Earthquake Geotechnical Engineering for Protection and Development of Environment and Constructions – Silvestri & Moraci (Eds)

ISBN 978-0-367-14328-2

A comparative study on performance of bridges with rocking pile foundations in different soils

O. El-Hawat & B. Fatahi
School of Civil and Environmental Engineering, University of Technology Sydney, Sydney, Australia

C. Edmonds
EIC Activities (A Member of The CIMIC Group), Sydney, Australia

ABSTRACT: Rocking pile foundations is a new design approach used in bridges that enables both the structure and foundation to remain elastic during intense earthquakes. However, this system offers less energy dissipation than conventional rocking foundation designs, and hence the dynamic response is expected to be larger. Moreover, the soil medium could increase the dynamic response due to rocking by prolonging the period of the bridge and amplifying the earthquake signal. This paper seeks to numerically investigate the dynamic response of bridges with rocking pile foundations subjected to four different soil types. 3D numerical models of the bridge structure and soil foundation are developed and subjected to two earthquake records. The results show that rocking pile foundations perform best under stiff soils. However, in softer soils, the structure experiences much larger structural actions that could lead to bearing failure, pier and pile damage and even unseating of the bridge decks resulting in collapse.

1 INTRODUCTION

Under large earthquakes, bridges are conventionally designed to develop plastic hinges in their piers as a conservative design approach which offers energy dissipation at the cost of damage (Mander 1983). This design philosophy relies on the concept of ductility allowing the pier to withstand large drift ratios while still carrying the gravity loads from the deck. Despite this design being effective in preventing collapse under intense earthquakes, the damage caused to the piers often leads to lengthy and costly repairs, impaired traffic flows, and in some cases demolition of the entire bridge (Priestley et al. 1996). For this reason, engineers are searching for more cost effective and sustainable solutions.

After the 1960 Chile earthquake, Housner (1963) studied the motion of rigid bodies rocking on rigid base subjected to ground excitations and concluded that rocking can be used as an effective earthquake isolating system that reduces the seismic actions on the structure. Since then, the rocking motion of structures has been studied using extensive conceptual models to prove that they are effective as a seismic isolation technique. The stability of rocking structures was found to be controlled by both the frequency of the excitation and the size of the rocking structure. Generally, a signal with a higher frequency will have a smaller potential to cause overturning (Yim et al. 1980, Zhang & Makris 2001).

Antonellis & Panagiotou (2013) investigated the effects of rocking shallow foundations over piles in order to significantly reduce the inelastic behavior of the soil. The structure is shown to behave elastically under intense earthquakes. However, due to the absence of energy dissipation seen in the formation of plastic hinges, the structure experiences higher inertial actions and deck displacements that could lead to other damages and failures in the bridge (Hao & Daube 2012).

Moreover, the rocking behavior of the bridge is affected by the soil medium which provides stiffness to the piles, affecting the period and sway of the foundation (Van Nguyen et al. 2017). Bridges constructed in softer soil will have a larger period. In addition, the softer soil amplifies the earthquake signal changing its amplitude and frequency which ultimately affects the rocking motion of the structure (Choi & Stewart 2005). Therefore, it is of importance to investigate the effect of soil stiffness on the performance of bridges adopting rocking piles. This paper investigates the effects of soil types on the performance of rocking pile foundations. Performances of four identical bridge structures built on four different site Classes including Class A, B, C and D soil categories as defined in Eurocode-8 (European Committee for Standardization 2005) are assessed. Three-dimensional numerical models are developed using SAP2000 finite element software, and nonlinear time history analysis are conducted using two near field earthquakes. Both horizontal components of the earthquakes are adopted and scaled to a particular site hazard level. The bridges are then compared in terms of their dynamic response including the deck displacements and structural actions and bending moments in the piers and piles.

2 SITE CHARACTERISTICS AND SEISMIC HAZARD

In this study, four identical bridges are investigated, each situated in different site classes. The selected four soil categories are as defined in Eurocode-8 (European Committee for Standardization 2005), as site Classes A, B, C and D. In this study only rock and clay types of ground are considered in all four categories with their properties displayed in Table 1.

The bridges are constructed on a 25 m deep deposit (Class A, B, C or D) underlined by a rock layer (Class A). The European seismic hazard map was used to determine the PGA for two hazard levels including: 10% probability in 50 years (PGA=0.4) for the Design Earthquake (DE) and 2% probability in 50 years (PGA=0.6) for the Maximum Considered Earthquake (MCE). The bridges are designed using the DE scaled earthquakes, whereas the response of the bridge subjected to MCE is discussed in the paper. The 1995 Kobe and 1994 Northridge earthquakes were selected and scaled with a damping ratio of 5% adopted to obtain the response spectrum (Xu & Fatahi 2019). Both horizontal components were considered for each earthquake, whereas the vertical component was ignored. It should be noted that the earthquakes were scaled between the periods ranging from T=0.05s to T=3s.

3 BRIDGE DESCRIPTION

All four bridges are created with geometry and properties similar to existing bridges and designed to perform elastically under the DE level shaking and fulfil the two design objectives: (1) prevent plastic behavior in piers, and (2) prevent any inelastic soil behavior that leads to excessive settlements and residual rotations.

Table 1. Soil properties of the four type of soils used for each bridge.

Site Class	Soil Type	$V_{s,30}$ (m/s)	S_u (kPa)	ε_{50}	γ (kN/m^3)	Unconfined Compressive Strength (MPa)
A	Rock	1000	-	-	25.5	10
B	Stiff Clay	700	300	0.004	19.8	-
C	Medium -Stiff Clay	300	150	0.005	19.2	-
D	Soft Clay	150	65	0.006	18.6	-

Note: $V_{s,30}$ is the average shear wave velocity in the top 30m of the ground, S_u is the undrained shear strength of the soil, ε_{50} is the strain corresponding to one-half of the maximum principal stress difference, and γ is the unit weight of the ground.

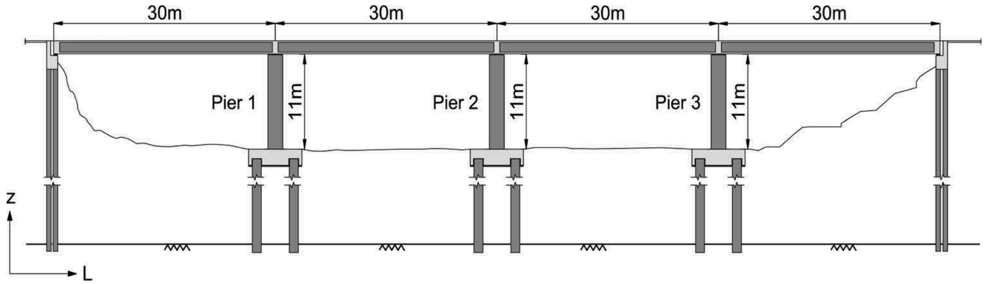

Figure 1. Longitudinal section of the bridge adopted in this study.

The concrete box girder bridge shown in Figure 1 has four spans, each span being 30m long, with a total length of 120m. The concrete box girder has a total width of 14 m, depth of 1.8 m, and slab thickness of 0.25 m. The concrete in the bridge has a compressive strength of 50 MPa and a unit weight of 25 kN/m^3. Additionally, the reinforcing steel used in the bridge has a yield strength of 500 MPa and a unit weight of 77 kN/m^3.

The bridge is supported by three single column bents that are equal in height at 11m tall. Each column has a diameter of 2.2 m and a longitudinal reinforcement ratio of 2.4%. The piers form pin connections with the deck and monolithically fixed to the pile cap foundation.

The concrete abutment is connected to six piles with a diameter of 0.8 m and a length of 35 m. The concrete deck is supported by the abutment through four elastomeric bearings with a circular diameter of 0.4 m and thickness of 0.2 m. The concrete deck is restrained in the transverse direction by concrete shear keys. The expansion joint between the deck and the abutment back wall is 0.3 m which accommodates the seismic displacements as well as the creep and shrinkage movements of the deck.

The rocking pile foundation consists of the pier and pile cap rocking about the pile heads. The piles adopted in site Class A are only 1 m deep as the foundation material is sufficient to carry the vertical loads, however piles are still used to simulate the same rocking behavior for the sake of comparison. For site Classes B, C and D, the piles are end bearing on rock with a total length of 26 m (piles are 1m socket in the rock). The pile cap is supported on four piles and has a plan dimension of 7 m x 7 m and a thickness of 2 m. The supporting piles have a grid spacing of 4.2 m in each direction. The piles for all four bridges have a diameter of 1.2 m and steel reinforcement ratio of 1.2%. The piles are embedded a distance of 0.5 m into the pile cap through keys to transfer the shear forces during the earthquakes. To guarantee a smoother transfer of forces, a thick rubber pad is placed on the pile heads. Additionally, the embedded part of the pile is wrapped with neoprene for the smooth transfer of shear forces. Both these components are designed to have dimentions to withstand the DE level ground excitations.

4 NUMERICAL MODEL

The computer software SAP2000 V20.1 was used to develop the 3D finite element models of bridges as shown in Figure 2. The bridge deck is modelled as a spine by using multiple frame elements and lumped masses as suggested by Kappos et al. (2012). Frame elements were used to model the bridge deck, piers and piles as they are capable of capturing biaxial bending, biaxial shear, axial deformation and torsional actions. The piers are connected to the deck using pin connections that allow rotations but restrain translational movement. All the bridge piers have been assigned with flexural plastic hinges taking into effect the interaction of axial and bending moments about the longitudinal and transverse directions at the base of the piers to capture possible inelastic behavior. Additionally, the piers have been assigned a stiffness reduction factor of 0.65, as recommended by Eurocode-8 (European Committee for Standardization 2005) to capture possible cracking that would occur in the tensile region of the pier during the earthquake.

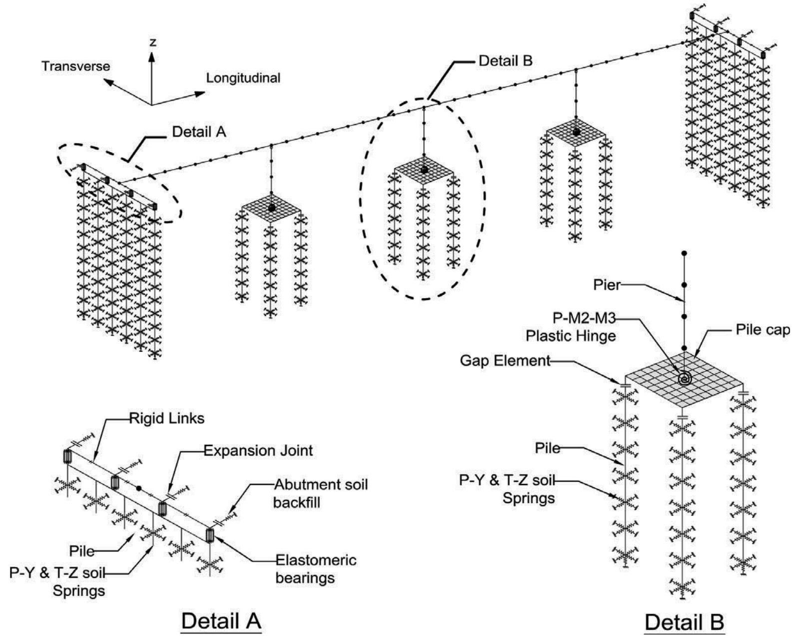

Figure 2. Numerical model of the bridge.

The abutment back-wall is modelled as per Caltrans seismic design criteria v1.6 (Caltrans 2010) recommendations using linear springs with stiffness of 28.7 kPa/mm/m and a maximum capacity of 6.4 kN. A total of four springs spaced at 2.5 m were used and are activated in compression once the expansion joint is closed. The expansion joint is modelled using a gap element with an opening of 300 mm and a compressive stiffness that is equal to the deck stiffness. Moreover, the concrete shear keys were modelled using multilinear springs with a backbone curve adopted from Silva et al. (2009), designed to resist a transverse force of 800 kN.

The elastomeric bearings were modelled using multilinear springs with a horizontal stiffness given by $k_h=G_rA/t_r$; where $G_r=0.9$ MPa is the shear modulus, A is the plan area, and t_r is the total height. The maximum bearing strain defined in Eurocode-8 is 200%. Once the bearing exceeds a horizontal displacement equal to twice the height, the bearing will rupture and its stiffness decreases to 0.

The ground behavior is modelled through various spring properties that depict the behavior of the clay and rock as required. The lateral resistance of the soil for clay is modelled using multilinear springs with a p-y backbone curve recommended by Welch & Reese (1972). The p-y springs can only take compression forces and are assigned to the pile at one meter intervals in horizontal directions to model separation of pile from soil during earthquake loading. The skin friction of the pile was also modelled using t-z springs adopted from the API recommended practice 2A-WSD (API 2000) and were assigned to the pile at one meter intervals. The bearing capacity of the pile was modelled using a linear spring with the compressive strength of the rock. All the springs used to model the effects and behavior of the soil include hysteretic damping with a Takeda hysteresis model.

The pile cap in the rocking pile foundation is modelled as a shell element, while the piles are modelled as frame elements. To model the rocking interface, a gap element is assigned between the pile head and the pile cap that transfers compression and shear forces only (Fatahi et al. 2018). Nonlinear time history analysis was used to simulate the earthquake excitations. Rayleigh damping ratio of 5% corresponding to the first two fundamental periods of vibrations was also adopted (Van Nguyen et al. 2016).

5 RESULTS AND DISCUSSION

5.1 *Modal analysis & period*

A modal analysis was conducted for the bridges, built on different soil types, using Ritz vectors with 98% mass participation ratio, where their periods were then computed and summarised in Table 2.

The first two fundamental modes of the bridge are in the longitudinal and transverse directions, respectively. As shown in Table 2, the natural excitation frequencies of the bridges are affected by the ground characteristics due to the stiffness that ground provides to the piles. Class D type soil has the lowest stiffness and therefore the bridge constructed in this soil has the largest natural excitation frequency. This is due to the reduced stiffness in the soil leading to the increased deflection of the piles. Class A type ground (i.e. rock) on the other hand provides the smallest natural excitation frequency as the rock strata is quite stiff. By increasing the period of the bridge structure, its dynamic response can be significantly reduced depending on the earthquakes' response spectrum.

5.2 *Non-linear dynamic time history analysis*

Non-linear dynamic time history analyses are carried out on the bridges built in different site classes. Two earthquakes excitations, namely 1994 Northridge and 1995 Kobe were applied to the bridges and their structural response in the longitudinal and transverse directions are reported in Tables 3a and 3b, respectively.

5.2.1 *Deck displacements*

The results presented in Tables 3a and 3b and Figure 3 indicate the rocking bridge constructed in ground Class A (rock) experienced the smallest relative deck displacements whereas the bridge constructed in site class D (soft soil) experienced the largest relative displacements of all the soil types. This observation in soft soil is due to the amplification of the earthquake

Table 2. 1st and 2nd Fundamental periods of the bridges in different soil types.

Site Class	1st Mode Period: Longitudinal (First)	2nd Mode Period: Transverse (second)
A	2.55	2.31
B	2.61	2.36
C	2.65	2.38
D	2.72	2.44

Table 3a. Dynamic response of the bridge in the longitudinal direction.

Earthquake	Site Class	Maximum Pier Moment (MN.m)	Pier Design Capacity (%)	Pier Max Drift Ratio (%)	Deck Displacement (m)	Bearing Strain (%)	Maximum Pile Moment (MN.m)	Pile Design Capacity (%)
1995 Kobe	A	28.7	48%	1.94%	0.21365	107%	NA	N/A
	B	36.3	61%	3.23%	0.35532	178%	9.8	54%
	C	40.7	68%	3.41%	0.37456	187%	12.2	68%
	D	45.9	77%	5.62%	0.61832	309%	15.5	86%
1994 Northridge	A	28.3	47%	1.44%	0.1586	79%	NA	N/A
	B	35.7	60%	3.02%	0.33193	166%	9.2	51%
	C	40.3	67%	4.09%	0.44995	225%	11.8	66%
	D	44.7	75%	5.48%	0.60327	302%	15.1	84%

Table 3b. Dynamic response of the bridge in the transverse direction.

Earthquake	Site Class	Maximum Pier Moment (MN.m)	Pier Design Capacity (%)	Max Drift Ratio (%)	Deck Displacement (m)	Maximum Pile Moment (MN.m)	Pile Design Capacity (%)
1995 Kobe	A	34.8	58%	1.94%	0.12198	NA	N/A
	B	37.7	63%	3.23%	0.28299	9.1	51%
	C	44.2	74%	3.41%	0.31665	11.9	66%
	D	48.2	80%	5.62%	0.665	15.2	84%
1994	A	33.6	56%	1.44%	0.15965	NA	N/A
Northridge	B	42.4	71%	3.02%	0.28646	8.7	48%
	C	43.8	73%	4.09%	0.33967	11.4	63%
	D	44.5	74%	5.48%	0.64216	14.7	82%

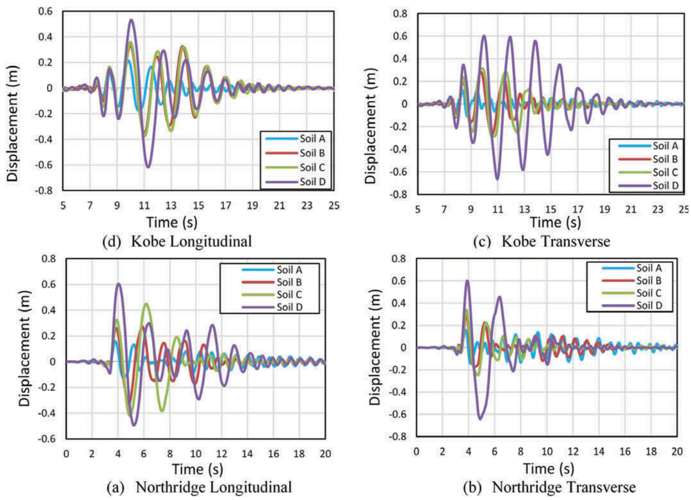

Figure 3. Relative displacement time history of the bridge deck.

signal that travels from rock strata through the soil medium before reaching the structure. The rocking response is not only affected by the amplitude but is also impacted by the frequency of the earthquake signal. For the bridge constructed in ground Class A, the original earthquakes' amplitude and frequency did not change significantly before reaching the structure, and hence the rocking motion of the bridge was kept at a minimal due to its large period and high frequency content of the earthquake. However, for the bridge constructed in soil Class D, the earthquake signal was significantly amplified and changed to a high amplitude and low frequency signal. The rocking motion of the bridge was highly receptive towards lower frequency signals and hence this bridge experienced larger deck deformations.

The results show that the deck displacements for all the bridges except for the bridge constructed in site Class D (soft soil), were within tolerable limits. For the bridge constructed in site Class D, the maximum bearing strain experienced was 309%, and 302% for both the Kobe and Northridge earthquakes, respectively. This strain exceeded the ultimate shear strain of 200%, leading to rupture of the bearing. Additionally, these deck displacements exceeded the seat width at the abutment which was 0.5m resulting in unseating and collapse of the bridge decks. Similarly, for Class D site in the transverse direction, the relative deck displacements reached 0.665 m, and 0.642 m for both the Kobe and Northridge earthquakes respectively, compared to a maximum 0.316 and 0.339 m for Class C sites indicating the great effect of soil stiffness has on the bridge response. This indicates that the shear keys at the abutment were damaged for the bridge built on site Class D, leading to unseating of the bridge deck. Therefore, when designing a rocking bridge foundation in softer soils, it is vital to consider the increase in deck displacements in design to prevent failure of the bridge superstructure.

5.2.2 *Pier bending moments*

The most critical pier in the bridge arrangement is the central pier (Pier 2), hence only the maximum bending moment in that pier is reported in Tables 3a and 3b. It can be observed that the bridge constructed in site Class A (rock) experienced the smallest moments in the pier, whereas in site Class D (soft soil) the pier experienced the largest bending moments. The moment in the piers increased with decreasing soil stiffness as observed in Figure 4. The highest design capacity ratio (ratio between the seismic bending moment force and ultimate bending capacity of the member) experienced in both directions was 80% (site Class D – soft soil), whereas the lowest design capacity ratio was 47% (corresponding to site Class A - rock), indicating a large variation between the induced forces and structural actions depending on the site classification and ground characteristics. All the piers behaved elastically under the MCE loading, however; the bridge built on site Class D (soft soil) experienced the highest moment due to the site amplification of the earthquake signal. Rocking bridges constructed in this soil medium would require the use of bigger and stronger columns in order to perform elastically.

5.2.3 *Pile bending moment*

The maximum pile bending moments experienced under both applied earthquakes are provided in Tables 3a and 3b and Figure 5. It can be observed that the bending moments in the pile increased as the stiffness of the soil decreased. This is due to the decrease in stiffness of the soil resulting in less resistance to pile lateral displacement. Referring to Tables 3a and 3b, the average design capacity ratio of the piles under both earthquakes in the longitudinal direction was 52%, 67%, and 85% for site Classes B, C, and D respectively, and hence the piles required for sites with softer soils must be larger and stronger in order to withstand the bending moments induced by the seismic events.

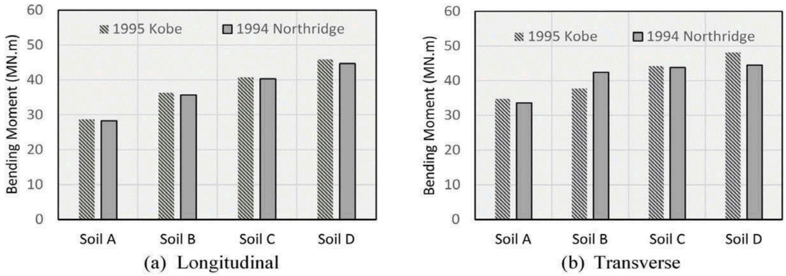

Figure 4. Maximum moment in the central pier of the bridge - Pier 2.

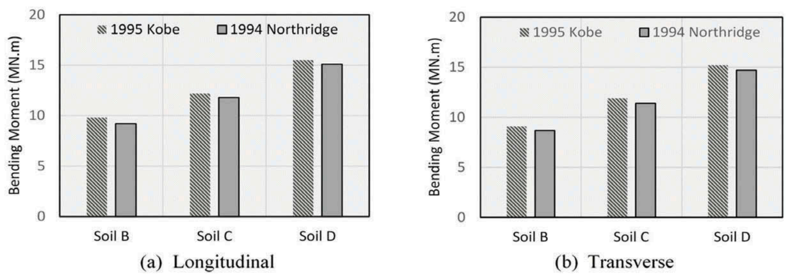

Figure 5. Maximum Bending moment in the piles.

6 CONCLUSIONS

This paper provides a comparative investigation on the performance of bridges with rocking pile foundations in different ground conditions. Four identical bridges with rocking pile foundations were designed on site Classes A, B, C and D as defined in Eurocode-8 and subjected to two near field earthquakes namely, 1994 Kobe, and 1995 Northridge. The dynamic response of the bridges were then compared in both the longitudinal and transverse directions. It was observed that bridges with rocking pile foundations built in softer soils experienced larger deck displacements, larger bending moments in the piers, and in the piles. In general, rocking pile foundations are most efficient when constructed in stiff ground such as site Class A, since rocking structures have large fundamental periods and respond best when subjected to low amplitude, high frequency earthquake signals, which is transferred by stiffer soils or rock strata. On the other hand, when designing rocking pile foundations on weaker soils, design provisions are required to accommodate for induced large displacements and high structural actions.

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