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ORIGINAL PAPER

Seismic Requalification of Pile Foundations in Liquefiable Soils

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Abstract Pile-supported structures founded on liquefiable soils continue to collapse during earthquakes despite being designed with required factors of safety against bending due to lateral loads and axial capacity (shaft resistance and end-bearing). Recent research identified a few weaknesses in the conventional design approach: (a) when soil liquefies it loses much of its stiffness and strength, so the piles now act as long slender columns, and can simply buckle (buckling instability) under the combined action of axial load and inevitable imperfections (e.g. out-of-line straightness, lateral perturbation loads due to inertia and/or soil flow). In contrast, most codes recommend that piles be designed as laterally loaded beams; (b) Natural frequency of pile supported structures may decrease considerably owing to the loss of lateral support offered by the soil to the pile and the damping ratio of structure may increase to values in excess of 20 %. These changes in dynamic properties can have important design consequences. The immediate need is not only to rewrite the design code to incorporate these effects, particularly buckling instability but also to requalify and, if necessary, strengthen the existing important piled foundations in liquefiable soils. This paper aims to provide a methodology for carrying out requalification studies. A practical example is taken to show the application of the methodology.

Keywords Pile foundations · Requalification · Liquefaction · Constitutive model

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Introduction

Pile-supported structures still collapse and/or get severely damaged during most major strong earthquakes. This is despite the fact that a large factor of safety is apparently employed in the design. Case histories of failure of pile- supported structures during earthquakes indicated that the pile foundations were damaged. Superstructures were intact/undamaged and they as a whole tilted or rotated rendering them useless following an earthquake (Yoshida and Hamada [1], Kawamura et al. [2]). This strongly indicates that the correct failure mechanisms governing the failure of pile foundations have not been properly taken/followed while designing these structures.

Overview of a Typical Loading on a Pile Foundation

Figure 1 shows a schematic diagram of the various stages of loading on a pile-supported structure during a seismic liquefaction process. P_{static} (Stage I) represents the axial load on the piles in normal condition and this axial compressive load may increase/decrease further due to inertial effect of the superstructure ($V_{inertial}$) as shown in Stage II. When soil liquefies, the pile becomes unsupported and acts like a long slender column as described in Stage III condition. Ground movement such as flow failures or lateral spreading can also induce additional kinematic loads on the pile foundations and is shown in Stage IV loading. A detailed discussion

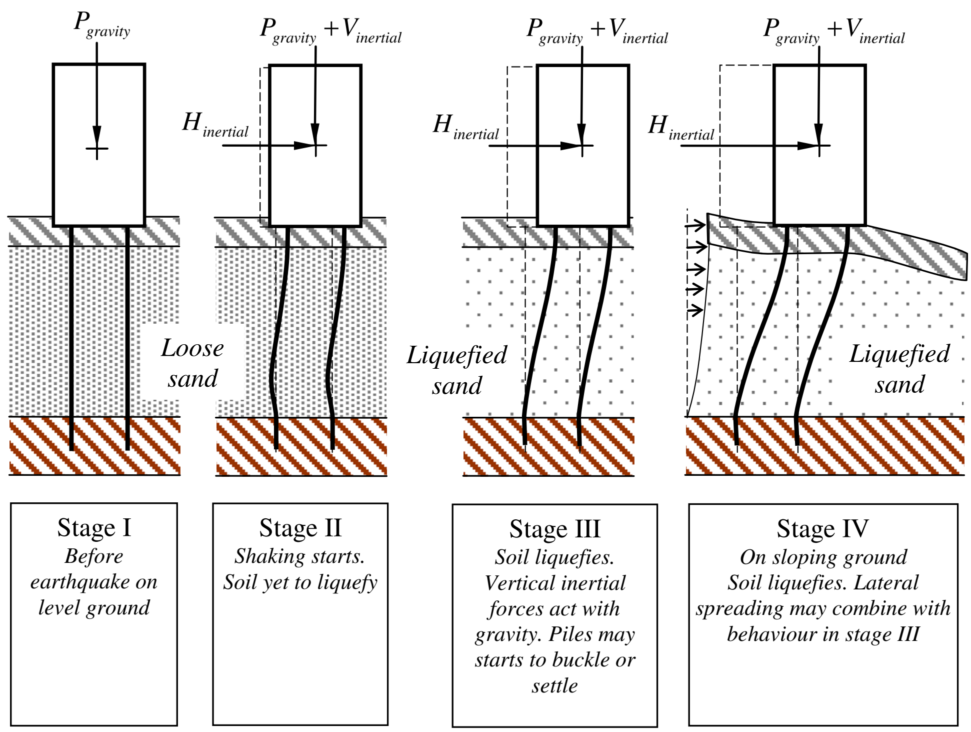


Fig. 1 Different loads acting on a pile

including validation of the aforementioned stages of loading through experiments and analytical solutions can be found in [1–5]. Apart from the loading, the modal parameters i.e. the dynamic properties of the building (first resonant frequency and damping) in Stage II will be very different from Stage III. In most cases, the time period of the building will increase a few times during the transition from Stage II to Stage III. Damping of the structure, on the other hand, will also increase a few times and in some instances can go up to 20 %. Details of this change in dynamic properties can be found in Lombardi and Bhattacharya [5] and a simple model (see Fig. 2) is proposed and is discussed later in the paper.

It is clear that a design method or a requalification study should ensure that the stresses in the pile should not exceed the yield stress of the pile material at any point during the whole earthquake and also during the transition from "no-liquefaction" to "full-liquefaction". It is of interest to review the codes of practice in this regard and it will become apparent that not all the worst load combinations are taken into consideration and that seismic requalification is necessary for important lifeline structures.

A Short Review of the Codes of Practice

The Japanese Highway Code of practice (JRA) advises practicing engineers to consider two different loading conditions:

(i) Inertial force due to the oscillation of the superstructure i.e. Stage II loading in Fig. 1;

(ii) Kinematic loading exerted by the lateral pressure of the liquefied layer and any non-liquefied crust resting on the top of the liquefied deposit i.e. Stage IV loading in Fig. 1. The code also suggests designers to check against bending failure due to kinematic and inertia forces separately.

Similarly, Eurocode 8 [6] advises engineers to design piles against bending due to inertia and kinematic forces arising from the deformation of the surrounding soil. In the event of liquefaction, Eurocode 8 [6] also suggests that *"the side resistance of soil layers that are susceptible to liquefaction or to substantial strength degradation shall be ignored"*. Other provisions, such as the NEHRP code [7] and Indian Code [8] also focus on the bending strength of the piles. In summary, the codes of practice simply treat piles as laterally loaded beams and assume that the lateral load due to inertia and soil movement causes bending failure.

Importance of Inclusion of Axial Load and P-delta Effect

This section shows the implication of axial load considerations in pile design. Figures 3 and 4 show the effect of the axial load on the bending response of a pile foundation, in terms of normalised pile displacement *y/D* and pile bending moment. In these figures *y* is the lateral pile head displacement evaluated considering the effect of axial load

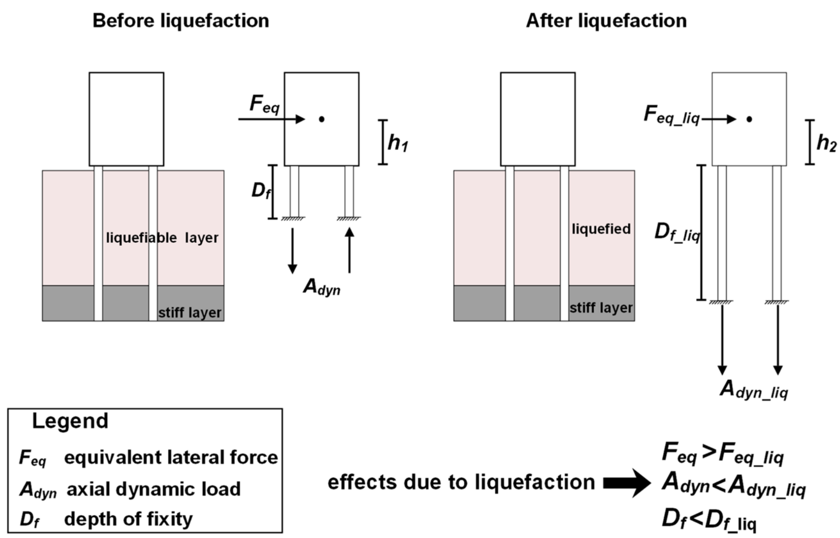


Fig. 2 Changes in modal parameters of a pile-supported structure

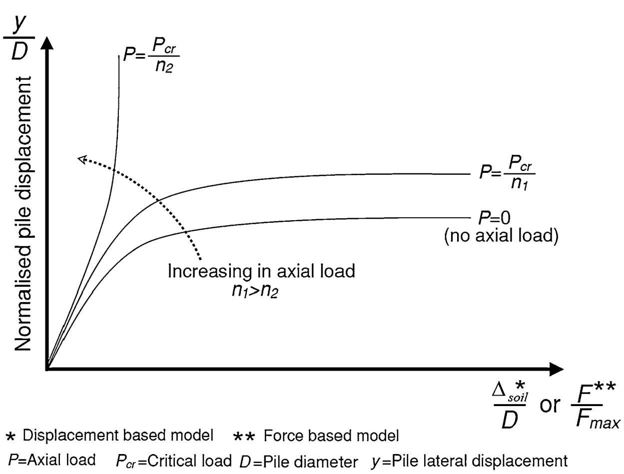


Fig. 3 Pile head deflection response due to lateral and axial load

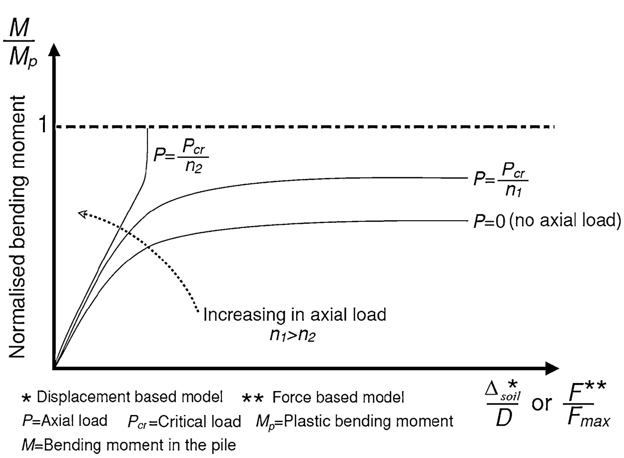


Fig. 4 Pile bending moment response due to lateral and axial load

against either Δ_{soil}/D in displacement based analysis method or F/F_{max} in force based analysis method. Clearly, Figs. 3 and 4 show that when the axial load is higher with respect to critical buckling load of the pile, the pile head deflection (y) and maximum bending moment in pile get larger. During liquefaction, as the soil loses its stiffness, the elastic buckling load (P_{cr}) also reduces. If a constant static axial load on pile is assumed, it can be seen that P/P_{cr} ratio increases. With the increasing P/P_{cr}, the pile head deflection and bending moment in pile also increase. When this ratio is close to 1, i.e., axial load is close to the buckling load, the bending moment amplification factor becomes very high, which leads the bending moment in pile to reach its plastic moment capacity, M_p, at a much lower value of lateral load. The sudden rise in pile head deflection demonstrates the failure of the pile where bending moment reaches M_p and pile continues to deflect without any additional loading. This clearly shows the importance of considering axial load in the pile design. Elastic critical load (P_{cr}) of a pile can be computed using the concept of effective length (L_{eff}) of the pile in the liquefiable zone and is a function of boundary condition of the pile at the top and bottom of the liquefiable zone and depth of liquefiable zone. A table for calculating the effective lengths of pile can be found in Bhattacharya and Goda [9].

Importance of Considering Dynamics and Constitutive Behavior of Liquefied Soil

The dynamics of the problem can be considered in a simplified way as described in Fig. 2. A pile can be modelled as a free-standing column fixed at some depth below the ground surface and it is often referred to as depth of fixity (D_f). Figure 2 shows a schematic diagram of the simplified pseudo-static analysis, in which the pile-supported structure (in Stage II loading as shown in Fig. 1) is modelled as a Single Degree of Freedom system and the seismic action is represented by an equivalent shear force, F_{eq}, that is proportional to the total mass of the superstructure and spectral acceleration. The equivalent force is applied to the centre of mass of the superstructure, and hence generates an overturning moment, M, at the point of fixity, which is given by Eq. (1).

$$M = F_{eq} \times (D_f + h_i) \tag{1}$$

where h_i is the height of application of the shear force measured from the foundation level. For equilibrium, the overturning moment must be counterbalanced by two axial loads, which are indicated in the Fig. 2 with the symbol A_{dyn}, in which the subscript *dyn* stands for *dynamic,* and it highlights the fact that A_{dyn} are a result of the overturning moment rather than the weight of the superstructure.

Effects of Depth of Fixity

With the onset of liquefaction and the subsequent reduction in the stiffness of the soil layer, the depth of fixity can increase significantly which primarily depends on the depth of liquefaction and is shown schematically in Fig. 2b denoted by D_{f_liq}.

Depth of fixity may be calculated following the procedure stipulated in IS 2911 [8] or Davisson and Robinson [10] with the simplified assumption to consider the lower end of the pile as fixed at some depth in the non-liquefied soil layer below the liquefied layer. After a certain value of the depth of liquefaction, the depth of fixity can be approximated by a constant value depending on the amount of the degradation of the liquefiable layer (Lombardi et al. [11]). Davisson and Robinson [10] concluded that the depth of fixity is insensitive to the embedment ratio after a certain exposed length for constant soil stiffness. This observation was also reaffirmed by Kerciku et al. [12]. It is observed that depth of fixity of 5–7 times the diameter of the pile is enough as the anchorage in the non-liquefied layer.

Since the total shear force will now be carried by the pile section, one need to check the capacity of the pile section at the bottom of the liquefiable soils for the combined stresses for bending moment (M), shear force (V) and axial force (P). For this, simplified procedures are based on an approximation of interaction surface which may be visualized in 3D plot of P–V–M as shown in Fig. 5. The surface may be generated as the envelope of a number of design interaction curves. The pile section may be considered to be safe for the loads falling within the interaction surface.

Consideration of Pile Groups

For group of piles, the resistance force after liquefaction, A_{dyn_liq} (as shown in Fig. 2) needs to be evaluated. Distribution of forces in the individual piles may be determined by simple static analysis. The steps may be summarized as following

(a) The superstructure can be replaced by an equivalent single degree of freedom (SDOF) system so as to have the same dynamic effects on the foundation. The mass of the equivalent SDOF system may be calculated by equating the base shears and may be represented as following (Wolf [13]) Eq. (2)

$$m = \frac{\sum_j (m_j \varphi_j)^2}{\sum_j m_j \varphi_j^2} \tag{2}$$

where m_j is the mass at story j and ϕ_j is the mode shape at story j.
Once we know the fundamental time period of the superstructure (for normal buildings, the period may be taken as $0.1n$, where n is the number of stories) and the most dominant modes, we can calculate the mass and stiffness of the equivalent SDOF system.

(b) The next step is to model the piles and the raft or grade beams and place the SDOF on the centre of mass of the raft. Then evaluate the forces on the piles by simple static analysis.

For illustration purpose, a typical soil–pile-structure system with pile groups is shown in Fig. 6. The considered building was the Port and Customs Tower situated at the Kandla Port area, Gujarat, India. This building had tilted during the Jan 26th 2001 Bhuj Earthquake and liquefaction have been observed around the building. The building, foundation and soil data used may be found in Dash et al. [14]. The superstructure is modelled as an equivalent SDOF system which gives the same fundamental period of the actual building.

Effects of Liquefaction on the Overall Response

Clearly the time period of the building/structure will increase and so does the damping of the structure. The simplified depth of fixity approach incorporates two important features related to the reduction in soil stiffness caused by liquefaction: (a) reduction in inertia force and consequently the change in maximum bending moment, due to the lengthening of the fundamental period of the models; (b) lowering of the location of the maximum bending moment i.e. location of maximum bending moment moves to deeper locations along with the point of fixity. The transient bending moment is also to be considered and is explained in the next section through an example.

A typical 5-storey building supported on piled foundations will have a period of 0.5 s and is mainly dictated by the building dimensions. If the soil at the site liquefies to a reasonable depth, the period of the building may increase

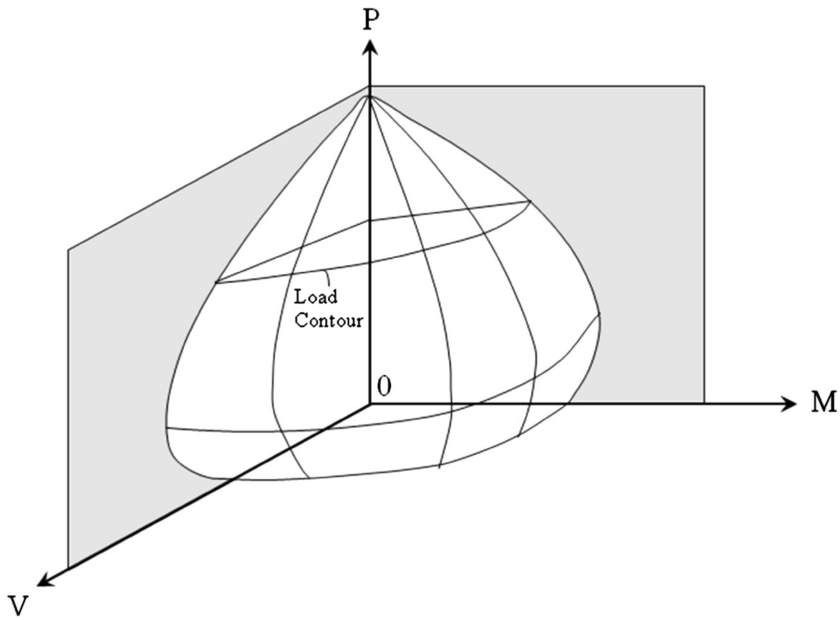


Fig. 5 Interaction surface for combined action of axial force (P), shear (V) and moment (M)

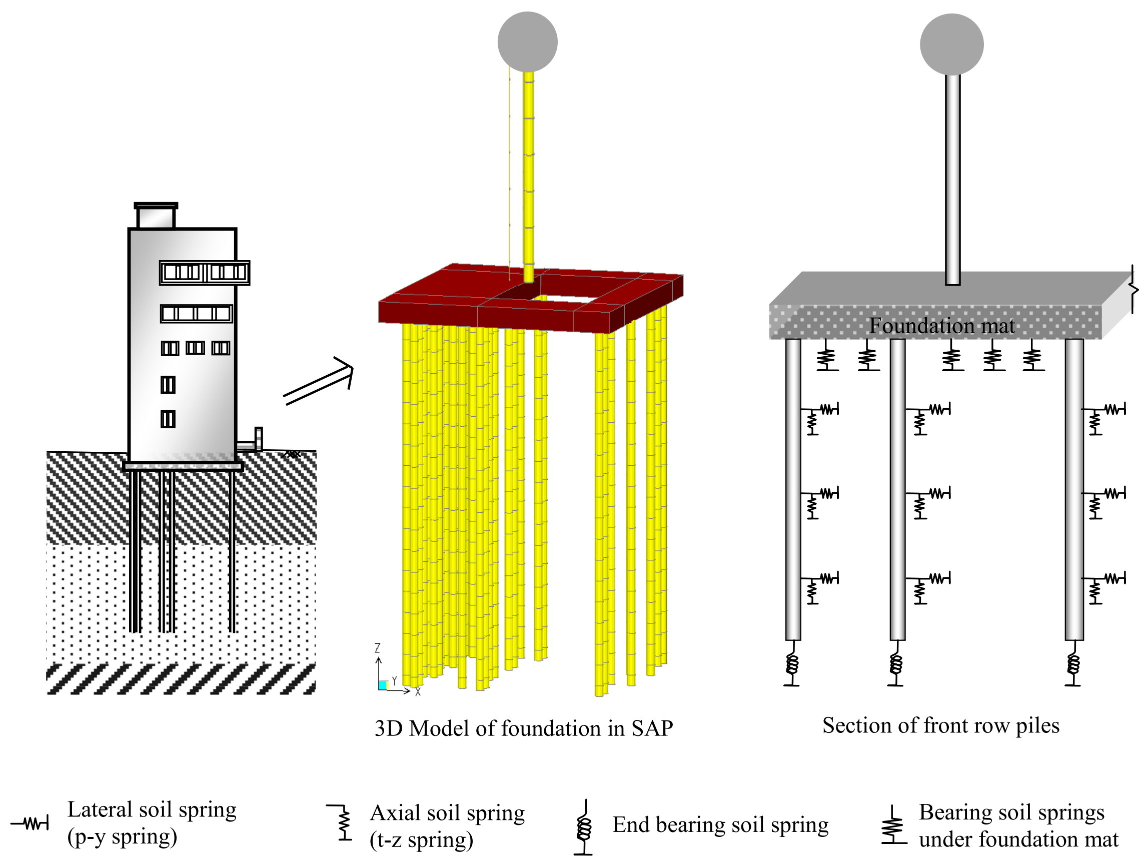


Fig. 6 Schematic of the soil–pile-structure interaction model

and let us assume in this example that this increases to 5 s. Of course, the lengthening of the time period of the structure (in this case from 0.5 to 5 s) depends on many factors including EI (bending stiffness) and length of the pile, depth of liquefaction, stiffness of the soil beneath the liquefied layer. As the structure transits from 0.5 s (Stage II in Fig. 1) to 5 s (Stage III in Fig. 1), the bending moment profile in the pile will constantly change and this transition time may vary and in most cases may, depending on the input motion and the soil profile, range between 8 and 20 s. In some cases, during the transition, the frequency of the structure may get tuned with the frequency of the earthquake causing amplification of the bending moments. However, due to enhanced damping of the structure owing to liquefaction the amplification of the responses will be limited. While depth of liquefaction can be obtained from empirical methods, the time to reach full liquefaction is very important for predicting transient bending moments in the pile. Therefore seismic requalification studies require constitutive modelling of liquefied soil.

It must be mentioned that through analysis of pile foundations from recent earthquakes, Bhattacharya et al. [15] showed that large diameter piles performed better than a group of small diameter piles. As diameter of the pile increases, the stiffness of the pile section increases by four folds and many of the static (buckling) and dynamic (tuning of the structure and the earthquake) instabilities disappear. However how large is large enough is a scientific question that needs to be addressed in a new design method and is beyond the scope of the current paper.

The present paper evaluates the seismic safety of a pile foundation in Haldia due to a scenario earthquake using the various analysis methods such as Beam on Nonlinear Winkler Foundation (BNWF) and three-dimensional finite element.

An Example Application to Show the Requalification Studies

A pile-supported building at Haldia (a small industrial town in the eastern part of India) is considered for the requalification studies. According to IS 1893 (Part 1): 2002 [16] Haldia is placed in Zone IV which has an associated zone factor of 0.24. As per the code, the zone factor is a reasonable estimate of the peak ground acceleration (PGA) of the site. Hence PGA of 0.24 g is assumed for the study.

Geotechnical Data Considered for the Study

The present study has been carried out for a project site at Haldia in West Bengal, India. The site is very near to a river and the ground conditions at the site comprise superficial deposits (Alluvium and River Terrace Deposits) underlain by the clay formation. The generalized soil profile and the soil parameters recommended in the geotechnical investigation are provided in Table 1.

Evaluation of Liquefaction Potential for the Site

It is essential to be able to identify whether or not a soil at a site is susceptible to liquefaction. In order to liquefy, soils must be saturated, able to contract under shear and its permeability must be low during shaking i.e. undrained condition (rules out coarse gravels). Factor of safety (FOS) against liquefaction for the present site is then evaluated by the approaches developed by Idriss and Boulanger [17] or Boulanger and Idriss [18]. When FOS at any depth of a soil layer is less than 1.0 then that layer is assumed to be susceptible against liquefaction and vice versa. It may be mentioned that in absence of real data, static shear stress correction factor K_α has been considered to be 1.0. The factor of safety with depth of the soil layers is shown in Fig. 7 and it is clear that the layers IIIA, IIIB and IV are potentially liquefiable since the FOS against liquefaction is much less than 1.0.

Details of Pile Foundation

Based on the recommendations of the geotechnical investigations report, driven cast in situ piles were used to support the structure. Following are the details of the pile.

- Type: Driven cast in situ concrete piles (Concrete grade M25)
- Diameter, D: 600 mm

Table 1 Generalised soil profile with soil parameters

Strata no.	Basic description	Thickness (m)	$(N_1)_{60}$-value	Unit weight, γ (kN/m^3)	Cohesion, c_u (kPa)	Friction angle, ϕ (°)
I	Soft silty clay	2.0	3	19.1	40	–
II	Soft clayey silt	5.0	2	18.2	23	–
IIIA	Loose sandy silt	12.5	8	18.0	–	28
IIIB	Medium dense silty sand	3.0	8	19.0	–	30
IV	Stiff clayey silt	2.5	8	18.4	49	–
V	Medium dense silty sand	3.0	28	19.0	–	32

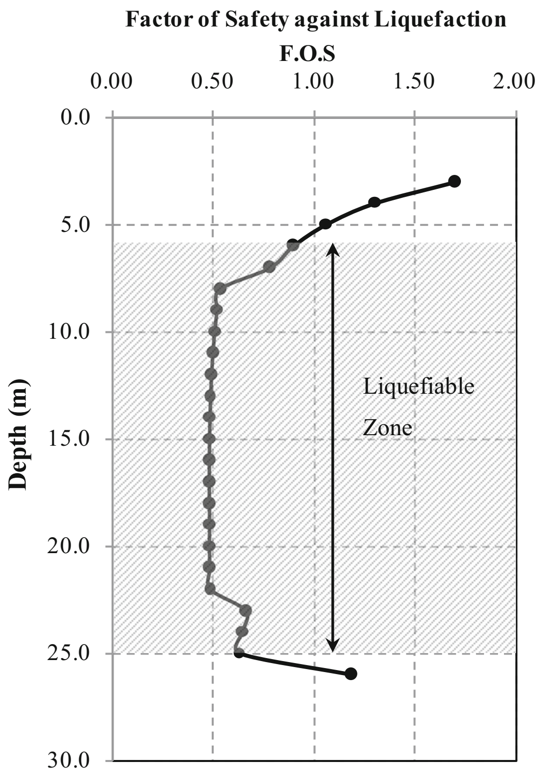


Fig. 7 FOS against liquefaction for Haldia project site by Idriss and Boulanger approach

- Length: 26.5 m below the cut-off level terminating at Strata-V

Vertical Pile Capacity

The pile capacity has been determined based on the codal stipulation of IS 2911 (Part-I/Sec-1) 1979 [19]. Strata-I has been neglected for computation of the pile capacity. Safe vertical pile capacity (P) of 1,100 kN is adopted for the study.

Expected Ground Motion at the Site

During the liquefaction process (i.e. from the onset of liquefaction to the stage when the soil is fully liquefied in the transient phase), the intrinsic properties of the soil-structure system change. The dynamic behavior of the pile i.e. the change in bending moment is dependent on the time required to reach full liquefaction. Usually, this time will vary depending on the earthquake motion at the site and the properties of the soil profile. So it is necessary to adopt appropriate time history as input bed rock motion. Engineers often need to choose a time-history motion at a site where such information is unavailable. An approach for selecting an input motion under such condition is described below. This method is based on spectral matching whereby the spectrum of the input motion is matched with the code-specified response spectrum.

Generation of Spectrum Compatible Time History

Traditionally seismic hazard at a site for design purposes has been represented as design spectra. Thus all seismic codes and guidelines require scaling of selected ground motion time histories so that they match the controlling design spectrum within a period range of interest. Several methods of scaling time histories have been proposed. For example, an input motion is selected first and the motion is manipulated to obtain a motion that matches design spectra. Many programs are available to carry out the spectral matching: *WAVEGEN* (Mukherjee and Gupta [20]), *RSP-Match2005* (Abrahamson [21] and Hancock et al. [22]), *SPEC3* (Kumar [23]). In this paper, scaling of the time histories to match the target design hazard spectra (target spectrum) was carried out using *SPEC3*. Spectrum compatible time history has been generated for the Haldia site and is discussed below.

Spectrum Compatible Time History for Haldia Site

No strong motion records are available for Haldia site. Hence a strong ground motion recorded has been selected based on the PGA value of the site and then the software *SPEC3* has been employed for generating spectrum compatible time history. Details of the program may be found in Kumar [23] and the program may be downloaded free. Based on the expected PGA at the site, the recorded earthquake motion data from the September 20, 1999 Chi–Chi Earthquake having a magnitude of 7.6 with PGA of 0.25 g from TCU122 station was selected. This time history is then modified using *SPEC3* program to obtain the spectrum compatible time history to be used for the study. The target spectrum was considered as per IS 1893 (Part 1: 2002) [16] with medium soil condition. The comparison of different spectra is shown in Fig. 8a, b shows the spectrum compatible acceleration time history for the particular site of for damping of 5 %. This time history may be adopted for input bed rock motion for ground response analysis and also may be useful for evaluation of liquefaction potential and seismic analysis of pile foundations considering soil-structure interaction effects. It may be noted that if the free field time history is used for generating spectrum compatible time history then the generated time history must be de-convoluted at the required depth of input motion.

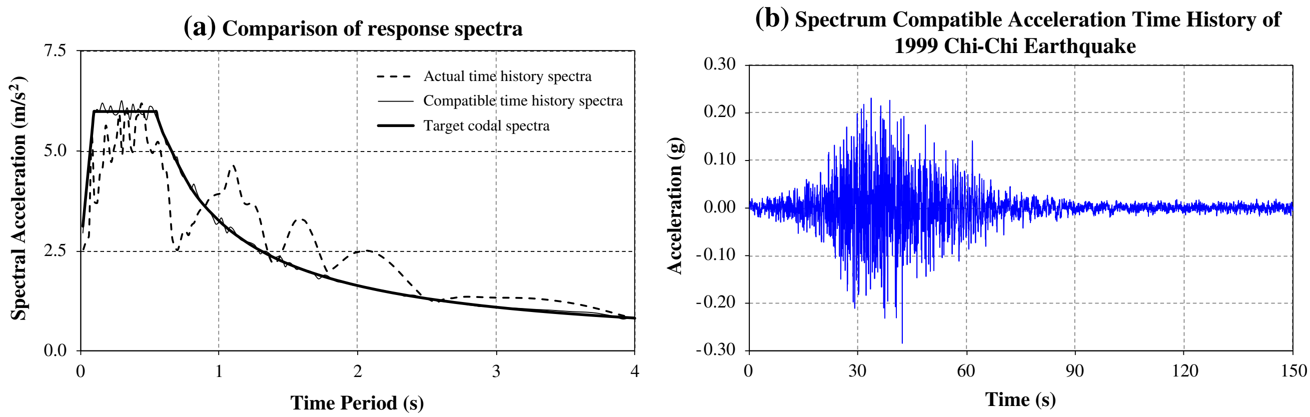


Fig. 8 **a** Comparison of target and compatible response spectrum of Chi–Chi Earthquake time-history. **b** Spectrum compatible acceleration time history used in the study

Check Against Buckling Instability

When the soil medium is liquefiable, the pile section needs to be checked against the buckling instability criterion (Bhattacharya [24], Bhattacharya et al. [25]). The thickness of the liquefiable soil layer (L_0) is considered to be as unsupported length of the pile foundation. For buckling analysis, each pile needs to be evaluated with respect to its end conditions i.e. fixed, pinned or free. Since the embedment depth of the pile in the bottom non-liquefied soil layer (Layer V) is less than the 5 times diameter of the pile section, the bottom boundary may be considered as pinned boundary. If we consider the pile head to be free, then the effective length of the pile section (L_{eff}) may be considered to be $2L_0$.

The Euler's buckling load of the pile (P_{cr}) may be calculated from the well known formula given by Eq. (3).

$$P_{cr} = \frac{\pi^2}{L_{eff}^2} EI \tag{3}$$

Where EI is the flexural rigidity of the pile section and for the considered section P_{cr} calculated from Eq. 3 is equal to 1,211 kN. Hence the ratio of the vertical load and the critical load (P/P_{cr}) is 0.91. As discussed in Bhattacharya [26], Bhattacharya and Madabhushi [27] and Bhattacharya and Goda [9], a pile should not be close to its critical load at full liquefaction. The actual failure load ($P_{failure}$) is some factor, ψ ($\psi < 1$) times the theoretical Euler's buckling load given by Eq. (3). It was inferred that instability may be expected at around 0.35, i.e. ψ is taken as 0.35. However, this factor will depend on the axial load, imperfections or the residual stresses in the pile due to driving. Hence based on the above discussion, it may be stated that the pile section considered for the study needs to be revised to avoid buckling instability during full liquefaction condition.

Estimation of Lateral Inertial Load

For an efficient seismic design of the foundation, it is important to estimate the loads that are being transferred to the foundation during an earthquake. These loads depend on the seismic loads that act on the superstructure during an earthquake. Different codes around the world propose different methods of estimation of these seismic loads on the super structure (e.g. Indian standard IS 1893–2002 [16], Eurocode EN 1998–2004 [6]). In absence of availability of the super-structural details, 10 % of the vertical capacity (i.e. 110 kN) is adopted as inertial force acting on the pile head.

Simplified Analysis Procedure

A relatively simple but detailed nonlinear BNWF (Beam on Nonlinear Winkler Foundation) model is prepared to study the response of a typical pile foundation subjected to a combination of axial load and lateral inertial load (Fig. 9a). The analysis of the BNWF model is carried out by a finite element based structural analysis program SAP 2000 (CSI 2004) [28]. The soil surrounding the pile is modeled as lateral soil springs (p-y spring). The present analytical model considers the boundary condition at pile head as free. Present analysis also assumes that the pile is stable under vertical settlement, hence the support condition is considered as a hinged support at the tip of the pile.

From the evaluation of liquefaction potential, it is clear that the soil layers IIIA, IIIB and IV are liquefiable in case of earthquake loading. The nonlinear spring properties (p-y curve) to represent the non-liquefiable soil layers are calculated according to the API (2003) [29] guidelines. The in situ relative density (D_r) of the soil is established from the experimental value of '*N*' of standard penetration test as per the correlation reported in (Meyerhof [30]).

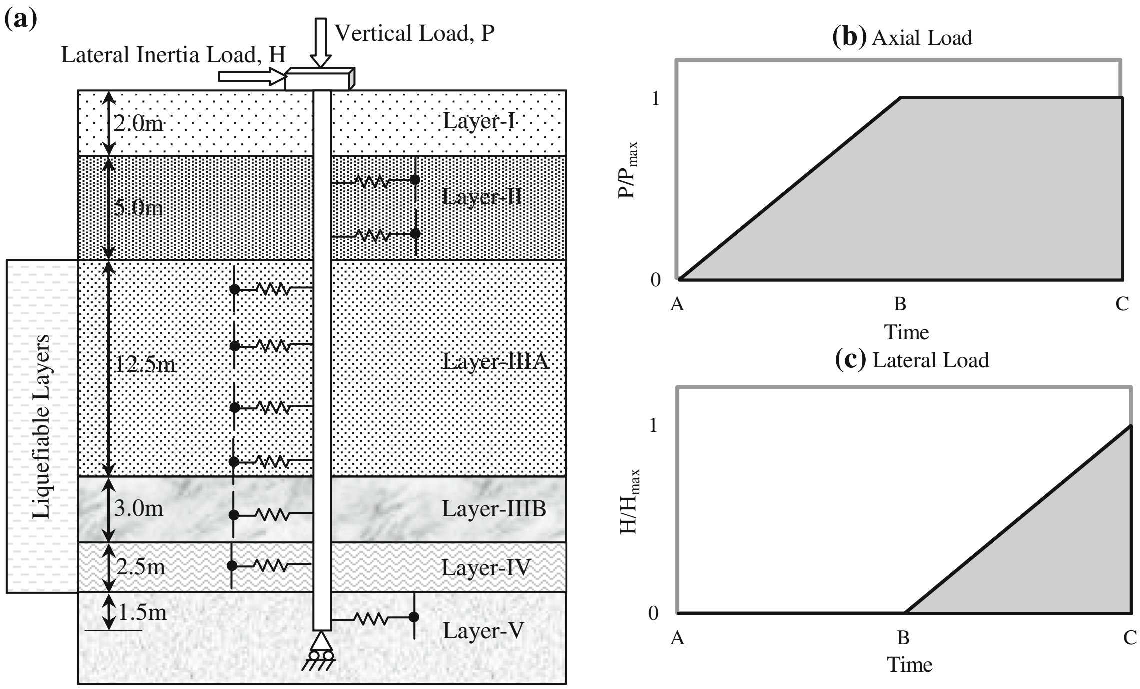


Fig. 9 **a** Soil–pile model considered for the study. **b** Axial load pattern used for the study. **c** Lateral load pattern used for the study

The p-y springs of the liquefied soil are modeled by reducing the strength and stiffness of the springs using a reduction factor, the p-multiplier. This study uses representative $(N_1)_{60}$ value of 10 for the liquefied soil to obtain the p-multiplier value. Though many p-multiplier values are reported in literature, p-multiplier value of 1/50 as suggested by Brandenberg [31] has been adopted.

A nonlinear pseudo-static analysis was performed by using SAP 2000 (CSI, 2004) [28], which is essentially a modified time history analysis. The axial load is present throughout the lateral loading phase. In the time history analyses, the damping and mass of the system was forced to be near zero value to make it pseudo-static. As shown in Fig. 9b, the pile is first subjected to the full axial load (P_{max}) and then the lateral pile load was applied by increasing linearly up to its maximum (H_{max}), keeping the axial load constant. To ensure slow gradual increase of loading, time values at A, B and C in Figs. 9b, c were defined arbitrarily as 0, 60 and 400 s for both axial and lateral loading for the nonlinear pseudo-static analysis carried out in the study. The analysis includes P-delta and large displacement effects. Details of the methodology of analysis can be found in Dash et al. [32].

Combined Action of Axial Load and Bending Moment

If a hinge forms under the combined action of bending moment (M) and axial load (P), the yield condition for a circular section may be represented by the following equation (Heyman [33]).

$$\left(\frac{P}{P_Y}\right)^{3/2}+\left(\frac{M}{M_P}\right)=1 \tag{4}$$

where P_Y: Squash load in absence of bending, i.e. the pile fails in compression; M_P: Plastic moment capacity in absence of axial load i.e. the element fails in pure bending.

Figure 10a shows the plot of the Eq. (4), which is often termed as “yield surface for a plastic hinge under bending and compression”. Any point within the yield surface would imply that the stress in the section has not exceeded the yield stress. For the pile section considered in the analysis, the available plastic moment capacity (i.e. 356.5 kN m) is also marked in the figure for the axial load of 1100 kN.

The distribution of bending moment along the depth of the pile obtained from the pseudo-static analysis as stated in the previous section is shown in Fig. 10b. It may be observed that the moment induced in the pile section at the depths from around 2.5 to 7.5 m is more than the available plastic moment capacity of the section. This indicates that at these depths the factor of safety against formation of plastic hinge will be less than 1.0; hence initiating the failure of the section. Thus when we consider the liquefaction of soil medium for the generated ground motion of the site the pile section of 600 mm seems to be inadequate.

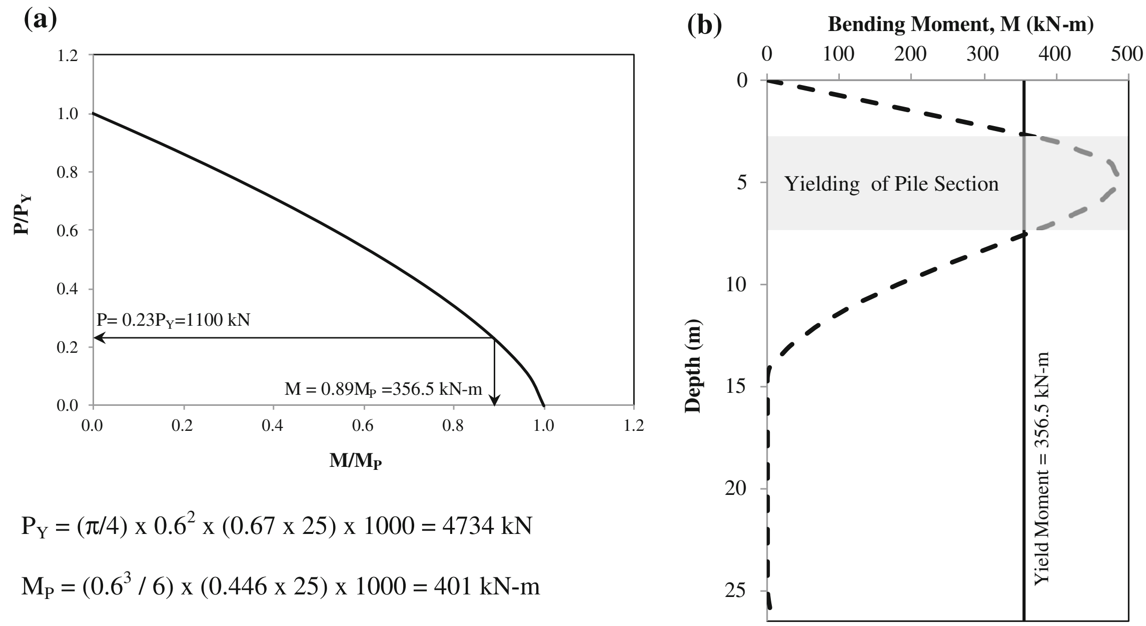


Fig. 10 **a** Yield surface for a plastic hinge for a circular solid pile section under bending and compression. **b** Bending moment distribution along the depth of the pile

Continuum Approach for Soil–Pile System Analysis

When a soil layer undergoes ground acceleration and liquefies subsequently, the effective stiffness of the soil decreases to near zero because the pore pressure in the liquefiable layer increases. The reduction in soil stiffness reduces the overall stiffness of the foundation. The foundation stiffness will not fall to zero as there will be residual stiffness in the soil and of course the stiffness of the unsupported piles. Along with the change in stiffness, the mass will also decrease. The liquefiable soil layer behaves similar to a liquid during liquefaction. In addition to the stiffness and mass reduction, the damping will increase as liquefaction occurs and can reach as high as 20 %. If we assume the stiffness, mass and damping in the superstructure to be constant throughout, the period the soil–pile system changes with change of stiffness, mass and damping of the foundation system. But it does not happen instantaneously but over a time. Hence the dynamic behaviour of the pile i.e. the change in bending moment, shear force, time period and damping are dependent on the time required to reach full liquefaction. Usually, this time will vary depending on the earthquake motion at the site and the properties of the soil profile. Stages of liquefaction may be reasonably determined by implementing three-dimensional finite element model with a reasonable constitutive model of soil which can simulate soil behaviour during liquefaction with certain accuracy.

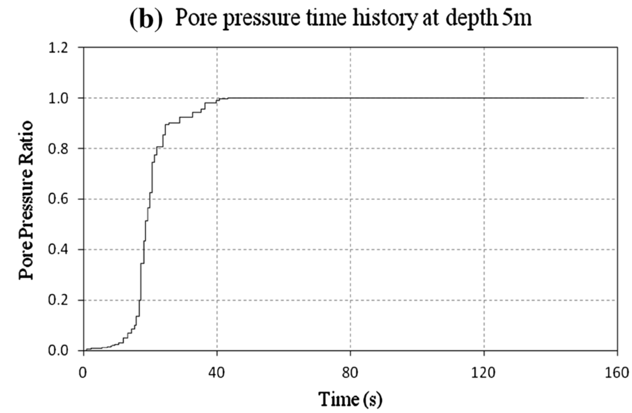
Application of 3-D Finite Element Model

Two/three dimensional behavior of the soil–pile system especially under dynamic condition can be studied by FE modeling of the soil–pile system. Advanced soil plasticity models can also be employed in these analyses. Bentley and El Naggar [34] have reported nonlinear analysis for single piles with a particular reference to Loma Prieta Earthquake (1989) time history for sandy soil. Wu and Finn [35, 36] proposed a quasi three dimensional finite element method of analysis for the dynamic response of the pile foundations feasible for practical applications. The method was presented for elastic response and was validated against Kaynia and Kausel [37], Novak et al. [38] and Fan et al. [39]. Finn and Fujita [40] have investigated the behavior pile foundation under liquefying soil conditions with 2D finite element formulations. Manna and Baidya [41] investigated vertical vibration of a full-scale pile experimentally as well as 2D FE analyses. Sarkar and Maheshwari [42] and Maheshwari and Sarkar [43] investigated the three-dimensional behavior of single pile and pile groups considering liquefaction of the soil medium with work hardening Drucker–Prager soil model.

Sarkar [44] developed a 3D finite element code in MATLAB with Drucker–Prager plastic cap model with Byrne [45] approach to evaluate the effect of soil plasticity and liquefaction on the behaviour of the soil–pile system. The present soil–pile system of Haldia is analyzed in the developed program considering single-pile configuration.

Taking the advantage of symmetry, only one-half of the actual model is built (it may be noted that a quarter model is not suitable for nonlinear analyses). This significantly improved the efficiency of computation. Radiation boundary condition is imposed at all the lateral boundaries of the soil–pile subsystem. Square cross section with each side equal to 0.5 m was used for piles. The length (L) of the piles considered is 26.5 m. The size of the full model was 52 × 26 m^2 in plan and 26.5 m height. In plan, elements of fine size are used near the pile and the mesh size is increased gradually towards the boundary. The element size is kept uniform in the vertical direction to allow for an even distribution of vertically propagating shear waves. Kelvin elements (Novak and Mitwally [46]) were used at the boundary of the soil–pile system as the radiation boundary condition. Drucker–Prager soil model has been used for soil plasticity with the properties shown in the Table 1. The Young's modulus values for the soil layers have been computed using empirical relationship based on SPT-N values proposed by Bowles [47]. Finite element mesh of the developed model is shown in Fig. 11. Various feature of finite element modeling may be found in Maheshwari and Sarkar [43] and Sarkar and Maheshwari [42]. The spectrum compatible time history for Haldia project site as generated and shown in Fig. 8b was applied as the input base acceleration for soil–pile system and the responses of the soil–pile system were computed.

The pile head displacement time history is shown in Fig. 12a. It may be observed that the pile cap displacement



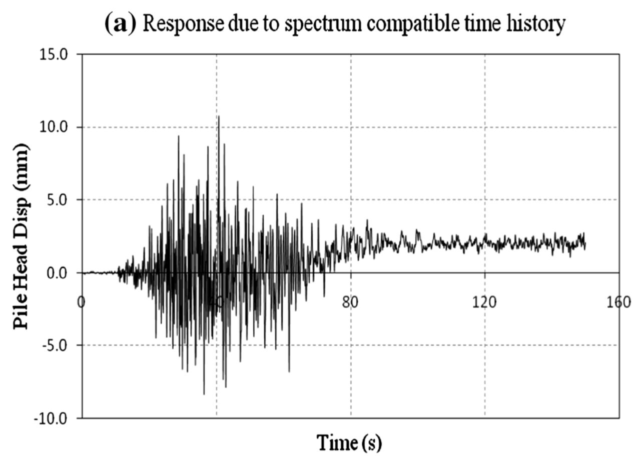


Fig. 12 **a** Pile head displacement time history for spectrum compatible time history. **b** Pore pressure ratio at depth 5 m

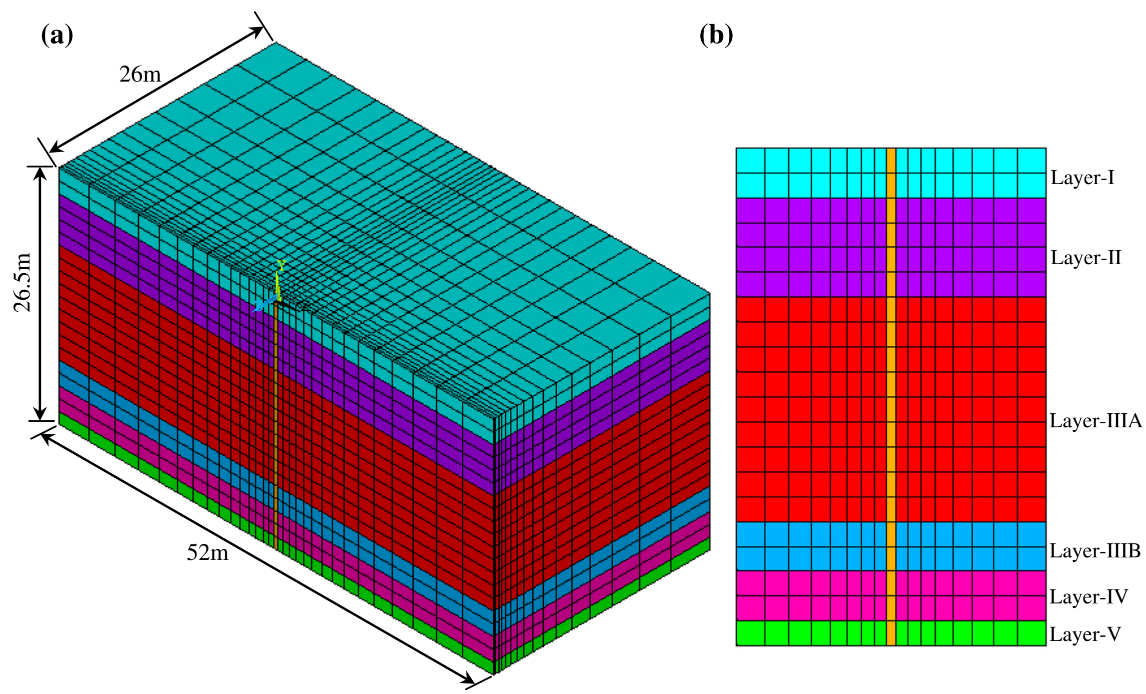


Fig. 11 **a** Three-dimensional FE model for a single pile. **b** Zoomed view of the pile with layer details

reaches maximum at around 40 s of the time history. The pore pressure ratios (ratio of excess pore pressure and effective stress) were calculated at different depths from the ground level for the spectrum compatible time history to investigate the extent of liquefaction along the depth. The soil medium is assumed to be liquefied when the pore pressure ratio reaches 1.0. The time history of pore pressure ratio at depth 5.0 m is shown in Fig. 12b. The pore pressure ratio reaches 1.0 at about 40 s of the earthquake indicating that the soil medium is completely liquefied at this time. The pile head displacement is also at the peak at this time (Fig. 12a). From the pore pressure time histories, it was also observed that the pore pressure ratio reaches 1.0 only up to a depth of 7.5 m from the ground surface. It means that the soil medium gets completely liquefied up to the depth 7.5 m for the applied time history. Permanent pile head displacement of about 2.5 mm is observed during post-liquefaction phase.

It may be noted here that the material behaviour is not affected by the orientation for the soil plasticity model considered here i.e. the model is applicable for isotropic material. The plastic cap model in general cannot predict the pore pressure build-up in the post-liquefaction phase. Hence study with more advanced soil plasticity model (viz. multi-surface plasticity model, bounding surface models etc.) is envisaged for better prediction of behaviour of soil–pile system under liquefying soil condition.

Summary and Conclusion

Recent research highlighted various limitations of the conventional pile design and calls for seismic requalification. It is shown that a typical requalification study involves generation of site specific synthetic input motion compatible with the seismic hazard, estimation of depth of liquefaction, time required to reach full liquefaction, estimation of inertial and kinematic loading on structure at various stages of the earthquake and the determination of the bending moment and shear force profiles under the expected loading, which will lead to checking against various failure mechanisms. This paper presents a comprehensive requalification study of a typical pile-supported foundation at a site in Haldia. Different failure mechanisms were evaluated using simplified and FE analysis. Dynamic behaviour of the soil–pile system is very much dependent on the time required to reach full liquefaction. To demonstrate the applicability of continuum approach in determining the effect of stages of liquefaction, a three-dimensional finite element model has been developed and analyzed for the spectrum compatible time history generated for the Haldia site. The methodology presented can be used to carry out similar studies.

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