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CRITERIA FOR DESIGN OF PILED FOUNDATIONS IN SEISMICALLY LIQUEFIABLE DEPOSITS

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ABSTRACT

Collapse and/or severe damage to pile-supported structures are still observed in liquefiable soils after most major earthquakes. Therefore this remains a great concern to the earthquake engineering community. This paper critically reviews the current understanding of pile failure in seismically liquefiable soils. The plausible failure mechanisms of these foundations that have been identified so far are summarised. Essential criteria for design of pile foundations have been proposed.

Keywords: Pile, Liquefaction, Earthquake, Pile Design Criteria, Buckling

INTRODUCTION

Collapse and/or severe damage to pile-supported structures are still observed in liquefiable soils after strong earthquakes, such as the 1995 Kobe earthquake, 1999 Koceli earthquake or the 2001 Bhuj earthquake, despite the fact that a large factor of safety is employed in their design. The collapse of these foundations are often accompanied by plastic hinges in the piles. Bhattacharya (2003) has shown that the overall factor of safety against plastic yielding of a typical concrete pile can range between 4 and 8. This is due to the multiplication of safety factors on load (1.5), material (1.5 for concrete), fully plastic strength factor (Z_P/Z_E = 1.67 for circular section) and practical factors such as minimum reinforcements or minimum number of bars. This implies that the actual moments or shear forces experienced by the pile are 4 to 8 times those predicted by their design methods. The current methods of pile design under earthquake loading, such as Eurocode 8, NEHRP (2000), Japanese Highway Code (JRA) or the Indian Code (IS 1893) is based on a bending mechanism where inertia and slope movement (lateral spreading of soil) induce bending moments in the pile. Bhattacharya et al (2005) has shown that bending mechanism due to lateral loads cannot always explain a pile failure. They have used the well-known case study of the Showa Bridge failure to illustrate that although the design of the piles in the bridge satisfies the latest guidelines of the JRA (2002) code, the bridge actually failed during the 1964 Niigata earthquake.

Background behind the current understanding of pile failure

Buildings and bridges on loose to medium dense sands are often built on piles because the sand is not strong enough to support the structure on its own. In an earthquake if these sands are saturated they lose strength and liquefy. This means that if the soil is on a slope it will flow downhill which is often termed as “Lateral spreading”. Lateral spreading is a term used to represent the permanent lateral ground displacement after an earthquake. Such displacements have been reported to be detrimental to pile foundations and are often regarded to be the root cause of many bridge failures, see for example Hamada (1992a, 1992b), Ishihara (1997), Tokimatsu et al. (1998), Goh and O’Rourke (1999), Abdoun

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and Dobry (2002), Finn and Fujita (2002), Berrill and Yasuda (2002), Finn (2005) and Miwa et al. (2006). Figure 1 explains the hypothesis of this failure mechanism which is based on bending failure. JRA (1996, 2002) is one of the advanced codes of practice and they have codified this mechanism. They advise the practicing engineers to design the pile considering passive earth pressure for non-liquefied crust and 30% of the total overburden pressure for liquefied zone. Inertia load will act on the pile head and will also induce bending moments in the pile. Also the code prescribes to verify these effects separately.

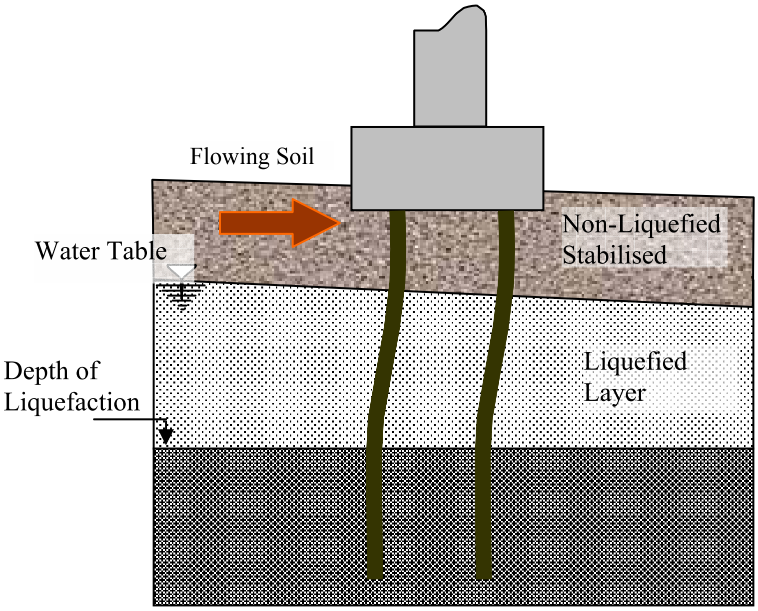


Figure 1: Current understanding of pile failure

Recent research and the buckling mechanism

However, recent research [Bhattacharya (2003), Bhattacharya and Bolton (2004), Bhattacharya et al. (2004), Lin et al. (2005), Knappett and Madabhushi (2005)] have suggested an alternative explanation. When the soil around the pile liquefies it loses much of its stiffness and strength, so the piles now act as unsupported long slender columns, and simply buckles under the action of the vertical superstructure loads. The stress in the pile section will initially be within the elastic range, and the buckling length will be the entire length in the liquefied soil. Lateral loading, due to slope movement, inertia or out-of-line straightness, will increase lateral deflections, which in turn can cause plastic hinge to form, reducing the buckling load, and promoting more rapid collapse. Therefore, this hypothesis is based on a buckling mechanism.

Bending and buckling instability

Beam bending and column buckling require different approaches in design. 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 at first yield, M_Y; or plastic moment capacity, M_P) of the member. 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 configuration of the member, i.e., slenderness ratio, and not on the yield strength of the material. 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 is a need for a minimum pile diameter depending on the thickness of the liquefiable soil.

The next section describes the various loading stages in the pile.

DIFFERENT STAGES OF LOADING IN THE PILE

During earthquakes, soil layers overlying the bedrock are subjected to seismic excitation consisting of numerous incident waves, namely shear (S) waves, dilatational or pressure (P) waves, and surface (Rayleigh and Love) waves which result in ground motion. As the seismic waves arrive in the soil surrounding the pile, the soil layers tends to deform. This seismically deforming soil tries to move the piles and the embedded pile-cap with it. Subsequently, depending upon the rigidity of the superstructure and the pile-cap, the superstructure may also move with the foundation. The pile may thus experience two distinct phases of initial soil-structure interaction.

(1) Before the superstructure starts oscillating, the piles may be forced to follow the soil motion, depending on the flexural rigidity (EI) of the pile. Here the soil and pile may take part in kinematic interplay and the motion of the pile may differ substantially from the free field motion. This may induce bending moments in the pile (Figure 2-Stage I).

(2) As the superstructure starts to oscillate, inertial forces are generated. These inertia forces are transferred as lateral forces and overturning moments to the pile through the pile-cap. The pile-cap transfers the moments as varying axial loads and bending moments in the piles. Thus the piles may experience additional axial and lateral loads, which cause additional bending moments in the pile (Figure 2-Stage II)

Above two effects occur with only a small time lag. If the section of the pile is inadequate, bending failure may occur in the pile. The behaviour of the pile at this stage may be approximately described as a beam on an elastic foundation, where the soil provides sufficient lateral restraint. The available confining pressure around the pile is not expected to decrease substantially in these initial phases. The response to changes in axial load in the pile would not be severe either, as shaft resistance continues to act. However, the pile should be strong enough to take the additional axial load induced by the inertia of the superstructure mass.

In loose saturated sandy soil, as the shaking continues, pore pressure builds up and the soil starts to liquefy. With the onset of liquefaction, an end-bearing pile passing through liquefiable soil will experience distinct changes in its stress state. The following two distinct states may be used to describe the sate of soil-structure interaction during the earthquake.

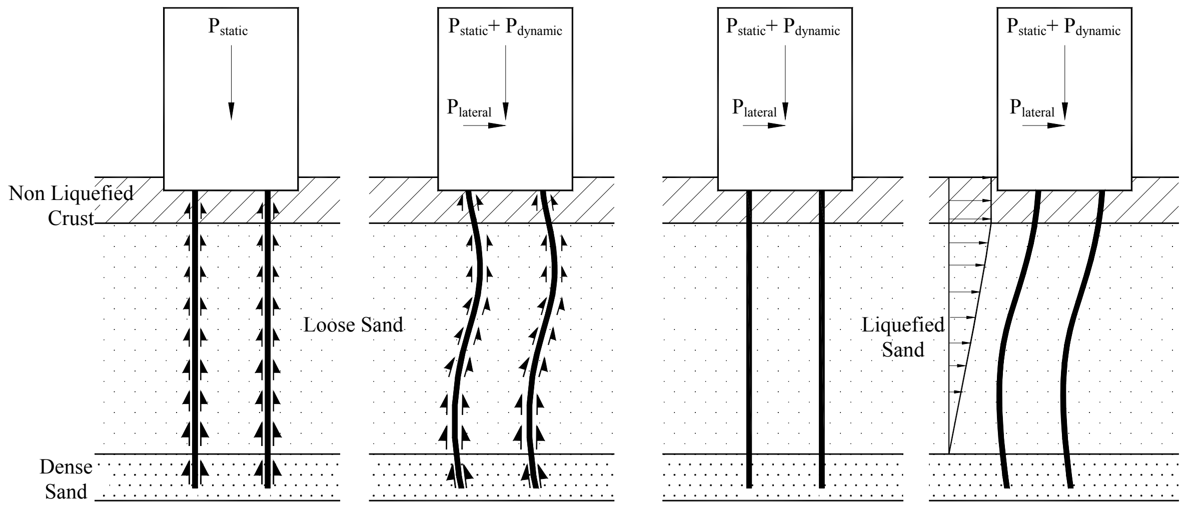
(1) The pile will start to lose its shaft resistance in the liquefied layer and shed axial loads downwards to mobilize additional base resistance. If the base capacity is exceeded, settlement failure will occur (Figure 2-Stage III). As the unsupported length of piles increases de to the liquefaction, the piles may fail due to buckling under high axial load.

(2) The liquefied soil will begin to lose its stiffness so that the pile acts as an unsupported column as shown in Figure 2-Stage IV. Piles that have a high slenderness ratio will then be prone to axial instability, and buckling failure will occur in the pile, enhanced by the actions of lateral disturbing forces and also by the deterioration of bending stiffness due to the onset of plastic yielding. Dynamic centrifuge tests, study of case histories and analytical work carried out by Bhattacharya (2003), Bhattacharya et al (2004) has conclusively shown the above failure mechanism. This particular mechanism is currently missing in all codes of practice.

In sloping ground, even if the pile survives the above load conditions (i.e. safely carry the axial load and the lateral inertial loads at fully-liquefied condition), it may experience additional drag load due to the lateral spreading of soil. Under these conditions, the pile may behave as a beam-column (column with lateral loads).

Dynamic considerations and the change in natural frequency of vibration

The above section describes the stages of loading in the pile as if the loads are acting statically on the foundation. However, there will be effects due to the dynamic nature of the non-linear earthquake loading. Normally, the frequency of the pile-supported structure is estimated based on the dimension of the superstructure without any consideration to the foundations which is based on internationally calibrated data; see for example Chopra and Goel (2000). This assumption can be justified for Stage II loading shown in Figure 2. However, during and after liquefaction, as the pile loses its lateral confinement, it becomes an integral part of the superstructure. The frequency of the structure may alter substantially and in most cases will reduce. Designers must ensure that the frequency of the structure at full liquefaction should not come close to the driving frequency of the earthquake to avoid resonance effects. This provision is currently not explicitly mentioned in the codes of practice.



Stage-I: Before earthquake in a level ground	**Stage-II:** Shaking starts. Soil yet to liquefy. Pile acts as a beam	**Stage-III:** Soil liquefied. Inertia forces may act. Pile acts as a column and may buckle or settle down	**Stage-IV:** In sloping ground lateral spreading may start

Figure 2: Loads and collapse mechanisms on a piled foundation.

PREDOMINANT LOADS ACTING ON PILED FOUNDATIONS DURING EARTHQUAKES

During earthquakes, the predominant loads acting on a pile are:
(1) *Axial load (P)* that acts at all times. The axial load may increase due to inertial effect of the superstructure and kinematic effects.
(2) *Inertial loads* due to the superstructure which is oscillating in nature.
(3) *Kinematic Loads* due to ground movement. This load can be of two types, such as transient (during shaking due to the dynamic effects) and residual (after the shaking ceased due to lateral spreading).

FUNDAMENTAL FAILURE MECHANISMS OF PILE FOUNDATION

The fundamental mechanisms that can cause formation of failure in a pile are:

(1) ***Shear failure*** in the pile due to the lateral loads such as inertia or kinematic loads or a combination of the above. This is particularly damaging to hollow circular concrete piles (non-ductile) with a low shear capacity.

(2) ***Bending failure*** due to the combined effect of lateral and axial loads i.e. formation of a collapse mechanism.

(3) ***Buckling failure*** in slender piles due to the effect of axial load and the loss of surrounding confining pressure provided by the soil owing to liquefaction, see Bhattacharya (2003), Bhattacharya and Bolton (2004), Bhattacharya et al (2004). Buckling is sensitive to imperfections such as lateral loads, out-of-line straightness. This would imply that in presence of lateral loads, a pile would buckle at a load much lower than the Euler's Critical Load.

The buckling instability load of a piled structure can be calculated by estimating the buckling load of one pile and multiplying by the number of piles forming the foundation. The buckling load of one pile can be estimated using the Euler's buckling load formulation.

The Euler's buckling load of the pile (P_{cr}) is calculated from the well known formula given by:

$$P_{cr} = \frac{\pi^2}{L_{eff}^2} EI$$, where

EI = Bending stiffness of the pile and
L_{eff} = Effective length of the pile and is defined in the next few paragraphs.

Figure 3 shows some cases of piled foundations likely to be encountered in practice. The fixity at the bottom of the liquefied soil can be classified into two cases:

(a) Embedment more than 5 times the diameter of the pile, such as Cases 1, 3, 4 and 6 in Figure 3. In such case, that the slope of the deflected pile is zero at the bottom of the liquefied layer.

(b) Less embedment and therefore the pile can rotate at the bottom of the liquefied soil. Examples are Cases 2 and 5 in Figure 3.

Table 1 lists various cases of piles in liquefiable soil with their critical buckling load with some examples. Apart from Table 1 and Figure 3, other boundary conditions of the pile at the top and bottom of the liquefiable soil may exist and similar expressions can be worked out.

(4) ***Dynamic failure****:* The above three failure mechanisms assume that the loads are pseudo-static in nature. However, during the earthquake, the dynamic soil-pile interaction becomes much complicated and the additional stresses may be induced in the pile due to the altered dynamic properties of the soil and the structure. It must be mentioned that the change in the dynamic properties of the soil and structure will be a function of the characteristics of the earthquake. Piles will experience additional dynamic forces due to shaking of the bedrock and surrounding medium and change in the dynamic property (stiffness and ductility) of the structure as a whole. The structure in fully liquefied soil behaves like inverted pendulum/open ground story structure with piles resembling the long ground columns, which is not considered an ideal design for seismic vibration. Hence this dynamic failure of piles should not be ignored in the design process; see for example Bhattacharya and Adhikari (2007).

The above four forms of failure is stated as "Limit State of Collapse". Each of these failure mechanisms can cause a completely collapse of the foundation. However, a real failure is perhaps a nonlinear combination of the above mechanisms.

It is worth noting that the pile will also lose its shaft resistance in the liquefiable region due to the loss of effective stress, and thus have to settle for vertical equilibrium. In order to be functional after the earthquake, the settlement of the piled foundation should be within the acceptable limits for the structure. This can be termed as "Serviceability Limit-State".

Table 1: Various cases of Effective length of piles in liquefiable soils

Case ID in Fig 3	**Boundary condition of the pile at the top and bottom of the liquefied layer.**		**Effective Length**	**Buckling load of each pile**	**Example**
	Top	**Bottom**			
Case 1	Fixed [θ = 0, δ = 0]	Fixed [Sufficient embedment at the dense layer] [θ = 0, δ = 0]	$L_{eff} = 0.5L_0$	$\frac{4\pi^2 EI}{L_0^2}$	Pile groups with raked piles. Large non-liquefied crust which will not slide
Case 2	Free to translate but restrained against rotation– sway frame [θ = 0, δ ≠ 0]	Pinned [Less embedment at the dense layer] [θ ≠ 0, δ = 0]	$L_{eff} = 2L_0$	$\frac{\pi^2 EI}{4L_0^2}$	NFCH building, Hamada (1992a)
Case 3	Free to translate but restrained against rotation– sway frame [θ = 0, δ ≠ 0]	Fixed [Sufficient embedment at the dense layer] [θ = 0, δ = 0]	$L_{eff} = L_0$	$\frac{\pi^2 EI}{L_0^2}$	Most cases fall under such category.
Case 4	Fixed in direction but free to rotate [θ ≠ 0, δ = 0]	Fixed [Sufficient embedment at the dense layer] [θ = 0, δ = 0]	$L_{eff} = 0.7L_0$	$\frac{2\pi^2 EI}{L_0^2}$	Pile groups with raked piles. Improper pile-pilecap connection
Case 5	Fixed in direction but free to rotate [θ ≠ 0, δ = 0]	Pinned [Less embedment at the dense layer] [θ ≠ 0, δ = 0]	$L_{eff} = L_0$	$\frac{\pi^2 EI}{L_0^2}$	Pile groups with raked piles. Improper pile-pilecap connection.
Case 6	Free i.e. unrestrained against rotation and displacement [θ ≠ 0, δ ≠ 0]	Fixed [Sufficient embedment at the dense layer] [θ = 0, δ = 0]	$L_{eff} = 2\ L_0$	$\frac{\pi^2 EI}{4L_0^2}$	Piles in a row such as the Showa Bridge piles

Case 1: In such cases, it is necessary to verify if the non-liquefied crust at the top of the liquefied soil can slide laterally and load the pile .If the non-liquefied crust can slide, the unsupported length will be L_0 + the thickness of the non-liquefied crust, and will be similar to Case 3.

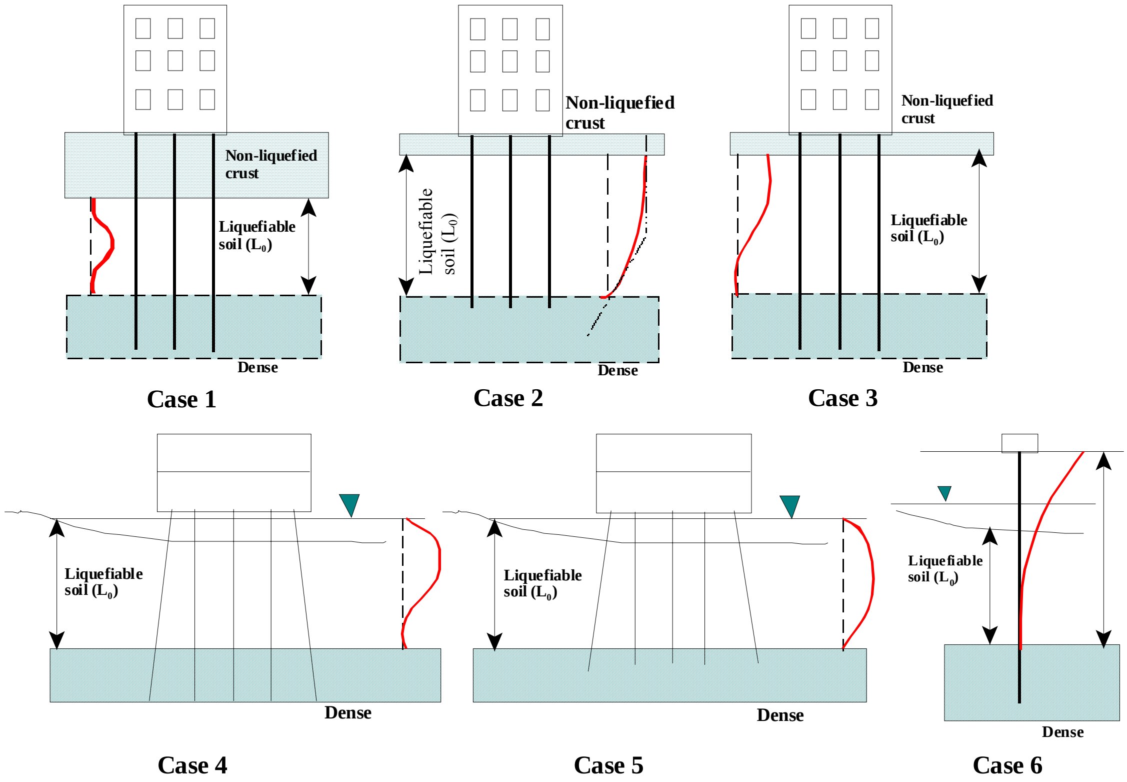


Figure 3: Effective length of piled foundation likely to be encountered in practice.

ESSENTIAL CHECKS THAT A SAFE DESIGN PROCEDURE SHOULD ENSURE

A safe design procedure should ensure that the piles have enough strength and stiffness to sustain 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. At any section of the pile, bending moment should not exceed allowable moment of the pile section. The shear stress load 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. Typical calculations carried out using the method proposed by Davisson and Robinson (1965) 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 (2003).

(3) A pile should have sufficient capacity to carry the axial load acting on it during full liquefaction without buckling. 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 pile should have sufficient capacity to carry the additional dynamic forces along with the static forces without exceeding yield. During earthquakes, the frequency of the pile-supported structure should not be close to the driving frequency of earthquake.

(5) 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.

PROPOSED FAILURE CRITERIA IN SIMPLIFIED DESIGN APPROACH

The proposed design criteria for piles are as follows:

(1) During the entire earthquake, the pile should be in stable equilibrium, the amplitude of vibration should be such that no section of the pile should have an ultimate limiting strain for the material. For example in the case of concrete piles, the ultimate strain in the pile should not exceed 0.003. At this strain, visible cracks appear in concrete leading to deterioration of bending stiffness. This criterion automatically ensures that no plastic hinge will form and no cracks will open up. Steel tubular piles are ductile i.e. they can withstand large amount of inelastic strain before yield and thus can be a good choice.

(2) The settlement of the piled foundation should be within acceptable limits for the structures. However, the settlement should be limited to a maximum of 10% of the pile diameter to avoid base failure (end-bearing failure) based on Fleming et al (1992).

CONCLUSION

For seismic pile design in liquefiable deposits, most codes of practice focus on bending strength and do not mention the bending stiffness required to avoid buckling instability in the event of soil liquefaction. The current design codes needs to address buckling of piles due to the loss of soil support owing to liquefaction and must consider the dynamic response of structure during earthquake. A pile must also be sufficiently embedded in the non-liquefiable hard layer below the liquefiable soil to ensure fixity and avoid sliding. The frequency of pile-supported structure at full liquefaction should not be close to the driving frequency of the earthquake. The settlement of the structure due to loss of shaft resistance of the pile in the liquefiable soils should be within acceptable limits.

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