Instantaneous Deformation Analysis of a Gravity Structure

by

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Introduction

THE Atlantic Generating Station of Public Service Electric and Gas Company will be an offshore nuclear generating facility. It will be located about three miles off the New Jersey Coast between Long Beach Island and Atlantic City. The conceptual development has been explained by Kehnemuyi and Nicholas (1973). The plans for the proposed Atlantic Generating Station call for two floating nuclear power units enclosed in a protective basin. The protective basin will consist of U-shaped main breakwater constructed in an average depth of about 35 feet (19.7 m) of water. A closure breakwater consisting of a row of caissons with protective rubble mounds at the end will complete the basin. The cross sections of the main breakwater is the same as most of its lengths. The proposed height of the main breakwater is at Elevation + 66.0 feet (20.2 m). The seaward face (outward) of the breakwater slopes 2 horizontal to 1 vertical. A scour blanket is proposed to extend 60 feet (18.30 m) from outer toe of the breakwater. On the interior side of a caisson core covered with suitable berm is proposed.

Though the subsurface soils will not have a direct influence on the floating power plant, the stability and performance of the breakwater and mooring caissons will definitely depend on the foundation soils.

This paper will deal with only one part of the foundation investigation problems for a complex structure like the breakwater. The analysis described in this paper accounts only for the deformations due to instantaneous strains in the foundation and structural strains in the foundation and structural fill material caused by the application of structural loads.

Geology

The geology of the site area has been described in detail in Reference (1). The reference cited described several cycles of sea level fluctuations. Kenny (1964) has presented a comprehensive summary of eustatic sea level changes. Figure 1*a* shows the age data obtained by radiocarbon tests on samples obtained from the AGS site together with the data presented by Kenny, plotted as a function of depth below present sea level (present defined at 1950 A.D.). Most age data from the AGS site correlate closely to changes in sea level reported at other parts of the world. As a result of these

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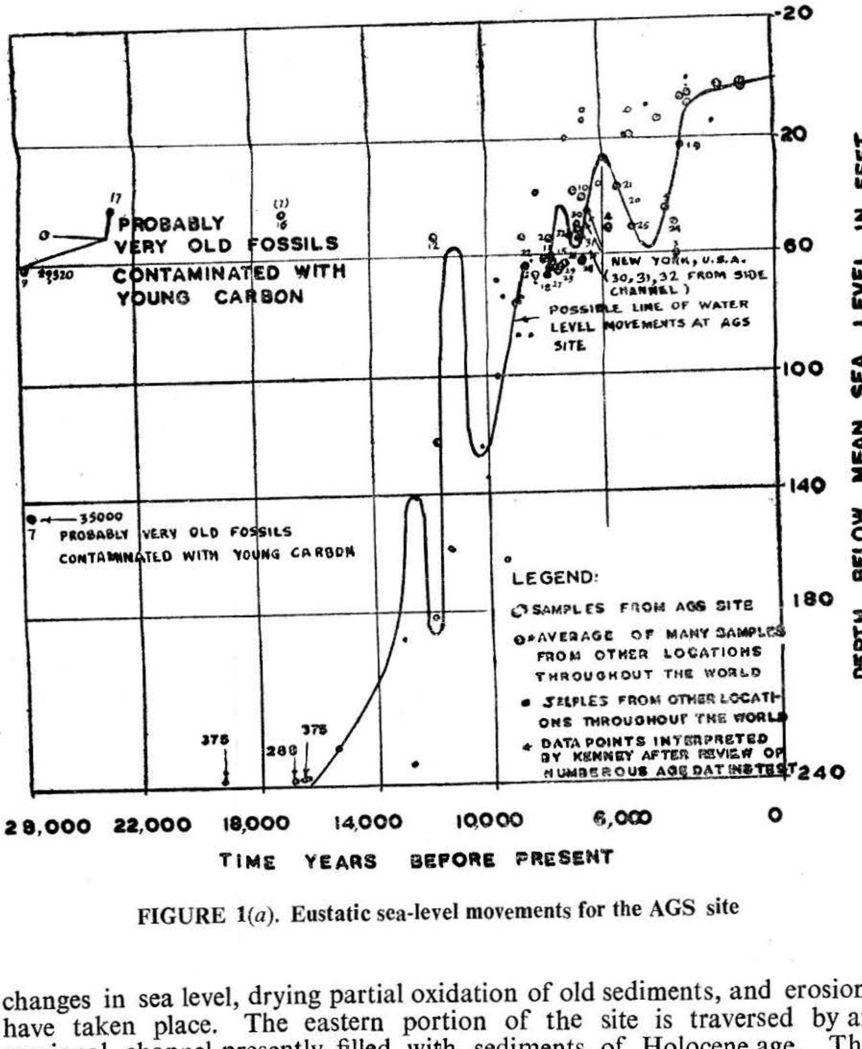

FIGURE 1(*a*). Eustatic sea-level movements for the AGS site

changes in sea level, drying partial oxidation of old sediments, and erosion, have taken place. The eastern portion of the site is traversed by an erosional channel presently filled with sediments of Holocene age. The western boundary of the channel is shown in Figure 1*b*. The eastern boundary falls way off to the east and hence not shown. The deepest point of that channel is approximately 40 feet (12.2 m) below the ocean bottom and is believed to have extended to, or slightly into, the top of the Tertiary sediments.

The upper soils of the site are a series of marine and continential deposits. The western portion beyond the west boundary of the channel is occupied by a thin layer of Holocene sediments. These sediments consist mainly of a medium-to-stiff silty-clay surface layer, underlain by silty sand. The Holocene sediments are underlain by Pleistocene age sediments consisting of a layer of stiff fissured silty clay. This stratum apparently had been subject of desiccation in the past. The fissured clays are underlain by interbedded gravels, sand and a stiff clay, all of Pleistocene age. Beneath these Pleistocene sediments is a thick deposit of sands and gravels of Tertiary age. Lenses of stiff silty clay have been encountered within the

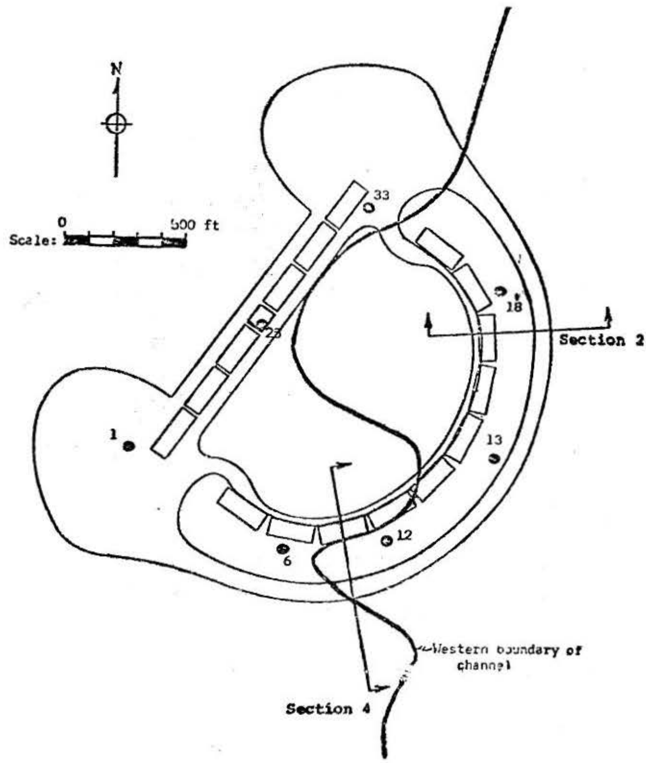

FIGURE 1(*b*). Location plan—the static cone soundings

top part of these sands and gravels. The water content of the clay lenses is very close to the plastic limit, suggesting that the soils are heavily preconsolidated.

Subsurface Investigations and Stratigraphy

The subsurface investigations were performed in several stages, from 1971 to 1974. Each stage provided some field data, which was evaluated to develop guidelines for the following stage of the subsurface investigation. Details of the field investigations and different field and laboratory tests performed will be reported separately when completed. In total one hundred and two vibracores, sixty four test borings, thirteen field vanes and seven cone penetration tests were completed. In addition 40 miles (64.36 km) of seismic reflection profiling was performed. In Figure 1*b* are shown only the location of static cone soundings.

The resulting of borings and vibracores were correlated with geophysical data to provide the required information on stratification. These studies also delineated the boundaries of the buried channel in the eastern portion of the site (shown in Fig. 1*b*), which is oriented roughly north-south. The stratigraphy at two sections of the breakwater is shown in Figures 2*a* and 2*b* (Fig. 1*b* shows the location of the sections).

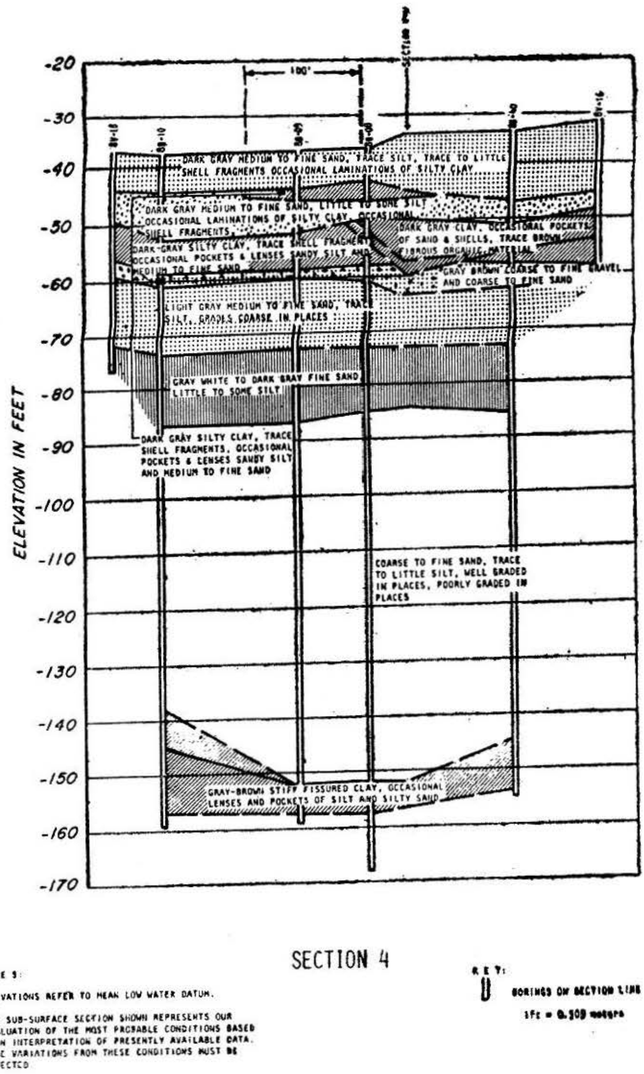

FIGURE 2*a*

The surface layer consists of fine to medium to dense sand which disappears towards west. Immediately below this layer lies two to five feet (0.6-1.5 m) of dark gray silty clay (hereinafter called CL layer). Where the top surface layer is nonexistent (in west), this layer of silty clay forms the surface layer. The silty clay layer is underlain by two to four feet of silty sand, which in turn lies over a layer of plastic clay, approximately 18 feet (5.5 m) thick at the deepest part of the channel. The plastic clay (hereinafter called CH) does not exist beyond the western boundary of channel. Below the plastic clay layer exists a deep deposit of dense sand which extends to a depth of 100 feet (30 m) or even more below the seabed level.

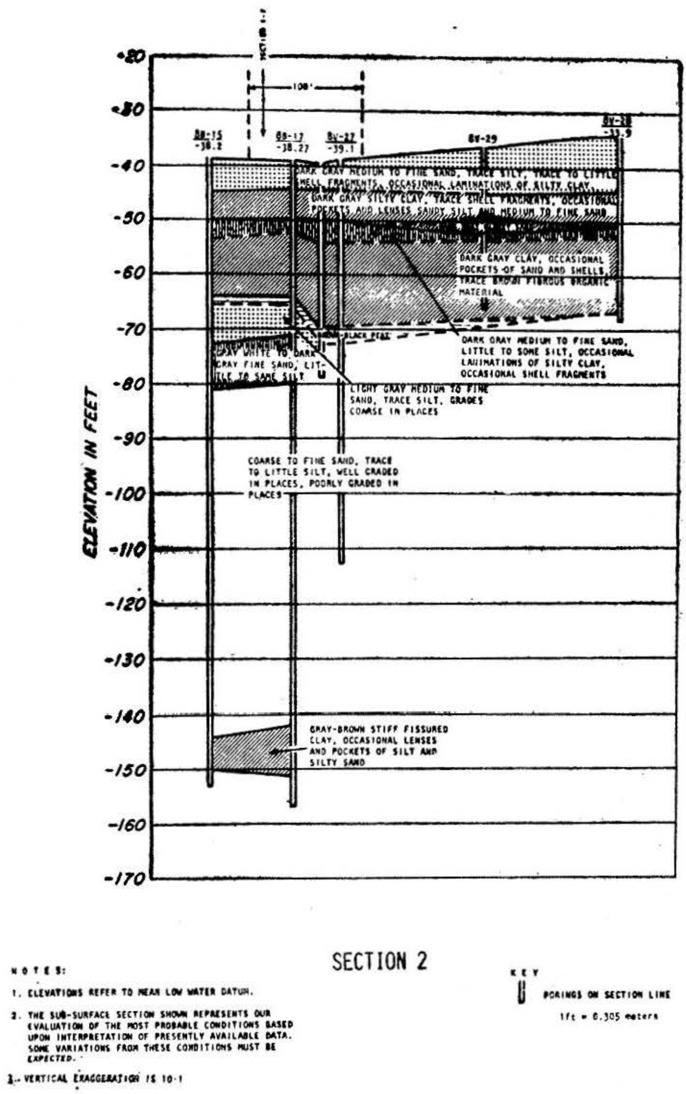

FIGURE 2*b*

Scope

It is obvious that innumerable combination of stages of construction can be analyzed. However, the analysis presented herein is the outcome of construction operations planning of the contractor selected for the breakwater construction. The breakwater shall be constructed in three stages (Figure 3). These stages represent the evaluations controlled by the equipment the contractor is planning to use and the construction times presently are his best estimates. The scope of this paper is only to present the methods to evaluate the magnitude of instantaneous settlement with the first stage loading. The section selected for the analysis was the one with deepest layer of plastic clays—Section 2 as shown in Figures 1*b* and 2*b*.

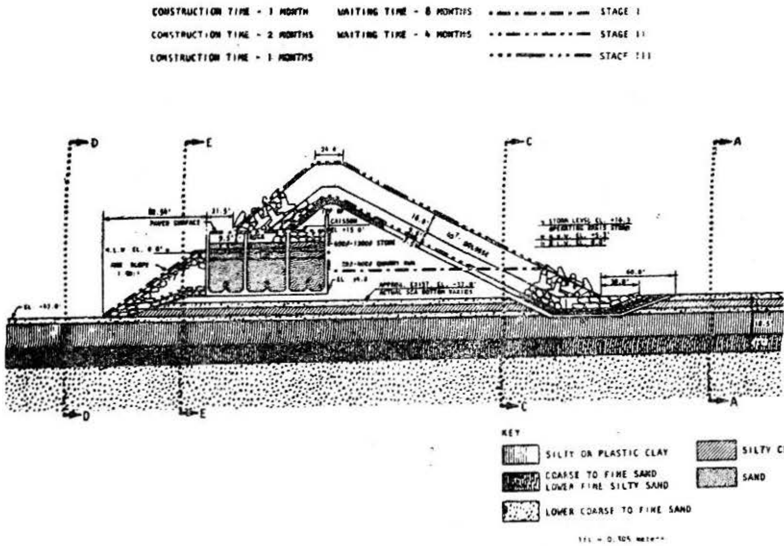

FIGURE 3. Breakwater section 2—updated subsurface information with stage construction

Local yielding and its extent is one of the major worries of an engineer, especially in the marine clays. Many failures are progressive in nature and the initiation of a failure is from the yielded zone. The main intent was therefore to find whether or not yielding will occur in the subsoil clays.

Soil properties

Only the soil properties concerned with the scope of analysis presented herein are discussed.

The approach aimed to finally evaluate the deformation-shear strength properties of the clays is called SHANSEP (Ladd—1972, 1974). However at the time this analysis was performed all the testing being not compete, extrapolations had to be used. The initial undrained soil strengths were evaluated by field vanes. Typical field vane strengths from probes near the Section 2 are shown in Figure 4.

Conventional and constant strain rate consolidation test (Wissa et al. 1971) were used to obtain maximum past pressures. Based on computed over consolidation ratios and average plasticity index, the values of K_o were estimated from data presented by Brooker and Ireland (1965). Later the same K_o-consolidation tests were also completed. Figure 6 shows K_o versus OCR plots. The estimated curve used in this analysis was very close to the experimental curve. Figure 5 is a plot of the overburden pressures (σ_{vo}), maximum past pressures (σ_{vm}) and Atterberg limits on the two different clays (CL and CH). These values are from test samples obtained at elevations shown in the figure from various boreholes within the channel.

Normalized Secant modulus from direct-simple shear tests are presented in Figure 7.

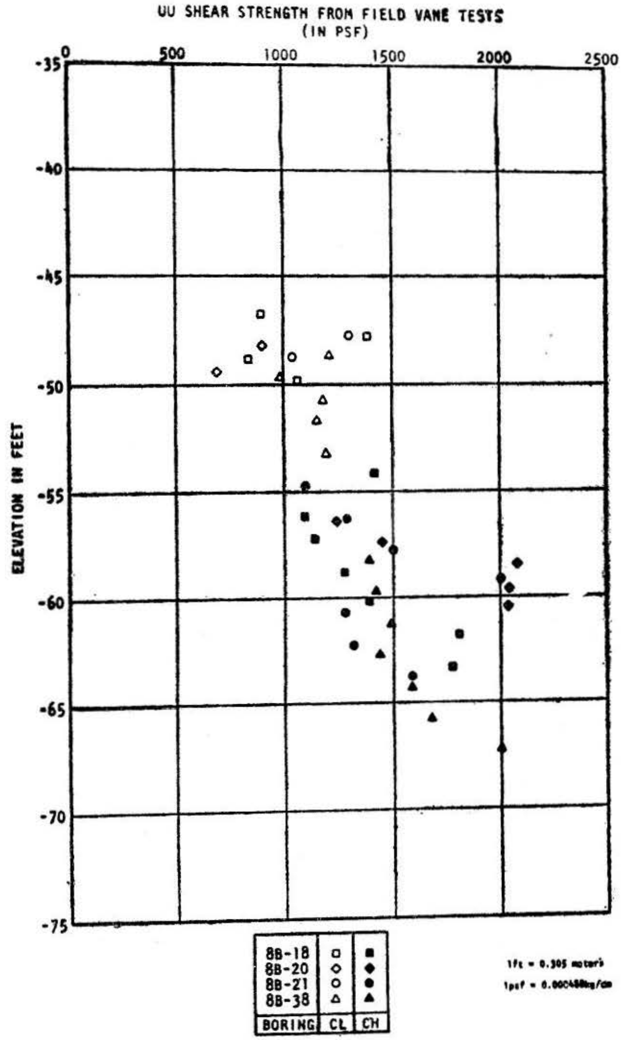

FIGURE 4. Field vane strength from probes near Section 2

Prediction of Immediate Deformations and Stresses

The immediate deformations may be due to undrained shear in the case of cohesive soils in which the drainage of the pore water requires a long time or drained shear in case of clean sand and gravels in pore pressures dissipate rather immediately. The finite element program used to predict the immediate deformations is called FEECON (Simon et al. 1972 and 1974). The program models the deformations and stresses in a grid of elements composed of up to ninety nine different soil types and subject to external loading. The finite element grid can contain up to five hundred elements and nodes, and elements can be activated and deactivated during the program operation to simulate the effect of both embankment construction and simple excavation. The program output provides the element stresses, modal displacements and element pore pressure increments, along with

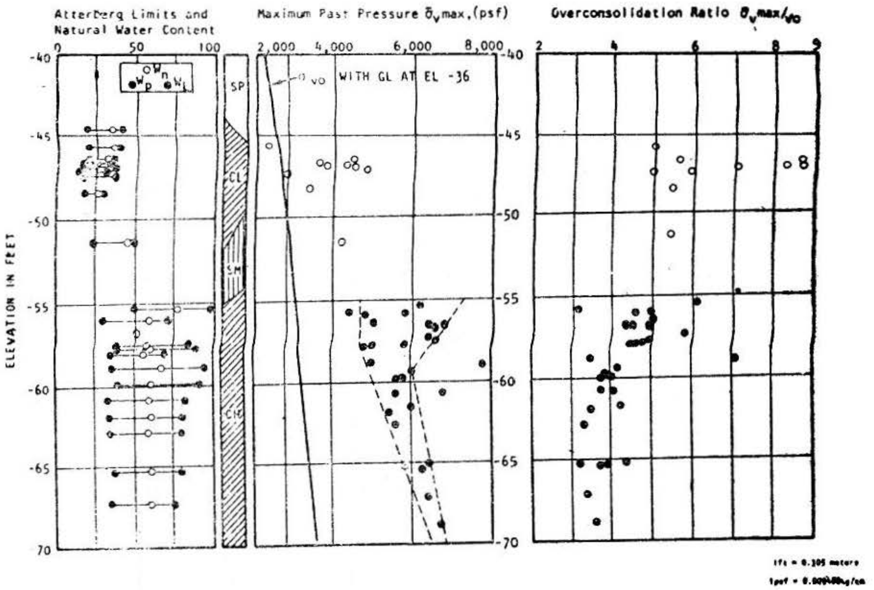

FIGURE 5. Index properties and stress history within the buried channel

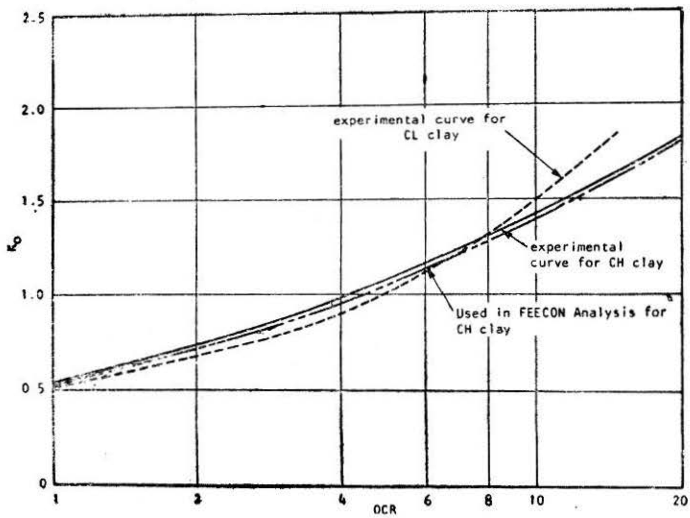

FIGURE 6. K_o *vs OCR*

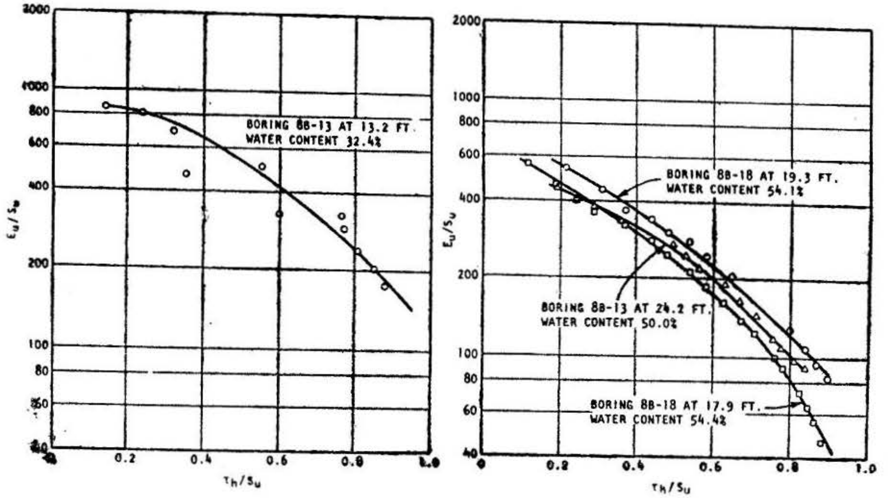

FIGURE 7. Normalised secant modulus $\overline{CK_oU}$ *DSS* Tests

information regarding the stress level in the elements. The evaluation of stress levels can also be used as indication of how close the elements are to yielding. The program treats the drained properties of sand and gravel in terms of effective stresses and undrained properties of clays in terms of total stresses. The program takes into account the initial stresses in the clay. Five models are available for describing the stress-strain behaviour of the soils.

Deformability models used

The main interest in the prediction was the undrained loading of the clay material. The choice of an appropriate model for the breakwater material and sands was less emphasized than for the clay deposits. An hyperbolic undrained shear-stress strain function which can account for anisotropic yield strength as well (Simon et al. 1972) was used for the clays. For the sands and silty sands the hyperbolic shear stress-strain function (Duncan & Chang 1970) was adopted. All the material of the breakwater was simulated by a lilnear stress-strain model with Tresca yield criterion. The program uses the incremental method in the nonlinear stress-strain models. Properties were defined for each layer of soil. Figure 8 lists the properties used for this analysis.

Input parameters

Clays : To obtain the input parameters for analysis, a combination of SHANSEP approach, measured strength data and extrapolation was used. The E_u values used in the hyperbolic model were based on measured data obtained from $\overline{CK_oU}DSS$ tests. The initial shear modulus is obtained from the relationship.

$$G_I = \frac{E_u}{2\,(1+\nu_I)} \simeq \frac{E_u}{3}$$

The yield modulus was taken as one per cent of the initial modulus. The bulk modulus was kept constant at all stress levels. It is related to Young's modulus (undrained) by the formula

$$K_I = K_y = \frac{E_u}{3\,(1-2\nu_I)}$$

For $\nu_I = 0.48$, $K_I = K_y = 25\, G_I$

The initial stress levels are accounted for in the program with the expression $q_o = 0.5\,(1-K_o)\,\sigma_{vo}$

Undrained shear strengths: The average field vane strength with Bjerrum's correction factor (1972) yields a very good average strength, as far as total stress stability is concerned. This has been ascertained by many investigators. Bjerrum's recommended field vane correction factor for a clay with a *P.I.* of 15 per cent (as the *CL* clay in here) is about 1.05 which applies to circular arc failures. Ladd and Edger (1972) concluded that $\overline{CK_oU}$ *DSS* tests yield good to slightly conservative values of S_u for a homogeneous clay for circular arc analyses. It was therefore assumed that measured field vane strength was equal to S_u for the *DSS* mode of failure which inturn is a good average strength.

For the *CH* clay, Bjerrum's recommended field vane correction (for a clay with a *P.I.* of 40 per cent) is 0.86. Based on certain significant lower values of correction factor in several case studies a value of correction factor equal to 0.75 was selected and the corrected field vane strength was assumed equal to S_u for the *DSS* mode of failure. Having established the average strengths, the values of $S_u(V)$ and $S_u(H)$ were calculated K_s, the ratios of the strengths in the horizontal and vertical directions. The K_s values used are extropolations from those measured with other clays of similar plasticity. Currently consolidated-undrained triaxial compression and extension tests, direct-simple shear tests are being performed on undisturbed samples of the clays. Few tests in plane strain condition will also be completed. The K_s values used will be ascertained once the tests are completed. Appendix III presents a sample calculation to obtain the values of $S_u(V)$ and $S_u(H)$ from the average $\bar{S}_u$ value. The method is based on Lacasse and Ladd (1973).

Pore pressure parameters

FEECON analyses requires the use of material constants a and k in the following equation

$$\triangle u = \triangle\sigma_{oct} + a\,\triangle\tau_{oct}^{k}$$

In the CL and CH clays, a and k were set equal to unity, thereby using the effect of the change in octahedral shear stress.

Sands and silty sands: Since the contribution of these materials as far as immediate settlements are concerned will not be significant, the properties to be used for the shear stress-strain function were based on extrapolation from Kulhaway, Duncan and Seed (1969). Tests are under progress on these materials and revised analyses shall be made with the test properties in the future.

Breakwater material: The breakwater is made up of different materials and the properties used in this analysis are best estimates. The rigidity of the breakwater caisson was incorporated by providing a thick base slab (material No. 11 Figure 8 and Table 1) whose rigidity shall be equal to that of caisson as a whole.

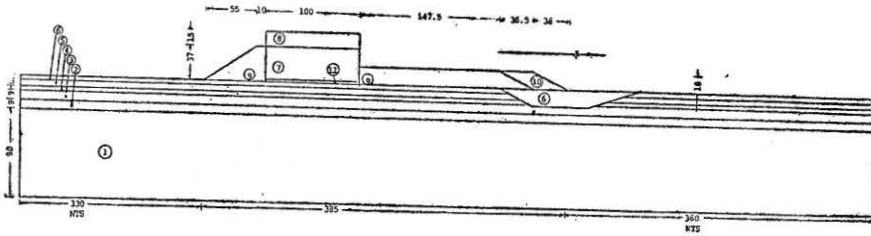

FIGURE 8. Breakwater materials (*NTS*—not to scale. All dimensions in feet)

Results of Analysis

The vertical and lateral undrained deformations due to stage one loading are presented in Figures 9 and 10. The vertical deformations are maximum under the caisson where the maximum loads exist but are of the order of one inch only. The horizontal deformations are maximum at inward side of breakwater and are of the order of two inches. It can be seen in Figure 9 that the imposed boundary conditions made an assumption that there shall be no movement at a depth of ninety feet below the CH clay layer. The results do indicate that the assumptions are realistic.

No zone of yielding was found either in CL or CH clays for this stage of loading. There shall be a waiting period of eight months before further load is applied, which shall allow the clays to gain in strength. Ideally the finite element programme should predict a zone of yielding that is compatible with a failure mode obtained in a stability analysis. Independent slope stability computations by Bishop (1955) and Morgenstern & Price (1965) methods also confirm high factors of safety (of the order of 1.7) for this stage of loading. No yielding should therefore be expected.

Figure 11 shows the central portion of FEECON mesh. Maximum shear stress elements are marked. Also drawn in the same figure is the surface of least resistance based on slope stability analysis (Morgenstern and Price 1965). This surface passes through the zones of highly stressed elements.

Conclusions

A preliminary check for overstressing of the marine deposits of clays under the first stage loading of a breakwater is presented. The selection of soil properties has been based on a combination of SHANSEP approach, measured strength data and extrapolation. The finite element program used for evaluation of the immediate deformation is FEECON.

The results of the analysis indicate that the clayey soils will not yield under the stresses imposed by the first stage of construction. The suitability of the site for the construction of the proposed nuclear power plant is therefore affirmed.

TABLE 1

(Soil Properties Used in Feecon Analysis)

Material type	Mat.	$S_u(V)$ (p.s.f.)	K_s	$\overline{a/b}^{*}$	G	F	d	ϕ (deg)	K	n	R_f	$\bar{\gamma}$ (p.s.f.)	K_o	K_i (p.s.f.)	K_y p.s.f.	G_i (p.s.f.)	G_y (p.s.f.)	q_o (p.s.f.)
Sand	1	0.54			0.54	0.23	4.3	39	1850.0	0.3	0.84	66	0.35					
Clay (CH)	2	1600.00	0.69	1.0							0.95	51	1.05	1,535,000		61,400	307	—39.13
Clay (CH)	3	1180.00	0.69	1.0							0.95	51	1.09	885,000		35,400	177	—49.73
Sand	4				0.54	0.23	4.3	34	540.0	0.54	0.85	61	0.4					
Clay (CL)	5	1250.00	0.60	1.0							0.95	51	1.4	1,800,000		72,000	360	92
Sand	6				0.54	0.23	4.3	34	540.0	0.54	0.85	61	0.4					
Caisson	7	10000.00	1.00	1.0								58	0.2	6,000,000	6,000,000	2,500,000	2,500,000	
Caisson	8	10000.00	1.00	1.0								122	0.2	6,000,000	6,000,000	2,500,000	2,500,000	
Rockfill	9	10000.00	1.00	1.0								66	0.3	2,000,000	2,000,000	870,000	870,000	
Dolosse	10	10000.00	1.00	1.0								40	0.3	3,000,000	3,000,000	1,150,000	1,150,000	
Base	11	576000.00	1.00	1.0								58	0.2	8.24×10^8	8.24×10^8	3.58×10^8	3.58×10^8	

(*) *as defined in Davis and Christian (1971)*

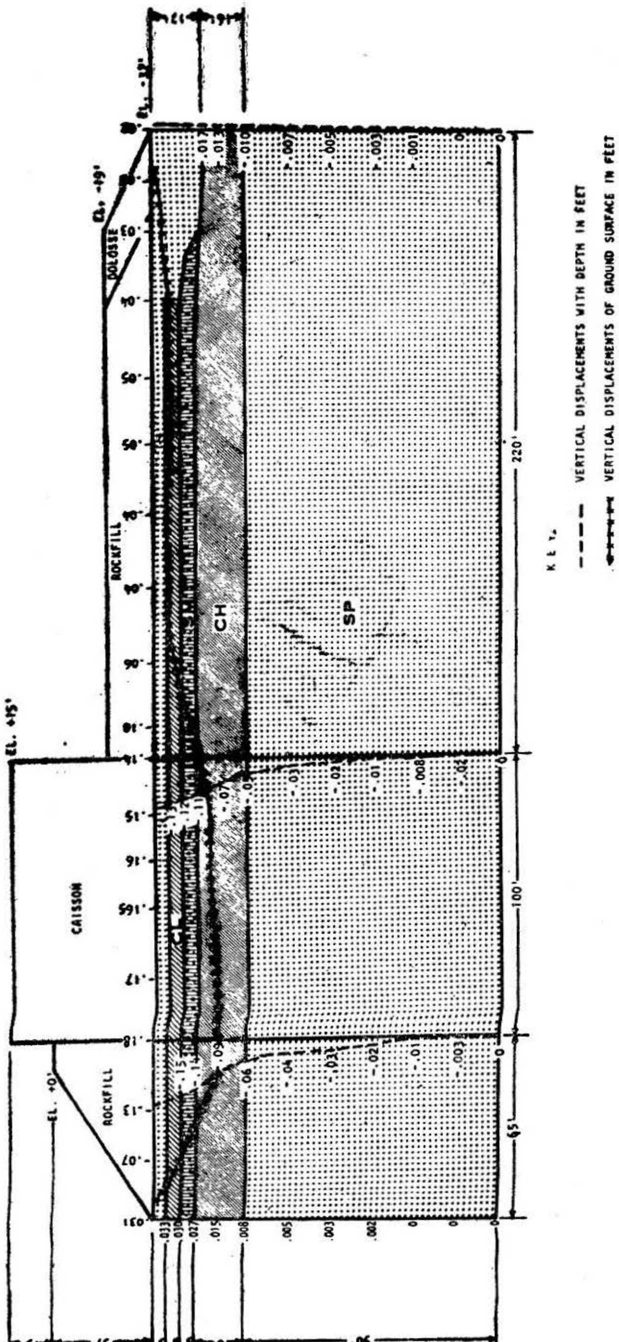

FIGURE 9. Vertical deformation

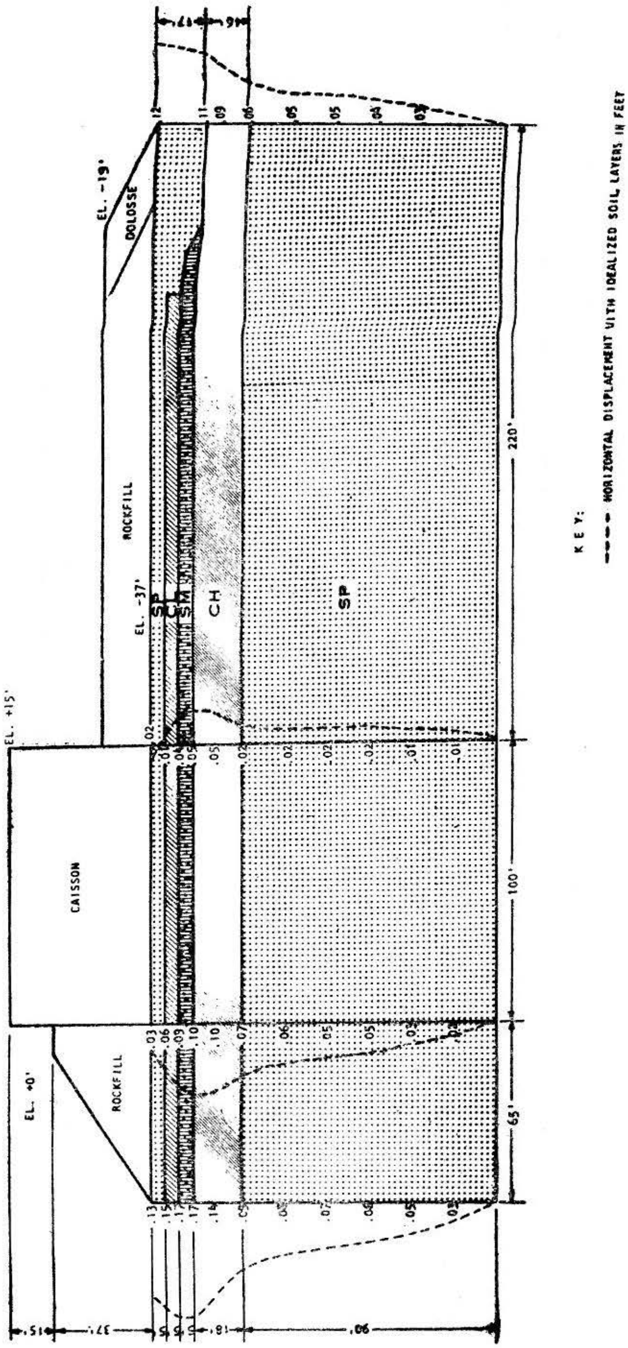

FIGURE 10. Horizontal deformations

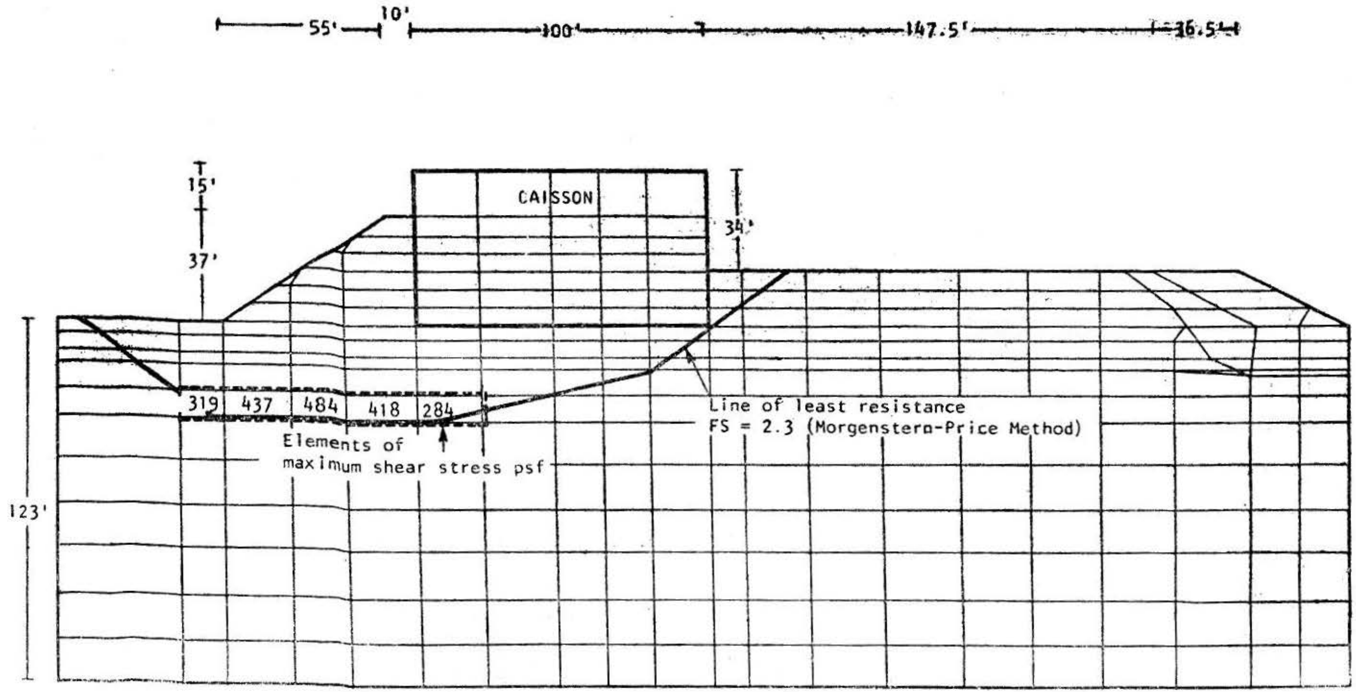

FIGURE 11. Central portion of FEECON mesh

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