

# FAILURE OF SHOWA BRIDGE DURING THE 1964 NIIGATA EARTHQUAKE: LATERAL SPREADING OR BUCKLING INSTABILITY?

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### **ABSTRACT :**

Following the 1964 Niigata earthquake many bridges, including the Showa Bridge, over the Shinano river collapsed. The newly-constructed Showa Bridge demonstrated one of the worst instances of damage, and there are still uncertainties and controversies regarding the causes of collapse. The collapse of the Showa Bridge has been, throughout the years, an iconic case study for demonstrating the devastating effects of the lateral spreading of liquefied soil. In this paper, this widely accepted collapse hypothesis has been challenged. The documented eyewitnesses' observations and post-collapse damage reports have been reanalysed, and the all the major studies on the collapse of the bridge compared and contrasted. It has been shown that the current, widely accepted, failure mechanism based on bending due to lateral spreading, cannot explain the failure. This paper presents a new hypothesis based on buckling failure due to axial loads in conjunction with residual, earthquake-induced, lateral displacements. This alternative explanation has been evaluated quantitatively using the method suggested by Kerciku et al. (2008) for estimating the buckling capacity of piles in liquefied soil, and Eurocode 3 (1993) recommendations for steel members subjected to bending and axial compression.

KEYWORDS: Pile Foundation, Liquefaction, Buckling, Showa bridge, Depth of fixity, Lateral Spreading

# **1. INTRODUCTION**

Roads and bridges are vital parts of the infrastructure and therefore critical elements should continue to function even after a natural disaster such as a hurricane or an earthquake. This is to facilitate the relief operations and speed the recovery process. Most small to medium span bridges founded on potentially 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 earthquakes of Niigata (1964, Japan), Kobe (1995, Japan), Kocaeli (1999, Turkey) and Bhuj (India).

The Niigata earthquake occurred on the 14<sup>th</sup> of June 1964 and registered 7.5 on the Richter scale. Located some 55km from the epicentre, the Showa Bridge which crosses the Shinano River was one of the worst instances of damage (Figure 1). The 1964 collapse of the Showa Bridge has been, throughout the years, an iconic case study for demonstrating the devastating effects of the lateral spreading of liquefied soil (Fukuoka, 1966; Hamada and O'Rourke, 1992, and Yasuda and Berrill, 2000, Kramer, 1996). In this paper the events and circumstances that led to the collapse of the bridge have been reanalysed. The authors have critically re-evaluated the effects of lateral spreading on the bridge pier-piles. Using tools derived by Kerciku et al. (2008), the depth fixity of the piles in liquefied soils has been derived, and its structural performance calculated. Finally, we have shown that these calculations provide an alternative explanation for the failure of the Showa Bridge.

# 2. THE COLLAPSE OF THE SHOWA BRIDGE.

The Showa bridge had a total length was 307m with main girder spans of 28m. Each span was composed of 12 composite girders, making the bridge about 24m wide, and was supported on nine 600mm diameter steel piles (in a single line) of wall thickness of 9 to 16mm.





Figure 1: Aerial photograph of the Showa Bridge after the 1964 Niigata Earthquake, from Towhata (1999).

The bridge girders rested on movable and fixed joints to allow for thermal expansion of bridge, as shown in Figure 2. The construction of the bridge was completed just one month before the earthquake. Figure 2 shows the structure of the Showa bridge – the dashed lines describing position of the girders and piles after the collapse. Judging from how the expansion joints were bent and the eyewitness reports, Horii (1968) suggests that girder E was the first to fall in the water. Pier-piles  $P_5$  and  $P_6$  collapsed, consequently, bending above the riverbed by about 90° in opposite directions, as shown in Figure 2. Judging from eyewitness's testimonies in Horii (1966), we can conclude that immediately after the collapse of girder E, in a "domino effect", girders F, D, C, and B slipped off their movable joints on pier-piles  $P_6$ ,  $P_5$ ,  $P_4$  and  $P_3$  respectively, and partially collapsed into the river. Hamada and O'Rourke (1992) concluded that the soil liquefied to a maximum depth of 10 m below the Showa Bridge riverbed, as shown in Figure 3.



Figure 2: Showa Bridge detail, showing movable and fixed joints. Adapted from Ishihara (1984)

Reliable eyewitness reports quoted by Horii (1968) and Hamada and O'Rourke (1992) suggest the superstructure collapsed between one and two minutes after the strongest ground motion had ceased. Evidence suggests that the phenomenon of delayed pile foundation failure during earthquakes is typically caused by liquefaction related effects (Berrill et al., 2001). This is because pore pressure increases during liquefaction may be transmitted from other, adjacent soil regions/layers, and this leads to soil liquefaction occurring at a finite time after the strong motion has ceased. Lateral spreading, which commonly accompanies liquefied soil on slopes or discontinuities, was observed at the Showa Bridge riverbanks. Based on these arguments, it has been widely accepted that the Showa Bridge failed due to bending of the pier-piles from the liquefaction induced lateral spreading of the riverbed soil (Fukuoka, 1966; Hamada and O'Rourke, 1992, and Yasuda and Berrill, 2000, Kramer, 1996).





Figure 3: Showa Bridge detail: Soil Liquefaction profile (in grey). From Hamada and O'Rourke (1992). The soil behind the left abutment is subjected to lateral spreading and creates a hinge near the abutment.

### 3. QUALITATIVE ANALYSIS OF THE FAILURE OF THE SHOWA BRIDGE

#### 3.1. Lateral Spreading of the Liquefied Soil

Fukuoka (1966) was the first to suggest that the Showa Bridge may have failed due to lateral spreading. Through a qualitative analysis, based on observations of soil movements at the riverbank, Fukuoka (1966) concluded that lateral spreading affected piles  $P_5$  and  $P_6$ . Furthermore local buckling observed at pile  $P_4$ , at a depth of 10m under the riverbed, suggested that it's bending of piles  $P_5$  and  $P_6$  was caused by the soil surface slide.

Hamada and O'Rourke (1992) thoroughly reanalysed the Niigata earthquake lateral spreading observations and concluded that lateral spreading of the liquefied soil caused the bending failure of the Showa Bridge piles. The permanent ground displacements in the area were estimated via aerial photography - reaching a maximum of 4 m near the Showa Bridge. Hamada and O'Rourke (1992) strongly supported the lateral spreading failure mechanism, discarding inertial effects on the basis that failure occurred after the main ground motion had ceased. Stemming from the conclusions of Fukuoka (1966) and Hamada and O'Rourke (1992) the Showa Bridge failure has been unquestionably related to lateral spreading by the wider research community (Yasuda and Berrill, 2000; Kramer, 1996 etc).

Whilst it is clear that lateral spreading occurred on the left riverbank, there is no evidence that the riverbed soil (directly underneath the bridge) was significantly affected by lateral spreading, as hypothesised by Hamada and O'Rourke (1992) and Fukuoka (1966). Hamada's conclusion regarding lateral spreading of the riverbed is based on the categorisation of lateral spreading ground-types (topographical circumstances leading to lateral spreading), defined in his 1986 work (Hamada et al., 1986; as shown in Figure 4). Following these ground categorisation, the riverbed overlain by the Showa Bridge would correspond to a type "C" ground surface, whilst the soil mass behind the left abutment would be a type "B" ground (free-face) as shown in Figure 5.



CASE C

Figure 4: Ground types for lateral spreading after Hamada et al (1986). Type A: slightly inclined "gently sloping", Type B: discontinuity "free face", Type C: inclined boundary.

However, theories stating that the Showa Bridge riverbed soil spread laterally by any significant degree may be disproved by a simple potential energy argument. When the shear stress required for static equilibrium is greater

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than the shear strength of the soil in its liquefied state, the liquefied soil mass will attempt to flow towards a lower potential energy form/location. Therefore, liquefied soil on a slope will flow to the bottom of the slope; and liquefied soil behind a discontinuity (such as a retaining wall) will tend to flow across the discontinuity. As shown in Figure 5 the liquefied soil of the riverbed (directly underneath the Showa Bridge piers) could not have flown laterally because is already in its lowest potential energy from. It is likely that some of the riverbank soil heaved near the left abutment due to pressure from the soil above the abutment, as shown in Figure 5, but it is reasonable to believe that the riverbed soil did not move significantly.

The soil of the left riverbank is, however, is clearly in a higher potential energy form and it is likely that it liquefied towards the centre of the riverbank. Hamada and O'Rourke (1992) conclude that the riverbank soil moved by about 4 m towards the centre of the river. The movement of this "thin" layer of soil coming from the abutment, if it reached piles  $P_5$  and  $P_6$ , is unlikely that it provided enough lateral pressure to affect the collapsed piles.



Figure 5: Showa Bridge Liquefaction profile divided into ground types defined by Hamada et al. (1986) cases (B) and (C). The soil profile has been adapted from Hamada (1992)

Further opposition of the argument that the failure of the piles was caused by lateral spreading of the riverbed is the fact that piers  $P_5$  and  $P_6$  collapsed in opposite directions. Had lateral spreading caused the bending failure of the piles, they would have collapsed in the same direction. Finally, Bhattacharya et al. (2005) conducted a JRA (1996) code check for bending of piles due to lateral spreading (assuming a soil depth of 10m spread laterally), and showed that the bending capacity of the piles was about 2 times larger than the induced lateral moments. In light of the aforementioned arguments it is be reasonable to conclude that the 1964 collapse of the Showa Bridge may not be attributed to lateral spreading of the liquefied soil.

#### 3.2 Inertial Earthquake Motion

Iwasaki (1986) suggests that failure occurred due to differential pile-head displacement resulting from excessive inertial earthquake loading. Iwasaki (1984) computed the inertial structural response of the bridge suggesting that due to liquefaction-induced soil stiffness degradation the pile head displacements exceeded the maximum allowed values, which lead to the collapse of the girders. This hypothesis clashes with the eyewitness evidence reported in Horii (1968) and Hamada and O'Rourke (1992) which suggest that the bridge failed 1-2 minutes after the earthquake peak ground acceleration (PGA) had ceased. Further, recent work by Haldar et al. (2008) suggested that inertial effects could not have lead to the bending failure of the piles. However, judging from the sides of the sole plate composing the movable shoes, Horii (1968) identified longitudinal marks which suggests that relative displacement did occur during strong ground shaking, but it was non-catastrophic. Therefore, it is reasonable to believe that the strong earthquake motion resulted in some permanent lateral deformation of the piles, but was not large enough to directly cause the failure of the bridge.

#### 3.2 Buckling Failure of the Piles

Bhattacharya et al. (2005) analyse the buckling stability of the Showa Bridge piles. However, Bhattacharya et al. (2005) assumptions on the buckling effective length of the pile may have led to un-conservative conclusions on the buckling capacity of the piles. Assumptions made on the "free" boundary condition at the pile head give an effective length equal to twice the actual length of the pile and may have lead to an underestimation of the



(4.1)

buckling capacity of the piles by a factor of four. The moment restriction at the pile head, due to the weight of the girders, would have more likely lead to a sway buckling mode (which gives a buckling effective length equal to the actual length). Furthermore assumptions on the depth of fixity of the pile (unsupported length) may also have lead to inaccuracies in calculating the buckling capacity. However, the increment in unsupported length due to liquefaction of the soil surrounding the pile decreases considerably the axial capacity of the pile. This phenomenon is also liquefaction related and would have occurred after the strong earthquake motion had ceased. The increase in unsupported lengths would affect the bending due to residual loads and initial lateral imperfections of the pile. These second order effects, also known as  $P-\Delta$  moments may prove to be catastrophic for slender, axially loaded structural elements such as piles.

### 4. QUANTITATIVE ANALYSIS

The qualitative analysis in the previous section suggests that lateral spreading cannot explain the failure of the Showa bridge piles. Further, earthquake inertial motion or buckling instability alone do not produce valid failure mechanisms. However, the combined effect of bending due to residual earthquake displacements (suggested by Iwasaki, 1984), and the decreased axial capacity of the piles due to increase of the unsupported length (suggested by Bhattacharya et al., 2005) may explain the failure of the Showa Bridge. In this section we have derived the effective length of the piles using the method suggested by Kerciku et al. (2008) for buckling of piles in liquefied soils, and computed a standard elastic utilisation check recommended by the Eurocode 3 - Part 1 (1993) for structural members bearing, axial and moment forces.

The residual inertial lateral displacements may have acted as additional imperfections of the piles resulting in large P- $\Delta$  moments. The slenderness of the piles increased progressively as the soil liquefied from top to bottom. Haldar et al. (2008) concluded that the soil of the riverbed under the bridge liquefied sequentially with the top layers liquefying in the first instances after the strong ground motion and the bottom layer at 10m depth liquefying more than 30 seconds after the strong ground motion. In the study below is shown that the P- $\Delta$  moments, combined with the increased slenderness of the piles proved to be catastrophic and lead to the overall bending moments exceeding the moment capacity of the central piles.

#### 4.1 Effective Length of the Piles

The unsupported pile length, L' has been derived using the recommendations in Kerciku et al. (2008). From post earthquake SPT tests of the Showa Bridge riverbed soil (Iwasaki, 1986), it is possible to estimate that the pre-earthquake, *N-value*, for the top 15m strata, was an average N=9.3. Following the JRA (2002) recommendations the relationship between the soil's stiffness and the STP *N-value* is:

$$E_{\rm s}$$
=2800N

According to this correlation the pre-earthquake, static soil's Yong's modulus is  $E_S=26$  MPa. Assuming a soil stiffness degradation factor of  $\varphi=0.1\%$  the liquefied soil stiffness would be:

$$E_{\rm s}' = \varphi \cdot E_{\rm s} = 26 \, kPa. \tag{4.2}$$

The depth of liquefaction, h, estimated from Figure 3, for pile P<sub>5</sub>, is approximately 9 *m*. From Kerciku et al. (2008) we can then derive the depth of fixity of the pile:

$$R = \sqrt[4]{\frac{1.35EI}{E_s'}} = \sqrt[4]{\frac{1.35 \times 160 \times 10^6}{26 \times 10^3}} = 9.547$$
(4.3)

$$P_R = \frac{h}{R} = \frac{9}{9.547} = 0.943 \tag{4.4}$$

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The value of  $S'_R$  can be derived from Figure 6. From Figure 6,  $S'_R = 1.1$ . Therefore, the depth of fixity  $L'_s$  is:

$$L'_{s} = S'_{R}R = 1.1 \times 9.547 = 10.5m \tag{4.5}$$

And, the total equivalent unsupported length (depth of fixity plus the exposed pile length) for pile  $P_5$  is:



Figure 6: Extracted from Kerciku et al. (2008) for  $0 < S_R < 1.6$  and  $0 < P_R < 1.4$ , from curve  $\delta = 0.5$ 

#### 4.2 Forces due to Residual Lateral Deformations

The earthquake caused a permanent lateral deformation of the piles,  $\Delta$ . These permanent deformations were documented by Iwasaki (1986), as shown in Figure 7. The quasi-static lateral load, *F*, responsible for this displacement, may be derived by solving the moment-curvature differential equations, as suggested by Kerciku et al. (2007) (for no lateral restraint, *k*=0), and may be derived from Eqn. 4.7.



$$F = \frac{\Delta P}{L'} \frac{\frac{\pi}{2} \sqrt{\frac{P}{P_E}}}{\tan\left(\frac{\pi}{2} \sqrt{\frac{P}{P_E}}\right) - \frac{\pi}{2} \sqrt{\frac{P}{P_E}}}$$
(4.7)

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The corresponding lateral forces F, derived from these lateral displacements using the aforementioned equation have been expressed into the following Eurocode 3 (1993) non-dimensional values and have been plotted on the graph in Figure 8 where:

 $\frac{F_{Ed}L'}{2W_{eff}f_y/\gamma_{MO}}$  which is the Eurocode 3 (1993) bending utilization – moment applied divided by moment

capacity.  $F_{Ed}$  is the quasi-static lateral force calculated from Eqn 4.7,  $W_{eff}$  is the elastic modulus as defined in Eurocode 3, and  $f_y$  is the yield stress of steel.

The axial load derived from the weight of the girders and structure, as suggested by Bhattacharya et al. (2005) has been presented in the following non-dimensional value:

 $\frac{N_{Ed}}{N_{b,Rd}}$  which is the Eurocode 3 (1993) axial utilization – axial load applied,  $N_{Ed}$  divided by the buckling

capacity of the pile.  $N_{b.Rd}$  is the buckling capacity of a fixed-free steel pile according, for the effective lengths as derived by the method above (Eqns 4.1 – 4.6), according to Eurocode 3 (1993).



Value	Numerical Values			Reference
	P <sub>1</sub>	P <sub>3</sub>	P <sub>5</sub>	
				Iwasaki
$\Delta$ (m)	0.1	0.3	0.5	(1986)
				Bhattacharya
N <sub>Ed</sub> (kN)	750	750	750	et al. (2005)
				Hamada and
				O'Rourke
EI (MNm <sup>2</sup> )	160	160	160	(1992)
				Kerciku et al
L' (m)	20	20	19.5	(2008)
F <sub>Ed</sub> (kN)	19	39	55	Equation1
W <sub>eff</sub> f <sub>y</sub>				Bhattacharya
(kNm)	629	629	629	et al. (2005)
$N_{b,Rd}$ (kN)	2623	2623	2623	Eurocode 3
F <sub>Ed</sub> L/				
$2W_{eff}f_{y/\gamma_{MO}}$	0.30	0.62	0.80	
NEd/ Nb,Rd	0.28	0.28	0.28	

Figure 8: Design performance for pile  $P_1$ ,  $P_3$  and  $P_5$  of the Showa Bridge after the earthquake, assuming a liquefied effective length of 19.5m. Chart adapted form Eurocode 3 – Part 1 (1995) section 6.2.9.9.

The section geometry and material properties used in this calculation are given in Bhattacharya et al. (2005). The axial and bending utilisation of piles  $P_1 P_3$ , and  $P_5$  have been calculated and data-points are plotted in Figure 8. It can be observed that pile  $P_1$  is a safe design, whilst pile  $P_3$  and  $P_5$  are outside the failure envelope and therefore prone to failure. Since this is an elastic analysis and the plastic moment capacity generally is 20% higher than the elastic one, pile  $P_3$  can also be considered as a safe design.

#### 5. CONCLUSIONS AND DISCUSSION

This paper shows that the riverbed soil directly underneath the Showa bridge could have not spread laterally. The riverbed soil was at its lowest potential energy and it remained static. The riverbank soil did laterally spread, but it probably heaved near the abutment (See Figure 5). The movement of the riverbank soil could not have caused any significant loads on the central pile rows, due to the negligible thickness of the spreading soil.



The failure of the piles was caused by to the combined effect of the axial loads, and second order P- $\Delta$  moments (caused by earthquake-residual-lateral displacements) on the piles. As shown in Figure 8, pile P<sub>5</sub> had been displaced by a magnitude large enough to produce second order P- $\Delta$  moments responsible for 80% of the stress utilisation. This, combined with the reduced axial capacity of the pile due to the increased slenderness, produced stresses in the pile which surpassed the steel's elastic (and plastic) stress limit. The same analysis also shows how piles P<sub>1</sub> and P<sub>3</sub> did not fail, because the residual lateral imperfections were much lower and the second order P- $\Delta$  effects were non-catastrophic.

In the context of a new failure hypothesis it is reasonable to believe that the liquefaction front travelling from top to bottom reached a critical depth a few moments after the strong ground motion (Haldar et al. 2008). At this critical depth, the combination of superstructure loading, earthquake imposed lateral imperfections, and reduced lateral support from the liquefied soil, resulted in collapse due to the structural instability of the central piles  $P_5$  and  $P_6$  and the subsequent collapse of the Showa Bridge.

# **5. REFFERENCES**

Bartlett, S. F. and Youd, T. L., 1992, "*Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction Induced Lateral Spreads*," Tech. Rept. NCEER 92-0021, National Center for Earthquake Engineering Research, Buffalo, NY, USA.

Bhattacharya S., Bolton M.D. and Madabhushi S.P.G. (2005): "A reconsideration of the safety of the existing piled bridge foundations in liquefiable soils", Soils and Foundations (Japan) Vol. 45, No 4.

Berrill J. B., Christensen S. A., Kennan R. P., Okada W. and Pettinga J. R. (2001). "*Case study of lateral spreading forces on a piled foundation*". Geotechnique, Vol. 51, No. 6, p501-517.

Eurocode 3, Part 1-1, BS EN (1993), *Design of steel structures – General rules and rules for buildings*. European Committee for standardization, Brussels.

Fukuoka M., (1966). *Damage to civil engineering structures*. Soils and Foundations, Vol. VI, No. 2, p45-52. Haldar S., Babu S., Bhattacharya S., (2008). "*Buckling and bending response of slender piles in liquefiable soils during earthquakes*" Geomechanics and Geoengineering An International Journal, Vol, 3, Issue 2, p 129-143. Hamada M., Yasuda S., Isoyama R. and Emoto K. (1986). *Study on liquefaction-induced ground displacements*," Report by the Research Committee, Association for the Development of Earthquake Prediction, Tokyo, Japan, Hamada M., and O'Rourke T. D. (1992). *Case studies of liquefaction and lifeline performance during past earthquakes – Volume 1: Japanese case studies*. Technical Report NCEER-92-0001, National Centre for

Earthquakes – Volume 1: Japanese case studies. Technical Report NCEER-Earthquake Engineering Research, Buffalo, NY, USA.

Horii K. (1968). *General report on the Niigata earthquake*, Tokyo Electrical Engineering College Press. Part 3: Highway Bridges, p431-450.

Iwasaki T. (1984). A case history of bridge performance during earthquakes in Japan. Int'l Conf. Case Histories in Geotechnical Engineering, University of Missouri-Rolla, Vol. 3, p981-1008.

Iwasaki T. (1986). Soil liquefaction studies in Japan, State-of-the-art. Technical Memorandum No. 2239, Public Works Research Institute, Tsukuba, Japan.

Japan Road Association (2002). Specification for Highway and Bridges, Part IV, JRA, Tokyo Japan Japanese Road Association (1996), Specification for Highway Bridges, Part V, Seismic Design. Tokyo, Japan Kerciku A.A., Battacharya S., Burd H.J., Lubkowsky, Z.A. (2008). "Fixity of pile foundations in seismically liquefied soils for buckling calculations – an eigenvalue analysis." Proceedings of the 14<sup>th</sup> World Conference in Earthquake Engineering. October 2008, Beijing, China

Kerciku A. A., Bhattacharya S., Burd H. J. (2007) "Why do pile supported bridge foundations and not abutments collapse in liquefiable soils during earthquakes?" Proceedings of the 2<sup>nd</sup> Japan Greece Workshop on seismic design, observation and retrofit of foundations. Japanese Society of civil engineers, p440-452.

Krammer, S. L. (1996) Geotechnical Earthquake Engineering, Prentice Hall New Jersey

Towhata I. (1999). *Photographs and Motion Picture of the Niigata City Immediately after the 1964 earthquake (CD)*. The Japanese Geotechnical Society

Yasuda S. and Berrill, J.B. (2000) Observations of the earthquake response of foundations in soil profiles containing saturated sands, Proc Geo Eng2000 Conf, Melbourne.