Evaluation of Vehicle-Bridge Interaction during Earthquakes

H. Wibowo, D. M. Sanford, I. G. Buckle, & D. H. Sanders
Center for Civil Engineering Earthquake Research, University of Nevada, Reno, U.S.A.

15 WCEE
LISBOA 2012

SUMMARY:
Live load provides not only additional gravity load on a bridge but also dynamic force effects during earthquakes due to the flexible nature of suspension-tire systems. However, the significance of these dynamic effects on the seismic response of a bridge is unclear. Most bridge design specifications have few requirements or are silent about the inclusion of live load in the seismic design of bridges. The main objective of this study is to investigate these dynamic effects using both experimental and analytical techniques. This paper focuses on the experimental work, which includes shake table testing of a 0.4-scale model of a horizontally curved steel girder bridge loaded with a series of representative trucks. Preliminary experimental results show that the presence of the live load had a significant beneficial effect on the performance of this bridge during small amplitude motions, but that these effects became less significant with increasing amplitude of shaking.

Keywords: live load, seismic response, vehicle-bridge interaction, multiple shake tables, bridges

1. INTRODUCTION

Although earthquake reconnaissance reports have shown that live load is present during earthquake events, design procedures for earthquake-resistant bridges in most countries do not require the simultaneous presence of live load and earthquake load to be considered. This decision is based on two major assumptions. First, it is unlikely that the full design live load will be on the bridge at the time of the design earthquake, and second, the seismic response of a bridge is dominated by its dead load and live load inertial effects are negligible by comparison. However for bridges in urban and metropolitan areas where congestion is a frequent occurrence, some fraction of the design live load (usually taken as 50%) is now recommended to be included with the dead load when computing gravity load effects (AASHTO, 2012). But this recommendation applies only to gravity load effects and not to inertial effects.

The omission of inertial effects in design is the result of a prevailing attitude that the suspension system of a heavy vehicle acts in a manner similar to a tuned mass damper and reduces the motion in the bridge. It is therefore believed to be conservative to ignore these effects. But in fact little is understood about the dynamic interaction between heavy vehicles and bridge systems during strong shaking and there is no hard evidence that the tuned mass damper model is universally applicable. It is equally possible that the added weight increases the inertial loads in the bridge and the corresponding displacements and forces.

2. LITERATURE SURVEY

Currently, very little research has been conducted to resolve the live load issue. Previous work has shown that live load can either have a beneficial or an adverse effect on the structure during earthquake shaking. However, there are still uncertainties about the reason why this is so and there has been no large-scale experimental work to investigate the effects of live load on the seismic response of bridges prior to this experiment.

An earlier study by Sugiyama *et al.* (1990) used a single degree-of-freedom vehicle system that can model rolling in the transverse direction and pitching in the longitudinal direction, but the properties are not given. The bridge was idealized as a nine-mass system with transverse and rotation inertia connected by linear springs and damper elements. A vibration test is reported on an existing steel girder bridge with and without trucks in the longitudinal and transverse directions to verify the results. In the test, two large trucks were parked facing the same direction on a portion of an existing off ramp whose girders were vibrated with an electro-hydraulic exciter. The bridge was tested with the vehicles empty and loaded to various capacities. The results show that the dynamic effect of the vehicle is more dominant in the transverse direction and the vehicle tends to reduce the response of the bridge. They also mentioned that as the exciting force level increases, the effects of nonlinearity become more apparent since the dynamic characteristics of the vehicle itself are nonlinear. These results are corroborated by Kameda *et al.* (1992) who used a 5 degrees-of-freedom model in their study. These authors state that the vehicle tends to increase the bridge response when the vehicle is in the in-phase mode with the bridge and decrease the bridge response when it is in the out-of-phase mode. Moreover, they also concluded that the ratio of the fundamental frequencies of the bridge and the vehicle plays an important role for the response of the bridge.

Furthermore, another study of seismic response of a bridge with live load was done by Kawatani *et al.* (2007). They analyzed the seismic response of a steel plate girder bridge under vehicle loadings during earthquakes. The vehicles were modeled with 12 degrees-of-freedom that took sway, yaw, bounce, pitch, and roll into account. The observations from the numerical analysis showed that heavy vehicles acting as a dynamic system can reduce the seismic response of bridges under a ground motion with low frequency characteristics, but the vehicles have the opposite effect and slightly amplify the seismic response of the bridge under high frequency ground motions.

Kawashima *et al.* (1994) and Otsuka *et al.* (1999) performed a series of study to determine the effect of live load on a bridge when combined with seismic load. The study modeled a two-span simply supported girder bridge with a mix of ordinary cars, modeled as additional dead load, and large trucks, each modeled with 5 degrees-of-freedom. The bridge was only analyzed in the transverse direction because it was estimated that the deck response would be significantly affected by the rolling of the large trucks. The studies found that the displacement response of the girders increased by 10% when the live load was included; ductility demand at the bottom of the column also increased by 10% with live load on the bridge. This study concluded that this was not enough of an effect to be significant and safety factors can be modified to take this effect into account if they are not already sufficient enough. It was also concluded that the increase in response was due to the increase of weight, however, the effect of the large trucks was not just to increase the dead weight, and they also behaved as a mass damper.

Scott (2010) developed a simplified modeling approach for dynamic analyses to account for combined live load and seismic load. It is shown that for short-span bridges, the displacement responses are mainly due to the fundamental bridge mode. In addition, for long-span bridges, vehicle speed has small influence on the displacement and acceleration responses of the bridge.

A recent study on the effects of live load a highway bridge under moderate earthquake in the horizontal and vertical directions was reported by Kim *et al.* (2011). The study concluded that the seismic responses of the bridge are amplified when the vehicle is considered as merely additional gravity load or mass and the amplification is dependent on the relationship between the fundamental frequency of the bridge and the response spectra if the ground motion. However, when the vehicle is considered as dynamic or mass-spring-damper system, which is more realistic, the dynamic effect of the vehicle is greater than its gravity load addition effect and thus it reduces the seismic response. In addition, the study also showed that the effect of moving vehicle as compared to stationary vehicle is negligible. Therefore, it is sufficient to model the vehicle as stationary for this purpose.

3. BRIDGE MODEL AND REPRESENTATIVE VEHICLE

A three-span, curved bridge model was tested on the NEES Shake Table Array in the Large-Scale Structures Laboratory at University of Nevada, Reno. This 0.4-scale model has a steel plate girder superstructure, single-column reinforced concrete substructures, and seat-type abutments. Overall dimensions are shown in Table 1 below.

Table 1. Bridge Geometry Summary

Parameter	Prototype	Model
Total Length	362.5'-0" [110.49 m]	145'-0" [44.196 m]
Span Lengths	105'-0" [32.004 m], 152'-6" [46.482 m], 105'-0" [32.004 m]	42'-0" [12.802 m], 61'-0" [18.593 m], 42'-0" [12.802 m]
Radius at Centerline	200'-0" [60.96 m]	80'-0" [24.384 m]
Subtended Angle	104° (1.8 rad)	104° (1.8 rad)
Total Width	30'-0" [9.144 m]	12'-0" [3.658 m]
Girder Spacing	11'-3" [3.429 m]	4'-6" [1.372 m]
Total Superstructure Depth	6'-6.125" [1.984 m]	2'-7.25" [0.794 m]
Column Height	19'-2' [5.842 m]	7'-8" [2.337 m]
Column Diameter	5'-0" [1.524 m]	2'-0" [0.61 m]

The bridge model has a total length of 145 ft [44.196 m], a total width of 12 ft [3.658 m], and subtended angle of 104° as shown in Figs. 1 and 2. Each bent has a single circular column. The column height is 7 ft - 8 in [2.337 m] with a diameter of 24 in [0.61 m]. The superstructure is a three-span, three-girder steel bridge with concrete deck. The detail of the superstructure and the column can be seen in Fig. 3. The superstructure is supported by fixed (rotation-only) pot bearings at the bent locations and sliding bearings at the abutments. Moreover, shear keys are provided at the abutments to restrain movement in the radial direction during small amplitude earthquakes, but are designed to fail at higher events to protect the abutment foundations against damage.

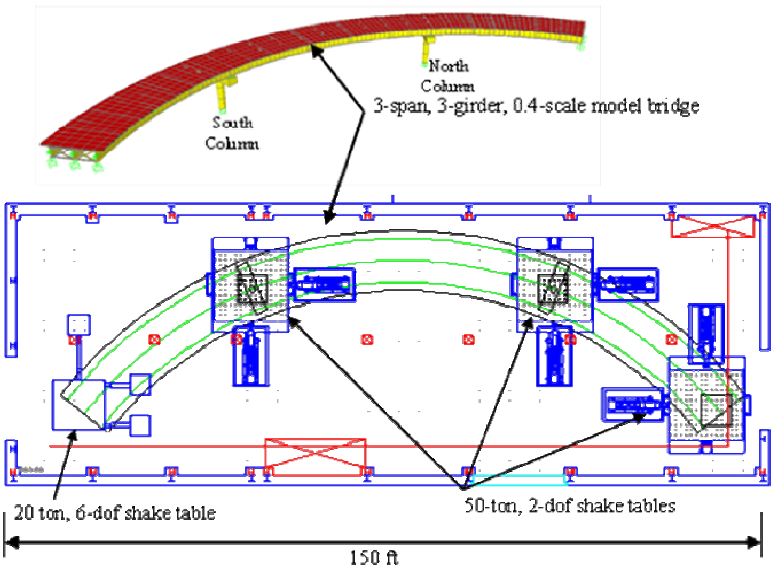


Figure 1. Bridge Model and Layout in Large-Scale Structures Laboratory

The prototype bridge was designed for a site in Seismic Zone 3 (AASHTO, 2012) with a 1,000-year spectral acceleration at 1.0 second (S_1) of 0.4 g. Under this Design Earthquake (DE), the bridge is expected to be damaged but not collapse. The record selected as the input motion for the experimental studies was the Sylmar record from the 1994 Northridge Earthquake near Los Angeles, scaled to have the same spectral acceleration at 1.0 second. A scale factor of 0.475 was therefore applied to both the

NS and EW time histories of ground acceleration from this station.

Figure 2. Bridge Model Assembled in Large-Scale Structures Laboratory

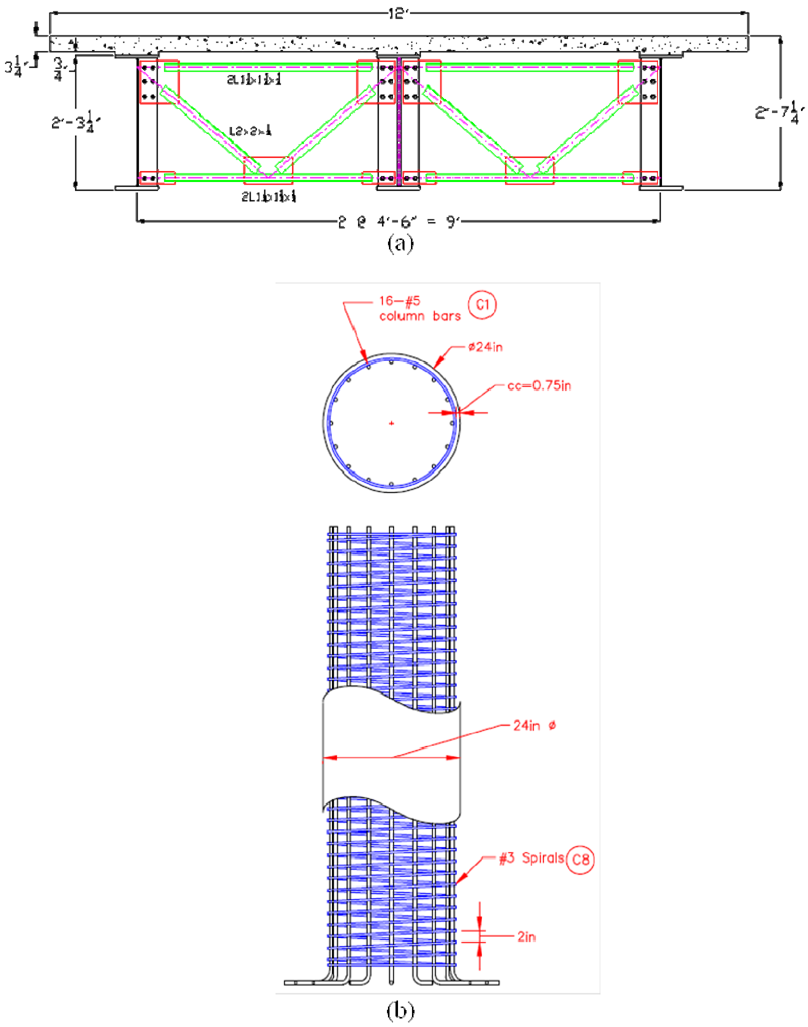


Figure 3. Typical Superstructure and Column Details

The starting point for selection of the test vehicle was the H-20 truck, which is a two-axle vehicle weighing 40 kip (8 kip on the front axle and 32 kip on the rear axle) [178 kN: 35.6 kN and 142.4 kN] with a 14 ft [4.267 m] wheel base. For a 0.4-scale model, the model truck would have a wheel base of 5.6 ft [1.707 m], a width of 2.4 ft [0.732 m], and weigh 6.4 kip [28.48 kN]. Since such a vehicle would most likely have to be custom-built and thus not economically feasible, the decision was made to select from commercially available vehicles. The closest vehicle to match the modeling requirements and constraints of the experimental setup was found to be the Ford F-250. Although the similitude requirements are not fully satisfied, the dynamic properties of the chosen vehicle can produce similar effects to those of the target vehicle.

Table 2. Ford F-250 Dimensions and Weight Ratings

Parameter	Value
Overall Length	247” [6.274 m]
Overall Width	68” [1.727 m]
Overall Height	80” [2.032 m]
Wheel Base Length	156” [3.962 m]
Ground Clearance	7.9” [0.201 m]
Curb Weight	6.7 kip [29.815 kN]
Gross Vehicle Weight Rating	10 kip [44.5 kN]
Max Allowable Payload	2.3 kip [10.235 kN]

4. EXPERIMENTAL SETUP

The bridge model was assembled on the four NEES shake tables in the Large-Scale Structures Laboratory and the vehicles positioned on the deck as shown in Figs. 4 and 5. Instrumentation has been installed on the columns, bridge girders, and trucks to gather response data during testing. The types of instruments range from strain gauges on the column rebar, string pots on the bridge girders and trucks (to measure displacements), and accelerometers on the bridge deck and trucks (to measure accelerations). During the experiment, 383 data acquisition channels were used.

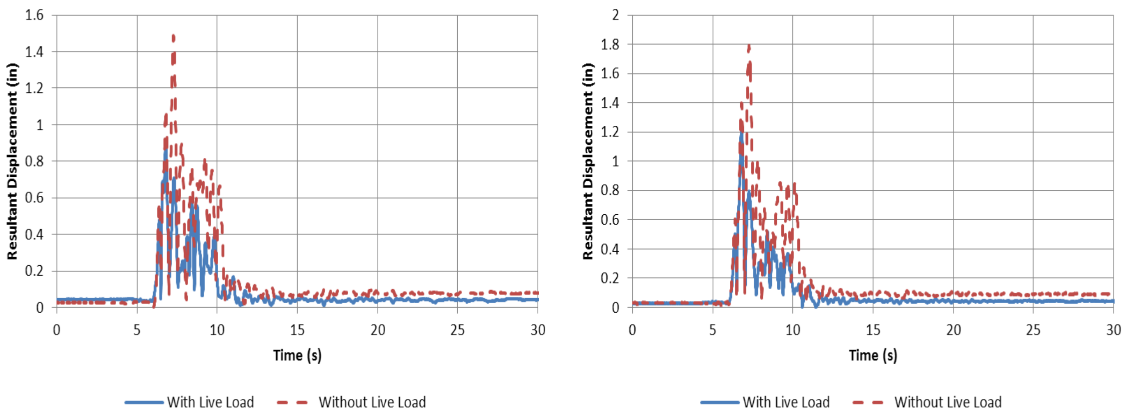
Figure 4. Bridge Model with Live Load (Courtesy of M. Wolterbeek, 2011)

Figure 5. Fish-Eye View of Experimental Model in the Laboratory

The test protocol followed for this experiment started with 10% of the DE and then the motion was increased in successive increments to 20%, 50%, 75%, 100%, 150%, 200%, 250%, 300%, and 350% of the DE. Before each run, a series of low level white noise excitations were run to characterize the system's dynamic properties.

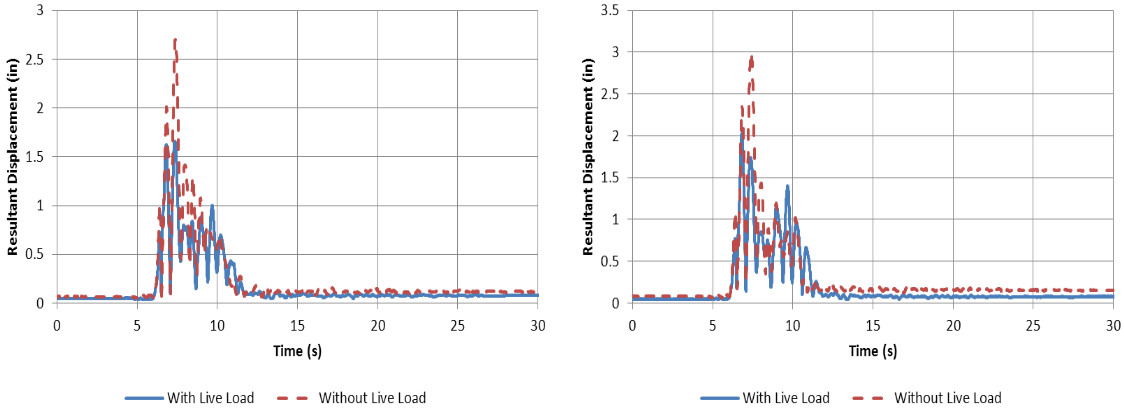
5. EXPERIMENTAL RESULTS

One of the parameters that may be used to quantify the effect of live load is the column displacement. Figs. 6 and 7 show the north and south column displacements with and without live load under 75% and 100% of DE, respectively. It is shown that for these two runs, the maximum displacement is less when live load is present. It is also important to note that during the no-live load case, the shear keys at the abutment failed during the 75% DE run, whereas it took a stronger ground motion (100% DE) to fail these keys when live load was present, i.e. the live load reduced the forces in the shear keys at the same level of excitation. Maximum shear key forces with and without live load are summarized in Table 3. This observation shows that at these levels of shaking, the existence of live load caused less demand in the column and reduced the radial shear forces at the abutments. The damage in the column was also found to be minor and not as severe as for the no-live load case.



(a) (b)

Figure 6. (a) North and (b) South Column Displacement Histories during 75% DE Run



(a) (b)

Figure 7. (a) North and (b) South Column Displacement Histories during 100% DE Run

Table 3. Maximum Radial Shear in the Shear Key at North Abutment (NA) and South Abutment (SA)

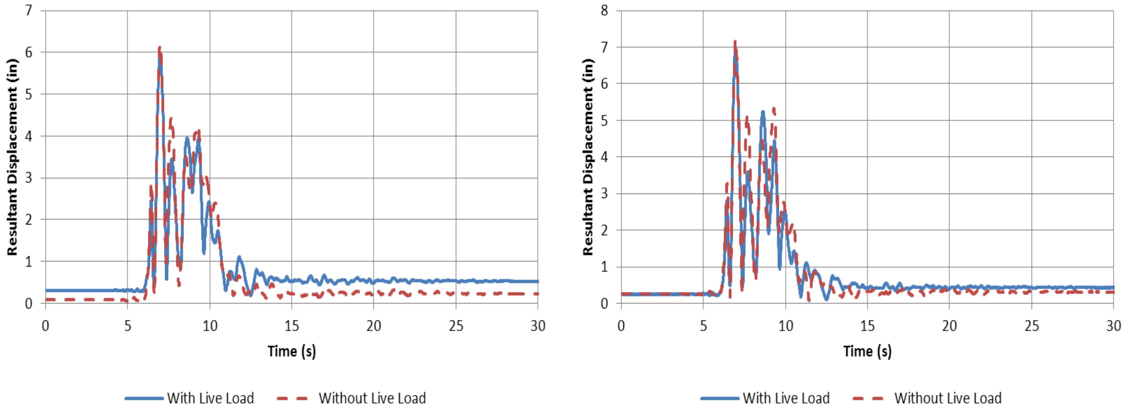
Run	With Live Load (kip) [kN]		Without Live Load (kip) [kN]	
	NA	SA	NA	SA
10% DE	0.76 [3.38]	3.06 [13.62]	5.48 [24.39]	4.30 [19.14]
20% DE	3.29 [14.64]	7.27 [32.35]	11.70 [52.07]	8.51 [37.87]
50% DE	16.54 [73.60]	17.32 [77.07]	29.38 [130.74]	18.00 [80.10]
75% DE	22.05 [98.12]	22.65 [100.79]	*33.51 [149.12]*	*1 252.40 [5 573.18]*
100% DE	*23.31 [103.73]*	*23.31 [103.73]*	N/A	N/A

Note: Values in italics are values at instant when shear key failed

Observations from the higher amplitude runs, after the shear keys at the abutments had failed, show maximum displacements that are almost the same in the two cases. Figs. 8 and 9 show the displacements in the north and south columns with and without live load after 250% and 300% of DE, respectively. It is seen that at these levels of shaking (and after the keys had failed), the live load exercises the columns to a similar extent and the maximum displacements at the top of the columns became closer to the no-live load case. It is also seen that the residual displacements in the columns for the live load case are about double those without live load. These larger residual displacements indicate greater distress to the columns, and especially the south column, due to the presence of the live load.

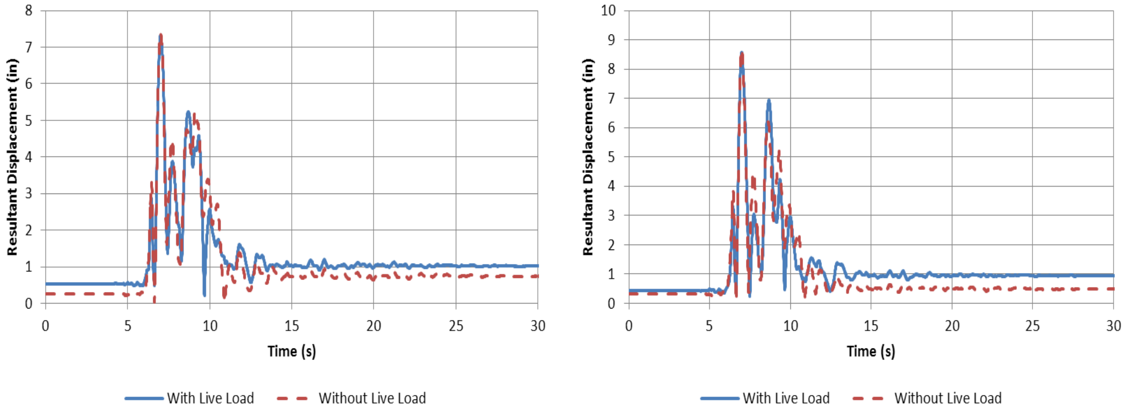
Another parameter to quantify the effect of live load on seismic response of the bridge is the extent of spalling in column's plastic hinge zone. Fig. 10 shows comparison of the spalling that occurred at the bottom plastic hinge zone on the south face of south column with and without live load. It can be observed that the spalling on column without live load is more extensive and the plastic hinge zone is greater than on the column with live load. This phenomenon is not that apparent on the north column, as depicted in Fig. 11 where the column experienced less damage.

Abutment uplift is observed during the experiments with and without live load. This upward displacement at the abutment becomes larger at higher earthquake intensity runs. Figs. 12 and 13 show the vertical displacements of the north and south abutments measured at the bottom of the outer and inner bays during the 350% DE runs with and without live load. Positive displacement means that the bridge deck is moving upward or experiencing uplift. The bridge uplifts due to the torsional behavior of the curved bridge. It tends to uplift towards the inner girder at the north abutment while the south abutment remains relatively in place. It can be observed from the graphs that maximum abutment uplift when live load is present is about the same or less as the abutment uplift without live load.



(a) (b)

Figure 8. (a) North and (b) South Column Displacement Histories during 250% DE Run



(a) (b)

Figure 9. (a) North and (b) South Column Displacement Histories during 300% DE Run

(a) 

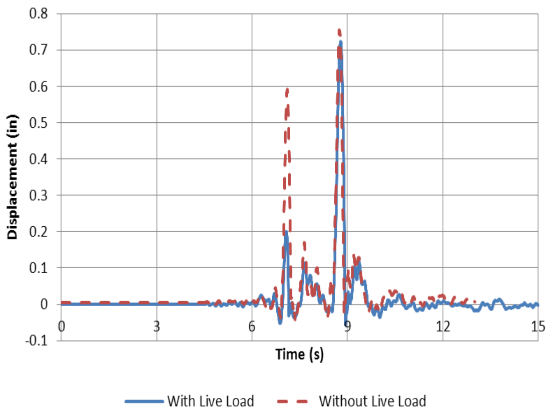
(b) 

Figure 10. Spalling on South Face of South Column (a) With and (b) Without Live Load After 350% DE Run

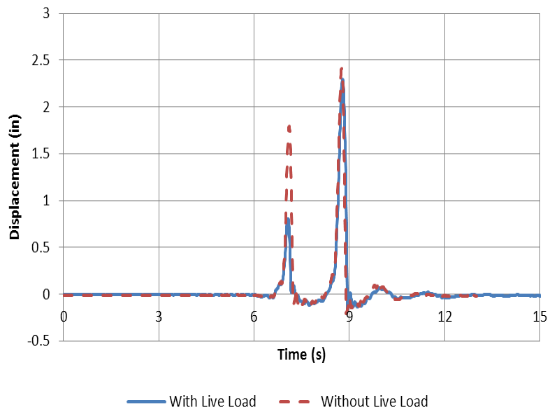
(a) 

(b) 

Figure 11. Spalling on South Face of North Column (a) With and (b) Without Live Load After 350% DE Run

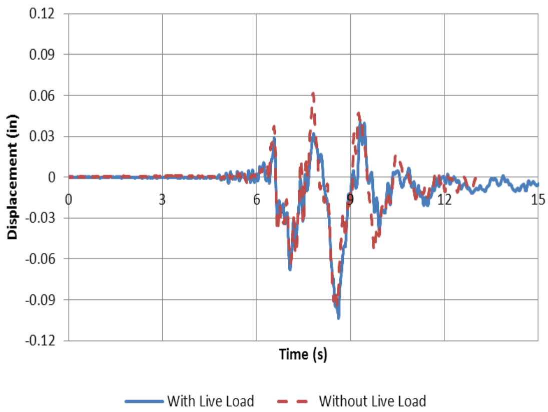


(a)

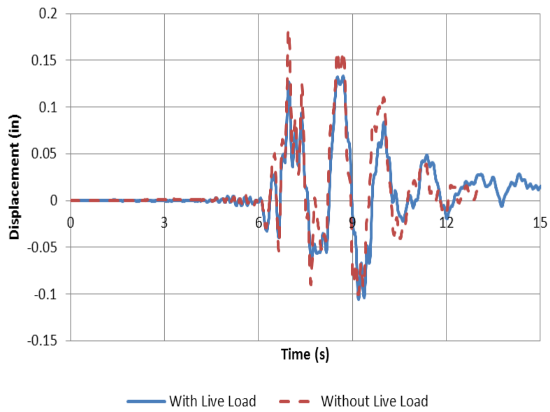


(b)

Figure 12. North Abutment's (a) Outer and (b) Inner Bays Displacement Histories during 350% DE Run



(a)



(b)

Figure 13. South Abutment's (a) Outer and (b) Inner Bays Displacement Histories during 350% DE Run

6. CONCLUDING REMARKS

From the experimental results with and without live load presented herein, some observations can be made. In lower amplitude motions, when the shear keys were still intact, live load gave an apparent beneficial effect. In higher amplitude motions, after the abutments were free to move, the effect of the live load was less significant. This may be due to (1) the deteriorating nature of the bridge under increasing levels of shaking and thus changing vehicle-to-bridge frequency ratio and increasing the structural damping, or (2) the changed configuration of the bridge when the abutments were released in the radial direction after the shear keys failed, or (3) both of the aforementioned.

Studies are continuing to better understand this phenomenon. Analytical models have been developed to further extend the study numerically to obtain some limitations on when the live load gives beneficial or adverse effect to the structure.

ACKNOWLEDGMENT

This project is funded by California Department of Transportation (Caltrans) under contract number 59A0695 and the Federal Highway Administration (FHWA) under contract number DTFH61-07-C-00031: Improving the Seismic Resilience of the Federal Aid Highway System. The Caltrans Program Manager is Dr. Allaoua Kartoum and the FHWA Contracting Officer's Technical Representative is Dr. Wen-huei (Phillip) Yen. The Principal Investigator is Dr. Ian Buckle and the co-investigators include Drs. Ahmad Itani, Manos Maragakis, Gökhan Pekcan, and David Sanders. The project manager is Kelly Lyttle. A number of graduate students are working on this project including Nathan Harrison, Ebrahim Hormozaki, Michael Levi, Eric Monzon, Ahmad Saad, Chunli Wei, and Joseph Wieser. In addition, Dr. Arash Esmaili Zaghi (Post-doctoral Scholar) and Moustafa Al-Ani (Visiting Researcher) have also provided valuable assistance with the project. The authors also acknowledge the National Science Foundation for the use of NEES Shake Table Array at the University of Nevada, Reno under a Shared-Use Agreement with NEEScomm at Purdue University.

REFERENCES

AASHTO. (2012). AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 6th Edition. Washington, DC: AASHTO.

Kameda, H., Murono, Y., Maekawa, Y., and Sasaki, N. (1992). Dynamic Structure-Vehicle Interaction for Seismic Load Evaluation of Highway Bridges. *Proceedings of the Tenth World Conference on Earthquake Engineering*, Madrid, Spain, July 19-24, 4861-4866.

Kawashima, K., Unjoh, S., and Mukai, H. (1994). Research on Live Load Effect to Earthquake Response of Bridges (Part 1). PWRI Research Report No. 3316, 59 pp. (In Japanese)

Kawatani, M., Kim, C. W., and Yasui, K. (2007). Seismic Response of a Highway Bridge under Traffic Loadings. *Proceedings of Pacific Structural Steel Conference 2007: Steel Structures in Natural Hazards*, Wairakei, New Zealand, March 13-16, 183-188.

Kim, C. W., Kawatani, M., Konaka, S., and Kitaura, R. (2011). Seismic Responses of a Highway Viaduct Considering Vehicles of Design Live Load as Dynamic System during Moderate Earthquakes. *Structure and Infrastructure Engineering*, **7:7-8**, 523-534.

Otsuka, H., Unjoh, S., and Mukai, H. (1999). Research on Live Load Effect to Earthquake Response of Bridges (Part 2). PWRI Research Report No. 3355, 40 pp. (In Japanese)

Scott, M. H. (2010). Combined Seismic Plus Live Load Analysis of Highway Bridges, Report OTREC-RR-09-261. Portland: Oregon Transportation Research and Education Consortium.

Sugiyama, I., Kameda, H., Sasaki, N., and Kawakita, S. (1990). Dynamic Structure-Vehicle Interaction of Highway Bridges and Its Implication to Seismic Design. *Proceedings of the 6th U.S.-Japan Bridge Engineering Workshop*, Lake Tahoe, NV, May 7-8, 379-392.

CONVERSION TABLE

From	To	Multiply by
in	mm	25.4
ft	mm	304.8
kip	kN	4.45