PROGRESS REPORT
RECENT EXPERIENCE FROM THE PRACTICE OF SEISMIC ANALYSIS FOR STRUCTURES

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SUMMARY

Since the last World Conference on Earthquake Engineering in New Delhi in 1977, many advances have been made in the fields of Engineering Seismology and Earthquake Engineering. The incorporation of new methodologies into consulting practice generally requires a degree of caution on the part of the consultant as the State-of-the-Art is far from unanimous in certain key issues.

The following is a brief account of D'Appolonia's experience and contribution to the practice of earthquake engineering.

INTRODUCTION

The aim of the paper is to discuss examples of the application of seismic analyis in practice. This is done by discussing methodologies as applied to specific cases.

The performance of seismic analyses in practice requires that the results of State-of-the-Art research are used to determine practical engineering solutions. Where these analyses are not simple, they require the understanding of soil and structural behavior under seismic loading. Thus, for example, nuclear power plants have been successfully licensed using a State-of-the-Art engineering approach, rather than by adhering strictly to regulatory guidelines. The same engineering approach can also be beneficial to other types of structures, where safety or protection of sensitive equipment is important, such as offshore platforms, industrial and research buildings.

The following discussion covers a range of topics including:

- Seismic input,
- Soil liquefaction,
- Soil-structure interaction,
- Structural analysis, and
- Bearing capacity.

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SEISMIC INPUT

In recent years, considerable thought has been given to the effects of surface waves and near-field earthquakes on the seismic response of structures. Each of these phenomena has been investigated and applied in recent projects.

Surface Waves

All earthquakes generate a wide variety of waves including:

- Obliquely propagating compression (body) waves,
- Obliquely propagating shear (body) waves,
- Rayleigh waves, and
- Love waves.

The last two types of waves are known as surface waves since they propagate horizontally and are confined to the near-surface soil or rock layers. These waves travel with about the same velocity as a shear wave at a depth of roughly one wavelength and as they impinge on a structure they may impose torsional and rocking motions.

Love waves are always dispersive whereas Rayleigh waves are dispersive in layered media.

The usual simplification in the present state of practice is to assume vertically propagating body waves which implies that all points on the foundation of a structure move together. However, since surface waves travel across the base of the foundation at a finite velocity, seismic analysis for surface waves must consider that different points along the foundation are excited out of phase. These can be modelled by specifying as input the foundation vertical and horizontal time histories of motion, with a time lag between the input to the various points along the foundation consistent with the velocity of propagation of the seismic wave.

Using this approach, the torsional and rocking effects of traveling waves on the foundation mat were investigated for a nuclear power plant founded on aseismic bearing pads. The soil compliance for the soil-structure interaction was represented by a higher order Winkler model (Hall et al., 1979) in order to compute the horizontal, vertical and rocking responses simultaneously. The structural response obtained from the traveling wave excitation was found to be approximately equal to that obtained from the usual assumption of vertically propagating body waves (Richli et al., 1980).

Near-Field Earthquakes

For most structures which have been seismically analyzed, the response of the structure has been computed considering excitation by distant earthquakes i.e., those events which occur at a distance from the site over which the use of an attenuation relationship is plausible. Design

seismic excitation is usually in the form of one or several appropriate real acceleration time histories or an artificial acceleration time history matching a specified response spectrum. Such design time histories generally have a duration of 20 to 30 seconds, exhibit moderate accelerations (0.1 to 0.3g), and have predominant frequencies between one and five Hertz. Since important structures such as nuclear power plants are generally located in areas remote from major zones of seismic activity these events are qualified as distant earthquakes. However, it is generally not feasible to have the site in a totally aseismic area, therefore where seismotectonic conditions dictate a near-field earthquake is postulated. A near-field earthquake is an event of small Magnitude, shallow focal depth and short duration of strong ground motion (about three seconds). The radius of effect of such earthquakes is typically 5 to 15 kilometers. The ground motion is characterized by a high frequency content (four to eight Hertz) and a few exceptionally high acceleration spikes, typically in the range of 0.4 to 0.7g. The acceleration amplitudes appear to be very dependent on factors such as the nature of the causative fault. The seismic design of a structure based on a distant earthquake is, when appropriate, verified for near-field effects by using one or several suitable acceleration time histories recorded at a station located near an epicenter.

Analyses using near-field time histories have recently been performed to check the design of nuclear power plants. Examples of a design artificial time history and that of a near-field earthquake utilized by D'Appolonia are shown in Fig. 1.

SOIL LIQUEFACTION

The possibility of soil liquefaction under seismic excitation has required many analyses of liquefaction potential for nuclear power plants founded on sandy profiles, and an increasing number of such analyses for offshore structures, such as gravity bases for single point moorings. Once the design earthquake has been evaluated, the first step should be a study of the soil profile, and in particular the results of standard penetration tests.

Quite convincing correlations have been developed between the field evidence of liquefaction or its absence, and the standard penetration resistance (Peck, 1979). Furthermore an understanding of the process of liquefaction and relative importance of the various factors involved leads an experienced engineer to a better position to assess the liquefaction potential of a site. If the empirical correlations indicate no liquefaction (e.g. consistently high values of SPT count) parallel analytical approach can be forgone. However, if low blow counts are measured a two-fold approach is adopted:

- Judicious examination of the blow counts, e.g., consideration of clay content of location where low blow counts were observed.

- Seismic assessment of the site and appropriate laboratory tests.

The results of both approaches will provide the basis for the engineer to decide on the liquefaction potential of the site. This approach was used in a recent study to rule out liquefaction at the site where a few low SPT N-values corrected for the in-situ overburden pressure were recorded.

A liquefaction study based on analysis and laboratory results involves the following major steps: the laboratory determined shear stresses to cause liquefaction, the computation of in-situ seismic shear stresses and finally the computation of the factor of safety.

The in-situ seismic shear stresses can be determined using a simplified procedure (Seed and Idriss, 1971) or by using the computer program SHAKE or FLUSH. The factor of safety is determined by dividing the real resistance (corrected laboratory resistance) by the in-situ seismic shear stresses.

The usual procedure to date, in determining the in-situ shear stresses has been to use one or more appropriate real acceleration time histories as the seismic excitation, since the use of an artificial acceleration time history with its high frequency content yields unrealistically high shear stresses in the soil. However, an artificial time history can be used as the seismic excitation if the appropriate correction factors are employed (Lu et al., 1976).

SOIL-STRUCTURE INTERACTION

Our soil-structure interaction analyses emphasize the use of the lumped parameter approach and our research efforts have been directed toward improving the lumped parameter modeling techniques for situations such as distributed compliances and deeply embedded structures, since this method is economic and flexible.

Structures on Soil

The analysis of a nuclear power plant founded on aseismic bearing pads incorporated a recent advance in soil-structure interaction modeling known as a higher order Winkler model (Hall et al., 1979). This model is shown schematically in Fig. 2.

In a conventional Winkler model, the soil springs do not differentiate between vertical and rocking motions. In actuality, however, the soil pressure beneath a point due to vertical translation is different from the soil pressure due to the vertical component associated with rocking of the foundation. The Winkler model cannot handle this condition since only the diagonal terms of the soil stiffness matrix are represented. Therefore, a higher order Winkler model was developed which includes off-diagonal terms in the soil stiffness matrix so as to simultaneously account for translational and rotational modes of soil-structure interaction. This is achieved by introducing elements with the capability of converting rotations into translations. The higher order Winkler element is physically an assemblage of two rigid beam elements connected by a moment spring. The rigid links connect two adjacent soil nodes with the two corresponding foundation nodes and have the capability of

transmitting shear only due to the rotation of the moment spring. In this way, the moment from the rotational spring is converted into a force at the foundation and soil nodes so that rotation of the foundation is resisted only by the force applied at the foundation nodes. The total soil reaction at any soil node is then given by the sum of the direct translational component and of the higher order Winkler spring. A similar procedure was adopted for the distribution of damping parameters.

In the case of uniform vertical translation (vertical mode) the coupling elements are not activated and, therefore, the vertical stiffness is correctly predicted.

The derivation of the complete higher order Winkler model involves the following steps:

- Compute the distributed linear springs compatible with each translational and rotational mode of vibration (vertical, horizontal, rocking and torsional). A set of springs is computed for each mode.

- Retain the distributed springs corresponding to the translational modes and compute coupling springs so that the stiffness for the rotational modes is correctly predicted.

These two steps are described in more detail by Hall et al. (1979).

The off-shore platform described by Gurpinar et al. (1980) required the modeling of a complex three-dimensional structure founded on piles. The dynamic analysis also had to consider the inertia effect of the platform body accelerating under water and the effect of hydrodynamic drag on the structural members. The inertia was included in the analysis by means of the so-called "added" or "virtual" mass, while the drag was found to be insignificant. The four groups of piles supporting the platform structure were modeled as vertical and horizontal springs and dashpots. The spring constants were computed using the formulation of Poulos (1979) at a stress level compatible with the stresses induced by the design earthquake ground motion, while the radiation damping for the soil pile system was evaluated from a study by Roesset and Angelides (1979). The structure itself, was modeled as a network of beam elements with "virtual mass" added to the elements located below the water level. Of the two time histories presented in Fig. 1 the near-field earthquake was predicted to give negligible response in comparison with the artificial time history although the peak acceleration for the near-field is three times greater.

The dynamic analyses of the above structures were performed with the computer code DAPSYS (D'Appolonia, 1977). This general purpose structural analysis program can be used with spring and dashpot models of the foundation soil and various lumped mass or finite element models of the structure. The dynamic analysis can be performed by several methods:

Response history analysis by mode superposition,

- Response history analysis by direct integration, or
- Response spectrum analysis.

Modal damping in the program is computed by the method proposed by Roesset et al. (1973) which is an extension of Biggs (Whitman, 1970) energy method. Therefore, the composite modal damping for each mode of the dynamic soil structure interaction model of a structure includes both hysteretic (material) and viscous (radiation) damping.

Structures In Soil

The seismic analysis of structures buried or deeply embedded in soil has advanced considerably in recent years. Two types of buried structures, shallow horizontal tunnels (Constantopoulos et al., 1980) and vertical deep well casings (Michalopoulos and Ries, 1980) have been studied in detail. In addition, a method of analysis for deeply embedded structures (Hall and Kissenpfennig, 1975) has been applied to a nuclear power plant structure having a deep foundation.

Relatively flexible buried structures (such as pipelines) are frequently analyzed by postulating strain compatability between the structure and the soil, i.e., the structural strains are considered equal to the strains of the surrounding soil and the stresses are computed by multiplying these strains by the modulus of the tunnel material (Newmark and Hall, 1977). Such an analysis, which neglects soil-structure interaction is applicable due to the high coupling stiffness, the high natural frequency, the high radiation damping and low structural stiffness associated with these buried structures. For a stiff tunnel, however, this method of analysis is very conservative and soil-structure interaction effects must be considered to reduce this conservatism. Constantopoulos, et al. (1979, 1980) have presented a simplified method of analysis for tunnels which considers bending and longitudinal loads induced by traveling surface waves and transverse loads induced by compression, shear and Rayleigh waves. Constantopoulos et al. (1980) paper also provides stress modification factors based on finite element soil-structure interaction analyses which can be employed in the proposed analytical method.

The seismic analysis of a dewatering system presented by Michalopoulos and Ries (1980) analyzes a flexible diaphragm wall and deep wells. The analysis of the diaphragm wall is similar to that for a tunnel but it can be shown that soil-structure interaction effects can be ignored since the diaphragm has approximately the same stiffness as the surrounding soil. The seismic analysis of the deep wells also considered shear, axial and bending stresses induced by shear and compression waves. An interesting aspect of the project was the analysis of the pump and discharge pipe supports within the casing using a traveling wave analysis for a vertically propagating shear wave.

In order to compute a more realistic response for deeply embedded foundations, a recent development in lumped parameter modeling (Hall and Kissenpfennig, 1975) has been applied to a nuclear power plant structure having an embedment ratio (embedment depth to minimum foundation width) of

0.6. The soil stiffnesses were computed using a procedure developed by Johnson et al. (1975) to determine the horizontal and rocking springs for an embedded foundation. In addition to the horizontal and rocking springs, this procedure also yields a stiffness coupling parameter which defines the location of the resultant horizontal stiffness in terms of its distance above the base of the foundation. For simplicity the coupling term may be removed from the stiffness matrix by applying the horizontal spring at this point. The rocking spring at the base of the foundation must then be adjusted to account for the rocking stiffness induced by the offset horizontal spring. This approach has been applied to a narrow, deeply embedded building and yielded considerably lower seismic moments than the usual lumped parameter approach considering all the springs and dashpots attached at the base of the foundation. This result is reasonable since it is known that embedment significantly increases radiation damping (Novak and Beredugo, 1971) and, therefore, reduces structural response.

Sensitivity Analysis

One of the main advantages of the lumped parameter approach for seismic analysis is the ease and economy with which parametric studies can be performed. In a lumped parameter model, the stiffness and damping parameters can easily be varied to determine the effects on the seismic response of the structure. Recently an approach has been presented to evaluate the effects which field and laboratory measurements of soil properties have upon the values of the stiffness parameters and the range necessary to be considered in the analysis (Micharlopoulos et al., 1979).

The soil-structure interaction stiffness parameters, as determined from half space solutions are a function of the following soil properties:

- Shear wave velocity,
- Poisson's ratio,
- Total unit weight, and
- The reduction in shear modulus with increasing shear strain.

Since all of the above field and laboratory measurements were performed using the same State-of-the-Art techniques, variations in the measurements are due primarily to differences in the soil properties rather than to systematic errors in the measuring techniques. For each type of measurement, the values of mean and standard deviation were determined. A series of stiffness coefficients were generated for each mode of vibration by selecting random values of each soil property within the range of the mean plus and minus one standard deviation (with the exception of Poisson's Ratio which showed very little variation). For each mode, the values of the mean and standard deviation were then computed for the generated set of stiffness parameters. A sensitivity factor, defined as the ratio of the mean stiffness plus or minus the standard deviation divided by the mean stiffness was computed for each mode of vibration and was found to have a maximum value of about 1.3. It should be noted that the sensitivity factor is site-specific and is a function of the variation in the measured soil properties. This method was applied to two structures at a nuclear power plant in order to determine an appropriate range of stiffness parameters to be used in the parametric analyses of structural response.

BEARING CAPACITY ANALYSIS

Existing bearing capacity formulae for foundations subjected to eccentric and inclined loads (e.g., Meyerhof, 1953), have been developed using small scale experiments and lead to conservative factors of safety when utilized for large structures such as nuclear power plants. A simplified method involving an iterative solution which accepts horizontal loads and moments as well as vertical loads has, therefore, been developed. It should be noted that this procedure is still somewhat experimental in nature but is presented herein to promote discussion.

Briefly, the method amounts to an initial strain non-linear analysis whereby the stiffnesses are obtained from half space closed form formulation.

The loads and moments imposed on the foundation are modeled by a combination of uniform vertical and horizontal loads and linearly varying loads. The foundation strata are modeled by a grid of points in an elastic half space, chosen to give a depth and breadth appropriate to contain the expected yield zones. This arrangement is shown in Fig. 3. Also input into the model is the initial undrained shear strength (S_u) of the foundation material at each level of the grid, as cohesion (c) and friction angle (ϕ).

The stress situation due to the loads in the given grid of points is computed using elastic half space theory (Gerrard and Harrison, 1974). The formulation was derived for a plane strain environment and a weightless soil, so at each point the stresses due to the weight of the overlying soil are added to give the complete state of stress within the soil.

The soil is assumed to be elastic plastic, that is with a stress-strain curve as illustrated in Fig. 4A.

Once the stresses and strengths at each point in the half space as illustrated in Fig. 3 are computed, the grid is scanned for points where the maximum shear stress (τ_{max}) exceeds the shear strength. The initial computation is generally made with the design loads acting on the foundation, and the shear strength is usually not exceeded at any point in the soil mass. The foundation loads are then linearly increased in small increments until the maximum shear stress exceeds the shear strength at a point. Under the assumption of an elastic plastic soil, the shear stress in excess of shear strength is computed. This excess shear stress is resolved into horizontal and vertical normal stresses and shear stresses, and balancing external load is assumed to act on the element around the grid point in such a way as to give a resulting stress situation which produces a τ_{max} equal to the strength. The excess and balanced stress conditions are represented by Mohr's circles M_0 and M_1, respectively, in Fig. 4B, and the method of application of the redistributed stresses is illustrated in Fig. 4C. The effect of balancing external load on the yielding grid point is then computed at all grid points.

The effect on the other points is computed following Melan's (1932) equations (Poulos and Davis, 1974; 1977), which give the resultant horizon-

tal, vertical and shear stresses for a given subsurface point load. To account for a uniform loading on the element sides, the equations are integrated over the appropriate length, resulting in a set of influence factors to give the resultant stresses at any point for a given unit load.

The excess stress on one element is thus redistributed over the whole grid system (i.e., the half space), and the resulting stresses are added to the existing stresses. The theoretical state of stress at each yielding point at this stage, falls between the initial state of stress and that defined by the failure criterion. The procedure is repeated, and in the subsequent iterations the effects on an element of its own excess stresses becomes smaller and smaller. Eventually, after an appropriate number of iterations, the shear stress at all grid points does not exceed the relevant shear strength by more than an allowed tolerance, and the material satisfies equilibrium without violating the failure criterion, as illustrated in Fig 4A.

The applied foundation load is further increased and, since the material is considered elastic, the effect of the increase of the surface load is added linearly to the pre-existing stresses. After increasing the stresses the procedure is repeated: the strength is recomputed, the stresses are checked against the strength, and the excess stresses are redistributed.

For each load stage the excess stresses are used to compute the additional vertical displacement of the center of the footing over and above the elastic displacement. This is done using the program HSPACE (Lysmer, 1971) which uses Mindlin's (1936) equations, integrated numerically. It is noted that this calcualtion employs one constant value of elastic modulus, and so the computed displacements cannot be used directly to assess settlements.

A plot of vertical surface displacement against applied load is thus obtained, as well as a computer print-out (and video image) of the failed grid points at each iteration. When the plastic zone in the grid considered is fully developed, increase of load does not lead, practically, to yield of additional points. Thus, the conditions within this plastic zone are nearly the same and result in an almost constant increment of plastic flow and vertical deflection with each successive load increment. These conditions are depicted in the load deformation diagram as the curve approaches a straight line for higher loads as shown in Fig. 6. The failure load is defined as the intercept on the load axis of the straight line into which the load deformation curve degenerates. The method of redistribution of stresses from yielding points described above, is similar to the method developed by Wittke (1977).

The analysis presented in this section incorporates the following assumptions:

- Elastic plastic soil that is with a stress-strain curve as illustrated in Fig. 5A.

- Overall stress equilibrium, i.e., statically admissible stress field is obtained, but plastic flow is not modeled, per se

- Dilatancy effects in sand after yield are not modeled.

- Melan's (1932) equations (Poulos and Davis, 1974; 1977) are used to distribute the excess stresses throughout the soil mass.

The above assumptions will give realistic results before yield, but then are approximate once yield has been initiated. Verification analyses comparing the results obtained with this method to conventional soil mechanics solutions indicate good agreement. The method is, therefore, considered to be adequate to assess the bearing capacity of structures subjected to horizontal and vertical loads and moments.

RESEARCH NEEDS

The need for energy is accelerating the construction of more nuclear power plants and offshore structures for production or transportation of oil or gas. However, there is also an ever increasing awareness of the environmental impact these structures will cause in case they fail or malfunction. Seismic effects are some of the most critical considerations for the design of such very important structures which have to operate at very low risk levels. The conflict between the energy demand and environmental awareness will inevitably result in more detailed and sophisticated requirements especially for nuclear power plants.

Engineering practice is relatively capable of incorporating well tested methods and sound data in conventional earthquake engineering. However, for the seismic analysis of nuclear power plants the practice and research are advancing at about the same rate. More research is needed to give the practising engineer increased confidence in the sophisticated methodologies he is increasingly required to use or develop. This would also have the benefit of shortening the time required for licensing procedures.

ACKNOWLEDGEMENTS

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LIST OF REFERENCES

Constantopoulos, I.V., J.T. Motherwell, and J.R. Hall, Jr., April 1979, "Dynamic Analysis of Tunnels," Third International Conference on Numerical Methods in Geomechanics, Aachen, Germany.

Constantopoulos, I.V., J.R. Cole-Baker and A.P. Michalopoulos, September 1980, "Seismic Analysis of Buried Tunnels," to be presented at the Seventh World Conference on Earthquake Engineering, Istanbul.

D'Appolonia Consulting Engineers, Inc., 1977, "DAPSYS: A Computer Code for Soil-Water-Structure Interaction Effects," Proprietary, Brussels.

Gerrard, C.M. and W.J. Harrison, 1974, "Stresses and Displacements in a Loaded Orthorhombic Half Space," Appendix A to Elastic Solutions in Soil and Rock Mechanics, by H.G. Poulos and E.H. Davis, John Wiley and Sons, Inc.

Gürpinar, A., B.J. Gryspeert, and J.R. Cole-Baker, September 1980, "Dynamic Analysis of Offshore Platforms under Seismic Excitation," to be presented at the Seventh World Conference on Earthquake Engineering, Istanbul.

Hall, J.R., Jr. and J.F. Kissenpfennig, September 1975, "Special Topics on Soil-Structure Interaction," International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures, Paper No. U2/2, West Berlin.

Hall, J.R., Jr., I.V. Constantopoulos and A.P. Michalopoulos, April 1979, "Higher Order Winkler Model for Soil-Structure Interaction," Proceedings of the Third International Conference on Numerical Methods in Geomechanics, Aachen, Germany, pp. 933-938.

Johnson, G.R., P.P. Christiano and H.I. Epstein, August 1974, "Stiffness Coefficients for Embedded Foundations," Proceedings of the ASCE Power Division Specialty Conference, Boulder, Colorado.

Lu, B.T.D., J.A. Fischer, and J. Peir, August 1976, "Effects of Earthquake Input in Seismic Responses on Nuclear Power Plant Sites," International Symposium on Earthquake Structural Engineering, St. Louis.

Lysmer, J., April 1971, "HSPACE - A Digital Computer Program for the Determination of Stresses and Displacements in an Elastic Half-Space," Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley.

Melan, E., 1932, "Der Spanningszustand der durch eine Einzelkraft im Innern Beanspruchten Halbschiebe, Z. Angew.," Math. Mech., Vol. 12.

Meyerhof, G.G., 1953, "The Bearing Capacity of Foundations under Eccentric and Inclined Loads," Proceedings of the Third International Conference on Soil Mechanics and Foundations Engineering, Vol. 1, Zürich, Switzerland.

Michalopoulos, A.P., K.R. Hansen, R.P. Arias, and D.A. Raynaud, September 1979, "Measurement, Selection and Use of Dynamic Soil Properties and Design," Seventh European Conference on Soil Mechanics and Foundation Engineering, Brighton, England.

Michalopoulos, A.P. and E.R. Ries, September 1980, "Seismic Analysis of a Dewatering System," to be presented at the Seventh World Conference on Earthquake Engineering, Istanbul.

Mindlin, R.D., 1936, "Force at a Point in the Interior of a Semi-Infinite Solid," Journal of Applied Physics, Vol. 7, No. 5, pp. 195-202.

Newmark, N.M. and W.J. Hall, 1977, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," NUREG/CR-0098, N.M. Newmark Consulting Engineering Services," Prepared for USNRC.

Novak, M. and Y.O. Beredugo, May 1971, "The Effect of Embedment on Footing Vibration," First Canadian Conference on Earthquake Engineering Research, Vancouver, British Colombia.

Peck, R.B., March 1979, "Liquefaction Potential: Science versus Practice," Journal of the Geotechnical Engineering Division, ASCE, GT3, pp. 393-398.

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

Poulos, H.G. and E.H. Davis, August 1977, Corrections to Elastic Solutions for Soil and Rock Mechanics (Unpublished).

Poulos, H.G., 1979, "Group Factors for Pile Deflection Estimation," the University of Sydney, School of Civil Engineering, Research Report No. R-351.

Richli, M., I.V. Constantopoulos and N.R. Vaidya, September 1980, "Seismic Design Considerations for Structures Suppored on Aseismic Bearing Pads," to be presented at the Seventh World Conference on Earthquake Engineering, Istanbul.

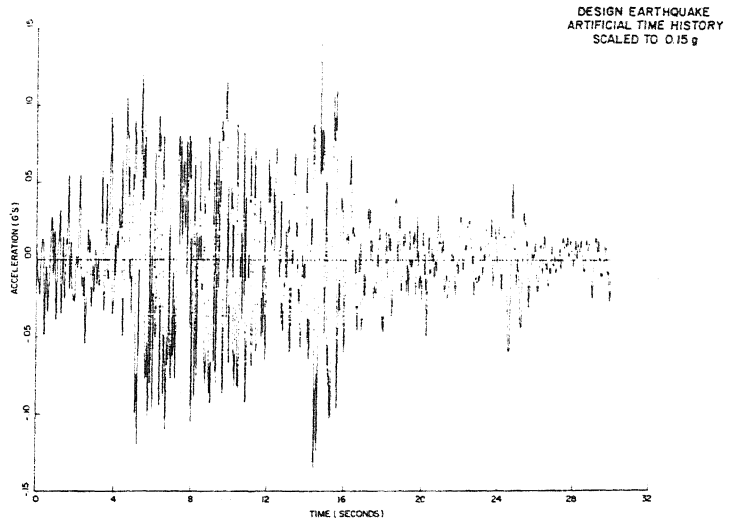
Roesset, J.M., R.V. Whitman, and R. Dobry, March 1973, "Modal Analysis for Structures with Fondation Interaction," Journal of the Structural Division, Vol. 99, No. ST3, pp. 399-416.

Roesset, J.M. and D. Angelides, 1979, "Dynamic Stiffness of Piles," Conference on Numerical Methods in Offshore Piling, Institution of Civil Engineers, London.

Seed, H.B. and I.M. Idriss, September 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM9.

Whitman, R.V., 1970, "Soil-Structure Interaction," Seismic Design for Nuclear Power Plants, MIT Press, Cambridge, Massachusetts.

Wittke, W., 1977, "Static Analysis for Underground Openings in Jointed Rock," Chapter 18 in Numerical Methods in Geotechnical Engineering, by C.S. Desia and J. Christian, McGraw-Hill Book Company, New York, 783 pages.



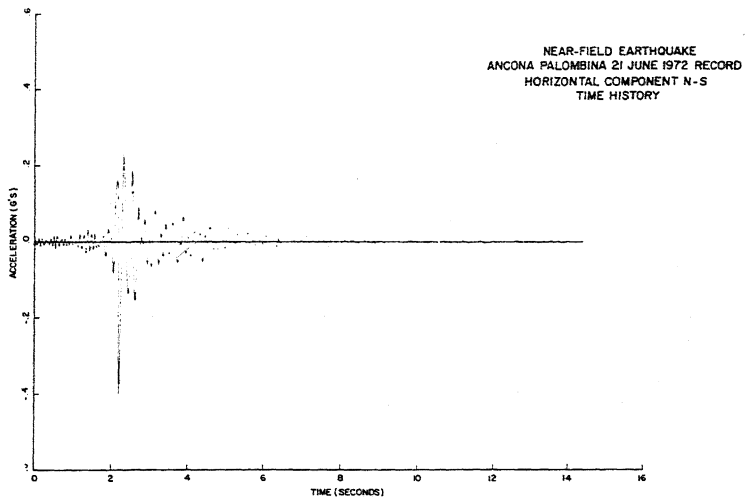


FIGURE I

DESIGN ARTIFICIAL
AND NEAR FIELD REAL TIME HISTORIES

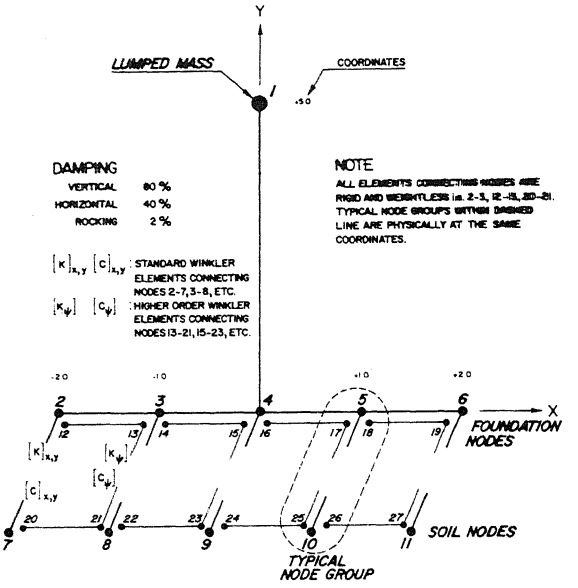


FIGURE 2

SCHEMATIC REPRESENTATION
OF HIGHER ORDER WINKLER MODEL

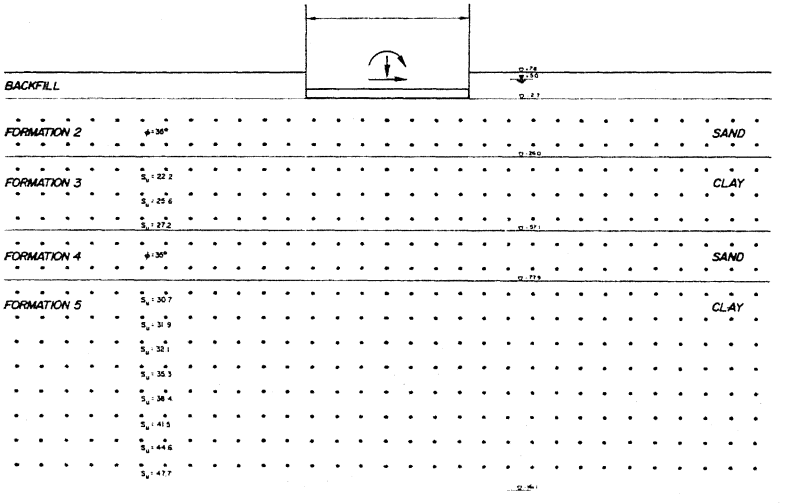
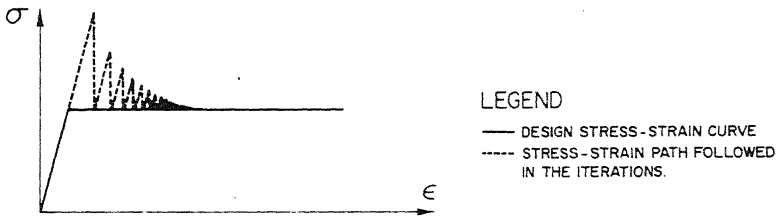
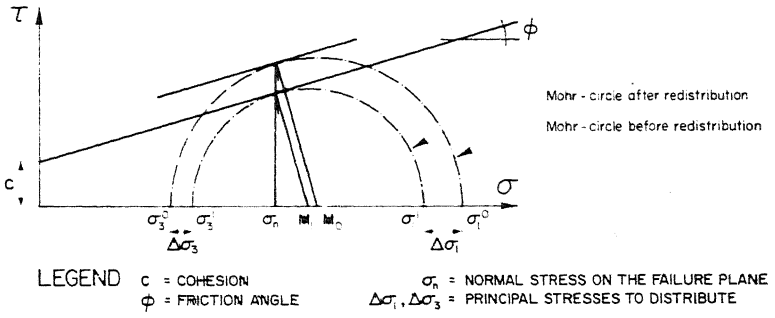


FIGURE 3

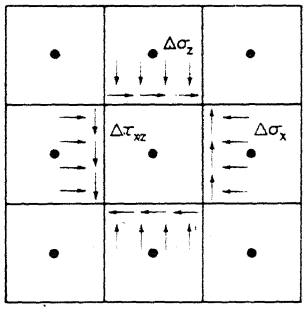
MODEL USED IN DAPCAP PROGRAM
TO COMPUTE DYNAMIC BEARING CAPACITY



A. Repeated loading and unloading due to redistribution of the excess stresses



B. Determination of redistributed stresses



C. Application of redistributed stresses

LEGEND

● GRID POINT

□ ELEMENT AROUND THE GRID POINT

FIGURE 4

STRESS REDISTRIBUTION IN CASE OF YIELDING

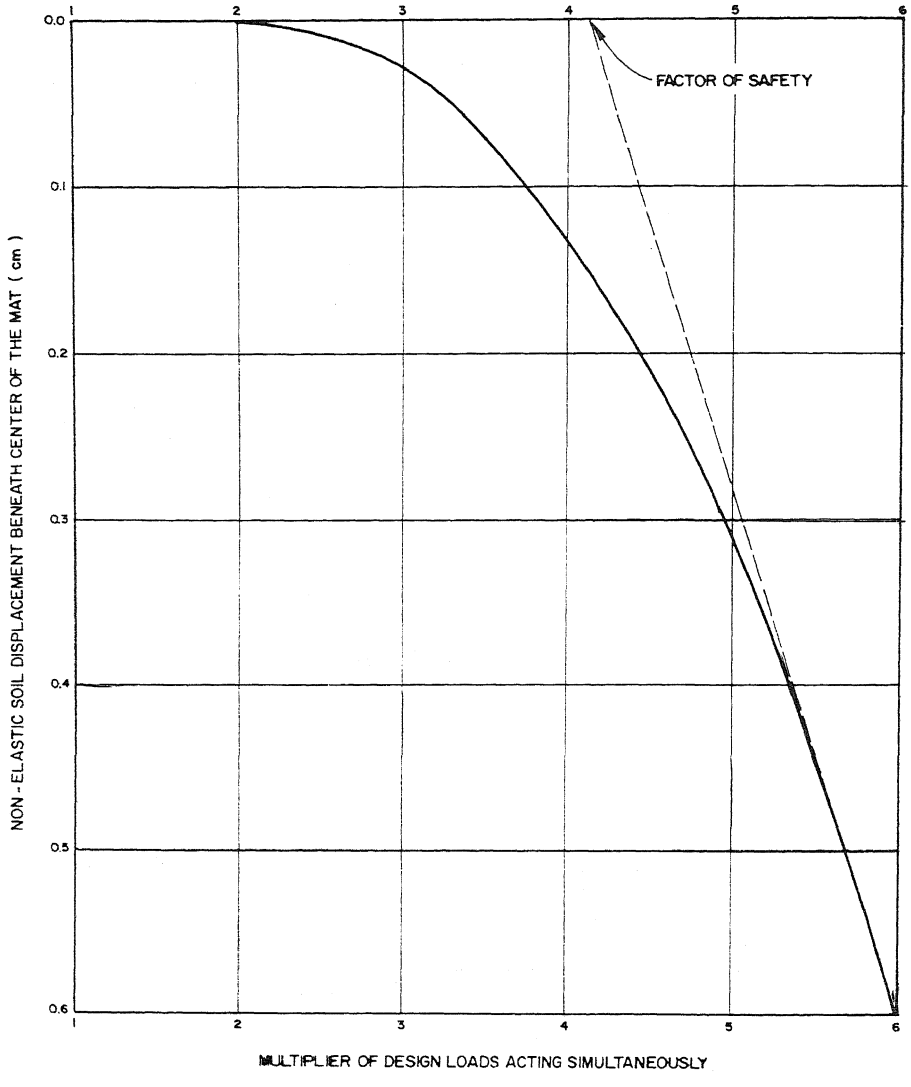


FIGURE 5

RESULTS OF BEARING CAPACITY ANALYSIS