

Variation in modal parameters of bridges due to soil-structure interaction and pier inelasticity

M.T.A. Chaudhary

Department of Civil Engineering, Kuwait University, Kuwait.

ABSTRACT: Modal properties are important parameters of a bridge. This paper presents the results of an analytical study carried out to find out the effect of soil-structure interaction (SSI) and pier column inelasticity on modal parameters of a 4-span bridge typically used for long urban elevated viaducts. The bridge was designed in a moderate seismic zone for five soil conditions with deep foundations and five rock profiles with shallow foundations. Impedances of both types of foundation systems were computed by well-established methods. Non-linear behaviour of reinforced concrete pier columns was modelled for material and geometric non-linearity and was incorporated in the analysis by equivalent linear method. SSI effect was incorporated by the sub-structuring method in the 3-D finite element method (FEM) model of the bridge. FEM model of the bridge was subjected to fifteen actual ground motions varying in peak ground acceleration (PGA) from 0.01g to 0.64g. Results of more than 300 FEM analysis cases were evaluated to delineate the effect of SSI and pier column inelasticity on modal properties of the bridge; which showed wide variation for different soil-foundation systems and variation in seismic ground motion. It was found that pier column inelasticity influenced the modal frequencies of the bridge more significantly than the SSI effect for majority of the analysis cases.

1 INTRODUCTION

Modal properties (mode shapes, modal frequencies and modal damping ratio) are important parameters of a bridge structural system. These parameters can be evaluated from acceleration measurements at discrete locations in a bridge subjected to ambient vibration, forced excitation or seismic activity (Doebling et al 1998; Fan & Qiao 2011 etc.). Changes in the modal properties have been used to determine the structural parameters of the system and any change from the benchmark values can indicate a change in the physical state (i.e. damage) of the bridge (Brownjohn 2007; Chaudhary & Fujino 2008 and others). This analytical parametric study is undertaken to determine the influence of two important phenomena that introduce non-linearity to the bridge system and can cause variation in the modal parameters: (i) soil-structure interaction (SSI) and (ii) pier column inelasticity.

SSI is an important phenomenon in the response of bridges subjected to seismic forces. It has been demonstrated that SSI cannot be ignored for all classes of bridges (Mylonakis & Gazetas 2000) subjected to seismic forces. SSI in bridges has been studied analytically (Ciampoli & Pinto 1995; Olmos & Roesset 2012 among others) as well as from field testing and recorded seismic accelerations (Chaudhary et al 2001; Fraino et al 2012 etc.).

The influence of reinforced concrete pier column inelasticity on the performance of bridges has also been investigated from strength and ductility point of views (Priestley & Park 1987). As pier columns are the main supporting elements of the bridge, therefore any change in their shape, size and strength directly influence the modal properties of the bridge.

Modal frequencies have shown variations of up to 30% between various earthquakes as determined from system identification and analytical studies on bridges (Gomez et al 2013) and buildings (Todorovska 2009; Chaudhary 2009; Butt & Omenzetter 2012). The difference in frequencies is generally attributed to non-linear behaviour in structural components (columns, soil-foundation system, base-isolation bearings, etc.) as well as ambient environmental effects (Laory et al. 2014).

The main objective of the current study is to delineate the effect of SSI and pier column inelasticity on the modal properties of an analytical FEM model of a 4-span bridge with ten different soil-foundation systems when subjected to a suite of fifteen actual seismic ground motions with PGA varying between 0.036 to 0.64 g.

2 BRIDGE DESCRIPTION

This study focused on ordinary and standard bridges which are defined as "those using normal weight concrete, with span lengths less than 90 m, and located in areas with no liquefiable soil" (Caltrans 2006). A multi-span continuous bridge with medium span length (30 m), which is extensively used for long elevated urban viaducts as shown in Fig. 1 is investigated in this study. An interior part of the bridge is selected such that the influence of abutments can be safely ignored.



Figure 1. Longitudinal elevation of the bridge

The bridge comprised of AASHTO Type V prestressed concrete girder superstructure and 11 m tall two-column reinforced concrete bent as sub-structure. Foundations consisted of spread footings for AASHTO site classes A to C while pile foundations were used in AASHTO site classes C and D. Site classes A and B in the AASHTO code (AASHTO 2010) are rock profiles, while class C is a 'soil rock' and class D represents the normal soil profiles. Rock site classes in the AASHTO code were further divided into five rock classes in this study based on the CSIR classification. Similarly, the soil sites represented by classes C and D were further divided into five soil profiles to fill in the gaps in the wide range of V_s in these profiles. The rock and soil profiles used in the study are identified in Table 1 and Table 2 respectively along with their salient mechanical properties.

AASHTO	Rock	Vs	ρ	ν	G	qa
Site Class	Description	m/s	kg/m ³		GPa	kN/m ²
	Very good	3350	2920	0.15	32.60	3816
A	Good	2250	2610	0.20	13.18	2051
	Fair	1300	2320	0.25	4.00	839
В	Poor	780	2090	0.30	1.22	385
С	Very Poor	600	2060	0.35	0.74	215

Table 1: Rock profiles and their mechanical properties

Table	2: Soil pi	ofiles an	d their mec	hanıcal	properties	5

AASHTO Site Class	Soil Profile	Vs (m/s)	ρ (kg/m3)	ν	G (MPa)	β (%)
G	IIupper	600	2060	0.35	741	3
C	II _{avg}	475	2020	0.35	456	4
D	IIIupper	350	1980	0.40	243	5
	III _{avg}	275	1900	0.40	144	7
	III _{low}	175	1850	0.42	57	8

The 2 m square reinforced concrete pier columns, group of eleven 1 m diameter pile foundations, spread footings of various plan dimensions and superstructure components were designed according to the AASHTO code stipulated load combinations of dead, live and seismic loads. The bridge was designed for HL-93 live load and was located in a moderate seismic zone with A = 0.2g.

3 METHODOLOGY

This study utilized a sub-structuring method in which the SSI problem was split into two parts. Frequency-independent dynamic impedance of shallow footings in rock (Table 3) and pile foundations in the soil profiles (Table 4) was computed in the lateral, vertical and rocking modes by the procedures available in the literature. These impedances were incorporated in the 3-D FEM analytical model of the bridge as Winkler springs.

5	Foundation size	K_v	K _{Hx}	K_{Hz}	K _{Rx}	K _{Rz}
Rock	(L x B x D)	x10 ⁷	x10 ⁷	x10 ⁷	x10 ⁸	x10 ⁹
Description	(m x m x m)	(kN/m)	(kN/m)	(kN/m)	(kN-m/rad)	(kN-m/rad)
Very good	12.6 x 3.2 x 1.75	61.9	61.7	56.6	19.4	15.7
Good	12.6 x 3.4 x 1.75	27.1	26.0	23.8	9.43	6.99
Fair	12.8 x 4.0 x 2.00	9.29	8.49	7.79	4.28	2.56
Poor	13.2 x 5.0 x 2.25	3.25	2.82	2.62	2.25	0.99
Very Poor	14.8 x 6.0 x 2.50	2.47	1.97	1.83	2.33	0.95

Table 3: Shallow foundation stiffness in various modes for the selected rock profiles

Table 4: Pile group stiffness in various modes for the selected soil profiles

<i>a</i>	Pile	K _v	K _{Hx}	K _{Hz}	K _{Rx}	K _{Rz}
Soil	length	x10 ⁶	x10 ⁶	x10 ⁶	x10 ⁸	x10 ⁸
Profile	(m)	(kN/m)	(kN/m)	(kN/m)	(kN-m/rad)	(kN-m/rad)
IIupper	22	29.15	11.23	10.97	13.28	8.22
II _{avg}	25	19.54	7.64	7.40	8.94	5.55
III _{upper}	30	11.78	4.78	4.54	5.43	3.39
III _{avg}	32	7.31	3.16	3.01	3.93	2.14
III _{low}	35	3.07	1.53	1.47	1.47	0.95

The bridge system was modelled in a commercially available FEM package utilizing equivalent linear properties of non-linear soil-foundation system and sub-structure components. The FEM bridge model was subjected to a suite of fifteen actual ground motions with PGA varying between 0.036g and 0.64g. Acceleration spectra of these ground motions are presented in Fig. 2.

Member forces and displacements in various components of the bridge system were computed along with the modal properties of the bridge for each seismic ground motion simulation. Stiffness of the pier columns was adjusted for the computed displacement values in the next iteration to account for inelasticity in the pier column as determined from the load-deflection curve in Fig. 3.

4 FINITE ELEMENT METHOD (FEM) MODEL

The analytical model of the bridge was made in FEM package STAAD (Bentley Systems, 2015) and is depicted in Fig. 4. The bridge super-structure was modeled by two different finite elements. Beam elements with six degrees of freedom were used for modeling the girders and diaphragms, while four node plate elements were employed for the bridge deck. Full composite action was assumed between the girders and the deck slab in the FEM model. Sub-structure consisted of pier columns, pier cap and pile cap; all of which were modeled by beam elements. Foundation-soil system was modeled by Winkler springs. The pile cap which connects the two columns was modeled as a rigid beam to accurately estimate the effect of seismic forces transferred to piles and soil.



Fig. 2. Spectra of used seismic motions

Fig. 3: Load deflection curve for pier



Fig. 4. 3D FEM model of the bridge

Non-linear behavior of the pier columns was captured by employing equivalent linear stiffness of the pier columns computed from the load-displacement (P- Δ) curve of the columns as shown in Fig. 4. P- Δ curve of the column was computed by integration of the moment-curvature (M- ϕ) relationship of the column section. Reduction in pier column stiffness was computed for the maximum elastic displacement determined from the case of linear elastic pier columns.

FEM analysis of the bridge model was conducted for the suite of fifteen selected ground motions for two basic conditions, i.e. linear elastic pier columns and non-linear pier columns. For each of the two pier column conditions, eleven boundary conditions were investigated. One boundary condition was with a fixed foundation, i.e. no SSI and the remaining ten represented boundary conditions with SSI with varying values of the rock/soil-foundation springs corresponding to the five rock and five soil profiles as outlined in Sections 2 and 3. Altogether 330 FEM analyses were conducted for all cases of pier elasticity, rock/soil-foundation systems and ground motion records. Results of these analyses related to the modal parameters of the bridge are presented in the next section.

5 MODAL PARAMETERS OF THE BRIDGE

5.1 Mode shapes

The first mode represented the longitudinal translation of the bridge while transverse translation was the 2nd mode. First torsional mode was the 3rd mode and the 4th mode consisted of combined torsion and transverse translation. The 5th mode was the 1st vertical mode. No visual change was noted in the mode shapes due to changes in the boundary conditions from the fixed base and elastic pier case (representing no SSI and no pier inelasticity) to various cases of rock/soil-foundation springs (representing SSI cases) as well as for the cases including inelastic pier columns. Average MAC values for the first five modes varied from 0.85 to 0.98 for the considered cases of SSI and pier inelasticity, which implies that these mode shapes were rather insensitive to the variations in soil-foundation system and inelasticity in the pier columns.

5.2 Modal frequencies

5.2.1 Elastic pier column

Variation of the first five modal frequencies of the bridge for the case of elastic pier with fixed base and various conditions of soil/rock-foundation springs is presented in Fig. 5. It is to be noted that the modal frequencies for the elastic pier case are independent of the used ground motion. An examination of Fig. 5 reveals that frequencies in modes 1 to 4 for the soil profiles displayed variation of 8%, 14%, 11% and 5% respectively with respect to the various soil conditions. On the other hand, modal frequencies in rock profiles showed a variation between 0.5% and 2.6% for the changes in boundary conditions (i.e. fixed base or different cases of soil/rock-foundation springs). A general trend can be observed that the frequencies in any particular mode progressively reduce from the fixed base condition to the weaker soil/rock cases.



Fig. 5. Variation in modal frequencies of bridge with elastic pier columns

5.2.2 Inelastic pier column

For the inelastic pier column case, the modal frequencies are dependent on the stiffness of pier columns and the support boundary conditions (fixed or soil/rock-foundation springs). Pier column stiffness varied due to the level of lateral displacement induced by the ground motion. Therefore, modal frequencies changed when the ground motion was changed. Figures 6 and 7 depicts the variation of first four modal frequencies as a function of the used ground motions (in terms of PGA) for the various soil and rock profiles respectively.

6 DISCUSSION ON VARIATION IN MODAL FREQUENCIES

A clear trend can be observed from Figures 6 and 7. That is, modal frequency decreases with increasing PGA as well as decrease in the soil/rock-foundation stiffness. The decrease in modal frequency with PGA is relatively pronounced for the first three modes of vibration for the soil profiles. The first modal frequency decreased by 23% for the fixed base case between earthquake records 1 and 15 (i.e. from PGA of 0.036 g to 0.64 g) and for the weakest soil (III_low) the decrease was 29%. The decrease in the 2nd modal frequency was 16% and 31% for the fixed base and soil III_low cases respectively across EQ records 1 and 15. The same numbers were 11% and 28% for the 3rd modal frequency. However, the 4th modal frequency changed by only 6% and 8% for the fixed base and soil III_low cases respectively. These changes for the bridge in rock profiles were: 22%, 16%, 11% and 5% for modes 1 to 4 respectively for the fixed base case and 25%, 24%, 21% and 6% for the weakest rock profile.

These observations pointed to the fact that it can become increasingly difficult to identify change in frequency as the order of modal frequency increases due to a smaller range of such change. This observation can have implications for identification of localized changes (resulting from damaged members or change in boundary conditions) in the bridge system which is usually associated with higher modes. The smaller difference in higher modal frequencies for various soil/rock profiles also implies that these modes are not significantly affected by SSI and/or changes in pier column stiffness.



Fig. 6. Variation in modal frequencies of bridge in soil profiles with inelastic pier columns



Fig. 7. Variation in modal frequencies of bridge in rock profiles with inelastic pier columns

7 EFFECT OF SSI & PIER INELASTICITY ON MODAL FREQUENCIES

Modal frequencies remain constant for the elastic pier case for all levels of ground motions and show variation only due to the change in the stiffness of the boundary support / foundation springs. However, for the case of inelastic pier, modal frequencies change due to change in stiffness of the pier as well as foundation spring as discussed in Section 6.







Fig.9. Components of change in modal frequencies - block foundation bridge (rock profiles)

Figs. 8 and 9 present the breakdown of the components (i.e. SSI and pier column inelasticity) causing change in modal frequencies for soil profile and rock profile bridges respectively. The contribution of each component is computed by the following relationships:

1

$$Pier inelasticity \ component = \ (f_{fixed}^{elastic} - f_{fixed}^{inelastic}) / f_{fixed}^{elastic}$$
(1)

$$SSI \ component = \ (f_{fixed}^{inelastic} - f_{SSI}^{inelastic}) / f_{fixed}^{elastic}$$
(2)

Pier column inelasticity accounted for an average change of 66%, 49%, 40% and 84% in 1st to 4th modal frequencies respectively for the pile supported bridge in the weakest soil profile as shown in Fig. 8. Fig. 9 reveals that the share of pier column inelasticity to average change in 1st to 4th modal frequencies was

85%, 72%, 67% and 91% respectively for the weakest rock profile bridge. This indicates that most of the change that resulted in the lower modal frequencies was due to inelasticity in the pier columns. Contribution of pier inelasticity towards change in the 3rd modal frequency in both soil and rock profile bridges was relatively less. This is due to the fact that the 3rd mode was a torsional mode and torsional stiffness of the bridge sub-structure was substantially more than the torsional stiffness of the foundation.

Modal frequencies computed from actual measurements in bridges have shown variation under earthquakes of various intensities. This variation can only be captured in the theoretical calculations of modal frequencies if the pier column stiffness is changed corresponding to the level of pier column displacement induced by the earthquake motion along with proper treatment of SSI. Furthermore, results of this analytical investigation should be validated by actual field measurements before adoption of these findings in practice.

8 CONCLUSIONS

- i) Lower vibration mode shapes of the bridge studied herein were insensitive to SSI as well as pier column inelasticity.
- ii) Majority of the lower modal frequencies were affected more by pier column inelasticity than by SSI. iii)SSI affected torsional modal frequency more than the translational modal frequencies.
- iv) Pier inelasticity has to be incorporated in the structural model updating procedure when using identified modal parameters from a bridge which is instrumented as part of a Structural Health Monitoring (SHM) scheme.

9 **REFERENCES**

- AASHTO, 2010. AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, Washington, DC.
- Bentley Systems, Inc., 2015. STAAD V8i, 3D Structural Analysis and Design Engineering Software, Exton, PA.
- Brownjohn, J.M.W. 2007. Structural health monitoring of civil infrastructure. *Phil. Trans. Roy. Soc., Physical and Engineering Sciences*, 589-622, London.
- Butt, F. & Omenzetter, P. 2012. Evaluation of seismic response trends from long-term monitoring of two instrumented RC buildings including soil-structure interaction. *Advances in Civil Engineering*, Article ID 595238 (doi:10.1155/2012/595238), 18 pages.
- CalTrans, 2006. Seismic design criteria, SDC-2006. California Department of Transportation, Sacramento.
- Chaudhary, M.T.A., Abe, M. & Fujino, Y. 2001. Identification of soil-structure interaction effect in base-isolated bridges from earthquake records. Soil Dynamics and Earthquake Engineering, 21(8), 713-725.
- Chaudhary M.T.A. & Fujino, Y. 2008. System identification of bridges using recorded seismic data and its application in structural health monitoring. *Structural Control and Health Monitoring*, Volume 15, 1021-1035.
- Chaudhary, M.T.A. 2009. Options for support tower design of a sensitive equipment under ambient vibrations. *Structural Design of Tall and Special Buildings*, 18(8): 861-875.
- Ciampoli, M. & Pinto, P.E. 1995. Effects of soil-structure interaction on inelastic seismic response of bridge piers. *Journal of Structural Engineering*, 121(5), 806-814.
- Doebling, S. W., Farrar, C. R., & Prime, M. B. 1998. A summary review of vibration-based damage identification methods. *Shock and Vibration Digest*, 30(2), 91-105.
- Fan, W., & Qiao, P. 2011. Vibration-based damage identification methods: a review and comparative study. *Structural Health Monitoring*, *10*(1), 83-111.
- Fraino, M., Ventura, C.E., Liam Finn, W.D. & Taiebat, M. 2012. Seismic soil-structure interaction effects in instrumented bridges. *Proceedings of the 15th World Conference on Earthquake Engineering*, Lisbon.
- Gomez, H.C., Ulusoy, H.S. & Feng, M.Q. 2013. Variation of modal parameters of a highway bridge extracted from six earthquake records. *Earthquake Engineering and Structural Dynamics*, 42, 565-579.
- Laory, I., Trinh, T. N., Smith, I. F., & Brownjohn, J. M. 2014. Methodologies for predicting natural frequency variation of a suspension bridge. *Engineering Structures*, 80, 211-221.
- Mylonakis, G. & Gazetas, G. 2000. Seismic soil-structure interaction: beneficial or detrimental? *Journal of Earthquake Engineering*, 4(3), 277-301.
- Olmos, B.A. & Roesset, J.M. 2012. Inertial interaction effects on deck isolated bridges. *Bulletin of Earthquake Engineering*, 10(3), 1009-1028.
- Priestley, M. J. N., & Park, R. 1987. Strength and ductility of concrete bridge columns under seismic loading. *ACI Structural Journal*, 84(1).
- Todorovska, M. I. 2009. Soil-structure system identification of Millikan Library North–South response during four earthquakes (1970–2002): What caused the observed wandering of the system frequencies? *Bulletin of* the Seismological Society of America, 99(2A), 626-635.