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SETTLEMENT PREDICTION OF PILE-SUPPORTED STRUCTURES IN LIQUEFIABLE SOILS DURING EARTHQUAKE

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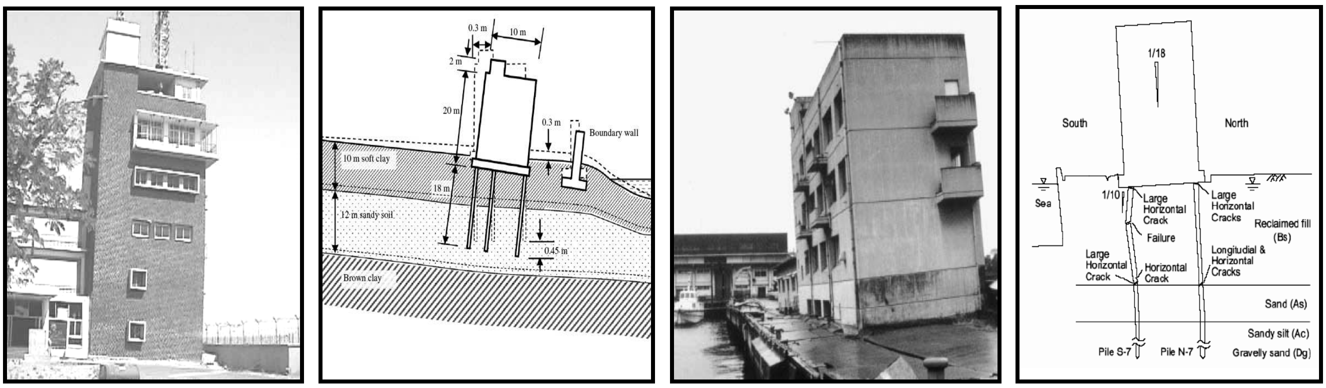
ABSTRACT :

It is observed from the past earthquakes that the pile foundations in liquefiable soil are very susceptible to damage or failure. Often, the structures in liquefied soil experience excessive tilting and/or settlement. Though, the failure is often attributed to lateral spreading of the ground, however detailed study of some case histories shows that the settlement of pile foundation could be a potential failure mode that can cause tilting of the structure. The settlement of a pile when loaded axially is considered to be of three parts: (a) Axial compression of pile; (b) Slip between soil-pile interface; (c) Settlement of the soil mass as a whole. This present addresses the issue of settlement of pile-supported structures due to the loss of pile capacity in liquefied soil. A simple mathematical model that can be implemented in an EXCEL type program has been proposed for characterizing the above phenomenon. The method uses envelopes of unit load transfer curve that describes the axial load transfer mechanism of the pile foundation in liquefied soil.

KEYWORDS: Settlement prediction, Axial load transfer, Liquefaction, Pile foundation

1. INTRODUCTION

Pile foundations are commonly used to transfer axial loads from a superstructure to the ground in cases where: (a) the structural loads are very high; (b) where the surface soil or soils at shallow depths cannot carry the imposed loads. Also, piles are used to support structures in areas of seismic risk especially where the soils can liquefy due to the seismic shaking. Following a moderate to strong earthquake in liquefiable areas, it has been observed that piled foundation suffer tilting along with settlement. Figure 1(a & c) shows two such cases. In each of the cases, the piles supporting the building either passed through liquefiable soils or were founded on liquefiable soils. This paper investigates some aspects of the vertical settlement of the piled foundation when the soil surrounding the pile liquefies.



(a) (b) (c) (d)

Figure 1 (a) Tilting of Customs Tower House following the 2001 Bhuj EQ, Dash et al (2008); (b) Schematic diagram of failure of 1(a); (c) Tilting of Pile-supported building following the 1995 Kobe EQ, Bhattacharya (2006); (d) Schematic diagram of failure of 1(c)

2. PROBLEM DEFINITION

Often in practice, engineers need to design a piled foundation where the soil profile is layered and one of the layers may liquefy. Figure 2 shows typical site conditions to be encountered at site. Table 1 lists some case histories where such conditions have been encountered.

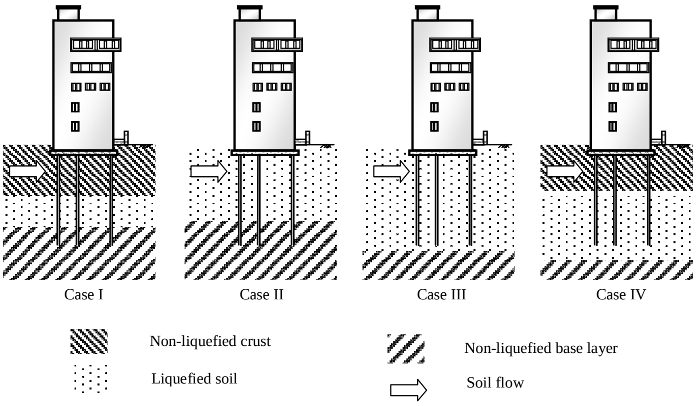


Figure 2 Different field conditions of soil-pile interaction

During seismic shaking, loose to medium dense soil liquefies. As the result of liquefaction, the effective stress in the liquefied soil zone reduces to a near zero value. Therefore, the shaft resistance of the pile in liquefiable layer also becomes zero. The pile has to therefore transfer the axial load to deeper layers below the liquefiable zone which may result in pile head settlement – often termed as axial load transfer mechanism. If the pile head displacement is more than the allowable limit for the structure, then the structure may fail by Serviceability Limit State.

For Case I and Case II in figure 2, due to the loss of shaft resistance in the liquefied layer, the pile will settle but the catastrophic collapse or failure may be avoided if the pile is sufficiently embedded in the non-liquefiable layer below the liquefiable layer. In these cases, buckling of slender piles and bending due to lateral loads may form plastic hinges in the pile, Bhattacharya (2003). On the other hand, for Case III and Case IV, where the pile rests on liquefiable soil deposits, settlement or tilting of the structure is inevitable. Structural failures such as plastic hinges etc., are most unlikely noticed in these situations.

Table 1 Classification of case histories according to different cases as shown in figure 2

Cases in figure 2	Examples
Case I	Pile supported building in Figure 1(c), near to Quay wall. More details of building 1(c) can be found in Bhattacharya (2006). Here the non-liquefied crust above liquefiable soil is about 2m.
Case II	This case is typically encountered in bridge pier piles (Showa bridge piles). Details compiled in Bhattacharya (2003).
Case III	Piled foundations for Saiseikai Hospital, Ishizue Primary School, Irifune Primary School and East Police Station. Details of performance of these foundations during the 1964 Niigata Earthquake can be found in Hideaki (1966).
Case IV	Building in Figure 1(a). Customs Tower House in Kandla Port following the 2001 Bhuj Earthquake. Details can be found in Dash et al (2008).

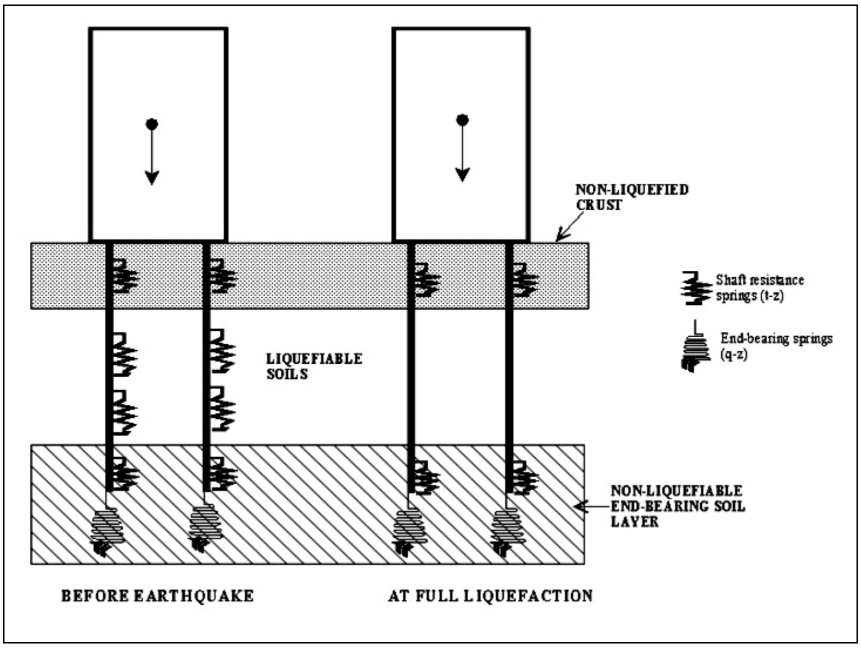
The aim of this paper is therefore the following:

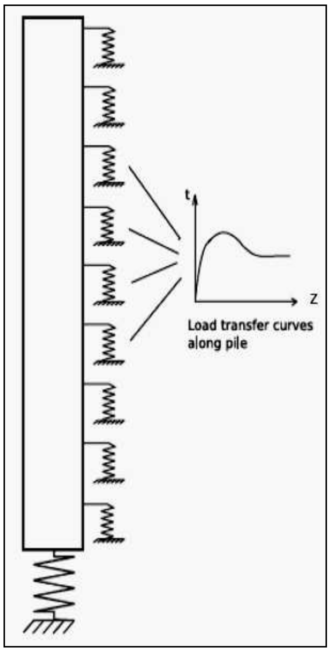
1. Develop a simple framework to predict the pile head settlement based on the various mechanisms that control the overall settlement.
2. Validate the simple frame work through the analysis of a field case record.

3. A SIMPLE FRAMEWORK TO PREDICT SETTLEMENT

3.1 Physical understanding behind the framework

Figure 3 shows a simple schematic diagram of a pile-supported building in two stages: (a) before earthquake; (b) at full liquefaction.





(a) (b)

Figure 3 (a) Schematic representation of a pile-supported structure. The soil is replaced by t-z and q-z springs; (b) Idealized single pile

Before earthquake, the axial load is shared between the shaft resistance and the end-bearing resistance (point resistance). One of the practical ways to analyse the settlement of an axially loaded pile is to consider the soil as a spring. The resistive capacity of the spring varies with the amount of deflection i.e. the relative movement between the pile and the soil for shaft resistance and the settlement of the pile tip for end-bearing resistance. The resistance rises to a maximum value which is then maintained for any further displacement. In other words, the maximum resistance is mobilised. The maximum value of this resistance can be obtained from the soil properties and is directly proportional to the effective stress of the soil. Conventionally, the shaft resistance is denoted by t-z spring and the end-bearing resistance is denoted by q-z springs (Vijayvergiya, 1977). The natures of the springs are shown in figure 4.

Figure 4 Load-movement and load transfer characteristics of an axially loaded pile

In loose saturated sandy soil, as the shaking continues the pore pressure will rise and the soil may start to liquefy. The pile will then start to lose its shaft resistance in the liquefied layer and shed axial loads down the pile. As a result, the springs in the liquefied zone cease to act as shown in figure 3. If the bearing capacity at is exceeded, settlement failure may occur.

3.2 Mathematical formulation behind the framework

Figure 5 shows the pile and it is divided into N elements. In the model, *i* is a general element having a length of *dx*. The value of F_i represents a proportion of the maximum resistive force mobilized defined by the deflection of the element u_i.

$$F_i = f_{si}A_{si} = \beta_i f_{si\max} A_{si} \qquad (1)$$

In the above equation $f_{si\,max}A_{si}$ represents the maximum shaft resistance for the pile element. Also, the change is axial load can be expresses as follows:

$$\frac{dP}{dx} = -\pi.D.f_s = -F \qquad (2)$$

The settlement of the pile element can be expressed by eqn. 3 where *EA* is the axial stiffness of the pile.

$$\frac{du}{dx} = -\varepsilon_x = -\frac{P}{(EA)_P} \qquad (3)$$

Combining eqns. 2 and 3, we arrive at eqn. 4.

$$\frac{d^2u}{dx^2} = \frac{\pi D}{(EA)_P} f_s \qquad (4)$$

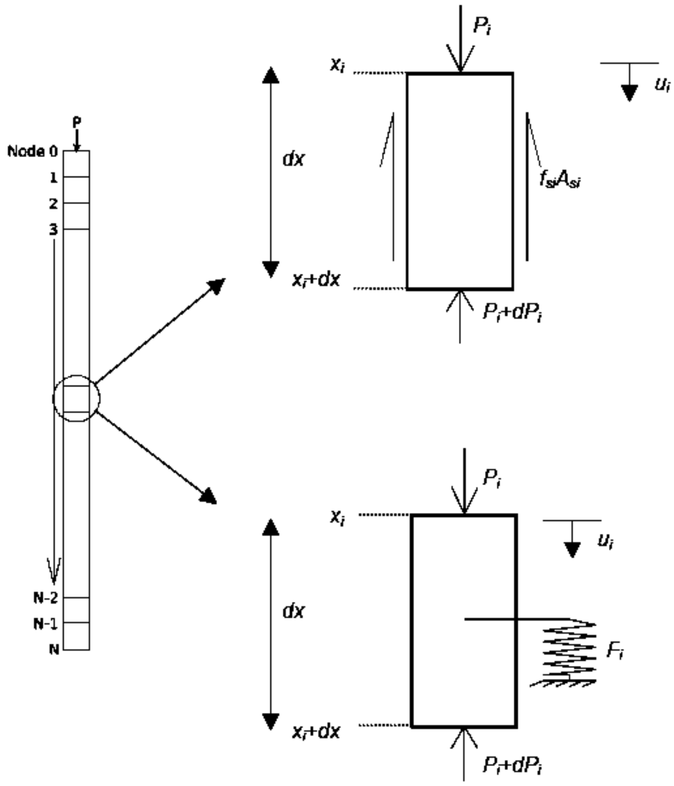


Figure 5 An element of a pile with spring

Equations 3 and 4 are approximated to form the basis for a computational method of calculating the deflections in the pile.

$$\frac{\left(\frac{du}{dx}\right)_i - \left(\frac{du}{dx}\right)_{i-1}}{dx} = \frac{\pi D.f_{s(i-1)}}{(EA)_P} \qquad (5)$$

$$\left(\frac{du}{dx}\right)_i - \frac{-P_{i-1}}{(EA)_P} = \frac{-dP_{i-1}}{(EA)_P} \tag{6}$$

$$\left(\frac{u_i - u_{i-1}}{dx}\right)_i = -\frac{(P+dP)_{i-1}}{(EA)_P} \tag{7}$$

$$u_i = u_{i-1} - \frac{(P+dP)_{i-1}}{(EA)_P} dx \tag{8}$$

Therefore, we obtain that the

$$\boxed{deflection_i = deflection_{i-1} - pile\ compression_{i-1}}$$

The other equations are:

$$u_i = u_{i-1} - \frac{(P_{i-1} - F_{i-1})}{(EA)_P} dx \tag{9}$$

$$P_i = P_{i-1} - F_{i-1} \tag{10}$$

3.3 Algorithm for implementation of the above in a spreadsheet

The above equations can be easily implemented in a spreadsheet type program to compute the loads and displacement of the pile. The steps are:

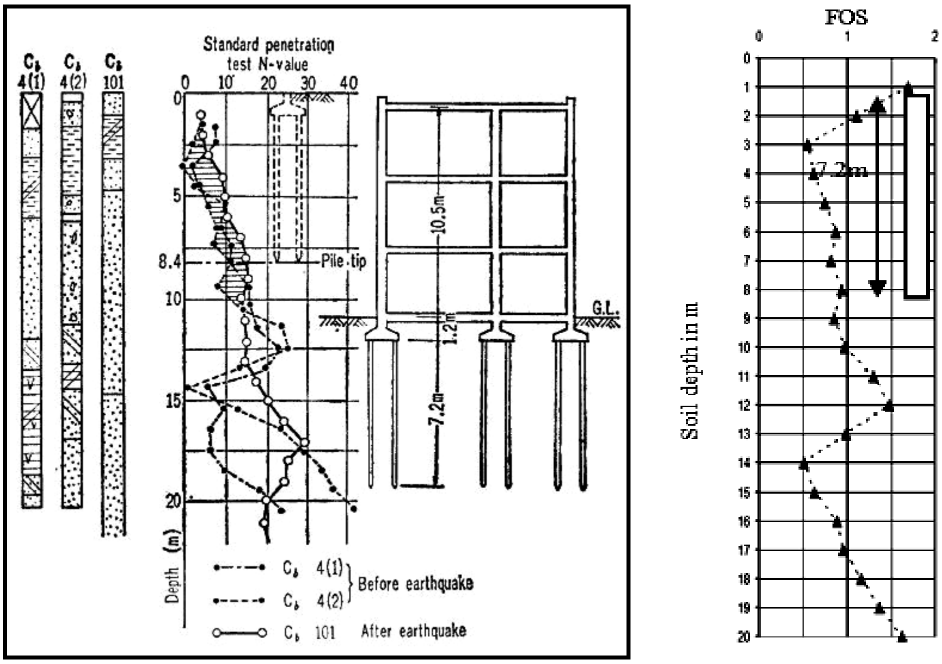
1. Calculate the pile head load P_0 acting at node 0 in figure 5.
2. Choose an incremental length, *dx*.
3. For each element, calculate the ultimate shaft resistance and point bearing for N^{th} element. By adding all the resistances, find the ultimate capacity of the pile and compare with P_0.
4. Assume an initial pile head deflection.
5. Use Eqn.1 to find the value of ultimate shaft resistance force F (or $-dP$) in the top element. The value of factor β can be found using figure 4.
6. Use computational Eqns. 9 and 10 to calculate the deflection and the load on the next element.
7. Repeat steps 5 and 6 for every element down the pile.
8. Using Eqn.1 and a value of β from figure 4 find the final resistive force under the base of the pile.
9. Use Eqn.10 to find the load that would be on the top of an element below this point, should one exist.
10. Now return to step 4 and carry out an iterative procedure using steps 4 to 10 until the resulting load in step 9 is zero.

Equilibrium requires that the sum of the loads carried by the soil (modelled as springs) must equal the applied load. When this is true the load resulting from step 9 must be zero as there are no further springs below the base. The pile head deflection that results in this equilibrium is the deflection predicted for the load applied in step 1.

4. STUDY OF A CASE STUDY: TILTING OF SAISEIKAI HOSPITAL BUILDING

4.1 Damage to the building

During the 1964 Niigata Earthquake (magnitude, $M_w = 7.6$) many reinforced concrete buildings supported on pile foundations experienced tilt or subsidence. The maximum ground acceleration at the site is reported as 0.14g (Fukuoka, 1966). The subsoil condition in Niigata city is mostly sandy (Yorihiko, 1966). The investigated Saiseikai hospital building is a three-storied reinforced concrete structure. The piles supporting the hospital are of made of concrete, 7.2m in length and 0.18m in diameter. The allowable bearing capacity was 60kN per pile. The tip of pile is placed 8.4m below ground level. The building subsided 0.6-1.0m and tilted about 5.6° towards east direction in the short span.



(a) (b)

Figure 5 (a) Schematic diagram of Saiseikai hospital building along with soil profile; (b) Factor of Safety against liquefaction of the Saiseikai hospital building site

4.2 Soil profile at the Saiseikai hospital building site

The soil profile at the site consists of silty sand to about 5m depth and then medium dense sand till 20m depth. The Niigata city straddles very nearer to Shinano river at its mouth and it influences the depth of water table and its location is assumed of about 1.5m below ground level. The dry unit weight of the soil is assumed as 16kN/m^3 and also the saturated unit weight of the soil as 17kN/m^3. The Standard Penetration Test (SPT) 'N' value can be seen in figure 5.

The overall observed settlement of the pile is a combination of the settlement of the soil mass as a whole, axial compression of pile and slip between soil-pile interfaces due to loss of shaft resistance owing to liquefaction. It is therefore required to identify the liquefaction potential of the site before performing the prediction of pile foundation settlement.

4.3 Evaluation of liquefaction potential at the site

Idriss and Boulanger (2004) have developed a semi-empirical approach for liquefaction assessment, which is an extension of Seed and Idriss (1970) work. The factor of safety against liquefaction for the site is shown in figure 5(b). The analysis suggests that the soil liquefied to about 10m. Therefore, the foundation was fully embedded in liquefiable soil i.e. Case III in figure 2.

4.4 Computation of pile head settlement based on the simplified framework

Based on the framework discussed in section 3, it has been estimated that the ultimate load of the pile is about 80kN. Therefore, the strength of soil is adequate to carry the allowable load on the pile under service condition. But in case of seismic condition, the soil springs along the shaft and end bearing area weakens. Hence, to predict the behaviour of pile in bearing its allowable load under this seismic case, the soil springs along the pile is reduced accordingly. Here a factor is introduced as α, called as soil strength reduction factor. It is done by reducing the shaft and bearing resistance of the soil in liquefiable area to 20%, 40%, 60%, 80%, 90%, 95%, 99% and finally 100%. Figure 6 and table 2 shows the results of the analysis.

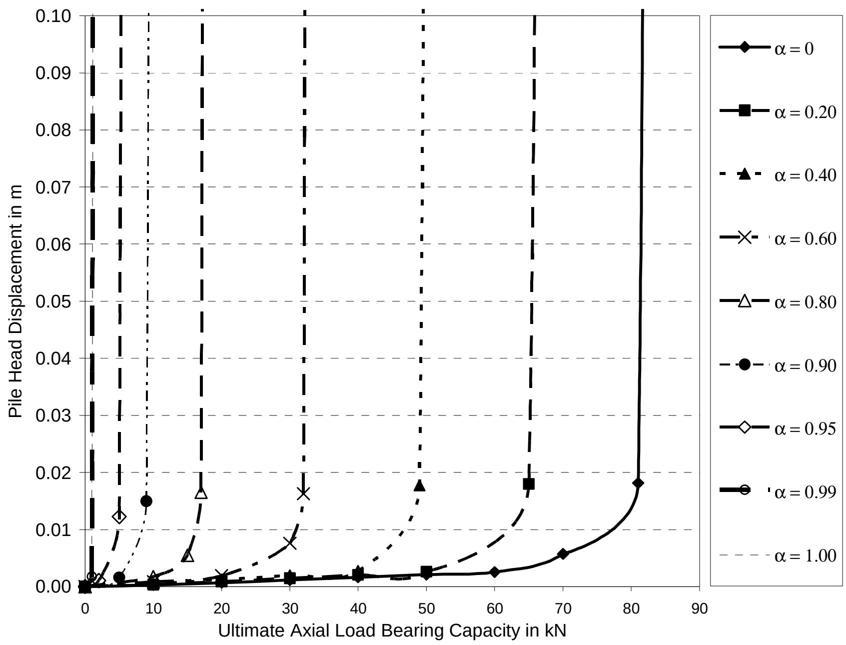


Figure 6 Predicted pile head settlement of Saiseikai hospital building for both service and seismic condition (α = 0, denotes the service condition)

Table 2 Results of pile head settlement

Case history/ Comments	Soil profile	Pile length, diameter and allowable load under each pile	Thickness of liquefied layer	Estimated pile head settlement	
				Service condition	Seismic condition of various % shaft and base resistance in liquefaction area
Saiseikai Hospital Building (case III) / Pile tip at liquefied area	0 – 5m : silty sand 5m – 20m : medium sand	7.2m, 0.18m and 60kN	8m (6.5m pile embedded)	0.003m	0.003m (90%) 0.010m (80%) Infinity - Sink/tilt (70%)

4.5 Estimation of ground settlement

One of the case histories, Saiseikai hospital building that falls under Case III is considered for liquefaction-induced ground settlement estimation which is calculated using two methods proposed by Tokimatsu et. al (1987) and Ishihara et. al (1992). The result of this analysis is shown in table 3.

Table 3 Post liquefaction ground settlement

Case History	Pile Length	Soil Depth Considered	Ground Settlement	
			Method 1 Tokimatsu et al (1987)	Method 2 (Ishihara 1992)
Saiseikai Hospital Building, 1964 Niigata EQ	7.2m	20m	0.368m	0.430m

5. DISCUSSION AND CONCLUSIONS

The analysis of the case study showed that when the reduction in soil strength is about 30%, the pile will lose it bearing capacity and will tilt or sink. The case falls under Case III in figure 2. The ground will settle by 400mm The prediction of liquefaction depth shows that the pile rests on liquefied layer itself, hence the settlement or tilting failure, see table 2. The pile tip rests on liquefied area and structural failure in the pile is unlikely. One of the limitations of this method is that dynamic effects are ignored.

A simplified approach of quantifying the settlement of a pile due to loss of effective stress owing to liquefaction has been discussed. The method is based on axial load transfer (t-z, q-z) curves extensively used in offshore engineering practice (API, 2000). An example is considered to demonstrate the application of the method.

REFERENCES

API (2000). 2A-WSD:Recommended practice for planning, designing and constructing fixed offshore platforms – working stress design. *American Petroleum Institute Recommended Practice*, 56-63.

Bhattacharya, S. (2003). Pile instability during earthquake liquefaction. *PhD thesis*, University of Cambridge, UK.

Bhattacharya, S. (2006). Safety assessment of existing piled foundation in liquefiable soils against buckling instability. *ISET Journal of Earthquake Technology*, **43:4,** 133-147.

Dash, S.R., Govindaraju, L. and Bhattacharya, S. (2008). A case study of damages of the Kandla port and customs office tower supported on a mat-pile foundation in liquefied soils under the 2001 Bhuj earthquake. *Soil Dynamics and Earthquake Engineering*, doi:10.1016/j.soildyn.2008.03.004 (Article in Press)

Hideaki, K. (1966). Damage to reinforced concrete buildings in Niigata city with special reference to foundation engineering. *Soils and Foundations*, **6:2,** 71-88.

Idriss, I.M. and Boulanger, R.W. (2004). Semi-empirical procedures for evaluating liquefaction potential during earthquakes. *Proceedings of the 11th International conference on soil dynamics and earthquake engineering*, **1**, 32-56.

Ishihara, K. and Yoshimine, M. (1992). Evaluation of settlements in sand deposits following liquefaction during earthquakes, *Soils and Foundations*, **32:1,** 173-188.

Fukuoka, M (1966). Damage to civil engineering structures, *Soils and Foundations*, **6:2**, 45-52.

Seed, H.B. and Idriss, I.M. (1970). Simplified procedure for evaluating soil liquefaction potential. *Journal of Soil Mechanics and Foundation Engineering, ASCE*, **97:9**, 1249-1273

Tokimatsu, K. and Seed, H.B. (1987). Evaluation of settlements in sand due to earthquake shaking. *Journal of Geotechnical Engineering, ASCE*, **23(4)**, 56–74.

Vijayvergiya, V.N. (1977). Load-movement characteristics of piles. *Conference report*, Long Beach, California

Yorihiko, O. (1966). Niigata earthquakes, 1964 building damage and soil condition. *Soils and Foundations*, **6:2,** 14-37.