Attachment C

GEI Consultants, Inc. Report

Pilgrim 1 IPEEE
Plymouth, Massachusetts

Ref 3-16 Soil Failures Analysis

GEI Consultants, Inc.

PILGRIM 1 IPEEE
PLYMOUTH, MASSACHUSETTS

Submitted to

Stevenson & Associates
Ten State Street
Woburn, MA 01801

1021 Main Street
Winchester, MA 01890-1943
617-721-4000

Project 92012
July 9, 1992

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PLYMOUTH, MASSACHUSETTS

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Project 92012

Eugene Marciano

Eugene A. Marciano, Ph.D.
Project Manager

EXECUTIVE SUMMARY

The purpose of this report is to provide estimates of the liquefaction potential, and the seismically induced settlements, permanent horizontal displacements, and transient horizontal displacements of the ground versus peak ground surface acceleration due to an earthquake, for the Pilgrim Nuclear Station in Plymouth, Massachusetts.

The stratigraphy of the Pilgrim site consists of 30 to 50 feet of compacted fill materials above approximately 30 to 50 feet of glacial outwash deposits, which are underlain by bedrock at a depth of approximately 80 feet. The fill consists of sand and gravelly sands with less than 6% fines. The outwash deposits are granular, consisting predominately of poor- to well-graded sands with some zones of gravelly sands. The fill is heavily compacted. The outwash deposits are very dense as a result of loading due to glaciation.

For the outwash deposits, previous cross-hole testing by Weston Geophysical gave shear wave velocities ranging from about 1,700 to 2,700 feet per second (fps) (soil profile 2). In addition, shear wave velocities were calculated for the outwash soils using empirical correlations available in the literature and blowcount and laboratory test data from the Pilgrim site. The results range from 800 to 1,400 fps (soil profile 1). For the outwash, the shear wave velocity profile that produces the more severe loading should be used. It is reasonable to expect that the two shear wave velocity profiles bound the true shear wave velocity profile at the Pilgrim site.

For the compacted fill, cross-hole test results were not available. The shear wave velocities for the fill were calculated using empirical correlations and the results of laboratory tests and range from about 300 fps at the ground surface to over 1,000 fps at a depth of about 40 feet.

Liquefaction

The soils at the Pilgrim site are highly dilative due to their dense state, and thus their undrained steady-state shear strengths are greater than their drained strengths. Therefore, a liquefaction stability failure is not possible, regardless of the magnitude and peak ground acceleration of the earthquake. A conservative stability analysis conducted using the drained shear strengths gives a factor of safety of 1.9 against failure.

Settlements

For soil profiles 1 and 2, the maximum calculated settlements at the ground surface are 0.29 inch and 0.72 inch, respectively. These settlements correspond to peak ground

accelerations of 0.35 g and 0.7 g, which are the largest peak accelerations that would develop at the ground surface for soil profiles 1 and 2, respectively.

Differential settlements can be expected within the foundation imprint of any one building and within the areas between buildings due to natural variability of the compressibility of the soil deposits. These can be taken equal to 50% of the total settlements and can be taken to occur over a distance of about 25 feet for structures on individual spread footings and for the areas between buildings. For structures founded on a continuous mat foundation, the differential displacement can be taken to occur over a distance of about 50 feet.

Differential settlements can also be expected between any one building and the ground and between adjacent buildings, such as those within the Power Block, due to the different thicknesses of the soil strata beneath the various structures and beneath the ground surface. Those between a building and the surrounding ground will occur over a distance of only a few feet. The distance over which the differential settlements between adjacent buildings will occur is dependent on the interaction of the foundation mat with the foundation soil and can occur abruptly at construction or expansion joints between or within the buildings.

For a given value of acceleration, the settlements for the NUREG/CR-5250 and EPRI hazard results are identical.

Permanent Horizontal Displacements

For soil profile 2, the maximum calculated permanent horizontal displacement occurs for the Intake Structure for the NUREG/CR-5250 hazard results. It equals 1.7 inch and occurs for a peak ground acceleration of 0.7 g. The permanent horizontal displacements for soil profile 1 are slightly less than those for soil profile 2. Estimates of the differential permanent horizontal displacements between structures can generally be taken equal to the difference between the permanent horizontal movements of each structure, which will be less than 1.7 inches.

The permanent horizontal displacements for the EPRI hazard results are about one third of those calculated using the NUREG/CR-5250 hazard results.

Transient Horizontal Displacements

For soil profiles 1 and 2, maximum transient horizontal displacements at the ground surface are 1.05 inches and 1.12 inches, respectively.

Differential transient horizontal displacements can be expected between any one building and the surrounding ground, between buildings separated by some distance, and between adjacent buildings within the Power Block. The differential displacement can be conservatively taken equal to the absolute sum of the peak displacements of the building and the surrounding ground or the absolute sum of the peak displacements of the two buildings.

The differential displacement between a building and the surrounding ground can be conservatively taken to be uniformly distributed over a distance of about 25 feet from the foundation. The differential displacement between two separated buildings can be reasonably taken to be uniformly distributed over the distance between the two buildings.

The differential displacement between any two buildings within the Power Block is conservatively bounded by the absolute sum of the peak displacements of the two structures. The differential displacement between any two buildings within the Power Block may occur abruptly across construction joints or expansion joints between or within the building foundations or structures.

For a given value of peak ground acceleration, the settlements for the NUREG/CR-5250 and EPRI hazard results are identical.

The above described procedures for determining the differential transient displacements are generally conservative procedures that do not account for the effects of flexibility and rocking of structures. The effect of rocking is likely to be small due to the high stiffness of the soils at the Pilgrim site and is likely to be compensated by the effects of soil-structure-interaction or, i.e., flexibility of the foundations. The Condenser Tanks appear to be the most likely structures to be significantly affected by rocking, due to their high center-of-gravity and small depth of embedment.

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1. INTRODUCTION

1.1 Purpose and Scope

The purpose of this report is to provide estimates of liquefaction potential and seismically induced permanent and transient displacements of the ground at the Pilgrim Nuclear Station. These are provided as a function of the peak ground acceleration.

The scope of the work is:

a. Perform an evaluation of the liquefaction potential and seismic stability.

b. Estimate seismically induced displacements of sloping ground using published correlations of displacement with ground motion intensity.

c. Estimate seismically induced settlements using published correlations of volumetric strain with ground motion intensity.

d. Estimate transient displacement of the ground and building foundations during a seismic event.

e. Present the results of the above items as a function of the peak ground surface acceleration.

1.2 Background

The Pilgrim Nuclear Station is located in Plymouth, Massachusetts. The location of the site is shown in Fig. 1.

A site plan showing the major structures, including the Power Block and the Intake Structure, is shown in Fig. 2. The power block consists of the Reactor Turbine, Auxiliary Bay, and Rad-Waste Buildings. Three sections through the main structures are shown in Figs. 3, 4, and 5.

Subsurface investigations have been performed for Pilgrim 1, as well as for a potential expansion referred to as Unit 2 that was planned but was not built. The site for Unit 2 is located immediately east of Unit 1, and thus subsurface data in Unit 2 were also used to develop results presented in this report.

1.3 Project Personnel

The following GEI personnel were responsible for performing the majority of the work for this project:

Eugene Marciano, Ph.D.	Project Manager
Paul Joseph	Project Engineer
Edmund Williams	Project Engineer
Gonzalo Castro, Ph.D., P.E.	Project Principal

1.4 Authorization

This work was authorized by a contract signed by Mr. Thomas J. Tracy of Stevenson & Associates. The work was performed under GEI Consultants, Inc.'s Quality Assurance Program, which complies with the requirements of 10CFR50 Appendix B.

2. SUBSURFACE CONDITIONS AND SOIL PROPERTIES

2.1 Introduction

The information presented in this report is based on Bechtel Drawings C1 through C9 and M15 through M29, Bechtel's Soils Report for the Pilgrim 2 site (Bechtel, 1976), and a compendium of Pilgrim boring logs conducted for both the Pilgrim 1 and 2 sites (GEI, 1978). The Pilgrim 2 Soils Report contains GEI soils data reports conducted for the Pilgrim 2 site. In addition, GEI's project files for geotechnical investigations conducted for the Pilgrim 1 and 2 sites were used.

2.2 Soil Profile

The stratigraphy of the Pilgrim 1 site consists of 30 to 50 feet of compacted fill materials, designated as type A and type B fills on Bechtel Drawing C8, above approximately 30 to 50 feet of glacial outwash deposits. The soil deposits are underlain by bedrock at a depth of approximately 80 feet.

The type A and B fills were specified to be compacted to a minimum of 98% and 96%, respectively, of the maximum dry density as determined by ASTM D1557. The type A and B fills consist of sand and gravelly sands with less than 6% fines.

The outwash deposits are very dense as a result of loading due to glaciation subsequent to their deposition. The outwash deposits are granular, consisting predominantly of poor- to well-graded sands with some zones of gravelly sands.

A comparison of the boring logs for the Pilgrim 1 and 2 sites and the geological history of the sites indicates that the outwash deposits have similar soil descriptions and ranges of blowcounts at the Pilgrim 1 and 2 sites. They have the same depositional history and were both subjected to glacial loading.

Plots of blowcounts versus effective vertical stress for several boring logs from the Pilgrim 1 and 2 sites are presented . Figs. A-1 and A-2 in Appendix A. These blowcounts correspond to the outwash soils and are limited to blowcounts obtained using the standard penetration tests for which no gravel was observed in the split-spoon samples. It can be seen that the blowcounts at the two sites are similar and very high, indicating similar properties of the outwash materials in Units 1 and 2. Note that the shallower soils were excavated below the Power Block structures in Unit 1, as shown in Figs. 3, 4, and 5. The range of effective overburden pressures at the excavation bottoms is shown in Figs. A1 and A2.

Boring logs from the Soils Report for three borings (Borings 505, 609, and 610) representative of the Pilgrim site conditions are included in Appendix A, and the blowcounts are plotted in Fig. A2 where they are identified by the open square symbols. Corrections for the effect of the gravel content were made to the blowcounts, if required. However, the majority of the blowcounts did not require correction. Nearly all of the blowcounts in these borings are in excess of 50 blows per foot, and many exceed 100 blows per foot and are similar to the blowcounts from the other borings.

2.3 Existing Foundations

The Reactor Building's foundation bears directly on the glacial outwash. The Turbine Building is underlain by about 10 feet of compacted fill above the glacial outwash. The foundations of the Diesel Generator Building and the Condenser Tanks are close to the ground surface and thus are founded on the fill. The Intake Structure is founded on the glacial outwash. The foundation elevations for these structures are given in Figs. 3, 4, and 5.

The following vertical bearing pressures were obtained from the Soils Report (Bechtel, 1976) for the Pilgrim 1 structures:

Structure	Gross Bearing Pressure (ksf)	Net Bearing Pressure (ksf)
Reactor Containment Building	7.7	1.7
Reactor Auxiliary Building	2.3	-0.5 to 3.0
Radwaste Building	4.5	-0.3
Turbine Building	2.2	-0.9
Intake Structure	4.1	-1.8

The net bearing pressure is equal to the gross bearing pressure minus the total vertical stress in the soil at the bearing level prior to excavation.

The dimensions of the Condenser Tanks were estimated based on Bechtel Drawing C8 and a recent site visit by GEI. The tank load was estimated assuming that the tank is filled with water, giving a value of about 3.5 kips per square foot (ksf) at the ground surface. The load exerted by the Diesel Generator Building near the ground surface was taken to be 2 ksf.

2.4 Ground Water Table Elevations

The elevation of the ground water table in this area can be expected to undergo fluctuations due to tidal effects and normal rainfall. Based on observation well readings conducted by GEI (1983) over nearly a 3-year period within and surrounding the Pilgrim 1 area, the highest recorded ground water elevations varied from +4 feet at approximately 100 feet from the shoreline to +8 feet at the southern end of the Turbine Building, about 600 feet from the shoreline. The corresponding values for the lowest recorded ground water elevations are +1 to +5 feet. The above values do not include the potential effects of flooding, storm surges, or other extreme events on the ground water table.

The mean high water and mean low water tidal elevations from the nearest National Oceanic and Atmospheric Administration tide station, located in Boston, are +4.98 and -4.56 feet, respectively, GEI (1983).

2.5 Total Unit Weights

Based on the data available in the Soils Report for Pilgrim 2, the average total unit weights are 126 pounds per cubic foot (pcf) for the compacted fill above the water table, 137 pcf for the compacted fill below the water table, and 129 pcf for the outwash deposits. Bechtel indicates in the Soils Report (1976) that the unit weight of the bedrock is 168 pcf.

2.6 Shear Strengths

The compacted fills and glacial outwash at the Pilgrim site are very dense, as evidenced by the high blowcounts in the outwash and the minimum specified relative compaction values of 98% and 96% (ASTM D1557) for the class A and B fills, respectively.

The results of triaxial compression tests conducted by GEI were reported in the Soils Report (Bechtel, 1976). Consolidated drained (S) triaxial compression tests and consolidated undrained (R) triaxial compression tests with pore pressure measurement were conducted for undisturbed tube samples of the glacial outwash, as well as for samples representative of the fill.

Drained Strengths

Because of the dense compacted state of the fill and the dense state of the outwash due to glacial loading, these materials are dilative during shearing. Therefore, during drained loading they increase in volume (dilate). The shear stress increases to a maximum value as the soil dilates, reaching the drained peak shear strength of the soil. The shear stress then drops off until the soil is completely remolded. At that point, the soil reaches the steady-state condition, which is a state of continuous deformation at constant volume,

constant normal effective stress, constant shear stress, and constant rate of shear strain (Poulos et al, 1985). The shear stress at the steady-state condition is the minimum drained strength of the soil and is referred to as the drained steady-state shear strength. The strength is equal to:

peak strength: $S_{DP} = \sigma'_f \tan\phi_p$

steady state strength: $S_{DS} = \sigma'_f \tan\phi_s$

where S_{DP} = drained peak shear strength
S_{DS} = drained steady-state shear strength
σ'_f = effective normal stress on the failure surface
ϕ_p = peak friction angle
ϕ_s = the steady-state friction angle

Based on the results of the S tests, the average peak and steady state friction angles for the outwash are 38.8 degrees and 33.4 degrees, respectively. Tests on samples representative of the compacted fill indicated peak friction angles of 40.5 to 43 degrees and steady state friction angles of 36 to 39 degrees.

Analysis of the blowcount data for Borings 505, 609, and 610 using correlations by Peck, Hanson, and Thornburn (1974) and Gibbs and Holtz (1957) indicate a drained peak friction angle of about 42 to 45 degrees for the glacial outwash. This is higher than the value of 38.8 degrees from the triaxial compression tests.

<u>Undrained Strength</u>

During undrained shearing of a dilative cohesionless soil, the shear stress increases gradually and approaches the undrained steady-state shear strength as the soil is remolded, i.e., the peak strength is about equal to the steady-state strength. The undrained steady-state shear strength, S_{us}, and thus in effect the peak undrained shear strength as well, is given by the same expressions as for drained strengths given above. However, for dilative material, the pore pressure decreases during shearing, resulting in an increase in effective stress and consequently larger shear strength for undrained conditions than for drained conditions.

The results of the R tests give a friction angle of 33.2 degrees for the peak and the steady-state strength. Note that this is about equal to the drained steady-state friction angle of 33.4 degrees, as expected. The results of R tests indicate that dilation continues until the pore pressure becomes low enough to cavitate. Thus there is no well-defined undrained strength since it is a function of the effective stress at which cavitation occurs which, in turn, is a function of the initial pore pressure and effective stress. In the

analyses presented in this report, the strength available during the earthquake is conservatively assumed to be equal to the drained strength, thus neglecting the possibility of negative-induced pore pressures.

2.7 Shear Wave Velocities

The results of seismic cross-hole testing conducted by Weston Geophysical at the Pilgrim 2 site in 1972 and 1976 are available in the Soils Report (Bechtel, 1976). The results are plotted in Fig. 6, and the shear wave velocity ranges from 1,700 to 2,700 fps. There is no compacted fill in the Pilgrim 2 area, and thus the cross-hole test results are not available for the fill. For the outwash deposits, the following shear wave velocities based on the cross-hole results were recommended by Bechtel (1976) for design of Pilgrim 2.

Depth (feet)	Elevation (feet)	Shear Wave Velocity (fps)
35 to 51	-13 to -29	1,950
51 to 71	-29 to -49	2,300
71 to bedrock	-49 to bedrock	2,650
bedrock	bedrock	5,900

We have also calculated the shear wave velocities of the outwash soils and the compacted fills based on blowcount and unit weight data, which were input into empirical correlations, and on laboratory testing data from the Soils Report (Bechtel, 1976).

The shear wave velocities were calculated using the following data:

a. Blowcount data within the glacial outwash corrected for the influence of gravel content, if necessary, and an empirical correlation between shear were velocity and blowcount by Ohta and Goto, 1978, as presented in Sykora (1987).

b. Impulse shear wave velocity tests on undisturbed samples of glacial outwash.

c. Resonant column test results on specimens prepared by compaction of materials from bulk samples obtained from the glacial outwash. The bulk samples were obtained from borings in the vicinity of the Pilgrim 2 cross-hole survey tests and are believed to be representative of both the outwash and the fill.

d. Hardin and Drnevich's (1972) empirical relationship for granular materials. The range of unit weight for the outwash deposits was determined from *in situ* field density test results conducted by GEI (Bechtel, 1976). The range of unit weight of the compacted fills was estimated using the results of compaction tests conducted by GEI on samples of the outwash materials (Bechtel, 1976).

The results of the various estimates of the shear wave velocities are shown in Fig. 6. All of the plotted points and curves in this figure are based on a ground water table elevation of +5 feet, i.e., a depth of about 17 feet below the ground surface.

The shear wave velocities estimated from the empirical correlations and laboratory test results described above fall within a relatively narrow band (see Fig. 6). However, for the outwash soils, plotted for depths larger than about 25 feet in Fig 6, these results are lower than those obtained from the cross-hole measurements by a factor of about two.

The empirical correlations are an average of data for a wide range of soils and thus involve considerable scatter. The laboratory tests may underestimate somewhat the *in situ* shear wave velocity because of unavoidable disturbance of the "undisturbed" samples used in the tests. However, it is not likely that the laboratory tests would underestimate the shear wave velocities by a factor of two.

The shear wave velocities measured using the cross-hole method appear to be unusually high even considering the high densities and high overconsolidation of the outwash soils at Pilgrim. For example, recent shear wave velocities determined in glacial till in Boston were about 1,800 fps as compared to 2,500 fps for the Pilgrim outwash at similar depths. The glacial till is denser than the outwash, and it is also highly overconsolidated. The procedures used to perform the cross-hole tests for Pilgrim included the use of explosives for the signal source and relatively large spacings between the source and receiver holes. The use of explosives for the source generates a larger percentage of compressive wave (P wave) energy than shear wave (S wave) energy. The velocity of the S wave is typically about half that of the P wave, and thus the P wave arrives before the S wave. The result of this is that the high P wave content tends to obscure the arrival time of the S wave recorded at the receiver holes. In addition, the large spacings (approximately 150 feet) between the source and receiver holes may have resulted in refraction of the wave through deeper, denser layers, which would result in an overestimate of the shear wave velocity.

Based on the above discussion, it is our opinion that the actual shear wave velocities of the outwash soils are bounded by the results obtained from the empirical correlations and laboratory test data and those obtained from the cross hole measurements. Since the range between the lower and upper bound is large, it is recommended that analyses be performed for both lower and upper bound values as shown in Fig. 8, designated as profiles S1 and S2, respectively. Alternatively, one could perform additional cross-hole determinations using closely spaced boreholes (10 to 15 feet) and signal generation devices that enhance shear wave energy.

For the compacted fill, we recommend that the empirical/laboratory results be used, as shown in Fig. 8.

3. SOIL LIQUEFACTION EVALUATION

The potential for seismically induced liquefaction involves consideration of several related phenomena and mechanisms of failure. In this section we deal with the potential for pore pressure build-up and with the potential for overall instability of the ground. These two subjects are discussed separately in the following sections.

3.1 Potential for Seismic Pore Pressure Build-Up

Two approaches were used to evaluate the potential for pore pressure build-up induced by earthquakes of various levels of peak ground surface acceleration.

The first approach is based on an empirical chart that relates manifestations of pore pressure build-up in sand deposits as a function of earthquake seismic shear stresses induced in the ground by the earthquake. The sites are level ground sites, and the soil characteristics are represented by SPT blowcounts normalized to a standard confining pressure (1 ton per square foot [tsf]).

The field observations consisted generally of sand boils or sand volcanoes, which are evidence of high pore pressures and settlement. They are not an indication of instability, as instability was not possible for the level ground deposits.

As part of the liquefaction evaluation for the Pilgrim 2 site (Bechtel, 1976), the blowcounts were analyzed using the then available correlation by Seed et al (1975), shown in Fig. 7. The curve shown was based on the data plotted by Seed et al (1975) and shown as open and closed circles. In addition, we have added to the plot the more recent curves by Seed et al (1983) for earthquake magnitudes 7.5 and 8.5.

The square and triangular data points in Fig. 7 are the average shear stress determined using Seed and Idriss's (1971) simplified expression for shear stress and are plotted versus the corrected blowcount for each standard penetration test in Borings 505, 609, and 610. Only the blowcounts for which the spoon samples had no gravel content were shown. The square and triangular data points are for peak ground surface accelerations of 0.15 and 0.25 g, respectively.

For a peak acceleration of about 0.5 g, the shear stresses computed using Seed and Idriss's (1971) expression would be double those indicated by the triangles. Therefore, for a peak acceleration of about 0.5 g and higher, a few of the lowest Pilgrim blowcounts will plot on or close to the boundary lines in Fig. 7, indicating a potential for pore pressure build-up in localized zones of the outwash. Note, however, that the N_1 values of the Pilgrim site are 34 or higher, while the empirical data are based on sites which,

with one exception, have N_1 values lower than 25. Thus the analysis of the Pilgrim case is based on extrapolation of field data.

A second approach is to estimate pore pressure increases based on the shear strains induced by the earthquake and the laboratory data collected by Dobry (National Research Council, 1985). Dobry's data indicate that about 10 cycles of a seismic shear strain of about 0.3% or higher is needed to reach 100% pore pressure. Based on the results of SHAKE analyses discussed in Section 4.2, an effective strain of 0.3% (defined as 65% of the peak strain) is reached somewhere in the profile when the earthquake has a peak ground surface acceleration of 0.40 g and 0.75 g for the high and low estimates of shear wave velocity defined in Section 2.7, respectively.

3.2 Seismic Stability

A liquefaction stability failure will occur if: 1) the undrained steady-state shear strength is less than the driving stresses (i.e., the stresses required to maintain equilibrium) and 2) sufficient deformation is induced, such as by an earthquake, to reduce the shear strength to the undrained steady-state shear strength.

For the dilative soils at the Pilgrim site, the undrained steady-state shear strength is greater than the drained strength. Therefore, a liquefaction stability failure, as defined above, is not possible at the Pilgrim site, regardless of the magnitude and peak ground acceleration of the earthquake.

A stability analysis was conducted to determine the minimum factor of safety based on the drained steady-state shear strength. The computer program, STABL5, was used to perform the stability analyses. The Modified Bishop method of slices for circular failure surfaces was used.

The geometry used for the stability analysis is shown in Fig. 3 and is based on Bechtel Drawings No. C1 through C9 and M15 through M29. The gross loads given in Section 2.3 for the structures shown in Fig. 3 were applied at the level of the foundations of these structures.

The stability analysis was conducted using steady-state friction angles of 36 and 33 degrees for the compacted fill and the glacial outwash, respectively. These are based on the results of the triaxial compression tests discussed in Section 2. These were used to compute the drained steady-state shear strength of the soil along the potential failure surface.

The slopes are covered with riprap, consisting of large blocks of stone. The riprap was taken to be about 10 feet thick, as shown in Fig. 3, based on a recent site visit by GEI.

The unit weight and friction angle for the riprap were estimated to be 130 pcf and 40 degrees based on previous experience with similar materials.

The maximum measured ground water table elevations described in Section 2 were used to define the phreatic surface on land for the stability analysis. These are shown in Fig. 3. The elevation of the water surface of the channel was taken to be the mean low water tidal elevation discussed in Section 2.

A search was conducted to determine the critical surface for stability failure. The resulting critical circle for the stability analysis is shown in Fig. 3. The critical factor of safety for this circle is 1.9. This factor of safety is based on the drained steady-state shear strength of the soil. The undrained steady-state strength of the dilative soils is higher than the drained strength, and therefore, the actual factor of safety against a stability failure is much higher.

4. PERMANENT DISPLACEMENTS DUE TO SEISMIC EVENTS

Permanent displacements and settlements were calculated at critical locations throughout the site. Newmark's (1965) sliding block analogy was used to calculate the permanent displacements due to sloping ground. Previously integrated solutions available in the literature were used for this purpose. Correlations of volumetric strain with the seismic shear strain (Castro, 1987) were used to calculate the settlements due to seismic shaking. The computer program, SHAKE (Schnabel et al, 1972), was used to calculate the shear strains and shear stresses versus depth for the soil profile. The results of the SHAKE analyses represent the case of one-dimensional wave propagation through the soil profile in the free field. These results do not account for the effects of soil-structure-interaction or rocking of the structure, which are discussed later in this report.

4.1 Critical Locations for Evaluation of Permanent Displacements

Displacements due to slope movements were calculated for the Intake Structure, the Condenser Tanks, the Reactor Containment Building, the Turbine Building, and the Diesel Generator Building.

Vertical settlements due to densification during seismic shaking of the soil above the ground water table and due to dissipation of pore pressures developed below the water table during seismic shaking were calculated for these structures. Vertical settlements were calculated for the structures and for the ground surface to enable calculation of differential settlement of piping and ducts.

4.2 SHAKE Analyses

The computer program, SHAKE (Schnabel et al, 1972) was used to analyze the response of the soil deposit to an earthquake. The purpose of the SHAKE analyses is to determine the maximum shear strains and the maximum shear stresses versus depth as a function of the peak ground surface acceleration.

The bedrock was taken to be at about El. -58 feet based on the available boring logs for the Pilgrim 1 site. The strata thicknesses and total unit weights used for input into the SHAKE analyses consist of the following:

Stratum	Thickness (feet)	Total Unit Weight (pcf)
Compacted fill above water table	18	126
Compacted fill below water table	17	137
Glacial Outwash	45	129
Bedrock	Halfspace	168

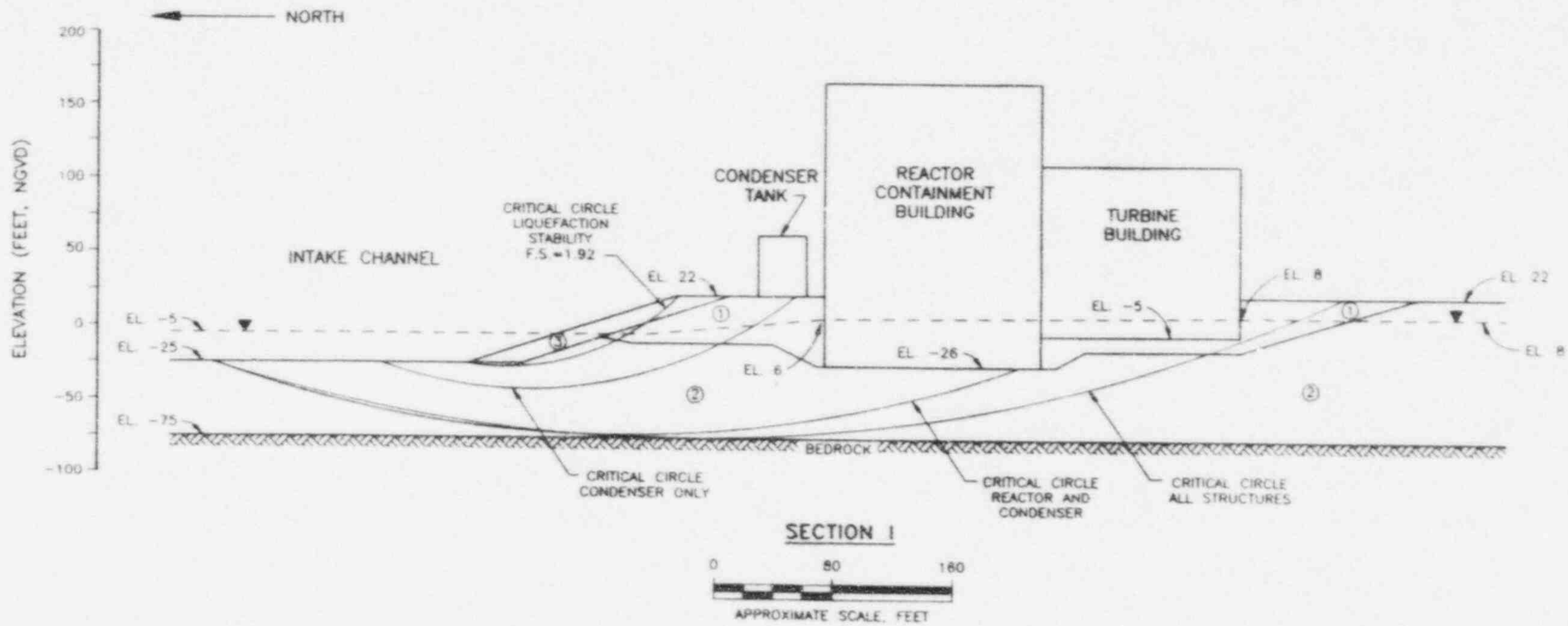
The SHAKE analyses were conducted for the two alternative *in-situ* shear wave velocity profiles discussed in Section 2 and shown in Fig. 6. The specific values of shear wave velocity used for each of the two profiles are shown in Fig. 8.

For each of the two soil profiles, analyses were conducted for a range of peak ground accelerations for each of two seismic records. The two records were generated by Stevenson & Associates and are referred to as NR0098-1 and NR0098-2 in this report. NR0098-1 is a synthetic record generated using the NUREG-0098 response spectrum scaled to a peak acceleration of 0.5 g. NR0098-2 was generated by changing the peak value of record NR0098-1 from 0.5 g to 1.0 g; the acceleration values for all other points in this time history are identical to those of NR0098-1.

Analyses were first conducted using NR0098-1, which provided strain-compatible moduli for the soil profile. Then, analyses using NR0098-2 were conducted using the strain-compatible moduli determined for NR0098-1. An analysis for NR0098-2 scaled to a peak acceleration of 0.2 g was conducted using the strain-compatible moduli determined by the analysis for NR0098-1 scaled to 0.1 g and so forth for higher levels of peak acceleration. It is our understanding that this procedure was used by Stevenson & Associates to conduct their soil-structure-interaction (SSI) analyses for the Pilgrim site.

4.2.1 Results for Profile 1

Profiles 1 and 2 refer to the low and high values of shear wave velocity defined in Section 2.7, respectively. SHAKE analyses were conducted using NR0098-1 scaled to peak accelerations of 0.1, 0.2, 0.3, 0.35, 0.4, and 0.45 g. The record was applied to the surface of the deposit. Results were not obtained for 0.45 g since the program did not converge to strain-compatible values of shear moduli due to the severity of the ground motion. This indicates that for the NUREG-0098 design spectrum, a peak ground acceleration higher than 0.4 g is not possible for soil profile 1. In addition, for peak ground surface accelerations of 0.3, 0.35, and 0.4 g, the peak acceleration at the bedrock is 0.67, 1.9, and 11 g, respectively. This

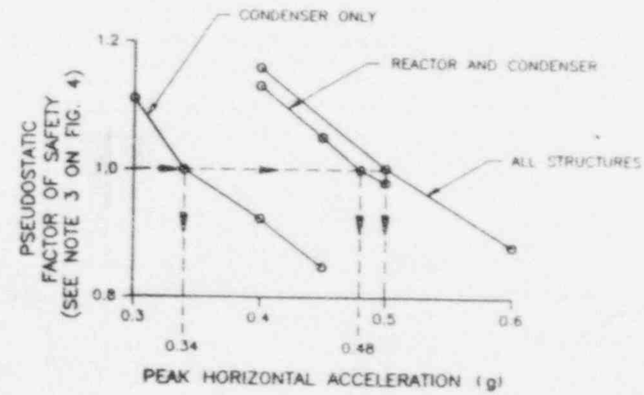


SECTION I

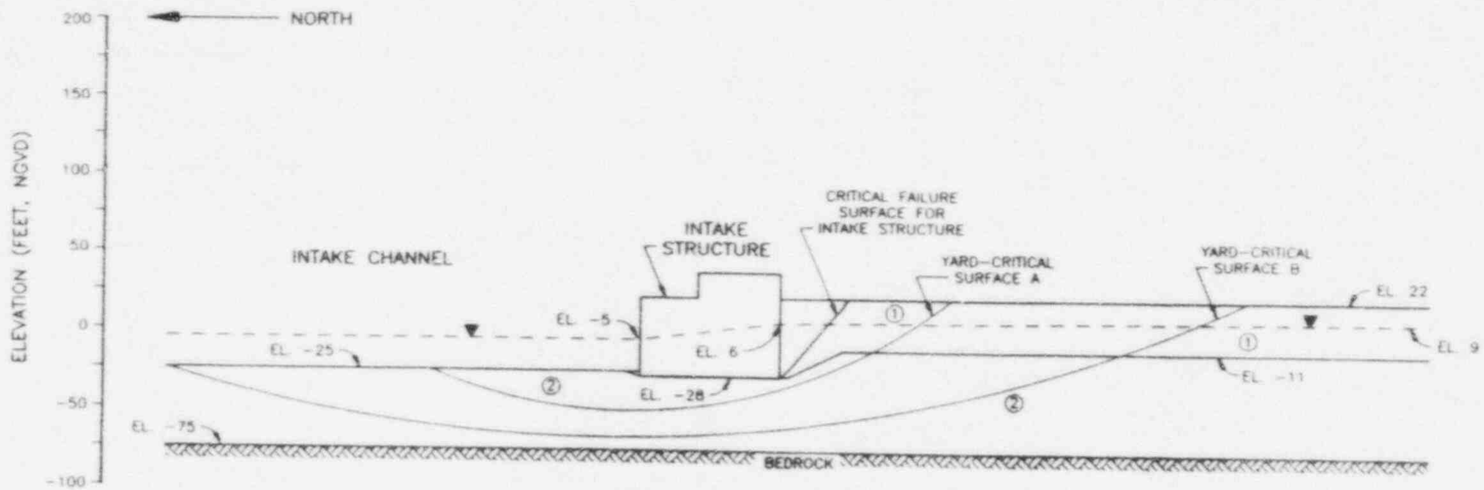
SOIL PROPERTIES

LAYER NO.	DESCRIPTION	γ_{moist} (pcf)	γ_{sat} (pcf)	ϕ'_s (deg)	ϕ'_p (deg)
①	COMPACTED GRANULAR FILL	126	137	36	40
②	GLACIAL OUTWASH	112	129	33	40
③	RIPRAP SLOPE	130	130	40	40

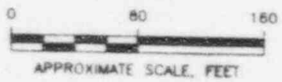
NOTES:
1. REFER TO FIG. 2 FOR PLAN LOCATION OF SECTIONS.
2. SECTION I IS BASED ON "YARDWORK PLANT SITE EXCAVATION SECTION B" (DRAWING NO. C-7) BY BECHTEL.



Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	STABILITY ANALYSES FOR SECTION I
GEI Consultants, Inc.	Project 92012	July 1992 Fig. 3

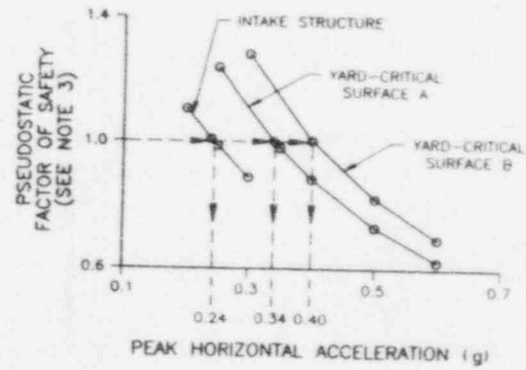


SECTION II



SOIL PROPERTIES

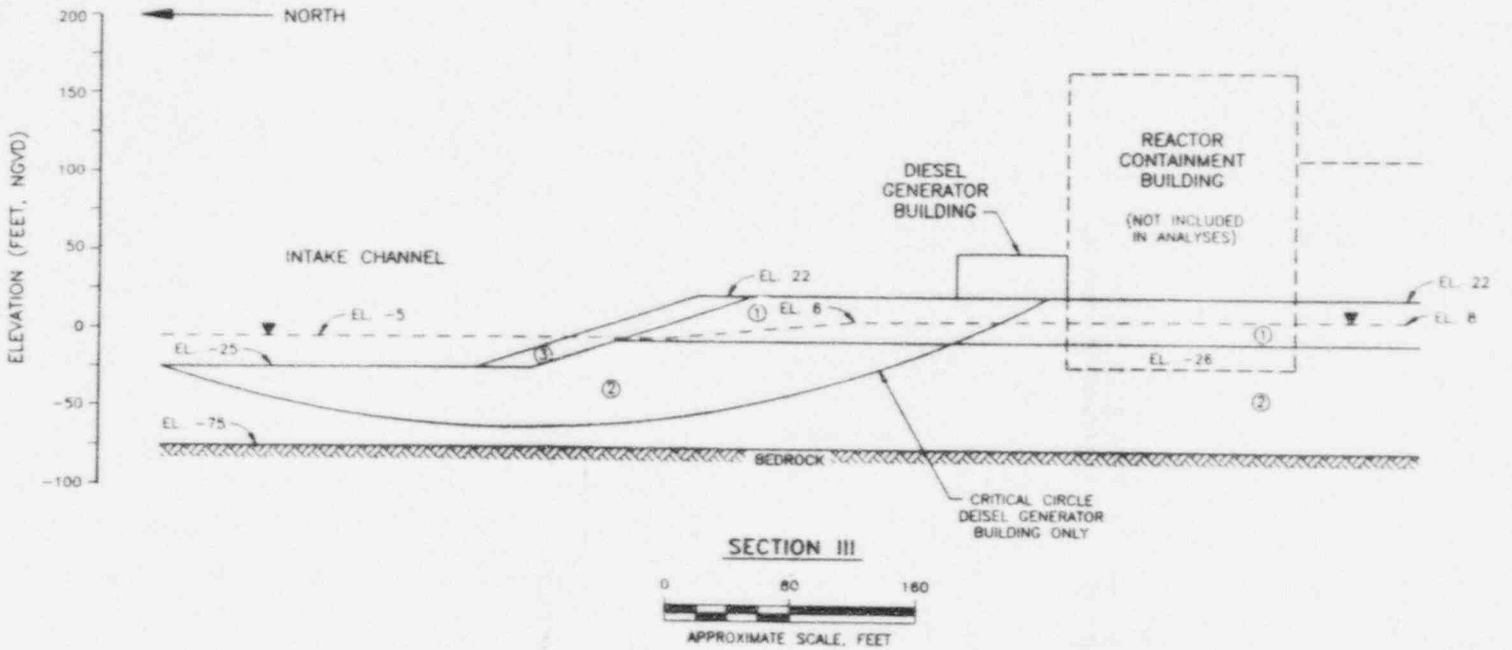
LAYER NO.	DESCRIPTION	γ_{moist} (pcf)	γ_{sat} (pcf)	ϕ'_s (deg)	ϕ'_p (deg)
①	COMPACTED GRANULAR FILL	126	137	36	40
②	GLACIAL OUTWASH	112	129	33	40



NOTES:

1. REFER TO FIG. 2 FOR PLAN LOCATION OF SECTIONS.
2. SECTION II IS BASED ON "YARDWORK PLANT SITE EXCAVATION SECTION C" (DRAWING NO. C-7) BY BECHTEL.
3. PSEUDOSTATIC FACTORS OF SAFETY EQUAL TO AND LESS THAN 1 CAN OCCUR MOMENTARILY DURING SHAKING FOR PEAK ACCELERATION VALUES GREATER THAN 0.24g, CAUSING A PERMANENT BUT FINITE HORIZONTAL DISPLACEMENT. THE PSEUDOSTATIC FACTOR OF SAFETY DOES NOT REFLECT THE STABILITY OF THE SOILS AND STRUCTURES AT THE SITE. THE FACTOR OF SAFETY AGAINST A STABILITY FAILURE AT THE PILGRIM SITE IS GREATER THAN 1.9.

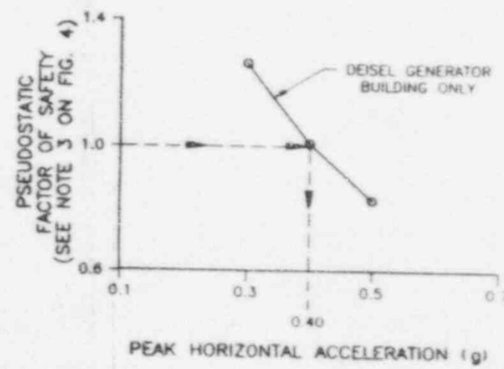
Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	STABILITY ANALYSES FOR SECTION II
GEI Consultants, Inc.	Project 92012	July 1992 Fig. 4



SECTION III

SOIL PROPERTIES

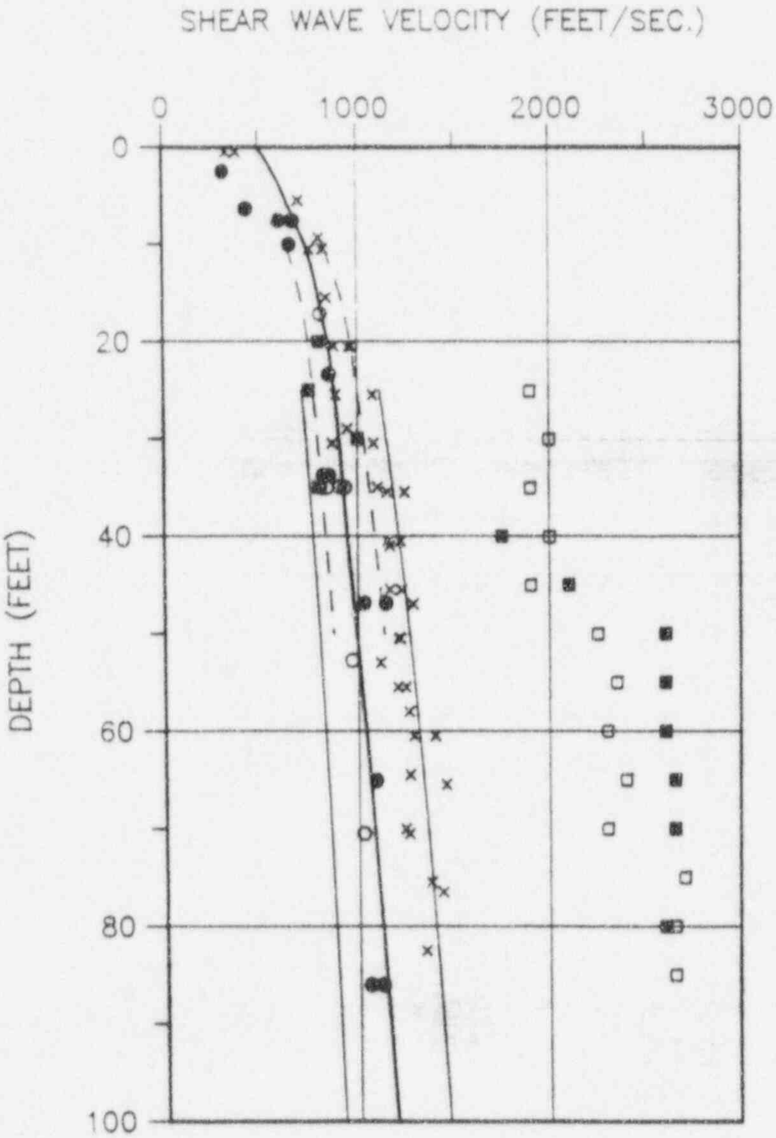
LAYER NO.	DESCRIPTION	γ_{moist} (pcf)	γ_{sat} (pcf)	ϕ'_s (deg)	ϕ'_p (deg)
①	COMPACTED GRANULAR FILL	126	137	36	40
②	GLACIAL OUTWASH	112	129	33	40
③	RIPRAP SLOPE	130	130	40	40



NOTES: 1. REFER TO FIG. 2 FOR PLAN LOCATION OF SECTIONS.

2. SECTION III IS BASED ON "YARDWORK PLANT SITE EXCAVATION" DRAWING NO. C-7 BY BECHTEL.

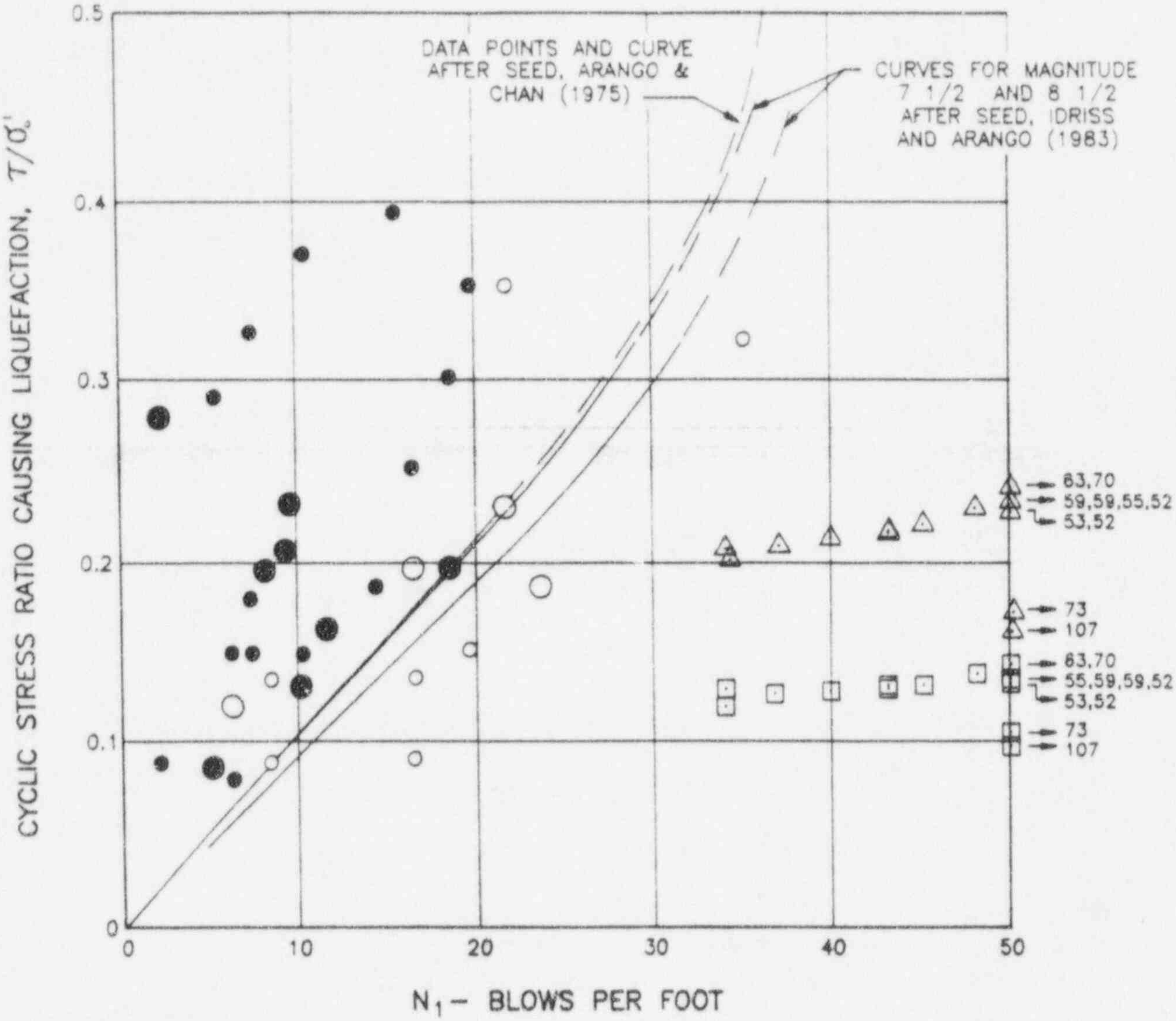
Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	STABILITY ANALYSES FOR SECTION III	
GEI Consultants, Inc.	Project 92012	July 1992	Fig. 5



Legend:

- × Correlation with N values
- ○ Impulse test results
- ● Resonant column test results
- □ Crosshole test results, 1972
- ■ Crosshole test results, 1976
- — — Range for compacted fills (based on Hardin and Drnevich expression)
- ——— Range for outwash deposits (based on Hardin and Drnevich expression)
- ——— Best fit all data, excluding crosshole tests

Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	IN-SITU SHEAR WAVE VELOCITY VERSUS DEPTH
Φ GEI Consultants, Inc.	Project 92012	May 1992 Fig. 6

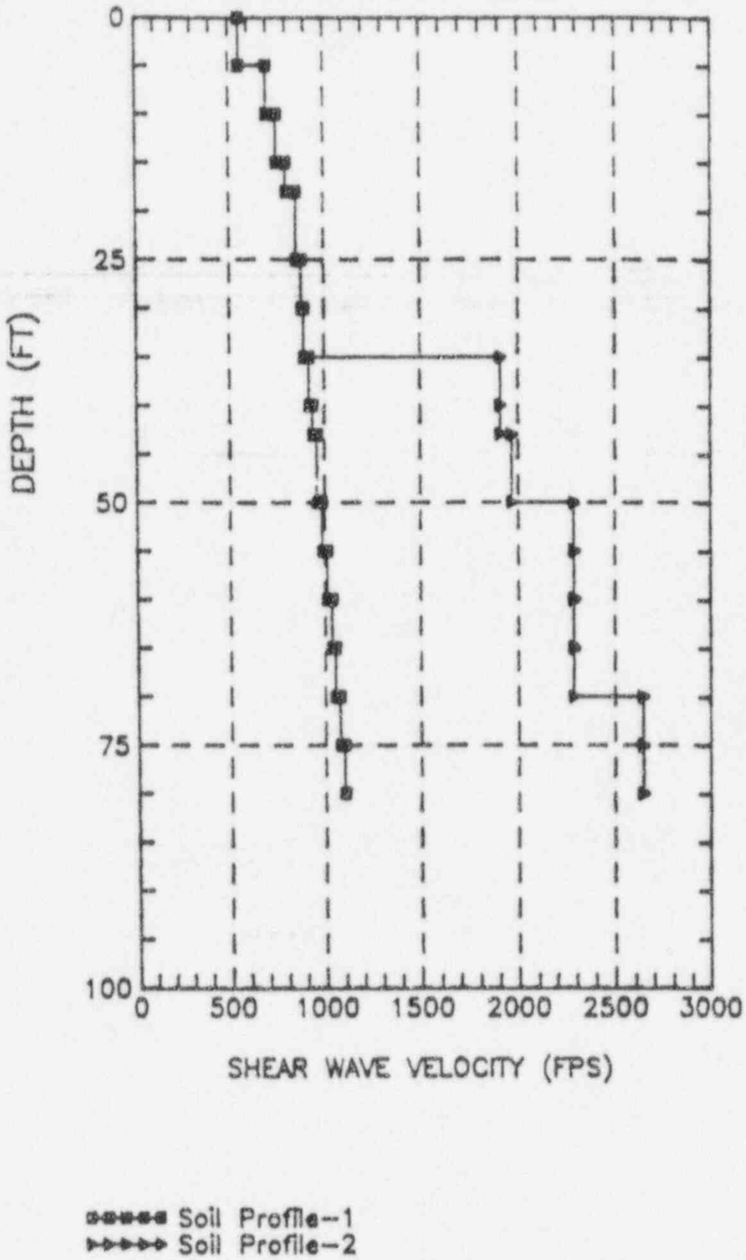


LEGEND

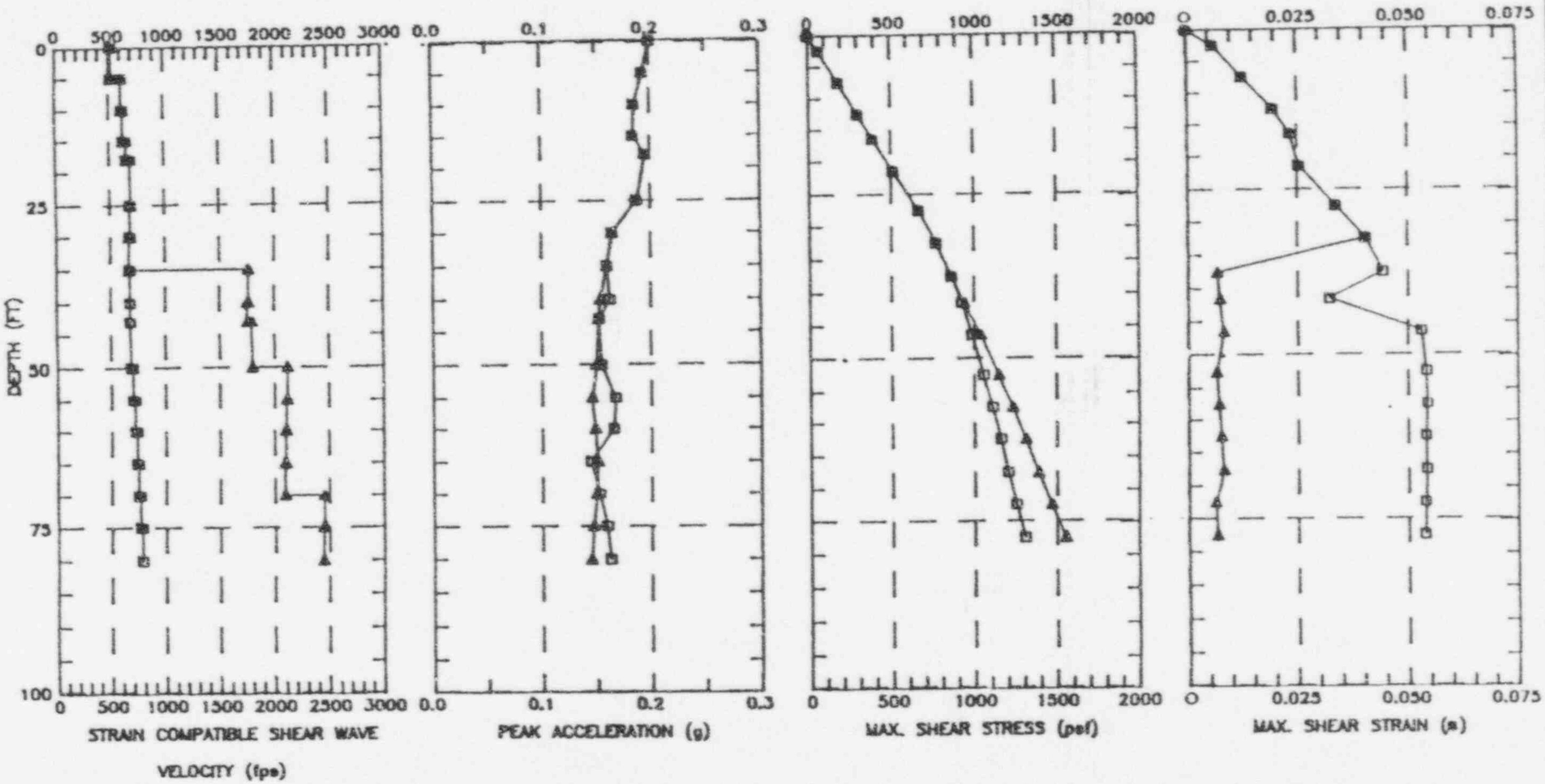
- ●, • LIQUEFACTION OBSERVED (1975)
- ○, ○ NO LIQUEFACTION OBSERVED (1975)
- △ PILGRAM DATA FOR a_{max} = 0.25 g
- ⊡ PILGRAM DATA FOR a_{max} = 0.15 g
- △← 63 VALUES OF N_1 > 50

2012LIQ1.DWG 5/21/92 MJR/AJS

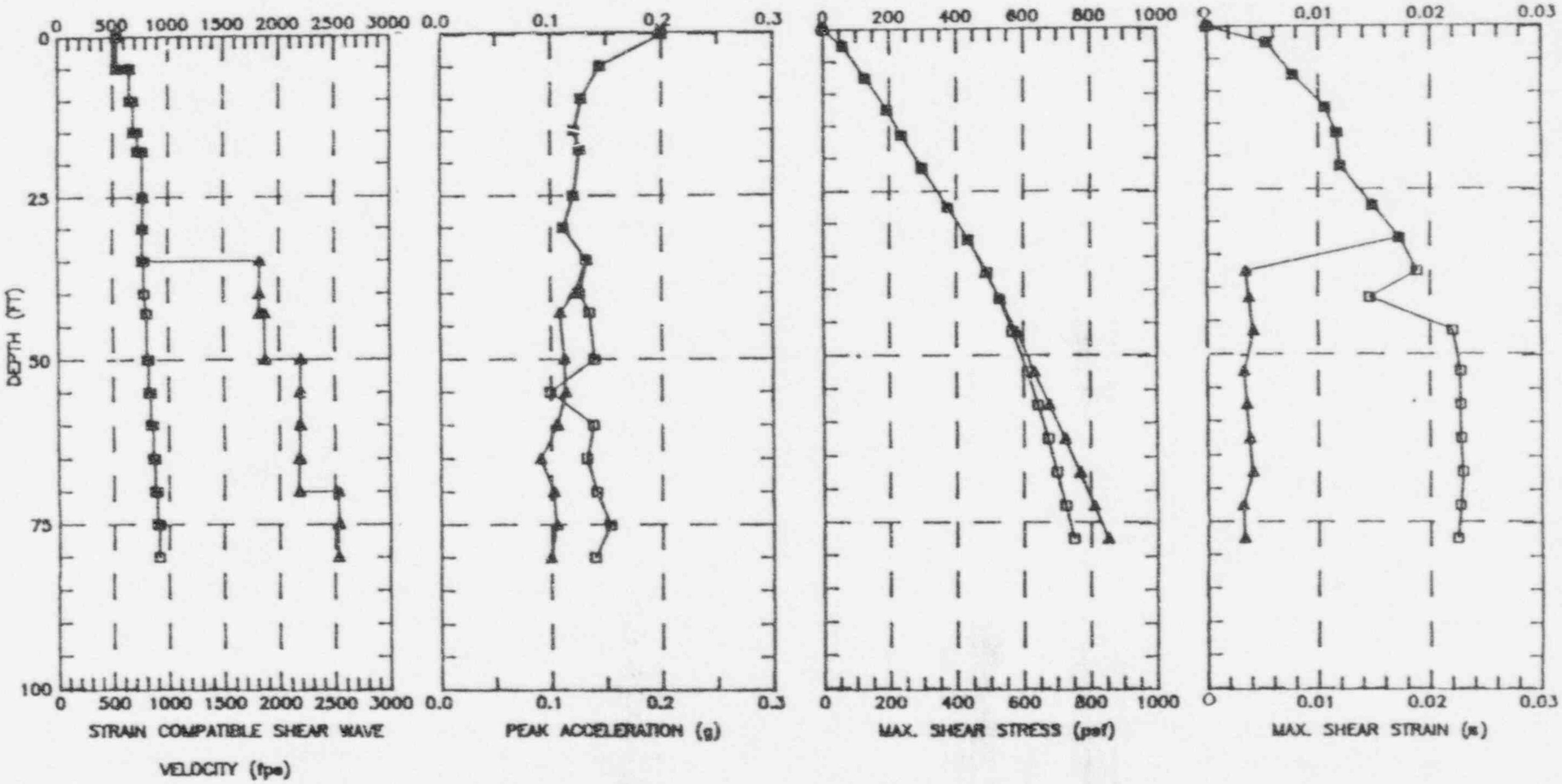
Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	LIQUEFACTION POTENTIAL BASED ON STANDARD PENETRATION TEST DATA
GEI Consultants, Inc.	Project 92012	May 1992 Fig. 7



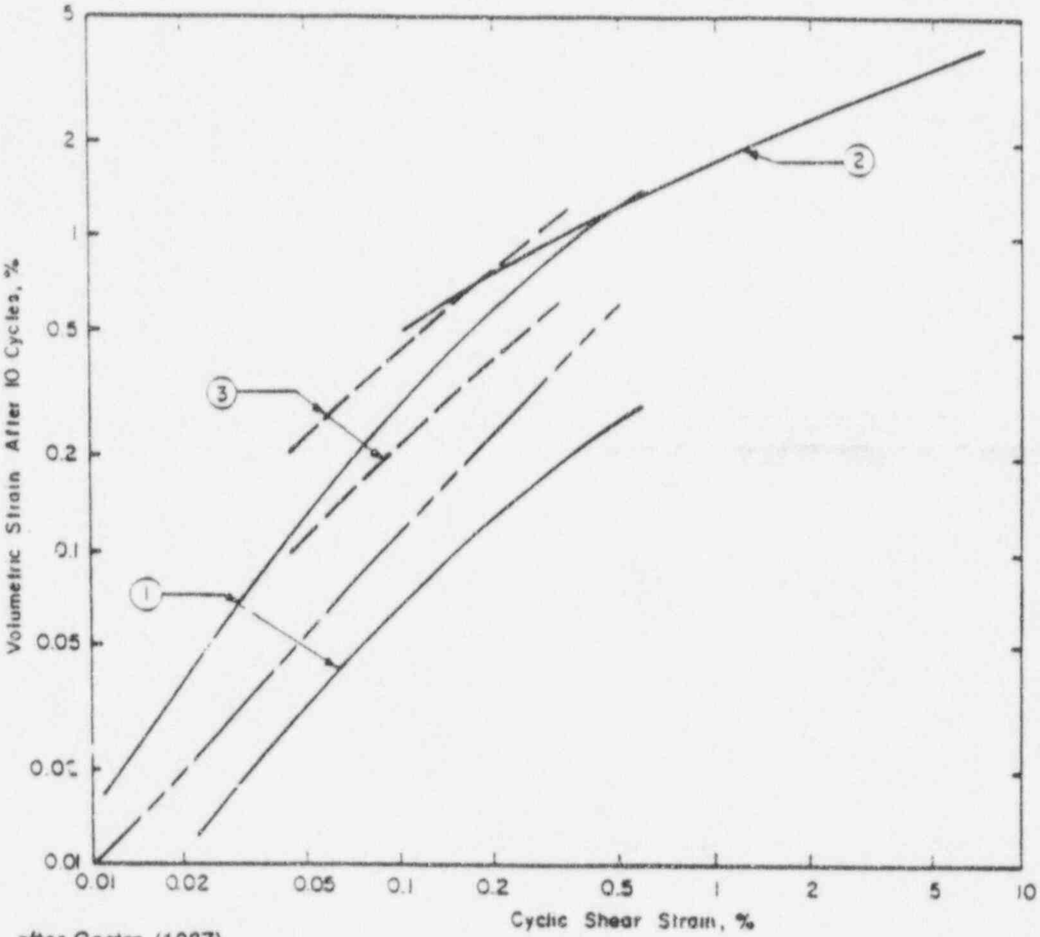
Stevenson & Associates Woburn, Massachusetts	Deformation Analysis Pilgrim 1 IPEEE	Shear Wave Velocity Input versus Depth (Feet) for SHAKE Analyses
Φ GEI Consultants, Inc.	Project 92012	May 1992 Fig. 8



Stevenson & Associates Woburn, Massachusetts	Deformation Analysis Pilgrim 1 IPEEE	SHAKE RESULTS Record NR0098-1 Peak Acc. = 0.2g
Φ GEI Consultants, Inc.	Project 92012	Fig. 9



Stevenson & Associates Woburn, Massachusetts	Deformation Analysis Pilgrim 1 IPEEE	SHAKE RESULTS Record NR0098-2 Peak Acc. = 0.2g
GEI Consultants, Inc.	Project 92012	Fig. 10

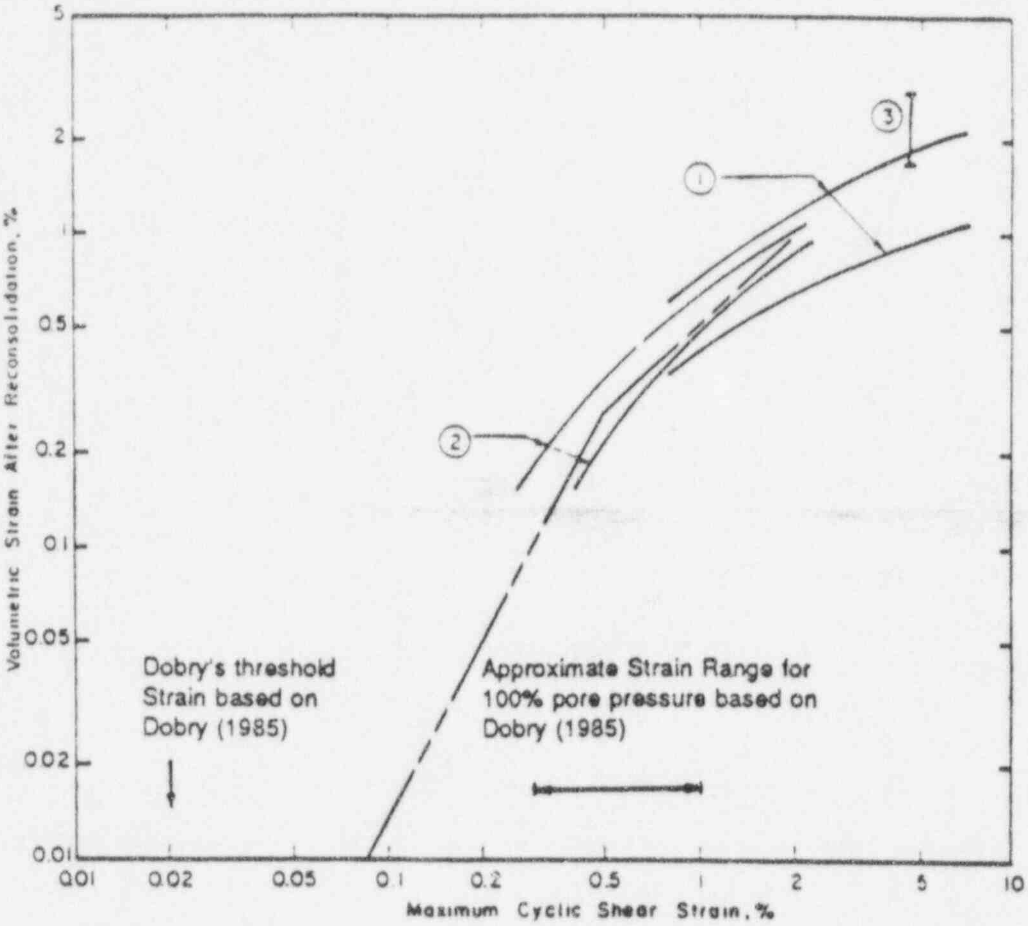


after Castro (1987)

	1	2	3
References:	Silver and Seed, 1971	Youd, 1972	Ishibashi et al, 1985
Type of Test:	Simple Shear	Simple Shear	Torsional Shear
Type of Sample:	Dry Sand	Saturated Sand	Saturated Sand
Relative Density:	45% to 80%	75%	42% to 76%
Confining Pressure:	500 to 4000 psf	100 to 4000 psf	2880 psf

— — — — Estimated relationship for fill above water table at Pilgrim 1

Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	SUMMARY PLOT OF VOLUME DECREASE OF SANDS UNDER CYCLIC STRAINING
GEI Consultants, Inc.	Project 92012	May 1992 Fig. 11



after Castro (1987)

① Tatsuoka et al, 1984
Torsional Tests
Clean Sand, loose to dense
$\bar{\sigma}_{mc}$ = 1000 to 6000 psf

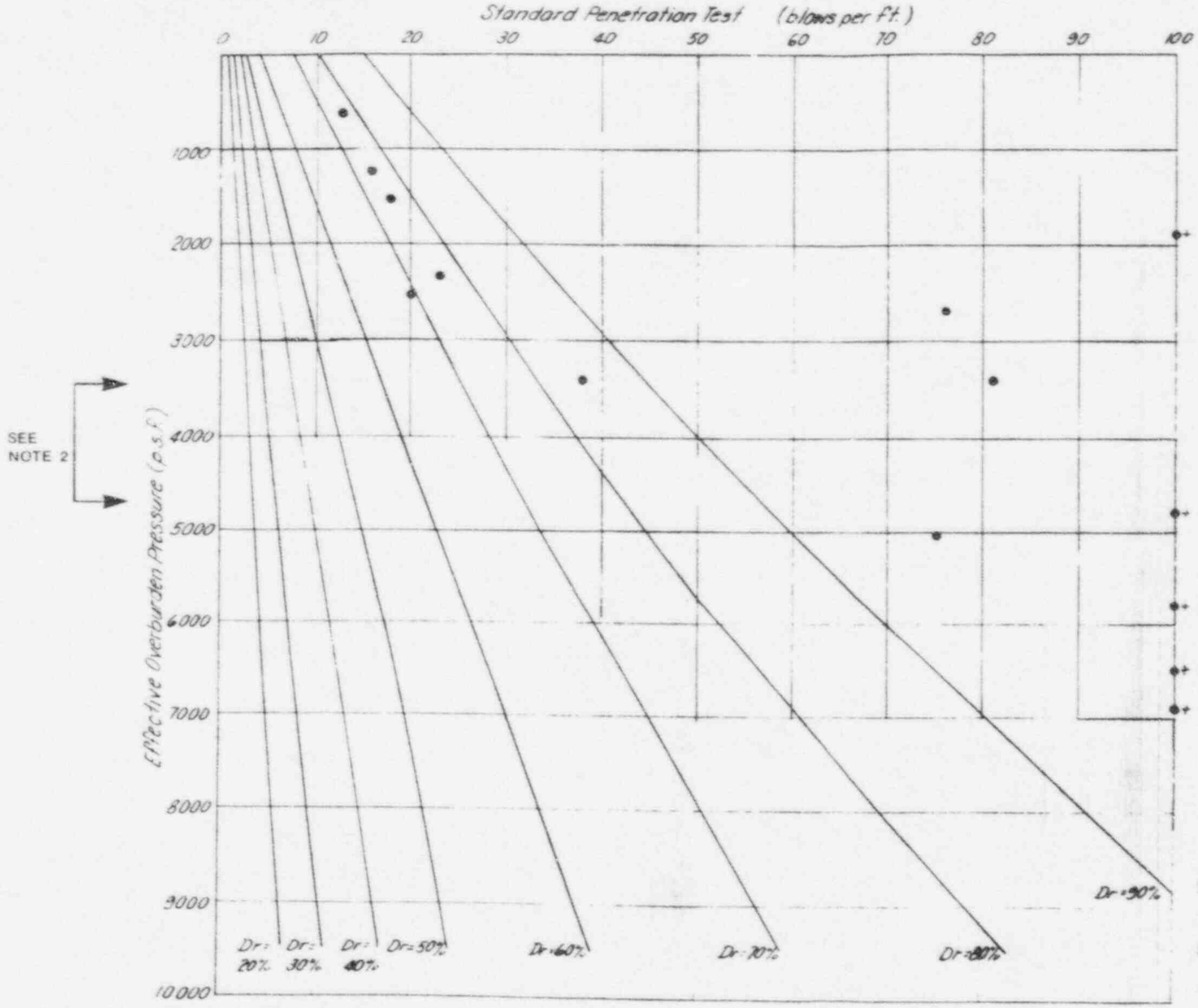
② Castro, 1968
Triaxial Tests
Silt, Undisturbed Samples
$\bar{\sigma}_c$ = 6000 psf

③ Castro et al, 1987
Triaxial Tests
Loose Silty Sand, Undrained Samples
$\bar{\sigma}_c$ = 5000 to 6400 psf

— — — — Estimated relationship for fill and glacial outwash below water table at Pilgrim 1

Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	VOLUME CHANGES OF SANDS AND SILTS DUE TO RECONSOLIDATION AFTER CYCLIC LOADING
GEI Consultants, Inc.	Project 92012	May 1992 Fig. 12

26



NOTES:

1. Blowcounts were determined from borings drilled in the 1967 to 1968 investigation: B-3, B-32, B-37, B-50 B-55, B-59A.
2. Effective overburden pressure was determined based on ground waterlevel readings taken at the time of drilling in each boring.
3. SPT refers to the standard penetration test made in accordance with ASTM Designation D-1586.
4. Blowcounts were plotted on Gibbs & Holtz Relative Density chart for normally consolidated soil (1).
5. Effective overburden pressures were determined from the following unit weights:
   δ_s Saturated Unit Weight = 137 pcf
   δ_b Buoyant Unit Weight = 75 pcf
6. The points plotted are from blowcounts made on soils classified as: Sand (SP) and Silty Sand (SM). Blowcounts made on soils classified as Silt (ML) Clay (CL) or on Soils containing gravel were not used in preparing this Figure.

LEGEND

Dr - Relative Density.

●+ Indicates blowcount was greater than 100 blows per foot.

(1) See Reference 1 in list of references.

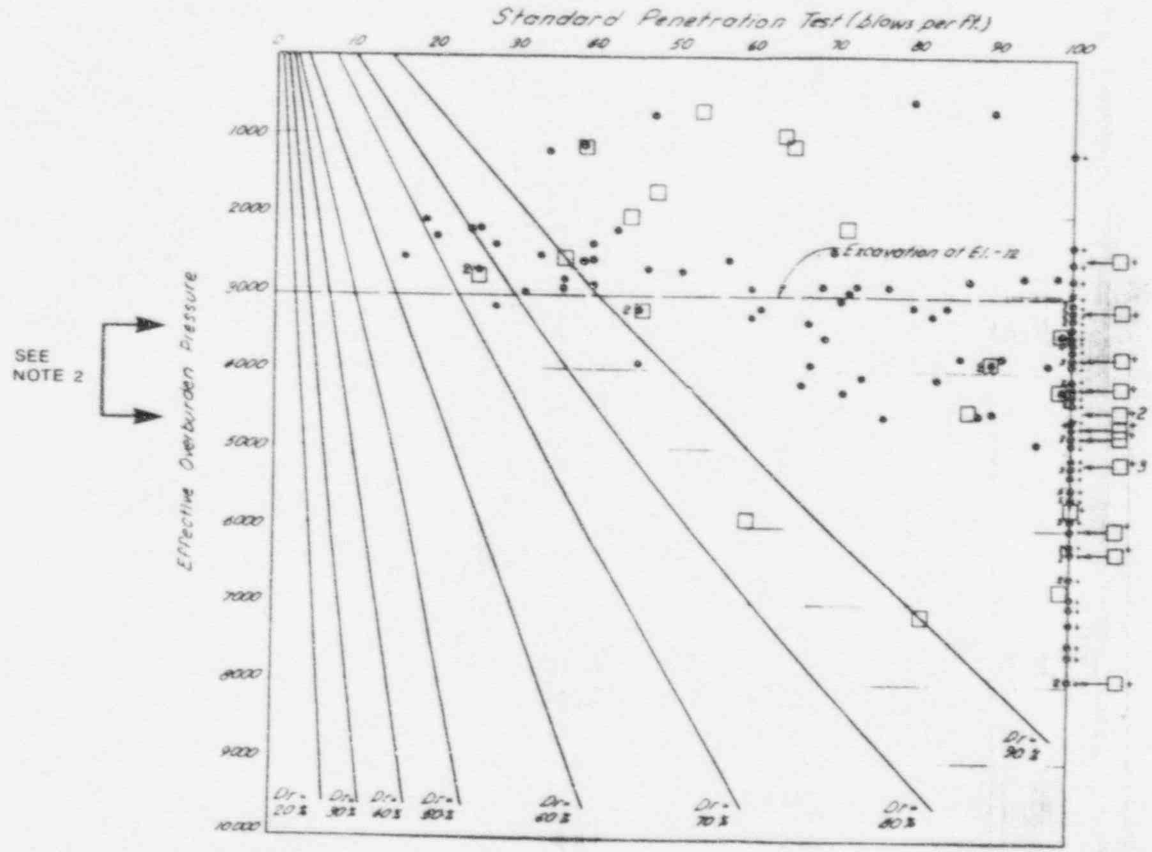
NOTES:

1. This was Fig. 40 in Bechtel (1976).
2. Range of effective stresses from bottom of fill to base of Reactor Containment Building (added by GEI, 1992).

PILGRIM STATION

STANDARD PENETRATION TEST VS. EFFECTIVE OVERBURDEN PRESSURE-UNIT 1

FIGURE ~~40~~ A-1



NOTES

1. Blowcounts were determined from borings B402, B403, B405, B408, B412, B415, B416, B417, B419, B421, B422, B424, B431, B435, B501, B502, B504, B505, B604, B609, B610.
2. Effective overburden pressure was determined based on ground water level readings taken at the time of drilling in each boring.
3. Effective overburden pressure was determined from average of all water level readings taken at the time of drilling, to define bottom of excavation as shown on plot.
4. Blowcounts were plotted on Gibbs & Holtz Relative Density chart for normally consolidated soil (1).
5. Effective overburden pressures were determined from the following unit weights
   γ_t - Saturated Unit Weight = 129 pcf
   γ_m - Moist Unit Weight = 112 pcf
   γ_b - Buoyant Unit Weight = 67 pcf
6. SPT refers to the standard penetration test made in accordance with ASTM Designation D-1586.
7. The points plotted are from blowcounts made on soil classified as Sand (SP), Sand (SW), Silty Sand with less than 20% minus #200 material (SM) and Clayey Sand with less than 20% minus #200 material (SC).
8. Blowcounts made on soils containing gravel or on soils classified as Silt (ML) or Clay (CL) were not used in preparing this figure.

LEGEND

Dr - Relative Density

2• Number of tests at point when more than one

□* or •* Indicates blowcount was greater than 100 blows per foot.

(1) See reference 1 in list of references

□ Tests determined from Penetration Depth VS Cumulative Number of Blows plots by GEI for borings 505, 609, 610.

NOTES

1. This was Fig. 22 in Bechtel (1976).
2. Range of effective stresses from bottom of fill to base of Reactor Containment Building (added by GEI, 1992).

PILGRIM STATION

STANDARD PENETRATION TEST vs. EFFECTIVE OVERBURDEN PRESSURE-UNITS 2 AND 3

FIGURE A-2

BORING LOCATION N9995.025 E5448.629	INCLINATION Vertical	BEARING N.A.	DATE START/FINISH October 20, 1975 / October 28, 1975
CASING ID 3 in.	CORE SIZE N.A. in.	TOTAL DEPTH 91.0 ft	DRILLED BY American Drilling & Boring Co., J. Tassira
GROUND EL (MSL) +20.9 ft	DEPTH TO WATER/DATE (1) ft / NA		LOGGED BY W. E. Pitt/ F. D. Leathers

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
20.9									Gravelly sand. Widely graded; about 35% subangular to subrounded gravel, max size 25 mm; about 5% nonplastic fines; gray (SW).
20		S1A	(2)	2 in.	12	12		3-7/8 in. Tricone roller bit from 0 to 91.0 ft	
		S1B	(3)	2 in.	4	6			Sand. Uniform; medium to fine grained; ~5% fine gravel; brown, with about 5% black matter which appears to be ash (SP).
16	6	S2	46	2 in.	12	18			Sand. Predominantly fine and medium grained; about 5% fine gravel to 8 mm max size; with layers and pockets to 15 mm thickness of fine-sandy silt; rapid dilatancy reaction; low dry strength; brown (SP with ML-SM).
14	8						Roller bit boulder from 7.0 to 7.6 ft		
12	10	S3	64	2 in.	11	18			Silty sand. Widely graded; about 35% slightly plastic fines; about 5% fine gravel to 25 mm max size; gray-brown (SM-ML).
10	12						Advanced NW casing to 13.8 ft		
6	16	S4	(3)	2 in.	2	2	Roller bit boulder from 14.8 - 15.4 ft		WASH: Silty sand. Widely graded; fines slightly plastic; contains one rock fragment 35 mm in size; brown.
2	20						Advanced NW casing 19.1 ft		
0		S5	152	2 in.	8	18			Silty sand. Widely graded; about 25% slightly plastic fines; about 10% fine gravel; one rock fragment 35 mm size; brown with several rust brown pockets (SW-SM).
-2	23.0							(23.0)	Approximate strata change determined from wash.
-4	24						Advanced NW casing to 23.9 ft		
-8	30	S6	46	2 in.	10	18			Sand. Uniform; fine grained with several pockets of medium sand; about 5% nonplastic fines; brown (SP).

(continued)

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample ▽ Groundwater
U - Undisturbed samples
S - Shelby tube D - Denison
F - Fixed piston P - Pitcher
O - Osterberg G - GEI
SAMP OD - Outside diameter of sampling spoon

NOTES
1) Test pit dewatering system in operation during boring, no water level readings obtained.
2) Blows of 4/6", 8/6", 12/6".
3) Blows of 100/2".

PILGRIM STATION NO. 600
UNIT NO. 2
BOSTON EDISON COMPANY

Date: February 27, 1976 (Revised)

PAGE 1 of 3

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Bechtel Job No. 8791

GEOTECHNICAL ENGINEERS INC

BORING LOCATION N9995.025 E5449.829 INCLINATION Vertical BEARING N.A. DATE START/FINISH October 20, 1975 / October 28, 1975

CASING ID 3 in. CORE SIZE N.A. in. TOTAL DEPTH 91.0 ft DRILLED BY American Drilling and Boring Co; J. Texeira

GROUND EL (MSL) -20.9 ft DEPTH TO WATER/DATE (1) ft / NA LOGGED BY W. L. Pitt/F. D. Leathers

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
								3-7/8 in. Tricone roller bit.	
		S7	90	2 in.	15	18			Sand. Uniform, fine grained; clean with two 25 to 35 mm thick layers of silty fine sand; nonplastic fines; brown (SP).
		S8	103	2 in.	12	18			Sand. Medium to fine grained with about 5% coarse grained; clean; one 25 mm layer of medium to coarse sand; one 8 mm layer of nonplastic silty fine sand; brown (SP).
		S9	156	2 in.	15	18			Sand. Uniform, fine grained; less than 5% nonplastic fines; about 5% subangular fine gravel; several 1-3 mm thick medium to fine sand layers; brown (SP).
		S10	59	2 in.	15.5	18			Sand. Medium to fine grained; about 5% nonplastic fines; about 5% subrounded fine gravel; several 3-8 mm thick layers silty fine sand; brown (SP).
							Hole caved 53 to 57 ft. Advanced NW casing to 57 ft		
		S11	110	2 in.	9.5	18			Sand. Layered; 38 mm silty fine sand with about 20% nonplastic fines; 90 mm fine to medium sand, about 10% nonplastic fines, about 10% coarse sand and fine gravel; 114 mm clean medium to fine sand, about 10% coarse sand and fine gravel; brown (SP).
							Advanced NW casing to 59 ft		

Elevation scale (EL. MSL, ft): -10, -12, -14, -16, -18, -20, -22, -24, -26, -28, -30, -32, -34, -36, -38

Depth scale (ft): 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50, 52, 54, 56, 58, 60

(continued)

LEGEND

N - Standard penetration resistance, blows/ft of a 140 lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample   Groundwater
U - Undisturbed samples

S - Shelby tube   N - Denison
F - Fixed piston   P - Pitcher
O - Osterberg   G - GEI

SAMP OD - Outside diameter of sampling spoon.

NOTES

1) Test pit dewatering system in operation during boring, no water level readings obtained.

PILGRIM STATION NO. 600
UNIT NO. 2
BOSTON EDISON COMPANY

Date: February 27, 1976 (Revised)

505

Bechtel Job No. 8791

GEOTECHNICAL ENGINEERS INC

BORING LOCATION N9995,025 E5449,429 INCLINATION Vertical BEARING N.A. DATE START/FINISH October 10, 1975 / October 14, 1975

CASING ID 3 in. CORE SIZE N.A. in. TOTAL DEPTH 91.0 ft DRILLED BY American Drilling & Boring, J. Taxeira

GROUND EL (MSL) -20.9 ft DEPTH TO WATER/DATE (1) ft / NA LOGGED BY W.E. PHI/F. D. Leathers

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
-40	60							2-7/8 inch Tricone rolled bit.	
-42	62						Advanced NW casing to 63 ft		
-44	64	S12	99	2 in.	11	18			Sand. Layered; 25 mm medium sand with about 10% nonplastic fines and about 10% coarse sand and fine gravel; 25 mm fine sand with about 20% nonplastic fines; 50 mm medium to fine sand with about 5% nonplastic fines; 180 mm fine sand with about 10% nonplastic fines, contains several 2 mm layers medium sand; brown (SP).
-46	66								
-48	68						Advanced NW casing to 69 ft		
-50	70	S13	81	2 in.	15	18			Sand. Medium to fine grained; about 10% subangular to subrounded gravel to 30 mm maximum size; about 5% coarse sand; about 5% nonplastic fines (SP).
-52	72								
-54	74						Gravel felt on roller bit at 75 ft		
-56	76	S14	169	2 in.	12.5	18			Sand. Medium to fine grounded; stratified; less than 5% nonplastic fines; one 2 mm layer dark brown silty fine sand; brown (SP).
-58	78						Gravel felt on roller bit 78.5 ft		
-60	80						Gravel felt on roller bit, 80 ft		
-62	82	S15	104	2 in.	10.5	18			Sand. Fine to medium grained; less than 5% nonplastic fines; brown (SP).
	84						Caving of hole at 75 to 88 ft	(84.0)	Approximate strata change determined from wash)
-64	86								
-66	88								
-68	90	S16	(2)	2 in.	12	17	Roller bit to 91.0 ft to confirm bedrock	(89.9)	Silt. Rapid dilatancy reaction; low dry strength; less than 2% scattered medium sand grains; one 40 mm thick layer silty medium to fine sand with about 5% fine subangular gravel (ML). (BEDROCK)

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample
U - Undisturbed samples
Groundwater

S - Shelby tube N - Denison
F - Fixed piston P - Pitcher
O - Osterberg G - GEI

SAMP OD - Outside diameter of sampling spoon

NOTES
1) Test pit dewatering system in operation during boring, no water level readings obtained.
2) Blows of 22/6", 64/6", and 91/5".

PILGRIM STATION NO. 400
UNIT NO. 2
BOSTON EDISON COMPANY

Date February 27, 1976 (Revised)

505

Bechtel Job No. 8791

BORING LOCATION N992-464 E5351.36 INCLINATION Vertical BEARING NA DATE START/FINISH March 15, 1976 / March 14, 1976

CASING ID (1), (2) in. CORE SIZE NA in. TOTAL DEPTH 87.1 ft DRILLED BY American Drilling & Boring Co. - K. Allen

GROUND EL (MSL) +21.55 ft DEPTH TO WATER/DATE (3) ft / LOGGED BY W. E. Pitt

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	SAMPLE N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
21.55		S-1	43	2 in.	16.5	18	Uncased hole using bentonite drilling mud	6 in. Tricone roller bit	Sand. Widely graded, with about 30% subangular to subrounded gravel to 55 mm max. size; brown. (SW)
							Occasional cobbles and boulders near surface		
		S-2	(4)	2 in.	None	2	Roller bitted cobbles & gravel		No Recovery.
	(7.0')								
		S-3	39	2 in.	11	18			Sand. Uniformly graded; medium to fine grained; about 5% nonplastic fines; contains occasional rounded lumps of silt; brown. (SP)
		S-4	47	2 in.	11.5	18			Sand. Layered: 50 mm slightly silty fine sand; 25 mm clean medium to coarse sand with trace subangular gravel; 90 mm clean fine to medium sand; 12 mm clean fine to coarse sand; 75 mm clean fine sand with a trace medium sand; 35 mm fine sand with a trace of fine gravel; brown. (SP)
		S-5	44	2 in.	12	18			Sand. Layered; 55 mm clean fine to coarse sand with a trace fine gravel; 51 mm clean fine to medium sand; 12 mm finely laminated silty fine sand; 175 mm clean uniform fine to medium sand with a trace coarse sand and fine gravel; brown. (SP)
		S-6	112	2 in.	11.5	18			Sand. Uniformly graded; fine grained; about 5% nonplastic fines; contains several 1 mm thick layers of silty fine sand and occasional small brown spots near top of sample; brown. (SP)

(Continued)

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample ⊽ Groundwater
U - Undisturbed samples
S - Shelby tube N - Denison
F - Fixed piston P - Pitcher
O - Osterberg G - GEI
SAMP OD - Outside diameter of sampling spoon

NOTES

(1) Drove 6 in. ID SW casing to 4 ft. Bentonite drilling fluid used to advance hole.
(2) After boring completed, 3 in. ID NW casing installed to 87.1 ft, but not grouted or backfilled.
(3) No water level readings obtained due to bentonite drilling fluid.
(4) Blows of 50/2".

PILGRIM STATION NO. 600
UNIT NO. 2
BOSTON EDISON COMPANY

Date: May 19, 1976

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GEOTECHNICAL ENGINEERS INC

BORING LOCATION N9928,468 E5351,36 INCLINATION Vertical BEARING NA DATE START/FINISH March 15, 1976 / March 18, 1976

CASING ID (1), (2) in. CORE SIZE NA in. TOTAL DEPTH 87.1 ft DRILLED BY American Drilling & Boring Co. - K. Allen

GROUND EL (MSL) -21.55 ft DEPTH TO WATER/DATE (3) ft / LOGGED BY W. E. Pitt

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	SAMPLE N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
		S-7	132	2 in.	13	18	Uncased hole using bentonite drilling mud	6 in. Tricone roller bit	Sand. Layered; 200 mm clean uniform fine to medium sand; 50 mm slightly stratified fine to coarse sand; 50 mm slightly silty fine sand; orange brown, rust brown, and tan. (SP)
-10	32								
-12	34								
		S-8	176	2 in.	11	18			Sand. Uniformly graded; medium to fine grained; trace of coarse sand; contains several 1 mm black-stained layers; brown (SP)
-14	36								
-16	38								
-18	40	S-9	136	2 in.	12.5	18			Sand. Layered; 150 mm clean fine to coarse sand; 3 mm silty fine sand; 45 mm fine to medium sand with a trace coarse sand; 90 mm thinly laminated silty fine sands and fine sandy silts; 25 mm slightly silty fine to coarse sand and about 5% subrounded fine to coarse gravel up to 23 mm; brown. (SP and SW)
-20	42								
-22	44								
-24	46	S-10	121	2 in.	12	18			Sand. Layered; 275 mm widely graded fine to coarse sand and trace of subangular to subrounded gravel to 15 mm with several interbeds of silty fine sand; 25 mm slightly silty fine to coarse sand and gravel to 12 mm; brown. (SW)
-26	48								
-28	50	S-11	105	2 in.	12	18			Sand. Layered; 25 mm fine to medium sand; 25 mm fine to medium sand and fine gravel; 37 mm fine to coarse sand; 20 mm medium to coarse sand; 100 mm laminated silty fine sand and 1 mm layers of silt; 90 mm silty fine to coarse sand; brown. (SW and SP)
-30	52								
-32	54								
-34	56	S-12	107	2 in.	11.5	18			Sand. Layered; 135 mm clean uniform fine to medium sand; 135 mm slightly silty fine to medium sand and subrounded to subangular gravel to 35 mm; brown. (SP and SW)
-36	58								
-38	60								(Continued)

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample   Groundwater
U - Undisturbed samples
S - Shelby tube   N - Denison
F - Fixed piston   P - Pitcher
O - Osterberg   G - GEI
SAMP OD - Outside diameter of sampling spoon

NOTES
(1) Drove 6 in. ID SW casing to 4 ft. Bentonite drilling fluid used to advance hole.
(2) After boring completed, 3 in. ID NW casing installed to 87.1 ft, but not grouted or backfilled.
(3) No water level readings obtained due to bentonite drilling fluid.

PILGRIM STATION NO. 600
UNIT NO. 2
BOSTON EDISON COMPANY

Date: May 19, 1976

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BORING LOCATION N9928.468 E5351.36 INCLINATION Vertical BEARING NA DATE START/FINISH March 15, 1976 / March 18, 1976

CASING ID (1), (2) in. CORE SIZE NA in. TOTAL DEPTH 87.1 ft DRILLED BY American Drilling & Boring Co. - K. Allen

GROUND EL (MSL) -21.55 ft DEPTH TO WATER/DATE (3) ft / LOGGED BY W. E. Pitt

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	SAMPLE N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
	60	S-13	188	2 in.	12	18	Uncased hole using bentonite drilling mud	6-in. Tricone roller bit	Sand. Uniformly graded; medium to fine grained with about 5% coarse sand and about 10% subangular gravel to 10 mm max. size; brown. (SP)
-40	62								
-42	64								
-44	66	S-14	(4)	2 in.	12	17			Sand. Layered; fine to medium sand with occasional 1 mm layer of silty fine sand; 25 mm fine to medium sand with about 5% subrounded to subangular gravel to 15 mm max. size; brown. (SP)
-46	68								
-48	70	S-15	180	2 in.	11	18			Sand. Layered; 125 mm medium to coarse sand with a trace fine sand and fine gravel; 75 mm finely stratified fine to medium sand; 75 mm medium to coarse sand with a trace round to subround fine gravel; brown. (SP)
-50	72								
								(73.0')	
-52	74								
-54	76	S-16	133	2 in.	16	18			Gravelly sand. Widely graded; coarse to fine grained; about 30% subrounded to subangular fine to coarse gravel to 30 mm max. size; contains about 10% nonplastic fines; brown. (SW)
-56	78								
-58	80	S-17	(5)	2 in.	11	18			Sand. Uniformly graded; medium to fine grained with about 5% coarse sand and about 10% fine to coarse subrounded to subangular gravel to 30 mm, and about 5% nonplastic fines; brown. (SP)
-60	82								
-62	84								
-64	86	S-18	(6)	2 in.	8.5	10	3-in. NW casing placed in completed hole.		Sand. Widely graded; medium to fine grained with about 10% coarse sand and about 15% fine to coarse subrounded to subangular gravel to 30 mm, and about 10% slightly plastic fines; brown. (SW)
-65.55								(87.1')	

PROBABLE BEDROCK - BOTTOM OF BORING

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample ▽ Groundwater
U - Undisturbed samples
S - Shelby tube N - Denison
F - Fixed piston P - Pitcher
O - Osterberg G - GEI
SAMP OD - Outside diameter of sampling spoon

NOTES
(1) Drove 6 in. ID SW casing to 4 ft. Bentonite drilling fluid used to advance hole.
(2) After boring completed, 3 in. ID NW casing installed to 87.1 ft, but not grouted or backfilled.
(3) No water level readings obtained due to bentonite drilling fluid.
(4) Blows of 49/5" using 140-lb hammer and 96/6" and 61/6" using 300-lb hammer.
(5) Blows of 138/6" using 140-lb hammer and 83/6" and 106/4" using 300-lb hammer.
(6) Blows of 99/6" using 140-lb hammer and 31/2" using 300-lb hammer.

PILGRIM STATION NO. 600
UNIT NO. 2
BOSTON EDISON COMPANY

Date: May 19, 1976

PAGE 3 of 3

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GEOTECHNICAL ENGINEERS INC

BORING LOCATION N9478.03 E5530.64 INCLINATION Vertical BEARING NA DATE START/FINISH March 9, 1976 / March 12, 1976

CASING ID (1), (2) in. CORE SIZE NA in. TOTAL DEPTH 89.7 ft DRILLED BY American Drilling & Boring Co. - K. Alian

GROUND EL (MSL) +22.05 ft DEPTH TO WATER/DATE (3) ft / NA LOGGED BY W. E. Pitt

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	SAMPLE N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
22.05	0	S-1	16	2 in.	11.5	18	Uncased hole using bentonite drilling mud	6 in. Tricone roller bit	Sand. Uniformly graded; medium to fine grained with about 5% coarse sand and about 5% subrounded to subangular fine to coarse gravel to 25 mm; brown. (SP)
20	2								
18	4								
16	6	S-2	54	2 in.	13	18			Sand. Uniformly graded; slightly mottled fine to medium grained with about 5% coarse sand and about 5% subrounded to subangular gravel to 15 mm; last 50 mm silt and fine sand with trace fine gravel; rusty brown and gray. (SP)
								(7.2')	
14	8								
12	10	S-3	65	2 in.	14	18			Sand. Layered; widely graded very silty fine to medium sand and coarse sand with subrounded to subangular gravel to 35 mm; contains a 25 mm layer of coarse sand and gravel midway through the sample; mottled rust brown to gray. (SW - SM)
10	12								
8	14	S-4	(4)	2 in.	0	4	Roller bit boulder and gravel to 20 ft		No Recovery
6	16								
4	18								
2	20	S-5	97	2 in.	Not meas.	18			Gravelly sand. Widely graded; fine to coarse grained with about 25% subangular to subrounded gravel to 40 mm and about 10% nonplastic fines; sample slightly mottled; brown to rusty brown. (SW - SM)
0	22								
-2	24						Roller bit boulder from 23.5 to 24.5 ft	(24.5')	
-4	26	S-6	36	2 in.	10.5	18			Sand. Widely graded; fine to coarse grained with about 10% subrounded to subangular gravel to 12 mm and about 15% nonplastic fines; brown. (SW - SM)
-6	28								
-8	30								(Continued)

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample
U - Undisturbed samples
Groundwater
S - Shelby tube N - Denison
F - Fixed piston P - Pitcher
O - Osterberg G - GEI
SAMP OD - Outside diameter of sampling spoon

NOTES
(1) Drove 6 in. ID SW casing to 4 ft. Bentonite drilling fluid used to advance hole.
(2) After boring completed, 3 in. ID NW casing set in hole to 89.7 ft but not grouted or backfilled.
(3) No water level reading obtained due to use of bentonite fluid.
(4) Blows of 68/4" before encountering boulder.

PILGRIM STATION NO. 400
UNIT NO. 2
BOSTON EDISON COMPANY

Date: May 12, 1976

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GEOTECHNICAL ENGINEERS INC

BORING LOCATION N9878.03 E5530.68 INCLINATION Vertical BEARING NA DATE START/FINISH March 9, 1976 / March 12, 1976

CASING ID (1), (2) in. CORE SIZE NA in. TOTAL DEPTH 89.7 ft DRILLED BY American Drilling & Boring Co. - K. Allen

GROUND EL (MSL) 22.05 ft DEPTH TO WATER/DATE (3) ft / LOGGED BY W. E. Pitt

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	SAMPLE N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
-8	30	S-7	36	2 in.	11	18	Uncased hole using bentonite drilling mud	6 in. Tricone roller bit	Sand. Layered: 100 mm uniform clean fine sand; 175 mm slightly silty fine to medium sand with about 5% subangular to subrounded coarse sand and fine gravel to 7 mm in size; brown. (SP)
-10	32								
-12	34								
-14	36	S-8	114	2 in.	15	18			Sand. Layered 75 mm clean, uniform fine sand; 3 mm clean medium to coarse sand; 95 mm clean fine to coarse sand; 50 mm uniform medium to coarse sand; 125 mm uniform slightly silty fine sand with 1 mm layers of apparently oxidized silty fine sand; gray brown. (SP)
-16	38								
-18	40	S-9	99	2 in.	14.5	18			Sand. Uniformly graded; fine to medium sand grading to medium to coarse grained; contains about 5% coarse sand and about 5% nonplastic fines; orange brown. (SP)
-20	42								
-22	44								
-24	46	S-10	91	2 in.	15	18			Sand. Uniformly graded; fine to medium grained with about 5% coarse sand; contains several 1 mm layers fine to coarse sand and evidence of stratification; light brown. (SP)
-26	48								
-28	50	S-11	99	2 in.	14.5	18			Sand. Similar to Sample S-10.
-30	52								
-32	54								
-34	56	S-12	88	2 in.	15	18			Sand. Uniformly graded; fine to coarse grained with about 5% subrounded to subangular fine gravel to about 7 mm and about 5% nonplastic fines; brown. (SP)
-36	58								
-38	60								(Continued)

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler

REC - Length of sample recovered

S - Split spoon sample

U - Undisturbed samples

S - Shelby tube N - Denison

F - Fixed piston P - Pitcher

O - Osterberg G - GEI

SAMP OD - Outside diameter of sampling spoon

Groundwater

NOTES

(1) Drove 6 in. ID SW casing to 4 ft. Bentonite drilling fluid used to advance hole.

(2) After boring completed, 3 in. ID NW casing set in hole to 89.7 ft but not grouted or backfilled.

(3) No water level reading obtained due to use of bentonite fluid.

PILGRIM STATION NO. 600
UNIT NO. 2
BOSTON EDISON COMPANY

Date May 12, 1976

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GEOTECHNICAL ENGINEERS INC

BORING LOCATION N9878.03 E5530.88 INCLINATION Vertical BEARING NA DATE START/FINISH March 9, 1976 / March 12, 1976

CASING ID (1), (2) in. CORE SIZE NA in. TOTAL DEPTH 89.7 ft DRILLED BY American Drilling & Boring Co. - K. Allen

GROUND EL (MSL) -22.05 ft DEPTH TO WATER/DATE (3) ft / LOGGED BY W. E. Pitt

EL. MSL ft	SAMPLE Depth ft	SAMPLE Type and No.	SAMPLE N	SAMP OD	LENGTH REC in.	LENGTH PENETRATION in.	REMARKS ON ADVANCE OF BORING	SIZE/TYPE BIT USED TO ADVANCE BORING	SOIL AND ROCK DESCRIPTIONS (ASTM D2487-69 and D2488-69)
-38	60	S-13	117	2 in.	14	18	Uncased hole using bentonite drilling mud Caving of hole at 34 ft	6 in. Tricone roller bit	Sand. Layered: 100 mm clean uniform fine to medium sand with a trace coarse sand and fine gravel; 12 mm clean medium to coarse sand and subrounded to subangular fine gravel to 10 mm; 62 mm clean uniform fine sand with several 1 mm layers of apparently oxidized fine sand; 25 mm clean uniform fine to coarse sand; 150 mm fine to medium sand with a trace coarse sand and fine gravel; brown. (SP)
-40	62								
-42	64								
		S-14	216	2 in.	16	18			Sand. Layered; 25 mm, clean uniform medium sand; 50 mm clean uniform fine to medium sand; 150 mm clean medium to coarse sand with about 5% fine sand; 175 mm medium to coarse sand with about 5% nonplastic fines and about 5% subrounded to subangular gravel to 30 mm; brown. (SP)
-44	66								
-46	68								
-48	70	S-15	(4)	2 in.	12	18			Sand. Uniformly graded; medium grained with about 10% fine and coarse sand, about 5% subrounded to subangular gravel 10 mm, and about 5% nonplastic fines; brown. (SP)
-50	72								
-52	74								
-54	76	S-16	154	2 in.	17.5	18			Sand. Layered; 50 mm clean uniform fine to medium sand with a 1 mm layer of silty fine sand; 38 mm clean fine to coarse sand with a trace of fine gravel; 25 mm clean uniform fine to medium sand; 62 mm clean fine to coarse sand; 6 mm uniform silt; 45 mm fine to coarse sand; 125 mm clean uniform fine to medium sand with trace gravel to 10 mm; 90 mm fine to coarse sand with about 10% subrounded to subangular gravel to 25 mm; brown. (SP)
-56	78								
-58	80	S-17	(5)	2 in.	14	18			Gravelly sand. Widely graded; fine to coarse grained sand with about 40% fine to coarse subangular to angular gravel to 35 mm and about 5% nonplastic fines; brown. (SW)
-60	82							(83.0')	
-62	84								Sand. Widely graded; fine to coarse grained sand with about 10% fine to coarse subrounded to subangular gravel to 20 mm and about 20% plastic fines; brown. (SW - SM)
								(85.5')	
-64	86	S-18	(6)	2 in.	NA	18			Sand. Uniformly graded; fine to medium grained sand with about 5% coarse sand and about 5% nonplastic fines; brown. (SP)
-66	88								
-67.65	90						3 inch NW casing placed in completed hole.	(89.7')	PROBABLE TOP OF BEDROCK BOTTOM OF BORING

LEGEND

N - Standard penetration resistance, blows/ft of a 140-lb hammer falling 30 in. to drive a split-spoon sampler
REC - Length of sample recovered
S - Split spoon sample
U - Undisturbed samples
Groundwater

S - Shelby tube N - Denison
F - Fixed piston P - Pitcher
O - Osterberg G - GEI

SAMP OD - Outside diameter of sampling spoon

NOTES

(1) Drove 6 in. ID SW casing to 4 ft. Bentonite drilling fluid used to advance hole.
(2) After boring completed, 3 in. ID NW casing set in hole to 89.7 ft but not grouted or backfilled.
(3) No water level reading obtained due to use of bentonite fluid.
(4) Blows of 88/6", 50/2" using 140-lb hammer and 31/4", 52/6" using 300-lb hammer
(5) Blows of 108/6" using 140-lb hammer; 47/6", 75/6" using 300-lb hammer.
(6) Blows of 84/6", 107/6" using 140-lb hammer and 57/6" using 300-lb hammer.

PILGRIM STATION NO. 800
UNIT NO. 2
BOSTON EDISON COMPANY

Date: May 12, 1976

610

GEI Consultants, Inc.

1021 Main Street
Winchester, MA 01890-1943
617•721•4000

July 10, 1992
Project 92012

Mr. Thomas J. Tracy
Vice President
Stevenson & Associates
Ten State Street
Woburn, MA 01801

Dear Mr. Tracy:

Re: **Uncertainties in Soil Failure Analyses**
Pilgrim 1 IPEEE, Pilgrim Station, Plymouth, Massachusetts

This letter is in response to Dr. Fred Mogolesko's request at the June 11, 1992, meeting with BECo, Stevenson & Associates, and GEI Consultants, Inc. for a letter describing any uncertainties associated with the soil failure analyses.

We understand that the Probabilistic Risk Analysis (PRA) involves mean point estimation to evaluate the seismic vulnerability of the plant, and therefore, best estimates of the soil displacements are needed for input into the fragility analyses for the PRA. Due to the fairly high density of the soils at the Pilgrim site, it was determined prior to performing the soil failure analyses that relatively simple methods of analysis would be adequate. Some reasonably conservative assumptions were made to apply these methods of analysis.

The uncertainty associated with the analyses conducted and the conservative nature of the assumptions made are described below for each set of results from the soil failure analyses. These results are: 1) stability against liquefaction failure, 2) permanent horizontal displacements, 3) settlements, and 4) transient horizontal displacements. Next, the earthquake input is discussed. The impact of the soil parameters, the earthquake input, and the methods of analysis on the results are then summarized.

The uncertainties for the values of shear wave velocity for the soil strata at the Pilgrim site are discussed in the revised report prepared by GEI. The analyses of settlements,

permanent horizontal displacements, and transient displacements were performed for two shear wave velocity profiles that are upper and lower bounds for the probable actual *in situ* values.

Liquefaction Stability

The blowcounts and laboratory test data for the Pilgrim site indicate that the outwash deposits are very dense and highly dilatant. The compaction specifications and laboratory test data indicate that the fill at the Pilgrim Unit 1 site is heavily compacted and also highly dilatant. The strength of these highly dilatant soils would be higher for the undrained shearing that occurs during earthquake shaking than they are for drained shearing.

A conservative determination of the factor of safety using the drained steady-state strengths of the soils gives a value of 1.9, which is quite high. Typically, factors of safety of 1.5 are considered to be adequate. Since the soil response will be undrained during shaking, the actual factor of safety against liquefaction instability is much higher than 1.9.

Permanent Horizontal Displacements

A finite amount of permanent horizontal displacement can be expected to occur during shaking as a result of movements of the ground downslope toward the waterfront. These displacements have been calculated for the structures as a function of the peak ground acceleration.

The expression used to estimate the permanent horizontal displacement bounds nearly all of the values of displacement calculated for a large number of western United States earthquake acceleration records by Newmark (1965) and later by Franklin and Chang (1977). The characteristics of eastern seismicity were taken into account by using the hazard results in NUREG/CR-5250 and EPRI Report No. NP-6395-D for the Pilgrim site to determine the values of ground velocity to input into the expression for displacement.

The values for the yield acceleration used to calculate the displacements are based on conservatively selected values of yield strength. The yield strengths were conservatively based on the peak drained strength of the soil, since there would be some uncertainty as to whether the higher undrained strengths could be fully mobilized.

The values for the maximum acceleration of the potential soil mass subject to permanent movement are based on the maximum shear stresses calculated using the computer program, SHAKE. The maximum shear stresses are conservative for the reasons described below in the section on the earthquake input.

Settlements

Correlations available in the literature were used to determine the relationships between the volumetric compression and peak seismic shear strain for the soils at the Pilgrim site. The correlations are based on test data involving several different granular materials and, therefore, involve considerable scatter. Mid-range values were used to determine the relationships. Better estimates can only be made by using site-specific test data.

The volumetric compression increases with the peak seismic shear strain. The peak seismic shear strains calculated using the computer program SHAKE are conservative for the reasons described below in the section on the earthquake input.

Transient Displacements

The results of the SHAKE analyses represent one-dimensional wave propagation through the soil in the free field and do not account for the effects of the weight and flexibility of the structure or the effects of rocking of the structure.

Since the weights of the buildings are approximately equal to the excavated soil that they replace, the presence of the building will have a small effect on the accuracy of the calculated shear strains in the soil.

The buildings were assumed to be rigid by taking the building displacement to be equal to the displacement calculated by SHAKE at the elevation of the base of the building foundation; this is conservative since any flexibility of the building foundation will result in smaller differential transient horizontal displacements between the building and the surrounding soil.

The effect of rocking on the structures is likely to be small due to the fairly high stiffness of the soils, with the possible exception of the Condenser Tanks due to their high center-of-gravity and small depth of embedment.

The differential transient displacements were taken to be the absolute sum of the peak transient displacements of the points being considered, e.g., the building and the adjacent soil, which is conservative.

The transient displacements were calculated by integrating the peak seismic shear strains calculated using the computer program SHAKE. The peak seismic shear strains are conservative for the reasons described below in the section on the earthquake input.

Earthquake Input

The acceleration time history used for the SHAKE analyses is conservative for determining the peak seismic shear stresses and strains in the soil deposit, which are thus conservative for calculating the settlements and the permanent and transient horizontal displacements. The time history was synthesized by Stevenson & Associates using the NUREG/CR-0098 spectrum. For the same value of peak acceleration, the NUREG/CR-0098 spectrum envelopes the response spectra provided in NUREG/CR-5250 and the EPRI Report No. NP-6395-D for the Pilgrim site. The NUREG/CR-0098 gives a maximum spectral velocity of 80 in./sec/g compared to values of 30 and 18 in/sec/g for the NUREG/CR-5250 and EPRI spectra, respectively.

The effect of the spectral shape on the peak seismic shear stresses and strains is illustrated in Figs. 9 and 10 in the report by GEI. The results for the acceleration record based on the NUREG/CR-0098 spectrum scaled to 0.2 g are presented in Fig. 9. The results for an acceleration record for which all the acceleration values except the peak value are half of those of the acceleration record for Fig. 9 are presented in Fig. 10. Thus the ordinates of the response spectrum for the acceleration record for Fig. 9 are essentially twice those for Fig. 10, with the exception of very high frequencies where the peak accelerations are identical. It can be seen that the peak seismic shear stresses and strains in Fig. 9 are about double those in Fig. 10.

Summary

The results of the soil failure analyses are conservative estimates of the soil displacements rather than mean value estimates. The degree of conservativeness for each set of results is affected by uncertainties associated with the values of the soil parameters, the earthquake input, and the methods of analysis.

The soil parameters relevant to the soil failure analyses are 1) shear wave velocity, V_s; 2) modulus reduction and damping curves, G/G_{max} and D versus shear strain; 3) density; 4) drained and undrained shear strengths; and 5) volumetric compression versus shear strain.

The probable actual shear wave velocities at the site are bounded by the two sets of values determined and used for the soil failure analyses. The average modulus reduction and damping curves presented in Seed and Idriss (1970) for sands were used for the SHAKE analyses. The shapes of these curves have little effect on the resulting accelerations, shear stresses, and shear strains, especially for very stiff soils such as those at Pilgrim.

The densities and drained strengths of the soils are well known from the results of *in situ* tests and laboratory testing on both compacted, remolded samples and undisturbed

samples. The test data also indicate that the undrained strength is greater than the drained strength.

The relationships between compression of the soil and seismic shear strain are reasonable.

The earthquake input record is reasonable and is conservative for the purpose of calculating the soil displacements. The effects of eastern United States seismicity have been taken into account by selecting the peak ground velocity, V, based on the hazard results for the Pilgrim site.

The results of SHAKE analyses represent one-dimensional propagation of shear waves vertically through the soil strata. The effects of soil-structure-interaction or rocking of the structures on the settlements and the permanent horizontal displacements are small. Conservative approximations have been made to determine values for the transient displacements from the results of the SHAKE analyses.

Please call me if you have any questions.

Sincerely yours,

GEI CONSULTANTS, INC.

Eugene Marciano

Eugene A. Marciano, Ph.D.
Project Manager

EAM:ms

indicates that the largest realistic peak ground surface acceleration for soil profile 1 is less than 0.35 g.

SHAKE analyses were conducted using NR0098-2 scaled to peak accelerations of 0.2, 0.4, 0.6, and 0.8 g. The record was applied to the surface of the deposit.

Typical results for each of the records scaled to 0.2 g are shown in Figs. 9 and 10. The maximum shear strains and shear stresses and peak accelerations are plotted versus depth.

Comparison of Figs. 9 and 10 indicates that, for the same peak acceleration, NR0098-1 produces larger values for the maximum shear stresses and maximum shear strains than NR0098-2. This result is not unexpected since the single, high frequency, narrow peak in the NR0098-2 record has little effect on the soil profile. This is confirmed by the fact that the stresses and strains in the soil profile are about the same for the NR0098-1 record and for the NR0098-2 record with twice the peak acceleration of the NR0098-1 record.

Based on the above, the results of the analyses obtained with record NR0098-1 were used to determine shear stresses and strains in the soil and accumulated deformations as a function of the peak ground acceleration.

4.2.2 Results for Profile 2

SHAKE analyses were conducted using NR0098-1 scaled to peak accelerations of 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.75, 0.80, 0.85, and 0.9. The record was applied to the surface of the deposit. Results were not obtained for 0.9 g since the program did not converge to strain-compatible values of shear moduli due to the severity of the ground motion. This indicates that for the NUREG-0098 design spectrum, a peak ground acceleration higher than about 0.85 g is not possible for soil profile 2. In addition, for peak ground accelerations of 0.6, 0.7, 0.75, and 0.80 g, the peak acceleration at the bedrock is 0.67, 1.2, 2.3, and 5 g, respectively. This indicates that the largest realistic peak ground surface acceleration for soil profile 2 is less than about 0.7 g.

SHAKE analyses were conducted using NR0098-2 scaled to peak accelerations of 0.2, 0.6, 0.8, 1.0, and 1.4 g.

Typical results for each of the records scaled to 0.20 g are shown in Figs. 9 and 10. The maximum shear strains and shear stresses and peak accelerations are plotted versus depth.

Comparison of Figs. 9 and 10 indicates that NR0098-1 produces larger shear stresses and shear strains than NR0098-2. Based on the discussion in Subsection 4.2.1, the results of NR0098-1 are the appropriate values to use.

4.2.3 Discussion of SHAKE Results

The results of the SHAKE analyses represent the case of one-dimensional wave propagation through the soil profile in the free field. These results do not account for the effects of soil-structure-interaction or rocking of the structure. These effects are not likely to have a substantial impact on the values of the settlements and the permanent horizontal displacements. They can have a significant effect on the transient displacements of the structures. In general, the effects of soil-structure-interaction would be to reduce the differential transient displacements. The effect of rocking could be to increase the transient displacements. However, the fairly high stiffness of the soil strata at the Pilgrim site make it likely that for most of the structures the effect of rocking will be small and will be compensated by the effects of soil-structure-interaction. The Condenser Tanks appear to be the most likely structures to be significantly affected by rocking, due to their high center-of-gravity and relatively small depth of embedment. If the issues of transient displacements and thus soil-structure-interaction and rocking are found to be critical, a two- or three-dimensional model of the soil and the structure should be used to more accurately estimate differential transient displacements.

4.3 Pseudostatic Analyses

4.3.1 Purpose and Method

Pseudostatic stability analyses were conducted for the critical structures in the three profiles shown in Figs. 3, 4, and 5. The purpose of these analyses was to determine the yield accelerations for each critical structure. The yield acceleration is the value of horizontal acceleration which gives a pseudostatic factor of safety of 1.

The computer program, STABL5, was used to perform the stability analyses. The Modified Bishop method of slices for circular failure surfaces was used. Circular failure surfaces are critical for all of the cases except for the Intake Structure. A wedge analysis using Janbu's method of stability analysis in STABL5 was conducted for the Intake Structure. The wedge was taken to coincide with the bottom of the structure. This is equivalent to analyzing the Intake Structure as a gravity-retaining structure.

The geometries used for the stability analyses are shown in Figs. 3, 4, and 5 and are based on Bechtel Drawings No. C1 through C9 and M15 through M29. The

bedrock was taken to be at El. -75 feet, which is the deepest elevation reported for the boring logs at the Pilgrim 1 and 2 sites. The gross bearing pressures given in Section 2.3 for the structures shown in Figs. 3, 4, and 5 were applied at the foundation levels of the structures. In addition, a horizontal shear load equal to the horizontal acceleration times the gross bearing pressure was applied at the foundation level.

The selection of the yield strengths for the fill and outwash materials is discussed in Subsection 4.3.2. The strength parameters for the riprap and the ground water table elevations used are the same as those for the stability analysis described in Section 3.

For each structure, a search was conducted to determine the critical surface and factor of safety for several values of horizontal acceleration. The search was conducted for surfaces restricted to intersecting at least part of the structure. The value of horizontal acceleration giving a pseudostatic factor of safety of 1 is the yield acceleration. This value was determined by interpolation between the acceleration values for factors of safety bracketing 1.0, as shown in Figs. 3, 4, and 5.

Note that the pseudostatic factor of safety will be equal to or less than 1 only if and when the peak value of the acceleration time history of the soil mass exceeds the yield acceleration and then only for fractions of a second during the course of shaking. The result will be a finite permanent displacement of the soil mass and not instability of the soil mass. The method for calculating the permanent displacement of the soil mass is described in Subsection 4.4.1. In general, the peak value of acceleration for the soil mass must be substantially larger than the yield acceleration for the calculated permanent displacement to be significant.

4.3.2 Soil Strengths for the Pseudostatic Analyses

During undrained shearing of dilative soils, the shear stress increases until it reaches the value of the undrained steady-state shear strength. As the soil is sheared, the pore pressure decreases, and therefore, the effective stress increases. For a highly dilative soil, such as is the case for the soils at the Pilgrim site, the pore water pressure can decrease to a net pressure of -1 atmosphere in theory. However, phenomena such as release of dissolved gases due to the reduced pore pressure can limit the net pore pressure to much smaller values, perhaps on the order of -0.5 atmosphere. A reasonably conservative estimate of the resulting shear strength for the stability analyses conducted at the Pilgrim site is the drained peak shear strength.

The peak friction angle in the triaxial compression tests discussed in Section 2 was 38.8 degrees and was estimated as 42 to 45 degrees based on the blowcount data. The peak friction angles in the triaxial compression tests for the compacted samples ranged from 40.5 to 43 degrees. Based on these results, a value of 40 degrees was selected to estimate the yield strengths of the compacted fill and the glacial outwash. The strength along the critical surface was calculated using the following expression:

$$S_y = \sigma'_f \tan\phi$$

where S_y = the yield strength
σ'_f = effective normal stress on the failure surface
ϕ = friction angle of 40°

with σ'_f computed assuming drained conditions. This provides a reasonably conservative estimate of the yield strengths.

4.4 Analytical Methods for Calculating the Permanent Displacements

4.4.1 Displacements Due to Slope Movements

The displacements due to slope movements were estimated using Newmark's (1965) method of deformation analysis. In this method, one considers the movements of the soil mass above the critical surface. Permanent displacement of the soil mass occurs when its acceleration time history exceeds the yield acceleration. The yield acceleration is the value of horizontal acceleration that gives a pseudostatic factor of safety of 1 for the given slope geometry when applied to the soil mass in question. The displacement is computed by double integration of the difference between the acceleration time history and the yield acceleration whenever the acceleration exceeds the yield value. Integration is continued until the relative velocity of the soil mass is zero. Integration resumes each time the acceleration exceeds the yield value. The displacement is assumed to occur in the downslope direction only.

The following approximate expression was used to calculate the permanent displacements.

$$D = \frac{V^2}{2gN}\left(1 - \frac{N}{A}\right)\frac{A}{N}$$

where:

D = Permanent displacement of the soil mass above the critical surface
V = peak particle velocity of the soil mass
N = yield acceleration in units of g
A = maximum value of the acceleration of the soil mass
g = acceleration of gravity

This expression was developed by Newmark (1965) and provides a reasonable upper bound for the displacements for values of N/A greater than about 0.15. It was determined based both on theoretical considerations and comparison to the results of integrating actual recorded acceleration time histories for different values of yield acceleration.

The above expression was used to calculate the permanent horizontal displacement for a range of values of the peak ground acceleration. For a given value of peak ground acceleration, the maximum value of the acceleration of the soil mass and the peak value of the velocity need to be determined to enter into the above expression for displacement.

The maximum value of the acceleration of the soil mass was computed by dividing the peak shear stress calculated by SHAKE at the depth of interest by the total vertical stress at that depth. The soil profile used for the SHAKE analyses was used to compute the total stresses. For each critical surface, the maximum average accelerations were computed just above the depth of the water table, at or near the bottom of each critical surface, and approximately midway between the water table and the bottom of the critical surface. These three values were generally close in value and decreased slightly with depth. The average of these three values was used for the ground motion parameter, A.

Based on previous work using acceleration time histories recorded in the western United States, the ratio of the peak ground velocity to the peak ground acceleration was estimated to be 48 in./sec/g for competent soil conditions. However, this ratio is dependent on the prevalent frequency content of the ground motion and thus can be expected to be different for earthquakes occurring in the eastern United States. The effect of the frequency content of eastern earthquakes is taken into account by calculating the ratio using the hazard results provided in NUREG/CR-5250 (1989) for the Pilgrim site. Using these hazard results, the ratio was calculated to be 20 in./sec/g. The method of calculation is described below.

Based on the hazard curves and the uniform hazard spectra provided in NUREG/CR-5250 for a return period of 10,000 years and a damping ratio of 5% of critical damping, the median values for the maximum spectral velocity and the

peak ground acceleration are about 23 cm/sec and 0.3 g, respectively. Based on NUREG/CR-0098 (1978), the median value of the amplification factor (i.e., the ratio of the maximum spectral velocity to the peak ground velocity) for spectral velocity is 1.65. Dividing the maximum spectral velocity of 23 cm/sec by 1.65 gives a value of 14 cm/sec for the peak ground velocity. Dividing 14 cm/sec by the peak ground acceleration of 0.3 g gives 46 cm/sec/g, or 18 in./sec/g, which for the analysis presented in this report was approximated to 20 in./sec/g.

A similar computation was performed using the median EPRI Response spectrum and peak ground acceleration for a 10,000 year return period. The result is a peak ground velocity of about 11 in./sec/g.

The displacements were calculated for the NUREG/CR-5250 hazard results. Therefore, the value of V was taken equal to 20 in./sec/g multiplied by the applicable value of peak ground surface acceleration.

4.4.2 Seismically Induced Settlements of Level Ground

Settlement of the ground surface and structures due to seismic shaking is the result of: 1) densification during shaking of cohesionless materials located above the water table and 2) reconsolidation of materials located below the water table, shortly after the earthquake, due to dissipation of pore pressures developed during shaking.

The results of several experimental investigations of the densification of cohesionless materials above the water table and the reconsolidation of materials below the water table is presented in Castro, 1987 (see Figs. 11 and 12).

4.4.2.1 Densification of Soils Above the Water Table

The strains due to densification of sands and silts for drained conditions increase with the number of cycles of uniform shear-strain amplitude and the value of the shear-strain amplitude. The strains decrease with increasing initial density of the materials. Based on the available correlations, the log of the vertical strain was taken to vary linearly with the log of the shear-strain amplitude between the following values for the compacted fill at the Pilgrim 1 site, as plotted in Fig. 11.

Shear Strain Amplitude (%)	Vertical Strain (%)
0.01	0.01
0.5	0.6

The above relationship is for 10 cycles of uniform shear-strain amplitude. For an earthquake, the effective shear-strain amplitude should be used, which is taken to be 65% of the maximum shear-strain amplitude induced by the earthquake.

The vertical strain increases with the number of cycles of shaking. The strain for 40 cycles is close to double that for 10 cycles. Cycles in excess of about 40 produce little additional strain.

Seed et al (1983) indicated the following number of equivalent, uniform amplitude cycles versus earthquake magnitude, which is applicable to earthquakes in the western United States:

Magnitude	Number of Cycles
8.5	26
7.5	15
6.75	10

Earthquakes in the eastern United States generally have had a higher frequency content than western earthquakes and consequently have a larger number of equivalent uniform cycles for a given magnitude. Therefore, the number of equivalent uniform cycles may be closer to 40 than 26 for a magnitude 8.5 and higher event. Therefore, we calculated the strains in the compacted fills above the water table based on 10 cycles and then increased the resulting values by 100%.

4.4.2.2 Consolidation of Soils Below the Water Table

The results of experiments conducted using constant amplitude cyclic stress tests (Castro, 1987) and recorded earthquake time histories (Nagase et al, 1988) indicate that the reconsolidation strain for materials below the water table is dependent on the maximum value of shear strain that occurs during the earthquake. Based on the available correlations, the log of the consolidation strain of the glacial outwash and the compacted fills below the water table was estimated to vary linearly with the log of the maximum shear strain between the following values, as plotted in Fig. 12:

Maximum Shear Strain (%)	Consolidation Strain (%)
0.09	0.01
0.20	0.05
0.50	0.30
2.0	1.0

4.4.3 Settlement of Structures

The effect of the structures on the vertical strains in the underlying foundation soils depends on the influence of the structure on the seismically induced shear strains. These strains are affected by the seismic base shear exerted by the structure at the bottom of its foundation. The seismic base shear depends on the mass and dynamic response of the building.

The net bearing pressures under the structures range between -2 and +2 ksf (see Section 2.2), with the extreme low and high values occurring at approximately El. -25 feet. A stress change of 2 ksf represents the equivalent of removing or adding approximately 15 feet of soil. This represents a change in total mass, which is fairly small compared to the total thickness of the soil deposits of about 80 feet. Therefore, the seismic base shear exerted by the building is not expected to alter the seismic shear strains in the underlying foundation soils appreciably. Consequently, the seismically induced settlements of the buildings should be about the same as those computed in the soil column at the elevation of the foundation base.

4.5 Results of Displacement and Settlement Calculations

4.5.1 Displacements Due to Slope Movements

The computed horizontal displacements for soil profile 2 for the critical surfaces shown in Figs. 3, 4, and 5 are presented in Table 1. For a peak ground surface acceleration of 0.7 g, all of the displacements are less than 1 inch, with the exception of the Intake Structure, which has a calculated displacement of under 2 inches.

The results for soil profile 1 are slightly lower than those for soil profile 2, since the maximum values of average acceleration are slightly lower than those for soil

profile 2. This can be seen from Figs. 9 and 10, where the maximum shear stresses for soil profile 2 are only a few percent higher than those for soil profile 1.

The results for the EPRI hazard levels would be about one third of those calculated using the NUREG/CR-5250 hazard results, due to the lower value of 11 in./sec/g for the peak ground velocity.

The computed displacements listed in Table 1 are those of the center of gravity of the soil mass above the potential failure surfaces shown in Figs. 3, 4, and 5. It is not possible to compute differential horizontal movements from the results of the analyses. The computed movements are toward the shoreline and are identified by the critical surfaces in Figs. 3, 4, or 5 and the name of the structures located within the soil mass above the critical surface. Thus estimates of permanent horizontal differential movements can be obtained by comparing the computed movements for each structure. For example, for a 0.7 g earthquake the Intake Structure would move 1.7 inches while the Reactor Containment Building would move only 0.2 inch, both movements toward the shoreline, i.e., roughly north. Thus the distance between the two structures would increase by about 1.5 inches.

4.5.2 Vertical Settlements

The vertical settlements within the soil profile were calculated by integrating the vertical strains calculated as described in Subsection 4.4.2 with respect to depth. The settlements are principally a function of the peak ground acceleration and the depth and density of the soil strata below the level at which the settlements are being evaluated. The settlements were calculated at El. +22, +12, +2, -3, and -28 feet to correspond to the approximate depths of the foundations of the critical structures. The results are presented versus the peak ground acceleration in Tables 2 and 3 for soil profiles 1 and 2, respectively.

The structures are founded at approximately the following elevations:

Structure	Elevation (feet)
Containment Tanks	+22
Diesel Generator Building	+22
Turbine Building	-5
Reactor Containment Building	-26
Intake Structure	-28

As discussed in Subsection 4.4.3, it is reasonable to assume that the building settlements are about equal to those of the soil profile at the depth of the foundation base.

The estimated building settlements are generally small, less than 1/3 inch up to a peak ground surface acceleration of about 0.35 g for soil profile 1 and less than 3/4 inch up to a peak ground surface acceleration of about 0.7 g for soil profile 2. This is consistent with the high blowcounts in the outwash materials, which plot well below the Seed et al (1975, 1983) curves in Fig. 7, which were discussed in Section 3. The compression of the fill above the water table is generally greater than for the material below the water table, due to its drainage during seismic shaking.

Differential settlements can be expected within the foundation imprint of any one building and within the areas between buildings due to natural variability of the compressibility of the soil deposits. Differential settlements can also be expected between any one building and the ground and between adjacent buildings, such as those within the Power Block, due to the different thicknesses of the soil strata beneath the various structures and beneath the ground surface.

In general, for a structure founded on individual spread footings, differential settlements equal to about 50% of the total settlement over distances of 25 feet can occur due to natural variability of the compressibility of the soil deposits below the structure. This is based on experience with settlements of foundations on cohesionless soil deposits. This can also be expected to be true for the settlements of the ground surface and at the various depths below the ground surface. For structures founded on a structurally continuous mat foundation, the differential settlement can be taken to be 50% of the total settlement distributed over a distance of about 50 feet.

The differential settlements between a building and the surrounding ground can be taken equal to the difference between the total settlement at the ground surface and the total settlement of the building. The differential settlements between the ground surface and the buildings will occur over a distance of a few feet from the building.

The differential settlements between adjacent structures due to the different elevations at which these structures are founded, such as for the Reactor Containment and Turbine Buildings in the Power Block, can be taken equal to the difference in their calculated total settlements given in Tables 2 and 3. The computed differential settlements are listed in Tables 2 and 3 for the maximum possible accelerations of 0.35 g and 0.7 g for profiles 1 and 2, respectively. Note that these differential settlements are in addition to the differential settlements resulting from natural variability of the soil deposits.

If the foundations of the adjacent structures within the Power Block consist of a structurally continuous mat, then this will smooth the settlements over a finite distance. The distance over which the differential settlement between adjacent structures is distributed will depend on the interaction of the foundation mat with the foundation soil with the building settlements imposed under the foundation imprint of each building. If there are construction or expansion joints between any of the adjacent building foundations, then the settlement profile is likely to be discontinuous across the joint.

5. TRANSIENT DISPLACEMENTS

The peak transient horizontal displacements of the ground due to an earthquake were calculated by integrating the maximum shear strains obtained from the SHAKE analyses with respect to depth. This is slightly conservative, since the peak shear strains do not all occur exactly simultaneously. The resulting transient horizontal ground displacements versus the peak ground acceleration are presented in Tables 4 and 5 for soil profiles 1 and 2. As discussed in Section 4, the effect of rocking of the structures is not taken into account by use of the results of the SHAKE analyses. It is likely that its effect is small for the structures at the Pilgrim site, with the possible exception of the Condenser Tanks.

The transient displacements of the embedded building foundations and the transient displacements of the ground at some distance away from the building would be different. For the purposes of computing differential displacements of piping and ducts entering the building, it is reasonable to approximate the displacement of the building by taking it equal to the displacement of the ground at the elevation of the base of the foundation.

Differential displacements can be expected between any one building and the surrounding ground, between buildings separated by some distance, and between adjacent buildings within the Power Block. The differential displacement can be conservatively taken equal to the absolute sum of the peak displacements of the building and the surrounding ground or the absolute sum of the peak displacements of the two buildings. This is based on the conservative assumption that the peak displacements will occur simultaneously with their directions 180 degrees out-of-phase.

The differential displacement between a building and the surrounding ground can be conservatively taken to be uniformly distributed over a distance of about 25 feet from the foundation. For example, this gives a differential displacement of about 1-1/4 inches over 25 feet for ducts entering the Reactor Containment Building near the ground surface for a peak ground acceleration of 0.7 g for soil profile 2. For the same conditions, this gives a differential displacement of about 1 inch over 25 feet for pipes entering the Reactor Containment Building at a depth of about 18 feet.

The differential displacement between two separated buildings can be reasonably taken to be uniformly distributed over the distance between the two buildings. For example, this gives differential displacements of about 1/4 inch between the Reactor Containment Building and the Intake Structure and about 1-1/4 inches between the Condenser Tank and the Reactor Containment Building over the distances between these structures.

The differential displacement between any two buildings within the Power Block depends not only on the soil profile, but also on the stiffness characteristics of the structures and on the nature of their structural connections, if any. The differential displacement cannot

be estimated based on the results of the SHAKE analyses, which represent the case of one-dimensional wave propagation through the soil profile in the free field. A realistic estimate of the differential displacement can be obtained by performing appropriate two- or three-dimensional soil-structure-interaction analyses that model the soil profile and the stiffness characteristics of the structures and the connections between the structures. The absolute sum of the peak displacements estimated using the results of the SHAKE analyses provides a conservative upper bound value for the differential displacement between adjacent structures within the Power Block. For example, this gives a differential displacement of about 0.78 inches between the Reactor Containment and Turbine Buildings for a peak ground acceleration of 0.7 g for soil profile 2.

The differential displacement between any two buildings within the Power Block may occur abruptly across construction joints or expansion joints between or within the building foundations or structures.

If it is determined based on the results of the fragility analysis that more refined estimates of the differential displacements are required, then two- or three-dimensional soil-structure-interaction analyses can be performed using existing computer codes to develop more realistic estimates.

The displacements of the structures can occur in any direction, and thus the differential displacements can be transverse or parallel to the distance between the structures. The effects of simultaneous occurrence of two horizontal components of differential displacement or of the resulting forces or stresses imposed on pipe and ducts can be taken into account using any of the methods available for combining loads due to seismic excitation in multiple directions.

The methods described above for estimating the differential displacements using the transient displacements given in Tables 4 and 5 account for motions of the structures relative to each other or of the structures relative to the adjacent ground. Buried conduits away from the effects of the structures are subject to strains due to the propagation of seismic motions across the site. Formulas to estimate these strains are presented in ASCE (1983, 1984).

6. SUMMARY AND CONCLUSIONS

The permanent and transient displacements and settlements due to an earthquake were calculated versus the intensity of the ground motion. The results were obtained using conservative methodology and previously existing information concerning the soils and structures at the site. The results were determined using the hazard curves and response spectra from NUREG/CR-5250 and EPRI (1989) and seismic time histories provided by Stevenson & Associates.

Based on the results of previous cross-hole testing by Weston Geophysical and the results of calculations using available empirical correlations and the Pilgrim soils test data, two shear wave velocity profiles were determined for the outwash deposits. Displacements and settlements are provided for both of the shear wave velocity profiles. It is reasonable to expect that the two shear wave velocity profiles bound the true shear wave velocity profile at the Pilgrim site.

The results of the SHAKE analyses conducted for the purpose of calculating the displacements indicate that the largest realistic peak ground accelerations for soil profiles 1 and 2 are less than 0.35 and 0.7 g, respectively. This conclusion is based on analyses using the acceleration time history provided by Stevenson & Associates having a peak ground acceleration of 0.5 g (NR0098-1) scaled to peak ground surface accelerations of 0.1 to 0.9 g in the SHAKE analyses conducted for this report.

For soil profile 1 and a peak ground acceleration of 0.35 g, the settlements, permanent horizontal displacements, and the transient horizontal displacements are less than 0.29 inch, 0.2 inch, and 1.05 inches, respectively. For soil profile 2 and a peak ground acceleration of 0.7 g, the settlements, permanent horizontal displacements, and transient horizontal displacements are less than 0.72 inch, 1.7 inches, and 1.12 inches, respectively. This is consistent with the characterization of the soils at the site as very dense.

A liquefaction stability failure is not possible at the Pilgrim site due to the dense state of the *in situ* soils and compacted fill.

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TABLE 1 - PERMANENT HORIZONTAL DISPLACEMENTS (INCHES) FOR SOIL PROFILE 2
Pilgrim 1 IPEEE, Plymouth, Massachusetts

Location	Yield Acceleration (g)	Peak Ground Surface Acceleration				
		0.2 g	0.3 g	0.4 g	0.5 g	0.7 g
Condenser Tank, Fig. 3	0.34	0	0	0	0.1	0.6
Reactor Containment Building, Fig. 3	0.48	0	0	0	0	0.2
Power Block, Fig. 3	0.50	0	0	0	0	0.1
Intake Structure, Fig. 4	0.24	0	0	0.2	0.4	1.7
Yard, Fig. 4	0.34	0	0	0	0.1	0.6
Yard, Fig. 4	0.40	0	0	0	0	0.4
Diesel Generator Building, Fig. 5	0.40	0	0	0	0	0.4

TABLE 2A - SETTLEMENTS (INCHES) FOR SOIL PROFILE 1
Pilgrim 1 IPEEE, Plymouth, Massachusetts

Depth (feet)	Elevation (feet)	Peak Ground Surface Acceleration			
		0.1 g	0.2 g	0.3 g	0.35 g
0	+22	0.02	0.07	0.17	0.29
10	+12	0.02	0.06	0.15	0.26
20	+2	0	0.02	0.09	0.18
25	-3	0	0.02	0.08	0.17
50	-28	0	0.01	0.06	0.12
80	-58	0	0	0	0

TABLE 2B - DIFFERENTIAL SETTLEMENTS (INCHES) FOR
PEAK GROUND SURFACE ACCELERATION OF 0.35 g
Pilgrim 1 IPEEE, Plymouth, Massachusetts

	Ground Surface	Condenser Tank	Diesel Generator Building	Turbine Building	Reactor Building	Intake Structure
Ground Surface, El. 22	0	0	0	0.12	0.17	0.17
Condenser Tank, El. 22	0	0	0	0.12	0.17	0.17
Diesel Generator Bldg., El. 22	0	0	0	0.12	0.17	0.17
Turbine Bldg., El. -5	0.12	0.12	0.12	0	0.05	0.05
Reactor Bldg., El. -26	0.17	0.17	0.17	0.05	0	0
Intake Structure, El. -28	0.17	0.17	0.17	0.05	0	0

TABLE 3A - SETTLEMENTS (INCHES) FOR SOIL PROFILE 2
Pilgrim 1 IPEEE, Plymouth, Massachusetts

Depth (feet)	Elevation (feet)	Peak Ground Surface Acceleration						
		0.1 g	0.2 g	0.3 g	0.4 g	0.5 g	0.6g	0.7 g
0	+22	0.02	0.05	0.10	0.17	0.27	0.45	0.72
10	+12	0.01	0.04	0.07	0.13	0.22	0.38	0.63
20	+2	0	0	0.01	0.03	0.07	0.14	0.27
25	-3	0	0	0.01	0.02	0.05	0.11	0.22
50	-28	0	0	0	0	0	0	0.01
80	-58	0	0	0	0	0	0	0

TABLE 3B - DIFFERENTIAL SETTLEMENTS (INCHES) FOR PEAK GROUND SURFACE ACCELERATION OF 0.7 g
Pilgrim 1 IPEEE, Plymouth, Massachusetts

	Ground Surface	Condenser Tank	Diesel Generator Building	Turbine Building	Reactor Building	Intake Structure
Ground Surface, El. 22	0	0	0	0.50	0.71	0.71
Condenser Tank, El. 22	0	0	0	0.50	0.71	0.71
Diesel Generator Bldg., El. 22	0	0	0	0.50	0.71	0.71
Turbine Bldg., El. -5	0.50	0.50	0.50	0	0.21	0.21
Reactor Bldg., El. -26	0.71	0.71	0.71	0.21	0	0
Intake Structure, El. -28	0.71	0.71	0.71	0.21	0	0

TABLE 4 - TRANSIENT HORIZONTAL DISPLACEMENTS (INCHES)
FOR SOIL PROFILE 1
Pilgrim 1 IPEEE, Plymouth, Massachusetts

Depth (feet)	Elevation (feet)	Peak Ground Surface Acceleration			
		0.1 g	0.2 g	0.3 g	0.35 g
0	+22	0.15	0.37	0.74	1.05
10	+12	0.14	0.36	0.72	1.03
18	+4	0.13	0.34	0.68	0.98
25	-3	0.12	0.32	0.64	0.93
50	-28	0.07	0.19	0.40	0.58
80	-58	0	0	0	0

Note: The building displacement can be assumed to be equal to the ground movements at the elevation of the foundation base, i.e., at the elevations given below.

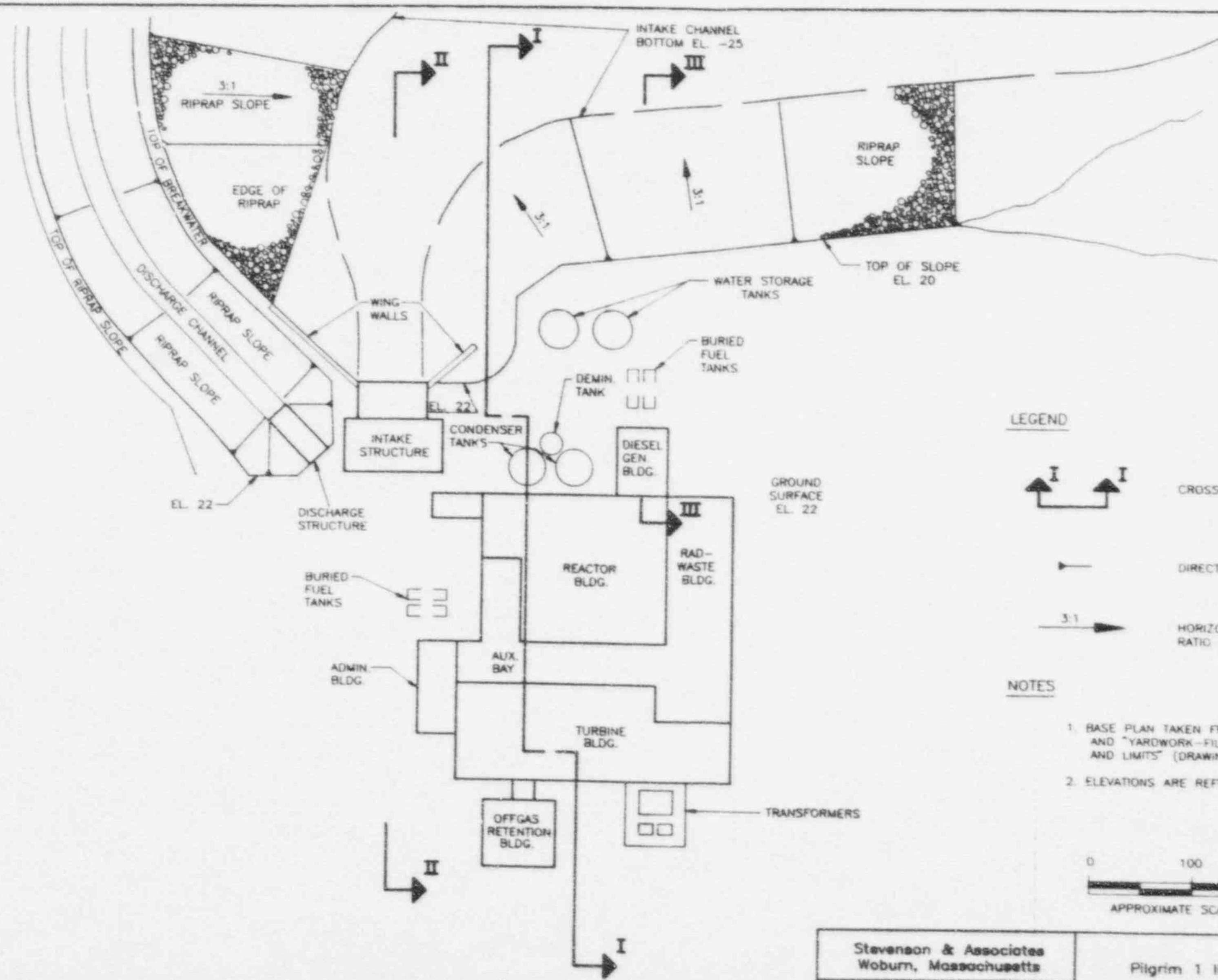
Building	Foundation Base Elevation
Ground Surface	+22
Condenser Tank	+22
Diesel Generator Bldg.	+22
Turbine Bldg.	-5
Reactor Bldg.	-26
Intake Structure	-28

TABLE 5 - TRANSIENT HORIZONTAL DISPLACEMENTS (INCHES)
FOR SOIL PROFILE 2
Pilgrim 1 IPEEE, Plymouth, Massachusetts

Depth (feet)	Elevation (feet)	Peak Ground Surface Acceleration						
		0.1 g	0.2 g	0.3 g	0.4 g	0.5 g	0.6 g	0.7 g
0	+22	0.06	0.14	0.24	0.38	0.56	0.80	1.12
10	+12	0.05	0.12	0.22	0.35	0.52	0.75	1.06
18	+4	0.04	0.10	0.18	0.30	0.43	0.62	0.87
25	-3	0.04	0.08	0.14	0.23	0.33	0.46	0.65
50	-28	0.01	0.03	0.04	0.06	0.07	0.09	0.13
80	-58	0	0	0	0	0	0	0

Note: The building displacement can be assumed to be equal to the ground movements at the elevation of the foundation base, i.e., at the elevations given below.

Building	Foundation Base Elevation
Ground Surface	+22
Condenser Tank	+22
Diesel Generator Bldg.	+22
Turbine Bldg.	-5
Reactor Bldg.	-26
Intake Structure	-28



LEGEND

I — I	CROSS SECTION LOCATION
	DIRECTION OF SLOPE
3:1	HORIZONTAL : VERTICAL RATIO OF SLOPE

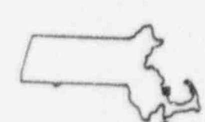
NOTES

1. BASE PLAN TAKEN FROM "PLOT PLAN" (DRAWING NO. C2) AND "YARDWORK-FILL COMPACTION REQUIREMENTS, AREAS AND LIMITS" (DRAWING NO. C-8), PREPARED BY BECHTEL.
2. ELEVATIONS ARE REFERENCED TO NGVD.

0 100 200

APPROXIMATE SCALE, FEET

Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE	SITE PLAN
GEI Consultants, Inc.	Project 92012	May 1992 Fig. 2



NOTES

Map is taken from U.S.G.S. Topographic 7.5 Minute Series Map of Manomet, Mass. Quadrangle, 1977.
Datum is National Geodetic Vertical Datum (NGVD).
Contour Interval is 10 Feet.

Stevenson & Associates Woburn, Massachusetts	Pilgrim 1 IPEEE Plymouth, Massachusetts	SITE LOCATION MAP	
GEI Consultants, Inc.	Project 92012	May 1992	Fig. 1