# Field Test on Group Piles under Machine Induced Coupled Vibration

**Sanjit Biswas**, *Research Scholar, Department of Civil Engineering, Indian Institute of Technology Delhi, New Delhi-110016, India; email: sanjit.jal@gmail.com*
**Shiva Shankar Choudhary**, *Research Scholar, Department of Civil Engineering, Indian Institute of Technology Delhi, New Delhi-110016, India; email: shv.snkr@gmail.com*
**Bappaditya Manna**, *Assistant Professor, Department of Civil Engineering, Indian Institute of Technology Delhi, New Delhi-110016, India; email: bmanna@civil.iitd.ac.in*
**Dilip Kumar Baidya**, *Professor, Department of Civil Engineering, Indian Institute of Technology Kharagpur, West Bengal - 721302, India; email: baidya@civil.iitkgp.ernet.in*

**ABSTRACT**: *Dynamic response characteristics of reinforced concrete group piles with embedded pile-cap condition are investigated in the field under varying levels of coupled harmonic excitations. The piles are constructed by bored cast-in-situ method. The site is located at Indian Institute of Technology Kharagpur, West Bengal, India. The soil properties are determined by laboratory tests on both disturbed and undisturbed soil samples collected from various boreholes at the site. Two in-situ tests, namely, standard penetration tests to determine N-value and cross hole seismic tests for determining the shear-wave velocity of soil are conducted at various depths of soil layer. Forced coupled vibration tests are conducted on pile groups using Lazan type mechanical oscillator with four different eccentric moments. Both the horizontal and rocking motions of pile groups are measured simultaneously for different operating frequencies of oscillator. Two resonant peaks are observed at two different frequencies. It is also observed that as the eccentric moment increases, the resonant amplitude increases, but the natural frequency decreases for both horizontal and rocking responses. The test results on piles are then compared with the numerical approach with two different soil-pile models – (i) A linear visco-elastic medium composed of outer infinite region and an inner weaker layer with reduced shear modulus, and (ii) a boundary zone with parabolic variation of shear modulus and with a non-reflective interface. From the comparison between the test results and numerical results using the first model it is found that the prediction of nonlinear response of the pile foundation is in good agreement with the test results. For the second model, the numerical results are not found satisfactory when compared to the test results. The nonlinear parameters like separation length, shear modulus ratio, weak zone damping factor and thickness ratio are also predicted for the group piles under coupled vibration.*

**KEYWORDS:** dynamic, group pile, coupled vibration, nonlinear, continuum approach, dynamic field test.

**SITE LOCATION:** IJGCH-database.kmz (requires Google Earth)

## INTRODUCTION

With the increasing popularity and demand of pile foundations for various civil engineering projects different analytical methods have been developed for the prediction of dynamic response of pile foundations. To verify the efficiency of these analytical methods, the experimental investigations are always necessary as the dynamic test data are a more reliable and unavoidable component. Due to the obvious difficulty and associated cost, dynamic testing on full-scale pile groups is rarely done in practice. On the other hand small-scale testing on model pile group is a reasonably good option for checking the usefulness and applicability of various available approaches for dynamic soil-structure interaction.

Field tests with small prototype piles are less demanding than full scale tests. In terms of equipment, cost, and effort, the field tests with small prototype piles are comparatively easier to numerically simulate and control the test. Novak and Grigg (1976) conducted dynamic tests with small pile foundations in the field and the test data were compared with theoretical predictions proposed by Novak (1974). El Sharnouby and Novak (1984) conducted a series of forced vibration

Submitted: 04 May 2013; Published: 22 August 2013
Reference: Biswas, S., Choudhary, S.S., Manna, B., and Baidya, D.K.. (2013). *Field Test on Group Piles under Machine Induced Coupled Vibration.* International Journal of Geoengineering Case histories, http://casehistories.geoengineer.org, Vol.3, Issue 1, p.10-23. doi: 10.4417/IJGCH-03-01-02

tests with a group of 102 closely spaced piles for vertical, horizontal and torsional modes separately. To compare the nonlinear response with the theoretical curves calculated using continuum approach, Han and Novak (1988) conducted dynamic tests with large scale model piles subjected to strong horizontal and vertical excitation in the field. Burr et al. (1997) performed the dynamic tests on 13 model pile groups at two separate sites to assess the effects of the spacing to diameter ratio on the dynamic response of the pile groups.

Among all the methods available in the literature one of the most established and widely used approach is the continuum approach to predict the dynamic response of a pile foundation. The elastic continuum method was mainly based on a closed form solution of Mindlin (1936). The complex dynamic soil stiffness of the composite pile-soil medium was first described by Novak et al. (1978) assuming plane strain condition. To calculate impedance functions (stiffness and damping), an approximate analytical solution was also introduced by Novak and Aboul-Ella (1978a, 1978b). To consider the nonlinear effect of soil, Novak and Sheta (1980) proposed a cylindrical boundary zone around the pile with lesser value of soil modulus and greater damping value relative to the outer region. Later the soil mass effects of the boundary-zone were also investigated by Novak and Han (1990). Then Han and Sabin (1995) proposed a model of ideal boundary zone with non-reflective interface. Another boundary zone model is also proposed by Han (1997) with non-reflective interface with parabolic variation of shear modulus. Manna (2009) performed tests in the field on model group pile and compared the test results with the analysis using continuum approach. Finally an empirical relationship to estimate the boundary-zone parameters and the extent of soil-pile separation was established.

From the literature review it is found that the prediction of the boundary zone parameters and the soil-pile separation lengths under coupled vibration are the fundamental parameters to predict the nonlinear response of soil-pile system which has not been studied in detail so far. Hence in this study, a coupled vibration test is performed on pile groups to determine the nonlinear responses of piles in layered soil medium. Then the continuum approach is used to predict the dynamic nonlinear response of the pile foundations obtained from the dynamic field tests of piles. The variation of stiffness and damping of pile group with operating frequencies of oscillator are studied. The boundary zone parameters and soil-pile separation lengths are also presented.

## SITE AND SOIL CONDITIONS

The investigation site is situated at Indian Institute of Technology, Kharagpur Campus, India. Both in-situ soil investigation and laboratory tesing is conducted to determine the soil properties and site characteristics. Three boreholes are explored at different locations of the site up to a depth of 3.0 m to collect disturbed as well as undisturbed soil samples for laboratory testing.

### Determination of Soil Profile and Soil Properties

The standard penetration test (SPT) is performed in all three boreholes at different depths (Bureau of Indian Standards 1981). The laboratory experiments such as natural moisture content (Bureau of Indian Standards 1973), specific gravity (Bureau of Indian Standards 1980a,b), Atterberg's limits test (Bureau of Indian Standards 1985b, 1972), particle size distribution analysis of soil (Bureau of Indian Standards 1985a), and unconsolidated undrained triaxial test (Bureau of Indian Standards 1993) are carried out in the soil testing laboratory. After compailation of all the field and laboratory test results it is found that the test site consists of three different soil layers as per Unified Soil Classification System. The soil profile consists of 1.20 m of soft yellow organic silty clay with low plasticity overlying a 1.10-m thick layer of brown medium stiff inorganic clay with low to medium plasticity and after the depth of 2.30 m a highly plastic red stiff inorganic clay mixed with gravel is found and extended up to the depth of 3.0 m. The different soil profiles and the uncorrected SPT-*N* value with depth of different soil strata are presented in Figure 1. The avarage soil properties of all three layers determined from laboratory tests are also presented in Table 1.

### In-situ Dynamic Soil Properties

To determine the dynamic shear modulus of soil, seismic crosshole tests (ASTM D 4428/D 4428M, 2000) are conducted in the field at every 0.5 m interval depth. Three boreholes (A, B and C) are drilled spaced 3.0 m apart, center-to-center on the ground surface. Two PVC (polyvinyl-chloride) pipes of inside diameter of 75 mm are inserted into the boreholes after drilling up to the desired depth. To capture the waves, accelerometers (B&K Type 4507) are attached at the bottom inner wall of each PVC pipe. These accelerometers are connected to the recording system (B&K Pulse 6.1, Type 3560c Sound and Vibration Meter). SPT hammer with a special wave generating device is used in one borehole (Borehole A) to generate

S-wave impulse energy source. Two receivers are placed at the same elevation in each of the designated boreholes (B and C). Typical variation of S-wave arrival with time at two boreholes of seismic crosshole tests at the depth of 1.50 m is shown in Figure 2. The variation of shear wave velocity and shear modulus of soil with depth is also shown in Figure. 3.

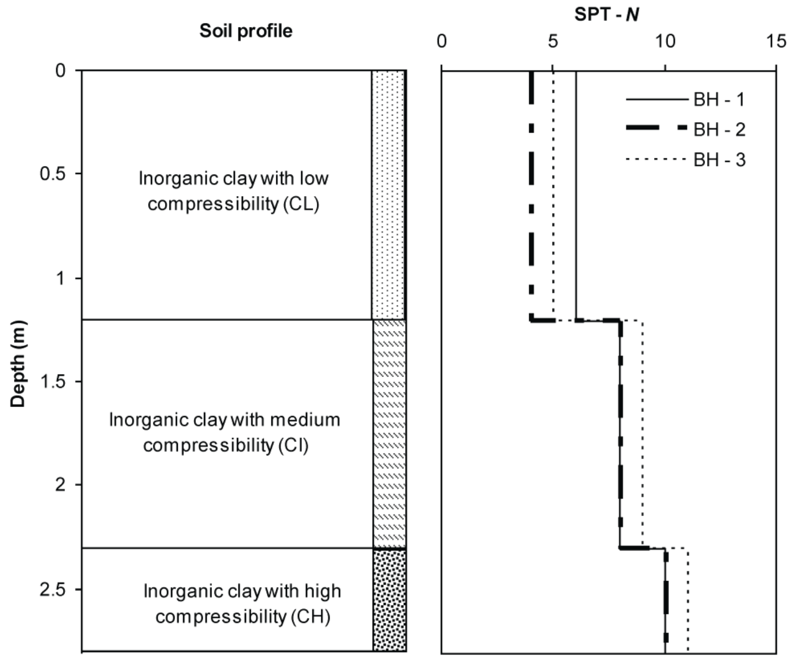


*Figure 1. Soil Profiles and Uncorrected SPT-N Value With Depth.*

*Table 1. Average Properties of All The Soil Layers*

| Property of soil | Layer 1 (0.00–1.20 m) | Layer 2 (1.20–2.30 m) | Layer 3 (below 2.30 m) |
|---|---|---|---|
| Moisture content (%) | 12.42 | 14.33 | 12.20 |
| Bulk density (kN/m$^3$) | 15.12 | 15.70 | 15.64 |
| Specific gravity | 2.57 | 2.61 | 2.60 |
| Liquid limit (%) | 33.60 | 49.33 | 51.35 |
| Plastic limit (%) | 13.80 | 15.01 | 17.38 |
| Shrinkage limit (%) | 12.76 | 14.21 | 12.93 |
| Cohesion (kPa) | 30 | 31 | 28 |
| Friction angle (deg) | 21 | 23 | 24 |

## CONSTRUCTION OF TEST PILE GROUPS

The bored cast-in situ piles are constructed in the field for the testing. A borehole is made using an auger of diameter 0.1 m up to a depth equal to the length of the pile. After boring, a reinforcement cage is placed into the hole. In each pile, three 8 mm diameter longitudinal bars are provided. Transverse reinforcement of 6 mm diameter in the form of circular ring at the spacing of 150 mm center to center is provided to support the longitudinal bars of the pile. The concrete cover of 20 mm is maintained for the construction of piles. Then the borehole is filled up to the neck of the longitudinal reinforcement of the pile by pouring the concrete (grade of concrete M25, as per Bureau of Indian Standards 2000) into hole with proper tampering. For the pile cap, a 8 mm diameter bar at the spacing of 90 mm center to center is provided at both directions with a clear cover of 100 mm. All the reinforcement used for construction of piles and pile caps are Fe 415 grade (characteristic strength is 415 N/mm$^2$ as per Bureau of Indian Standards 1985c).

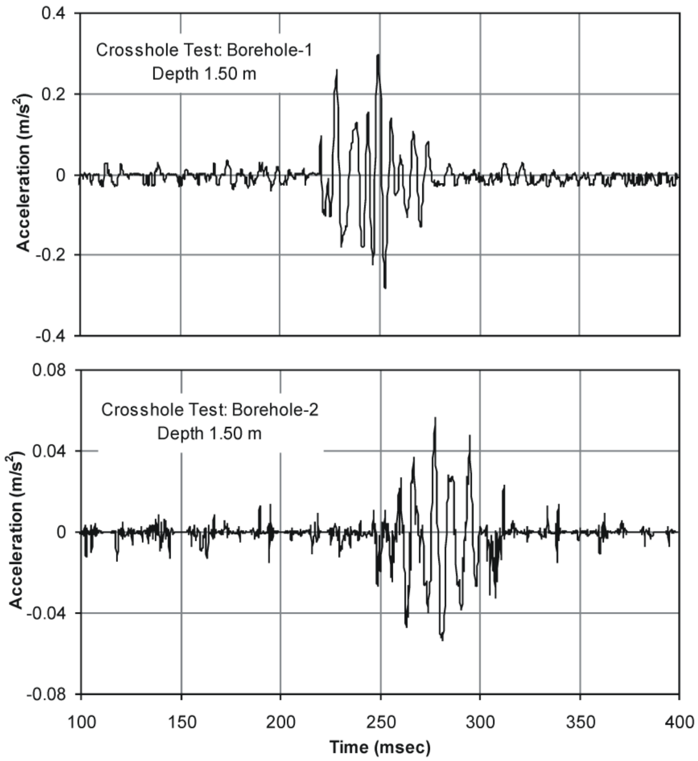


*Figure 2. Variation of S-Wave Arrival with Time at Two Boreholes of Seismic Crosshole Tests.*

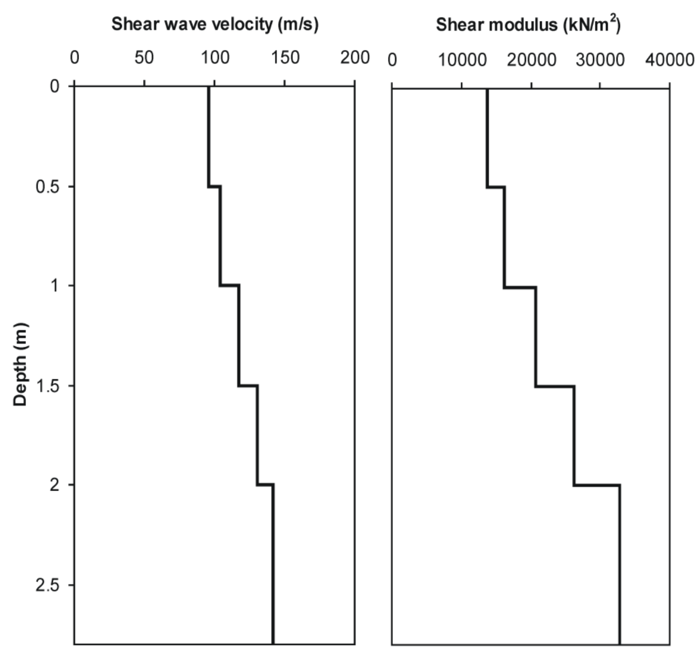


*Figure 3. Variation of Shear Wave Velocity And Shear Modulus of Soil With Depth.*

For monolithic action of pile cap and piles, the vertical bars of the piles are rigidly tightened with the reinforcement of the pile cap. The dimension of pile caps is 0.57 m × 0.57 m × 0.25 m and the pile cap is embedded up to 0.175 m into the soil. Three sets of 2 × 2 pile groups (length $L$ = 2m, spacing $s$= 2$d$, 3$d$, 4$d$and diameter $d$ = 0.1 m) are constructed for the investigation. In order to connect the pile cap to the loading system, four foundation bolts are attached on the top of pile cap. The stages for pile installation in the field are shown in Figure 4(a) through (e). To verify the shape of the pile a model pile is taken out of ground after one month as shown in Figure 4(f). The shape and size of the pile was found to be as intended (straight and circular).

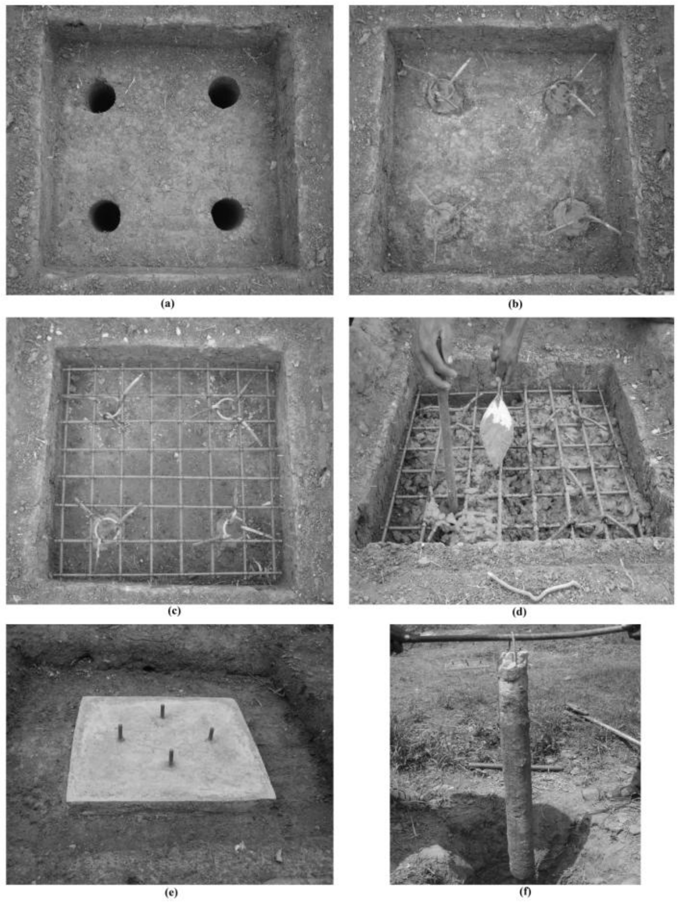


*Figure 4. Stages of Construction Process of Pile Groups.*

## COUPLED VIBRATION TEST AND TEST RESULTS

Coupled forced vibration tests are conducted on each pile group in the field. The mechanical oscillator (Lazan type) with two counterrotating eccentric masses is used to produce the harmonic excitation force on pile foundation. When one of them is driven by a motor, the arrangement induces unidirectional sinusoidal vibratory force passing through the center of gravity of the oscillator. The working principle of LAZAN type mechanical oscillator is shown in Figure 5. The magnitude of the exciting force is controlled by adjusting the angle ($\theta$) of the eccentric mass. When angle $\theta$ is set to a value, the mass on the shaft can generate an eccentric moment (*me*) and its value is given by

$$me = \frac{w}{g} e = \frac{0.9 \sin(\theta/2)}{g} \ \text{Nsec}^2 \tag{1}$$

The dynamic force ($P$) at any frequency that is proportional to the square of the excitation frequency can be expressed as

$$P = me\, \omega^2 \sin \omega t = \frac{0.9 \sin(\theta/2)}{g} \omega^2 \sin \omega t \quad \text{N} \tag{2}$$

where $W$ and $m$ are the weight and mass of eccentric rotating part in oscillator respectively, $e$ is the eccentric distance of the rotating masses, $g$ is the acceleration due to gravity, $\omega$ is the circular frequency of vibration and $t$ is the time.

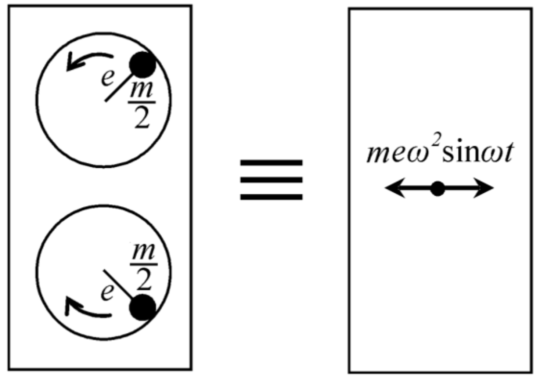


*Figure 5. The Working Principle of LAZAN Type Mechanical Oscillator.*

**Coupled Vibration Test Setup**

Forced vibration tests are carried out on small prototype piles under coupled vibration. The mechanical oscillator is first mounted on the pile cap by foundation bolts to provide the vibrating force. Then a number of mild steel ingots or test bodies are placed on the top of the oscillator to provide the desired static weight. The test body is comprised of steel ingots each weighing 650N (8 numbers) and 450N (10 numbers). The vibrating mass of the system are adjusted using these test bodies attached to the pile cap. The whole setup is properly connected with the foundation bolts of the pile cap in such a way that it acts as a single unit.

The unidirectional horizontal excitation force from the oscillator causes both horizontal and rocking motions of the pile foundation because of the distance between the center of oscillator where horizontal forces are generated and center of gravity (C.G.) of the entire loading arrangement. The oscillator is connected by means of a flexible shaft with a direct current (DC) Motor. The speed of DC Motor is controlled by a speed control unit. The vibration measuring system consisted of two piezoelectric acceleration pickups and a compatible vibration meter. The horizontal component is measured using one pickup connected to the side of the foundation at the level of center of gravity. While the rocking amplitudes were measured simultaneously by another pickup mounted vertically on the axis of the pile cap at a known distance from the center of projected pile cap center on the top of the loading system. The complete experimental setup of the coupled vibration test is shown in Figure 6.

Steady state dynamic response of the pile-soil system is measured for different eccentric moments ($W \cdot e$ = 0.187, 0.278, 0.366, and 0.450 Nm) for horizontal and rocking mode separately under the static load ($W_s$) of 12 kN. The oscillator is run slowly in a controlled manner through a motor using a speed control unit to avoid sudden application of high magnitude dynamic load up to a frequency of 50 Hz. The frequency and the corresponding amplitude of vibration are recorded by a photo tachometer and vibration meter, respectively.

**Test Results**

The frequency-amplitude response curves for the coupled mode of vibration are obtained from the test for different ecentric intensities. A typical frequency versus amplitude response curves for the group pile ($L/d$ = 20, $s/d$ = 3, and $W_s$ = 12 kN) are shown in Figure 7a and 7b for horizontal and rocking vibrations respectively. Two distinct resonant peaks are observed at two different frequencies for both horizontal and rocking responses which indicate the behaviour of a two-degree of freedom (couple) system. The first resonant peak of the coupled response is dominated by the horizontal translation as the rocking component is quite small, but completely opposite behaviour is observed for the second peak. It can be seen that the observed response curves display nonlinearity as the resonant frequencies decreases with increasing excitation intensity. Also it is observed that the amplitudes are not proportional to the excitation intensity indicating nonlinearity in the response. The experimental resonant frequencies and amplitudes of pile groups for horizontal and rocking components are summarized in Table 2. The variations of natural frequency and resonant amplitude are observed for different $s/d$ ratios of pile groups and it is found that the natural frequency increases and the resonant amplitude decreases with increasing pile spacing.

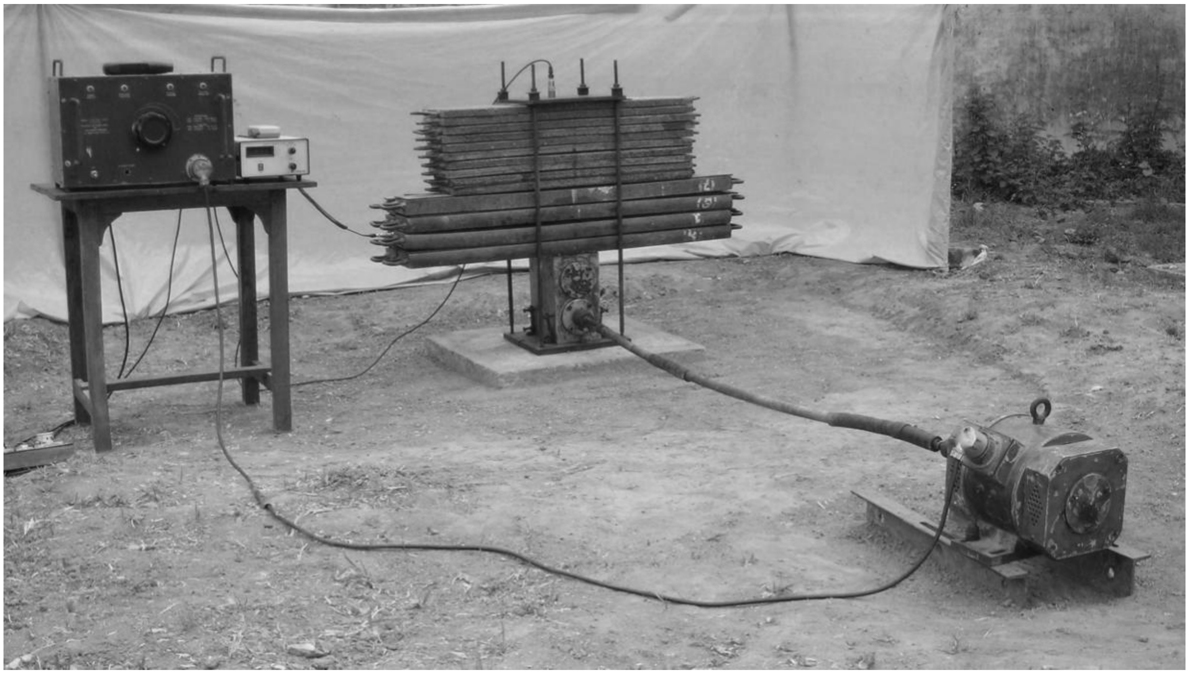

Figure 6. Experimental Setup of Coupled Vibration Test.

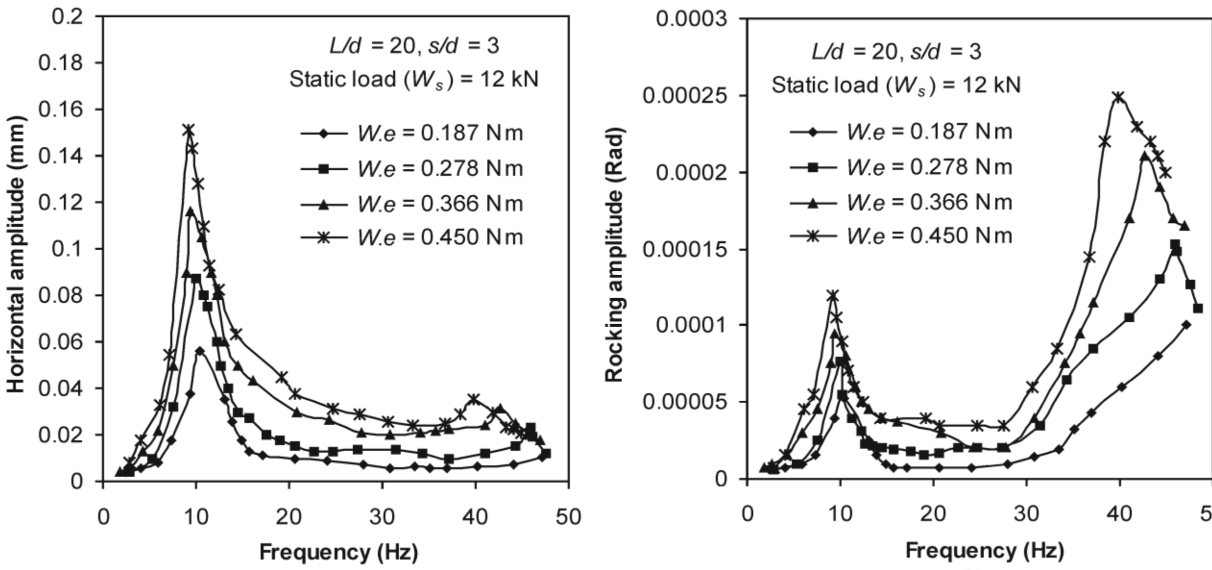


Figure 7. Response Curves obtained from Experiment [(a) Horizontal And (b) Rocking Mode].

**CONTINUUM APPROACH**

To determine the dynamic response of the pile groups in layered soil media Novak's continuum approach is used in this study. This theory is described by Novak and Aboul-Ella (1978a, b) for the calculation of the impedance function of the single pile in composite medium. The stiffness and damping are calculated based on the assumption that the soil is perfectly bonded to the pile and the soil behavior is governed by the laws of linear elasticity. The complex stiffness of pile groups in different modes of vibration is calculated by using the methodology given by Kaynia and Kausel (1982) and Novak and Mitwally (1990). With the stiffness and damping of the pile group, the horizontal and rocking response of the piles are calculated.

*Table 2. Coupled Vibration Test Results of (2 x 2) Group Piles (L= 2 m, d= 0.1m, Ws = 12 kN).*

| Eccentric moment (N m) | Pile cap embedded into the soil ($h$ = 0.175 m) | | | | | |
|---|---|---|---|---|---|---|
| | $f_{n1}$* (Hz) | $A_{H1\text{-res}}$** (mm) | $\psi_{r1\text{-res}}$*** (Rad) | $f_{n2}$* (Hz) | $A_{H2\text{-res}}$** (mm) | $\psi_{r2\text{-res}}$*** (Rad) |
| $L/d$ = 20, $s/d$ = 2 | | | | | | |
| 0.187 | 8.50 | 0.066 | 0.000046 | 43.55 | 0.017 | 0.000126 |
| 0.278 | 8.08 | 0.101 | 0.000077 | 41.23 | 0.033 | 0.000178 |
| 0.366 | 7.66 | 0.133 | 0.000112 | 38.86 | 0.025 | 0.000252 |
| 0.450 | 7.21 | 0.160 | 0.000130 | 37.00 | 0.044 | 0.000310 |
| $L/d$ = 20, $s/d$ = 3 | | | | | | |
| 0.187 | 10.50 | 0.056 | 0.000053 | - | - | - |
| 0.278 | 9.95 | 0.087 | 0.000076 | 45.83 | 0.023 | 0.000153 |
| 0.366 | 9.43 | 0.116 | 0.000095 | 42.68 | 0.031 | 0.000210 |
| 0.450 | 9.21 | 0.151 | 0.000119 | 39.85 | 0.035 | 0.000249 |
| $L/d$ = 20, $s/d$ = 4 | | | | | | |
| 0.187 | 11.68 | 0.051 | 0.000042 | - | - | - |
| 0.278 | 11.10 | 0.086 | 0.000059 | - | - | - |
| 0.366 | 10.43 | 0.112 | 0.000078 | - | - | - |
| 0.450 | 9.81 | 0.149 | 0.000091 | 45.61 | 0.030 | 0.000210 |

* $f_{n1}$, $f_{n2}$ = first and second resonant frequencies
** $A_{H1\text{-res}}$, $A_{H2\text{-res}}$ = first and second resonant amplitudes for horizontal motion
*** $\psi_{r1\text{-res}}$, $\psi_{r2\text{-res}}$ = first and second resonant amplitudes for rocking motion

The effect of nonlinearity and slippage is taken into account by considering a linear viscoelastic medium composed of two parts: an outer infinite region and an inner weak boundary zone around the pile with reduced soil modulus and higher soil damping as compared to the outer zone. Based on the energy dissipation of the composite medium through wave propagation, the complex dynamic stiffness is calculated. The soil reactions of the composite medium are introduced into dynamic analysis of pile foundations in place of the homogeneous medium without any other modifications in theory. In this study, two types of available soil models are used to analyze the nonlinear behavior of pile foundations.

- Novak and Sheta (1980): In this model a linear visco-elastic medium composed of outer infinite region and an inner weaker layer with lower shear modulus and higher damping factor as compared to the outer zone. This soil model with continuum approach is available in the computer software package DYNA 5 (Novak et al 1999).

- Han (1997): This is a boundary zone model with non-reflective interface between outer infinite region and inner weaker zone with a parabolic variation of shear modulus from the pile outer surface to the boundary of the outer zone. This type of soil model with continuum approach is programmed in a software form named DYNAN (ENSOFT INC, 2004).

The Poisson's ratio (μ) is considered 0.35 for all soils. With the value of shear modulus (G) and Poisson's ratio (μ), the elastic modulus (E) is calculated using the equation $E = G/2(1+\mu)$ for each layer.

**Theoretical Results and Prediction of Nonlinear Parameters**

To predict the actual material nonlinear properties of this linear-elastic based mathematical model (Novak's continuum approach) some nonlinear parameters like modulus reduction factor ($G_m/G$), weak zone soil damping ($D_m$), thickness ratio ($t_m/R$) and most importantly separation length are incorporated based on the literature review to predict the nonlinear frequency versus amplitude response. For different excitation intensities, the soil parameter in the weakened zone are adjusted so that the nonlinear theoretical response curves approached the dynamic test results. The variations of weak zone parameters with depth for different excitation levels are shown in Figure 8. An approximate step-variation trend is assumed for the nonlinear parameters which follows approximately a parabolic variation with the depth. It can be noted that as the excitation intensity increases, the shear modulus ratio ($G_m/G$) reduces, whereas the thickness ratio ($t_m/R$) and weak zone soil damping ($D_m$) increases. The values of $G_m/G$ are increased with depth but the $t_m/R$ and $D_m$ are decreased with depth for all excitation level. It can be seen that the value of shear modulus is very small up to the depth of 0.5 m from the ground level. However, the thickness and damping of the weak zone are significantly higher in top soil layers as the top portion of soil plays a significant role on the dynamic response of the pile under horizontal translation. In this analysis, 40% of weak zone mass is added to the pile for all excitation intensities.

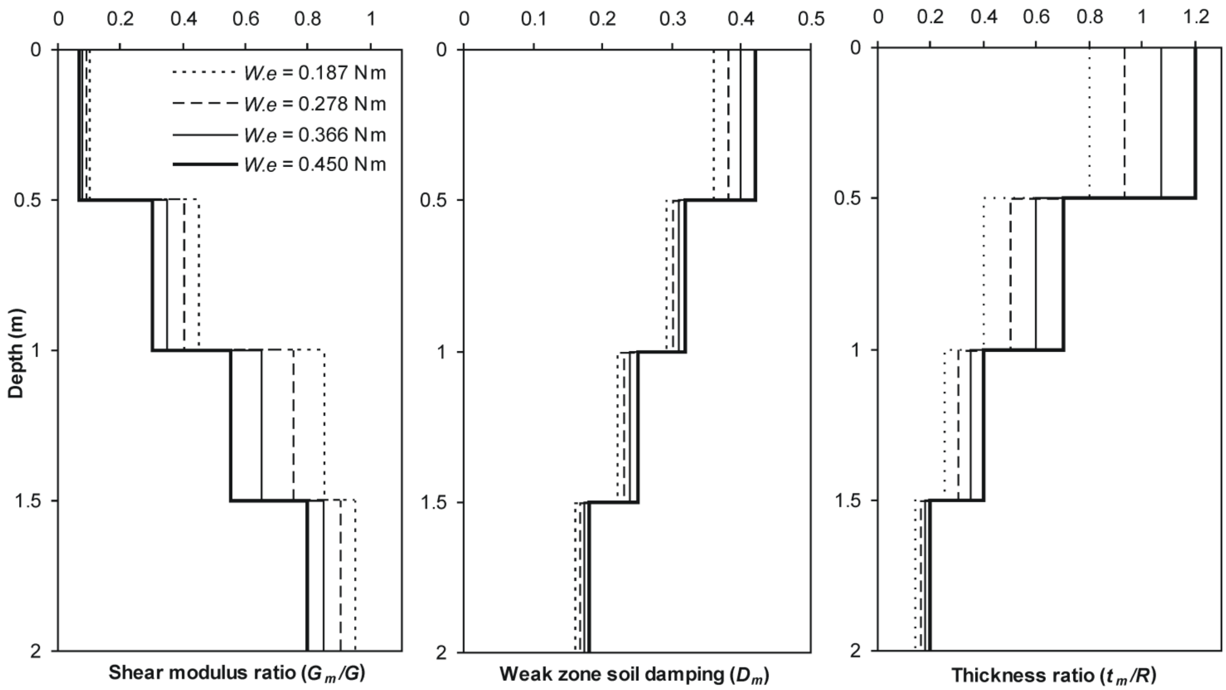


*Figure 8. Variation of Boundary Zone Parameters With Depth For Different Eccentric Moments.*

Theoretical analysis is performed using the usual properties of concrete for the pile material and soil properties from Table 1. Different separation lengths are assumed for different eccentric moments. Using the ratio $G_m/G = 0$ in the topmost layer, the separation between the pile and soil is implemented. The separations between pile and soil are considered 2.1$d$ (= 0.21 m) for $W\cdot e$ = 0.187 Nm and 2.7$d$ (= 0.27 m) for $W\cdot e$ = 0.450 Nm as the separation length is expected to be more at higher eccentricity. Theoretical response curves of piles under coupled vibration are calculated by two different approaches: (1) Novak and Sheta (1980), and (2) Han (1997). The results obtained from numerical analysis are compared with the experimental data to check the compatibility and efficiency of these different methods.

The comparison curves for horizontal and rocking response obtained from continuum approach of Novak and Sheta (1980) and experiments are presented in Figure 9. It can be noted from the comparison that the theoretical resonant frequencies match quite well with the measured results for both horizontal and rocking motions though in higher eccentric moments it differs a little from experimental results. It can also be seen that the theoretical model predicts the resonant amplitudes reasonably well for first mode but in the case of second mode a slightly high value is observed for rocking mode of vibration.

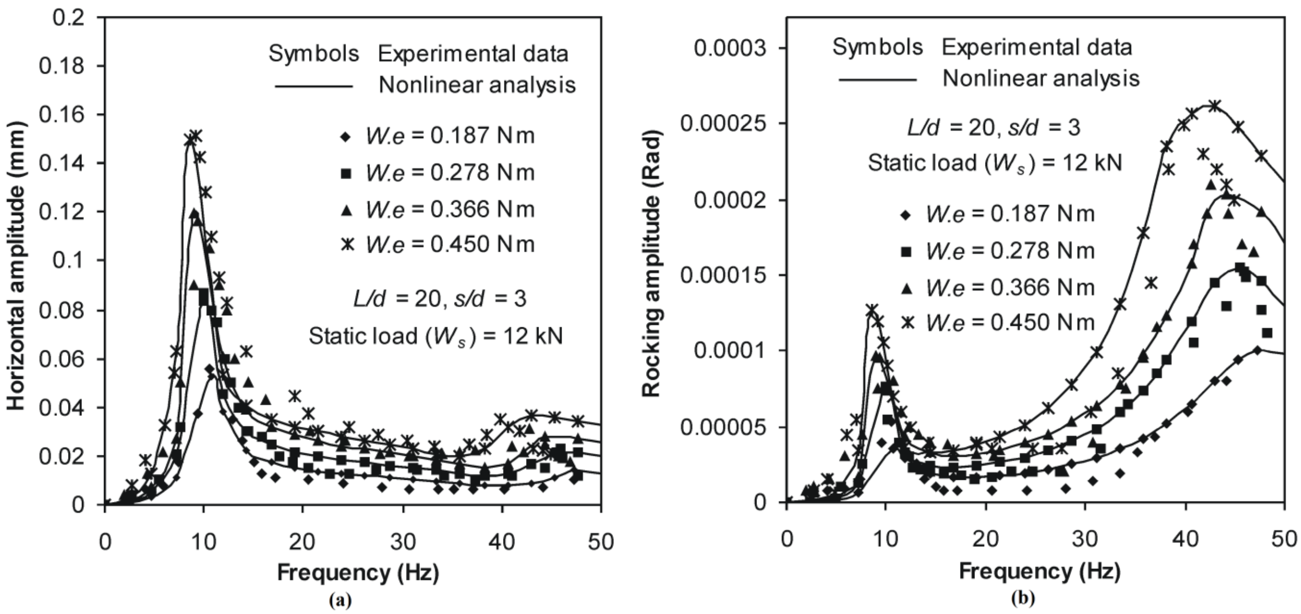


*Figure 9. Comparison between the Response Curves obtained from Experiment and Continuum Approach of Novak and Sheta (1980) [(a) Horizontal And (b) Rocking Mode]*

*Table 3. Nonlinear Analysis Results By Continuum Approach of Novak and Sheta (1980) of (2 x 2) Group Piles (L= 2 m, d= 0.1m, $W_s$ = 12 kN)*

| Eccentric moment (N m) | Pile cap embedded into the soil ($h$ = 0.175 m) | | | | | |
|---|---|---|---|---|---|---|
| | $f_{n1}$* (Hz) | $A_{H1\text{-res}}$** (mm) | $\psi_{r1\text{-res}}$*** (Rad) | $f_{n2}$* (Hz) | $A_{H2\text{-res}}$** (mm) | $\psi_{r2\text{-res}}$*** (Rad) |
| | | | *L/d* = 20, *s/d* = 2 | | | |
| 0.187 | 8.75 | 0.0437 | 0.000037 | 43.00 | 0.0162 | 0.000114 |
| 0.278 | 8.25 | 0.0533 | 0.000042 | 41.50 | 0.0287 | 0.000165 |
| 0.366 | 7.75 | 0.0973 | 0.000064 | 39.75 | 0.0389 | 0.000260 |
| 0.450 | 7.00 | 0.1815 | 0.000128 | 38.25 | 0.0429 | 0.000300 |
| | | | *L/d* = 20, *s/d* = 3 | | | |
| 0.187 | 10.75 | 0.0530 | 0.000035 | 47.25 | 0.0145 | 0.000101 |
| 0.278 | 10.00 | 0.0836 | 0.000073 | 45.50 | 0.0214 | 0.000155 |
| 0.366 | 09.00 | 0.1196 | 0.000097 | 44.25 | 0.0281 | 0.000204 |
| 0.450 | 08.25 | 0.1499 | 0.000127 | 43.00 | 0.0370 | 0.000262 |
| | | | *L/d* = 20, *s/d* = 4 | | | |
| 0.187 | 12.25 | 0.0847 | 0.000040 | 50.25 | 0.0136 | 0.000086 |
| 0.278 | 11.50 | 0.0823 | 0.000047 | 49.25 | 0.0189 | 0.000129 |
| 0.366 | 11.00 | 0.1020 | 0.000052 | 47.25 | 0.0246 | 0.000170 |
| 0.450 | 9.25 | 0.1871 | 0.000088 | 45.50 | 0.0295 | 0.000285 |

* $f_{n1}$, $f_{n2}$ = first and second resonant frequencies
** $A_{H1\text{-res}}$, $A_{H2\text{-res}}$ = first and second resonant amplitudes for horizontal motion
*** $\psi_{r1\text{-res}}$, $\psi_{r2\text{-res}}$ = first and second resonant amplitudes for rocking motion

The resonant frequency and amplitude values obtained from nonlinear analysis for both horizontal and rocking direction is shown in Table 3. The analysis is done approximately at 0.25 Hz interval for the detection of resonance frequency as well as resonant amplitude. It is observed from the analysis results that with increasing pile spacing the stiffness of the system is increased. The resonance frequency is increased and the resonant amplitude is decreased with an increase in pile spacing for the same eccentric moment.

The variation of stiffness and damping of piles computed from nonlinear continuum approach with frequency is presented in Figure 10 for horizontal and rocking mode separately. It can be observed that pile stiffness decreases with increasing excitation intensities for both horizontal and rocking vibration though the rate of change in stiffness value is more in the case of horizontal stiffness than rocking stiffness. However, at low frequencies, the pile system stiffness is almost constant since the dynamic stiffness is very close to the static one at low frequency. It is also noted that the pile system damping for both horizontal and rocking modes of vibration reduces as the excitation intensity increases. As the frequency approaches to zero the system damping increases significantly. With the increase in eccentric moment the stiffness as well as damping is reduced and as a result the resonance frequency and amplitude values are decreased as shown in Table. 3. This is primarily due to the fact of the development and spread of the weak soil zone and the soil-pile separation between pile and soil.

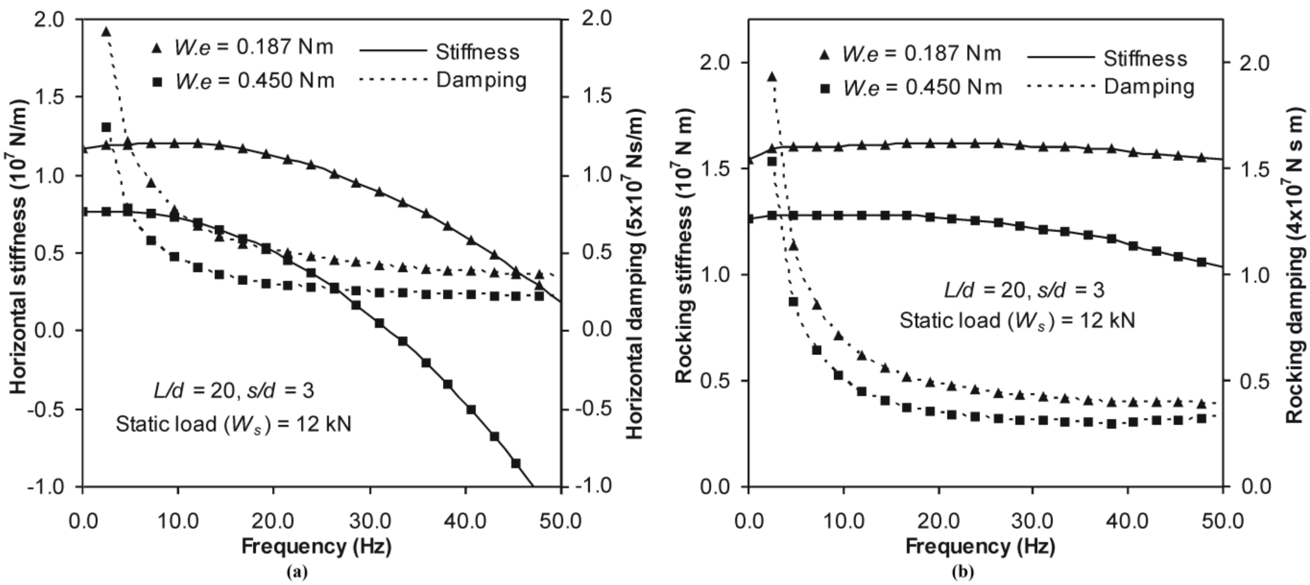


*Figure 10. Variation of Stiffness and Damping Coefficient of Soil-Pile System With Frequency [(a) Horizontal and (b) Rocking Mode]*

The experimental results are also compared with results obtained from Han's (1997) soil model and the results are presented in Figure 11. It is observed from the comparison curves that the predicted resonant amplitudes and the resonant frequencies are not in a comparable range with the test results except in the case of first resonant frequency. The resonant frequencies and amplitude values obtained from the analysis (Han, 1997) is listed in Table 4.

## CONCLUSIONS

The present study involves both field testing on model-scale group piles and analysis using various analytical approaches under coupled vibration. A large number of dynamic field tests with different excitation intensities are conducted to study the frequency amplitude behavior of piles for coupled vibration. The measured response curves of piles have been compared with those obtained using two different approaches, namely, nonlinear continuum approach analysis [Novak and Sheta (1980) and Han (1997)]. The effects of different influencing parameters on the dynamic response of pile have been investigated using the results obtained from the experimental investigation and analysis. The findings from this study have provided better understanding of pile-soil-pile interaction on the dynamic response of piles under coupled vibration. Some important conclusions that can be made from the analytical and experimental investigations are summarized as follows:

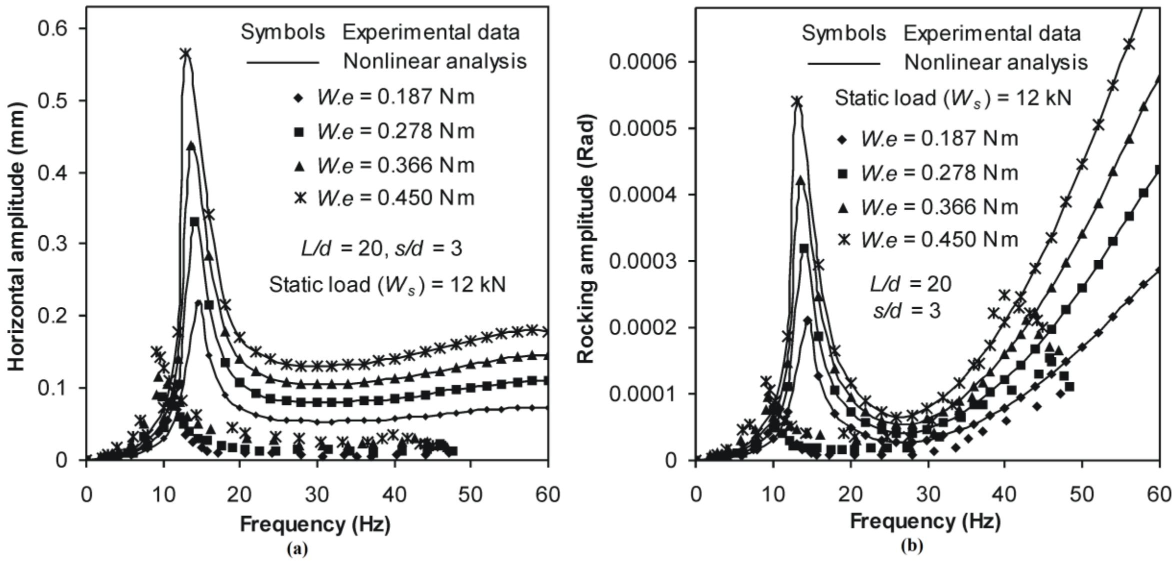


*Figure 11. Comparison between the Response Curves obtained from Experiment and Continuum Approach of Han (1997)[(a) Horizontal And (b) Rocking Mode]*

*Table 4. Nonlinear Analysis Results By Continuum Approach of Han (1997) of 2 x 2 Group Piles ($L$= 2 m, $d$= 0.1 m, $W_s$ = 12 kN)*

| Eccentric moment (N m) | Pile cap embedded into the soil ($h$ = 0.175 m) | | | | | |
|---|---|---|---|---|---|---|
| | $f_{n1}$* (Hz) | $A_{H1\text{-res}}$** (mm) | $\psi_{r1\text{-res}}$*** (Rad) | $f_{n2}$* (Hz) | $A_{H2\text{-res}}$** (mm) | $\psi_{r2\text{-res}}$*** (Rad) |
| | | | $L/d$ = 20, $s/d$ = 2 | | | |
| 0.187 | 13.75 | 0.2928 | 0.000296 | 61.25 | 0.0798 | 0.000330 |
| 0.278 | 13.25 | 0.3844 | 0.000384 | 60.25 | 0.0131 | 0.000542 |
| 0.366 | 12.50 | 0.4969 | 0.000503 | 59.00 | 0.1724 | 0.000714 |
| 0.450 | 11.75 | 0.6140 | 0.000661 | 58.00 | 0.1940 | 0.000883 |
| | | | $L/d$ = 20, $s/d$ = 3 | | | |
| 0.187 | 14.75 | 0.2185 | 0.000210 | 62.75 | 0.0737 | 0.000307 |
| 0.278 | 14.00 | 0.3320 | 0.000319 | 62.00 | 0.1101 | 0.000466 |
| 0.366 | 13.50 | 0.4371 | 0.000420 | 61.00 | 0.1449 | 0.000614 |
| 0.450 | 13.00 | 0.5638 | 0.000593 | 60.25 | 0.1793 | 0.000787 |
| | | | $L/d$ = 20, $s/d$ = 4 | | | |
| 0.187 | 15.75 | 0.1843 | 0.000174 | 65.50 | 0.0683 | 0.000251 |
| 0.278 | 15.25 | 0.3071 | 0.000272 | 64.50 | 0.1021 | 0.000383 |
| 0.366 | 14.75 | 0.3969 | 0.000380 | 63.25 | 0.1344 | 0.000505 |
| 0.450 | 14.00 | 0.5191 | 0.000553 | 62.25 | 0.1667 | 0.000654 |

* $f_{n1}$, $f_{n2}$ = first and second resonant frequencies
** $A_{H1\text{-res}}$, $A_{H2\text{-res}}$ = first and second resonant amplitudes for horizontal motion
*** $\psi_{r1\text{-res}}$, $\psi_{r2\text{-res}}$ = first and second resonant amplitudes for rocking motion

1. Dynamic response of the piles under strong horizontal excitation exhibits typical nonlinear behaviour for both horizontal and rocking components of motions and this nonlinearity depends on many parameters like shear modulus reduction factor ($G_m/G$), weak zone soil damping ($D_m$), thickness ratio ($t_m/R$) and most importantly separation length.

2. Stiffness and damping of the pile system are decreased for both horizontal and rocking modes of vibration with increase in the excitation level. This reduction is primarily due to the development of the weak boundary zone (reduction in stiffness) around the pile with increase in thickness and the separation between pile and soil.

3. The stiffness of the pile group system increases as the s/d increases which results in decrease in resonant amplitudes and increase in resonant frequencies for both horizontal and rocking motions.

4. The nonlinear approach by Novak and Sheta (1980) is found capable of predicting the stiffness and damping values of group piles reasonably well for both horizontal and rocking modes of vibration with reasonable estimation of nonlinear parameter such as modulus reduction factor ($G_m/G$), weak zone soil damping ($D_m$), thickness ratio ($t_m/R$) and separation length. On the other hand, the method proposed by Han (1997) was found to be unsatisfactory in this study.

5. The nonlinear parameters like modulus reduction factor ($G_m/G$), damping factor ($D_m$), thickness ratio ($t_m/R$) and the separation length play a major role for actual nonlinear response of the soil-pile system. It is found that the parameter assumed in the top layer has the most influence on the analytical results under coupled vibration.

## REFERENCES


American Society for Testing and Materials International. (2000). "Standard test methods for crosshole seismic testing."ASTM D 4428/D 4428M, 100 Barr Harbor Drive, West Conshohocken, PA, USA.

Bureau of Indian Standards.(1972). "Determination of shrinkage factors." IS 2720, Part 6, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards.(1973). "Determination of water content." IS 2720, Part 2, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards. (1980a). "Determination of specific gravity-Fine grained soils." IS 2720, Part 3/Sec. 1, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards.(1980b). "Determination of specific gravity-Fine, medium and coarse grained soils." IS 2720, Part 3/Sec. 2, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards.(1981). "Method for standard penetration test of soils." IS 2131, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards.(1985a). "Grain size analysis." IS 2720, Part 4, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards.(1985b). "Determination of liquid and plastic limit." IS 2720, Part 5, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards. (1985c). "Specification for high strength deformed steel bars and wires for concrete reinforcement." IS 1786, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards. (1993). "Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure." IS 2720, Part 11, Manak Bhavan, New Delhi, India.

Bureau of Indian Standards.(2000). "Plain and reinforced concrete- Code of practice." IS 456, Manak Bhavan, New Delhi, India.

Burr, J. P., Pender, M. J., and Larkin, T. J. (1997). "Dynamic response of laterally excited pile groups."J. of Geotech. and Geoenviron. Eng., ASCE, 123(1), 1-8.

El Sharnouby, B., and Novak, M. (1984). "Dynamic experiments with group of piles." J. of Geotech. Eng., ASCE, 110(6), 719-737.

ENSOFT INC, (2004). "DYNAN user's manual, Version 2".

Han, Y., and Novak, M. (1988). "Dynamic behavior of single piles under strong harmonic excitation."Cana. Geotech. J., 25, 523-534.

Han, Y. C., and Sabin, G. C. W. (1995). "Impedances of radially inhomogeneous viscoelastic soil media." J. of Eng. Mech., ASCE, 121(9), 939-947.

Han, Y. C (1997). "Dynamic vertical response of piles in nonlinear soil" J. of Geotech. and Geoenviron. Eng., ASCE, 123(8), 710-716.

Kaynia, A. M., and Kausel, E. (1982). "Dynamic behavior of pile groups." 2nd Int. Conf. on Numerical Methods in Offshore Piling, 509–532.
Manna, B. (2009). "Dynamic response of pile foundations- experimental and analytical study". Ph.D. Thesis, IIT Kharagpur.
Mindlin, D. (1936). "Force at a point in the interior of a semi-infinite solid." Physics, 7, 195-202.
Novak, M. (1974). "Dynamic stiffness and damping of piles." Can. Geotech. J., 11, 574-598.
Novak, M., and Grigg, R. F. (1976). "Dynamic experiments with small pile foundations."Cana. Geotech. J., 13, 372-385.
Novak, M., and Aboul-Ella, F. (1978a). "Impedance functions for piles embedded in layered medium." J. of Eng. Mech., ASCE, 104(3), 643-661.
Novak, M., and Aboul-Ella, F. (1978b). "Stiffness and damping of piles in layered media." Proc. of Earthq. Eng. and Soil Dyn., ASCE Specialty Conference, Pasadena, California, 704-719.
Novak, M., and Sheta, M. (1980). "Approximate approach to contact problems of piles." Proc. of Dyn. Response of Pile Foundations: Analytical Aspects, M. O'Neill et al., eds., ASCE, New York, 53-79.
Novak, M., and Han, Y. C. (1990). "Impedance functions of piles in layered media." J. of Geotech. Eng., ASCE, 116(6), 1008-1014.
Novak, M., and Mitwally, H. (1990)."Random response of offshore towers with pile-soil-pile interaction." J. Offshore Mech. Arct. Eng., 112, 35–41.
Novak, M., El Naggar, M. H., Sheta, M., El Hifnawy, L., El Marsafawi, H., and Ramadan, O. (1999). "DYNA 5 - a Computer program for calculation of foundation response to dynamic loads." Geotechnical Research Centre, University of Western Ontario, London, Ontario.