RECENT DEVELOPMENTS IN THE STATIC AND DYNAMIC ANALYSIS OF PILE GROUPS

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ABSTRACT

Procedures for the nonlinear dynamic effective stress analysis of pile groups is presented which analyze the group as a unit - not just as single piles with interaction factors. The procedures are validated using the data from centrifuge tests on a single pile and a 2×2 pile group. A major case history from practice involving the analysis of piles under seismic excitation and post-liquefaction deformations is described.

INTRODUCTION

Seismic soil-structure interaction analysis involving pile foundations is one of the more complex problems in geotechnical engineering. A very common example is the 3-D analysis of a pile foundation for a bridge abutment. The analysis involves modelling soil-pile-soil interaction, the effects of the pile cap, nonlinear soil response, and in many cases incorporates seismically induced porewater pressures. There are many approaches to solving the dynamic response of pile foundations. Novak (1991) gave an extensive review of the more widely accepted methods of analysis. His study showed that pile group response cannot be deduced from single pile response without taking pile-soil-pile interaction into account and that the dynamic characteristics of pile groups are strongly frequency dependent and may differ significantly from the characteristics of a single pile.

The methods for direct group analysis of pile foundations are based on linear elastic behaviour using either boundary element or finite element techniques. Very complete elastic analyses can be conducted using 3-D boundary element formulations (El-Marsafawi et al., 1992a, 1992b), but they require substantial computing time. They are exact for elastic isotropic conditions, however, they cannot take into account the nonlinear behaviour of soil under strong shaking or the effect of seismically induced porewater pressures on dynamic response. The reduction in soil stiffness and the increase in damping associated with strong shaking are sometimes modelled crudely in these analyses by making arbitrary reductions in

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the shear moduli and arbitrarily increasing the viscous damping. For this reason, the results of these studies have not proved very useful for the response of pile foundations to earthquake loading.

The offshore industry pioneered the seismic design of pile foundations by estimating the nonlinear behaviour of single piles using nonlinear Winkler springs and estimating the group stiffness by using static interaction factors. The nonlinear Winkler springs were obtained from load-deflection curves on prototype piles in various soil conditions. The original basic studies were conducted by Matlock (1963) and Reese et al. (1974a, 1974b). Their studies led to the p-y procedure for offshore pile design recommended by the American Petroleum Institute. These curves were derived under static conditions or low-frequency cycling. Therefore, they do not incorporate the frequency effect on damping and stiffness. However, in many low frequency applications, the dynamic stiffness is similar to the static stiffness. El-Marsafawi et al. (1992a, 1992b) presented developed approximate procedures for estimating dynamic interaction factors based on boundary element analysis, the work of Kaynia (1982), Kaynia and Kausel (1982), Davies et al. (1985), and Gazetas (1991a, 1991b). They extended the range covered by these researchers using the general 3-D formulation developed by Kaynia. These factors are limited to elastic response.

For seismic analysis of pile foundations under strong shaking, reliance is placed mainly on the p-y approach for individual piles. For these nonlinear problems, the effective stiffness of the pile groups is affected strongly by the load on the group because the inertial effects of the superstructure cause additional strains in the ground and hence modify further the effective moduli and damping. This effect was recognized by Matlock et al. (1978) in the development of the computer program SPASM. There are well recognized problems with the p-y approach; determining the appropriate p-y curves for the site, the approximate nature of the representation of field conditions, and the difficulty of simulating appropriate interaction between the piles. Traditionally, static interaction factors have been used, but with their wider availability dynamic interaction factors may be adopted in future. Of course both sets of factors are based on elastic analysis.

Two procedures for nonlinear analysis of pile groups, under development at UBC, will be described here. The first is a quasi-3D method which permits dynamic nonlinear effective stress analysis of pile groups in layered soils. By relaxing some of the boundary conditions associated with a full 3D analysis, the computing costs can be substantially reduced and the analysis is feasible on the equivalent of a 486 PC. The procedure is validated here by data from centrifuge tests on a single pile and a 2×2 pile group.

The second method is based on the nonlinear static and dynamic methods of analysis incorporated in the program TARA-3. The program is modified to include elastic piles. This method is suitable for the analysis of piles in large groups where a 2D approximation is reasonable for all but the peripheral piles. The application of this method is demonstrated by a case history from practice involving the nailing of the upstream slope of a dam to the foundation soils across a weak layer which liquefies during the design earthquake.

3-D ANALYSIS OF PILES

Dynamic finite element analysis in the time domain using the full 3-dimensional wave equations is not feasible for engineering practice at present because of the time needed for the computations. A simplified 3-dimensional (quasi-3D) wave equation appropriate for many practical problems is presented here which reduces the computational demands to a level which can be accommodated on a 486 type PC.

Under vertically propagating shear waves (Fig. 1) the soil undergoes primarily shearing deformations in xOy plane except in the area near the pile where extensive compressional deformations develop in the direction of shaking. The compressional deformations also generate shearing deformations in yOz plane. Therefore, the assumptions are made that dynamic response is governed by the shear waves in the xOy and yOz planes, and the compressional waves in the direction of shaking, Y.

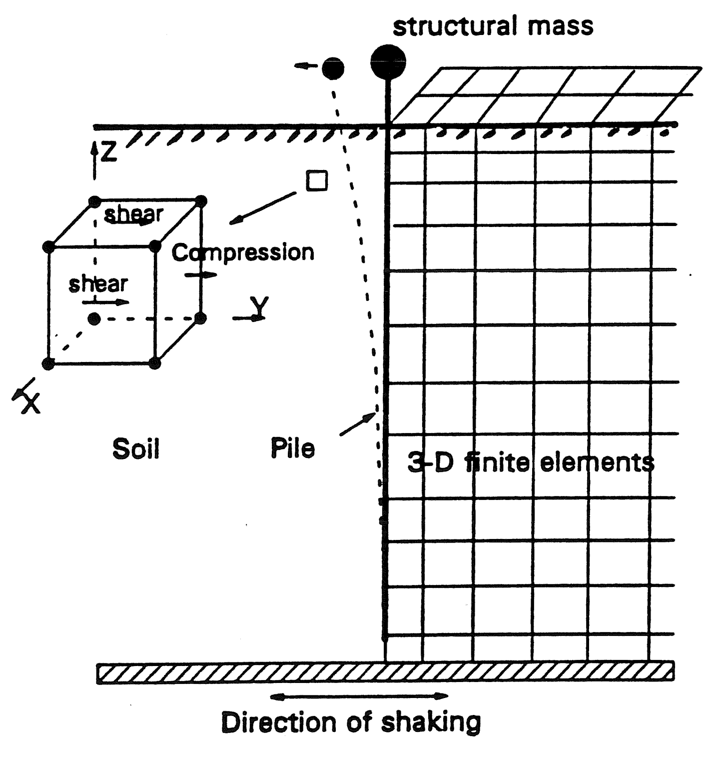


Fig. 1. Quasi-3D model of pile-soil response.

Let v represent the displacement of soil in the direction of shaking. The compression force is $\theta G(\partial^2 v/\partial y^2)$. The shear force in xOy plane is $G(\partial^2 v/\partial x^2)$ and the shear force in yOz plane is $G(\partial^2 v/\partial z^2)$. The inertial force is $\rho_s(\partial^2 v/\partial t^2)$. Applying dynamic equilibrium in Y-direction, the dyamic governing equation under free vibration of the continuum soil is written as

$$(1) \qquad G\frac{\partial^2 v}{\partial x^2} + \theta G\frac{\partial^2 v}{\partial y^2} + G\frac{\partial^2 v}{\partial z^2} = \rho_s\frac{\partial^2 v}{\partial t^2}$$

where G is the shear modulus, ρ_s is the mass density of soil, and θ is a coefficient related to Poisson's ratio of the soil.

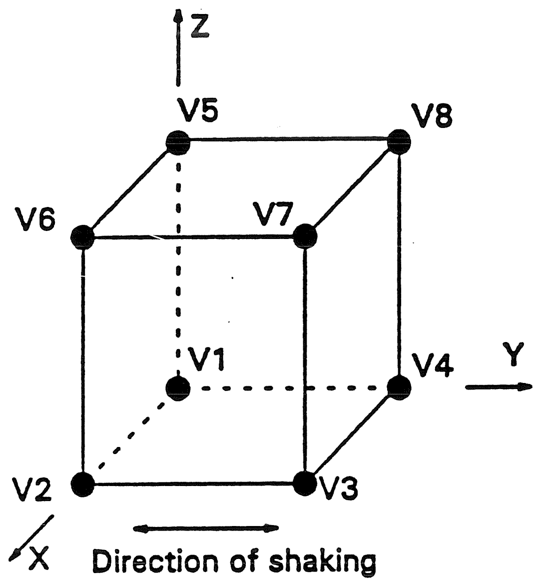
Piles are modelled using ordinary Eulerian beam theory. Bending of the piles occurs only in the yOz plane. Dynamic soil-pile-structure interaction is maintained by enforcing displacement compatibility between the pile and soils. An appropriate finite element procedure is employed that couples the motions of soil and pile, and enforces the displacement continuity between them. Any structural mass is attached at the pile head.

A quasi-3D finite element code PILE3D (Wu and Finn, 1994) was developed to incorporate the dynamic soil-pile-structure interaction theory decribed previously. An 8-node brick element is used to represent soil, and a 2-node beam element is used to simulate the piles, as shown in Fig. 2. The global dynamic equilibrium equation in matrix form is written as

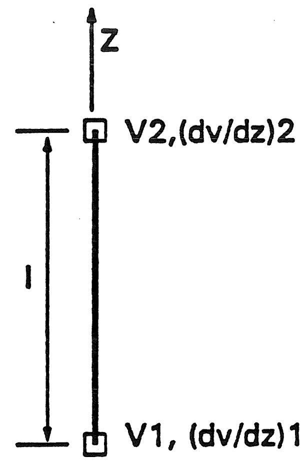
$$(2) \qquad [M]\{\ddot{v}\}+[C]\{\dot{v}\}+[K]\{v\} = -[M]\{I\}\cdot\ddot{v}_o(t)$$

in which $\ddot{v}_o(t)$ is the base acceleration, $\{I\}$ is a unit column vector, and $\{\ddot{v}\}$, $\{\dot{v}\}$ and $\{v\}$ are the relative nodal accelaration, velocity and displacement, respectively.

Direct step-by-step integration using the Wilson-θ method is employed in PILE3D to solve the equations of motion in (2). The non-linear hysteretic behaviour of soil is modelled by using a variation of the equivalent linear method used in the SHAKE program (Schnabel et al., 1972). Additional features such as tension cut-off and shearing failure are incorporated in the program to simulate the possible gapping between soil and pile near the soil surface and yielding in the near field.



Soil Element



Beam Element

Fig. 2. Finite element models for soil and pile.

SEISMIC RESPONSE ANALYSIS OF A SINGLE PILE

PILE3D was used to analyze the seismic response of a single pile in a centrifuge test which was carried out on the California Institute of Technology (Caltech) centrifuge by B. Gohl (1991). Details of the test may also be found in a paper by Finn and Gohl (1987). Figure 3 shows the soil-pile-structure systems used in the test. A 209.5 mm long stainless steel tube pile having an outside diameter of 9.52 mm and a wall thickness of 0.25 mm was embeded in a dry loose sand foundation. The model pile was instrumented by 8 pairs of foil type strain gauges mounted on the outside of the pile to measure bending strains at the locations shown in Fig. 3. An average centrifuge acceleration of 60g was used in the tests.

The pile has a free standing length of 16.5 mm above the soil surface. The effect of a super-structure was simulated by clamping a rigid mass to the head of the pile. The weight of the stuctural mass including the pile head insert and the pile head clamp was 2.416 N. The mass moment of inertia about the centre of gravity was I_{cg} = 0.0683 N.sec^2.mm. The centre of gravity of the mass was located 16.5 mm above the pile head. The model pile has an average flexural rigidity of 13.26 N.m^2 and a mass density of 74.7 kN/m^3.

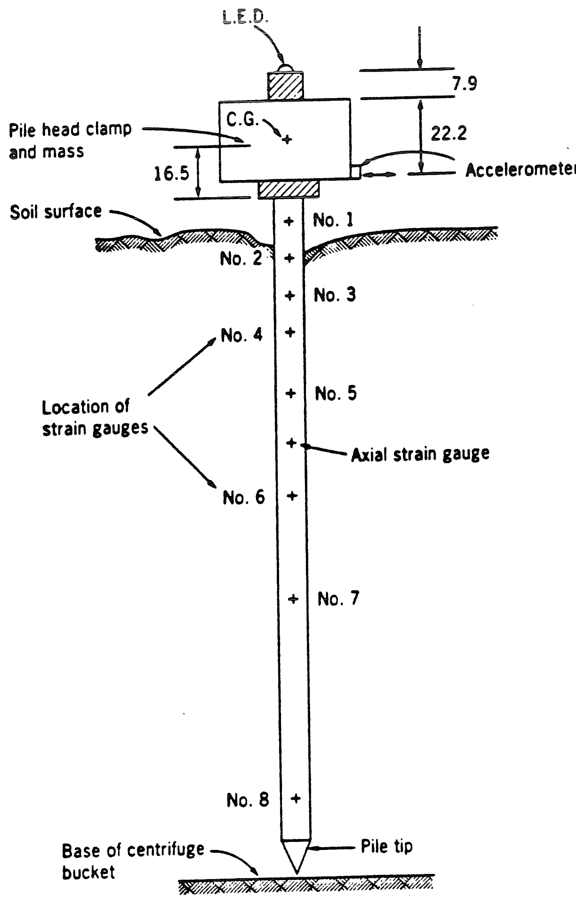


Fig. 3. The layout of the centrifuge test for a single pile.

The pile head mass was instrumented using a non-contact photovoltaic displacement transducer and an Entran miniature accelerometer. The locations of the accelerometer and light emitting diode (L.E.D.) used by the displacement sensor are shown in Fig. 3. The pile head displacements were measured with respect to the moving base of the soil container.

The sand used for the test was a loose sand with a void ratio $e_o = 0.78$ and a mass density $\rho = 1.50$ Mg/m^3. Gohl (1991) has shown that the low strain shear moduli of the sand foundation vary as the square root of the depth, and they can be quantitively evaluated using the Hardin and Black (1968) equation

(3) $$G_{max} = 3230 \frac{(2.973 - e_o)^2}{1 + e_o} (\sigma'_m)^{0.5}$$

where e_o is the in-situ void ratio of the sand and σ'_m is the mean normal effective confining pressure in kPa.

A horizontal acceleration record is input at the base of the system. The peak acceleration of the input motion is 0.158g. The computed Fourier amplitude ratios of the pile head response and the free field motion with respect to the input motions are given in Fig. 4(a) and Fig. 4(b). The natural frequency of the free field acceleration from Fig. 4(b) is estimated to be 2.75 Hz, and the fundamental frequency of the pile (from Fig. 4(a)) to be 1.1 Hz. The period of peak pile response is much longer than the period of the free field.

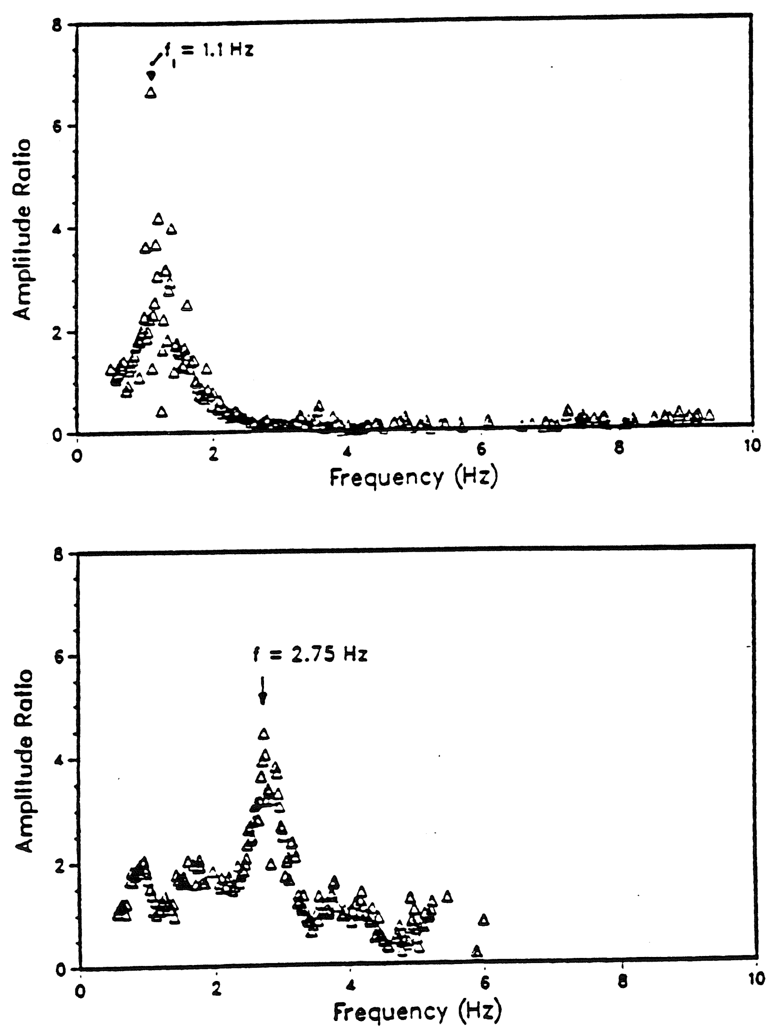


Fig. 4. The Fourier Spectra of accelerations (after Gohl, 1991).

The centrifuge test was analyzed by the quasi-3D finite element method of analysis using the program PILE3D. Figure 5 shows the finite element model used for analysis. The sand

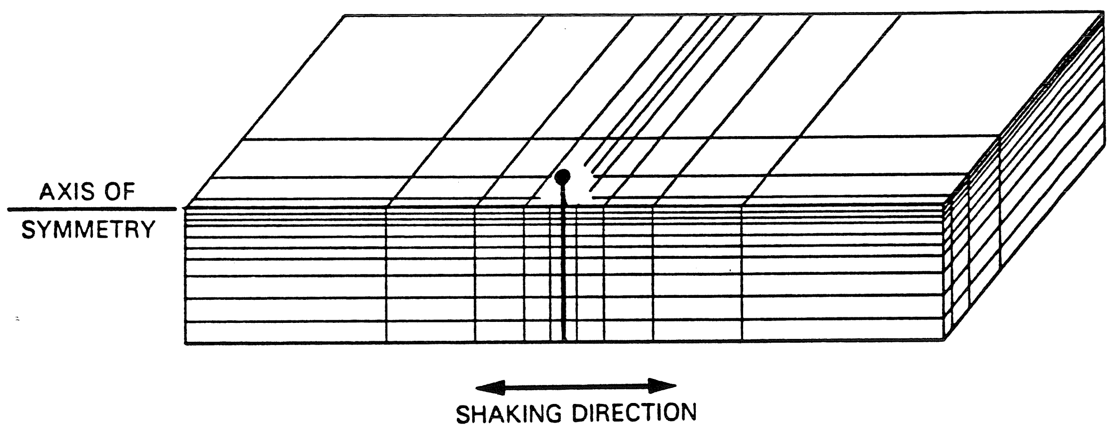


Fig. 5. The finite element modelling of centrifuge test

deposit is divided into 11 layers. Layer thickness is reduced as the soil surface is approached to allow more detailed modelling of the stress and strain field where lateral soil-pile interaction is strongest. The pile is modelled using 15 beam elements including 5 elements above the soil surface. The super-structure mass is treated as a rigid body.

The finite element analysis was carried out in the time domain. Nonlinear analysis was performed to account for the changes in shear moduli and damping ratios due to dynamic shear strains. The shear-strain dependency of both the shear modulus and damping ratio used in the analysis are shown in Fig. 6. The low strain shear moduli G_{max} were determined using Eq. (3).

The computed acceleration response at the pile head is plotted against the measured response in Fig. 7. Fairly good agreement between the measured and the computed accelerations is observed in the region of strong shaking.

The computed time history of moments in the pile at a depth of 3 m (near point of maximum moment) is plotted against the recorded time history in Fig. 8. There is satisfactory agreement between the computed and measured moments in the range of larger moments. The computed and measured moment distributions along the pile at the moment of peak pile head deflection are shown in Fig. 9. The computed moments agree quite well with the measured moments. The moments increase to a maximum value at a depth of 3.5 diameters, and then decrease to zero at a depth around 12.5 diameters. The moments along

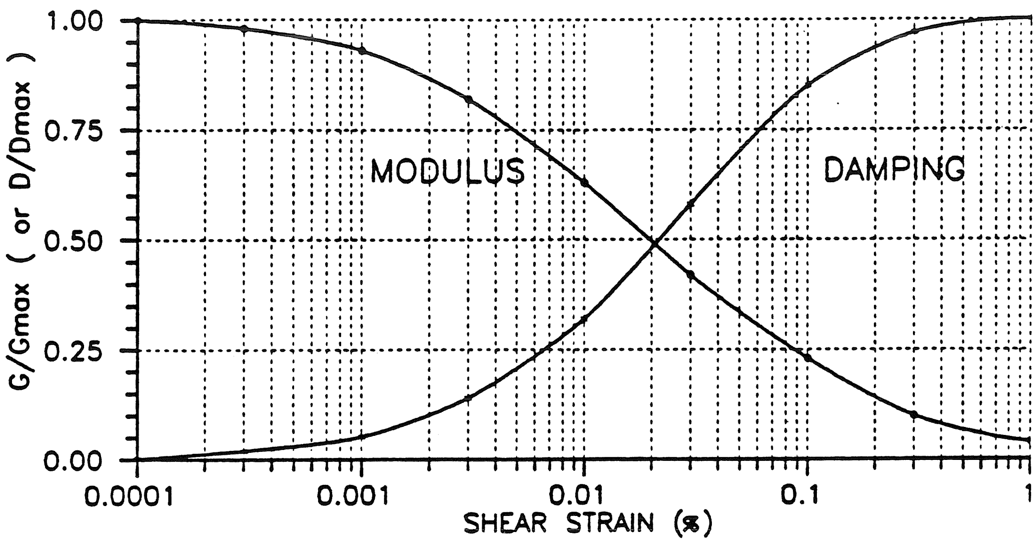


Fig. 6. The relationships between shear modulus, damping and the shear strain.

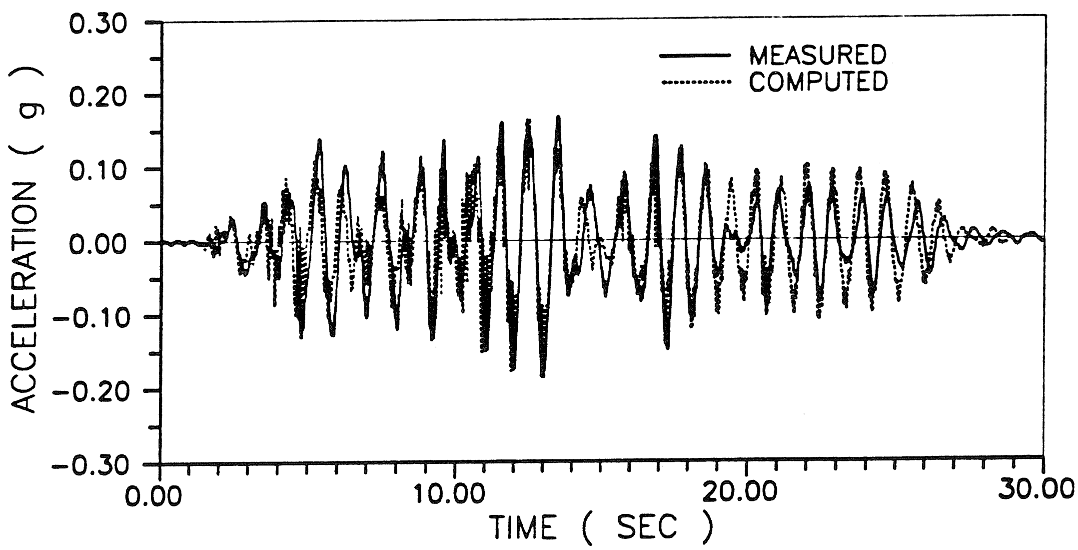


Fig. 7. The computed versus measured acceleration response at the pile head.

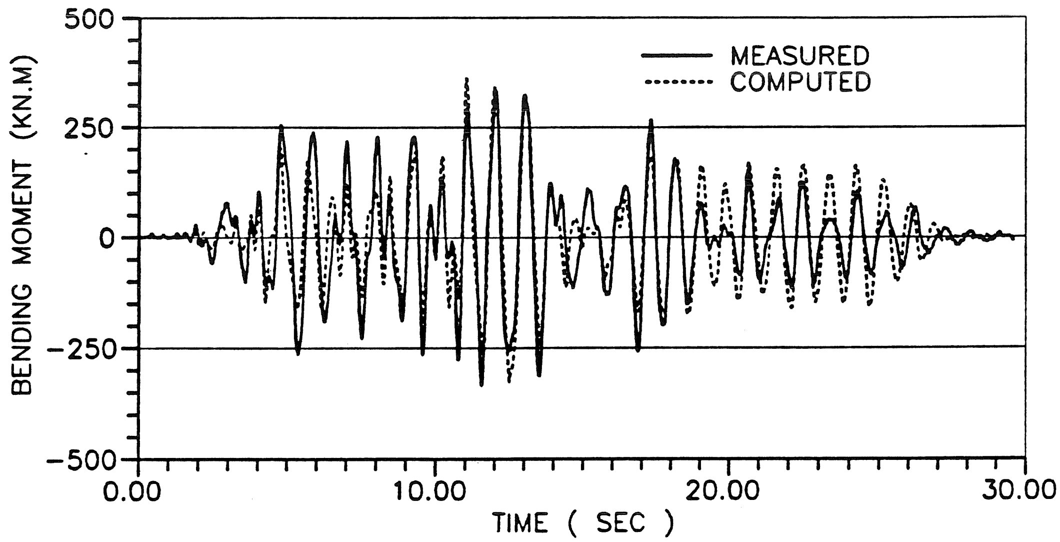


Fig. 8. The computed versus measured moment response at depth D = 3 m.

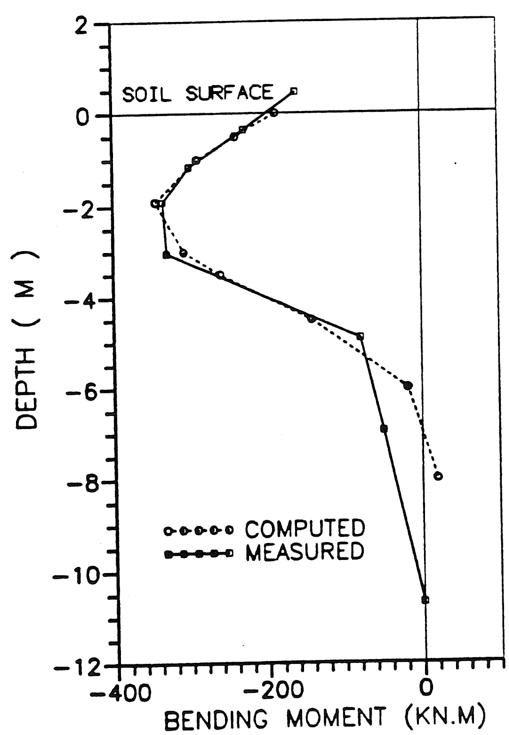


Fig. 9. The computed versus measured moment distribution of the pile at peak pile deflection.

the pile have same signs at any instant of time, suggesting that the inertial interaction caused by the pile head mass dominates response, and the pile is vibrating in its first mode. The peak moment predicted by the quasi-3D finite element analysis is 344 kNm compared with a measured peak value of 325 kNm.

Computational Time

Using a PC-486 (33MHz) computer, the average CPU time needed to complete one integration step is 7 sec for the finite element grid shown in Fig. 5, and 3 hours of CPU time are required for an input record of 1550 steps. The computational time would be shorter for a linear elastic analysis as no iterations are necessary in that case.

Pile Impedance

Dynamic impedances as functions of time were computed using the time-dependent nonlinear shear moduli from the PILE3D analysis. Harmonic loads with an amplitude of unity were applied at the pile head, and the resulting equations were solved to obtain the complex valued pile impedances. The impedances were evaluated at the ground surface.

The dynamic stiffness of the pile decreased dramatically as the level of shaking increases (Fig. 10). The dynamic stiffnesses experienced their lowest values between about 10 and 14 seconds, when the maximum accelerations occurred at the pile head. It can be seen that the lateral stiffness component K_{uu} decreased more than the rotational stiffness $K_{\theta\theta}$ or the coupled lateral-rotational stiffness $K_{u\theta}$. On the other hand the equivalent damping coefficients increased as the level of shaking increased because the hysteretic damping of the soil increased with the level of shaking.

At its lowest level, K_{uu} decreased to 22,000 kN/m, only 15.2 % of its initial stiffness of 145,000 kN/m. $K_{u\theta}$ decreased to 45,000 kN/rad or 36% of its initial stiffness of 125,000 kN/rad. $K_{\theta\theta}$ showed the least effect of shear strain. It decreased to 138,000 kN.m/rad or 63.6% of its initial stiffness of 217,000 kN.m/rad. The stiffnesses rebounded when the level of shaking decreased with time. Representative values of the pile stiffnesses K_{uu}, $K_{u\theta}$ and $K_{\theta\theta}$ for use in a structural analysis might be selected as 40,000 kN/m (31%), 65,000 kN/rad and 160,000 kN.m/rad, respectively. These stiffnesses are 27.6%, 52% and 73.7% of the original stiffnesses.

SEISMIC RESPONSE ANALYSIS OF A PILE GROUP

The seismic response of a 4-pile group in a centrifuge test was analyzed using the program PILE3D. The test set-up is shown in Fig. 11. The piles are set in a 2×2 arrangement at a centre to centre spacing of 2 diameters or 1.14 m. The properties of the piles are identical to those of the single pile described earlier. The group piles were rigidly clamped to a stiff

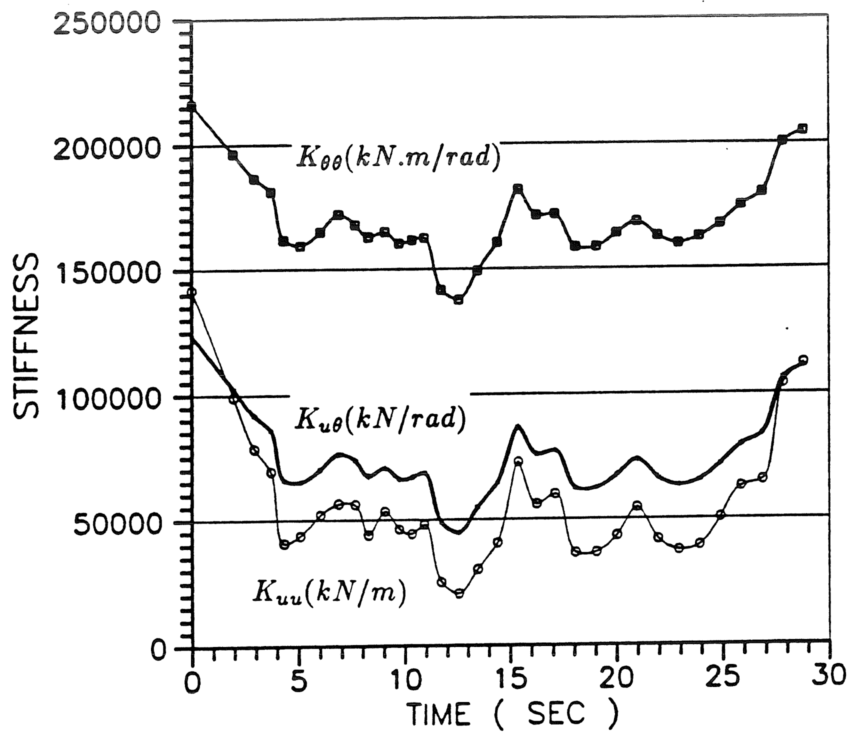


Fig. 10. Impedences K_{uu}, $K_{u\theta}$ and $K_{\theta\theta}$ of the single pile

pile cap and four cylindrical masses were bolted to the cap to simulate the inertia of a superstructure. The instrumentation on the pile cap assembly is also shown in Fig. 11. The displacements were tracked by means of an LED mounted on the pile cap assembly as in the case of the single pile test. The accelerations were recorded by an accelerometer mounted on the pile cap itself. The centre of mass of the total pile cap assembly is 0.96 m above the top of the pile cap and the free-standing pile length is 1.21 m (Gohl, 1991).

Under seismic excitation the pile cap rotation is resisted primarily by the rocking impedance of the pile group. This impedance cannot be included directly in the quasi-3D analysis because vertical and horizontal motions are uncoupled. Therefore, the following procedure was used to include the rocking stiffness.

At selected times during the horizontal mode analysis, the rocking stiffness and damping is computed using PILE3D in the vertical mode. This impedance calculation is made using the current values of strain dependent moduli and damping. The current rocking impedance is then transferred to the pile cap as rotational stiffness and damping. The accuracy of the representation of rocking impedance depends on the frequency with which it is updated.

The finite element mesh for the pile group analysis is shown in Fig. 12. Because of symmetry, the analysis can be conducted on half the pile group. This greatly reduces the

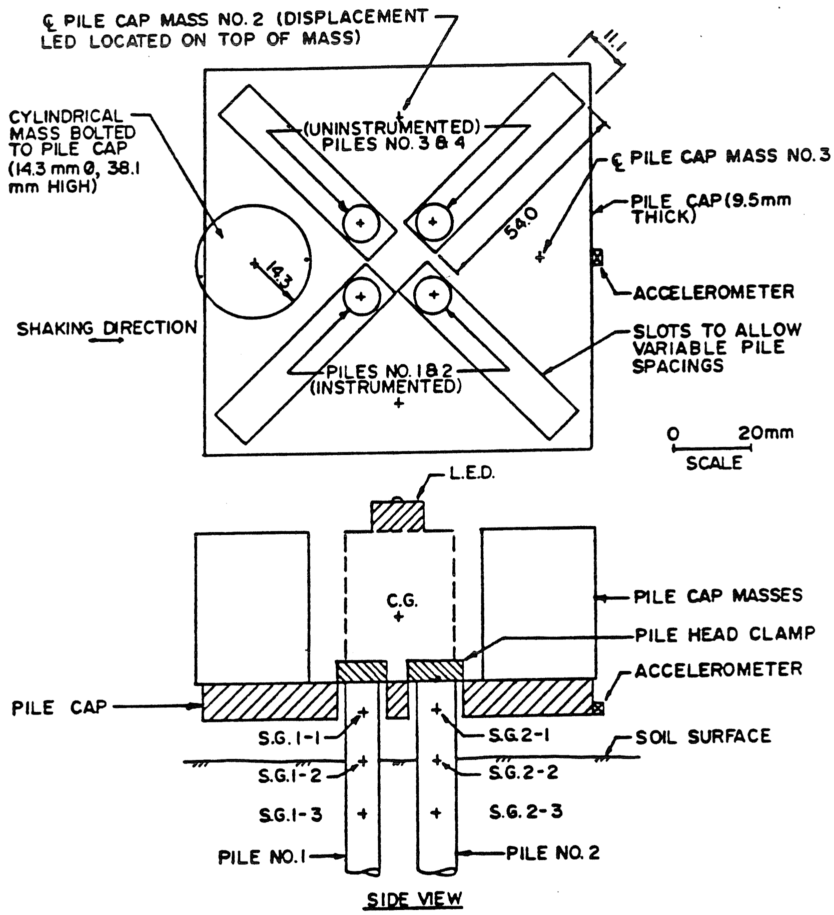


Fig. 11. Layout of centrifuge test for 4-pile group (after Gohl, 1991).

scope of the analysis. The mesh used in the present analysis has 946 nodes and 690 elements.

The pile group was shaken by an acceleration record with a peak acceleration of 0.14 g applied at the base of the soil layer. A peak acceleration of 0.24 g was recorded at the pile cap.

The computed acceleration response at the pile cap is plotted against the measured response in Fig. 13. There is fairly good agreement between the measured and the computed accelerations.

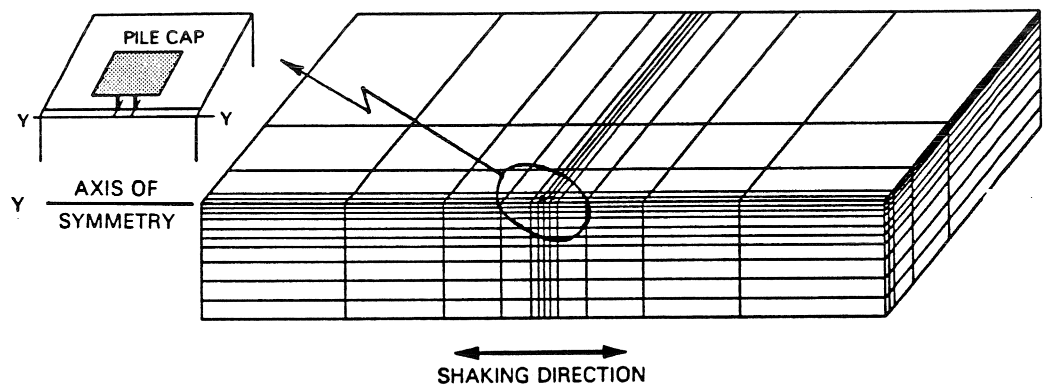


Fig. 12. Finite element model of centrifuge test on pile group.

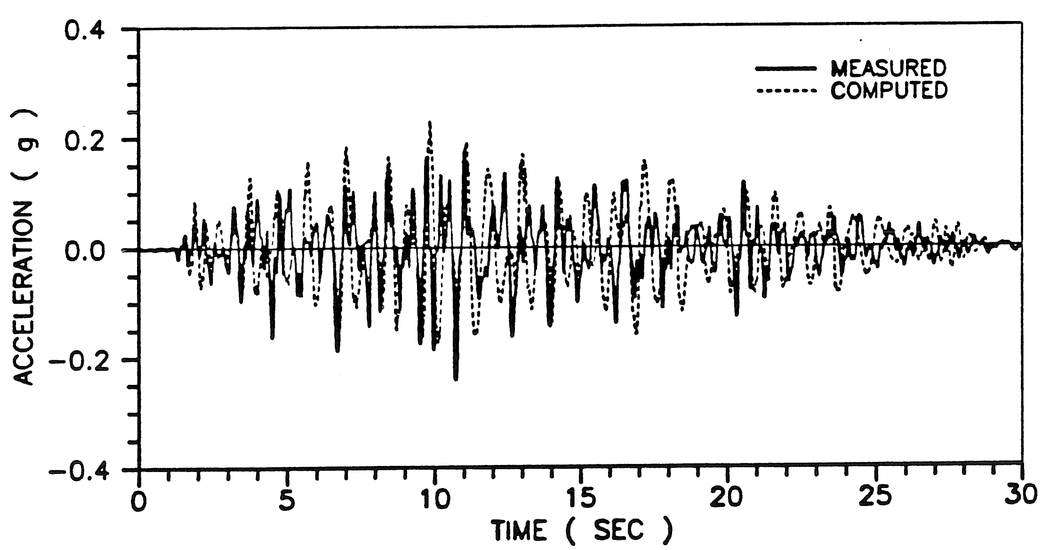


Fig. 13. The computed versus measured acceleration response at the pile cap.

The computed moment time history in the instrumented pile at a depth of 2.63 m corresponding to the location of the strain gauge in the area of maximum moment is plotted against the measured moment time history in Fig. 14. There is fairly good agreement

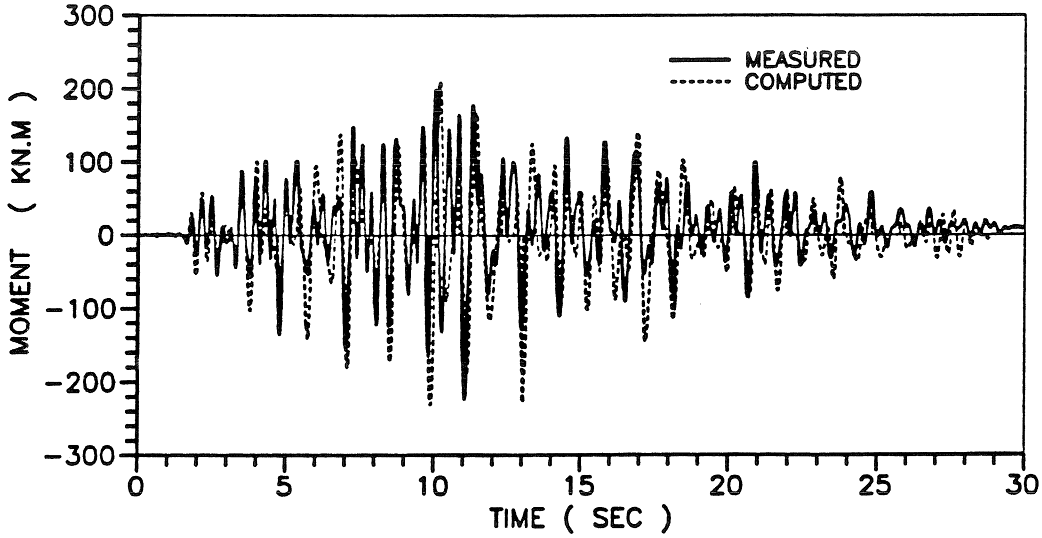


Fig. 14. The computed versus measured moment response at depth D = 2.63 m.

between the computed and measured moments. The distributions of computed and measured bending moments along the pile at the moment of peak pile cap displacement are shown in Fig. 15. The computed and measured moments agree reasonably well with the measured moments especially in the region of maximum moment.

ANALYSIS OF PILE REINFORCED SECTIONS OF SARDIS DAM

Previous studies have shown that the upstream section of Sardis Dam, Mississippi (Fig. 16), has the potential to undergo large deformations upstream along a layer of liquefiable clayey silt in the upper stratum clay should the design earthquake occur (Finn et al., 1991). The weak layer of clayey silt, 1.5 m (5 ft.) thick, is shown cross-hatched in Fig. 17. Due to liquefaction, the shear strength in this layer drops from an initial value in the region of 100 kPa (2000 psf) to a residual value of $S_{ur} = 0.075\ \sigma'_{vo}$ where σ'_{vo} is the effective overburden pressure. The method for restraining this deformation is to drive steel reinforced concrete piles through the upstream slope and the weak layer to an adequate penetration in the stronger layers below. In effect the upstream slope is nailed to the strong foundation layers (Fig. 17).

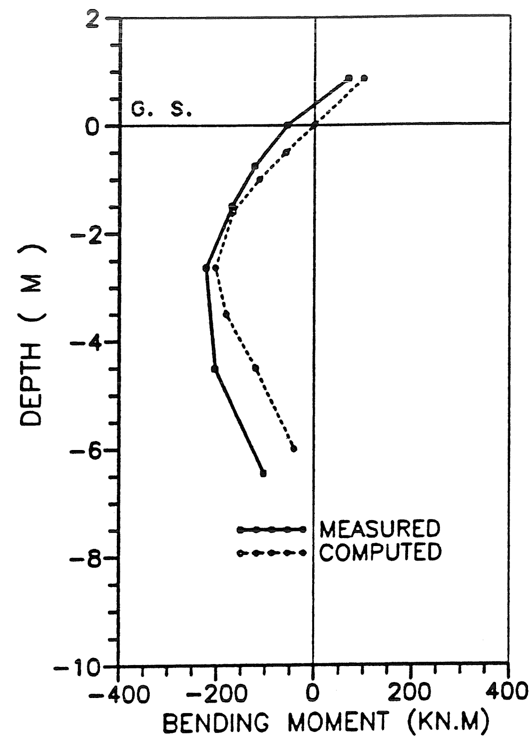


Fig. 15. The computed versus measured moment distributions along the pile at peak pile cap deflection.

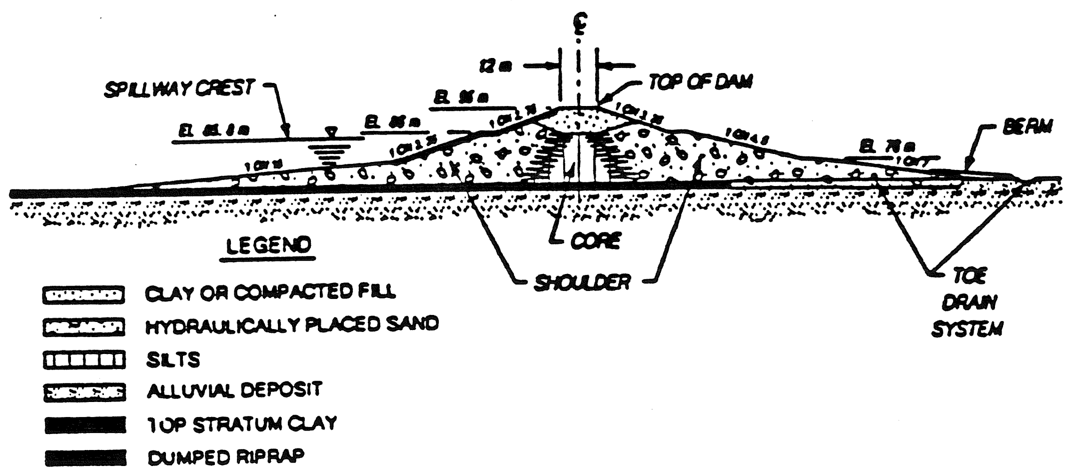


Fig. 16. Typical section of Sardis dam.

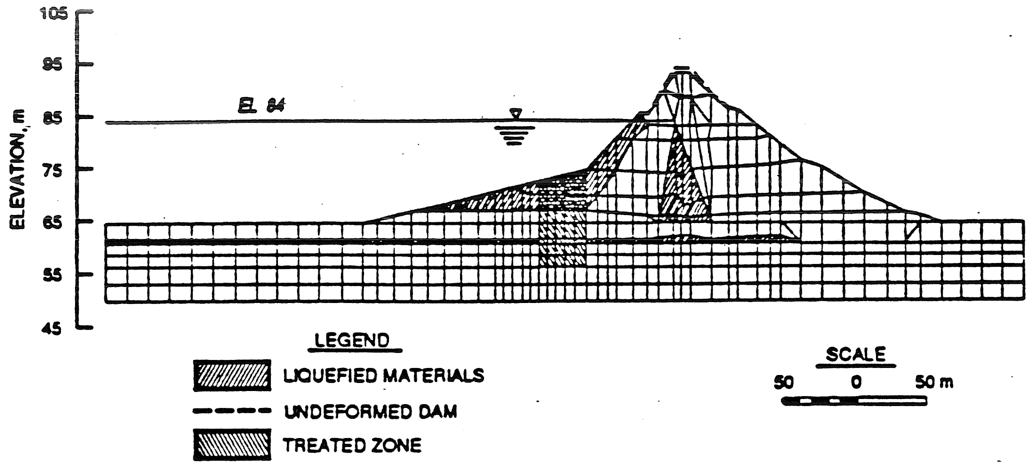


Fig. 17. Section of Sardis dam showing weak layer and restraining piles.

These piles must be designed to resist the combined effects of the pressures resulting from the restraint of the large deformations that might otherwise occur due to strength loss in the weak layer, and the moments and shears developed by seismic shaking during the design earthquake.

The key factors controlling the feasibility and cost of installing the pile to restrain the deformations are pile length and spacing, stiffness and strength of unliquefied soils surrounding the piles, residual strength of liquefied soils, the geometry of the structure, and the intensity of shaking before and after liquefaction has occurred. The ability to analyze such a complex problem while taking into account nonlinear behaviour of soil, potentially large strains in unremediated parts of the structure and a realistic interaction between piles and soil during both static and seismic loading is the essential requirement for determining the best location for the piles, an appropriate length and size and for categorizing the effects of soil properties.

Very little is known about the behaviour of piles under these complex conditions. Most of the evidence is from Japan where pile foundations have been severely damaged in liquefied ground as a result of ground displacements. However these piles were designed primarily for vertical static loading and both piles and the connections to the pile caps were inadequately designed to resist horizontal loading. Piles in Oakland Harbor were similarly damaged during the Loma Prieta Earthquake of 1989.

Detailed analytical studies were conducted to obtain data on the potential performance of piles resisting large upstream deformations in Sardis Dam and hence to provide a framework of understanding for the design of cost effective remediation using piles. These studies were conducted using the computer programs TARA-3 (Finn et al., 1986) and TARA-3FL (Finn and Yogendrakumar, 1989) which have proved useful in the analysis of flow deformations, settlements and the seismic response of earth structures and soil-structure interaction systems (Finn, 1988; Finn, 1994; Finn et al., 1994). The programs are not described here but detailed information is found in Finn (1988 and 1990).

The use of these analyses has allowed an acceptable displacement criterion to be used in defining satisfactory performance in addition to the common global factor of safety approach. This resulted in savings of several million dollars for the Sardis project. A problem with remediation measures designed solely on the basis of global factors of safety is that there is little understanding of what some of these factors mean in terms of displacements of the structure. Better and more cost-effective designs result from the implementation of Newmark's (1965) recommendation that the performance of earth structures be assessed using deformation criteria.

Analysis of Deformation Restraining Piles

A preliminary design option for restraining the flow deformations in Sardis Dam is to drive steel reinforced concrete piles, 0.6 m × 0.6 m (2 ft by 2 ft) in cross-section, through the upstream slope and the weak layer to a penetration of 4.6 m (15 ft) below the weak layer. The location for the piles shown in Fig. 17 was decided partly on analytical grounds and partly on practical considerations.

The piles are assumed to be at 3.7 m (12 ft) centres giving the plan layout shown in Fig. 18. Each row of piles normal to the axis of the dam is assumed to resist the potential movement of a plane strain section 3.8 m (12 ft) wide.

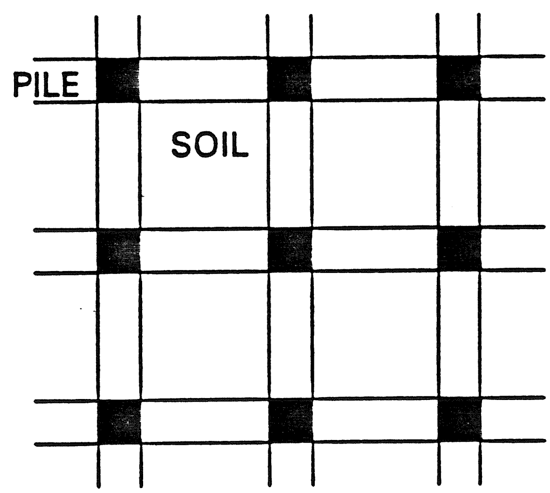


Fig. 18 Layout of restraining piles.

Analysis of pile response is conducted in two stages. First the effects of triggering residual strength in the layer of clayey silt are analyzed using the program TARA-3FL (Finn and Yogendrakumar, 1989). This analysis gives the moments and deflections in the piles arising from restraining the potential upstream movement of the embankment. Then the dam and piles, in the deformed condition, are subjected to earthquake shaking representative of design conditions using the program TARA-3 (Finn et al., 1986). This procedure for analysis simulates the condition where liquefaction and strength loss occur early in the earthquake. Subsequent studies confirm that this is the most conservative case.

Post-Liquefaction Deformation Analysis of Sardis Dam

The first step in the analysis is to construct the dam in layers to establish the initial stress conditions in the dam before remediation. Then the piles are inserted at the appropriate locations.

After liquefaction has occurred the initial stress state in the dam is no longer compatible with the stress-strain characteristics of the liquefied soils. The dam must therefore deform until a new equilibrium state is achieved. This involves very large strains in the unremediated section of Sardis Dam. To cope with these large strains, a Lagrangian formulation is used based on an adaptive mesh. When the upstream slope has been nailed to the foundation by piles, the inherent loss in resistance to lateral deformation along the weak layer of clayey silt is transformed to lateral load on the piles. The piles continue to deflect until an equilibrium state is achieved.

The distribution of lateral pressure on the lead pile after post-liquefaction equilibrium is reached is shown in Fig. 19.

The distribution of moments along the lead pile on the downstream side of the remediated zone is shown in Fig. 20. The peak moment is 1240 kNm (11,000 kip in), just above the weak layer.

The distributions of maximum shears and moments in piles along the remediated section are shown in Fig. 21 and Fig. 22, respectively. It is evident that the lead row of piles takes by far the largest moments and that these piles present the major challenge to effective design. The lead piles play such a dominant role because in the absence of a pile cap the only load transfer mechanism between the lead piles and the interior piles is the deflection of the lead piles towards the interior piles and the transmission of the associated increases in lateral pressures from one pile to another.

The deflection of piles in the leading row are shown in Fig. 23. The peak deflection is of the order of 0.075 m (3 inches).

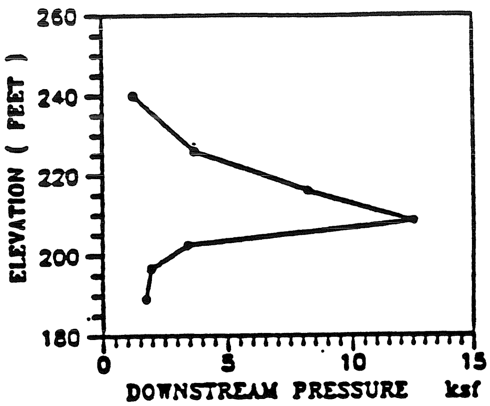


Fig. 19. Lateral pressures on lead piles after liquefaction (1 kip/ft^2 = 47.9 kPa; 1 ft = 0.3 m).

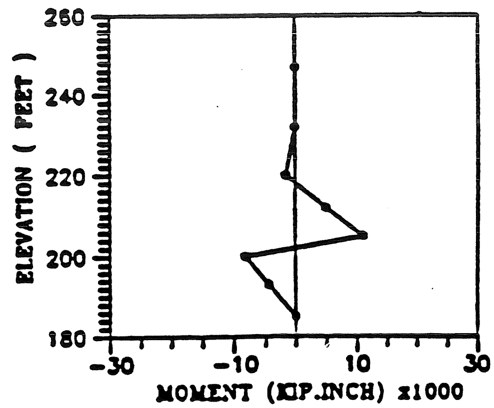


Fig. 20. Distribution of moments along lead piles after liquefaction (1 kip in = 113 Nm; 1 ft = 0.3 m).

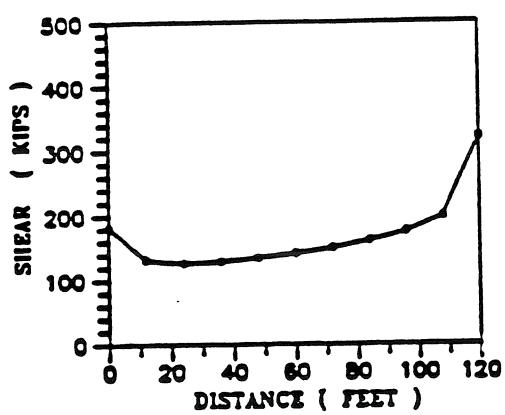


Fig. 21. Distribution of maximum shears in piles along remediated section (1 kip = 4.45 kN).

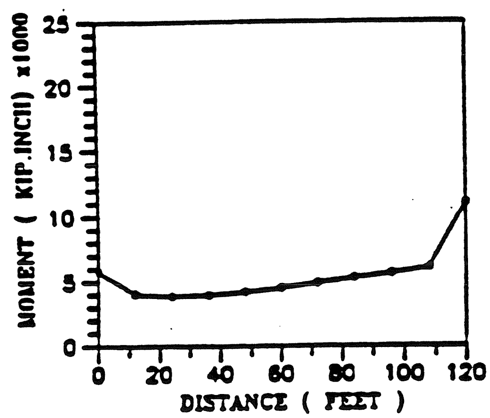


Fig. 22. Distribution of maximum moments in piles along remediated section (1 kip in = 113 Nm).

Seismic Analysis of Deformed Dam

The dynamic response of the remediated dam after triggering of residual strength by liquefaction was evaluated using the program TARA-3 (Finn et al., 1986). The input motions were the first 20 s of the S00E acceleration component of the 1940 El Centro earthquake scaled to 0.2 g.

The time history of dynamic moments in the leading row of piles at the top of the weak layer is shown in Fig. 24. The dynamic moments are plotted starting from the post-liquefaction moment. This plot therefore, gives the total moment at any instant during shaking. The peak moment occurring at t=2.2 s in the record is 1920 kNm (17,000 kip in) and the residual moment in the pile after the earthquake is about 1020 kNm (9,000 kip in).

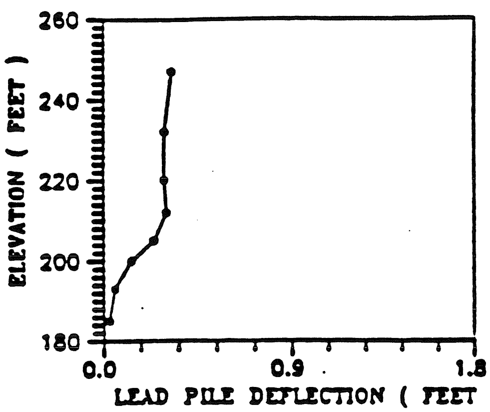


Fig. 23. Deflection pattern in the leading row of piles (1 ft = 0.3 m).

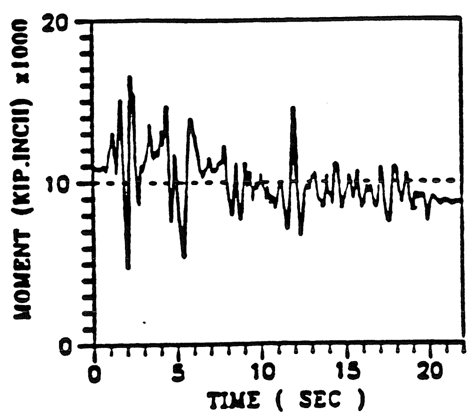


Fig. 24. Time history of total moment in the lead piles at top of weak layer (1 kip in = 113 Nm).

The time history of dynamic deformations at the top of the lead pile is shown in Fig. 25. The combined peak static and dynamic deflections of the lead pile upstream is of the order of 0.1 m (4 in).

The analyses demonstrated that potential large post-liquefaction deformations in Sardis Dam could be restrained. Both 2-D and 3-D parametric studies were conducted subsequently to optimize the location, design and spacing of the piles by nailing the upstream slope to the stable lower strata using piles. Pile nailing has now been adopted for stabilizing the upstream slope and construction is expected to start early in 1994.

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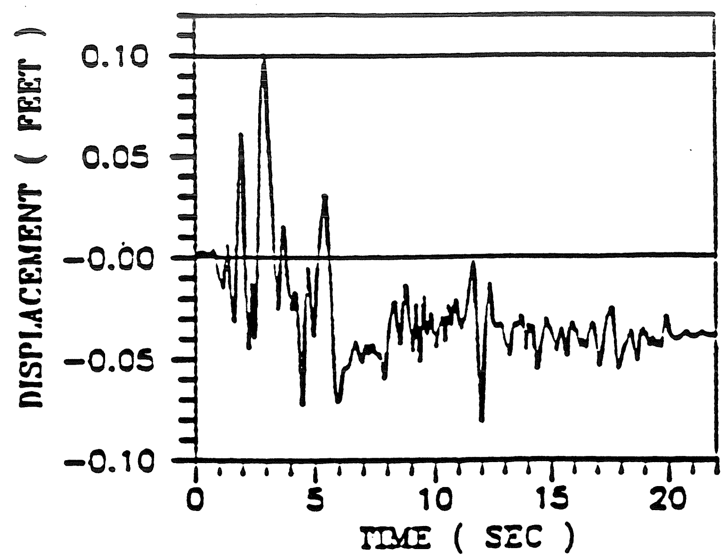

Fig. 25. Time history of deformations at the top of the lead piles (1 ft = 0.3 m).

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