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A Simplified Approach of Design Pile Foundation on Liquefiable Soil in India

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ABSTRACT

In India, piles are designed as per IS 2911:2010 (Part 1) for different soil conditions considering all the load criteria. But there is no such provision in Indian Standard Codes for designing piles in potentially liquefiable soils. In this paper, the design of pile foundation in liquefiable soil is discussed. The provisions for design of pile foundations in liquefiable soil from different codes of practice from different countries are studied, and the design approaches for the same are discussed based on those studies. The load bearing capacity of a pile in liquefied soil and non-liquefied soil is made based on the force equilibrium approach. A standardize graph which may be useful for practicing engineers has been plotted. Further, a comparative study is made based on the force equilibrium approach in Indian conditions.

KEYWORDS

Codal Provisions, Force-Equilibrium, IS 2911:(2010)Liquefaction, Load Bearing Capacity, SAP2000

INTRODUCTION

Deep foundations are adopted to support structural loads where the foundation soil is weak in a smaller depth and results bearing capacity failure and settlement problems. Piles are the popularly used deep foundations to support huge structural loads. In addition, piles must be resistive to the uplift loads due to hydrostatic or wind pressure. Piles are designed as per provisions of Indian Standard code (IS 2911:2010) for different soil conditions considering all the load criteria. But there is no such provision in Indian Standard Codes for designing piles in potentially liquefiable soils. Liquefaction is an earthquake induced phenomenon in soil strata with fine sand and silt. During potential earthquakes, soil behaves like liquefied one with lesser shear strength and starts flowing laterally. This phenomenon is called spreading of soil. Due to spreading a lateral load is applied on the piles. This lateral load produces bending and lateral deflection of piles (Unjoh et al, 2012). Besides, as there is lesser or no lateral support due to liquefaction, piles become critically susceptible to buckling. For these, well-designed structures with pile foundation collapse during different earthquakes despite being well designed considering high factor of safety. With increasing infrastructure growth and increasing earthquake activities researchers are giving more importance on this problem. Several researches were conducted by various researchers on the analysis and the design of pile foundations in liquefied soil

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and established different theories on this behalf (Ashour et al, 2011). Codes of practices available in other countries suggest some procedure for seismic design of pile foundations also (Ghosh et al, 2012).

In this paper, the behaviour of pile foundation in liquefied soil and the design of pile foundation in liquefiable soil are discussed. The provisions for design of pile foundations in liquefiable soil from different codes of practice are studied and the design approaches for the same are discussed based on those studies (Puri et al, 2008). Further, a comparison between the load bearing capacity of a pile in liquefied soil and non-liquefied soil is made based on the force equilibrium approach in Indian condition.

CODAL PROVISIONS FOR DESIGN OF PILES IN LIQUEFIABLE SOILS

Design codes from different countries adopted high values of partial factor of safety against plastic hinge formation. Those provisions suggested to design and construct the piles such a way that piles remain elastic to avoid subsurface repairment. But it is economical to allow a limited amount of yielding in the pile while designing. Details on specified codal provisions is presented below to access this design view.

Euro Code 8 (EN 1998-5:2004) Provisions

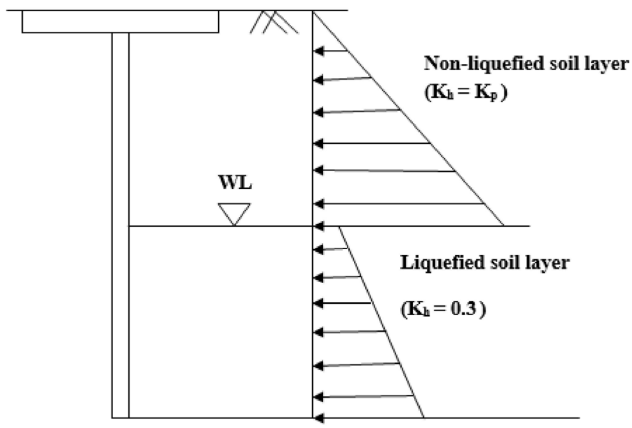
Euro code 8 prescribed that piles and piers should be designed to resist the action of two types of forces, one is Inertia forces from superstructure and second one is the kinematic forces. Kinematic forces are generated due to the deformation of surrounding soil during passing of shear waves. Bending moments are considered to be produced when there are consecutive soil layers with sharp change of soil stiffness in some high or moderately high seismic zone. Although the side resistance of soil layers that are susceptible to liquefaction or to substantial strength degradation is ignored. EN1998:5 suggested that pile designing should be done with a principle to remain elastic but may under certain conditions be allowed to develop a plastic hinge at their heads. According to Clause 5.8.4 the region of formation of potential plastic hinging is:

- A region of twice of pile diameters from the pile cap.
- A region of $\pm$ twice of pile diameters from any interface between two layers with excessive difference in shear stiffness (where ratio of shear modulus > 6).

JRA Provisions

The Japanese Highway Specification, JRA (2002) has suggested a new concept of “top-down” and “bottom-up” effects. The code has prescribed that “the design of piles against bending moment should be done considering that the non-liquefied crust layer exerts passive earth pressure on the pile and assuming the liquefied soil exerts 30% of total overburden pressure.” This statement interprets that the lateral earth pressure coming from the non-liquefied crust layer is equals to passive earth pressure. In this case the coefficient of earth pressure will be same as the coefficient of passive earth pressure. On the other hand, the coefficient of earth pressure that is considered for the liquefied soil layer is 0.3. Based on the shear parameters of soil, pressure diagrams and the total lateral load acting on the pile is estimated and the effect of that lateral load on pile is analysed. This estimation of pressure is based on back calculation of case histories of performance of pile foundations during the Kobe earthquake. The Japanese Code of Practice (JRA 1996) has incorporated this understanding of pile failure and is shown in Figure 1. This code also specify that the maximum bending moment is assumed to occur at interface between the liquefied and non-liquefied soil layer and suggested designers to check factor of safety against failure due to inertial force, bending moment and kinematic forces separately, not due to the combination of loads.

Figure 1. Diagrammatic representation of JRA (1996) provision



ASCE 7-10 Provisions

According to Clause 12.13.6.3 of American Society of Civil Engineers, "piles should be designed and constructed to withstand structural response and deformations from earthquake ground motions including both free-field soil strains (without the structure) and induced by lateral pile resistance to structural seismic forces, all as modified by soil pile interaction". This provision may be maintained only by calculating seismic forces with and without the superstructure and finally estimating the combined effect of those two seismic forces. This code also prescribes that bending moments, shear forces and lateral deflections in pile should be estimated for design considering the interaction of the pile shaft and surrounding soil layer (clause 12.13.6.7). According to this provision if the surrounding soil is potentially liquefiable then the soil stiffness degradation should be taken into account before going to estimate the stress resultants.

AASHTO (2010) Provisions

AASHTO (2010) provides a detailed explanation for liquefaction design requirements especially for piles under bridges. Clause 10.5.4.2 of AASHTO (2010) suggested that in Seismic Zone 3 and 4 if both the following conditions are present liquefaction assessment should be conducted.

- The groundwater level anticipated at the site is within 50 ft of the existing ground surface or the final ground surface, whichever is lower.
- Low plasticity silts and sands within the upper 75 ft are characterized by one of the following conditions: (1) the corrected standard penetration test (SPT) blow count, $(N_1)60$, is less than or equal to 25 blows/ft in sand and nonplastic silt layers, (2) the corrected cone penetration test (CPT) tip resistance, q_{ciN}, is less than or equal to 150 in sand, and nonplastic silt layers, (3) the normalized shear wave velocity, V_{s1}, is less than 660 fps, or (4) a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

For the sites which are found to be susceptible to liquefaction according to the above-mentioned criteria, the following effects should be considered for assessment.

- Loss in strength in the liquefied layer or layers.
- Liquefaction-induced ground settlement.
- Flow failures, lateral spreading, and slope instability.

With coefficient of acceleration larger than 0.5g, if liquefaction occurs then the bridge shall be designed and analysed for liquefied and non-liquefied conditions by the method mentioned below.

"Piles should in principle be designed to remain elastic, however, under certain circumstances a plastic hinge may be allowed to develop at the pile head, noting that this plastic rotation does imply that the piles and possibly other parts of the bridge will need to be replaced if these levels of deformation do occur". Specifically, for liquefaction induced soils, piles should be designed by allowing significant inelastic deformation (AASHTO 2010). In such cases the elastic moment capacity shall not be allowed more than a factor of 2. The code is not explicit about the using factored or unfactored loads, or whether lateral forces are considered or not or where liquefaction with no lateral spread is expected. It is also notified that pile group effects are not considered significant for liquefied soil.

DESIGN APPROACHES

The methods of design of piles in liquefiable soil require a reliable approach of calculating the effect of liquefaction on the pile foundation. The methods currently derived from the different codal provisions from different countries are:

1. The force equilibrium analysis; and
2. The displacement or p-y analysis.

The Force Equilibrium Analysis

Different Japanese codes recommend this approach for designing pile foundation in liquefiable soil undergoing lateral spreading. This approach involves estimation of lateral pressure of soil on pile and the pile response. The lateral soil pressure can be estimated by modifying the coefficient of lateral earth pressure from the history of effects of previous earthquakes. As an example, as per JWWA 1997 the lateral earth pressure coefficient for a liquefiable soil layer is considered as 0.3 considering the effect of Kobe Earthquake (Japan 1995).

The Load-Displacement or P-Y Analysis

P-Y analysis method involves incorporating Winkler type spring mass model. The post liquefaction free field displacements are estimated. It is assumed that this displacement varies linearly. Finally, the estimated displacements are applied to the springs which are designed to represent soil-pile interaction system (Finn and Thavaraj, 2001). Degraded p-y curves are used for such kinds of analysis. According to Japanese codal provisions linearly elastic-plastic springs are considered and the modulus of elasticity of soil can be determined using semi-empirical formulas (Finn and Fujita, 2004). Plate load test or standard penetration test is conducted to estimate the soil modulus. As per JRA recommendation (1996) reduced spring stiffness is considered to account for the effect of liquefaction.

DESIGN OF PILES ON THE BASIS OF FORCE EQUILIBRIUM APPROACH

From the first approach, the concept is to decrease the lateral supporting force from surrounding soil as the shear strength of soil reduces. But this approach is dependent on the case histories as the lateral earth pressure co-efficient of soil is calculated on the basis of the effects of previous earthquakes. Here in this calculation, the value of the co-efficient is taken as 0.3 (as for Kobe earthquake, Japan, 1995).

Plotting Standardized Curves

As per the Indian standard code for design and construction of pile foundations IS 2911 (2010) Part I, the pile load capacity is calculated for different diameters and depths of piles. The ultimate load capacity (Q_u) of piles, in kN, in granular soils is given by the following formula:

$$Q_u = A_p\left(\frac{1}{2}D\gamma N_\gamma + P_D N_q\right) + \sum_{i=1}^{n} K_i P_{Di} \tan\delta_i A_{si} \tag{1}$$

The first term gives end-bearing resistance and the second term gives skin friction resistance where:

A_p = cross-sectional area of pile tip, in m^2
D = diameter of pile shaft, in m
γ = effective unit weight of the soil at pile tip, in kN/m^3
N_γ = bearing capacity factors depending upon
N_q = the angle of internal friction, f at pile tip
P_D = effective overburden pressure at pile tip, in kN/m^2
$\sum_{i=1}^{n}$ = summation for layers 1 to n in which pile is installed and which contribute to positive skin friction
K_i = coefficient of earth pressure applicable for the ith layer
P_{Di} = effective overburden pressure for the ith layer, in kN/m^2
d_i = angle of wall friction between pile and soil for the ith layer
A_{si} = surface area of pile shaft in the ith layer, in m^2

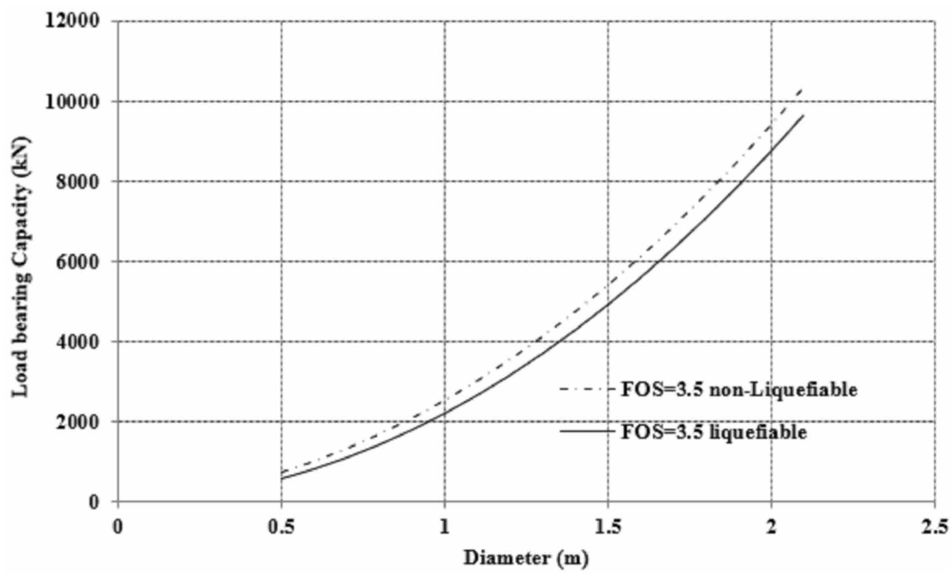
In this study, single piles are considered for analysis. These single piles are placed over a non-liquefied firm layer or hard rock and the overlain soil is considered to be loos-medium sand, saturated upto the ground surface. The ultimate load bearing capacity for two different piles one with 10 m length and variable diameter and another with 0.6 m diameter with variable length are calculated according to the formula given in equation 1. These values are plotted in two different graphs. Now, to compare the same with the load bearing capacity of piles in liquefiable soil following method should be followed. In case of liquefiable soil, the pile must also be able to resist the lateral loads and the moments caused during the earthquakes. One of the methods commonly used for the design of the piles for lateral loads is the Characteristic Load Method (Brettmann et al.1996).

Single piles are designed for the cases of without liquefaction and with liquefaction. The Characteristic Load Method developed by Brettmann et al. (1996) is used to design the pile for the estimated lateral load and moment in this study (Sadek et al.2004). This method is based on the pseudo-static approach and the formulations are based on non-dimensional relationships related to load and moment. In case of potential liquefaction, during design of pile the following conditions are considered:

1. The skin friction resistance of the pile is considered as 30% during liquefaction.
2. According to the study of Bhattacharya and Bolton (2004) due to loss of lateral support to the pile in the liquefied soil layer, the pile is essentially designed as a column against buckling. They suggested the minimum pile diameter required based on thickness of the liquefiable layer.
3. The bending moment capacity of the pile have to be increased as there are excessive extra pressure acts on the pile due to the movement of non-liquefied soil over the liquefied soil. Additional moment capacity can be achieved by increasing the reinforcement or by increasing the pile section.
4. The pile is considered to be placed on non-liquefied layer or firm layer; so the end bearing capacity is unchanged after liquefaction.

Considering above-mentioned points, the calculation has been done for the pile load capacity for different diameters and depths of the pile considering the soil is in liquefiable soil layer during

Figure 2. Variation of pile load bearing capacity of a 10 m long pile for different pile diameter



earthquake. The results have been plotted with respective graphs of non-liquefiable soils. The variation of load bearing capacity of pile with diameter and depth are shown below.

The plot in Figure 2 shows the variation of pile load capacity with diameter of a single pile of 10m deep and plot in Figure 3 shows the same with depth of a single pile for a 0.6m diameter pile.

Now, to emphasise this result obtained from the above concept a case is taken as an example to design the pile under a structure in potentially liquefiable soils. Further the same problem will be solved for non-liquefiable soil to compare the result. In this study a simple four storey building is taken as a model for this purpose.

Model of the Building and Various Parameters Considered

To calculate the loads that act on a structure during an earthquake occurrence, a typical multi storied building frame model is considered. The building frame is a moment resisting frame with reinforced concrete members. The plan and elevation of the concrete building frame considered are shown in figure 4. Here only a four-storied frame structure is modelled with equivalent dead and live load. The parameters used for the modelling of the building were based on the values used in general practice during the construction of a residential complex. Suitable cross-sectional dimensions of beams and columns were assumed (all in accordance with the Indian standards). The assumed values are shown in the Table 1. The grade of concrete and the grade of steel were considered to be M25 and Fe415 respectively. The modelling of the building without the staircase was done in the computer program SAP2000 with the assumed geometry and material properties. SAP2000 is a structural analysis and design software which is used to perform static or dynamic, linear or non-linear analysis of structural system by finite element program. The model is analysed statically. The default constitutive model for concrete is used for this purpose which is linear elastic model. In this default constitutive model, the Section Designer automatically generates concrete stress-strain curves. By default, these curves provide compressive strength. Stress-strain curves may be reviewed within the Section Designer by using the concrete model View controls. To model the tensile strength of concrete, a material stress-strain curve may be converted into a user-defined constitutive model by selecting Materials.

Figure 3. Variation of pile load bearing capacity of a 0.6 m diameter pile for different depth of piles

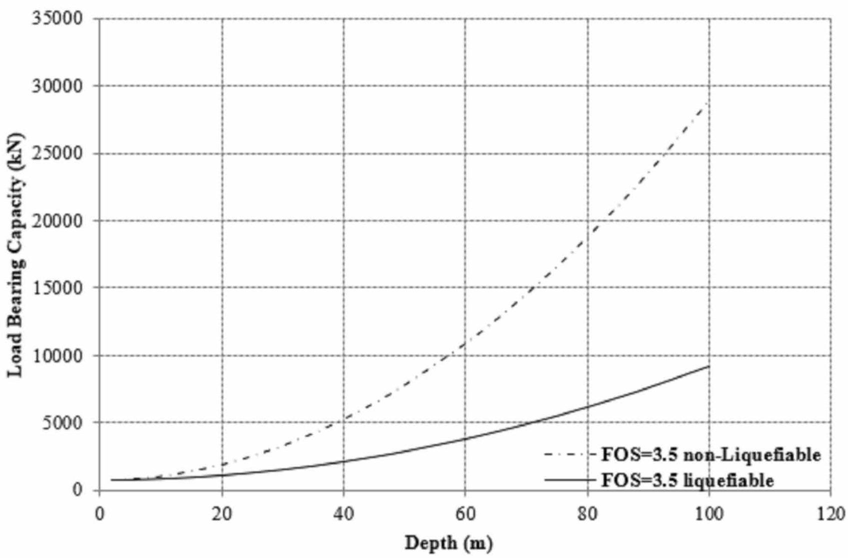


Figure 4. Model of the building in SAP platform

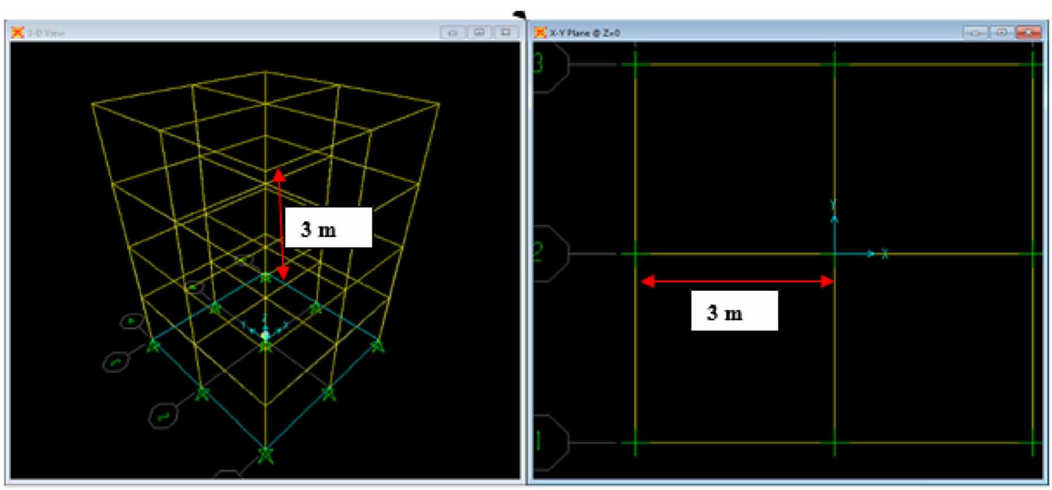


Table 1. Dimensions of the members of the RC Building Frame

Members	Length	Dimensions
Beams	3 m	300 mm x 300 mm
Columns	3 m	500 mm x 500 mm

The seismic weight of the building (W_s) is calculated as the total dead load plus one-fourth of the imposed load. The seismic weight of each floor of the structure is calculated to be 500kN and that of the roof to be 400kN. Then the seismic weight of the entire structure is four times the seismic weight of each floor plus the seismic weight of the roof. Thus, the seismic weight (W_s) of the considered structure is 3600kN.

Estimation of Seismic Loads on the Structure

It is important to estimate the loads those are being transferred to the foundation during an earthquake. These loads are used to design the pile foundation efficiently for a potentially liquefiable soil. These loads depend on the seismic loads that act on the super structure during an earthquake. Different codes around the world propose different methods of estimation of these seismic loads on the super structure. The methods proposed by the Indian standard (IS 1893) is reviewed and used to estimate the seismic loads. A case study of a typical multi storied structure is considered as a model super structure for this purpose.

Calculation of Seismic Loads as Per IS 1893

The total lateral force that acts at the base of the structure during an earthquake is called the design seismic base shear (V_B). As per IS 1893, base shear is calculated using the equation 2:

$$V_B = A_h \times W_s \tag{2}$$

The seismic weight of the structure (Ws) is as calculated above. The design horizontal seismic coefficient (A_h) is a function of the soil type (its stiffness and damping), the time period of the structure and the zone. Equation 3 is being used to calculate the design horizontal seismic coefficient:

$$A_h = \frac{Z \times I \times S_a}{2 \times R \times g} \tag{3}$$

The Zone factor 'Z' which is indicative of the effective peak ground acceleration, is assumed as 0.16 (as for Durgapur in zone III), 0.24 (as for zone IV) and 0.35 (as for zone V). The values for the Importance factor 'I', which depends on the functional use of the structure, are given in Table 6 of IS-1893. Considering the present structure as an important service and community building, the value of 'I' adopted is I = 1.5. The Response Reduction factor 'R', depends on the perceived seismic damage performance of the structure, characterized by brittle or ductile deformations. From Table 7 of the code, the value of R for a special moment resisting frame is taken as R = 5. The value of the average spectral acceleration coefficient 'Sa/g' depends on the soil type,time period (T) of the structure and the damping ratio. The time period of the structure is calculated for a RC frame building using the equation 4 as per IS code:

$$T = 0.075 \times h^{0.75} \tag{4}$$

The time period for the building frame considered with height (h) = 11m, T = 0.453s. Assuming 5% damping the value of spectral acceleration $\left(\frac{S_a}{g}\right)$ from acceleration response spectra = 2.5. The base shear acting on the structure is calculated and the value is 216 kN, 324 kN and 472.5 kN for three zones respectively. The base shear acting on each storey is calculated from equation 5:

$$Q_i = V_B \times \frac{W_i \times h_i}{\sum_{j=1}^{n} W_j \times h_j} \tag{5}$$

The results obtained from the calculations using above mentioned equations are shown below in tabular form. Table 2 shows the different values of factors and the obtained value of base shear. Table 3 shows the base shear acting on each storey in different earthquake Zones.

Foundation Loads

To determine the loads acting on foundation, building frame shown in 4 is taken subjected with normal loads and earthquake loads or seismic load calculated above. For structural analysis of theses building frame model, computer program SAP2000 is used. The analysis is performed for the dead, live and the earthquake loads for various load combinations prescribed in the code. The results of the analysis consisted of the forces, displacements and reactions of all the members of the structure. The results are sorted to find the maximum load that is transferred to the foundation of the system. Table 4 shows the maximum (design) loads transferred to the foundation in each case. Where 'P' is the axial load, 'V' is the lateral force and 'M' is the moment.

Table 2. Values of different factors and base shear

ZONE	Z	I	R	Sa/g	A_h	W_s	V_b (kN)
III	0.16	1.5	5	2.5	0.06	3600	216
IV	0.24	1.5	5	2.5	0.09	3600	324
V	0.35	1.5	5	2.5	0.13125	3600	472.5

Table 3. Values of base shear acting on each story in different earthquake zone

ZONE III				ZONE IV				ZONE V			
W_i (kN)	H_i (m)	$W_i.h_i$ (kN-m)	Q_i (kN)	W_i (kN)	H_i (m)	$W_i.h_i$ (kN-m)	Q_i (kN)	W_i (kN)	H_i (m)	$W_i.h_i$ (kN-m)	Q_i (kN)
900	2	1800	39.27273	900	2	1800	58.90909	900	2	1800	85.90909
900	3	2700	58.90909	900	3	2700	88.36364	900	3	2700	128.8636
900	3	2700	58.90909	900	3	2700	88.36364	900	3	2700	128.8636
900	3	2700	58.90909	900	3	2700	88.36364	900	3	2700	128.8636
	total	9900			total	9900			total	9900	

Table 4. Maximum values of axial load, lateral load and moment used for design

Zone	Max P (kN)	Max V (kN)	Max M (kN)
III	405	59	56
IV	677	105	107
V	918	167	166

Pile Design

Now from the standardized plots, required pile dimensions can be obtained. Here, a factor of safety 3.5 is considered in this regard. Then, from the plots the required diameter and depths are obtained. From the standardized plot in Figure 1 required diameter is obtained for a 10m deep pile. The result shows that the required diameter for a 10m long pile is 0.505m in case of non-liquefiable soil whereas 0.575m in case of liquefiable soil for the same load and same seismic zone (Zone III). Similarly, we can determine required diameter for a 10m pile for other earthquake zones. In case of Zone IV and Zone V the required pile diameter for the given condition are 0.71 m and 0.864 m respectively for liquefiable soil. Whereas in case of non-liquefiable soil 0.64m and 0.79m pile diameter is acceptable for Zone IV and V respectively shown in Table 5. From the standardized plot in Figure 2 required depth is obtained for a 0.6m diameter pile. Here, the result shows that the required depth for a 0.6m diameter pile is 2.6 m in case of non-liquefiable soil whereas 4.7 m in case of liquefiable soil for the same load and same seismic zone (Zone III). For the other earthquake zones calculations are done similarly and shown in Table 6.

The values of factored load and designed parameters are shown in Table 5 and Table 6.

Further, the pile performance is studied for different zones according to Indian standard provision. Considering three zones (zone III, IV, V) acceptable diameter for variable seismic weight of building is studied and the graphical representations are shown in the figures shown below. In Figure 6, the variation of diameter of a single pile with respect to seismic weight in Zone III (for a pile depth of 10m) is shown. In this figure a comparison between liquefiable soil and non-liquefiable soil is made. The same results for Zone IV and Zone V are shown graphically in Figure 5 and Figure 6 respectively. It is clearly shown that for a certain depth of pile (here 10m) the difference in diameter required for liquefiable soil over non-liquefiable soil is higher for lower earthquake zone.

All the previous curves are representing pile load bearing capacity for different seismic weight of superstructure when full soil layer through the pile depth is liquefied. But in practice, often limited liquefaction is observed when a part of the soil layer is liquefiable and pile load bearing capacity is affected by the liquefaction of that part of soil. For that reason, in this section a parametric study is carried to present the effect of liquefiable soil thickness on pile load bearing capacity. The plot

Table 5. Required diameter for a 10m pile

ZONE	Factored load on the pile (kN)	Required diameter for a 10m deep pile in non-liquefiable soil (m)	Required diameter for a 10m deep pile in liquefiable soil (m)
III	756	0.504	0.575
IV	1134	0.64	0.71
V	1653.75	0.79	0.864

Table 6. Required depth for a 0.6m diameter pile

ZONE	Factored load on the pile (kN)	Required depth for a 0.6m dia pile in non-liquefiable soil (m)	Required depth for a 0.6m dia pile in liquefiable soil (m)
III	756	2.6	4.7
IV	1134	12	21.7
V	1653.75	18.1	33

Figure 5. Variation of diameter of piles with seismic weight of structure for Zone III

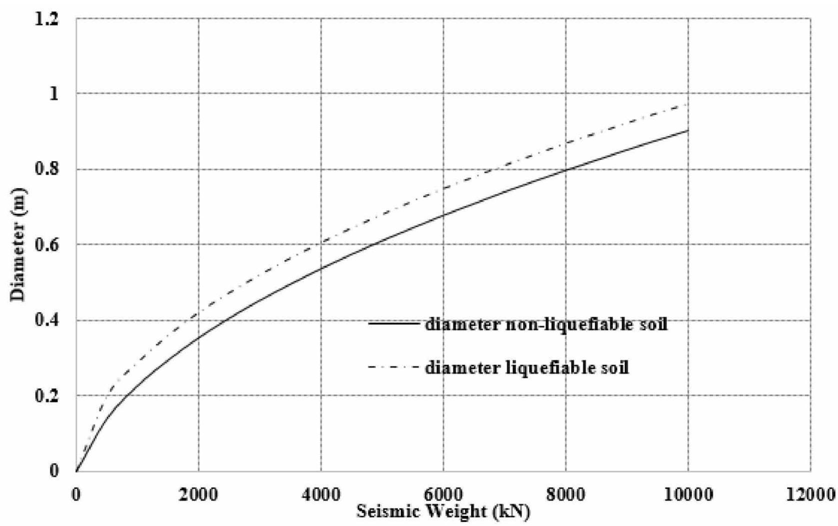
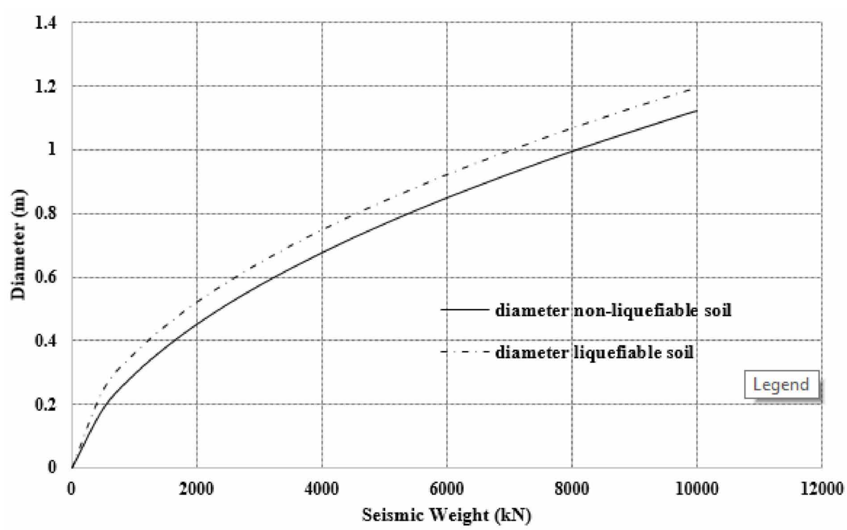


Figure 6. Variation of diameter of piles with seismic weight of structure for Zone IV



shown in Figure 8 represent the curves for variation of pile load capacity for different liquefiable soil thickness with pile diameter.

LIMITATIONS

In the above study, a simplified approach is followed to present a useful way to design pile foundation. From the curves presented above, the required design parameters like depth and diameter may be determined for a specific liquefiable soil thickness. However, some assumptions and limitations are considered through the study:

Figure 7. Variation of diameter of piles with seismic weight of structure for Zone V

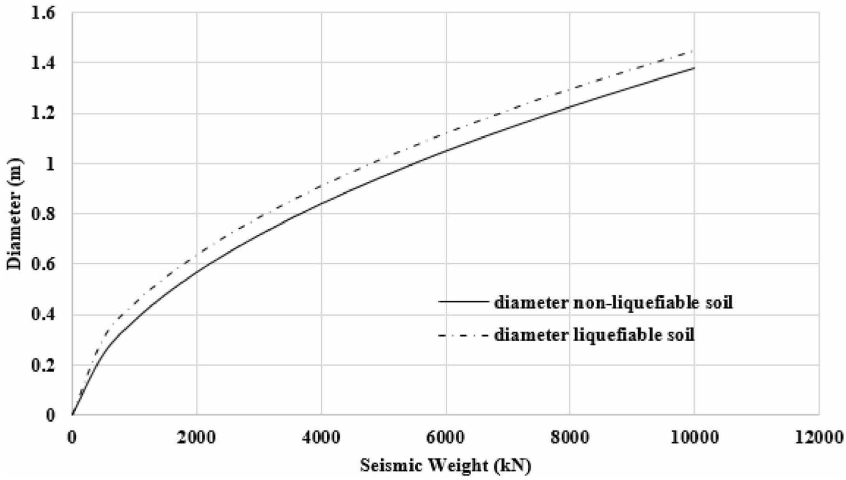
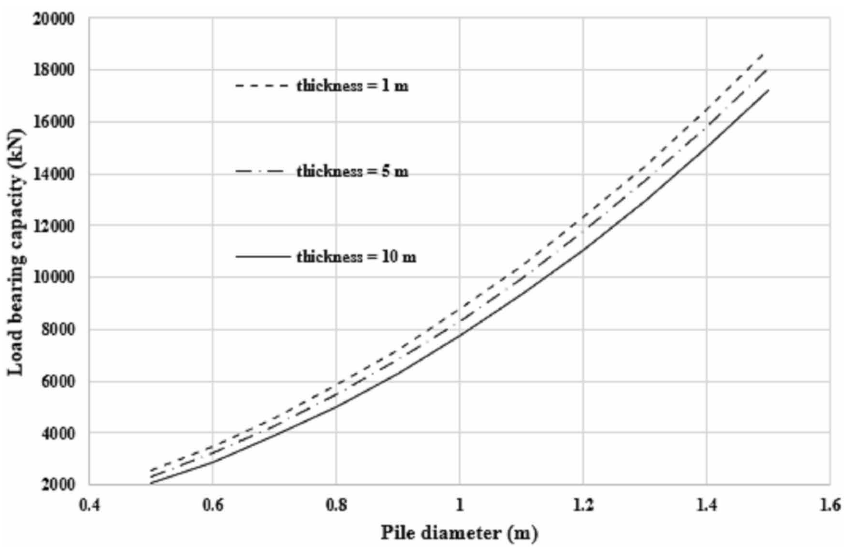


Figure 8. Variation of pile load bearing capacity with pile diameter for different liquefiable soil thickness



1. The single pile considered for the analysis is assumed to be placed on a firm non-liquefied soil stratum. Modification of end bearing capacity due to liquefaction is not considered in the study.
2. The load carrying capacity that is determined for different parameters are totally valid only for different seismic zones of India.
3. Though the lateral load carrying capacity of pile is an important parameter, this is not considered in the study.

DISCUSSION

From the above study, it is clear that for a certain depth of pile, diameter should be more for a given site condition in liquefiable soil. For a certain diameter of pile more embedded length is required for liquefiable soil. To withstand the extra bending moment and buckling in liquefied soil, we should consider the soil stiffness degradation before going to design a piled foundation in some potentially liquefiable soil. Indian standard codes for design of pile foundation IS 2911 (2010) does not prescribe any formulation for liquefiable soil. Structural failure of piles passing through liquefiable soils has been observed in many recent strong earthquakes. This suggests that the bending moments or shear forces those are experienced by the piles exceeding those predicted values estimated from the formulations prescribed in IS 2911. The limitations of the Indian Standard for pile design may be summarized as follows:

1. IS 2911 (2010) for pile design assumes that the pile remains in stable equilibrium condition and does not buckle as in case of liquefaction induced lateral spreading. In other words, the code ignores the structural nature of pile.
2. The effect of axial load that generates during liquefaction is ignored in this code.
3. There is no consideration of liquefaction criteria in the specifications or formulations prescribed for pile design.

To overcome this disadvantage the proposed standardized plots may be used to design the piles in liquefiable soil. The method used to formulate the pile design in liquefiable soil is totally based on the codal provisions from different countries. In this study, the formula given in IS 2911 (2010) part 1 to calculate bearing capacity of a piled foundation is modified with the help of the codal provisions from other countries. According to JRA provision, the lateral pressure coming from the soil as a support to the pile is reduced to 30% for liquefiable soil. Along with that, the pile is designed as a column to resist the buckling produced due to support reduction in liquefied soil. According to the estimated pile deformation, pile diameter or thickness is provided. Finally, structural reinforcement is increased in concrete pile to withstand the increased bending moment liquefied soil.

CONCLUSION

From the recent earthquake phenomenon, an efficient design of pile foundation was required to resist the estimated earthquake loads. From this point of view, there are different codal provisions for designing the piles in liquefiable soils in different countries. In Indian codes for design of piles there is no provision for liquefiable soil. From this study it may concluded that the pile diameter required for any estimated structural seismic load in liquefiable soil can be found from the graphical plots given in this study. Finally, there are three graphs plotted on the basis of performance study. From these plots required diameter can be calculated directly from estimated seismic weight of structure. The above graphical plots may be used satisfactorily in this regard.

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