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Effects of Live Load on Seismic Response of Bridges: A Preliminary Study

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Abstract: Although live load is well known to have a dynamic effect on bridge response in addition to its self-weight, the significance of these effects on seismic response is unclear. In addition, most bridge design specifications have few requirements concerning the inclusion of live load in their seismic design provisions. The main objective of this study is therefore to investigate and obtain insight into the effect of vehicle-bridge interaction during earthquake shaking. The study consists of both experimental and analytical investigations. This paper focuses on the experimental work, which includes shake table testing of a 2/5-scale model of a horizontally curved steel girder bridge loaded with a series of representative vehicles. Preliminary experimental results show that the presence of the live load had a clear beneficial effect on performance for small amplitude motions, but that this improvement diminished with increasing amplitude of shaking. Parameters used to measure performance include column displacements, abutment shear forces, abutment uplift, and concrete spalling.

Keywords: Live load, seismic response, large-scale, multiple shake tables, horizontally curved bridges.

Introduction

Even though 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, whilst 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 [1]. 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 as a mass damper, although not necessarily tuned, 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 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.

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 [2-8]. 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.

Bridge Model and Representative Vehicle

An experimental study was conducted on the NEES Shake Table Array in the Large-Scale Structures Laboratory at University of Nevada, Reno. This study took advantage of a FHWA project that studies the effect of curvature on seismic response of bridges. Therefore, the experimental portion was carried out on a three-span, horizontally curved bridge model. This 2/5-scale model has a steel plate girder superstructure, single-column bent reinforced concrete substructures, and seat-type abutments. Overall dimensions are shown in Table 1. The bridge model has a total length of 145 ft [44.196 m], a total

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width of 12 ft [3.658 m], and subtended angle of 104° as shown in Figures 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 Figure 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.

The prototype bridge was designed for a site in Seismic Zone 3 [1] with a 1,000-year spectral acceleration at 1.0 second (S_1) of 0.41 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.

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 1. Bridge Geometry Summary

Parameter	Prototype	Model
Total Length	362'-6"	145'-0"
Span Lengths	105'-0", 152'-6", 105'-0"	42'-0", 61'-0", 42'-0"
Radius at Centerline	200'-0"	80'-0"
Subtended Angle	104° (1.8 rad)	104° (1.8 rad)
Total Width	30'-0"	12'-0"
Girder Spacing	11'-3"	4'-6"
Total Superstructure Depth	6'-6.125"	2'-7.25"
Column Height	19'-2'	7'-8"
Column Diameter	5'-0"	2'-0"

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 Figures 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.

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 white noise excitations were run to characterize the system's dynamic properties.

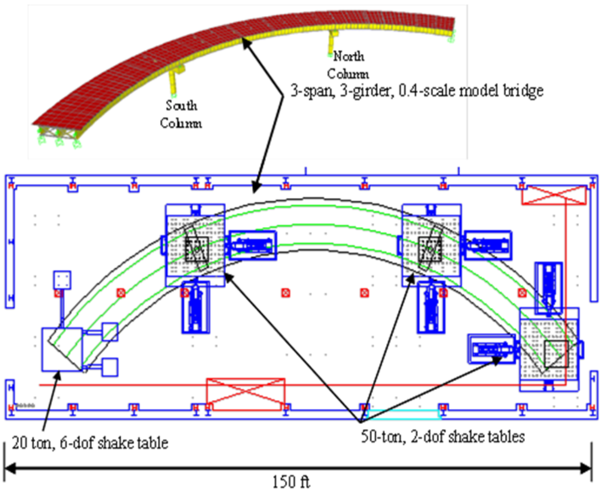


Figure 1. Bridge Model and Layout in Large-Scale Structures Laboratory

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

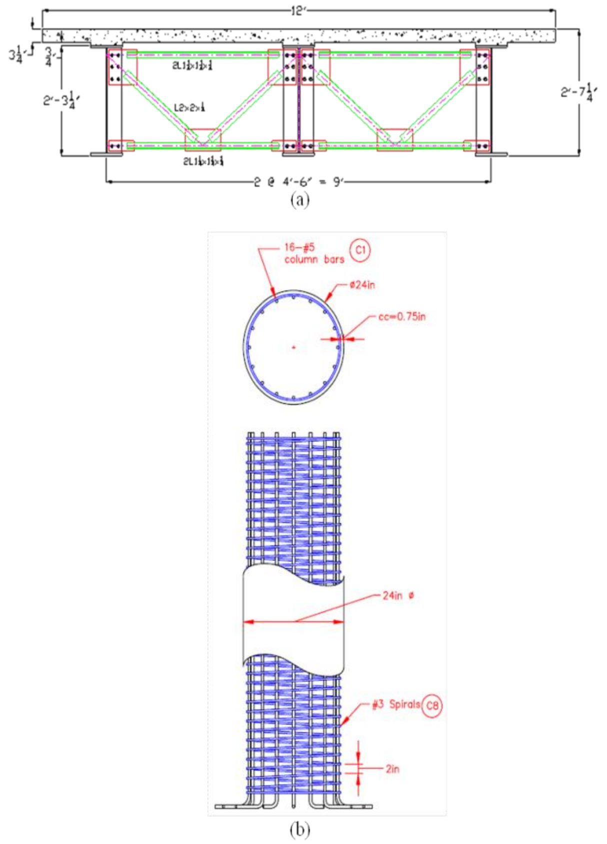


Figure 3. Typical Superstructure and Column Details

Table 2. Ford F-250 Dimensions and Weight Ratings

Parameter	Value
Overall Length	247 in
Overall Width	68 in
Overall Height	80 in
Wheel Base Length	156 in
Ground Clearance	7.9 in
Curb Weight	6.7 kip
Gross Vehicle Weight Rating	10 kip
Max Allowable Payload	2.3 kip

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

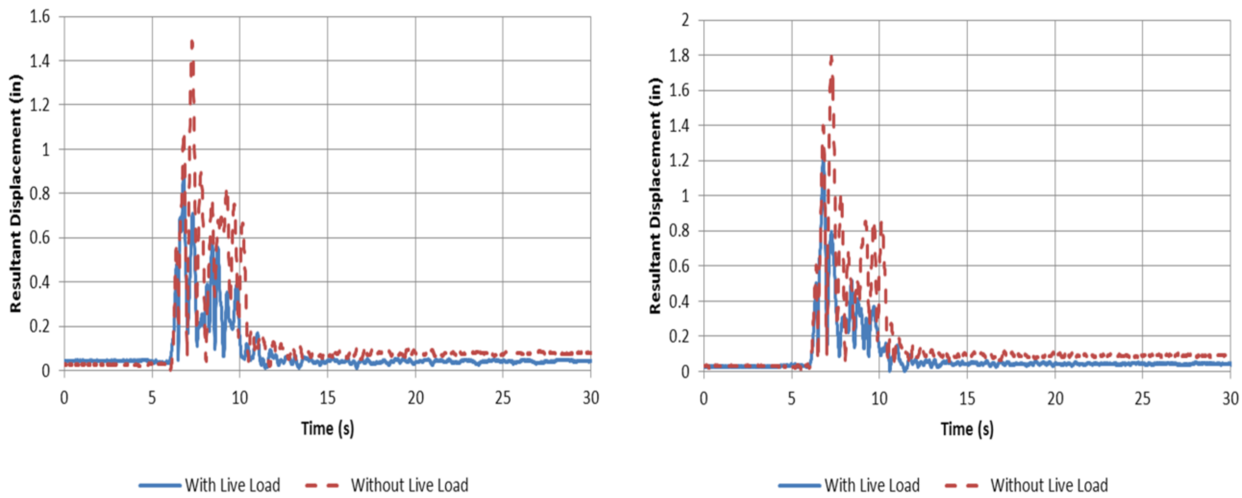
Figure 5. Wide-Angle View of Experimental Model in the Laboratory (apparent vertical curvature is due to distortion by camera lens)

Experimental Results

One of the parameters that may be used to quantify the effect of live load is the column displacement. Figures 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.

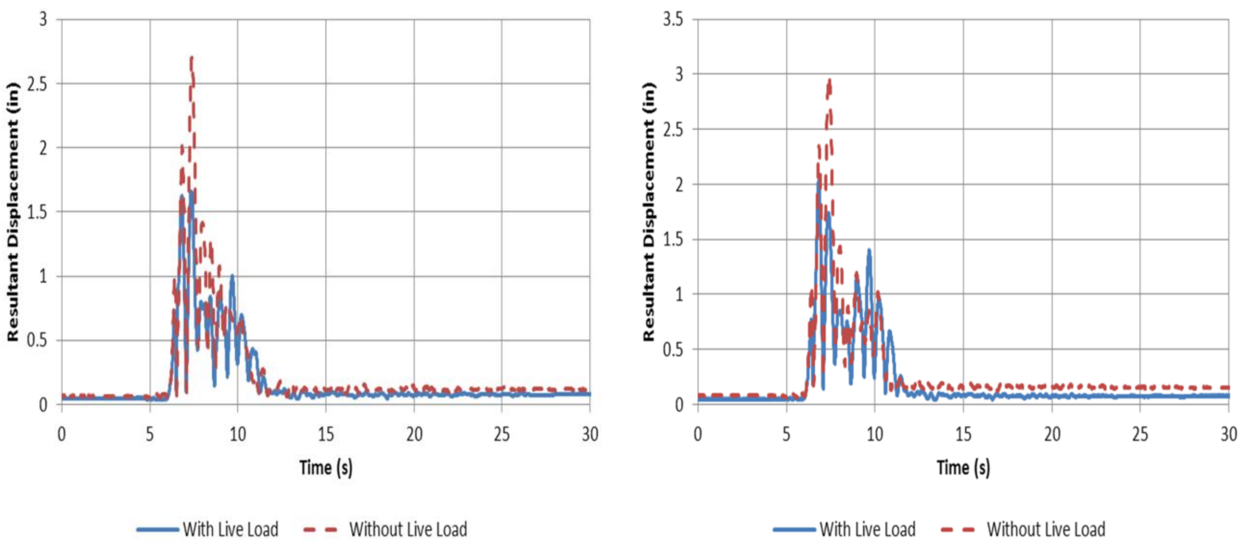
On the other hand, observations from the higher amplitude runs, after the shear keys at the abutments had failed, show maximum displacements that are almost the same in the two cases. Figures 8 and 9 show the displacements in the north and south columns with and without live load after 250% and 300% 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. Figure 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. On the other hand, this phenomenon is not that apparent on the north column, as depicted in Figure 11. 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. Figures 12 and 13 show the north and south abutments uplift measured at the bottom of the outer and inner bays during the 350% DE runs with and without live load. 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 abutment uplift when live load is present, is about the same as the abutment uplift without live load.



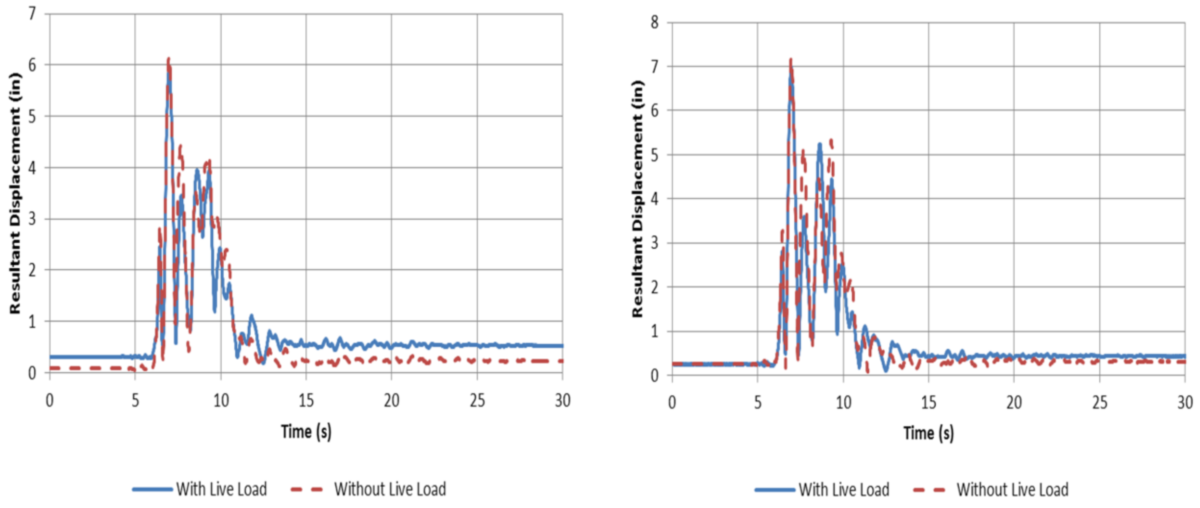
(a) (b)

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



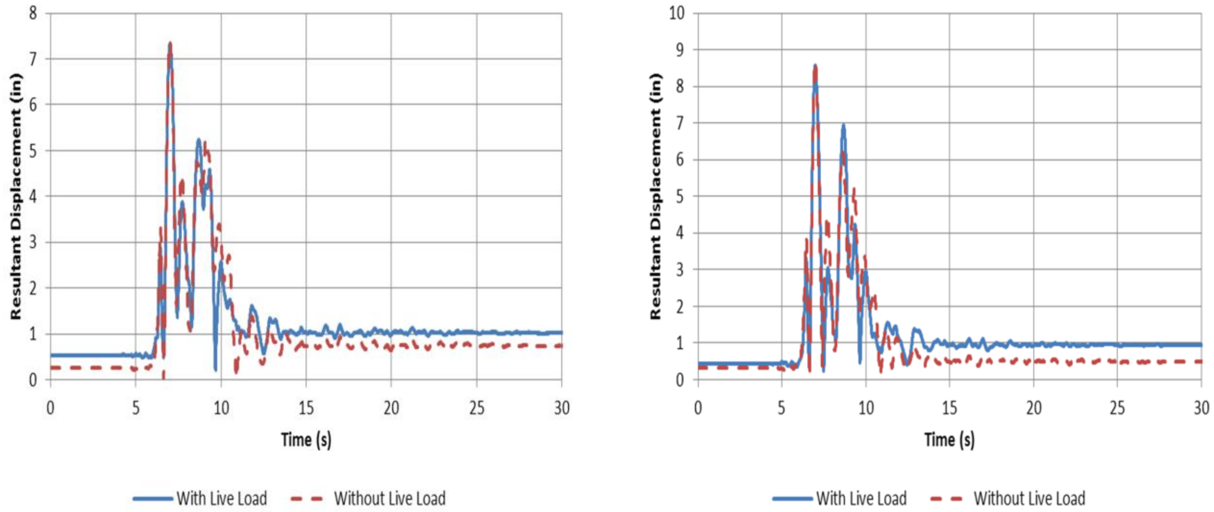
(a) (b)

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



(a) (b)

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



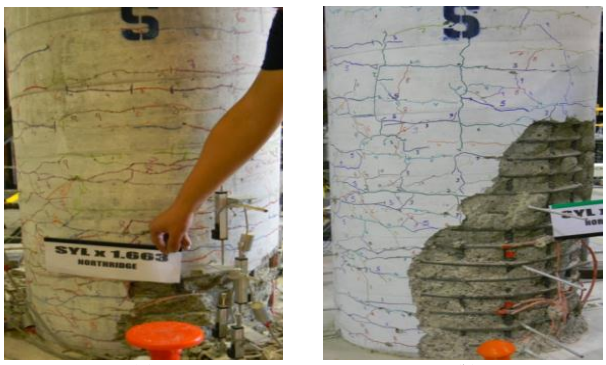
(a) (b)

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

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

Run	With Live Load (kip)		Without Live Load (kip)	
	NA	SA	NA	SA
10% DE	0.76	3.06	5.48	4.30
20% DE	3.29	7.27	11.70	8.51
50% DE	16.54	17.32	29.38	18.00
75% DE	22.05	22.65	*33.51*	*1 252.40*
100% DE	*23.31*	*23.31*	N/A	N/A

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

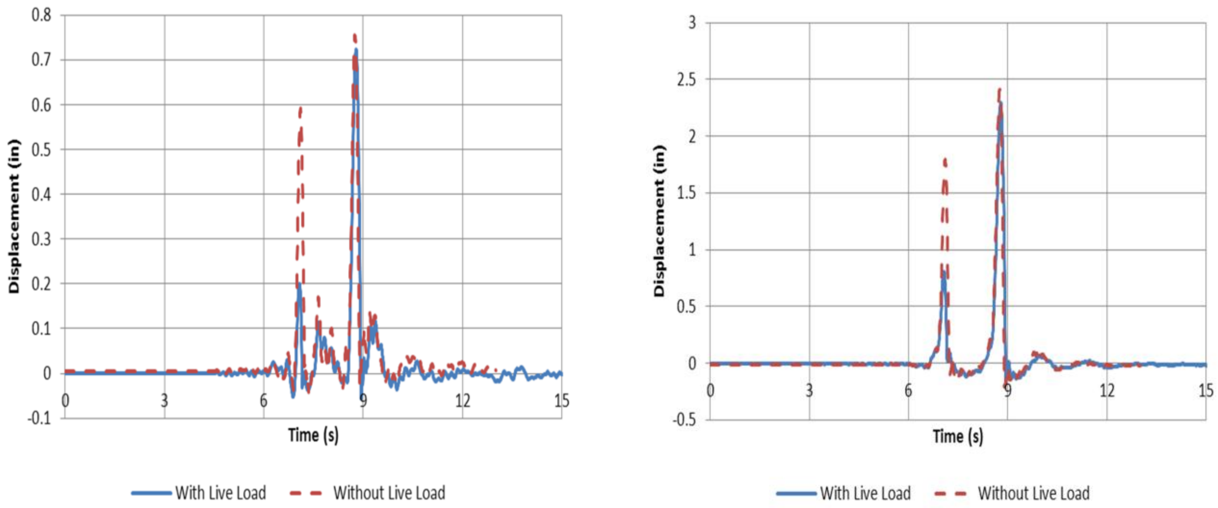


(a) (b)

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

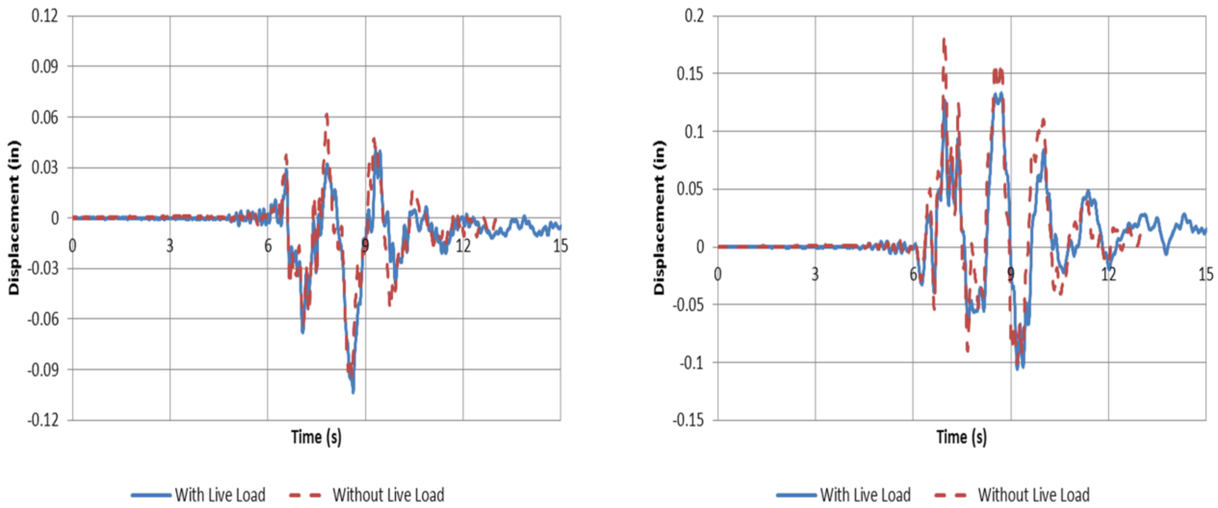
(a) (b)

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

Summary

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 due to 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. Also, 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.

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Conversion Table

From	To	Multiply by
in	mm	25.4
ft	mm	304.8
lb	N	4.45