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SEISMIC RESPONSE ANALYSIS OF PILE FOUNDATIONS AT LIQUEFIABLE SITES

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SUMMARY

Seismic response analyses were conducted on pile groups in liquefiable soils which were subjected to earthquake excitation on the large centrifuge at the University of California at Davis. The analyses were done using a simplified 3-D effective stress finite element program. Tests and analyses of a single pile and a 2×2 pile group subjected to an input acceleration record with a peak acceleration of 0.49 g are presented. There is good agreement between computed and measured free field porewater pressures, pile cap accelerations, time histories of moments and the distributions of maximum moments with depth

INTRODUCTION

In engineering practice, comprehensive methods for seismic response analysis of pile foundations are based primarily on linear elastic behaviour and use either boundary elements or finite element methods. Published graphs of the results are limited to relatively small pile groups because of the substantial computing time required. These methods do not take into account the nonlinear behaviour of soil under strong shaking. 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 shear moduli and arbitrary increases in viscous damping.

A new approach to 3-D nonlinear seismic response analysis of pile groups which removes some of the limitations of current methods has been developed at the University of British Columbia [Wu and Finn, 1994; Wu and Finn, 1997a,b]. It is based on a simplified 3-D representation of the foundation soils. By relaxing some of the boundary conditions associated with a full 3-D analysis, the computing time can be substantially reduced. The method is incorporated in the program PILE3D.

PILE3D analyzes the soil in terms of total stresses. The program has been modified to allow effective stress analysis by including a porewater pressure model to allow the generation of seismic porewater pressures due to shaking. The porewater pressure model is that developed by Martin, Finn and Seed [1975] but modified by adopting the two parameter model for volume change suggested by Byrne [1991]. During seismic response analysis, the soil properties are changed continuously to reflect the effects of the seismic porewater pressures on moduli and strength.

This paper describes validation of the modified program PILE3D [Wu, Finn and Thavaraj, 1998] using data from centrifuge tests on single piles and pile groups in liquefiable soils. These tests were run at the University of California at Davis and have been reported by Wilson [1995] and Wilson et al. [1997].

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SIMPLIFIED 3D SEISMIC ANALYSIS OF PILE FOUNDATIONS

The basic assumptions of the simplified 3D analysis are illustrated in Figure 1. Under vertically propagating shear waves 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. These compressional deformations generate shearing deformations in yOz plane. Therefore, the assumptions are made that dynamic response is dominated by the shear waves in the xOy and yOz planes and the compressional waves in the direction of shaking, Y. Deformations in the vertical direction and normal to the direction of shaking are neglected. Comparisons with full 3D elastic solutions confirm that these deformations are relatively unimportant for horizontal shaking [Wu and Finn, 1997a]. Applying dynamic equilibrium in Y-direction, the dynamic governing equation of the soil continuum in free vibration is written as

$$\rho_s \frac{\partial^2 v}{\partial t^2} = G^* \frac{\partial^2 v}{\partial x^2} + \theta G^* \frac{\partial^2 v}{\partial y^2} + G^* \frac{\partial^2 v}{\partial z^2} \tag{1}$$

where G^* is the complex modulus, v is the displacement in the direction of shaking, ρ_s is the mass density of soil, and θ is related to Poisson's ratio, μ of the soil. In this case, $\theta = (2-\mu)/(1-\mu)$.

Piles are modelled using ordinary Eulerian elastic 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 the soil.

The finite element code PILE3D-F [Wu, Finn and Thavaraj, 1998] incorporates these concepts of dynamic soil-pile-structure interaction. An 8-node brick element is used to represent the soil and a 2-node beam element is used to simulate the piles, as shown in Figure 1. The loss of energy due to radiation damping is modelled following the procedure proposed by Gazetas et al. [1993] in which a velocity proportional damping force per unit length is applied along the pile. The global dynamic equilibrium equation for the pile soil system is written in matrix form as

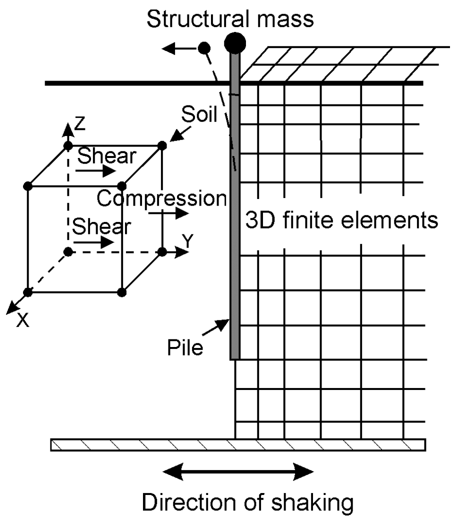

Figure 1. Quasi-3D model of pile-soil response.

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

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 acceleration, velocity and displacement, respectively.

Direct step-by-step integration using the Wilson-θ method is employed in PILE3D-F to solve the equations of motion in Equation (2). The non-linear hysteretic behaviour of soil is modelled by using an equivalent linear method in which properties were varied continuously as a function of soil strain. The seismic porewater pressures are generated continuously and their effects on soil properties taken into account. Additional features such as a 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.

CENTRIFUGE TESTS

Dynamic centrifuge tests of pile supported structures in liquefiable sand were performed on the large centrifuge at University of California at Davis, California. The models consisted of two structures supported by single piles, one structure supported by a 2x2 pile group and one structure supported by a 3x3 pile group. The typical arrangement of structures and instrumentation is shown in Figure 2. Full details of the centrifuge tests can be found in Wilson et al. [1997]. Only the single pile system (SP1) and the (2x2) pile group (GP1) are studied here.

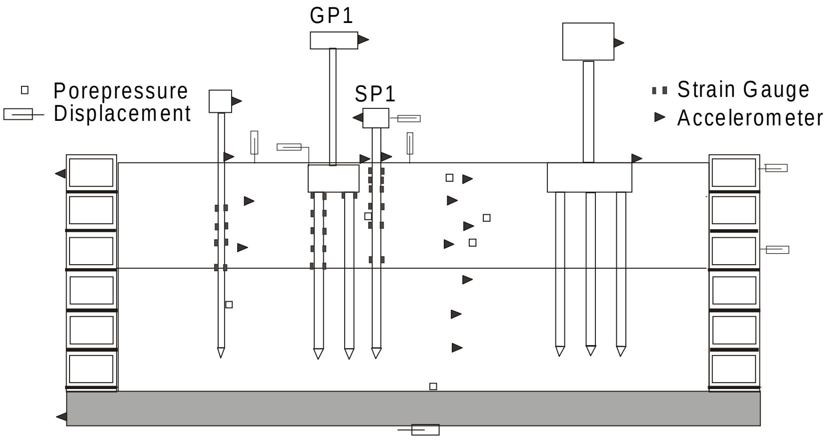

Figure 2. Layout of models for centrifuge tests.

The model dimensions and the arrangement of bending strain gauges in systems SP1 and GP1 are shown in Figs. 3 and 4, respectively. In the (2x2) group pile system GP1, the mass and the length of the column that carries the superstructures mass are 233 Mg and 10.9 m respectively. Model tests were performed at a centrifugal acceleration of 30g.

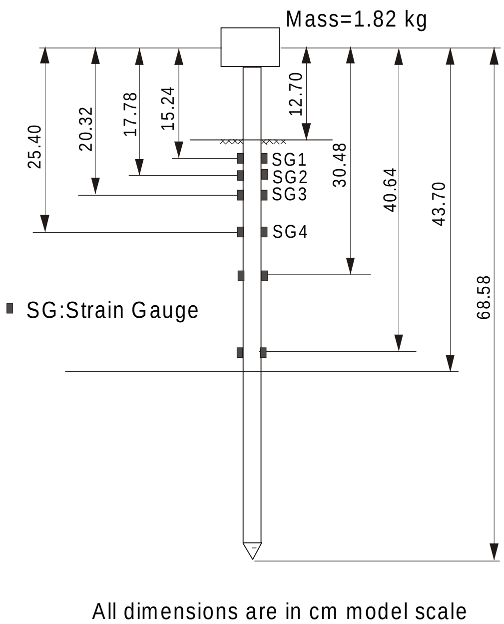

Figure 3. Instrumented pile for single pile test.

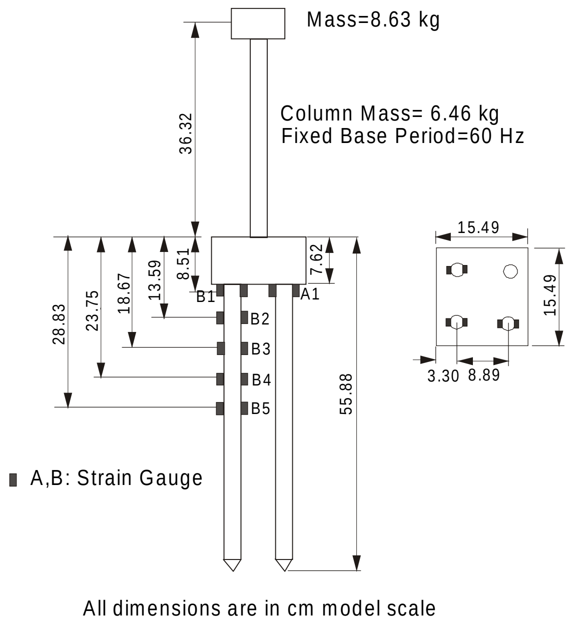

Figure 4. Instrumented test piles and details of superstructure for 2×2 pile group.

The soil profile consists of two level layers of Nevada sand, each approximately 10m thick at prototype scale. Nevada sand is a uniformly graded fine sand with a coefficient of uniformity of 1.5 and mean grain size of 0.15 mm. Sand was air pluviated to relative densities of 75%-80% in the lower layer and 55% in the upper layer. Prior to saturation, any entrapped air was carefully removed. The container was then filled with a hydroxy-propyl methyl-cellulose and water mixture under vacuum. The viscosity of this pore fluid is about ten times greater than pure water to ensure proper scaling. Saturation was confirmed by measuring the compressive wave velocity from the top to the bottom of the soil profile.

The responses of the single pile and the 2×2 pile group to the Santa Cruz acceleration record obtained during the 1989 Loma Prieta earthquake, scaled to 0.49 g is described and analyzed here.

EFFECTIVE STRESS DYNAMIC ANALYSIS OF SINGLE PILE

Finite Element Model of the Single Pile-Superstructure System

The finite element mesh used in the analysis is shown in Figure 5. The finite element model consists of 1649 nodes and 1200 soil elements. The upper sand layer which is 9.1 m thick was divided into 11 layers and the lower sand layer which is 11.4 m thick was divided into 9 layers. The single pile was modeled with 28 beam elements. 17 beam elements were within the soil strata and 11 elements were used to model the free standing length of the pile above the soil. The superstructure mass was treated as a rigid body and its motion is represented by a concentrated mass at the center of gravity. A rigid beam element was used to connect the superstructure to the pile head.

Soil and Pile Properties

The small strain shear moduli G_{max}, were estimated using the formula proposed by Seed and Idriss [1970].

$$G_{max} = 21.7\, k_{max}\, P_a \left(\frac{\sigma'_m}{P_a} \right) \tag{3}$$

in which k_{max} is a constant which depends on the relative density of the soil, σ'_m is the initial mean effective stress and P_a is the atmospheric pressure. The constant k_{max} was estimated using the approximation suggested by Byrne [1991]. The program PILE3D-F accounts for the changes in shear moduli and damping ratios due to dynamic shear strains at the end of each time increment. The shear strain dependencies of the shear modulus and damping ratio of the soil were defined by the curves suggested by Seed and Idriss [1970] for sand, shown in Figure 6. The friction angles of the upper and the lower layers were taken as 35° and 40°, respectively.

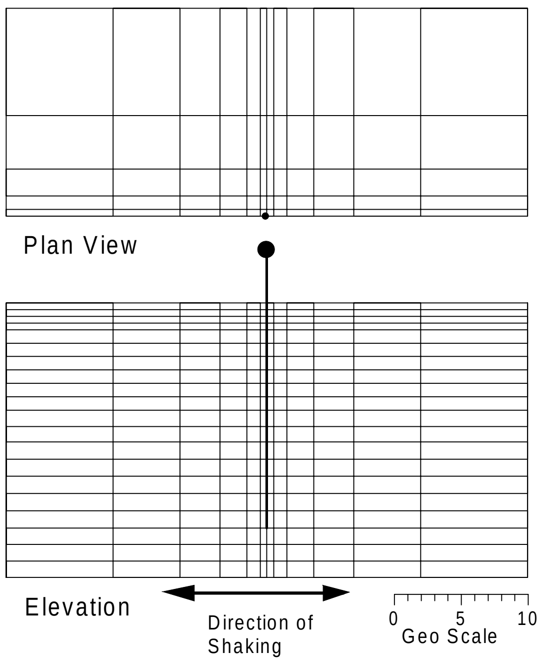

Figure 5. Finite element mesh for single pile.

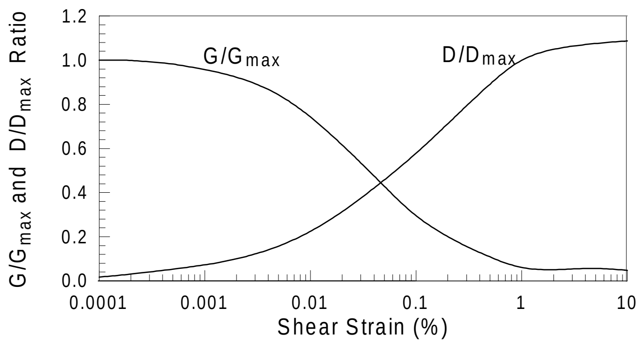

Figure 6. Variation of shear modulus and damping ratio with shear strain used in the analysis.

Porewater Pressure Effects

The seismic porewater pressures were generated in each individual element depending on the current volumetric strain prevailing in that element. The soil properties, the moduli and the strength were modified continuously to account for the effects of the changing seismic porewater pressures.

Earthquake Input Motion

The Santa Cruz acceleration record was scaled to 0.49 g and used as input to the shake table. The base accelerations of the model were measured at the east and west ends of the base of the model container. Wilson et al.[1995] showed that both accelerations agreed very well.

The base input acceleration is shown in Figure 7.

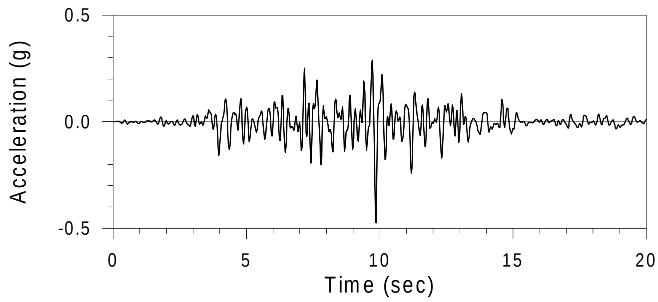

Figure 7. Input acceleration time history.

RESULTS OF SINGLE PILE ANALYSIS

Porewater Pressure Response
Figure 8 shows comparisons between measured and computed porewater pressures at three different depths; 1.14 m, 4.56 m, and 6.78 m in the free field. There is generally good agreement between the measured and computed porewater pressure responses.

Bending Moment Response
Figure 9 shows the measured and computed bending moment time histories at two different depths; 0.76 m and 1.52 m. Generally there is a very good agreement between the measured and computed time histories. Figure 10 shows the profiles of measured and computed maximum bending moments with depth. The comparison between measured and computed moments is fairly good, although the maximum moment is overestimated by 10% to 20% between 1 m and 4 m depths.

ANALYSIS OF 2×2 PILE GROUP

Effective stress analyses were also carried out to simulate the response of the (2x2) pile group- superstructure system. The finite element mesh is similar in type to that in Figure 5 except for the presence of the pile cap.

The pile cap was modeled with 16 brick elements and treated as rigid body. The superstructure mass was treated as a rigid body and its motion was represented by a concentrated mass at the center of gravity. The column carrying the superstructure mass was modeled using beam elements and is treated as a linear elastic structure. As the stiffness of this column element was not reported, it was calculated based on the fixed base frequency of the superstructure reported by Wilson et al. [1997] as 2 Hz.

RESULTS OF (2×2) GROUP PILE ANALYSIS

Acceleration Response
Figure 12 shows computed and measured pile cap acceleration time histories. There is a good agreement between the measured and computed values.

Bending Moment Response

Figure 13 shows time histories of measured and computed moments at a depth of 2.55 m. The measured and computed time histories compare quite well. Residual moments were removed from the time history of

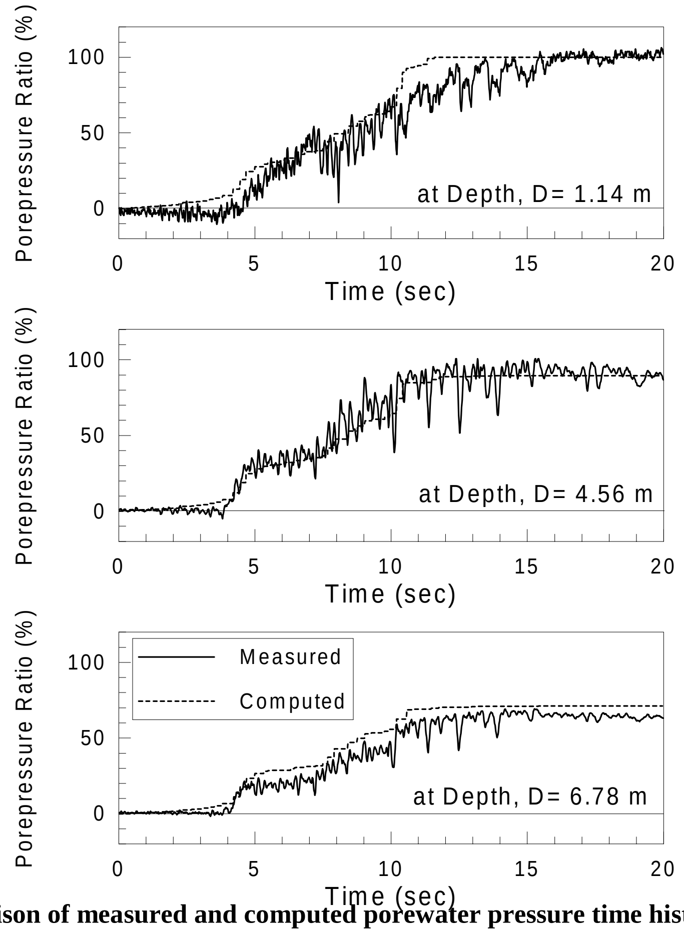

Figure 8. Comparison of measured and computed porewater pressure time histories at three depths.

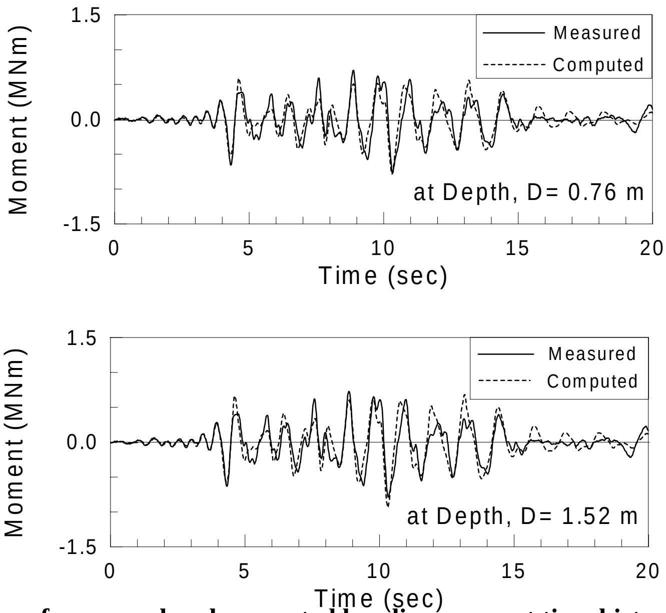

Figure 9. Comparison of measured and computed bending moment time histories at two depths.

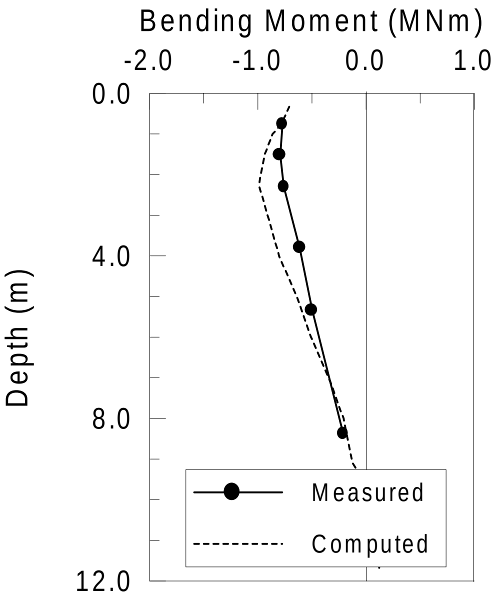

Figure 10. Comparison of measured and computed maximum bending moments profiles along the pile.

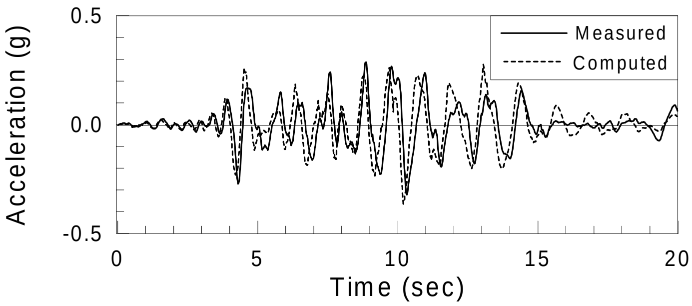

Figure 11. Comparison of measured and computed superstructure acceleration time histories.

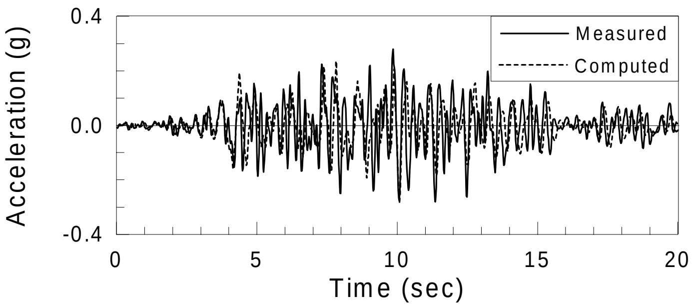

Figure 12. Comparison of measured and computed pile cap acceleration time histories.

measured moments before the comparison was made. Figure 14 shows the measured and computed bending moment profiles with depth. They also compare very well.

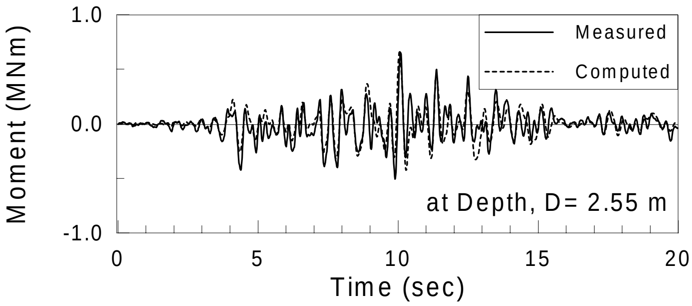

Figure 13. Comparison of measured and computed bending moment time histories.

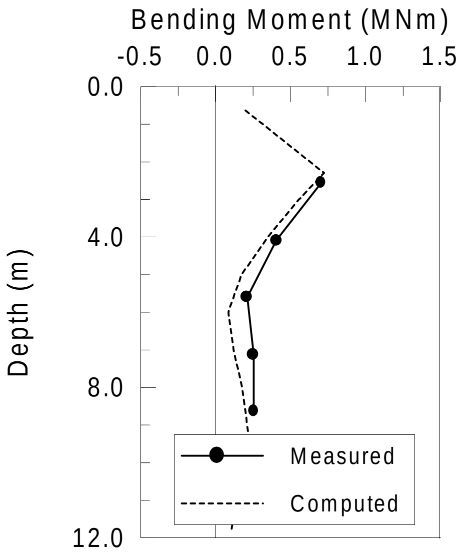

Figure 14. Comparison of measured and computed maximum bending moments along the pile.

CONCLUSION

The preliminary simulation studies conducted on the University of California centrifuge tests so far, including the two representative tests reported here, suggest that the program PILE3D-F has the capability to analyze pile foundations in potentially liquefiable soils with sufficient accuracy for engineering purposes.

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