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# BUCKLING OF PILES DURING EARTHQUAKE LIQUEFACTION

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## SUMMARY

Collapse of pile-supported structures by formation of plastic hinges in the piles is observed in the majority of recent strong earthquakes despite the fact that a large factor of safety is employed in their design. Studies show that when failure occurred in structures, they most often resulted due to loads that have been overlooked by the designer or considered as secondary; rather than inadequate factor of safety. Recent research into the pile failure mechanism has shown that there is also a fundamental omission of a load effect in the seismic pile design in liquefiable areas. The current codes of practice for pile design such as Euro code 8, NEHRP 2000, JRA1996 and IS1893 is based on a bending mechanism where lateral loads due to inertia or slope movement (Lateral spreading) 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 during soil liquefaction due to the diminishing confining pressure surrounding the pile. It is needless to mention that irrespective of lateral loads imposed by the earthquake, a pile has to sustain the axial load acting on it when the surrounding soil is at its lowest possible strength and stiffness owing to liquefaction. The provisions in the current codes are inadequate and buckling needs to be addressed. It must be mentioned here that buckling is the most destructive form of failure and it occurs suddenly. 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. In pile design, to avoid buckling there is a need to have minimum diameter depending on the depth of the liquefiable soil, typically length to diameter ratio of about 12 in the likely liquefiable zone. 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 the practical implications and new research needs.

## INTRODUCTION

It is not an overstatement 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 a 5km of 750mm diameter gas pipe line during testing from the Jamuna Bridge in Bangladesh on 11th June 1998. It has been reported NCE [1] that the most probable cause of the collapse was the failure to allow in design the weight of the water in the pipe needed for

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testing purposes. Other examples can be seen in Bhattacharya [2]. Collapse of pile-supported structures in liquefiable soils is still observed after strong earthquakes such as 1964 Niigata (Japan) earthquake, 1995Kobe (Japan) earthquake, 1999 Chi-Chi (Taiwan) earthquake, 1999 Koceli (Turkey) earthquake and 2001 Bhuj earthquake. Bhattacharya [2,3,4], Bhattacharya and Bolton [5,6,7] has shown that there is also a fundamental omission of a load effect in seismic pile design that may have contributed to the failure of many pile foundations.

**Pile failure and the Factor of Safety employed in their design**

Failure of piled foundations has been observed in the majority of the recent 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 as will be seen later in Figures 2(a & b). 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 [8] or a three storied building, Tokimatsu [9]. It has been shown by Bhattacharya [4] 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 before plastic yielding; a pile can sustain a load 4 to 8 times than that predicted by the code of practice. Thus unless wrong foundation design concepts are employed or the seismic forces are severely underestimated, failure by plastic yielding of piles is unlikely.

**Current understanding of pile failure and the codes of practice**

The current understanding of pile failure is based on a bending mechanism where lateral loads due to inertia and slope movements (lateral spreading) induce bending in the pile; see Figure 1(a). Permanent lateral deformation or lateral spreading is reported to be the main source of distress in piles, Abdoun and Dobry [10], Finn and Fujita [11], Hamada [12]. This unanimity has led the Japanese Code of Practice JRA [13] for example, (see Figure 1(b)) to include checks on bending moments in piles due to lateral spreading of the ground. The code advises practicing engineers to design piles against bending failure assuming that the non-liquefied crust offers passive earth pressure ($q_{NL}$) to the pile and the liquefied soil offers 30% of the total overburden pressure ($q_L$).

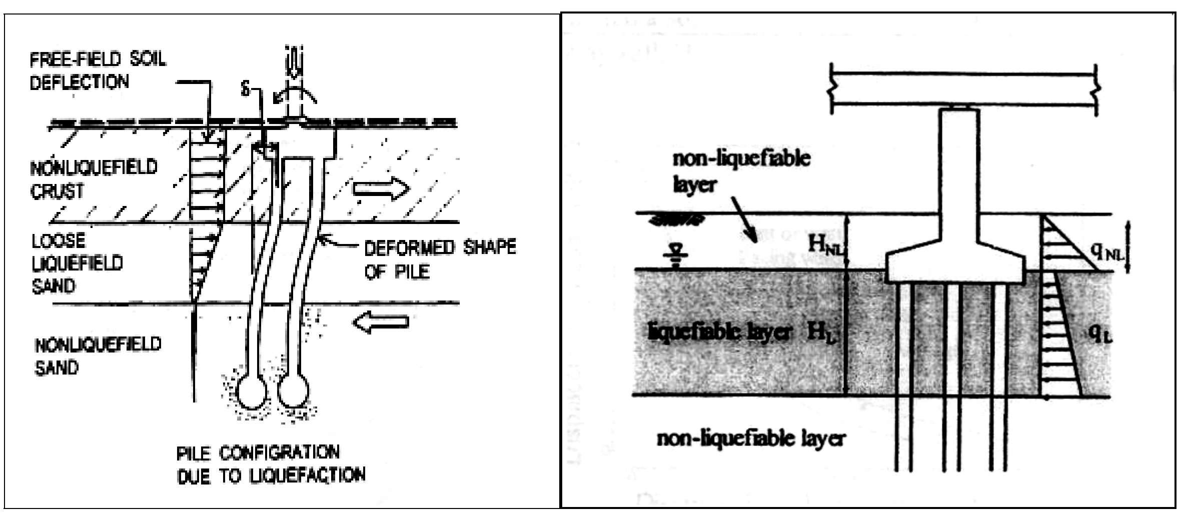


Figure 1: (a) Current understanding of pile failure, Finn and Thavaraj [14]; (b): JRA [13] code of practice.

The Eurocode 8 [15] advises designers to design piles against bending due to inertia and kinematic forces arising from the deformation of the surrounding soil. It goes on saying:
"*Piles shall be designed to remain elastic. When this is not feasible, the sections of the potential plastic hinging must be designed according to the rules of Part 1-3 of Eurocode 8*".
Eurocode 8 (Part 5) says
"*Potential plastic hingeing shall be assumed for:*
- *a region of 2d from the pile cap*
- *a region of ± 2d from any interface between two layers with markedly different shear stiffness (ratio of shear moduli > 6)*

*where d denotes the pile diameter. Such region shall be ductile, using proper confining reinforcements"*.

Other codes such as NEHRP code [16] and Indian Code (IS 1893) [17] also focus on bending strength of the pile. In summary, the current understanding of pile failure simply treats piles as beam elements and assumes that the lateral loads due to inertia and slope movement cause bending failure of 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.

## CRITICAL REVIEW OF THE CURRENT UNDERSATNDING OF PILE FAILURE (BENDING MECHANISM)

This section of the paper highlights some of the inconsistencies of the observations of pile failure. They are summarized below:

**Pile foundations were observed to collapse even in level grounds in a similar way to that observed in laterally spreading grounds**
After the detailed investigation of the failure of piles during 1995 Kobe earthquake, Tokimatsu and Asaka [18] report 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, …..".

Figure 2(a): Failure of pile foundation in level grounds, after the 1995 Kobe earthquake, Tokimatsu [19]; (b): Collapse of piled Kandla Tower in laterally spreading ground after the 2001 Bhuj Earthquake, Madabhushi et al [20].

Figure 2(a) shows the failure of a piled foundation in a level ground and Figure 2(b) shows the failure of a piled foundation in a laterally spreading ground. Both the foundations tilted in the direction of the asymmetrical mass i.e. in the direction of the eccentricity of vertical loading. It is surprising that a piled foundation collapses in a similar way in level ground i.e. in absence of lateral spreading and in laterally spreading grounds i.e. in the presence of lateral spreading. If lateral spreading is the main cause of failure, Hamada [12]; it is most unlikely that a piled foundation will collapse in level grounds. It must also be noted that most of PHC piles which had a high bending strength also failed.

**A note in the failure of bridges**

It is a common observation in seismic bridge failure that piers collapse while abutments remain stable, for example Figures 3(a&b). Figure 3(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 3(b)). Bhattacharya and Bolton [5] 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 4). 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 design methods only concentrate on lateral loads.

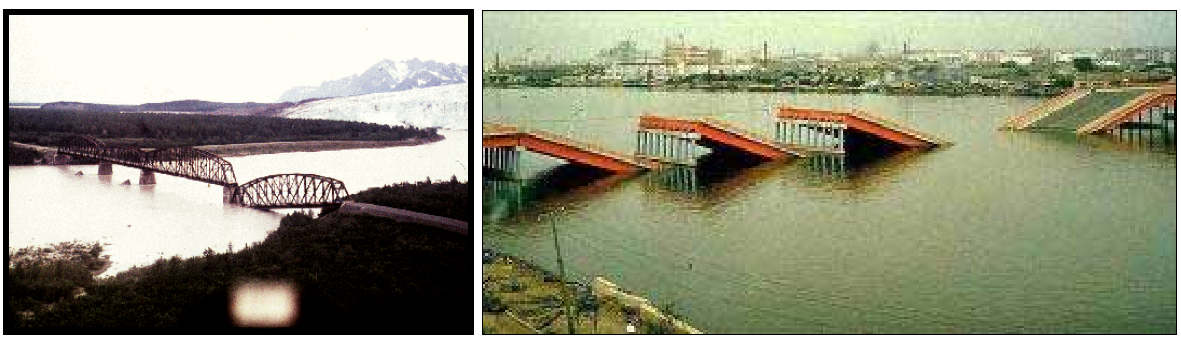

Figure 3: Failure of bridges during earthquakes; (a): Million Dollar Bridge during the 1964 Alaska earthquake (b): Failure of the Showa Bridge after the 1964 Niigata Earthquake. Photo courtesy [21].

**Location of hinge formation**

It has been revealed after the excavation of the NHK building, the NFCH building, Showa Bridge Hamada [8] and the three-storied building, Tokimatsu et al [9] that hinges formed in piles occurred within the top third of the pile or even at the middle of the liquefiable layer. 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.

**A case study of the well known failure of the Showa Bridge**

The example of the failure of the Showa Bridge (see Figure 3(b)) is extensively used to illustrate the effects of lateral spreading loads to piled foundations, see for example Hamada [8], Ishihara [22]. Figure 5 shows the schematic representation of the failure of the bridge. As can be seen from Figure 5, piles under pier no. $P_5$ deformed towards the left and the piles of pier $P_6$ deformed towards the right Takata et al [23], Fukuoka [24]. 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. Bhattacharya et al [25], Bhattacharya [4] has shown that the piles of the Showa Bridge are safe against the current codal provisions of the JRA [13] code with a factor of safety of 1.84 but the bridge actually collapsed in 1964. It is worth mentioning that the JRA code was revised few times after the 1964 Niigata earthquake i.e. in 1972, 1980 and 1996.

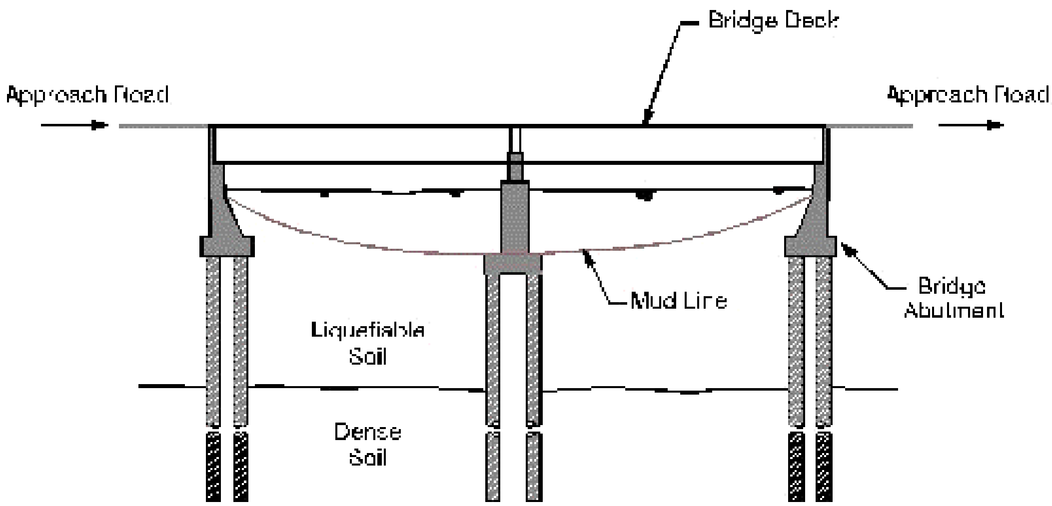


Figure 4: Schematic diagram of a bridge.

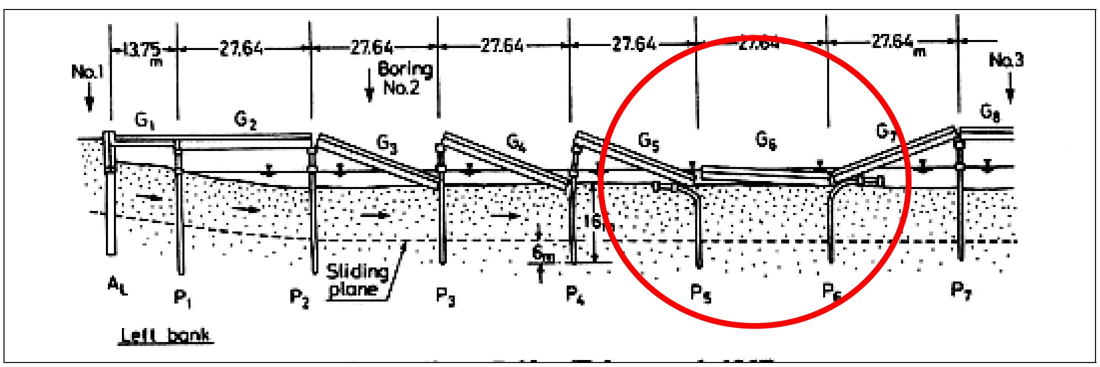


Figure 5: Schematic diagram of the failure of the Showa Bridge after Takata et al [23]. The diagram only shows half of the bridge. It must be noted that the direction of deflection shown by the red dashed circle contradicts the assumption of lateral spreading.

To summarize, the limitations of the current understanding of pile failure/ codes of practice 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 [4] that the pile foundation of Showa Bridge, which is considered safe by the 1996 JRA code, actually failed in 1964.

## A NEW THEORY OF PILE FAILURE

Earthquakes impose lateral loads in the pile through inertia of the superstructure and also through lateral spreading. It must be remembered that the pile has to carry the axial load throughout the design period needless to say even at full liquefaction i.e. when the soil surrounding the pile is at its lowest strength and stiffness. Structurally, piles are long slender columns with lateral support from the surrounding soil. If unsupported, these columns will fail in buckling instability and not due to the crushing of the pile material. During earthquake induced liquefaction, the pile does not get enough lateral support and it may simply buckle i.e. become unstable. Lateral loading due to slope movement (lateral spreading) or inertia or out-of-line straightness will increase lateral deflections, which in turn will reduce the buckling load and promote more rapid collapse. But these lateral loads or imperfections are secondary to the basic requirements that the piles is liquefiable soils must be checked against Euler's buckling. A new theory of pile failure based on buckling instability has been proposed by Bhattacharya [4], Bhattacharya et al [29], Bhattacharya and Bolton [5,7]. This theory is formulated by back analyzing 15 reported cases of pile foundations performance during earthquakes and later verified using dynamic centrifuge tests, Bhattacharya et al [25,29]. Analytical studies also validate the theory of pile failure; see Bhattacharya and Bolton [5], Bhattacharya [4]. A hypothesis of pile-soil interaction is also developed in the theory. The interaction can be well modeled using "Critical State" soil mechanics. The next section of the paper describes the basic ingredients of the theory.

## Basic ingredients of the theory of pile failure

*Structural nature of pile and buckling instability of frames supported on slender columns*

Bond (1989) showed through the collation of data of tubular piles used in different projects around the world that the length to diameter of the pile ranges from 25 to 100. Figure 6(a) shows the failure pattern of structures resting on slender columns (length to diameter ratio of 93) which would represent a piled building or a bridge in absence of soil but may not necessarily mean in the absence of soil support. 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 the "*absence of soil*". It must be remembered that the failures shown in Figure 6(a) is due to axial load alone. Example of failure of slender columns in a single row as in Showa Bridge (Figure 3(b)) can be seen in Bhattacharya et al [3].

The static axial load at which a frame supported on slender columns becomes laterally unstable is commonly known as the "Elastic Critical Load" of the frame or simply "Buckling load ($P_{cr}$)". The elastic critical load can be estimated based on the concept of "Euler's effective length of an equivalent pin-ended strut ($L_{eff}$)". Figure 7 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. The Euler's elastic critical load ($P_{cr}$) i.e. the well known buckling formula given by Equation 1 can be extended to predict the buckling load of frames supported on slender columns. The simplest way to estimate the buckling load of a piled building is to calculate the buckling load one pile depending on the boundary condition of the pile below and above the liquefiable soils and multiply by the number of piles forming the building. It must be recognized that the buckling of piles is different from the ones usually seen in the text books. This is because in the case of pile buckling, the top part of the pile is free to translate laterally unless raked piles are used, see for example the Landing Bridge; Berrill et al [27].

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

The failure shown in Figure 6(a) is only due to the effect of static axial load. In the presence of static lateral loads, the critical load would come down i.e. the frame will become unstable at much lower load typically at 50% of the elastic critical load. In presence of dynamic loads, i.e. if the entire set up shown in Figure 6(a) is shaken as in an earthquake, the frame would become unstable at a much lower load. Stability analysis of elastic columns, see Timoshenko and Gere, [28], shows that the lateral deflections caused by lateral loads gets amplified in the presence of axial loads. If $\delta_0$ is the deflection due to lateral loads alone, the final deflection $\delta$ gets amplified in the presence of axial loads. The term ($\delta/\delta_0$) can be termed as the "Buckling amplification factor (BAF)" given by equation 2.

$$B.A.F = \frac{1}{\left(1 - \frac{P}{P_{cr}}\right)} \tag{2}$$

This form of expression, sketched in Figure 6 (b), can be used with good accuracy (less than 2% error) for all beam-columns having ($P/P_{cr}$) less than 0.6, Timoshenko and Gere [28]. Beyond the ratio of 0.6 the induced plastic strains cause a deterioration of elastic bending stiffness leading to a reduction in the critical buckling load $P_{cr}$, and to premature collapse.

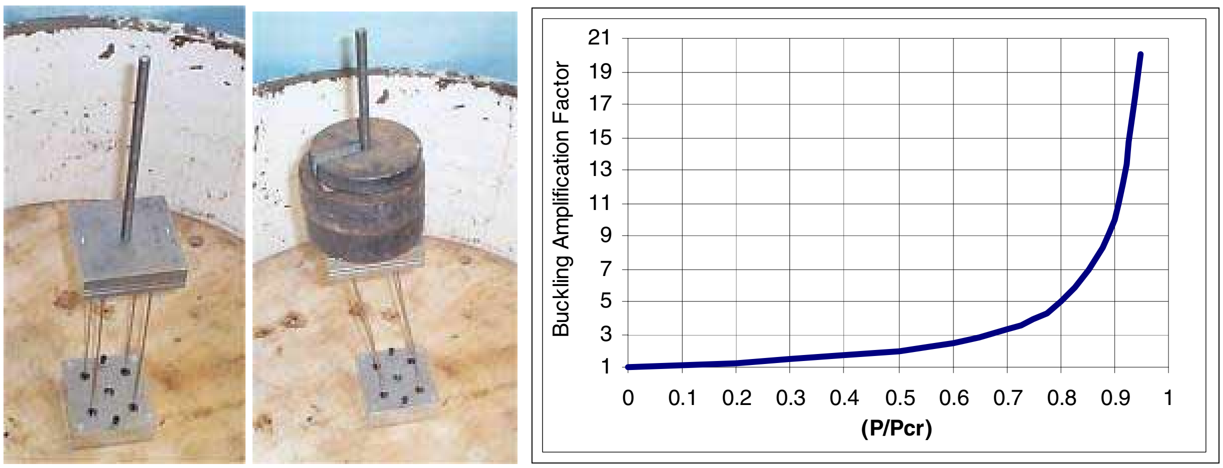


Figure 6(a): Failure of frame supported on long columns. (b): Buckling amplification factor.

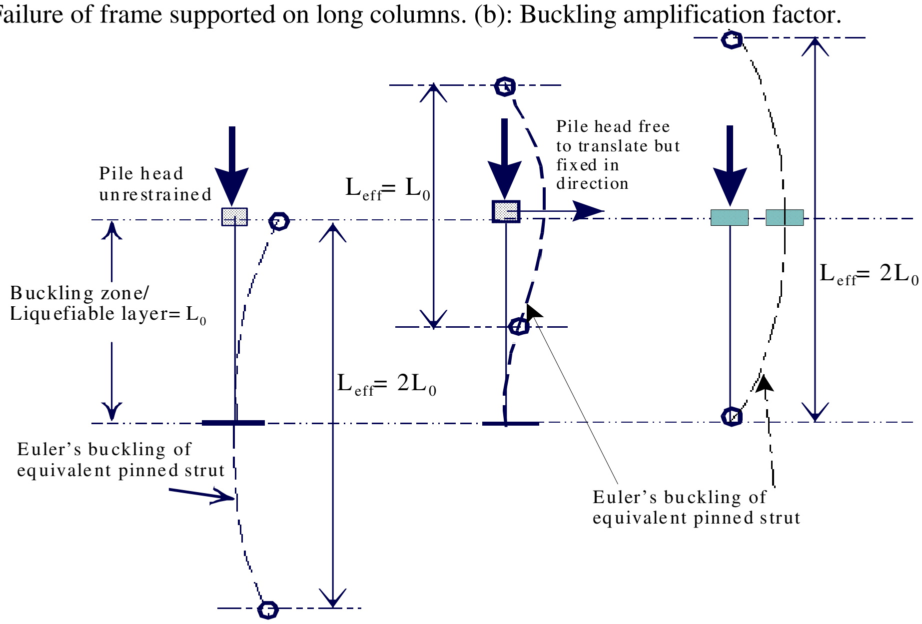


Figure 7: Concept of effective length of long slender column/ pile

The theory of pile failure is based on an idealization that a pile-supported structure such as a bridge or a building is a frame supported on slender columns as shown in Figures 6(a) with the support from the liquefiable soil. The part of the pile in liquefiable soil is the unsupported zone during seismic liquefaction. Each of these structures has a critical load i.e. the minimum axial load at which the frame becomes unstable.

*Allowable load and critical load of a pile. Is it practically important?*

Generally, as the length of the pile increases, the allowable load on the pile also increases primarily due to the additional shaft friction, but the buckling load (if the pile were laterally *unsupported* by soil) decreases inversely with the square of its length following Euler's formula (Equation 1). Figure 8(a) shows a typical plot for the variation of allowable load (P) and buckling load ($P_{cr}$) of a pile (if *unsupported*) against length of the pile. The pile in the above example has a diameter of 300mm (typical pile dimension in 1964 Japan) and is passing through a typical liquefied soil. The allowable load (P) is estimated based on conventional procedures with no allowance for liquefaction.

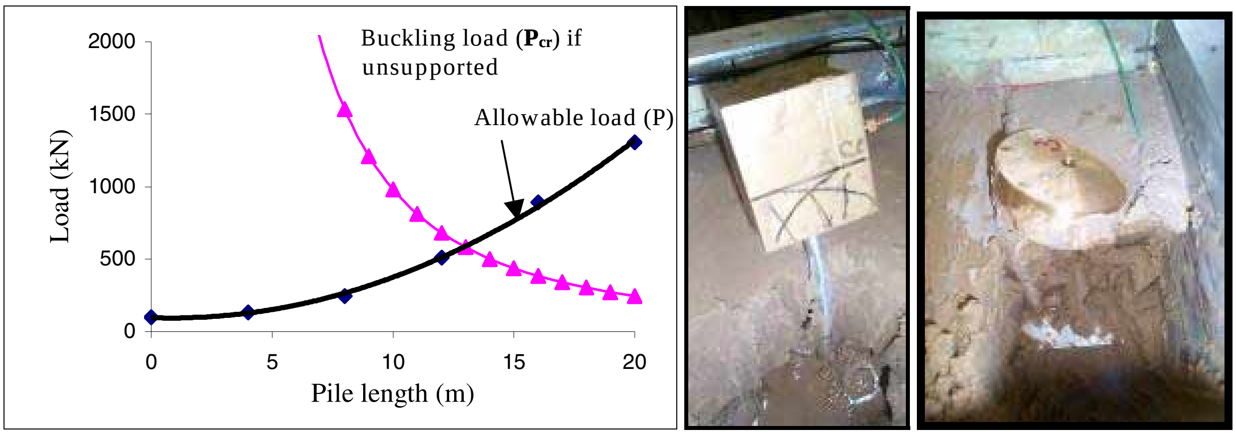


Figure 8: (a): Allowable load and buckling load of a pile (if unsupported); (b): Pile buckling in a centrifuge test.

*Dynamic centrifuge tests to verify the hypothesis of pile failure*

Dynamic centrifuge tests were carried to verify if fully embedded end-bearing piles passing through saturated, loose to medium dense sands and resting on hard layers buckle under the action of axial load alone if the surrounding soil liquefies in an earthquake. Details of the test can be seen in Bhattacharya et al [29], Bhattacharya and Bolton [5], Bhattacharya [4]. The centrifuge tests were designed in level grounds to avoid the effects of lateral spreading. Twelve piles were tested in a series of four centrifuge tests including some which decoupled the effects of inertia and axial load. Axial load was applied to the pile through a block of brass fixed at the pile head, Figure 8(b). With the increase in centrifugal acceleration, the brass weight imposes increasing axial load in the pile. Figure 8(b) shows two cases of pile failure. The stress in the pile section is well within the elastic range of the material (less than 30% of the yield strength) but it failed as the earthquake is fired. This confirms that the support offered by the soil was eliminated by the earthquake liquefaction and that the pile started to buckle to buckle in the direction of least elastic stiffness. The conclusions from the centrifuge tests can be summarized as below:

1. If the axial load is high enough, it may not be necessary to invoke lateral spreading of the soil to cause a pile to fail. A pile may collapse even before lateral spreading starts once the surrounding soil liquefies.
2. Liquefied soil cannot provide enough restraint to a pile susceptible to buckling. In other words, liquefied soil cannot stop the initiation of buckling. But as the pile starts to buckle pushing the initially liquefied soil, secondary resistance becomes available.

Thus a pile is unsupported against Euler's elastic buckling during seismic liquefaction. Lateral loads can make it worse.

**Study of case history**

In this study, fifteen reported cases of pile foundation performance during earthquake-induced liquefaction has been studied and analysed as listed in Table 1. Six of the piled foundations were found to survive while the others suffered severe damage; see Bhattacharya [4], and Bhattacharya et al., [29]. Details of each case history can be seen in Bhattacharya [4]. There are many uncertainties involved in the estimation of the parameters used to study the case histories. The parameters $r_{min,}$ however has no uncertainty. On the other hand, $L_{eff}$ depends on the correct estimation of the liquefiable layer and the boundary condition of the pile below and above the liquefiable layer.

Table 1: Summary of case histories.

| ID in Fig 9 | Case History and Reference | Pile section/ type | $L_0$* (m) | $L_{eff}$ (m) | $r_{min}$ (m) |
|---|---|---|---|---|---|
| A | 10 storey-Hokuriku building, Hamada[8] | 0.4m dia RCC | 5 | 5 | 0.1 |
| B | Landing bridge, Berrill et al [27] | 0.4m square PSC | 4 | 2 | 0.12 |
| C | 14 storey building, Tokimatsu et al [30] | 2.5m dia RCC | 12.2 | 12.2 | 0.63 |
| D | Hanshin expressway pier, Ishihara [31] | 1.5m dia RCC | 15 | 15 | 0.38 |
| E | LPG tank 101, Ishihara [31] | 1.1m dia RCC | 15 | 15 | 0.28 |
| F | Kobe Shimim hospital, Soga [32] | 0.66m dia steel tube | 6.2 | 6.2 | 0.23 |
| G | N.H.K building, Hamada [8] | 0.35m dia RCC | 10 | 20 | 0.09 |
| H | NFCH building. Hamada [8] | 0.35m dia RCC hollow | 8 | 16 | 0.10 |
| I | Yachiyo Bridge Hamada [8] | 0.3m dia RCC | 8 | 16 | 0.08 |
| J | Gaiko Ware House, Hamada [8] | 0.6m dia PSC hollow | 14 | 28 | 0.16 |
| K | 4 storey fire house, Tokimatsu et al [30] | 0.4m dia PSC | 18 | 18 | 0.10 |
| L | 3 storied building at Kobe university, Tokimatsu et al [9] | 0.4m dia PSC | 16 | 16 | 0.12 |
| M | Elevated port liner railway, Soga [32] | 0.6m dia RCC | 12 | 12 | 0.15 |
| N | LPG tank –106,107 Ishihara [31] | 0.3m dia RCC hollow | 15 | 15 | 0.08 |
| O | Showa bridge, Hamada [8] | 0.6m dia steel tube. | 19 | 38 | 0.21 |

$L_0^*$ = Length of pile in liquefiable region/Buckling zone

The parameters $r_{min}$ (minimum radius of gyration) and $L_{eff}$ (effective length of the pile in liquefiable region) are introduced to analyse the piles. The definitions of the parameters are given below.

1. $r_{min}$ : The minimum radius of gyration of the pile section about any axis of bending (m).This parameter can represent piles of any shape (square, tubular or circular) and is used by structural engineers for studying buckling instability and is given by Equation 3.

$$r_{min} = \sqrt{\frac{I}{A}} \tag{3}$$

where I is second moment area of the pile section about the weakest axis ($m^4$) and A is area of the pile section ($m^2$). For solid circular piles $r_{min}$ is 0.25 times the diameter of the pile and for tubular piles $r_{min}$ is approximately 0.35 times the outside diameter of the pile.

2. $L_{eff}$ = Effective length of the pile in the liquefiable region. The definition of effective length has been adopted from column stability theory. This parameter depends on the depth of liquefiable region ($L_0$) and the boundary condition of the pile as shown in Figure 7. The different boundary conditions of pile tip and pile head can be adopted as shown in Figure 7. $L_{eff}$ is also familiar as the "Euler's buckling length" of a strut pinned at both ends. In practice, designers may prefer to adjust effective length slightly to account for imperfect fixity.

The ratio $L_{eff}/r_{min}$ is termed as slenderness ratio of pile in the liquefiable region. Figure 9 shows the effective length of the piles in liquefiable zone plotted against the $r_{min}$ of the pile section with identification of whether the pile has failed or not during earthquakes. A line representing a slenderness ratio ($L_{eff}/r_{min}$) of 50 is drawn and it distinguishes poor performance piles from the good ones. This line is of some significance in structural engineering, as it is often used to distinguish between "long" and "short" columns. Columns having slenderness ratios below 50 are expected to fail in crushing whereas those above 50 are expected to fail in buckling instability. Thus, the analysis suggests that pile failure in liquefied soils is similar in some ways to the failure of long columns in air. The lateral support offered to the pile by the soil prior to the earthquake is removed during liquefaction. Thus the case histories validate the centrifuge test results.

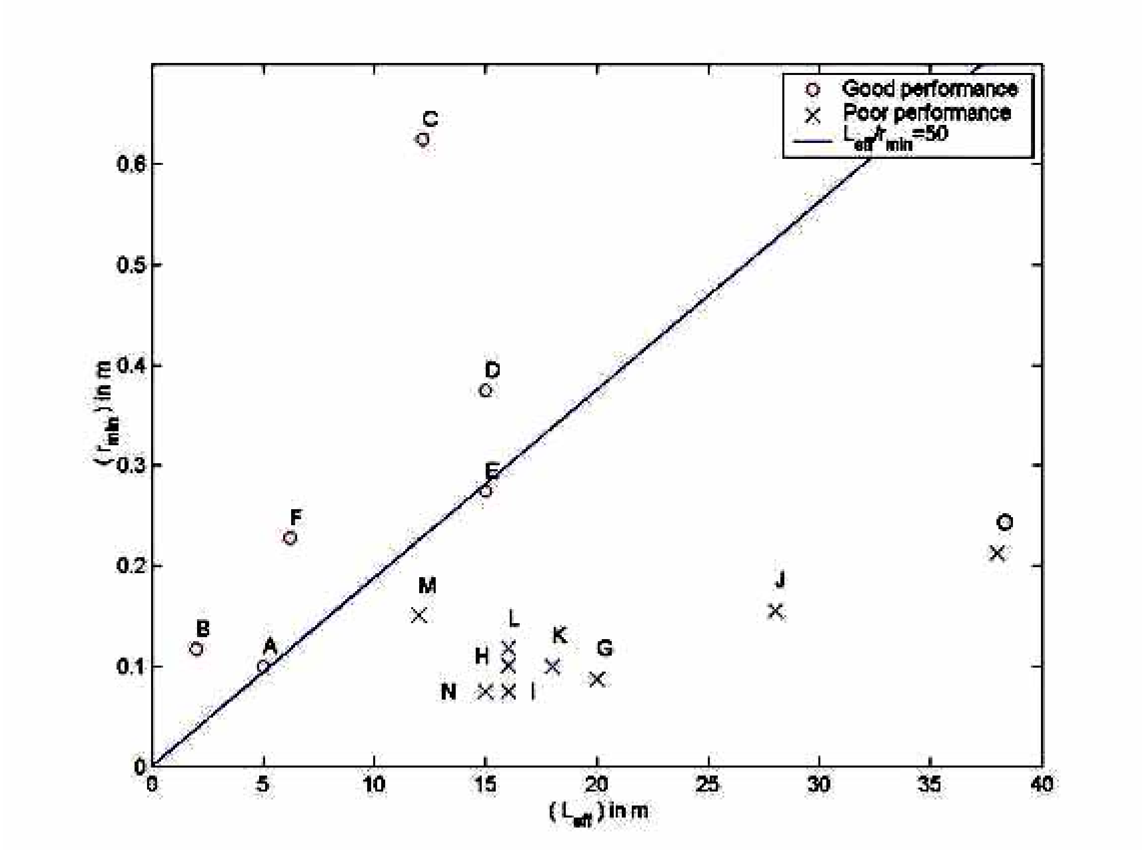


Figure 9: $L_{eff}$ versus $r_{min}$ for piles studied.

## NEW DESIGN APPROACH

In design, beam bending and column buckling are approached in two different ways. Bending is a stable mechanism, i.e. if the lateral load is withdrawn; the pile comes back to its initial configuration provided the yield limit of the material has not been exceeded. 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.

For example, steel pipe piles having identical length and diameter but having different yield strength [$f_y$ of 200MPa, 500MPa, 1000MPa] will buckle at almost the same axial load but can resist different amounts of bending. 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. In contrast, piles have been erroneously being designed as a beam.

**Essential criteria for design of pile foundations in liquefiable areas**

Bhattacharya and Tokimatsu (2004) identifies that a safe design procedure should ensure the following:

1. A collapse mechanism should not form in the piles under the combined action of lateral loads imposed upon by the earthquake and the axial load. Figure 10(a) shows such a mechanism. At any section of the pile, bending moment should not exceed allowable moment of the pile section. The shear stress at any section of the pile should not exceed the allowable shear capacity.
2. A pile should have sufficient embedment in the non-liquefiable hard layer below the liquefiable layer to achieve fixity to carry moments induced by the lateral loads. If proper fixity is not achieved, the piled structure may slide due to the kinematic loads. The fixity depth is shown by $D_F$ in Figure 10(b). Typical calculations carried out using the method proposed by Davisson and Robinson [34] shows that the point of fixity lies between 3 to 6 times the diameters of the pile in the non-liquefiable hard layer. Details can be seen in Bhattacharya [4].
3. The pile has enough strength and stiffness to carry the axial load acting on it during full liquefaction without buckling and becoming unstable. It has to sustain the axial load and vibrate back and forth, i.e. must be in stable equilibrium when the surrounding soil has almost zero stiffness owing to liquefaction. As mentioned earlier, lateral loading due to ground movement, inertia, or out-of-straightness, will increase lateral deflections which in turn can cause plastic hinges to form, reducing the buckling load, and promoting more rapid collapse. These lateral load effects are, however, secondary to the basic requirements that piles in liquefiable soils must be checked against Euler's buckling. This implies that there is a requirement of a minimum diameter of pile depending on the likely liquefiable depth.
4. The settlement in the foundation due to the loss of soil support should be within the acceptable limit. The settlement should also not induce end-bearing failure in the pile.

**Simplified procedure to avoid buckling**

As can be seen in Figure 11(a), lateral spreading loads and inertia loads may act in two different planes. Thus the pile not only has axial stress but also may have bending stresses in two axes. The pile represents a most general form of a "beam-column" (column carrying lateral loads) element with bi-axial bending. If the section of the pile is a "long column", analysis would become extremely complex and explicit closed-form solution does not exist. The solution of such a problem demand an understanding of the way in which the various structural actions interact with each other i.e. how the axial load influences the amplification of lateral deflection produced by the lateral loads. In the simplest cases i.e. when the section is "short column", superposition principle can be applied i.e. direct summation of the load effects.

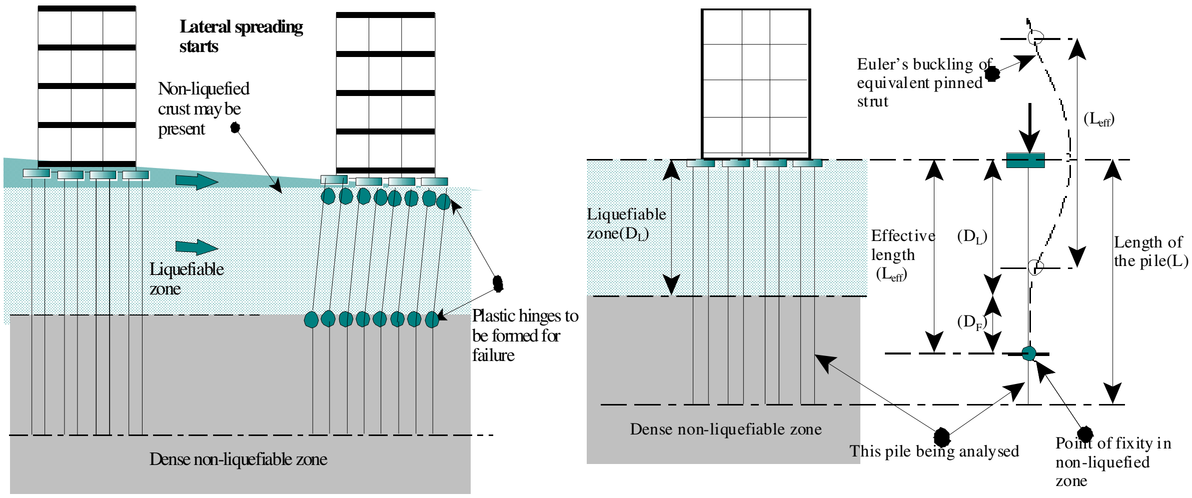


Figure 10: (a): Combined mechanism of axial and bending; (b): Buckling mechanism, Bhattacharya [4]

In other cases, careful consideration of the complicated interactions needs to be accounted. Designing such type of member needs a three-dimensional interaction diagram where the axes are: Axial (P), major-axis moment ($M_x$) and minor-axis moment ($M_y$).The analysis becomes far more complicated in presence of dynamic loads. The above complicated non-linear process can be avoided by designing the section of the pile as "short column" i.e. for concrete section - length to least lateral dimension less than 15 (British Code 8110) or a slenderness ratio (effective length to minimum radius of gyration) less than 50.

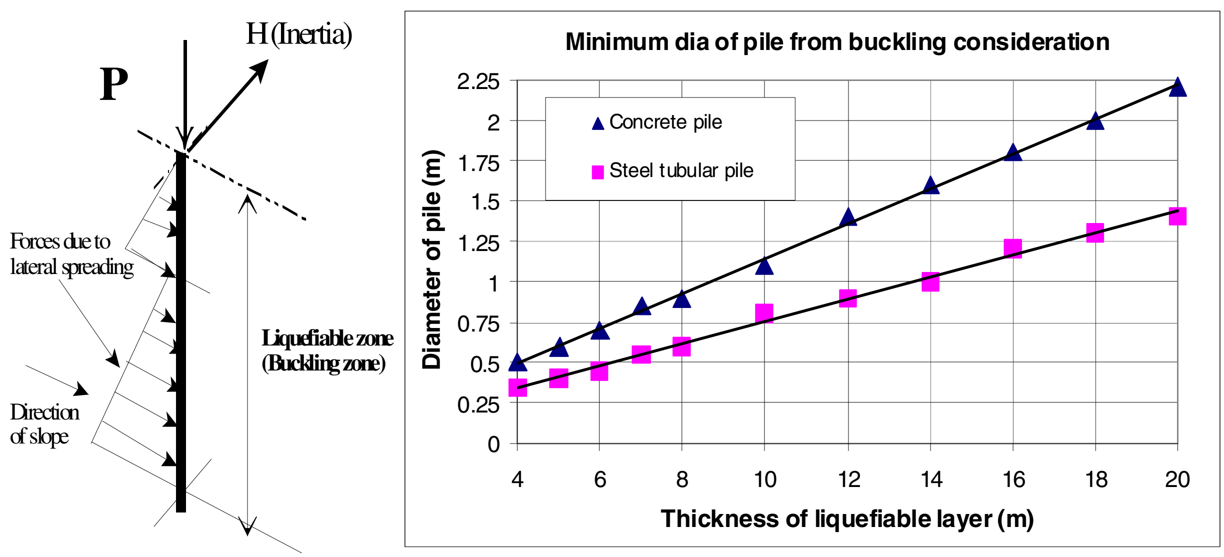


Figure 11: (a): Free body diagram showing the generalized loading acting on a pile; (b): Design chart to avoid buckling after Bhattacharya and Tokimatsu [33].

Figure 11(b) shows a typical graph showing the minimum diameter of pile necessary to avoid buckling depending on the thickness of liquefiable soil. The slenderness ratio is kept around 50. The main assumptions are that the piles are solid concrete section having E (Young's modulus) of $22.5\times10^3$MPa) and for steel E of 210GPa. The piles are not in a single row and at least in 2×2 matrix form. The thickness of the steel pile is based on API [35] code (American Petroleum Code) i.e. the minimum thickness is 6.35mm + (diameter of the pile/100). Details of the design chart can be seen in Bhattacharya (2003).

## CONCLUSIONS

It has been shown that buckling of a pile under the action of axial load alone due to the diminishing soil stiffness owing to liquefaction is a feasible pile failure mechanism during earthquakes. Lateral loads like inertia, slope movement loads will make the pile unstable at a much lower load. However, these lateral loads are secondary to the basic requirements that piles in liquefiable soil must be checked against Euler's buckling.

### Recommendations to Practice

1. Codes of practice need to include a criterion to prevent buckling of slender piles in liquefiable soils. The designer should first estimate the equivalent length for Euler's buckling, by considering any restraints offered by the pile cap, or the zone of embedment beneath the liquefiable soil layer. It is then necessary to select a pile section having a margin of factor of safety against buckling under the worst credible loads.
2. Designers should specify fewer, large modulus piles, in order to avoid problems with buckling due to liquefaction.
3. Cellular foundations of contiguous, interlocked sections should also be effective

### Further research

The research presented in this paper has identified the limitations of the existing design methods of piled foundations in liquefaction hazard areas for e.g. Japanese Road Association JRA, Euro code 8, and NEHRP. It seems that many of the bridges and buildings designed based on the existing design codes are unsafe. Based on the above fact the following research need is identified. The immediate need is to re-evaluate the safety of the structures designed based on the existing design methods and codes of practice. Structures that are unsafe will need retrofitting to withstand future impacts of earthquakes. Keeping this view in mind, the suggested future research work is outlined below.

Identifying the parameters for systematic evaluation of safety of existing structures founded on piles designed based on existing design methodologies. Some of the parameters identified are:
(a): Site characterisation i.e. depth of liquefiable soil at the site of the structure, slope of the ground, seasonal variation of ground water table. This would help to identify the non-liquefied crust at the site and expected lateral loads in the pile.
(b): Slenderness ratio of piles in liquefiable region. This would check the stability of pile against Euler's buckling.

Once unstable structures (for e.g. abutment/piers of bridges, or piled buildings) are identified, strategies for retrofitting have to be researched. This will involve means to improve stability of foundations. A simple method of identifying the existing unsafe structures can be seen in Bhattacharya [3].

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