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Proposed mechanism for mid-span failure of pile supported river bridges during seismic liquefaction

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ABSTRACT

Pile supported river bridge failures are still observed in liquefiable soils after most major earthquakes. One of the recurring observations is the mid span collapse of bridges (due to pier failure) with decks falling into the river while the piers close to the abutment and the abutment itself remain stable. This paper proposes a mechanism of the observed collapse. It has been shown previously through experiments and analytically that the natural period of bridge piers increases as soil liquefies. Due to the natural riverbed profile (i.e. increasingly higher water depth towards the centre of the river), the increase in natural period for the central piers is more as compared to the adjacent ones. Correspondingly, the displacement demand on the central pier also increases as soil progressively liquefies further promoting differential pier-cap displacements. If the pier-cap seating lengths for decks are inadequate, it may cause unseating of the decks leading to collapse. The collapse of Showa Bridge (1964 Niigata earthquake) is considered to demonstrate the mechanism. The study suggests that the bridge foundations need to be stiffened at the middle spans to reduce additional displacement demand.

1. Introduction

Collapse of pile-supported river bridges in liquefiable soils are still observed after most major earthquakes, see for example Fig. 1(a) and (b) taken from the past earthquake observations. It can be easily observed from the figures that the middle piers of the bridges collapsed without significantly affecting the other parts. The aim of this paper is to investigate the plausible failure mechanisms behind such collapse. Table 1 lists mid span failure for 10 bridges in the aftermath of seismic liquefaction where similar observations were noted. These examples indicate that seismic loads and its effects are inadequately considered and the current guidelines for pile design under such conditions needs revisiting. Therefore, the aims of the present work are as follows: (a) Review the codes of practice for pile design in liquefiable soils, (b) Postulate a plausible mechanism for such recurring mid-span failures, (c) Verify the mechanism through analytical calculations, (d) Validate calculations with the well-documented case study of Showa Bridge collapse (1964 Niigata earthquake, Japan).

2. Review of codes of practice and current understanding of pile design in liquefiable soil

Most codes of practice recommend that the pile be designed against bending failure due to inertial and kinematic loads (due to lateral spreading or wave propagation), see for example Japanese Highway Code of Practice [1], Eurocode 8 [2], National Earthquake Hazards Reduction Program Code [3] and Indian Code [4] etc. A comprehensive review of current theories of pile failures and hypothesis behind codes of practice may be found in [9]. Recent investigations carried out by Lombardi and Bhattacharya [5,6], Kimura and Tokimatsu [7] and Bhattacharya et al. [8] showed that when soil liquefies, part of the pile embedded in liquefiable zone becomes laterally unsupported and may buckle under the action of the axial load alone. Further, the dynamics of the soil-structure interaction in case of liquefied soil is explored in a recent study by Lombardi and Bhattacharya [5,6] where changes in the natural period and damping of the whole structure is examined during the liquefaction phase. Of particular interest in relation to the mid-span collapse of bridges is that the natural period of the piers of a bridge will increase many times with liquefaction. Bhattacharya et al. [10] showed that the natural period of pile supported piers of the Showa Bridge increased by more than 3 times due to liquefaction.

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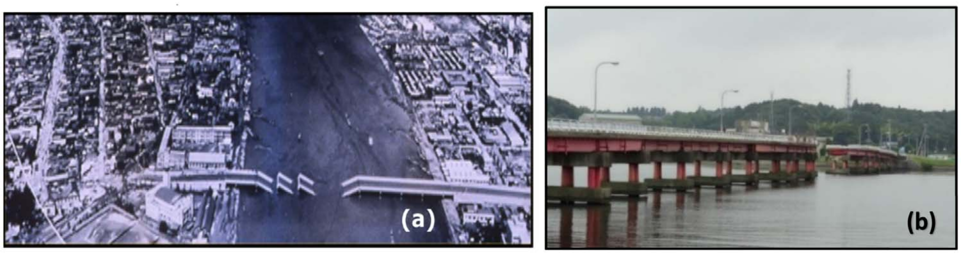

Fig. 1. Midspan Collapse of bridges (a)Showa bridge,1964 Niigata earthquake(b) Rokko bridge, 2011 Tohoku Earthquake.

3. Analysis of bridge pile in liquefiable soils

Fig. 2(a) shows a schematic diagram of a longitudinal section of a typical multi span bridge spanning across the river where piles of its abutments and piers passes through liquefiable deposits. Fig. 2(b) shows a section of the bridge to illustrate the difference between piles supporting a central pier and piers close to the abutments. Few points may be noted from the figures: (a) Due to the natural riverbed profile, water depth increases as we move from abutments towards the centre of the river channel. This would lead to relatively higher unsupported length of piles for central piers. (b) Due to continuous scouring and in the absence of scour protection work, water depth may increase at the centre of the river channel over time. Also, due to scour, the top soil in the central mudline usually consists of very loose soil deposit. During liquefaction, it may be reasonable to expect for a homogeneous ground that it will liquefy more or less equally to a given depth which is shown schematically in Fig. 2(a). This postulation is in broad agreement with the simplified method of obtaining the depth of liquefaction prescribed in codes of practices EC8 [2]. As the unsupported length of a pile is a function of water depth and depth of liquefaction, it may easily be derived from the above considerations that the piles supporting central piers will have higher unsupported length as soil liquefies. The effect of this higher unsupported length, is enhanced elongation of natural period for the central bridge piers as compared to the neighbouring piers. The impact of such elongation of natural period is differential demand of pier head displacement. Equivalent static analysis (single-mode method, AASHTO) may be used for carrying out such analysis. Fig. 3 shows mathematical idealisations of the problem, which is inspired from the Showa and Rokko Bridge configuration (Fig. 1(a) and (b)) and the assumptions used in the analysis are: (a) Each pier is considered separately and the interactions due to the adjacent piers are neglected; (b) The pier and the pile beneath are treated to be continuous, thus, same section property is used throughout as shown in Fig. 3; (c) Piles are flexible and therefore no rigid body rotation; (d) The effects of pile group are ignored; (e) The pile is axially stable i.e. enough base capacity to resist bearing failure; (f) Piles are laterally unsupported in liquefiable zone and hence, no p-y springs in the liquefiable part.

It may be appreciated that generalised analysis of real structural systems (in our case river bridge) is difficult and therefore bold simplifications are necessary. As the aim of this paper is to draw broad conclusions on the cause of collapse, a two-layered simplified ground profile (Fig. 3) is chosen. To evaluate the natural period of the bridge pier and to further predict the response, the parameters that are necessary are shown in Fig. 3 and defined below:

3.1. Unsupported length of the pile (L_O)

For a typical two layered soil system shown in Fig. 3(a) and (b), the unsupported length (L_0) can be estimated following the work of Bhattacharya and Goda [11] where a depth of fixity is further added to the length of unsupported portion of the pile. The unsupported length of the pile before and after the liquefaction is denoted as $L_{0\text{-}pre}$ and $L_{0\text{-}post}$ respectively.

3.1.1. Estimation of $L_{0\text{-}pre}$ and $L_{0\text{-}post}$

Before liquefaction only the portion of the piers, which is above the ground remains laterally unsupported. The depth of fixity is added to it to obtain $L_{0\text{-}pre}$ (see Fig. 3(a)). At full liquefaction, the depth of liquefiable soil strata and corresponding depth of fixity is considered in addition to estimate $L_{0\text{-}post}$ (see Fig. 3(b)).

3.2. Natural period (T)

Depending on the unsupported length and bending stiffness of piles, the natural period of the piers can be estimated. The natural period of the pier before and after liquefaction is denoted by T_{pre} and T_{post} respectively.

3.2.1. Estimation of T_{pre} and T_{post}

Following the work of Lombardi and Bhattacharya [6], the natural period is estimated based on idealization shown in Fig. 4, where the stiffness K_{lat2} and K_{lat1} are contributed by the upper liquefiable soil layer and the underlying non-liquefiable soil layer respectively and their damping is being represented by ζ_2 and ζ_1 respectively.

Eq. (1) is used to find out the natural period for the pier in pre-liquefaction stage, where $K_{e\text{-}pre}$ is the stiffness of the equivalent pile–pier system before liquefaction and M_e is the equivalent mass lumped at the top of the pier as shown in Fig. 4(a). The complete pile-pier system can be idealized as a fixed cantilever with an unsupported length $L_{0\text{-}pre}$ and its stiffness can be calculated by Eq. (2).

Table 1
Abridged List of Bridge failures due to seismic liquefaction.

Earthquake (Eq.)	Bridge	Remarks
Tohoku Eq. (2011)	Rokko Bridge	Middle span collapsed, see Fig. 1(b).
Wenchuan Eq. (2008)	Miaoziping Bridge	One of the approaching spans collapsed [15].
Wenchuan Eq. (2008)	Gaoyuan Bridge	Middle span fell off the piers due to liquefaction.
Costa Rica Eq. (1991)	Rio Viscaya Bridge	One internal supporting pier was missing.
Phillipines Eq. (1990)	Magsaysay Bridge	Piers settled and failed and the bridge fell into the river
Tangshan Eq. (1976)	Zhuacun Bridge	The girders of the middle spans collapsed.
Tangshan Eq. (1976)	Shahe Bridge	Bridge girder supports collapsed.
Phillipines Eq. (1976)	Quirino Bridge	Midspan collapsed for the truss bridge.
Haicheng Eq. (1975)	Panshan Bridge	One of the middle piers sank, causing collapse.
Niigata Eq. (1964)	Showa Bridge	Few middle spans collapsed, see Fig. 1(a).