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ON THE COLLAPSE OF BRIDGE FOUNDATIONS IN LIQUEFIABLE SOILS DURING EARTHQUAKES

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ABSTRACT

Collapses of bridges founded in liquefiable soils are still observed after most major earthquakes; see for example the bridge collapses following the 1964 Niigata (Japan) and Alaska earthquakes, 1975 Haicheng (China) earthquake, 1976 Tanshang (China) earthquake, 2008 Wenchuan (China) earthquake, 2010 Maule (Chile) earthquake. One of the observations is that the middle of the bridges collapses by falling of the deck without any noticeable damage to superstructure. It has long been argued that the cause of the bridge failure is due to liquefaction induced soil flow (also commonly known as lateral spreading of the ground) which pushed the pier causing large displacement of the pier which eventually dislodges the deck. This paper reviews the bridge failures observed in the past earthquakes from China, India, Japan and critically analyses the postulated hypothesis behind the failure. Parallels will be drawn with the recent findings on dynamic soil foundation structure interaction.

Piles are most common foundations for supporting small to medium span bridges and they are designed with required factors of safety against bending due to lateral loads (inertia and kinematic loads due to lateral spreading) and axial capacity (shaft resistance and end-bearing). Recent research identified a few weaknesses in the conventional design approach for pile foundations: (a) when soil liquefies it loses much of its stiffness and strength, so piles subsequently act as long slender columns, and can simply buckle (instability failure) under the combined action of axial load and inevitable imperfections (e.g. out-of-line straightness, lateral perturbation loads due to inertia and/or soil flow). In contrast, most codes recommend that piles be designed as laterally loaded beams; (b) Natural period of pile supported bridge structures may increase considerably (few times) owing to the loss of lateral support offered by the soil to the pile and the damping ratio of the structure may increase to values in excess of 20%. These changes in dynamic properties of the bridges can have important design consequences, the most important one being displacement demand on the pier.

The aim of the paper is to revisit the failure of bridges in light of the current understanding. It is concluded that the immediate need is not only to rewrite the design code to incorporate these effects, particularly buckling instability, but also to requalify and, if necessary, strengthen the existing important pile foundations in liquefiable soils. Research needs are also highlighted.

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INTRODUCTION

On the collapse of bridge foundations in liquefiable soils during earthquakes

Roads and bridges are vital parts of the infrastructure and therefore should remain in working condition even after any natural disaster such as a hurricane or an earthquake. This is to facilitate the relief operations. Most small to medium span bridges founded on seismically liquefiable deposits (loose to medium dense sands) are supported by pile foundations. Failure of these pile foundations has been observed in the aftermath of the majority of recent strong earthquakes such as the 1995 Kobe earthquake (JAPAN), the 1999 Kocheli earthquake (TURKEY) and the Bhuj earthquake (INDIA). It has widely been accepted that liquefaction-related effects are the cause of these failures.

In the context of bridge foundations, it is a common observation in liquefaction-related bridge failure that piers (intermediate supports) collapse, whilst the abutments (end supports) remain stable, see for example Figures 1,2 and 3. Figure 1 shows the collapse of the Showa bridge after the 1964 Niigata earthquake which will be discussed in details in this paper. Figures 2 and 3, on the other hand shows the collapse of the Million Dollar bridge after the 1964 Alaska earthquake and the collapse of Nakatsuno bridge following the 1948 Fukui earthquake. Similar observations were also observed in other earthquakes, see for example 1990 Luzon earthquake (Philippine), 2010 Chile earthquake.

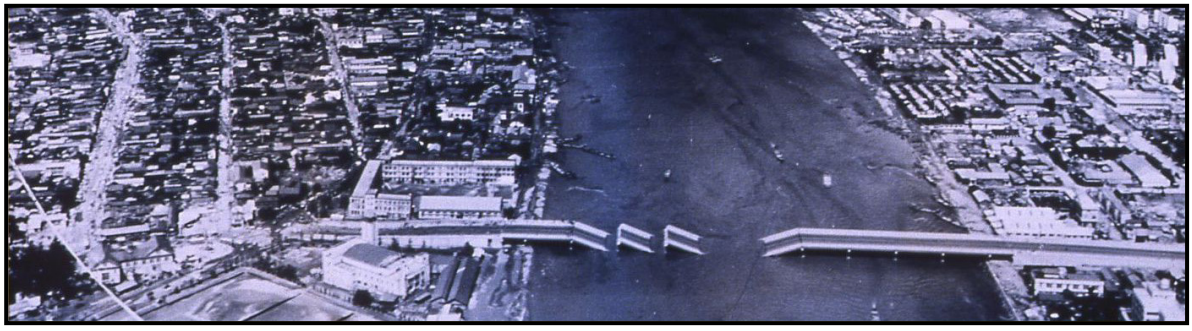

Figure 1: Showa Bridge collapse during 1964 Niigata Earthquake

Figure 2: Million Dollar Bridge collapse during 1964 Alaska Earthquake

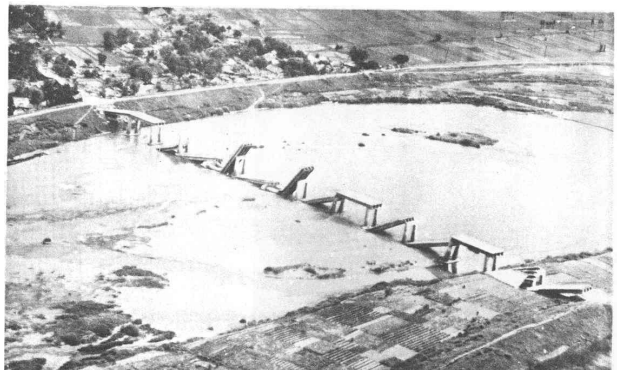

Figure 3: Nakatsuno Bridge collapse during 1948 Fukui earthquake (Japan)

Current understanding of pile failure

The most commonly adopted current hypothesis of pile failure is based on a bending mechanism. It is hypothesised that large inertial and kinematic lateral loads produce bending moments which exceed the capacity of the pile. The inertial lateral loads are the result of the earthquake induced inertial effects of the superstructure and the kinematic loads are due to flow of the soil following liquefaction and strength degradation. The later effect, is referred to as "*lateral spreading*".

The Japanese Code of Practice (JRA 2002) has incorporated this understanding of pile failure and as shown in Figure 4. The code advises practicing engineers to design piles against bending failure assuming that the non-liquefied crust offers passive earth pressure to the pile and the liquefied soil offers 30% of the total overburden pressure. Other codes such as the USA code (NEHRP 2000) and Eurocode 8, part 5 (1998) also focus on the bending strength of the pile. This simply treats piles as beam elements and assumes that inertial lateral loads and lateral spreading cause the bending failure of the pile. A good discussion can be found in Bhattacharya and Madabhushi (2008).

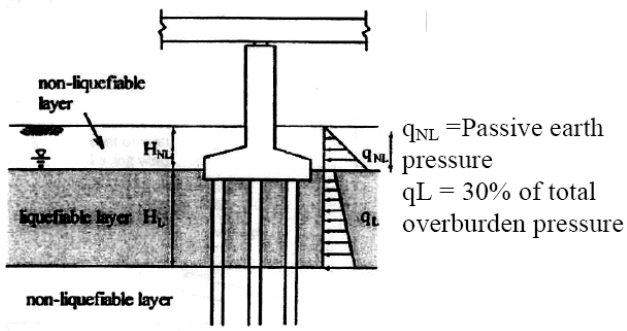

Figure 4: JRA (1996 or 2002) code of practice showing the idealization for seismic design of bridge foundation.

Therefore, two separate calculations are necessary considering two different loading conditions based on Japanese Highway Code of practice JRA (JRA, 2002): (a) kinematic loading exerted by the lateral pressure of the liquefied layer and any non-liquefied crust resting on the top of the liquefied deposit - see Figure 4; (b) inertial force due to the oscillation of the superstructure. The code recommends engineers to check against bending failure considering inertia and kinematic forces separately.

Inconsistency between the current understanding and the observed failure patterns

In bridge design, the number of piles required to support an abutment is governed by lateral load considerations since the abutment has to retain earth and fills of the approach roads. It is also noted that abutment piles carry the vertical load of the deck. On the other hand pile foundations supporting bridge piers are predominantly designed to support the vertical loads only. Figure 5 presents the schematic diagram of a typical two span bridge showing the abutment and pier foundations. From static equilibrium, for a multiple span bridge having similar span lengths, abutments support a vertical load equal to one-half of the vertical load supported by a pier.

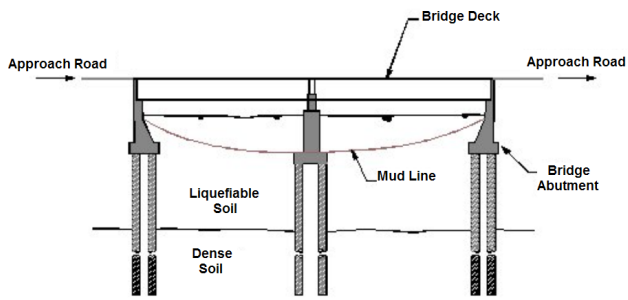

Figure 5: Schematic diagram of a typical two span bridge showing the abutment and pier foundations.

Typically, the lateral load carrying capacity of a pile is 10 to 20% of the axial load capacity. Therefore, for a typical multiple span bridge having similar span lengths, the number of piles supporting an abutment is larger than the number of piles supporting a pier. As it can be observed from Fig 1 and 2, collapse of pile-supported bridges in seismically liquefied soils is often characterised by tilting or failure of pier(s) and the subsequent collapse of the bridge deck(s). It is worthwhile to note that in these examples bridge piers collapsed whilst the abutments remained stable.

A CASE STUDY: COLLAPSE OF SHOWA BRIDGE

Showa Bridge, a simple steel girder bridge supported on piled foundations, were one of the many bridges that collapsed during the 14th June 1964 Niigata earthquake, see Figure 1. The total length of the bridge was about 307m. The bridge had 12 composite girders and its breadth was about 24m. The main span length was about 28m and side span length was about 15m (Fukuoka 1966), see Figure 6. The collapsed view from the southwest of the Showa Bridge is shown in Figure 1.

During the 1964 Niigata earthquake, the bridge site was subjected to extensive liquefaction and lateral spreading. Reliable eyewitnesses quoted by Horii (1968) and Hamada and O'Rourke (1992) along with the progressive damage simulation by Kazama et al. (2008) suggest that the bridge collapsed 1-2 minutes after the peak ground acceleration (PGA) had ceased. Yoshida et al. (2007) collated many eye witness statements and established the chronology of the bridge failure. Figure 6 shows a schematic diagram of the collapse of the bridge. The sequential failure initiated when piers P5 and P6 collapsed in opposite directions, accompanied by the fall of girder G5-6 (between P5 & P6) in the river. Immediately afterwards, in a domino effect, girders G6-7, G4-5, G3-4 and G2-3 partially fell in the river. Based on the eye witness reports, Kazama et al. (2008) also reported that the collapse of the bridge girders proceeded as G5-6→G6-7 →G4-5 →G3-4 →G2-3. As a result, five of the twelve spans fell off the pile heads in the earthquake.

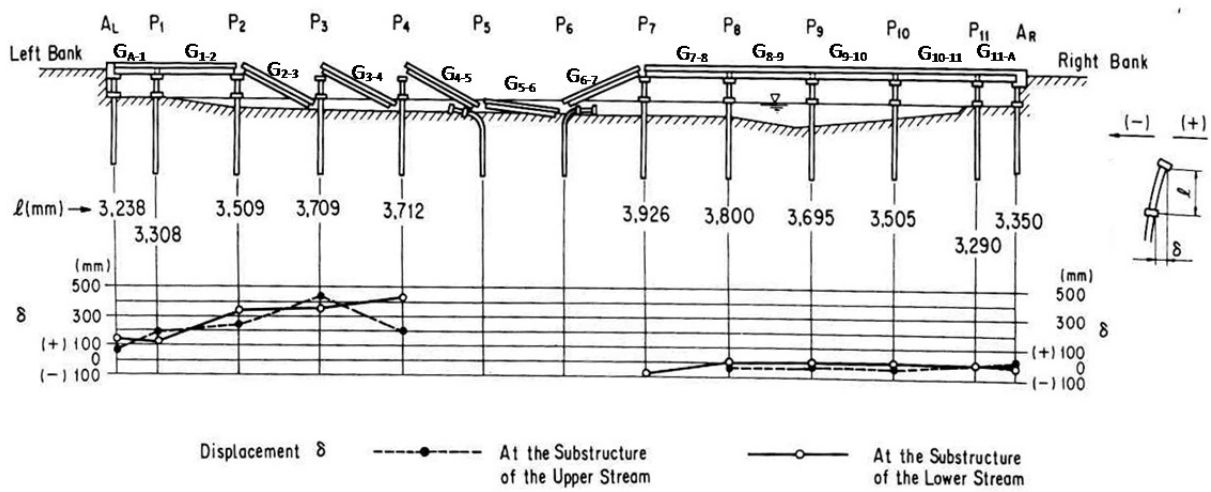

Figure 6: Schematic diagram of the collapse of the bridge along with the deflections of the pile caps (Iwasaki 1986)

Figure 7 shows the structural details and soil data for a pile of pier P4 after post earthquake recovery. On the other hand, Figure 8 shows the deck-pier support arrangement where there is alternating roller (movable) and pinned (fixed) except for pier P6 where both the supports are roller. Yoshimi (2003) commented on the lack of redundancy in the structural design of the bridge. It may be noted from Figure 6 that relative displacement of more than 30cm at the deck level will lead to unseating of the deck and hence may lead to collapse.

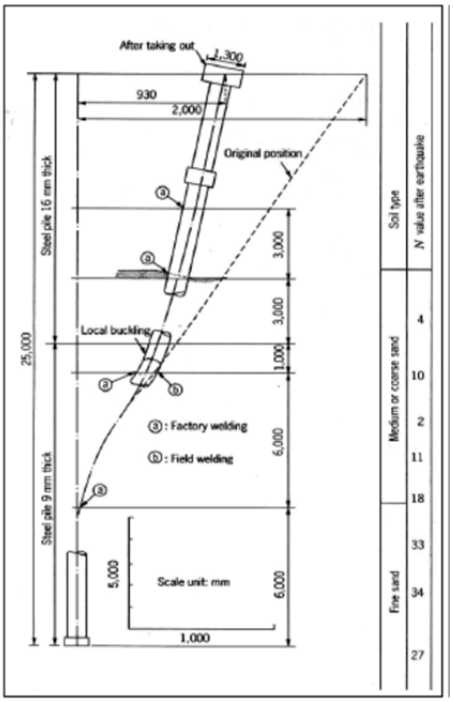

Figure 7: Structural and soil data for a pile of pier P4 (Fukuoka 1966)

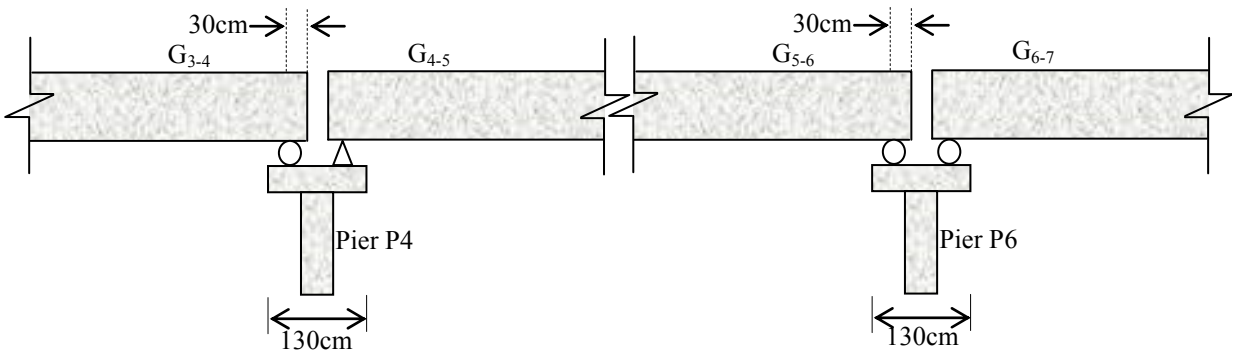

Figure 8: Support condition of the bridge at two piers

Figure 9 shows the time history of the acceleration, velocity and displacement recorded at the basement of a building (Kawagishi-Cho) at a location 1.25 km from the Showa bridge. It may be mentioned that records at Kawagishi-Cho are the only available strong motion records recovered near Niigata city and the site was fully liquefied. As a result, this ground motion is used to study the feature of earthquake at the time of collapse i.e. at about 70 seconds. Kudo et al. (2000) suggests that the long period ground motion was not produced by liquefaction but it radiated from the same source. They attributed the essential nature of the ground motion to the earthquake source, propagation path and deep sediments of regional scale. The plots on Figure 9 also show the window when the Showa Bridge collapsed. It may be observed that there is slight increase in acceleration i.e. a shock wave or a jolt during the time of collapse. It has been hypothesised (Kudo et al. 2000 and Yoshida et al. 2007) that this long period motion was presumably surface waves from the same earthquake source and travelled the same propagation path. From the ground motion, it is evident that the period of the ground is about 6-7seconds during the bridge collapse.

Inertial forces during the initial shock (within the first 7 seconds of the earthquake) or lateral spreading of the surrounding ground (which started at 83 seconds after the start of the earthquake) cannot explain the failure of Showa Bridge as the bridge failed at about 70seconds following the main shock and before the lateral spreading of the ground started. In this study, quantitative analysis is carried out for the various failure mechanisms that may have contributed to the failure. The study

shows that at about 70 seconds after the onset of the earthquake, the increased natural period of the bridge (due to the elongation of unsupported length of the pile owing to soil liquefaction) tuned with the period of the liquefied ground causing resonance between the bridge and the ground motion. This tuning effect (resonance) caused excessive deflection at the pile head adequate to unseat the bridge deck from the supporting pier and thereby initiating the collapse of the bridge.

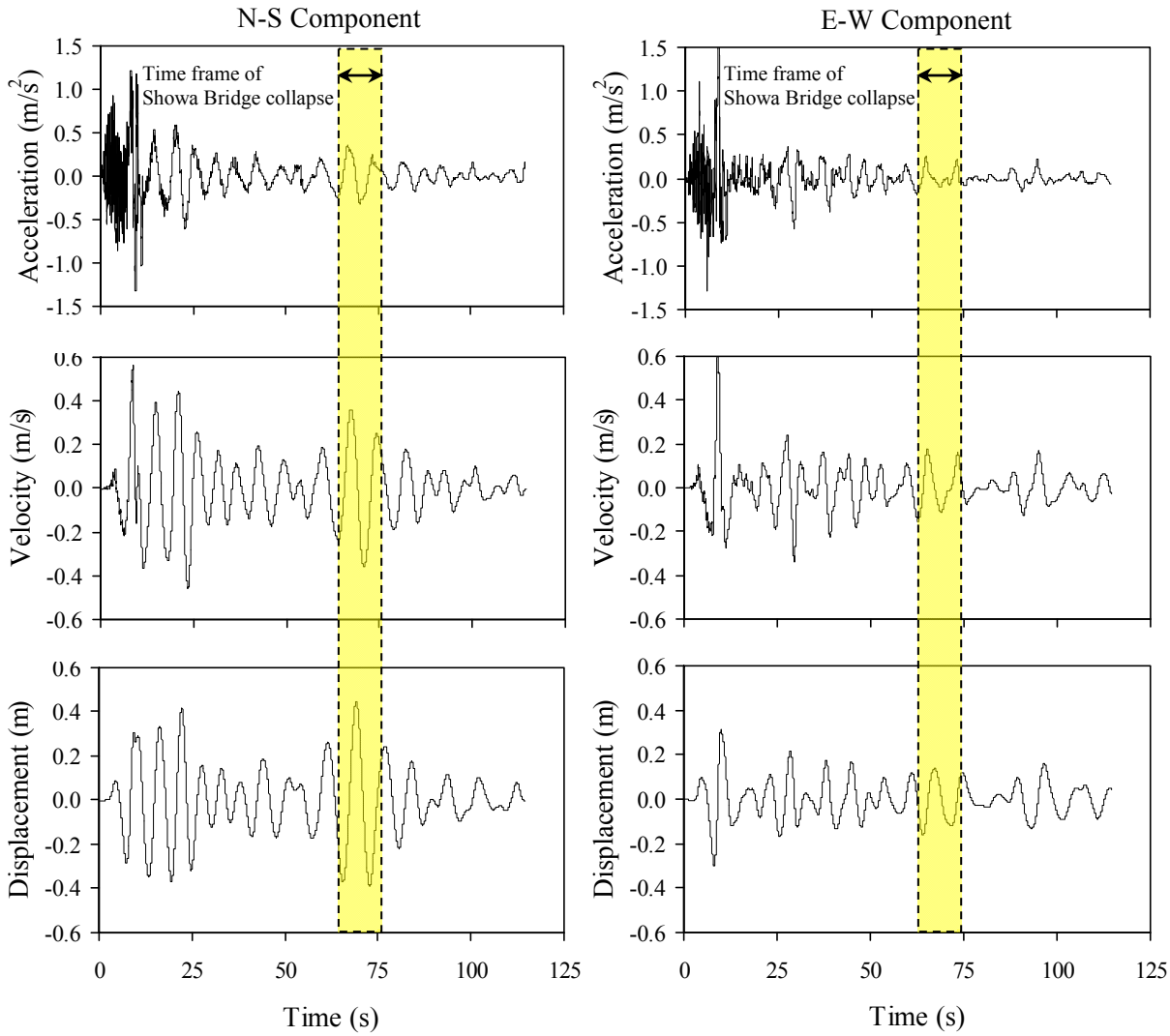

Figure 9: Recorded acceleration, velocity and displacement time histories, adapted from Kudo et al. (2000). Also in the diagram the time window (65s - 75s) when it is believed the Showa Bridge collapsed is indicated. (http://kyoshin.eri.u-tokyo.ac.jp/SMAD)

Estimation of period of the bridge at full liquefaction

The fundamental period of a bridge deck-pile-soil system will change with the liquefaction-induced-stiffness degradation of the soil surrounding the pile. The fundamental period, in most cases, will lengthen depending on the thickness of the liquefied soil layer. This has been shown through high quality experimental results carried out by Lombardi and Bhattacharya (2014) and analytical work by Bhattacharya et al (2008) and Adhikari and Bhattacharya (2008). To examine the effects of thickness of the liquefied soil layer on the period of pile foundations of the Showa Bridge, an idealised pile configuration has been adopted as shown in Figure 10(a). Further details can be found in Bhattacharya et al (2014).

The pile is assumed to be fixed at a depth of 4D below the liquefied soil layer (for further details see Bhattacharya et al. 2005). Weight from the deck on the pile is applied as mass, M on the free head of the pile. The fundamental period of the pile is then computed by considering the pile to be simple

cantilever for different length of the liquefied soil layer, L, and is shown in Figure 10(b). The simplified assumption is that liquefied soil offers no stiffness to small amplitude vibrations, the discussion of which can be found in Bhattacharya et al. (2009). It may be observed that the period increases with increasing thickness of the liquefied soil layer. For liquefied soil thickness of 10m as in case of pier P4, the fundamental period of the pile system increases from 2seconds before liquefaction to about 6seconds after liquefaction.

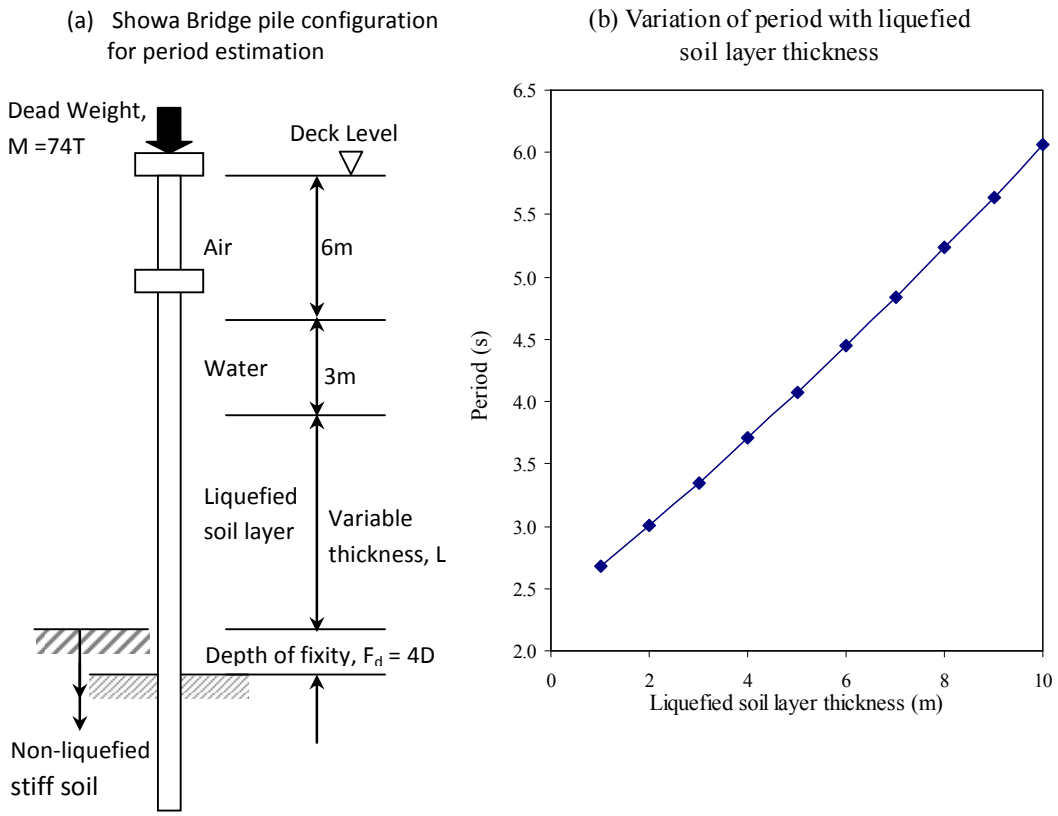

Figure 10: Period estimation for Showa Bridge pile: (a) pile configuration for period estimation (b) variation of period with liquefied soil layer thickness

Check for resonance

Based on the recorded motion, it can be estimated that the period of ground motion is 6 to7 sec at about 70 sec after the onset of the earthquake. Assuming 10m of resonant wavelength of liquefied layer, the equivalent shear wave velocity can be estimated to be about 6.7m/s (see equation 1):

$$\frac{v_s T}{4} = 10\ m \rightarrow v_s = \frac{40}{T} = \frac{40}{6} = 6.7 \quad m/\text{sec} \tag{1}$$

The acceleration, velocity and displacement spectra considering a Single Degree of Freedom (SDOF) system for the base motions as shown in Figure 11(a) are obtained with damping constant of 5% and 20%. The acceleration, velocity and displacement spectra are shown in Figure 11 and it may be observed that the spectral displacement reaches its peak at the period range of 6-7 seconds. An acceleration displacement response spectrum (ADRS) is plotted in Figure 12 for better comparison of the spectral behaviour.

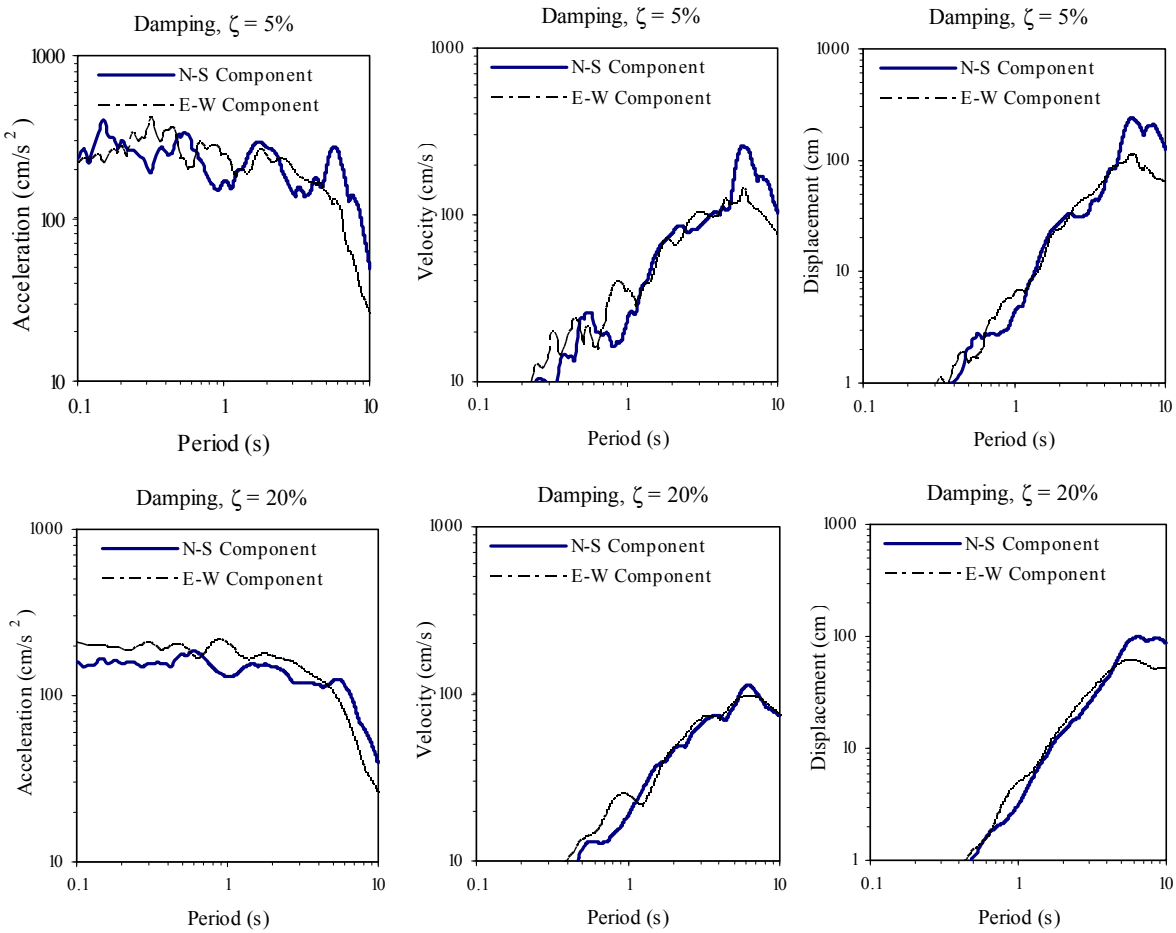

Figure 11: Acceleration, velocity and displacement spectra for a SDOF system for the time histories mentioned in Figure 7 for damping of 5% and 20%

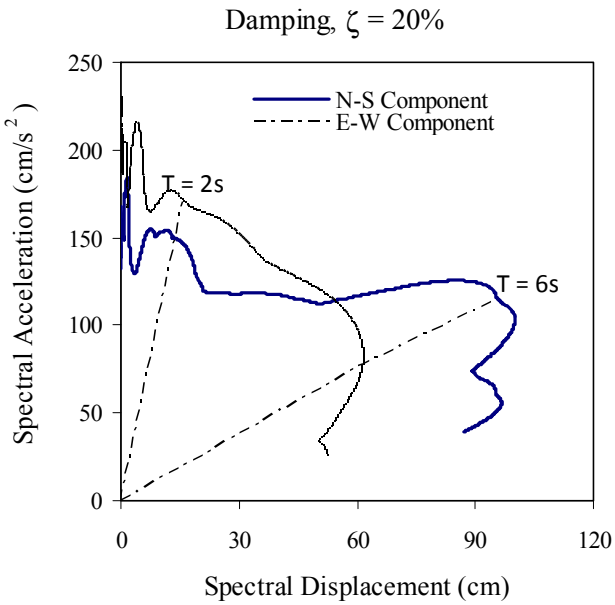

Figure 12: Acceleration-displacement response spectra for a SDOF system for the time histories mentioned in Figure 7 for damping of 20%

The role of in-depth analysis of case histories cannot be underestimated. Collapse of Showa Bridge provides a unique insight into various failure mechanisms that needs to be checked for any bridge. The next section provides some discussion on failure mechanisms for Showa Bridge foundations:

1. *Bending failure due to inertia in the first part of the strong motion*. This can be discarded as the bridge did not fail in the initial 10 seconds.
2. *Bending failure due to lateral spreading*: This failure mechanism can be discarded as lateral spreading started at about 83 seconds after the bridge collapsed. Also the piles close to the bridge abutments did not fail where the lateral spreading was known to be severe. On the contrary, the piles in the middle of the bridge failed where the lateral spreading is expected to be the least.
3. The proposed mechanism is the tuning of the bridge with the ground during the jolt causing large displacement at the pile which may have unseated the deck. It may also be mentioned that the depth of the liquefied layer is more towards the left half of the bridge as is evident from the liquefaction profile shown in Figure 6. Depending on the thickness of the liquefied soil layer, the flexibility of the soil-pile system is more towards the left half of the bridge and hence possibility of greater pile head deflection due to resonance. Therefore, depending on the thickness of the liquefied layer and resulting resonance (tuning with the earthquake), the deflections at the pile head is more on the left half of the bridge and is adequate to unseat the bridge deck. This could explain the reason why collapse was mainly observed on the left half of the bridge.

DISCUSSION AND CONCLUSIONS

It is a common observation in liquefiable soils that bridge pier fails while bridge abutments remained stable. In this study, quantitative back-analysis has been carried out to understand the failure mechanism of Showa Bridge. Following major conclusions may be summarised from the present study on Showa Bridge: (a) Due to liquefaction induced soil stiffness degradation, time period of the middle of the bridge (pile-soil-pier-deck system) increased from about 2seconds to about 6seconds. This resulting high period of the bridge falls in the displacement sensitive zone of the response spectra. Also the natural period of the liquefied soil falls in the range of 6-7seconds in the time window of 65s-75s leading to resonance between the ground motion and the bridge. This resonance coupled with the jolt at 70 seconds of the earthquake is thought to be a major contributor of failure of Showa Bridge. (b) Soil liquefaction profile as estimated by Hamada and O'Rourke (1992) shows more depth of liquefaction on left half of the bridge. Depending on the thickness of the liquefied soil layer and the corresponding period lengthening of the soil-pile system more tuning with the earthquake (i.e. resonance) and enhanced pile head deflection is expected on the left half side of the bridge. This may explain the observation that collapse occurred only on the left half of the bridge.

The current codes of practice or design guidelines do not consider all the above failure mechanisms. The main focus of the current codes are bending due to inertia and lateral spreading. It is therefore necessary to carry out seismic requalification studies of bridges in liquefaction-prone areas i.e. bridges designed and built using old codes of practice where all the above failure mechanisms are not considered.

According to most building codes, the seismic demand can be conveniently represented by the elastic response spectrum. The assessment of the spectrum ordinate requires an accurate estimation of both natural period and damping ratio of the structure. The majority of the seismic codes recommend to compute the vibration period based on empirical formulae, which only consider the dimensions, type and material of the superstructure, whereas the foundation is considered to be rigid. For example, to assess the fundamental period, T_1, of the structure, the Eurocode 8 (EN 1998-1:2004, 2004) recommends equation (2)

$$T_1 = C_t H^{3/4} \tag{2}$$

where T_1 is in seconds, H is the height of the building in m, the latter must be measured from the foundation or from the top of a rigid basement; C_t is equal to 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames, and 0.050 for all other structures. Equation 2 is quite similar to the well-known formula $T_1=0.1\times n$, where n is the number of stories of the building. It can be noted that by using empirical equations such as the one given by equation 2, the period of vibration is estimated based only the

characteristics of the superstructure but the foundation's flexibility is completely neglected. Although such an assumption may be conservative due to the beneficial effects of the SSI, i.e. de-amplification of spectral accelerations due to increase of damping and lengthening of the period, several studies demonstrated that the effects induced by Soil-Structure Interaction (SSI), especially in the presence of liquefaction, can be un-conservative and led to higher spectral acceleration and displacements.

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