Lateral Dynamic Response and Effect of Weakzone on the Stiffness of Full Scale Single Piles

A. Boominathan · S. Krishna Kumar · RM. Subramanian

Department of Civil Engineering, IIT Madras, Chennai 36, India
e-mail: boomi@iitm.ac.in

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Abstract The determination of dynamic characteristics of soil-pile system by full-scale lateral dynamic pile load testing is an important aspect in the design of pile foundations subjected to dynamic/seismic loads. Field test results are very useful for validating existing linear/nonlinear models, which are used to predict the dynamic stiffness and damping of the soil-pile system. This paper presents the results of field lateral dynamic load tests conducted at two different petrochemical complex sites in India and the measured dynamic constants of the soil-pile system. A 3D finite element analysis is performed using ABAQUS to predict the non-linear response of soil-pile system under dynamic lateral loads. The lateral stiffness estimated from the FE analysis shows good agreement with stiffness measured in the field tests. Dynamic analyses of single piles using improved Novak’s method were performed to study the effect of weak zone around the pile shaft on the lateral stiffness of the piles.

Keywords Dynamic load · Stiffness · Weak zone · Shear modulus

Introduction

There are many sources of ground-borne vibration such as earthquakes, construction, machine-foundation design, offshore structures, nuclear energy, and road and rail development. These sources produce ground-borne vibrations that can be transmitted into buildings via structural foundations, resulting in disturbances and structural damage. Pile foundations act as the major vibration transmission paths, and as such understanding their dynamic behavior is required. Analysis and design of piles subjected to dynamic lateral loads is complex due to the non-linear behavior of soil and the soil-pile separation that happens near the ground surface. Although, certain theoretical models consider soil non-linearity, the effects of pile installation procedure and the formation of soil-pile gaps are not rigorously modelled. The stiffness of the pile in the lateral direction is very low in comparison to its vertical stiffness; hence, the lateral capacity/stiffness of the pile governs the design in most cases, where the lateral loads are dominant. The lateral stiffness and the bending behaviour of piles depend mainly on the characteristics of the top few meters of soil below the ground. The topsoil layers mainly consist of weak deposits such as soft clay or loose sand, resulting in high degree of nonlinearity on piles subjected to lateral loading. Hence, the strong nonlinearity and soil gapping becomes a crucial step in the satisfactory design and performance of pile-supported foundations subjected to dynamic loads. The lateral stiffness constant of a pile is the most important parameter in the sub-structure approach, which is used to analyse pile-supported structures subjected to seismic loading [27].

In the recent past, many sophisticated linear and nonlinear models were proposed to study the lateral response of piles under dynamic loads [2, 4, 8, 10, 11, 14, 19, 21–23, 25], but there are only a few full scale experimental data available to confirm the reliability of these models. The major full-scale field testing carried out on piles embedded in clay and sandy clay sites by various authors [1, 5–7, 9, 12, 24, 28, 31] clearly demonstrate that the performance of existing linear and nonlinear models are highly dependent on in situ soil nonlinearity and dynamic loading characteristics. Hence, it is important to perform in situ full-scale
dynamic tests on piles in order to accurately assess the non-linear dynamic characteristics of the soil-pile system. This paper presents the results of two full-scale field dynamic lateral pile load tests carried out at two different sites in India (Chennai and Hazira) and the results of a nonlinear three-dimensional finite element analysis of piles under dynamic lateral loads using ABAQUS. Although the finite element analysis used in this study includes important features such as soil nonlinearity and gapping at the pile-soil interface, it does not account for the buildup of pore pressure due to cyclic loading. Thus, neither the potential for liquefaction nor the dilatational effect of clays and the compaction of loose sands in the vicinity of piles is accounted for, in the current analysis.

**Dynamic Testing of Piles**

Steady state forced lateral vibration tests were conducted on driven cast-in-situ piles installed, as per the procedure recommended by the Indian Standard code of practice IS: 9716, at two different sites in India (Site-I: Chennai, Tamil Nadu and Site-II: Hazira, Gujarat).

**Soil and Pile Properties**

The soil characteristics, pile dimensions, and properties for both sites are described below.

**Site-I: Chennai, Tamil Nadu**

The test pile is 450 mm in diameter and it extends to a depth of 20.15 m below the ground level. The length of the pile below the cut off level is 18.25 m. The pile is a M30 grade concrete pile, which corresponds to a dynamic Young’s modulus \( E_p \) of 37,000 MPa. The soil profile and characteristics of Site-I obtained from the borehole data are presented in Table 1. It can be observed from the table that four distinct layers characterise Site-I. A 5 m thick very soft silty-clay layer characterises the top 1/3rd length of the pile.

**Site-II: Hazira, Gujarat**

The pile is 500 mm in diameter and is 15.41 m in length below the cut off level. Pile is made of M30 grade concrete and the corresponding dynamic Young’s modulus \( E_p \) is 37,000 MPa. The typical soil profile of Site-II obtained from the borehole record is presented in Table 2. It can be observed from the table that Site-II is characterised by six layers, wherein the top three layers are characterised by a medium stiff silty-clay/stiff marine clay to a depth of 8.5 m and is followed by a 1.5 m thick loose sandy layer. Since direct measurement of shear wave velocity were not carried out at this site, shear wave velocity of layers was estimated using Eq. 1 based on the average SPT-N value. The maximum dynamic shear modulus was determined based on the shear wave velocity using Eq. 2 and the values are reported in Table 1.

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V_s = 91 \cdot N^{0.337} \quad (1)
\]

\[
G_{max} = V_s^2 \rho \quad (2)
\]

### Table 1: Properties of soil at Site-I

| Depth (m) | Thickness (m) | Soil type          | Avg. SPT \((N_{avg})\) | \(V_s\) (m/s) \(\text{Avg.}\) | Density \((\text{kg/m}^3)\) | \(G_{max}\) (MPa) | \(C_u^a\) (kPa)/friction angle |
|-----------|---------------|-------------------|-------------------------|-------------------------------|-----------------------------|-------------------|-----------------------------|
| 2.0–7.0   | 5             | Soft silty clay   | 0                       | 75\(^b\) (91\(^c\))           | 1,800                       | 10.1              | 2.0                         |
| 7.0–12.5  | 5.5           | Fine sand         | 40                      | 170\(^b\) (315\(^c\))         | 1,900                       | 54.9              | 38\(^d\)                   |
| 12.5–18.0 | 5.5           | Silty clay        | 60                      | 362\(^c\)                     | 2,000                       | 262.1             | 180                         |
| 18.0–20.2 | 2.2           | Yellow silty clay | >100                    | 430\(^c\)                     | 2,100                       | 388.3             | 250                         |

\(^a\) based on SPT \([20]\)
\(^b\) based on MASW test
\(^c\) based on SPT ‘N’ value

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Multichannel Analysis of Surface Waves (MASW) is a non-destructive technique that evaluates the elastic condition (i.e. stiffness) of the subsurface. The shear wave velocity structure is typically derived from the fundamental mode Rayleigh wavefield generated by an active and/or a passive source. The frequency-dependent properties of the Rayleigh-type surface waves can be utilised for imaging and characterising the shallow subsurface \([29]\). MASW surveys are typically used for geotechnical engineering purposes—such as \(V_s^{30}\) profiling, bedrock mapping and finite element modeling.

The MASW test was used to determine the shear wave velocities of different layers up to a depth of 12.5 m below the ground level. The shear wave velocity of soil layers below 12.5 m depth was estimated using Eq. 1 based on the average SPT-N value \([17]\). The maximum dynamic shear modulus required for the finite element analysis is determined based on the shear wave velocity using Eq. 2 \([30]\) and the values are reported in Table 1.
Test Setup and Procedure

Typical layout of a forced vibration test is shown in Fig. 1. A steady state sinusoidal force was generated with a 5-tonne capacity mechanical oscillator. The speed of the oscillator was controlled by a DC motor and a speed control unit. The forced vibration response of the piles were measured using two acceleration transducers fixed at the mid height of the pile cap, and at the pile cut off level as shown in Figure 1. A data acquisition system consisting of a multi-channel carrier-frequency amplifier system and a digital storage oscilloscope was used to monitor and record the time history of response of the pile measured by accelerometers. Each acceleration transducer was calibrated before and after conducting the tests. After every steady state lateral vibration test, the eccentricity of the oscillator was increased to raise the dynamic force and the test was repeated to cover a wide range of lateral displacements expected during a typical dynamic loading of the pile. The tests were repeated with five different eccentricities in the machine load. More information about interpretation of test data is presented in [6].

![Fig. 1 Typical forced vibration test layout (After [6])]  

Test Results

The displacement amplitude of vibration ($A_x$) was computed from the measured acceleration using Eq. 3.

$$A_x = \frac{a_x}{4\pi^2f^2}$$  \hspace{1cm} (3)

where, $a_x = \text{measured horizontal acceleration of vibration (mm/s}^2\text{)}$ at a particular frequency, $f$ (Hz).

The computed values of the amplitude of displacement corresponding to the pile cut-off level at each frequency for different eccentricities of the oscillator were plotted as frequency response curves. A typical frequency response curve obtained for Site-I and Site-II are presented in Figs. 2 and 3, respectively.

It can be noticed from Fig. 2 that the resonant frequency of soil-pile system at Site-I ranges from 10.5 to 14 Hz. The resonant frequency reduces from 14 Hz to 10.5 Hz as the magnitude of the dynamic force increases, indicating a non-linear response of the soil-pile system due to the degradation of soil stiffness. This observation is consistent for almost all the test piles at Site-I. The observed non-linearity could be due to the presence of a 5 m thick very soft silty-clay layer at the top with a SPT-N value of zero. It can be observed from Fig. 3 that the resonant frequency of soil-pile system at Site-II ranges from 13 Hz to 15 Hz.

![Fig. 2 Dynamic amplitude vs. frequency for Site-I]
i.e., the resonant frequency remains practically same irrespective of varying magnitudes of the dynamic force. This indicates that the degree of nonlinearity for Site-II is less, which is due to presence of relatively medium-stiff to stiff-clay layers in the top 8.5 m depth below the ground surface.

Non-Linear Finite Element Analysis

Finite-element models are extremely versatile, allowing for the introduction of nonlinear behavior in the soil–pile system and contact surfaces. Full 3D geometric models were used in ABAQUS to represent the pile–soil system [16]. The non-linearity in the lateral response of piles is because of the geometric and material non-linearity of the pile and the soil surrounding the top 1/3rd length of the pile. In the present study, the pile is idealised as an elastic linear isotropic material without any damping. The solid circular pile is modelled to the same scale as in the field conditions. The pile is discretized using solid tetrahedral elements. The pile is assumed to be pinned at its base and is free at the top simulating a free head condition similar to the large-scale field test. The soil profile in the FE analysis is modelled identical to the field conditions. The non-linear stress–strain behavior of soil is modelled using an elasto-plastic Drucker-Prager criterion. The input parameters, such as the Young’s modulus that is determined from the shear modulus, are presented in Tables 1 and 2. Soil is essentially a Tresca material, if the un-drained behaviour is being considered. The un-drained strength of the clays is used to model it as a Tresca material. Sand was modelled using the friction angle calculated (See Tables 1, 2) based on the correlations with SPT ‘N’ value [20]. The soil matrix of size 20D is adopted, where D is the diameter of the pile. Kelvin-Voigt elements are used at the soil mesh boundary to prevent reflection of stress waves on the boundaries and to eliminate the “box effect” (i.e., the reflection of waves back into the model at the boundaries) during dynamic loading, and they are assumed to be frequency independent given the low frequencies considered (0–20 Hz). The boundaries are restrained in the vertical direction. The base of the FE model is fixed in all directions and the top of the soil mass is not restrained. The discretized soil–pile model is presented in Fig. 4.

The modelling of the pile–soil interface is crucial because of its significant effect on the response of piles to lateral loading. The interface between the pile and the soil was modelled using surface-to-surface contact elements. The pile is considered as the contact surface and the soil as the target surface, to allow for the interpenetration of pile nodes into the soil surface. Penalty contact method with small sliding was utilised to simulate the normal and tangential contact behavior. The separation of contact under reversal of loading (tension) was also simulated.

The verification process followed incremental steps to ensure that pile, soil, and boundary conditions were separately accounted for to minimise error accumulation. The size of the mesh was mainly dependent on the loading conditions and geometry of the piles. The mesh was refined near the pile to account for the severe stress gradients and plasticity encountered in the soil, with a gradual transition to a coarser mesh away from the pile in the horizontal X and Y directions. The vertical Z direction subdivisions were kept constant to allow for an even distribution of vertically propagating SH waves. The maximum element size was less than one-fifth to one-eighth the shortest wavelength to ensure accuracy.

A cyclic ramped load similar to that in the field test was applied at the top of the pile at 0.75 m above the ground surface.
level. The acceleration, stresses, and displacements were recorded throughout the soil-pile system. The typical displacement contour obtained from finite element analysis is shown in Fig. 5.

The dynamic amplitude versus frequency response curves obtained from the finite element model for Site-I and Site-II are presented in Figs. 2 and 3, respectively. It can be observed from the figures that the lateral dynamic response of piles predicted by the FE simulation matches the field measurements. For the given soil conditions the FE simulations over-predict the amplitude by about 5–10%. The amplitude predicted by the FE simulations for the given soil conditions increases at higher eccentricities and near the resonant frequency. The inhomogeneity that presents in the soil means that experimental results will invariably differ from the idealized conditions. Another peak, but smaller in amplitude, is observed at a frequency of about 5 Hz, which is due to the resonant frequency of the pile in its 4th mode (see [3]). It can be noticed that the shear-wave velocity estimated from the SPT ‘N’ values are much higher than the direct measurements (see Table 1). Use of estimated shear-modulus instead of direct measurements of dynamic properties of the soil would have resulted in an estimation of a much stiffer response by FE.

The dynamic stiffness of the soil-pile system is typically obtained from a plot of dynamic force versus the equivalent static amplitude using the procedure described in [6] and as per IS 9716 [18]. The variation of the static amplitude with the dynamic force measured from the field tests and those predicted from the FE simulations for both the sites are presented in Figs. 6 and 7, respectively. It can be observed from Figs. 6 and 7 that the simulated response matches with the response observed in Site-I. In Site-II, although the simulated response matches with the measured response at low to moderate force levels, the simulated response at higher force level is about 30% more than what is measured in the field-test. This might be due to the estimation of dynamic properties, especially for the soil in the top 1/3rd length of the pile, from SPT ‘N’ values, instead of a direct measurement of the dynamic properties of the soil. This signifies the importance of measurement of dynamic soil properties.

The lateral stiffness of soil-pile system predicted by the finite element model and those measured from field tests are presented in Table 3. It can be observed from Table 3 that the FE simulation matches with the results obtained from the field tests. Site-I characterised by soft clay deposits at the top with a low shear wave velocity ($V_s$) of 75 m/s has lower stiffness in contrast to Site-II. Hence, it is evident that the site conditions, especially the top 1/3rd length of the pile, play an important role on the lateral stiffness of the pile, although the resonant frequency remains practically unchanged for piles of approximately same dimensions located in different strata. Comparison of FE simulations and large-scale field tests signifies the importance of measurements of dynamic characteristics of the sites.

**Effect of Weak Zone on Lateral Stiffness**

It is found that the site conditions, and in particular, the properties of the top soil layers greatly govern the degree of non-linearity and the dynamic lateral stiffness of the soil-pile system. Dynamic analysis were performed using Improved Novak’s method, where a non-reflective
boundary is formed between the near field and the far field to account for the mass of soil in the boundary, to understand the effect of presence of weak zone in the top layers (Fig. 8) on the lateral stiffness of the piles. DyNaN [13] is used to study the effect of weak zone on the dynamic response of piles. DyNaN employs well-established analytical solutions based on the improved Novak's approach to describe the soil-structure interaction under dynamic loading conditions. A non-reflective boundary is used between the near field and the far field to account for the mass of soil in the boundary.

Both theoretical and experimental studies have shown that the dynamic response of the piles is very sensitive to the properties of the soil in the vicinity of the pile shaft [15]. A rigorous approach to the nonlinearity of soil-pile system is extremely difficult and therefore approximate theories have to be used. Novak and Sheta [26] proposed including a cylindrical annulus of softer soil (an inner weak zone or so called boundary zone) around the pile in a plane strain analysis. The ideal model of boundary zone should have properties smoothly approaching those of the outer zone to alleviates wave reflections from the interface. The model of non-reflective interface assumed that the

Table 3 Measured and simulated lateral stiffness

| SITE   | Lateral Stiffness (MN/m) |
|--------|--------------------------|
| Field test | ABAQUS |
| Site-I  | 10.75  | 10.28 |
| Site-II | 56.98  | 52.85 |

Fig. 7 Force vs. static amplitude plot for Site-II

Fig. 8 Weak zone around pile

Fig. 9 Dynamic amplitude vs. frequency—effect of weak zone
boundary zone has a non-zero mass and a smooth variation into the outer zone by introducing a parabolic variation function, which may be best fit with use of experimental data. In the present analysis, DynaN is used to determine the stiffness of the soil-pile system considering the mass in the boundary zone.

The top soil layers were assumed to have developed weak zones due to pile driving. The weak zone effect is considered by reducing the shear modulus ratio by 0.5% and increasing the material damping of the soil layer by 0.5% (or reduced by 10% of the original damping) for every subsequent increase in eccentricities. Weak zone shear modulus ratio is the ratio of shear modulus in the disturbed zone around the pile to the shear modulus of soil present outside the disturbance zone. Weak zone thickness ratio is the distance of the disturbed zone from the outer diameter of the pile to the radius of the pile. A zone of 1.25 times the radius was assumed to be weak zone in the top layers. The soil and pile properties were similar to the non-linear Finite Element Analysis. The pile is assumed to have a fixed connection with the pile-cap. The dynamic amplitude versus the frequency plots for Site-I and Site-II are presented in Fig. 9.

It can be observed from Fig. 9 that the presence of weak zone shifts the predominant frequency to higher frequency range by about 5 Hz, but cannot be precisely attributed to the weak zone because of approximations involved in Novak’s approach. However, the lateral extent of the weak zone around the pile shaft has negligible effect on the frequency. The force versus the static amplitude plots is presented in Fig. 10. It can be observed from Fig. 10 that the stiffness of the soil-pile systems for both the sites decreases due to presence of weak zone. The stiffness of the soil-pile system obtained using improved Novak’s approach is 7 and 20 MN/m for Site-I and II, respectively. The decrease in the stiffness, due to the presence of the weak zone in the topsoil layers is significant for the very stiff soil site (Site-II).

Summary and Conclusions

Based on the lateral dynamic pile loads tests carried out on full scale single piles at two different sites in India, it is found that the site conditions, and in particular, the properties of the top soil layers greatly governs degree of non-linearity and the dynamic lateral stiffness of the soil-pile system. It is found that the piles installed in medium-stiff to stiff clay have higher lateral stiffness compared to the piles in very soft clays, however the resonant frequency of the pile-soil system is found to be unaffected by the stiffness of the soil strata. The finite element analysis is able to predict the dynamic lateral response of pile for soft as well as stiff

soil sites, thus signifying the efficiency of non-linear FE models in simulating the different degrees of nonlinearity of soil and pile separation. The inhomogeneity that presents in the soil means that field-test results will invariably differ from, say, the idealised conditions, making it more difficult to identify shortcomings in the models. Despite this,