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NONLINEAR SEISMIC ANALYSIS OF PILE FOUNDATIONS

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ABSTRACT

Procedures are presented for the dynamic effective stress analysis of pile groups including nonlinear stress-strain response. The pile group is analyzed as a unit and not as single piles with interaction factors. The procedures are validated using the data from centrifuge tests on a single pile and the 2×2 pile group. An interesting output of the analysis is the time variation of all the impedance factors to reflect the effects of nonlinearity.

KEYWORDS

3-D nonlinear seismic analysis; dynamic pile group-structure interaction; centrifuge tests; time dependent stiffness time dependent damping; inertial interaction

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.

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 new approach to the nonlinear analysis of pile groups, currently under development at UBC, will be described here (Wu, 1994). It 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.

QUASI-3D ANALYSIS OF PILES

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. Deformations in the vertical direction and normal to the direction of shaking are neglected. Comparisons with full 3-D elastic solutions confirm that these deformations are relatively unimportant for horizontal shaking. Applying dynamic equilibrium in Y-direction, the dynamic governing equation under free vibration of the soil continuum is written as

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

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.

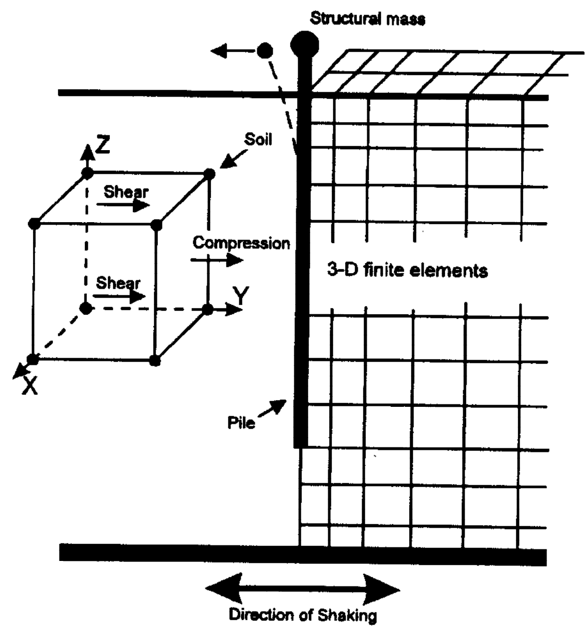

Fig. 1. Quasi-3D model of pile-soil response.

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.

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

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

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 PILE-3D 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.

SEISMIC RESPONSE ANALYSIS OF A SINGLE PILE

PILE-3D 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). An average centrifuge acceleration of 60g was used in the test. A horizontal acceleration record with a peak acceleration of 0.158g is input at the base of the system. Details of the test may be found in a paper by Finn and Gohl (1987). Figure 2 shows the model soil-pile-structure systems used in the test.

The centrifuge test was analyzed by the quasi-3D finite element method of analysis using the program PILE-3D. Figure 3 shows the finite element model used for analysis. The mesh has 666 nodes and 456 elements. The sand 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.

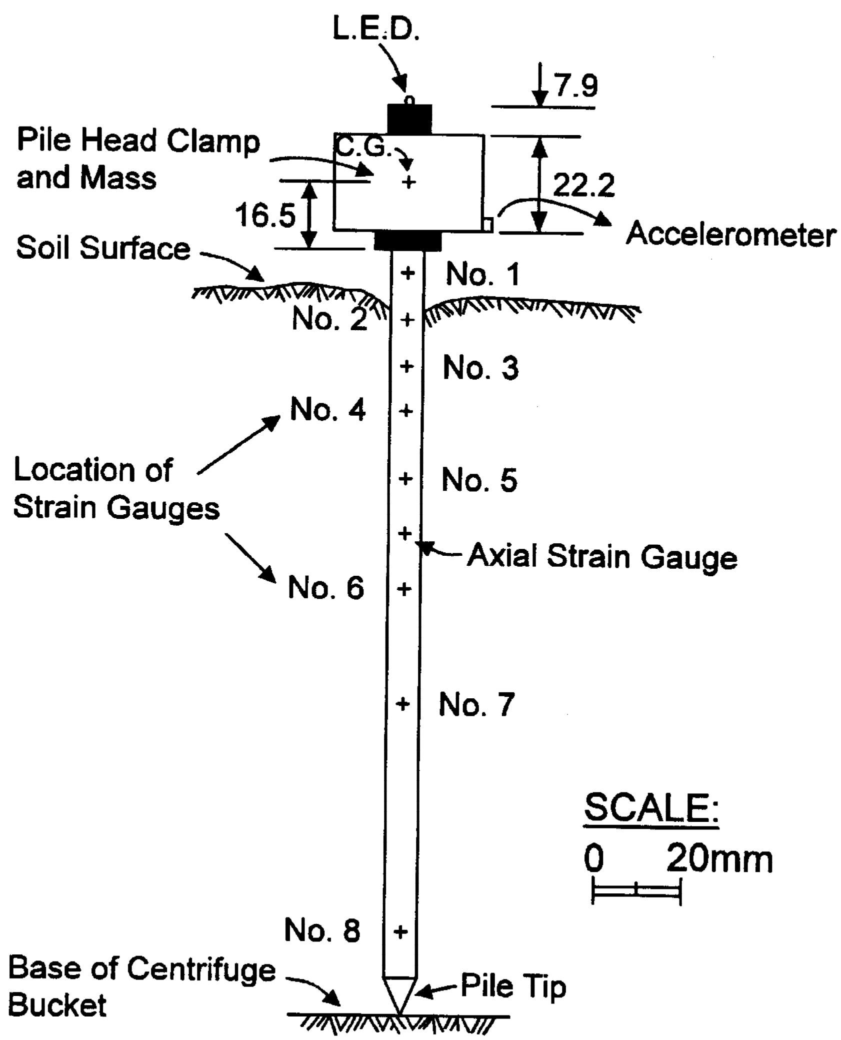

Fig. 2. Model instrumented pile used in centrifuge test.

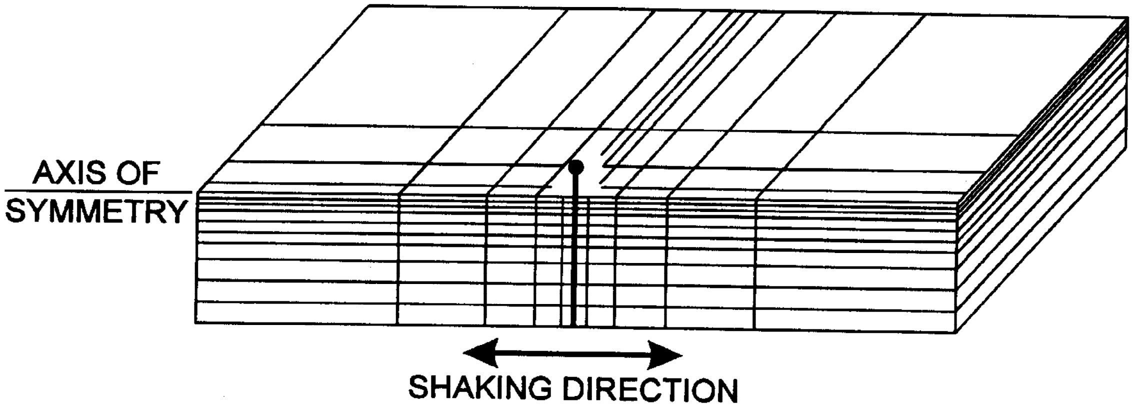

Fig. 3. Finite element mesh used for analysis of single pile response.

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 computed acceleration response at the pile head is plotted against the measured response in Fig. 4. Fairly good agreement between the measured and the computed accelerations is observed in the region of strong shaking.

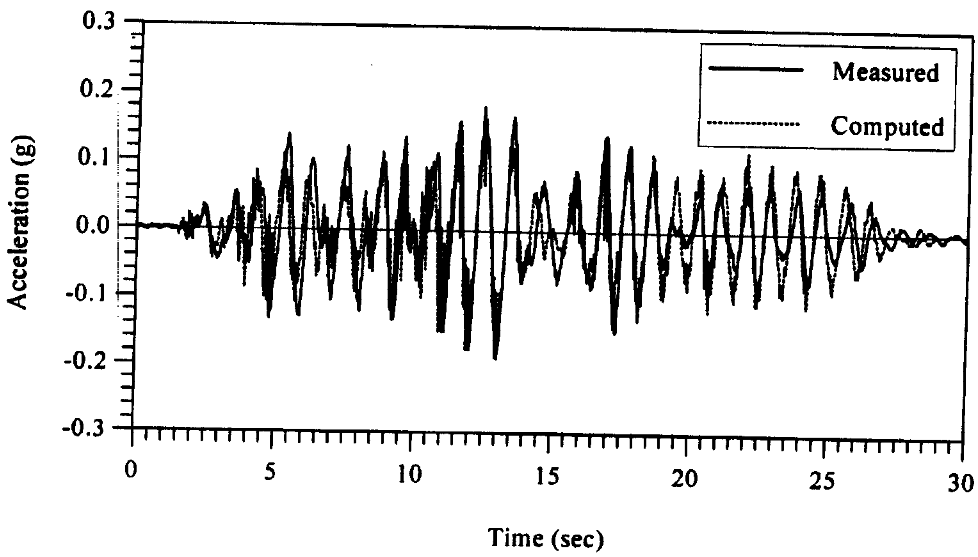

Fig. 4. The computed versus measured acceleration response at the pile head.

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. 5. 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 instant of peak pile head deflection are shown in Fig. 6.

Pile Impedance

Dynamic impedances as functions of time were computed using the time-dependent shear moduli from the PILE-3D analysis. The distribution of these moduli around the pile at a depth of 2.10 m at time t = 12.58s is shown in Fig. 7. Harmonic loads with an amplitude of unity were applied at the pile head, and the

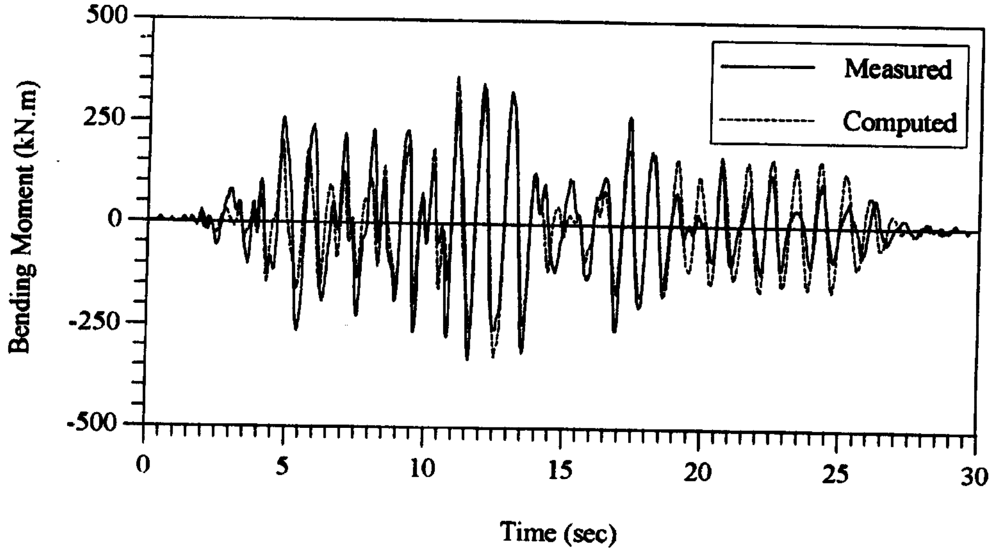

Fig. 5. The computed versus measured moment response at depth D = 3 m.

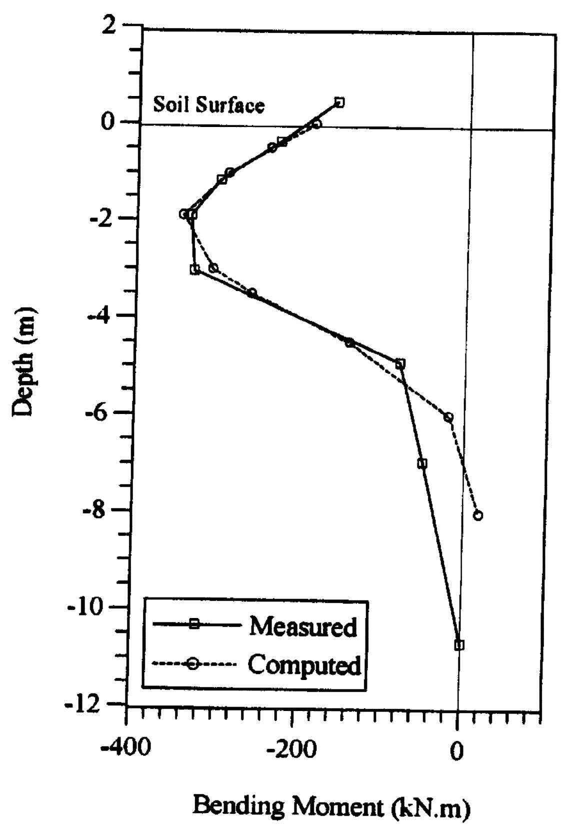

Fig. 6. Comparison between measured and computed bending moments at peak pile deflection.

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 (real part of the impedance) decreased dramatically as the level of shaking increases (Fig. 8). The dynamic stiffnesses experienced their lowest values between about 10 and 14

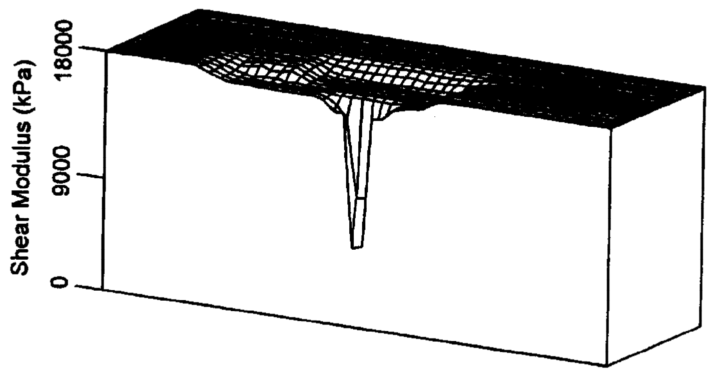

Fig. 7. Distribution of shear moduli around pile at a depth of 2.10 m at t = 12.58 s.

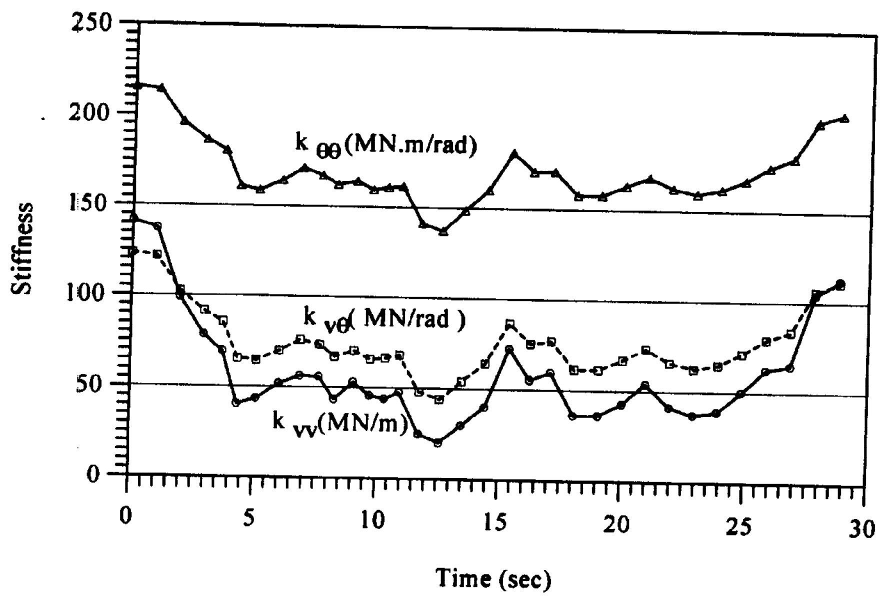

Fig. 8. Stiffnesses K_{vv}, $K_{v\theta}$ and $K_{\theta\theta}$ of the single pile.

seconds, when the maximum accelerations occurred at the pile head. It can be seen that the lateral stiffness component K_{vv} decreased more than the rotational stiffness $K_{\theta\theta}$ or the coupled lateral-rotational stiffness $K_{v\theta}$. 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.

To demonstrate the effect of inertial interaction, a single pile was analyzed, with and without a structural mass at the pile head. The variations in lateral stiffness, with and without inertial interaction, are shown in Fig. 9. In this case, inertial interaction has a major effect on pile stiffness. It is clear that inertial interaction should be considered when evaluating pile foundation stiffness under strong earthquake shaking.

SEISMIC RESPONSE ANALYSIS OF A PILE GROUP

The seismic response of a 4-pile group in a centrifuge test (Gohl, 1991; Finn and Gohl, 1987) was analyzed using the program PILE-3D. The piles are set in a 2×2 arrangement at a centre to centre spacing of 2 pile diameters. The properties of the piles are identical to those of the single pile described earlier. The group piles were rigidly clamped to a stiff pile cap and four cylindrical masses were bolted to the cap to simulate the inertia of a superstructure.

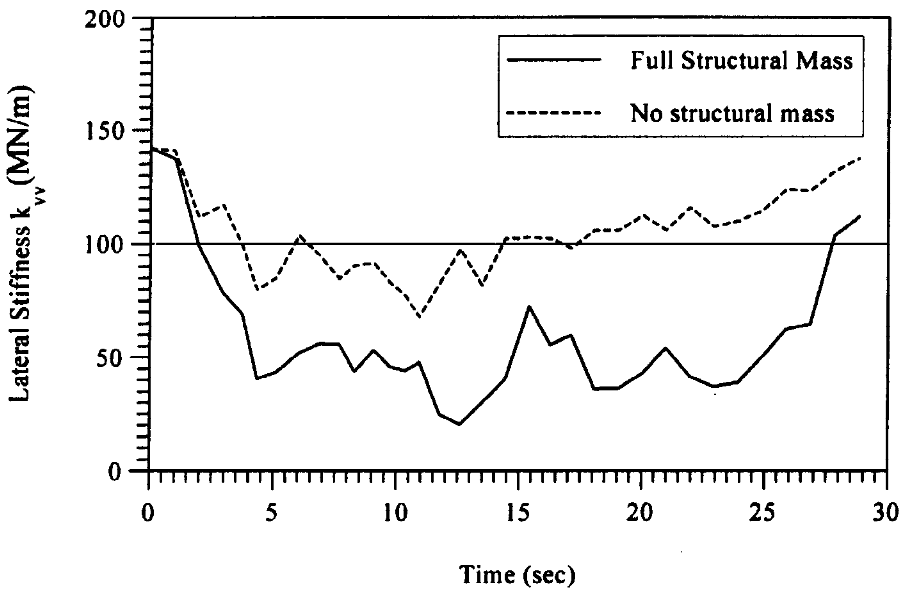

Fig. 9. The effect of inertial interaction between super-structure and pile foundation on pile stiffness.

At selected times during the horizontal mode analysis, the rocking stiffness and damping is computed using PILE-3D 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 distributions of computed and measured bending moments along the pile at the instant of peak pile cap displacement are shown in Fig. 10. The computed and measured moments agree reasonably well with the measured moments especially in the region of maximum moment.

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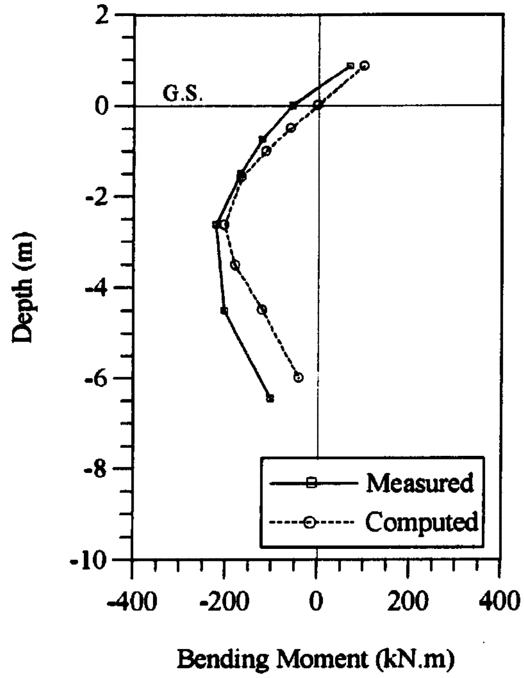

Fig. 10. Comparison between measured and computed bending moments at peak pile cap displacement.

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