A METHOD TO EVALUATE THE SAFETY OF THE EXISTING PILED FOUNDATIONS AGAINST BUCKLING IN LIQUEFIABLE SOILS

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Abstract: Recent research into the pile failure mechanism has shown that there is a fundamental omission in the seismic pile design in liquefiable areas. The current codes of practice for pile design such as EC 8, NEHRP 2000, JRA1996 and IS1893 is based on a bending mechanism where lateral loads due to inertia or slope movement induces bending failure in the pile. These codes omit considerations necessary to avoid buckling of a pile due to the axial load acting on it at all times in an event of soil liquefaction due to the diminishing soil support. These codes are inadequate and buckling needs to be addressed. Bending and buckling require different approaches in design. Bending is a stable mechanism and is dependent on strength whereas buckling is dependent on geometric stiffness and is almost independent of strength. Designing against bending would not automatically suffice the buckling requirements. For pile design, to avoid buckling there is a need to have minimum diameter depending on the depth of liquefiable soil. Thus there is a need to reconsider the safety of the existing piled foundations designed based on the current codes of practice. This paper discusses a way to identify the existing unsafe structures.

1. INTRODUCTION

1.1 Pile-supported Structures still collapse during earthquakes!

Collapse of pile-supported structures in liquefiable soils is still observed after strong earthquakes despite the fact that a large factor of safety against bending due to lateral loads is employed in their design. The failure of the structure is often accompanied by tilting and or settlement of the overall structure without any damage to the superstructure. During excavation following an earthquake, the piles are often observed to form plastic hinges, see for example the piles of NHK building, NFCH building and Showa Bridge Hamada (1992) or a three storied building, Tokimatsu (1997). It has been shown by Bhattacharya (2003) that using "Limit State Design Philosophy", the factor of safety against plastic yielding of a typical concrete pile ranges between 4 and 8. This high factor of safety is due to the multiplication of the partial safety factors due to load (1.5), material stress (1.5), plastic strength factor (ratio of Z_P/Z_E which is 1.67) and practical factors like minimum percentage of reinforcements due to shrinkage or creep in concrete. This suggests that unless wrong foundation design concepts are employed, failure by plastic yielding of piles is unlikely.

It is worth noting that study of failure of structures shows that when failure occurred in structures, they most often resulted due to loads that have been overlooked by the designer or regarded as secondary, rather than inadequate factor of safety. To cite an example is the collapse of 5km of 750mm diameter gas pipe line during testing from the Jamuna Bridge in Bangladesh on 11th June 1998. It has been reported (NCE, 1998) that the main cause of the collapse was the failure to allow in design the weight of the water in the pipe for testing purposes. Bhattacharya (2003), Bhattacharya and Bolton (2004a, 2004b) has shown that there is also a fundamental omission of a load effect in seismic pile design that has contributed to the failure of the many pile foundations. The current method of pile design under earthquake loading, for example JRA 1996, Eurocode 8, NEHRP 2000 is based on a bending mechanism where inertia and slope movement (lateral spreading of soil) induce bending moments in the pile. The next section of the paper aims to show that this hypothesis of pile failure is inconsistent with some of the observed mode of failure.

1.2 Inconsistency in Observations of Pile Failure with the Current Understanding

This section of the paper highlights some of the inconsistencies of observations of pile failure with respect to the current understanding. They are summarized below:

After the detailed investigation of the failure of piles during 1995 Kobe earthquake, Tokimatsu and Asaka (1998) reports that:
"*In the liquefied level ground, most PC piles (Prestressed Concrete pile used before 1980's) and PHC piles (Prestressed High Strength Concrete piles used after 1980's) bearing on firm strata below liquefied layers suffered severe damage accompanied by settlement and/or tilting of their superstructure, …..*".

If lateral spreading is the main cause of failure, why would most of high strength PHC piles collapse in level grounds i.e. in the absence of lateral spreading?

It is a common observation in seismic bridge failure that piers collapse while abutments remain stable, for example Figures 1 (a&b). Figure 1 (a) shows the collapse of one the piers of the Million Dollar Bridge leading to bridge failure. Similar failures were also observed of the Showa Bridge during the 1964 Niigata earthquake; see Figure 1(b).

Figure 1: Failure of bridges in earthquakes; (a): Million Dollar Bridge after the 1964 Alaska earthquake; (b): Showa Bridge after the 1964 Niigata earthquake. Photo courtesy NISEE.

Bhattacharya and Bolton (2004a) notes that in a bridge design, the number of piles required to support an abutment is governed by lateral load due to the fact that the abutment, as well as carrying the dead load of the deck, has to retain earth and fills of the approach roads to the bridge (see Figure 2). On the other hand, the bridge piers (intermediate supports) predominantly support the axial load of the deck. The lateral load acting on the pier during an earthquake is primarily the inertial force. The lateral capacity of a pile is typically 10 to 20% of the axial load capacity. Thus, for a multiple-span bridge having similar span lengths, the number of piles supporting an abutment will be more that that of a pier. It is worthwhile to note that in these examples only bridge piers collapsed while the abutments remained stable. This hints that the failure of bridge pier foundations may be influenced by axial load. In contrast, the current seismic design methods for pile foundations only concentrate on lateral loads.

Furthermore, the lateral loads acting on a bridge pier are only due to the drag force of the liquefied soil and any loads due to non-liquefied crust are unlikely to act. It is surprising that most often only bridge piers fail. This observation is in contradiction to the hypothesis of pile failure by Berrill et al (2001), Hamada (2000). Berrill (2001) analyzed the good performance of the Landing Bridge during the 1987 Edgecumbe earthquake and remarked that the chief threat to the piled foundations comes from the non-liquefied crust and not from the drag force of the liquefied soil.

Hamada (2000) in the 12th World Congress concludes that permanent displacement of non-liquefied soil is a governing factor for pile damage.

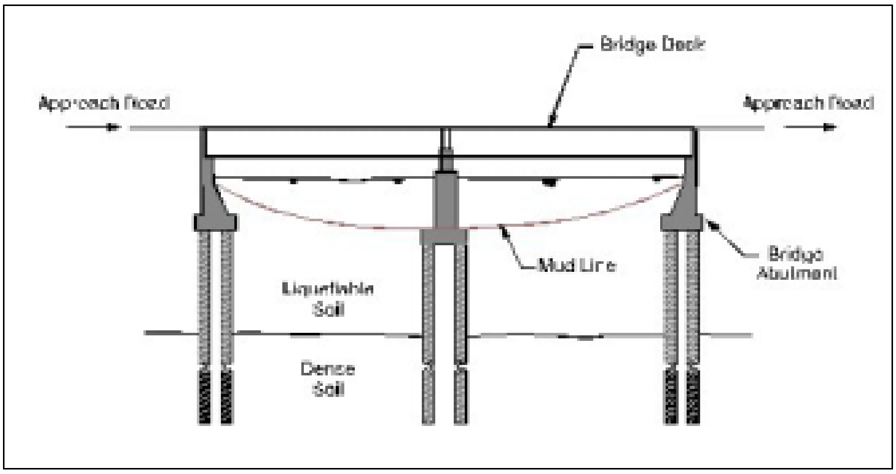

Figure 2: Schematic diagram of a bridge

Bhattacharya (2003), Bhattacharya et al (2004b), Bhattacharya and Bolton (2004a) studied in-depth the well known failure of the Showa Bridge, see Figure 1(b). This bridge was only one month old when the earthquake of Niigata occurred and the bridge collapsed. Details of the failure can be seen in Hamada (1992), Takata et al (1965), and Ishihara (1993). This bridge was studied in-depth as the bridge had steel tubular piles and was only one month old when it collapsed. Thus it is unlikely that the material of the pile corroded and it can be used as a benchmark problem for validation and verification of hypothesis or mechanisms. Figure 3 shows the schematic representation of the failure of the bridge which can be compared with Figure 1(b). As can be seen from Figure 3, piles under pier no. P_5 deformed towards the left and the piles of pier P_6 deformed towards the right, Takata et al (1965). Had the cause of pile failure been lateral spreading, the piers should have deformed identically in the direction of the slope. Furthermore, the piers close to the riverbanks did not fail, whereas the lateral spread is seen to be most severe at these places.

It must be mentioned here that this failure has been widely used as an example of piled foundations collapsed due to lateral spreading, Hamada (1992), Ishihara (1993). Bhattacharya (2003), Bhattacharya et al (2003) has shown that the piles of the Showa Bridge are safe against the JRA (1996) clause about the checks against lateral spreading by a factor of about 1.84. The bridge collapsed in 1964 which is safe against 1996 code and between these years the code was revised at least 3 times viz. 1972, 1980 and 1996.

It has been revealed after the excavation of the NHK building, Showa Bridge, Hamada (1992) and the three-storied building, Tokimatsu et al (1997) that hinges formed in piles occurred within the top third of the pile. Had the cause of pile failure been lateral spreading, the location of the plastic hinge would have been expected at the interface of liquefiable and non-liquefiable layer as this section would experience the highest bending moment. This observation also does not quite match with the current understanding of pile failure.

To summarize, the limitations of the current understanding of pile failure are:

1. The effect of axial load as soil liquefies is ignored.
2. Some observations of pile failure cannot be explained by the current hypothesis.
3. It has been shown, Bhattacharya (2003) that the pile foundation of Showa Bridge, which is considered safe by the JRA (1996) code, actually failed in 1964.

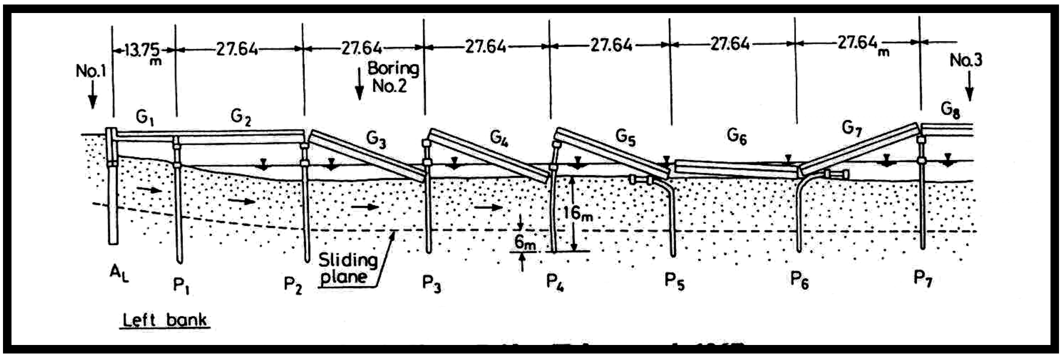

Figure 3: Schematic diagram of the failure of the Showa Bridge after Takata et al (1965). The diagram only shows half of the bridge. It must be noted that the direction of deflection of the piers P5 and P6 contradicts the current understanding.

Earthquakes induce lateral loads on the pile through inertia or slope movement (lateral spreading). But it must also be remembered that piles are normally used to carry vertical loads. These axial loads act at all times on the pile. Piles are long slender members having length to diameter ratio between 25 and 100 or even more, Bond (1989). The piles receive support from the surrounding soil and thus engineers did not see any problem of buckling. But during liquefaction, the stiffness of the soil surrounding the pile comes to near zero value and what will be the effect on these slender members? This is not addressed in the current understanding or codes of practice. An axial load has two distinct effects to a slender member being single or in a framework of many slender columns? They are (1) Reduces the bending strength of the member (Plastic moment capacity); (2) Causing a premature failure due to instability. This effect of axial load has been studied carefully and the missing link in the observations of pile failure and the current understanding could be bridged. Details can be seen in Bhattacharya and Bolton (2004a, 2004b and 2004c), Bhattacharya (2003).

2. A NEW THEORY OF PILE FAILURE

This section will describe a new theory of pile failure. A hypothesis of pile failure was formed based on the analysis of 15 reported case histories of pile foundation performance during earthquakes. This hypothesis was verified using dynamic centrifuge modeling; see Bhattacharya et al (2004a, 2004b), Bhattacharya (2003). Analytical studies also support the hypothesis, Bhattacharya and Bolton (2004a). The hypothesis being verified independently by three different approaches is thus called a theory of pile failure. This theory is based on the combination of two critical phenomena, such as Euler's elastic Critical Load and Critical State Soil Mechanics. The theory is described in Bhattacharya (2003), Bhattacharya and Bolton (2004b), Bhattacharya (2004). This section of the paper describes the basic ingredients of the theory.

2.1 Structural nature of pile

From a structural perspective, axially loaded piles are long slender columns with lateral support provided by the surrounding soil. If unsupported, these columns will fail in buckling instability and not due to crushing of the pile material. Figure 4(a) shows the failure pattern of structures resting on slender columns which would represent a piled building or a bridge in the absence of soil. Thus in the absence of soil, we would expect a pile-supported structure to fail in a similar pattern but it remains to be seen if liquefied soil behaves like "*absence of soil*". It must be mentioned that this failure is due to the effect of axial load alone. The static axial load at which a frame supported on slender columns (Figure 4(a)) becomes laterally unstable is commonly known as the "Elastic Critical Load" of the frame or the buckling load. The simplest way is to estimate the buckling load of the frame is to find out the Critical Load (P_{cr}) of one pile and multiply by the number of piles. P_{cr} is estimated using Equation 1 where L_{eff} is the "Euler's effective length of an equivalent pin-ended strut". Figure 4(b) shows the concept of effective length of pile adopted from column stability theory to normalise the different boundary conditions of pile tip and pile head.

$$P_{cr} = \frac{\pi^2 EI}{L^2_{eff}} \tag{1}$$

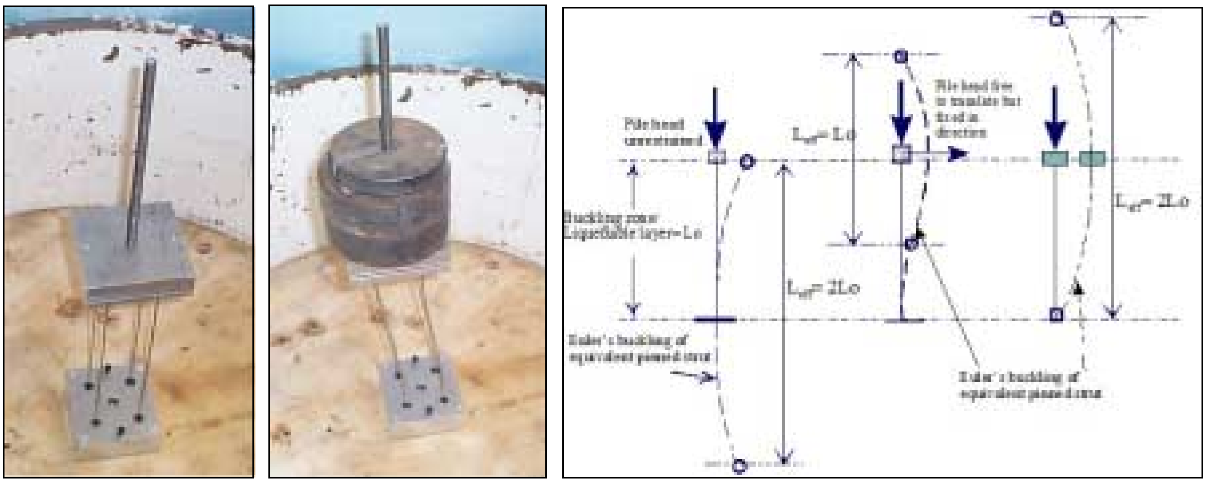

Figure 4: Simple concepts; (a): Buckling of slender columns; (b): Concept of Effective length for different boundary conditions of pile above and below the liquefiable layer.

Stability analysis of elastic columns, Timoshenko and Gere (1961) shows that the lateral deflections caused by lateral loads are greatly amplified in the presence of axial loads. In other words, in the presence of lateral loads a frame will buckle at a much lower load. Figure 5(a) shows a graph of buckling amplification factor plotted against the normalized axial load (P/P_{cr}) where P denotes the applied axial load. It can be observed from the graph that at 50% "Critical Load" the amplification of lateral deflection due to lateral loads is about 1.5 times. At these large deflections, secondary moments will generate which will lead to more deflections and thus more moment. It is thus advisable to keep the axial load within 35% of Euler's load and in these cases there are no chances of amplification. This is the approach of the structural engineers. It must be mentioned here that structural engineers generally prefer to keep a factor of safety of about 3 against linear elastic buckling to take into account the eccentricity of load, deterioration of elastic stiffness due to plastic yielding and imperfections.

Figure 5(b) shows a typical graph showing the allowable load of a pile and the buckling load, if unsupported. It may be observed that as the length of the pile increases the allowable load increases but the buckling load decreases by the square of the length. Thus, if unsupported for a particular length a buckling instability problem might occur for most piles. Dynamic centrifuge tests were carried out by Bhattacharya (2003) to verify the hypothesis of pile failure of buckling instability, Bhattacharya (2003), Bhattacharya et al (2004a, 2004b), Bhattacharya and Bolton (2004a,2004b,2004c). The tests were carried out in level grounds to avoid the effects of lateral spreading. Figure 6(a) shows the failure of piles observed in the centrifuge tests and in each case of the failure the pile head mass rotated. It is quite similar to the failure of the piled Kandla tower in Figure 6(b). Figure 6(c) shows the hinge formation in the model pile at the upper part which is similar to the observed hinge formation field case records, see Figure 6(d). Thus a failure mode observed in the field could be replicated in the centrifuge test. It shows that buckling is a feasible failure mechanism.

3. IMPLICATIONS TO PRACTICE

Buckling and bending has two different approaches in design. Current design codes are all based on bending. Bending is a stable mechanism as long as the pile is elastic, i.e. if the lateral load is withdrawn; the pile comes back to its initial configuration. This failure mode depends on the bending strength (moment for first yield, M_Y; or plastic moment capacity, M_P) of the member under consideration. On the other hand, buckling is an unstable mechanism. It is sudden and occurs when the elastic critical load is reached. It is the most destructive mode of failure and depends on the geometrical properties of the member, i.e. slenderness ratio, and not on the yield strength of the material.

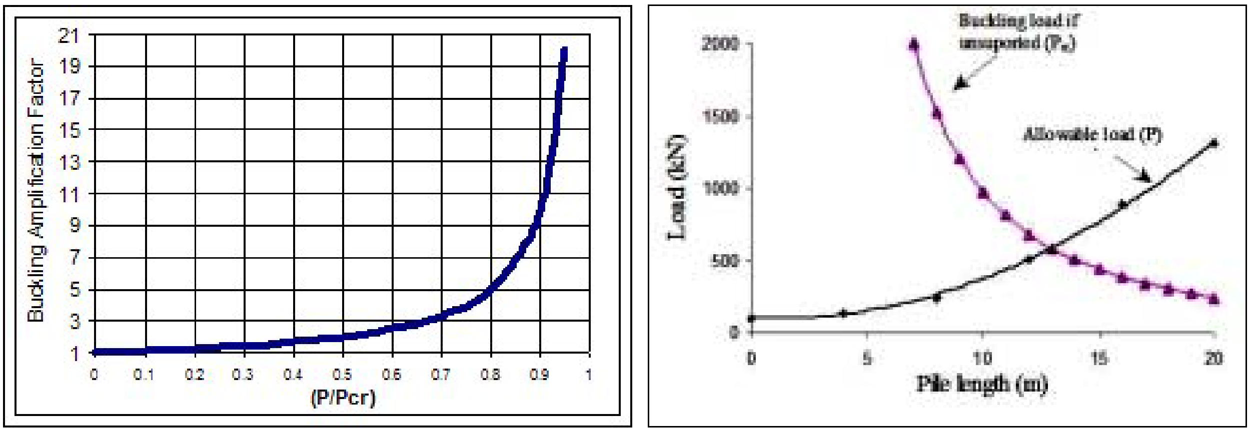

Figure 5: (a): Buckling amplification factor and normalized axial load; (b): Allowable load (P) and Buckling load (P_{cr}) of a pile, if unsupported.

Figure 6: Model test and field case history; (a): Pile foundation after the dynamic centrifuge test; (b): Failed Kandla Port tower; (c): Hinge formation in the centrifuge test; (d): Hinge formation in a building, Tokimatsu et al (1997).

Bending failure may be avoided by increasing the yield strength of the material, i.e. by using high-grade concrete or additional reinforcements, but it may not suffice to avoid buckling. To avoid buckling, there should be a minimum pile diameter depending on the depth of the liquefiable soil. This section of the paper describes a new approach. There is also a need to reconsider to safety of the existing piled foundations designed by current codes of practice i.e. based on bending mechanism.

3.1 Need of a new design method to include buckling

Figure 7(a) shows schematically the possible modes of pile failure. Details of the description can be seen in Bhattacharya (2003), Bhattacharya et al (2004a). However, it is clear that a piled foundation (a): has to sustain the axial load at all times without buckling; (b): should not form a collapse mechanism under the combined action of lateral load and axial load. Figure 7(b) shows a graph of minimum pile diameter required based on the depth of liquefiable soil from Bhattacharya (2003).

3.2 A simple method to identify the existing unsafe structures

This section of the paper will describe a way to identify the structures which may be unstable in an event of liquefaction. The examples of LPG tanks, Figure 8(a), during Kobe earthquake is taken to demonstrate the efficacy of the method. In the same site, tank 101 performed well while tank 106 failed. Let P be the axial load acting on each pile beneath the piled tank shown in Figure 8(b). It will be assumed that each pile is equally loaded. For a pile fixed at the tip and fixed in direction but free to translate at the top, the effective length (L_{eff}) is the unsupported length Figure 4(b). Considering the effect of lateral loads, it is reasonable to adopt a 35% of critical as safe load, following Figure 5(a) and as mentioned in the text. A parameter called critical depth (H_c) is now defined to identify the unsupported length of the pile required for instability. For this type of structure (i.e. bottom part of the pile embedded firmly in non-liquefiable layer and the top part is free to translate but fixed in direction) H_C, is given by equation 2.

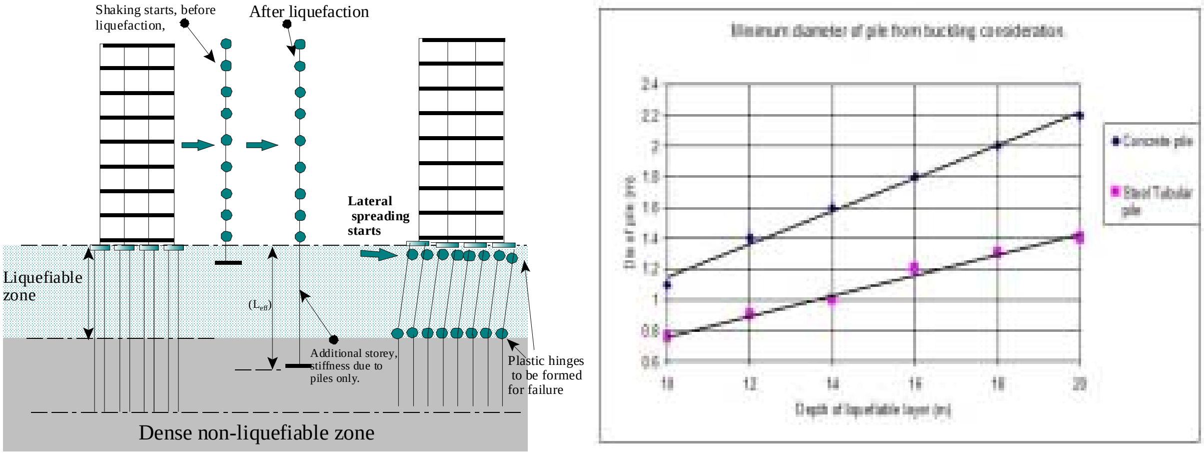

Figure 7: (a) Possible failure modes of pile foundation in a liquefiable region; (b): A proposed minimum pile diameter requirement to avoid buckling, details can be seen in Bhattacharya (2003).

$$P = 0.35P_{cr} = 0.35\frac{\pi^2 EI}{H_c^2} \text{, which leads to } H_c = \sqrt{\frac{3.45EI}{P}} \quad (2)$$

It must be noted that the critical depth concept is a by-product of L_{eff} but it is particularly fruitful from practical point of view to evaluate the safety of existing piled foundations prone to axial instability. A piled structure becomes unstable for $H_C<D_L$, where D_L is the depth of liquefiable layer. Table 1 estimates the critical depth for the tank foundations. P is estimated based on the allowable load concept, Bhattacharya (2003). Relevant details of the case history can be seen in Ishihara (1997). Analysis of the case history can be seen in Bhattacharya (2003).

Table 1: Estimation of critical depth

Case history	**EI of the pile**	**P**	**H_C (Eq2)**	**D_L (Depth of liquefiable soil)**	**Remarks**
LPG Tank 101	1.79×10^9N.m^2 1.1m dia RCC	4.1MN	38.8m	15m	Here $H_C>D_L$ and thus should be stable. It performed well
LPG Tank 106	11.15×10^6N.m^2 0.3m dia RCC hollow	0.46MN	9.14m	15m	Here $H_C<D_L$ and thus should be unstable. It failed

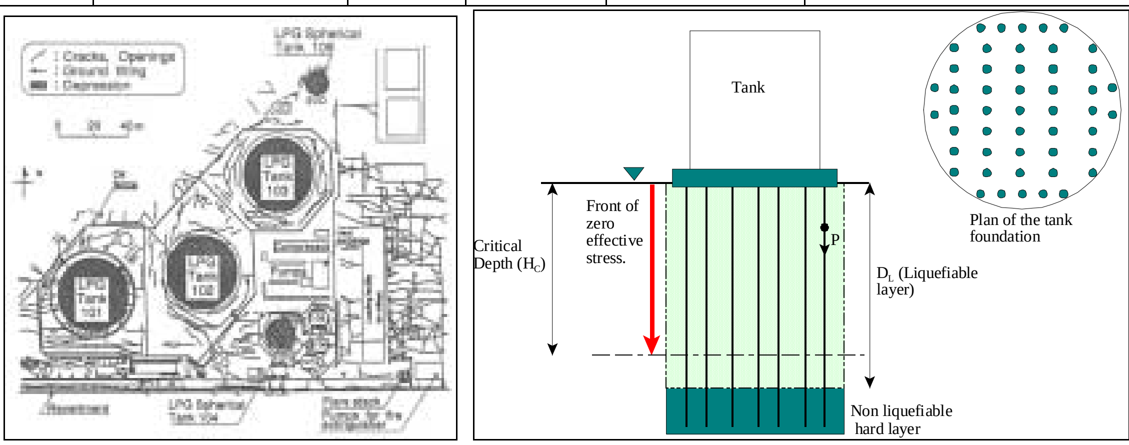

Figure 8: LPG tanks (a): Tanks 101 performed well while tanks 106,107 failed; (b): Schematic diagram of a piled tank.

The method is valid for the particular type of piled foundations mentioned in the text i.e. large pile caps or piled rafts with embedment of at least 6 times the diameter of the pile in the non-liquefied layer beneath the liquefiable layer. For other boundary conditions, the critical depth H_C will change depending on the fixity of the file below and above the liquefiable soil. This can be easily computed based on the equation 1 and Figure 4(b) but will be of the same form as equation 2. After checking the stability against buckling, the designer needs to check against the formation of a collapse mechanism as shown in Figure 7(a). While checking against the collapse mechanism, the reduction of bending stiffness of the pile needs to be considered to take into account the effect of the axial stress.

4. CONCLUSIONS

Buckling is a feasible pile failure mechanism in areas of seismic liquefaction. To avoid buckling there is a requirement of a minimum diameter of pile based on the depth of liquefiable soil. In contrast, current understanding is based on bending mechanisms. Buckling and bending requires different approach in design. Designing against bending would not automatically suffice the buckling requirements. Thus there is a need to reexamine the safety of existing piled foundations designed based on bending mechanism. A simple method to evaluate such safety is proposed in this paper. Designers should use large diameter pile is instead of a group of small diameter pile.

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