

PORE WATER PRESSURE GENERATION AND DISSIPATION NEAR TO PILE AND FAR-FIELD IN LIQUEFIABLE SOILS

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ABSTRACT: The degree of liquefaction as characterised by the excess pore water pressure plays an important role in defining soil strength and stiffness. The pile-soil interaction in liquefiable soil, if modelled using BNWF model, the strength and stiffness of the soil springs can be suitably reduced by using a reduction factor. This reduction mainly depends on the soil type, its SPT/CPT value and the degree of liquefaction. Ideally this reduction should be based on the excess pore water pressure near the pile. However, it is difficult to estimate the degree of liquefaction near the pile. Hence, the lateral resistance of liquefied soil at soil-pile interface is normally characterized by the degree of liquefaction expected in the soil at the site without considering the influence of pile. Though, excess pore pressure near to the pile could be the governing parameter of soil resistance, it is hard to characterize the expected value of it in a field condition, as it depends on many parameters including soil type, shear loading, pile dimension, gap formation near to pile that facilitates easy dissipation of excess pore water pressure (EPWP), soil densification during pile driving, etc. Hence, to understand the difference between the far-field and near-pile response of liquefied soil, one high quality centrifuge test results are studied in this paper. The pattern of excess pore water pressure generation and development has been compared for both near-pile and far-field. The results are critically reviewed and discussed in this paper.

Keywords: Pile foundation, Liquefaction, Pore water pressure, Earthquake

1. INTRODUCTION

Pile foundations are long slender structural elements mostly used for foundations in weak soils, including seismically liquefiable areas. It is routinely used to support foundations bridges and high rise buildings where the loads are very high and the top soil is weak.

To define strength and stiffness of liquefied soil, the degree of liquefaction as characterised by the excess pore water pressure plays an important role. The pile-soil interaction, if modelled using BNWF model, the strength and stiffness of the soil spring can suitably be reduced by using a reduction factor. This reduction mainly depends on the soil type, its SPT/CPT value and the degree of liquefaction. Ideally this reduction should be based on the excess pore water pressure near the pile. The liquefaction potential of the soil close to pile is affected by the factors that influence the local variations of stress and strain around the piles (e.g., flexibility of pile foundation, pile installation method, ground motions, lateral spreading displacements, pile diameter, etc. However, as it is difficult to estimate the degree of liquefaction near the pile, the strength and stiffness of the liquefied soil at soil-pile interface is normally characterized by the degree of liquefaction expected in the soil at the site without considering the influence of pile [1].

It is often observed that the excess pore water pressure close to pile stays lesser than that occurs at free field for similar shear loading ([2], [3]) in 1g model tests. Several centrifuge tests have also been carried out by [4] and [5] to study the effect of structure on liquefaction potential of the underlying soil, which showed that the pore water pressure built-up was quicker at free field than below the structure. Few field cases have also been reported by [6] and [7], which suggest apparent increase in liquefaction resistance due to the presence of structure.

In contrast to the above observation, Liu and Quio [8] have suggested that the conditions for liquefaction are worse near to a structure than free field in many situations, based on a back-calculated damage investigation following Tangshan, China earthquake of 1976. Rollins and Seed (1990) [1] have also showed that the cyclic stress ratio induced by earthquake near a building may be altered by at least two factors like (1) change in vertical effective stress in soil due to building load and (2) soil-structure interaction. They have shown that when the expected normalized spectra acceleration value (S_a/a_{max}) is higher than 2.4, the liquefaction potential is greater beneath the building than free field (Fig 2).

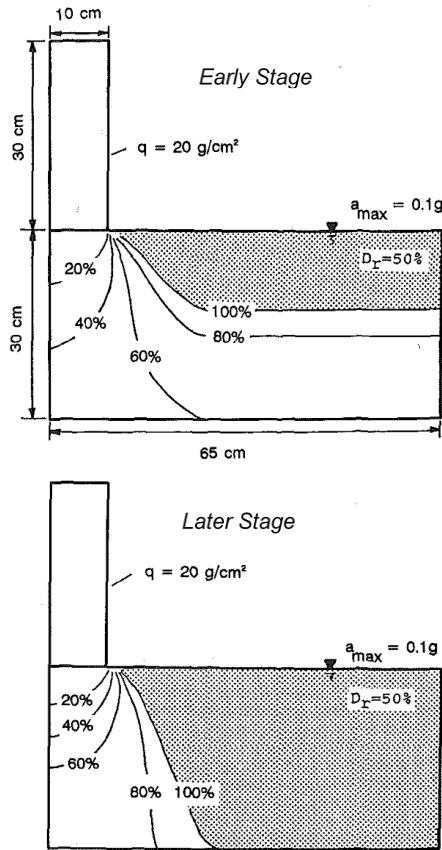


Fig. 1 Measured Pore water Pressure Ratio Development at free field and under a structure using Shake Table Test [3].

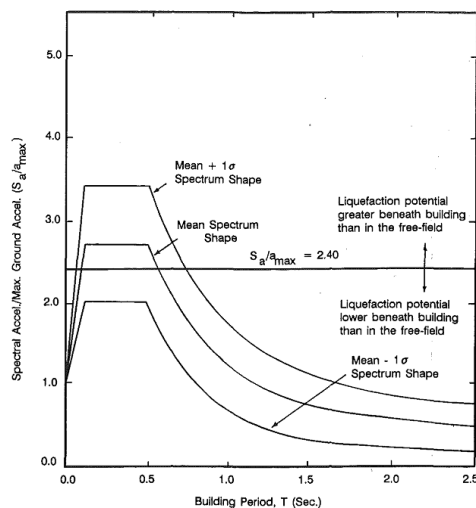


Fig. 2 Liquefaction potential of soil for various spectral acceleration values [1].

Although, it is expected that the liquefaction behavior of soil at far field (i.e., free field behaviour) and near field (close to structure behavior) will be different, the exact difference and their influence is very uncertain and depends on many site conditions. For pile foundation design, some researchers suggests to consider zero strength and stiffness of the liquefied

soil [9], whereas, many other researchers [10], [11], believes that the liquefied soil bears certain strength and stiffness which shall be considered during pile foundation analysis and design. The codes of practice of various countries does not say anything in this regard.

Hence, considering the far field behavior same as near field behaviour may not always provide conservative response. Fig 3 schematically illustrates two possible conditions of liquefaction at far field and near field during earthquakes.

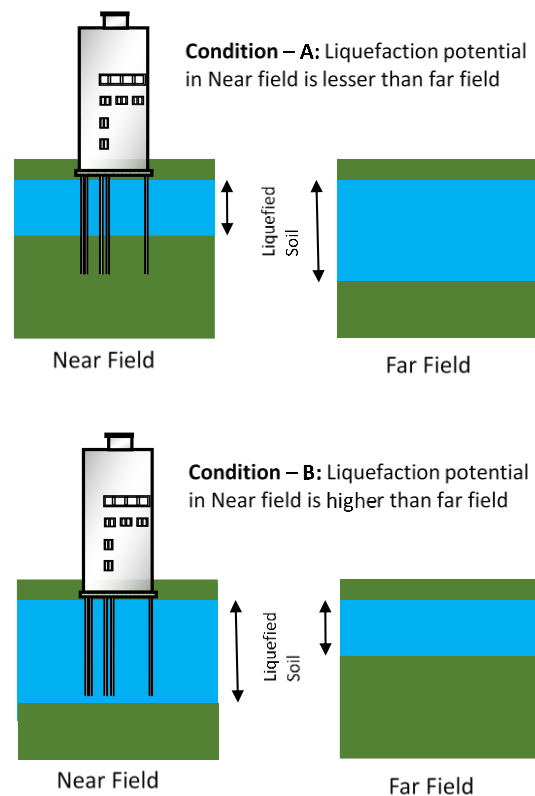


Fig. 3 Schematic representation of the difference in the occurrence of liquefaction in far field and near field during earthquakes.

Therefore, to understand the difference between the far-field and near-pile pore water pressure behavior for pile-soil interaction in liquefiable soils and its implication in lateral strength of liquefied soil, a high quality centrifuge test results are studied in this paper. The pattern of excess pore water pressure generation and development has been compared for both near-pile and far-field. The pore water pressure measured very close to pile is termed as near-pile measurement, and the measurements those are done more than 3D distance from pile is termed as far-field measurement, where D is the diameter of the pile. The results are critically reviewed and discussed in this paper.

2. EXPERIMENTAL SETUP OF PILE IN LIQUEFIABLE SOILS

A high quality centrifuge test data is used in this paper to study the near and far field pore water pressure response during seismic vibration.

The centrifuge test was carried out in the centrifuge facility of Shimizu Corporation, Japan. The test was carried out at centrifugal acceleration of 30-g. The stress and strain parameters were modelled by a factor of unity and the linear dimensions by the scale factor of 1: n (model: prototype), where $n = 30$. The scaling parameters related to the centrifuge tests at centrifugal acceleration of n -g are presented in Table-1.

Table-1: Scaling laws for centrifuge modelling at n -g.

Parameters	Unit	Model / Prototype
Stress	$ML^{-1}T^{-2}$	1
Strain	--	1
Length	L	1/n
Time (Dynamic)	T	1/n
Acceleration	LT^{-2}	N
Seepage velocity	LT^{-1}	N
Pile bending stiffness	$ML^{-3}T^{-2}$	$(1/n)^4$
Natural frequency	T^{-1}	1/n

A laminar box was used in the test, which had 14 rectangular frames made up of square steel tubes. The frames were connected by thin linear bearings of 2mm thickness placed in between them. The inside of the container was lined with a 1mm thick rubber membrane for waterproofing of the box and to protect the bearings from soil. The inside dimensions of the box were 807mm long, 475mm wide and 324mm high.

2.1 Test Layout

The model scale test setup used here is shown in Fig 4. For this centrifuge test, two pile groups were modelled on each side (side A and Side B) of the central partition wall. This kind of setup was beneficial in modelling both pile groups with the required soil/ structural parameter variation while keeping all other model parameters exactly same.

In this test setup, the free end quay wall was modelled near to one end of the container to simulate the soil flow condition (i.e. lateral spreading). In each of the tests, both the pile groups (A and B) were subjected to nearly identical conditions with respect to input motions and soil liquefaction.

Steel pipe of outer diameter 10mm and wall thickness of 0.2mm is used as pile in the test. Four layers of soil were placed in the test box, where top soil (soil-1) was unsaturated and all other three soils were saturated. The second layer of soil (soil-2) was

prepared as medium loose saturated sand, which liquefied during the test. The pile was initially fixed to the bottom of the box. Four layers of soil were then filled in the box with required relative densities. The geotechnical properties of the sand layers are provided in Table-2.

The model was subjected to a varying magnitude base acceleration of a 60 Hz sine wave (2 Hz at prototype scale). The magnitude of base acceleration was gradually increased to make the liquefaction process more realistic, which went up to $\sim 8g$ in 0.25s. This allowed the soil to liquefy in 5-6 cycles of loading in the model. The base acceleration input in prototype scale is shown in Fig 5.

Table 2 Geotechnical properties of the sand used in the test.

Item	Soil-1	Soil-2	Soil-3	Soil-4
Soil Type	Silica Sand-8	Silica Sand-8	Toyoura Sand	Silica Sand-3
e_{max}	1.385	1.385	0.951	0.974
e_{min}	0.797	0.797	0.593	0.654
D_r	50	50	90	90
γ'	12.85	7.652	9.908	9.496
	(γ_t)			
S_r	10	100	100	100

Note: e_{max} and e_{min} = Maximum and minimum void ratio, D_r = Relative Density (%), γ' = Effective unit weight (kN/m^3), γ_t = Dry unit weight (kN/m^3), S_r = Saturation Ratio.

3. OBSERVATION OF LIQUEFACTION FAR FIELD AND NEAR THE PILE

The near pile pore pressure observation was made behind the pile during the test. The value of excess pore water pressure (EPWP), being a stress parameter, remains same for both prototype and model scale measurement in the centrifuge test. But, the time axis changes with a scale of 1 : n(model : prototype). The results here are presented in prototype scale.

Fig.6 shows the excess pore water pressure recorded near to the pile and far field for vertical piles (Side-A). It has been observed that the pore water pressure generation is slower near the pile foundation as compared to far field. The pore water pressure ratio for these two cases are also plotted in Fig.7. The soil near to pile took long time (10s) to reach full liquefaction, whereas the soil at same depth in free field was fully liquefied in 5 seconds. This could be due to the availability of dissipation path at the pile-soil interface, which prevents the built up of pore water pressure. Hence it took more time for the soil to fully liquefy near the pile in comparison with far field.

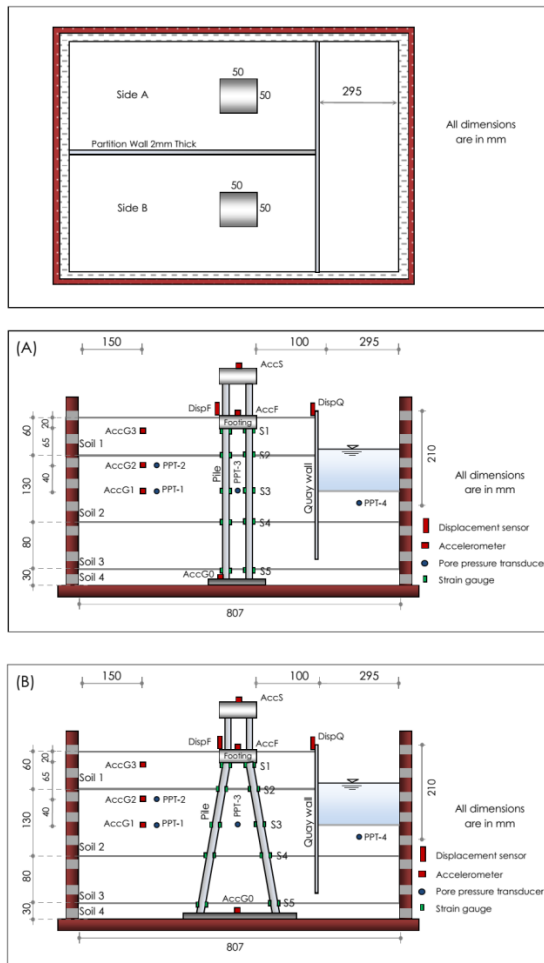


Fig. 4 Centrifuge test layout for pile foundation in liquefiable soils.

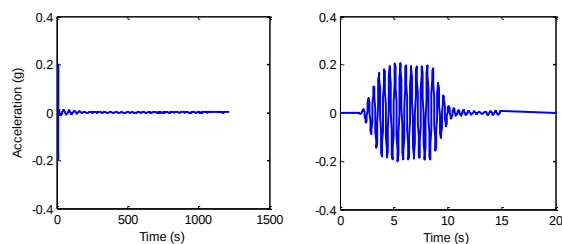


Fig.5 Base acceleration input in the centrifuge test (prototype scale).

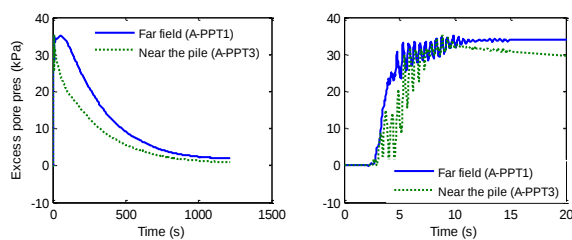


Fig. 6 Comparison of excess pore water pressure near to the pile and at the far field in Side-A model (Vertical pile group).

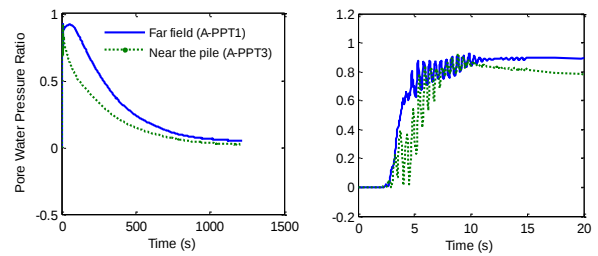


Fig. 7 Comparison of pore water pressure ratio near to the pile and at the far field in Side-A model (Vertical pile group).

Similar behavior has also been observed for inclined pile group (Side-B) as can be seen in Fig. 8.

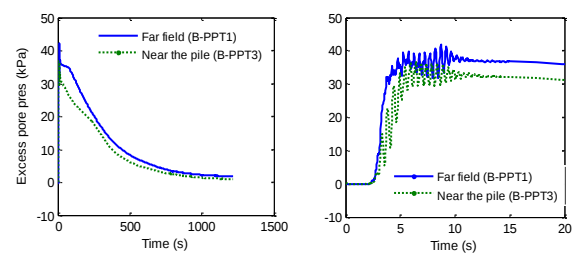


Fig. 8 Comparison of excess pore water pressure near to the pile and at the far field in Side-B model (Inclined pile group).

The pore water pressure ratio for inclined pile group is plotted in Fig. 9 for both near to pile and far field. It is clearly evident that even at the same depth, the soil close to the pile could not fully liquefy, but at free field full liquefaction has been observed. There is a slight overshooting of the pore water pressure ratio above 1. The maximum value of pore water pressure ratio should be 1, as this corresponds to full liquefaction. However the observance of overshooting of this value can be attributed to (1) the actual effective stress at the measuring location might be more than the calculated value due to densification, and/or (2) settlement/dislocation of pore water pressure sensor during the test.

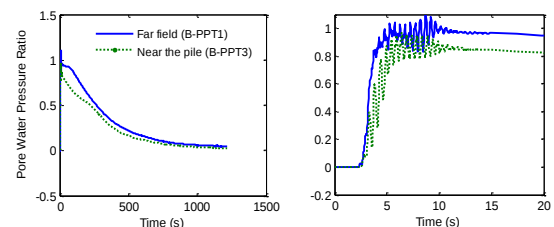


Fig. 9 Comparison of pore water pressure ratio near to the pile and at the far field in Side-B model (Inclined pile group).

Fig. 10 compares the excess pore water pressure near the pile for straight (Side-A) and inclined (Side-B) pile group. This figure shows that for inclined pile

group the pore water pressure generation is slower than that of vertical pile group. This is because, the presence of pore pressure dissipation path is smaller in vertical pile group as compared to longer inclined path in inclined pile group.

The lateral resistance of soil at pile-soil interface has been estimated from the strain gauge reading at different depths of the pile. This lateral soil resistance with respect to soil-pile relative deformation is generally known as p - y curves. The detailed procedure of estimating p - y curves for this test can be referred in Dash (2010).

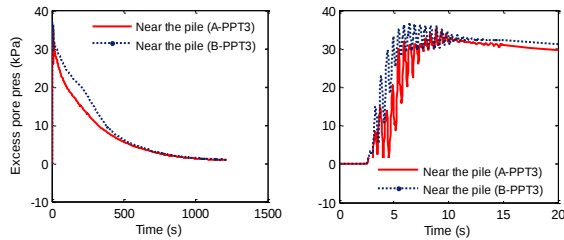


Fig-10 Comparison of excess pore water pressure near the pile for vertical (Side-A) and inclined (Side-B) pile group.

Fig.11 shows the lateral soil resistance - displacement behavior (p - y curves) at different times during the test. The p - y curves were also compared with the 10% API recommended p - y curves for saturated sand at corresponding depths. Viewing Fig. 11 in conjunction with Fig.6 and 7 shows that the resistance of liquefied soil subjected to shear was partly attributed from the decrease in EPWP. This behaviour is similar to that observed in the laboratory element tests during monotonic shearing of liquefied soil, for example: Yasuda et al., 1999, 1998; Sitharam et al., 2009. In the initial phase of loading, free field pore water pressure reached a maximum value (that corresponds to effective stress = 0) in 3 to 4 cycles. But near to the pile, development of pore water pressure was dependent on pile vibration, and in each cycle significant pore water pressure dissipation was also happening due to the availability of dissipation path at soil-pile interface.

Hence, even though the excess pore pressure near to the pile could be the governing parameter of soil resistance, it is not being used in practice because of the difficulty to exactly access it. The lateral resistance of liquefied soil at soil-pile interface is, thus, normally characterized by the degree of liquefaction expected in the soil at the site without considering the influence of pile.

Therefore, the lateral strength of liquefied soil (i.e., its p - y behavior) has to be suitably chosen to make the design safe and conservative for all failure modes and their possible combinations [16], considering the possibility of having different depth and degree of liquefaction near to the pile foundation than free field.

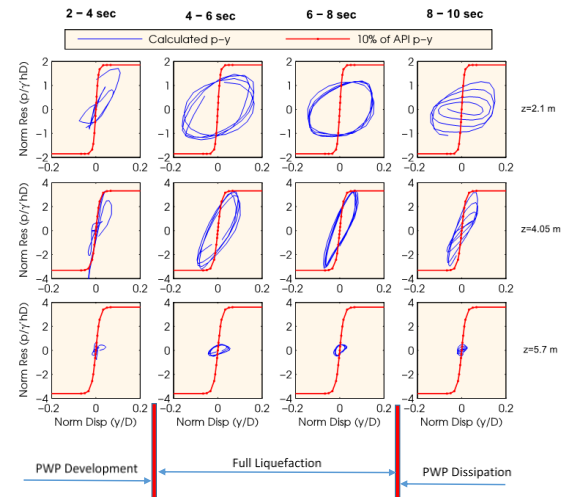


Fig. 11 p - y curves at different time range during the test for a pile in Side-A pile group.

4. CONCLUSION

Based on present investigation of the centrifuge test results and previous literatures, the extent of near field and far field liquefaction can notably be different with variation in both depth and the spread, which depends on many parameters.

PWP generation is slower for inclined piles than that of vertical pile group (due to presence of longer inclined pore water pressure dissipation path). However this difference may be treated as insignificant for design considerations.

In design, while considering the far field response of soil same as the near field behavior, it has to be considered only for the cases where depth of liquefaction at far field provide upper bound estimation.

For the cases where near field liquefaction can provide upper-bound value, suitable amplification factor of the far field response shall be chosen for the analysis.

Lateral strength of liquefied soil (i.e., its p - y behavior) has to be suitably chosen to make the design safe and conservative.

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6. REFERENCES

- [1] Rollins KM and Seed B, "Influence of Building on Potential Liquefaction Damage", J. of Geotechnical Engineering, Vol. 116, No. 2, Feb 1990, pp. 165-185.
- [2] Ishihara K and Matsumoto, "Bearing Capacity of Saturated Sand Deposits during Vibration", Proc.

- 4th Japan Earthquake Engineering Symposium, Tokyo, Japan, 1976, pp-431-438.
- [3] Yoshimi YK and Tokimatsu K, "Two Dimensional Pore Pressure Changes in Sand Deposits during Earthquakes", Proc. 2nd Int. Conf. on Micro-zonation, National Science Foundation," Vol. 2, 1978, pp. 853-863.
- [4] Whitman RV and Lambe PC, "Liquefaction: Consequences for a Structure", Soil Dynamics and Earthquake Engineering Conf., Southampton University, Vol.2, 1982, pp- 941-949.
- [5] TakemuraJ, Imamura S and Takashi Y, "Stability of Oil Tank Supported by Piled-Raft Foundation on Liquefiable Sand", Proc. of Sixth International Conference on Urban Earthquake Engineering, March 3-4, 2009, Tokyo, Japan
- [6] Watanabe T, "Damage to Oil Refinery Plants and a Building on Compacted Ground by the Niigata Earthquake and their Restoration", Soils and Foundation, Vol.15 No.3, 1966, pp. 27-40.
- [7] Ishihara K, Kawase Y, Nakajima M, "Liquefaction Characteristics of Sand Deposits at an Oil Tank Site during 1978 Miyagiken-Oki Earthquake", Soils and Foundation, Vol.20, No. 2, 1980, pp. 97-111.
- [8] Liu H and Quio T, "Liquefaction Potential of Saturated Sand Deposits Underlying Foundation of Structure", 8th World Conf. on Earthquake Engineering, Vol. 3, 1984, 199-206.
- [9] Bhattacharya S, "Pile instability during earthquake liquefaction", PhD thesis, University of Cambridge, UK, 2003.
- [10] Goh S, O'Rourke TD, "Limit state model for soil-pile interaction during lateral spread", Proc. of seventh US-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Soil Liquefaction, Seattle, 1999. p. 237-260.
- [11] Abdoun T and Dobry R, "Evaluation of Pile Foundation Response to Lateral Spreading", Soil Dynamics and Earthquake Engineering, Vol.22, No. 9-12, 2002, pp.1051-1058.
- [12] JRA. Specifications for highway bridges, part V. Seismic design, 1996.
- [13] Dash SR, "Lateral Pile Soil Interaction in Liquefiable Soils", PhD Thesis, 2010, University of Oxford.
- [14] Yasuda S, Terauchi T, Morimoto M, Erken A and Yoshida N, "Post liquefaction behavior of several sands", Proc. of 11th European conference on Earthquake Engineering, Balkema, Rotterdam, 1998.
- [15] Yasuda S, Yoshida N, Kiku H, Adachi K and Gose S, "A simplified method to evaluate liquefaction-induced deformation", Proc. of Earthquake Geotechnical Engineering, Balkema, Rotterdam, 1999, pp. 555-566.
- [16] Dash S R, Bhattacharya S and Blakeborough A, "Bending-Buckling Interaction as a Failure Mechanism of Piles in Liquefiable Soils", Journal of Soil Dynamics and Earthquake Engineering, Volume 30, Issues 1-2, January-February 2010, Pages 32-39.

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