NONLINEAR SEISMIC RESPONSE ANALYSIS OF A GRAVITY MONOPOD USING MODSAP-IV

D.V. Reddy, M. Arockiasamy and A.K. Haldar

SYNOPSIS

The development and application of a modified SAP-IV programme, MODSAP-IV, for nonlinear seismic response analysis of a large diameter offshore, prestressed concrete gravity platform-foundation system are presented. The soil-structure system is idealized by i) isoparametric plane strain, and ii) variable-number-nodes thick shell and three-dimensional isoparametric elements. The thickness of the soil elements in the plane strain model is varied parametrically until its fundamental frequency matches with that obtained from the three-dimensional formulation. The added water mass is assumed equal to the mass of water displaced, and lumped at the nodes of the tower and the caisson. The nonlinear behaviour due to shear deformation of the soil is considered by a method of equivalent linearization following the procedure of Seed and Idriss, and the equations of motion solved using the step-by-step direct integration to obtain an approximate solution. The stiffnesses and damping values are made compatible with effective shear strain amplitudes at the soil element centroids. Final values of the soil element stiffness and damping properties are determined by an iterative plane-strain analysis based on modification of SAP-IV which provides for variable soil damping in contradistinction to constant damping. Using beam elements in the superstructure, the responses are calculated to determine the effect of soil-structure interaction.

RESUME

Une modification du Programme SAP-IV permet l'analyse du comportement sismique de plates-formes de grand diamètre en mer construites en béton précontraint. Le sol est caractérisé à deux dimensions, tandis que des éléments à trois dimensions représentent la coque de béton, et des éléments de poutres modèlent la partie supérieure de la plate-forme. Le comportement non-linéaire du sol est représenté par des ressorts et amortisseurs équivalents selon le niveau des déformations au centre de l'élément en question. L'effet de l'interaction sol-structure est inclus et les masses des éléments et de l'eau déplacée sont prises aux noeuds du modèle.

Dr. D.V. Reddy is Professor of Engineering at Memorial University of Newfoundland, St. John's. His previous appointments include Professorship (and Headship) of Aeronautics and Applied Mechanics, Indian Institute of Technology, Madras, and Associate Professorship of Mechanical Engineering, The University of Calgary. He has held visiting professorial appointments at the University of Hannover, West Germany, South Dakota School of Mines and Technology, Rapid City, and the University of Illinois, Urbana.
Dr. M. Arockiasamy is a Postdoctoral Fellow at Memorial University of Newfoundland working in the area of Offshore Structural Dynamics. He is on leave of absence from his position of Professor of Structural Engineering, at the University of Madras, India.
Mr. A.K. Haldar, M.Eng., Memorial University of Newfoundland, 1977 is continuing his graduate work for a Ph.D. He has worked in industry as a Structural Engineer in Shawmont Newfoundland Ltd., St. John's, and as an Assistant Engineer in the Calcutta Metropolitan Development Authority, W. Bengal.
The authors have co-authored a number of publications on the dynamics of offshore structures and nuclear reactors.

INTRODUCTION

Offshore structures are designed and constructed in regions of high seismicity, and in areas where rough sea conditions often exist. Since these conditions develop high dynamic forces in the structure-foundation system, it is important that they must be predicted realistically for design purposes. The response of relatively flexible structures to earthquake motion has been studied extensively since the early 1930's when the first accelerograms were obtained. These structures were modelled as fixed base structures, and the adequacy of the base fixity assumption was first studied by Housner (1). Penzien and Tseng (2) presented the seismic analysis of gravity platforms using a structure-foundation system formed by coupling the dynamic modal properties of the linear fixed-base structure with the frequency-dependent foundation impedances. Penzien (3) presented dynamic analysis procedures for determining the seismic and wave response of fixed offshore structures, including soil-structure interaction effects. Both deterministic and stochastic analyses were carried out by Penzien (4) for predicting the seismic response of platform structure-foundation systems. The tower structure and the pile foundation were modelled by a combination of linear and nonlinear elements. Hydro-dynamic drag and inertia forces were included, and the soil-pile-tower interaction was treated using the theory of the elastic half-space. Rainer (5) presented a method for determining the structure-ground interaction effects of single-storey structures under earthquake loads. Whitman, Protonotarios, and Nelson (6) showed that inclusion of rocking in the dynamic analysis improved the agreement between predicted and observed frequencies. The effect of foundation sway was analysed considering the associated damping.

Kausel and Roesset (7) presented methods for estimating dynamic soil-structure interaction effects. The mathematical modelling of the soil and structures was based on finite elements and linear members, or finite difference schemes. Inertia forces were based on lumped or distributed masses. Kausel, Roesset and Christian (8) studied the effects of the presence of the structure on the nonlinear soil behaviour using an improved algorithm for the iterative linear approximation. The material properties were assumed to be linearly viscoelastic, and the dynamic equations were solved in the frequency domain by the complex response method. Hasselman, Bronowicki, and Chrostowski (9) presented a probabilistic method for evaluating the seismic response of offshore platforms considering soil-structure interaction. Idriss, Dobry and Power (10) considered the characteristics of strong ground motion and soil liquefaction in evaluating the soil response related to the seismic design of offshore platforms. Watt, Boaz, and Dowrick (11) computed the earthquake response of gravity structures in 200 m water depth using response spectrum and Fast Fourier transform solution techniques. Parametric studies of foundation compliance, damping behaviour, and soil stiffness were also carried out. Vaish and Chopra (12) presented an effective analysis procedure, based on the substructure approach, for the general linearly elastic structure-foundation system, idealised as an assemblage of finite elements. The foundation soil was first analysed independently of the structure to obtain its dynamic compliance characteristics, which were then incorporated into the structural equations of motion. The response of the structure was evaluated in the Ritz coordinate system with a high degree of refinement, with far greater computational efficiency. Prevost and Hughes (13) proposed an analytical model for the analysis of gravity offshore structure foundations subjected to cyclic wave loading. The model described the anisotropic, elasto-plastic, path-dependent, non-linear stress-strain-strength properties of inviscid, saturated soils subjected to cyclic loading paths. Liaw and Chopra (14) presented a method for the analysis of the response of cantilever axisymmetric tower structures, partly submerged in water, to earthquake ground motion. The tower and the surrounding body of water were treated as substructures, and the displacements of the tower were expressed by superposition of the first few 'dry' modes of vibration of the tower. The substructure approach permitted treating the surrounding water as a continuum, utilizing explicit mathematical solutions of the Laplace equation for the cylindrical tower-water interface.

Swamidas, Reddy and Purcell (15) presented the dynamic ice-structure interaction study of monopod platforms by hybridizing the relevant subroutines of the computer programmes, SAP IV (Structural Analysis Programme) and EATSW (Earthquake Response of Axisymmetric Tower Structures Surrounded by Water). The work included a study of the influence of soil properties on the frequencies and responses, and a comparison between the responses of fixed and elastically supported (by the soil foundation) monopod platforms. Haldar, Swamidas, Reddy, and Arockiasamy (16) studied the dynamic response to ice forces of offshore platforms considering soil nonlinearity. The method used was an equivalent linearization technique proposed by Seed and Idriss (17). Two structures (a pile-supported framed tower and a gravity type monopod) were analysed, and the response of the structures computed in three different ways: i) Time-history analysis, ii) Response spectrum method, and iii) Power spectral method. Later, the method was extended to the analysis of a prestressed concrete gravity platform subjected to a deterministic time-history wave loading by Arockiasamy, Haldar, Reddy, and Yen (18). Arockiasamy, Reddy, Haldar, and Yen (19) presented the dynamic response analysis of the gravity platform-foundation system considering the soil nonlinearity in terms of an elasto-plastic model obeying the Drucker-Prager yield condition.

The paper describes the dynamic analysis procedure for predicting the response of a large diameter, offshore prestressed concrete gravity platform-foundation system subjected to seismic forces. The nonlinear behaviour due to shear deformation of the soil is handled by a method of equivalent linearization, following the procedure of Seed and Idriss (20), with a special purpose programme developed by modification of SAP-IV to provide plane strain variable damping elements in the soil, in contradistinction to constant damping. Using beam elements in the superstructure, the responses are computed to determine the effect of soil-structure interaction.

STATEMENT OF THE PROBLEM

PROCEDURE

The gravity-type structure shown in Fig. 1, developed by the Sea Tank Co., is chosen for the example problem. The platform consists essentially of a large concrete cellular caisson into which the tower supporting the deck structure, is built (21). The caisson layout is a cellular platform formed by orthogonally interconnected walls. The concrete caisson serves as a platform float during towing and immersion and becomes the foundation on the sea bed. The deck supports considerable loads of the order of 10,000 tons including the main drilling and production equipment. The vertical part of the tower is modelled as a beam of hollow circular cross section with varying diameter. The concrete caisson, idealised as an assemblage of equidistant ribs, is modelled as a thick slab for determination of the bending, shear and torsional rigidities.

The soil-structure system is idealized by i) isoparametric plane strain, and ii) variable-number-nodes thick shell and three-dimensional isoparametric elements. The thickness of the soil elements in the plane strain model (Fig. 2b) is varied parametrically until its fundamental frequency matches with that obtained from the three-dimensional formulation (Fig. 2a). The added water mass is assumed to be equal to the mass of water displaced, and lumped at the nodes of the tower and the caisson. The Taft accelerogram (Fig. 3) is used for the excitation input at the bedrock level. The frequencies, mode shapes and responses are obtained by the specially developed programme, MODSAP-IV, for the structure-foundation system assuming an initial set of shear moduli for the soil elements. The stiffnesses and damping are made compatible with the effective shear strain amplitudes at all the soil element centroids. Published data on strain-compatible soil properties for clays and sands by Seed and Idriss (17) are used, and the equations of motion solved by step-by-step direct integration of the equations of motion.

ANALYSIS

VARIABLE DAMPING SOLUTION

The equations of motion for the structure, discretized as a finite element system, are

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{R(t)\} \tag{1}$$

where

$[M]$, $[C]$, $[K]$ = mass (lumped structural and added water masses), damping, and stiffness matrices respectively,

$\{u\}$ = nodal displacement vector,

and

$\{R(t)\}$ = earthquake load vector.

In the variable damping solution, the damping matrix is obtained by appropriate addition of the damping matrices of the soil and structural elements. The damping matrix of the structural finite elements is assumed proportional to the mass and stiffness matrices. In the case of soil finite elements, variable damping is used in which a damping submatrix is formulated for each individual element; all the element submatrices then added in an appropriate manner, and the damping matrix for the assemblage of the soil elements obtained (22).

The following relationship is used in the formulation of the submatrix for each element, q, in the soil:

$$[c]_q = \alpha_q [m]_q + \beta_q [k]_q \tag{2}$$

where

$[c]_q$, $[m]_q$ and $[k]_q$ = the damping, mass and stiffness submatrices respectively for element q,

and

α_q and β_q = parameters that are functions of the damping and stiffness characteristics of element q.

The parameters α_q and β_q are given by

$$\alpha_q = \lambda_q \, \omega_1$$

and

$$\beta_q = \frac{\lambda_q}{\omega_1} \tag{3}$$

The value of λ_q representing the damping ratio for element q is based on the strain developed in the element, and ω_1 is the undamped fundamental frequency of the system. The damping matrix for the entire assemblage of elements is obtained by appropriate addition of the damping submatrices of all the elements in the soil and the structure. The IJth term of the damping matrix of the entire system is given by

$$C_{IJ} = \sum_q c_{ij}^{(q)} \tag{4}$$

where

$c_{ij}^{(q)}$ = the ijth term of the damping submatrix $[c]_q$ of a typical element q.

The resulting damping matrix, [C], is symmetric and sparsely populated. The equations of motion, Eqs. 1, are solved using the step-by-step method (23) assuming linear variation of acceleration over the time increment of integration, Δt. The unknown response values at the nodal points at time, t, are expressed in terms of the known values at time, t - Δt, as

$$\{u\}_t = [\bar{K}]^{-1}\{\bar{R}\}_t \tag{5}$$

where

$$[\bar{K}] = [K] + \frac{6[M]}{\Delta t^2} + \frac{3[C]}{\Delta t} \quad ,$$

$$\{\bar{R}\}_t = \{R\}_t + \{A\}_t^T[M] + \{B\}_t^T[C] \quad ,$$

$$\{A\}_t = \frac{6}{\Delta t^2}\{u\}_{t-\Delta t} + \frac{6}{\Delta t}\{\dot{u}\}_{t-\Delta t} + 2\{\ddot{u}\}_{t-\Delta t} \quad ,$$

$$\{B\}_t = \frac{3}{\Delta t}\{u\}_{t-\Delta t} + 2\{\dot{u}\}_{t-\Delta t} + \frac{\Delta t}{2}\{\ddot{u}\}_{t-\Delta t} \quad ,$$

$$\{\dot{u}\}_t = \frac{3}{\Delta t}\{u\}_t - \{B\}_t \quad ,$$

and

$$\{\ddot{u}\}_t = \frac{6}{\Delta t^2}\{u\}_t - \{A\} \quad .$$

The stresses and strains developed in each element are then computed using the values of $\{u\}_t$, and the final values of the soil element stiffness and damping estimated by an iterative procedure. The programme, MODSAP-IV, specially developed by the authors, provides for variable damping in the soil for soil-structure interaction analysis in the time domain. At the time of initiation of this project, the available computer code LUSH (24), could only cope with plane strain finite elements for both the soil and the structures; MODSAP-IV provides for modelling the superstructure with beam or plane strain elements.

SOIL BEHAVIOUR

STRESS STRAIN RELATIONSHIP

The shear deformation occurring in soils due to seismic forces introduces nonlinear effects. The nonlinear soil behaviour is treated by an equivalent linear method presented by Seed and Idriss (20). The approximate relationship between the shear modulus and undrained shear strength for clays, established by Seed and Idriss (17) for a wide range of strain amplitudes (Fig. 4), and the variation of shear strain with shear modulus (Fig. 5) are used in the analysis. With an initial set of computed shear moduli for each soil element, the stress history is computed at each soil element centroid. The effective shear-strain amplitudes at each element centroid are estimated and checked for strain compatibility with those reported in Ref. 24. The properties of the soil elements, which do not exhibit compatible values, are modified and the procedure repeated until the shear moduli are compatible with the strain amplitudes. The response from the final iteration is assumed to be the approximate nonlinear response.

RESULTS AND DISCUSSION

The variation of undrained soil shear strength below the seabed is shown in Fig. 6. The natural frequencies and the maximum responses for the two cases, i) linear soil behaviour, and ii) nonlinear soil behaviour, are presented in Table I. The fundamental frequency reduction is not large (less than 15%) for the nonlinear case. The time histories of displacement, axial force and bending moment, and shear stress are presented in Figs. 7, 8, and 9. The axial force and moment induced at the bottom portion of the tower vary depending on the method of analysis used. Reduction in the shear stress, computed at the centroid of a typical soil element 22, is as high as 22%. The natural frequencies lie within the general range of values, reported by Shaw, Coates, Hobbs, and Schumm (25), in which the analytical and field data studies of the dynamic behaviour of gravity structures and foundations are correlated. The soil shear modulus and damping values (Table II) converge to reasonable limits within the first three iterations. The example illustrated has 118 D-O-F; computation time (CPU) for the eigenvalue and response analysis was 36.6315 min. on an IBM 370/158 computer.

The variation of pore pressures in the soil due to the cyclic loading phenomenon, variable dilatancy (volume changes caused by shear stresses), and anisotropy need to be considered in the determination of the sensitivity of the structural response. Also, the effects of energy dissipation into the soil mass have to be studied considering the standard viscous boundary as developed by Lysmer and Kuhlemeyer (26).

ACKNOWLEDGEMENT

The authors would like to thank Dr. R.T. Dempster, Dean, and Dr. A.A. Bruneau, Vice-President of Professional Schools and Community Services, Memorial University of Newfoundland, St. John's, Newfoundland, for their continued interest and encouragement. The support of this investigation by National Research Council of Canada, Grant No. 8119 is gratefully acknowledged.

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TABLE I FREQUENCIES AND MAXIMUM RESPONSES

Finite Element Model	Frequencies (Hz) Mode 1	Mode 2	Mode 3	Maximum Response: Displacement -top of tower (m)	Axial force -near base (kN)	Moment -near base (kN-m)	Shear Stress -at centroid (kN/m^2)
Linear Soil Behaviour	0.3866	0.7214	1.0550	0.0561	171.4259	895,244.0	11.1225
Nonlinear Soil Behaviour	0.3199	0.7196	1.0380	0.0570	183.7914	682,560.0	8.6567

TABLE II CONVERGENCE OF SOIL SHEAR MODULUS AND DAMPING

Iteration	Element No.	Shear Modulus (kN/m^2)	Damping Value
1	22	5038.41	0.1059
	32	25297.59	0.0331
	42	28294.83	0.0852
2	22	4401.89	0.1156
	32	35763.79	0.0456
	42	30704.59	0.0313
3	22	4550.80	0.1133
	32	47689.70	0.0384
	42	–	–

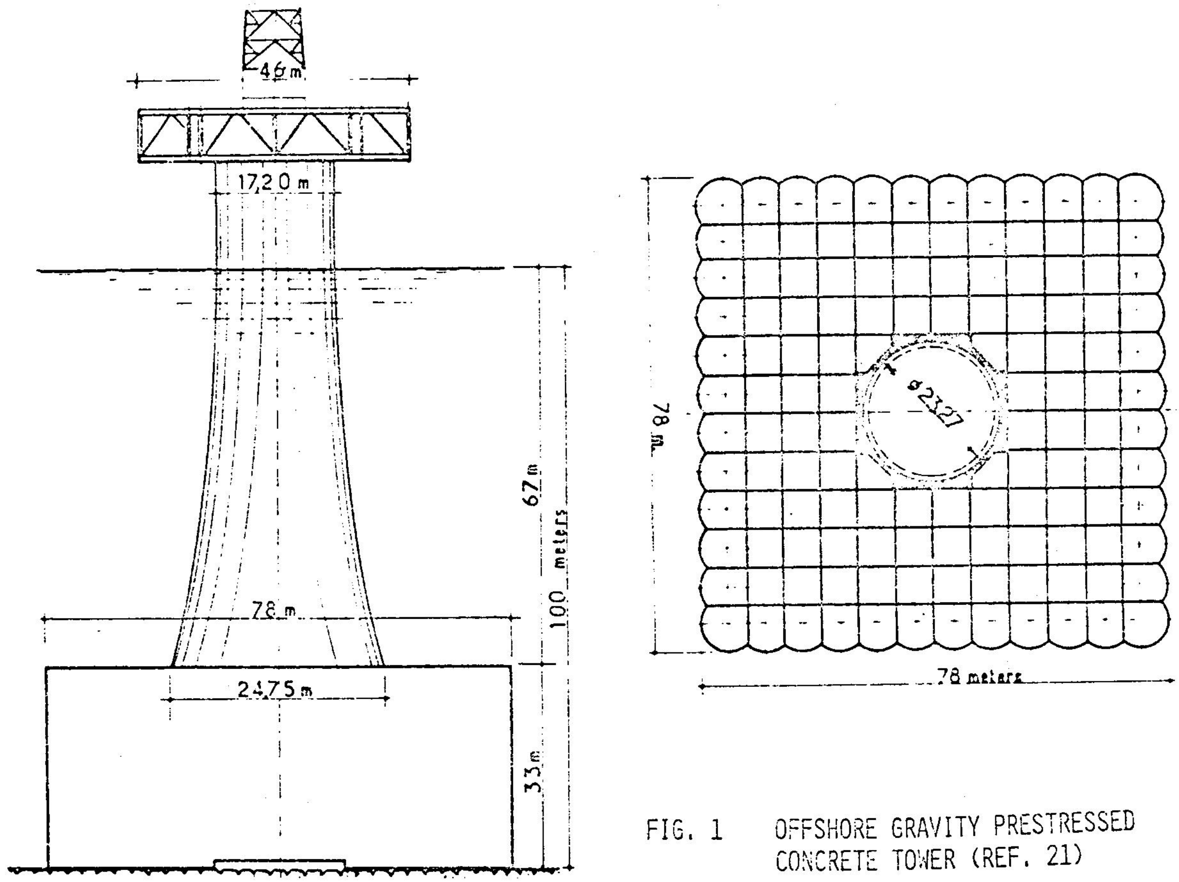

FIG. 1 OFFSHORE GRAVITY PRESTRESSED CONCRETE TOWER (REF. 21)

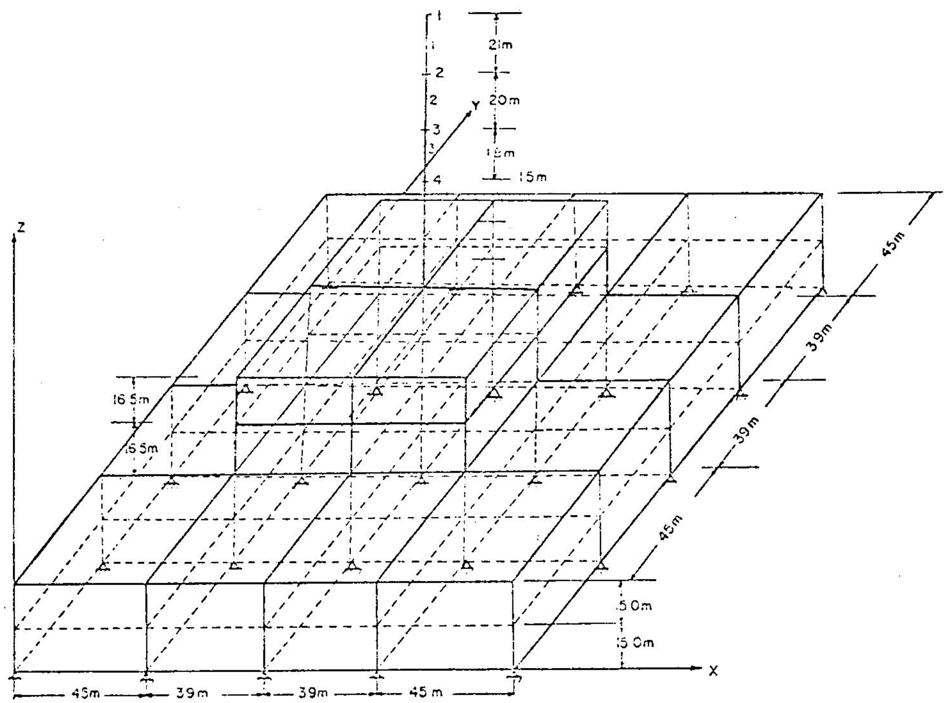

FIG. 2A THREE-DIMENSIONAL FINITE ELEMENT IDEALISATION OF SOIL-STRUCTURE SYSTEM

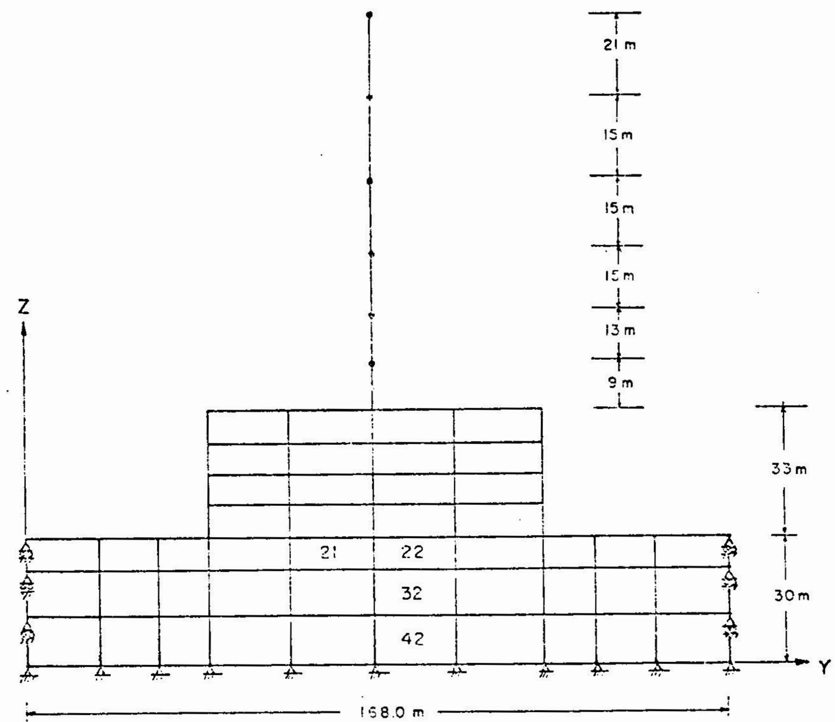

FIG. 2B TWO-DIMENSIONAL FINITE ELEMENT IDEALISATION OF SOIL-STRUCTURE SYSTEM

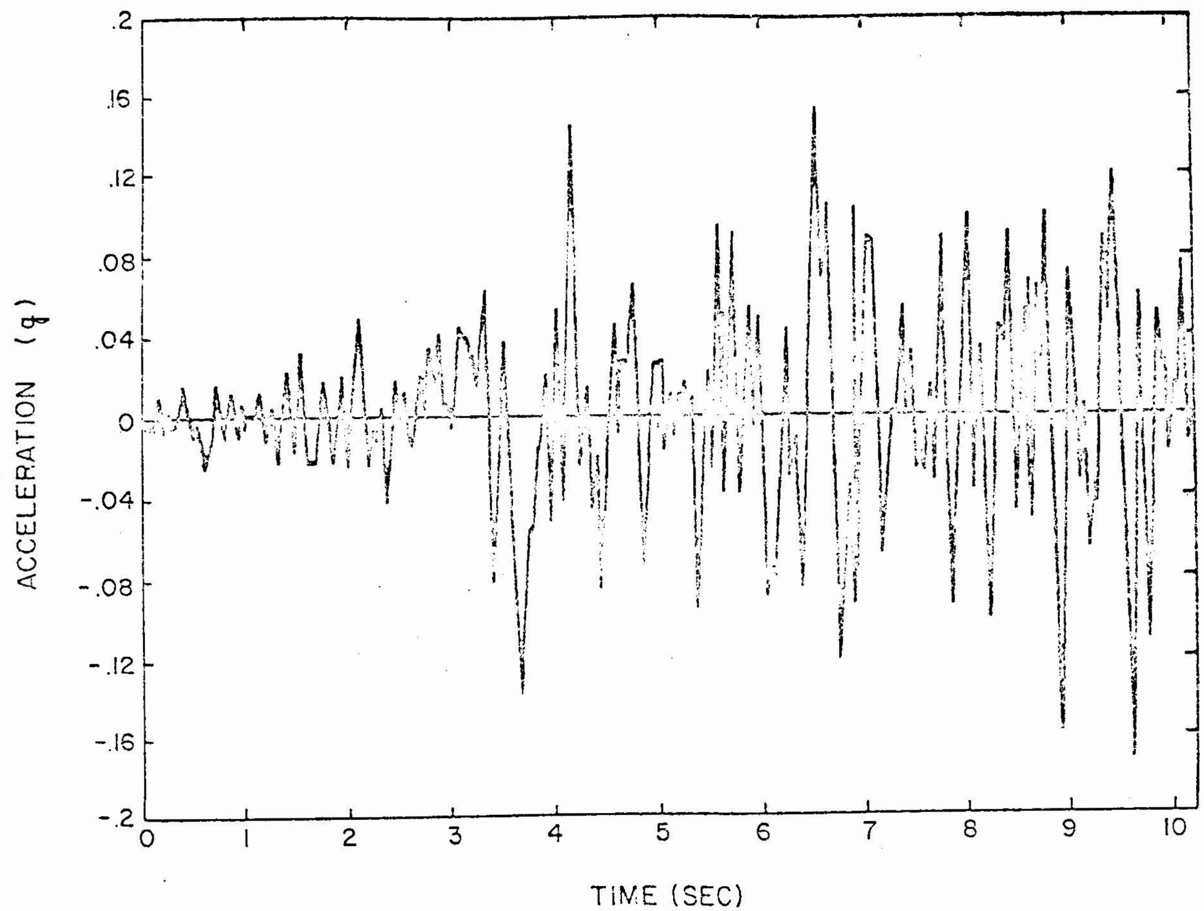

FIG. 3 INPUT MOTION FOR TAFT EARTHQUAKE

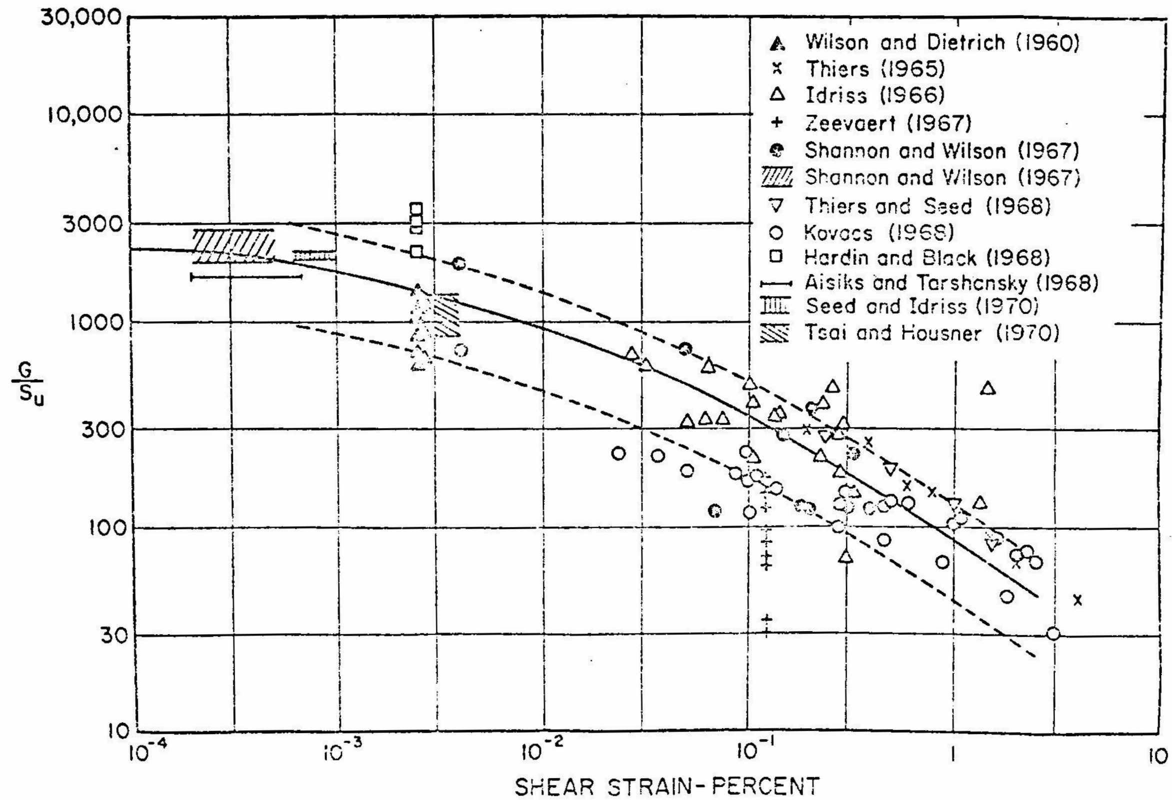

FIG. 4 IN-SITU SHEAR MODULI FOR SATURATED CLAYS (REF. 17)

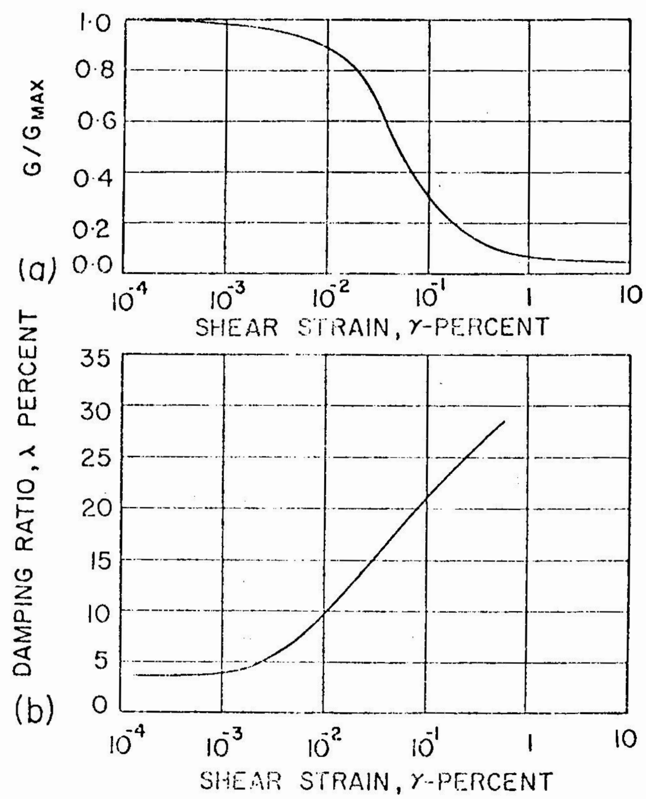

FIG. 5A EXAMPLE RELATIONSHIP-REDUCTION IN MODULUS VS. SHEAR STRAIN (CLAYS)

FIG. 5B EXAMPLE RELATIONSHIP-DAMPING RATIO VS. SHEAR STRAIN (CLAYS) (REF. 17)

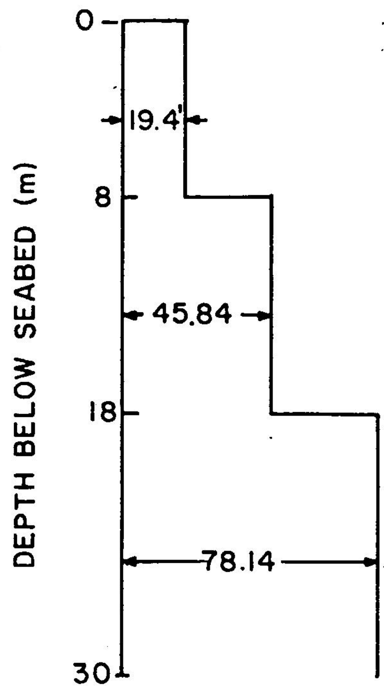

FIG. 6 VARIATION OF UNDRAINED SOIL SHEAR STRENGTH (kN/m^2)

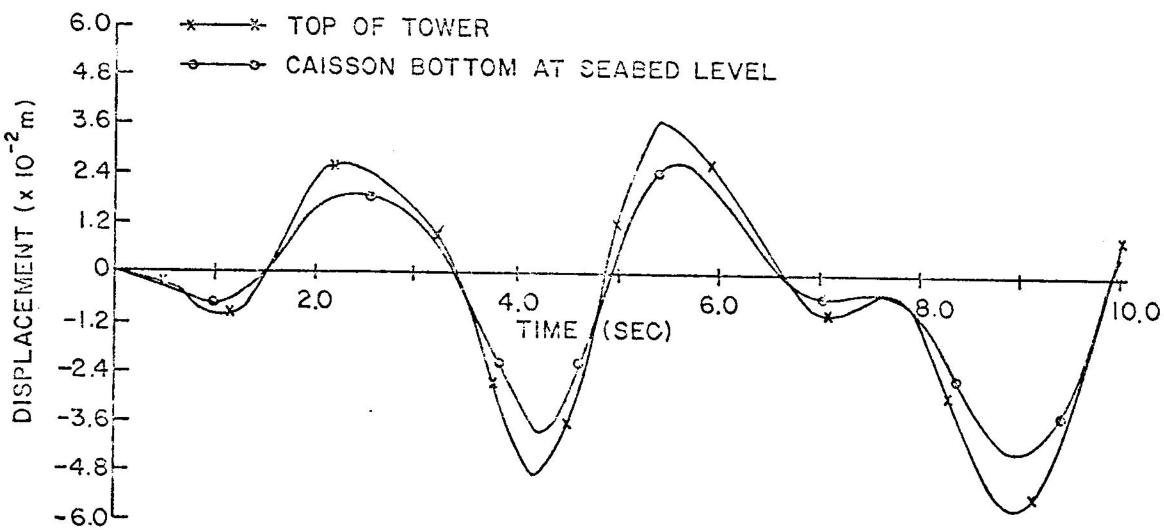

FIG. 7 TIME HISTORY OF HORIZONTAL DISPLACEMENT

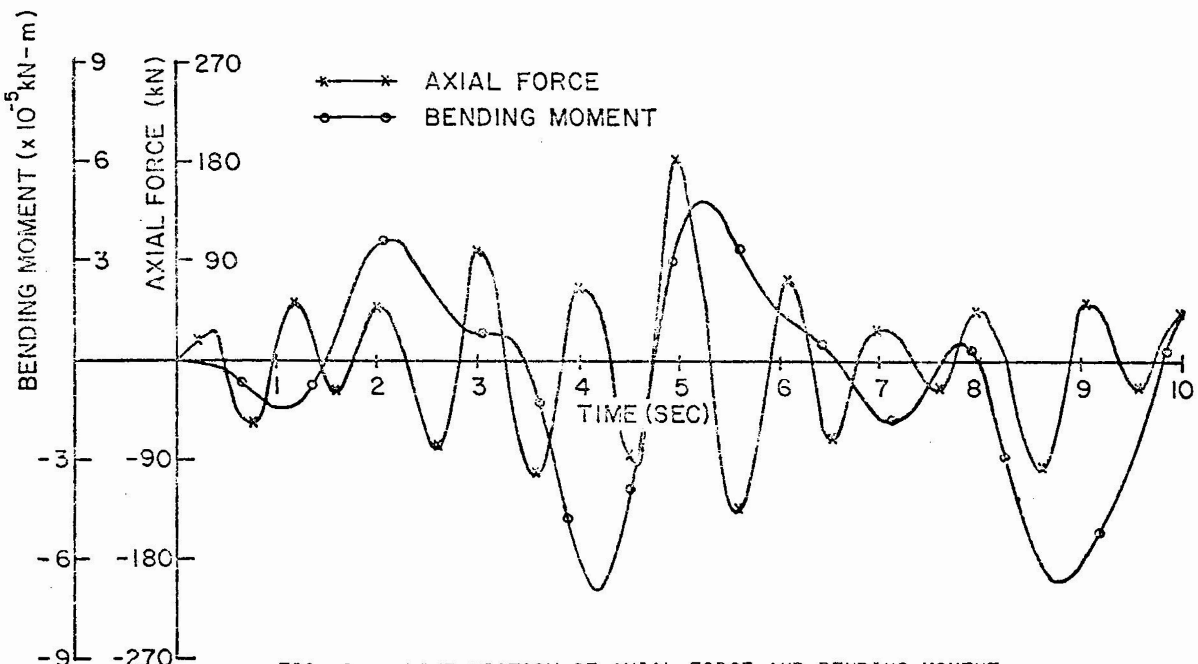

FIG. 8 TIME HISTORY OF AXIAL FORCE AND BENDING MOMENT AT BASE OF TOWER

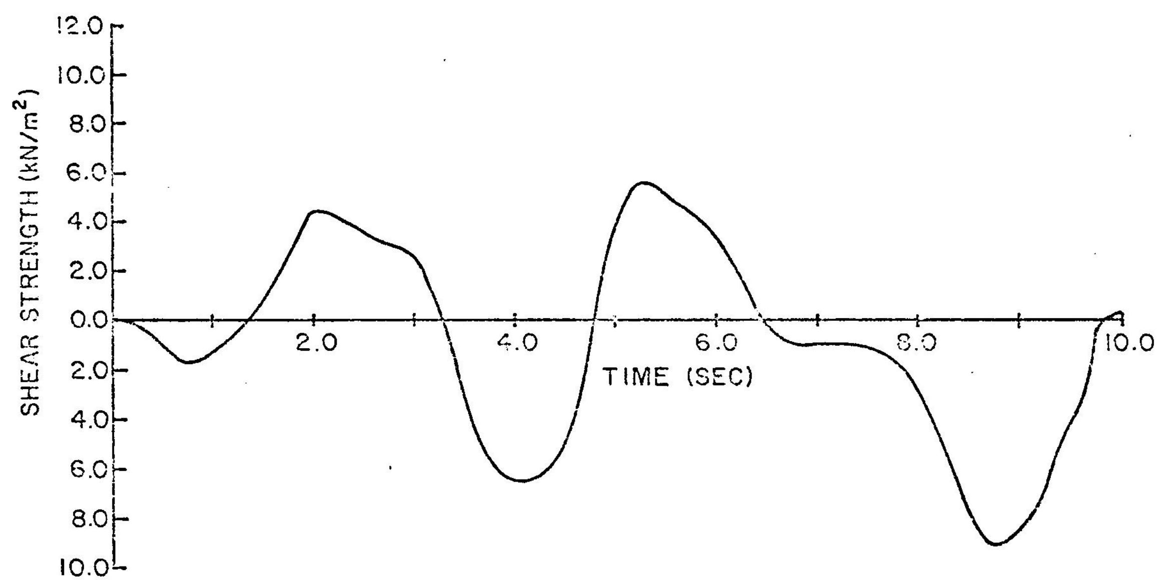

FIG. 9 TIME HISTORY OF SHEAR STRESS AT CENTROID OF SOIL ELEMENT 22