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Collapse Load Analysis of Reinforced Concrete Pile Group in Liquefying Soils under Lateral Loading

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Abstract**—**The ultimate load analysis of RC pile groups has assumed a lot of significance under liquefying soil conditions, especially due to post-earthquake studies of 1964 Niigata, 1995 Kobe and 2001 Bhuj earthquakes. The present study reports the results of numerical simulations on pile groups subjected to monotonically increasing lateral loads under design amounts of pile axial loading. The soil liquefaction has been considered through the non-linear p-y relationship of the soil springs, which can vary along the depth/length of the pile. This variation again is related to the liquefaction potential of the site and the magnitude of the seismic shaking. As the piles in the group can reach their extreme deflections and rotations during increased amounts of lateral loading, a precise modeling of the inelastic behavior of the pile cross-section is done, considering the complete stress-strain behavior of concrete, with and without confinement, and reinforcing steel, including the strain-hardening portion. The possibility of the inelastic buckling of the individual piles is considered in the overall collapse modes. The model is analysed using Riks analysis in finite element software to check the post buckling behavior and plastic collapse of piles. The results confirm the kinds of failure modes predicted by centrifuge test results reported by researchers on pile group, although the pile material used is significantly different from that of the simulation model. The extension of the present work promises an important contribution to the design codes for pile groups in liquefying soils.

Keywords**—**Collapse load analysis, inelastic buckling, liquefaction, pile group.

I. Introduction

PILES are considered to be the most promising deep foundations to support superstructures, when the underlying soil is having low bearing capacity. Along with the axial compressive loads of superstructures, piles also have to resist the lateral loads due to seismic activities in earthquake prone areas which lead to the liquefaction of soil. In such cases, the collapse load analysis of piles under liquefaction for lateral loading is quite important.

One of the most challenging issues in the field of geotechnical engineering is liquefaction, which can cause immense damage to structures during the earthquakes. Kramer [1] studied that liquefaction was observed as a chief cause of damage to the pile foundations during the post-earthquakes studies of the 1964 Niigata and 1995 Kobe earthquakes

Many previous researches [2], [3] have proved that under seismic loading, piles performance becomes a complex problem due to excess pore water pressures as well as due to decrease in the stiffness of saturated soils. Past earthquake studies prove that the most of damage in liquefiable soils has occurred due to inertial forces on piles and to lateral forces on the ground.

The study of Wahidy [4] proved that during the designing of reinforced concrete piles, if the length to diameter ratio for a pile is less than 30; in such cases, the stress on pile is always less than Euler's stress, so there is no chance of buckling failure and the pile section which is designed to resist the expected bending moments will also be safe against buckling and formation of plastic hinge.

The Beam on Nonlinear Winkler Foundation (BNWF) model is the most versatile used soil-pile interaction model which assumes soil to be a set of springs attached to the pile along the whole length of piles. Apart from the accelerations and seismic forces suggested by design response spectrum given in IS 1893:2002-Part I, Emani et al. [5] and Maheshwari et al. [6] suggested that structures supported on pile foundations are needed to be designed for larger accelerations and seismic forces as compared to other types of foundations.

For seismic loading, the lateral soil springs referred to as p-y springs, plays a vital role in determining the response of piles. Dash et al. [7] observed that these p-y relationship curves vary with excess pore water pressure, which in turn influences the lateral pile response subjected to displacement loading.

In the centrifuge model test, Lombardi et al. [8] has found that at low strain level, the liquefied soil shows negligible stiffness and strength but at large strain level, liquefied soil gains stiffness and shows strain-hardening behavior. This behavior is also needed to be included in the exact modeling of p-y curves for soil springs. Wang et al. [9] has found that numerical calculations with the implementation of damping is sufficient for modeling the preliminary centrifuge data, but for evaluating the reliability for BNWF procedures, physical model data is also required.

During elastic modeling of piles, generally beam elements are modeled with linear force-displacement relationship assuming zero tensile strength of concrete. Bosco et al. [10] studied that with constant axial forces and increasing bending moment, in the initial stage, the moment-curvature relationship may be linear but with variation of compressed part of section in later stages, the moment-curvature relationship becomes non-linear and results an inconsistent effective moment of inertia. This effect of tension shift provides a more realistic force-displacement relationship.

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II. Description of the Model

The results reported in this paper are of a socketed 2-pile group connected by a relatively rigid pile cap embedded in liquefiable soil strata and subjected to monotonously increasing static lateral load, under service axial load conditions.

Piles: The piles are RC piles, in which M25 grade concrete and Fe 415 grade steel are used. Each of the piles has an outer diameter of 0.5 m and is 10 m long (Fig. 1). The concrete cover to the confining reinforcement is 50 mm. The main longitudinal reinforcement is 0.82%. The confining reinforcement and its effect on the stress-strain behavior of confined concrete is described in Fig. 2 [11]. A value of Z = 52.6 describes the confinement of concrete in the RC pile.

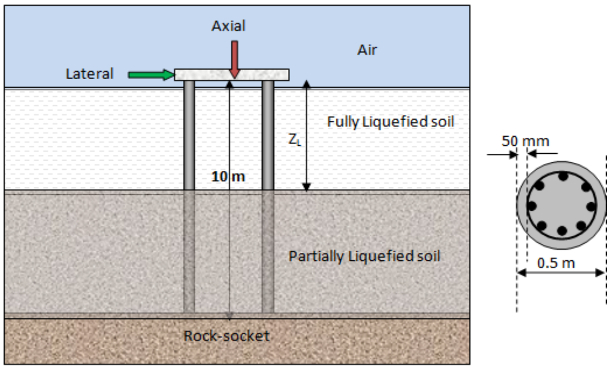

Fig. 1 Description of soil layers and embedded pile group

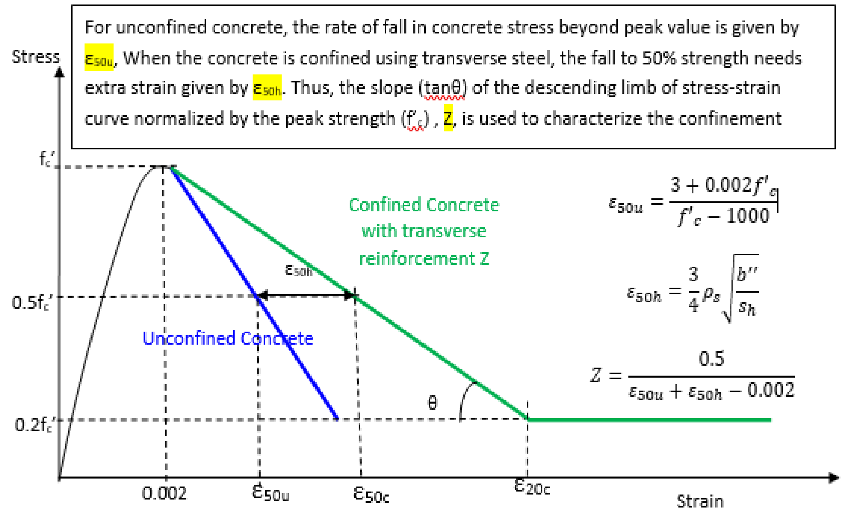

Fig. 2 Stress-strain behavior of confined concrete [11]

The moment-curvature relationship for such circularly confined concrete reinforced with strain-hardening steel reinforcement shall be as shown in Fig. 3.

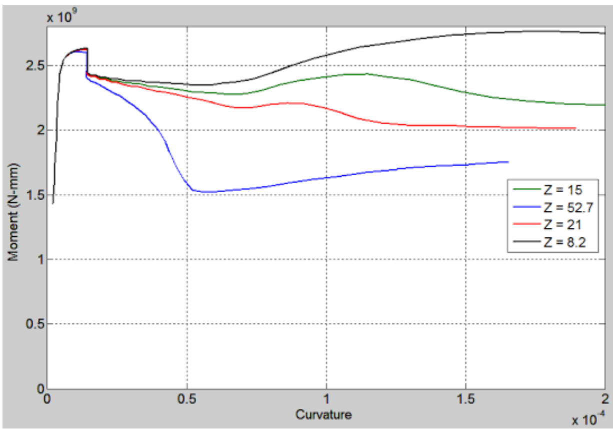

Fig. 3 Moment-Curvature relationship for the pile section under various confinements

The onset of strain-hardening of compression and tension steel can be seen from the plot. The moment-curvature behavior is also a function of the axial load, which can be assumed not to vary during seismic loading. The results reported in the present work are for High Yield Strength Deformed Bars (HYSD), and so the strain hardening effects are not present.

Soil conditions: The soil surrounding the pile is assumed to be nonlinearly elastic. The nonlinear p-y curves for the soil springs are obtained as per the API guidelines, but modified to consider the change in behavior under liquefaction. The ultimate soil resistance (P_u) is obtained using API formula for sand having internal friction of 30°, mass density of 1800 kg/m^3, and subjected to non-cyclic loading. Dash et al. [7] stated that the nonlinear p-y curves for the soil stiffness also varies based on the extent of liquefaction. The zone of liquefaction (Z_L) governs the ru-profile, and based on the ru-value the p-y curves are as given in Fig. 4. For the results presented herein, a depth of liquefaction Z_L =1 m, 3 m and 7 m is assumed to be completely liquefied.

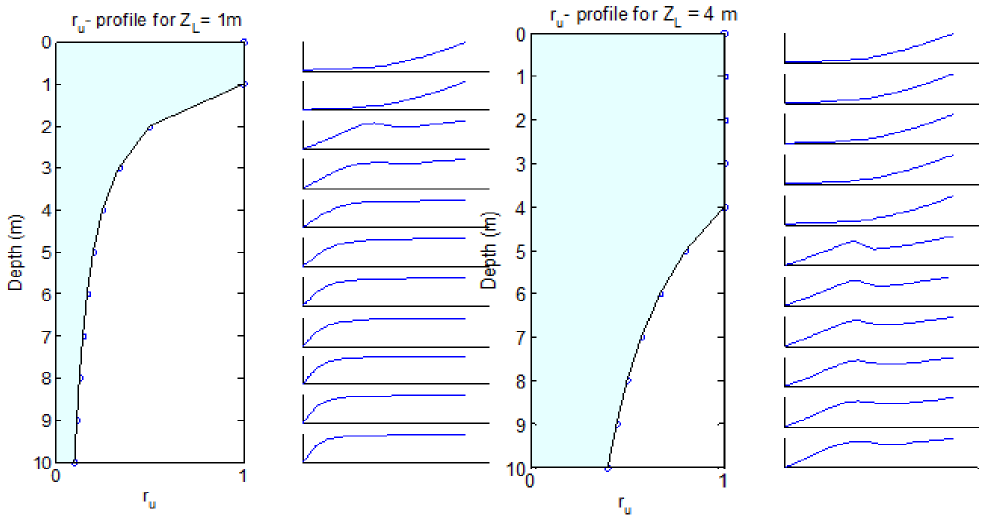

Fig. 4 Nonlinear p-y curves at different depths of liquefied strata

Loading: The pile group is subjected to static nonlinear loads, so that the analysis results are of some relevance to seismic conditions and liquefaction. Only the lateral load is varied, while the axial load is kept constant at $0.3f_{ck}D^2$, where f_{ck} is the characteristic strength of a concrete pile and D is the diameter of a pile. Basically, a push-over analysis is performed on the pile group while considering the possibility of formation of the failure mechanism. The failure is identified as the case for which convergence could not be reached.

III. Numerical (Finite Element) Modeling of Pile- Soil System

The pile-soil system is modeled in ABAQUS 6.13 software, as Beam on Non-linear Winkler Foundation (BNWF), where both the piles of the group are modeled as 2-noded cubic beam elements. The cap is also modeled as a rigid beam element. For ensuring plastic section behavior of the pile cross-section, rotational springs with desired moment-rotation properties are used at 0.5 m spacing, between the elastic beam elements. Also, the soil springs are modeled as axial connector elements with nonlinear elastic behavior. For a monotonic loading, the non-linear elastic behavior simulates the inelastic behavior as there will be no unloading.

IV. Results and Analysis

The collapse load is identified during Riks analysis [12], [13] as an unstable state, wherein numerical convergence is not achieved. This can happen due to (i) formation of enough number of plastic hinges leading to development of a mechanism, (ii) buckling of both the piles, or (iii) a combination of plastic hinge formations and buckling of individual pile(s). The collapse load for the three degrees of liquefaction is shown in Fig. 5.

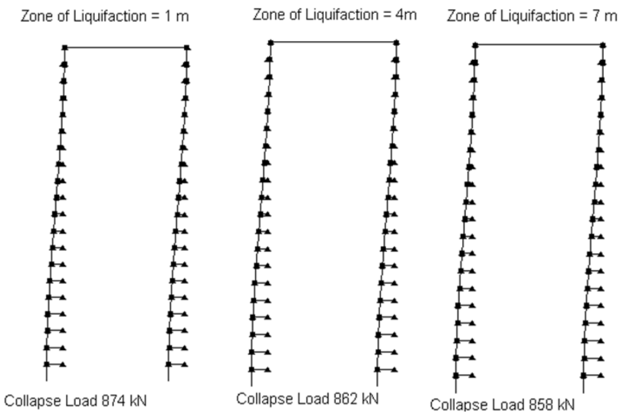

Fig. 5 Deformed shape just before collapse for various degrees of liquefaction

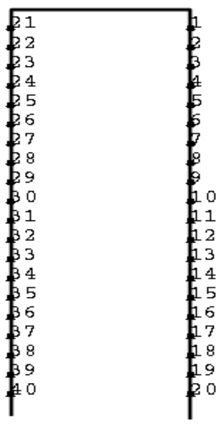

Fig. 6 Locations of connector joints between beam elements

Since the elastic beam elements are connected through the connector joints having similar moment-rotation relationship as of beam elements, the joints will serve as potential critical locations along the length of piles where plastic hinges can be formed. Fig. 6 shows the connector labels along the pile group, whereas Table I show the moment generated at these connector joints for various depths of liquefaction. In this case, the bending moment at the top and end connectors is almost equal to the plastic bending moment (Mp), so these locations along the piles can be considered to be behaving like plastic hinges under the given soil conditions.

TABLE I
BENDING MOMENTS AT CRITICAL LOCATIONS IN PILES AND LOCATIONS OF PLASTIC HINGES

Connector Joint No.	Moment (kN-m)		
	Z_L= 1m	Z_L= 4m	Z_L= 7m
1	**2256.487**	**2255.831**	**2254.917**
2	2055.156	2058.086	2058.526
3	1844.072	1850.578	1852.367
4	1620.804	1630.916	1634.065
5	1385.994	1400.514	1405.022
16	-1338.06	-1349.14	-1352.48
17	-1556.49	-1575.66	-1582.67
18	-1763.97	-1791.83	-1802.62
19	-1959.47	-1996.31	-2010.9
20	**-2142.01**	**-2187.83**	**-2206.17**
21	**2324.003**	**2323.643**	**2323.983**
22	2067.617	2071.838	2073.311
23	1815.705	1822.741	1824.819
24	1566.012	1575.945	1578.655
25	1317.395	1331.107	1334.471
36	-1276.22	-1286.57	-1289.55
37	-1507.49	-1525.51	-1531.87
38	-1739.92	-1766.36	-1776.28
39	-1974.12	-2009.52	-2023.1
40	**-2210.71**	**-2255.37**	**-2272.67**

Note: $\mathbf{M_p}$**= 2324 kN-m**

V. CONCLUSION

The above study represents a finite element approach for collapse load analysis of reinforced concrete piles groups which are subjected to lateral loading under varying liquefaction conditions. Through this work, the critical locations along the piles have been identified which are susceptible to behaving as plastic hinges and various modes of failures can be observed. This study can be further extended and results can be used for designing foundations of structures rested on piles in areas susceptible to liquefaction. With the capability of modeling foundation settlement, further modes of failure can be identified, especially for slender piles made of hollow steel sections.

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