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Dynamic structure-vehicle interaction for seismic load evaluation of highway bridges

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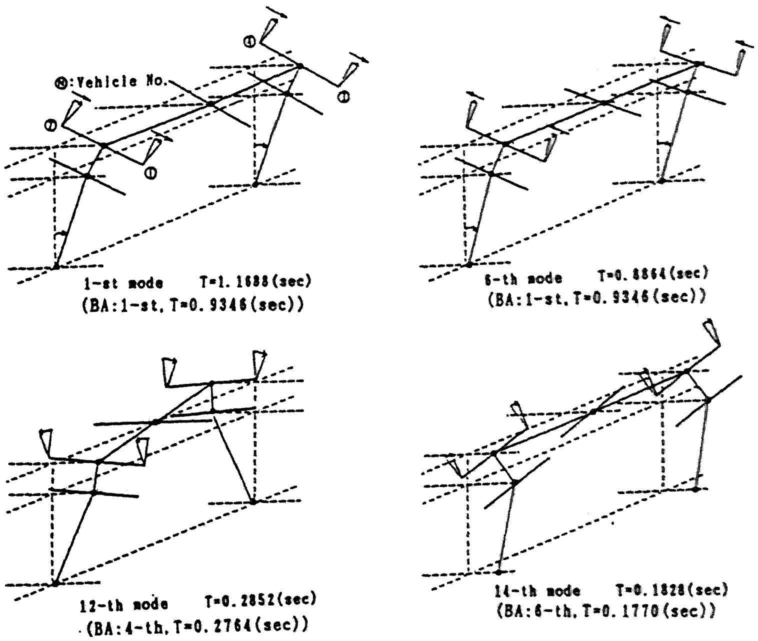
ABSTRACT: Seismic load of highway bridges are discussed with an emphasis on the dynamic effects of heavy vehicles at rest on the bridge decks in traffic congestion. Structure-vehicle interaction is formulated and an analytical model is developed. Detailed observation is made on dynamic properties of heavy vehicles. The developed model is verified by comparison with full-scale vibration tests. On this basis, frequency response and earthquake response time history of bridges loaded with heavy trucks are obtained, and their implication to the seismic load of bridge structures is discussed.

1 INTORODUCTION

The dynamic effects of heavy vehicles on the seismic load of highway bridges are an area that has not been studied enough to allow regorous engineering judgment. Whereas current seismic design codes for standard highway bridges all over the world ignore the effects of live loads, this subject is being a controversial issue among code developing engineers in Japan. This is particularly an important problem of access-controled urban highways that are subject to frequent occurrence of traffic congestions, take place in which case it is observed that heavy tracks are packed on bridge decks with small spacings at a very low speed or sometimes with a complete stop.

Heavy vehicles at rest on bridge decks can either increase the seismic load of bridge structures by their large mass or decrease it by dynamic effects similar to



1-st mode T=1.1688(sec)
(BA:1-st, T=0.9346(sec))

6-th mode T=0.8864(sec)
(BA:1-st, T=0.9346(sec))

12-th mode T=0.2852(sec)
(BA:4-th, T=0.2764(sec))

14-th mode T=0.1828(sec)
(BA:6-th, T=0.1770(sec))

Figure 1. Lateral vibration modes of bridge-vehicle interaction (cases of simple vehicle model).

that of tuned-mass dampers. These phenomena should be formulated in a proper way, and information should be provided so that engineering judgment can be made both on quantitative and qualitative basis.

This works is aimed at contributing to answering the question as a stated above. A realistic analytical model is developed by incorporating structure-vehicle interactions, which is verified by full-scale testing of an existing bridge. On this basis, frequency response and time history earthquake response are obtained, and discussion is made in relation to the seismic loads of bridges under the action of vehicles.

2 STRUCTURE-VEHICLE INTERATION MODEL

Important vibration modes of a typical analytical model of structure-vehicle interaction are shown in Figure 1. Its main structure has been modeled from a span of a four-lane highway viaduct composed of a sequence of simple composite plate girders supported by single-column type bents. This figure shows a case with four vehicles indicated by small arrows. In this particular case, the vehicles are represented by simple inverted pendula, but in the following analysis more detailed MDOF vehicle models are developed.

It is important to compare the 1-st mode and the 6-th mode in Figure 1. In both of these two modes, the bridge structure vibrates in an identical mode which is the 1-st mode of the bridge-alone system. These two modes are different in that the structure and vehicles are deformed in-phase in the first mode whereas they are deformed out-of-phase in the 6-th mode. How these modes are defined depending on the structural parameters and how they are activated in earthquake responses proves to be a major factor governing the dynamic effects of heavy vehicles on the seismic load of bridge structures.

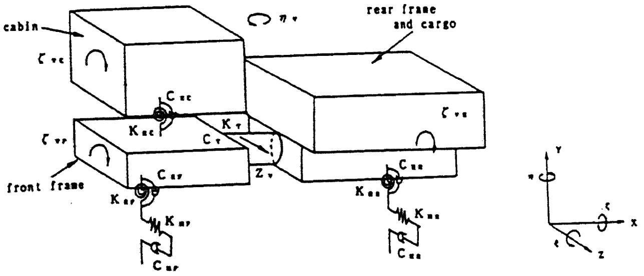
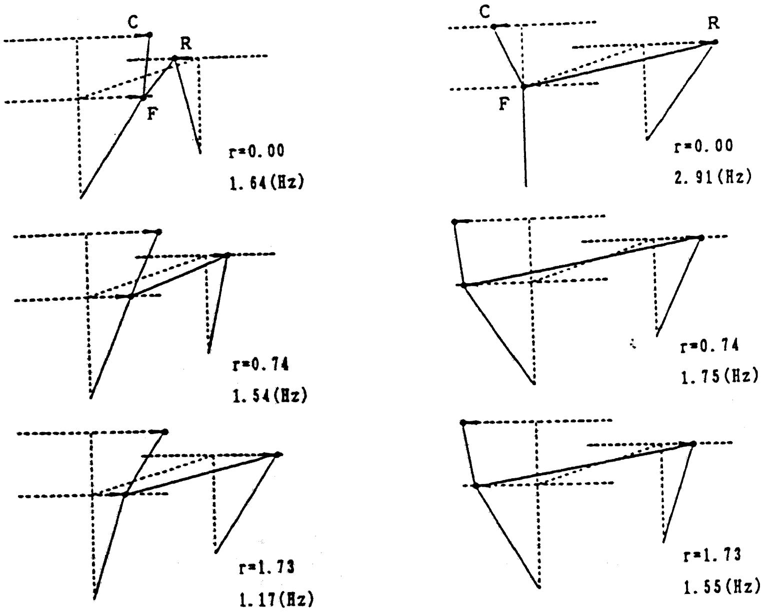


Figure 2. Five-degree-of-freedom vehicle model.



(a) 1-st mode (b) 2-nd mode

Figure 3. Lateral vibration modes of vehicles (5 D.O.F. model: cargo truck).

3 DETAILED VEHICLE MODEL

From a full-scale vibration tests described in the next chapter, it was found that participation of complex vibration modes of vehicles should not be neglected when the fundamental period of the bridge is close to that of vehicles. For this reason, the five-degree-of-freedom vehicle model shown in Figure 2 has been employed. The first and second modes shown in Figure 3 are the main contributors to the interaction with the bridge structure. In this figure, the "loading ratio" r is defined as

$$r = (\text{actual cargo weight/legal limit of cargo weight})$$

Models for two types of heavy vehicles were used in this study; i.e., dump trucks and long-range cargo trucks. Their natural frequencies are shown in Figure 4. The measured values of their damping ratio as shown in Figure 5 were employed.

4 FULL-SCALE TESTING

Full-scale testing was performed by using an existing rampway structure for Hanshin (Osaka-Kobe) Expressways loaded with two long-range cargo trucks (Sugiyama et al.1990). A hydraulic exciter was fixed

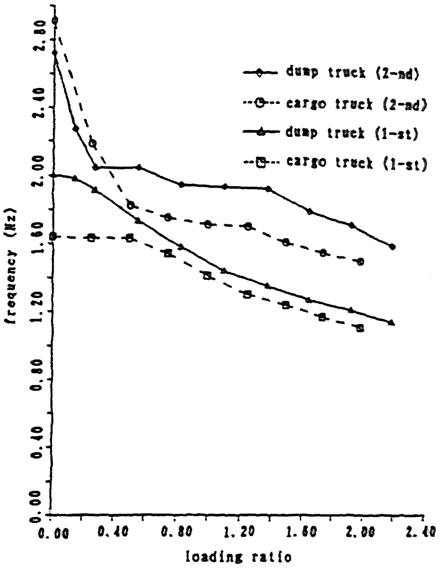


Figure 4. Natural frequencies of vehicle models.

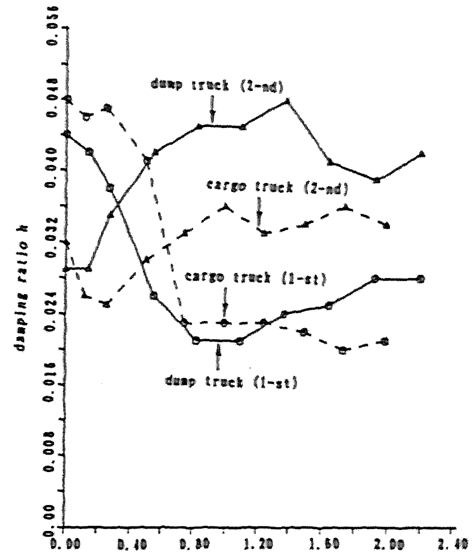


Figure 5. Measured damping ratio of vehicles.

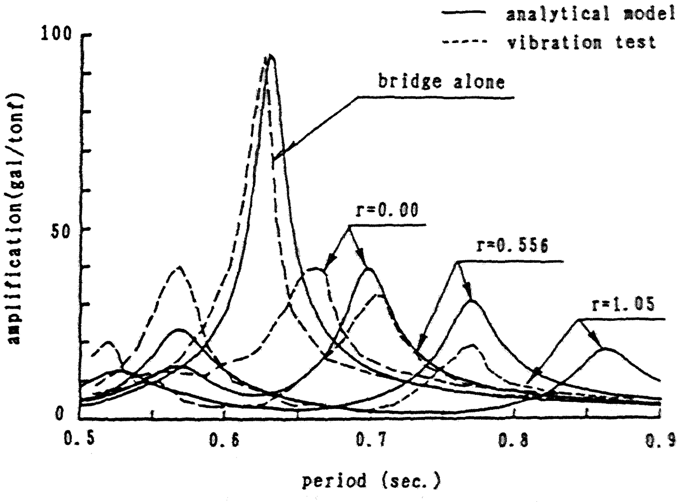


Figure 6. Frequency response of bridge-vehicle model compared with full-scale test results.

to the surface of the bridge deck, and harmonic excitation was applied to the bridge-vehicle system. A structure model was developed which fits to the particular case of tested bridge and also to the excitation from the surface of the deck.

Figure 6 shows the frequency response of the bridge girder from the test results, dashed line, and the corresponding analytical results, solid line. They agree in a sense and do not in another.

A remarkable aspect of this result is that all cases of the loading ratio *r* result in the resonance peaks much lower than the case of the bridge-alone. The vehicles affect the bridge response like tuned-mass dampers. When a simple inverted pendulum vehicle model was used, it was unable to explain this reduction for the case of $r = 0.0$ (Sugiyama et at.1990). But by using the five-degree-of-freedom vehicle model developed herein, all these test results are well understood.

Considerable difference in the resonance period between the test results and the analytical results remains unresolved. It has been found that the resonance periods are considerably sensitive to the stiffness of the vehicle model. This can be a reason for this difference, but more studies are needed regarding this point.

Figure 6 may give an impression that the presence of heavy vehicles will always reduce the bridge response. However, these results were obtained for a special case where the fundamental period of the bridge-alone, 0.633 sec, happened to be in the range of the first mode frequency of the cargo trucks shown in Figure 4. In other cases, we have different results as discussed in the following chapters.

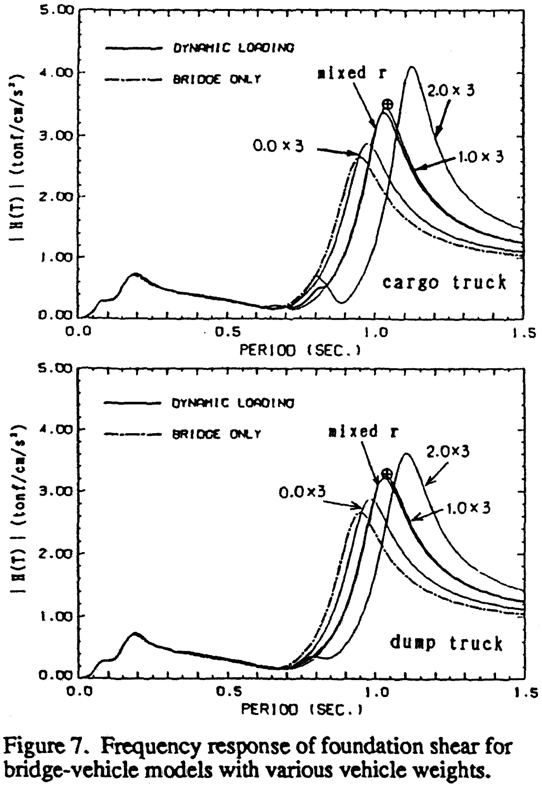


Figure 7. Frequency response of foundation shear for bridge-vehicle models with various vehicle weights.

5 FREQUENCY RESPONSE FOR BASE EXCITATION

Frequency response has been obtained for the case of transverse harmonic excitation from the pier foundations for the bridge type shown in Figure 1. Response curves for the foundation shear are plotted in Figure 7. these figures are for cases with three vehicles with different values of loading ratio, indicted as "mixed *r*." The response curves are also shown for cases with three vehicles with equal values of *r*, indicated by "(the value of *r*) × 3." The marks indicated by ⊕ are located at the points giving the arithmetic mean of the height of the resonance peaks and that of the resonance periods. It may be observed that the marks nearly coincide with the peaks for the "mixed r" case.

All frequency response curves in Figure 7 have resonance peaks that are higher than that for the bridge-alone, which is quite different from what we observed in Figure 6. The bridge structure used in Figure 7 has its fundamental period of 0.935 sec which is in the region of longer periods than the vehicles; i.e., a combination of soft bridge and stiff vehicles. In this case, the mass of vehicles tend to behave as an increase in the weight of the girder, and consequently gives higher resonance peaks at longer periods than the bridge-alone.

To see that response reduction can take place under this model, the fundamental period of the bridge was reduced to 0.64 sec, which corresponds to the case of

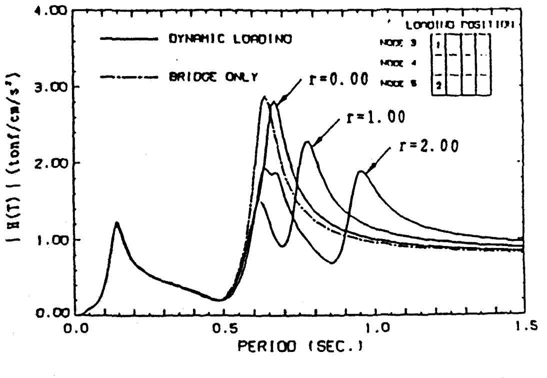


Figure 8. Frequency response of foundation shear for bridge-vehicle models for a case of short-period bridges.

the tested bridge in Figure 6. The frequency response curves for this case are shown in Figure 8. Considerable reduction of the resonance peaks is observed.

To illustrate the variation of the resonance peaks, their height have been normalized to that of bridge-alone case, and shown in a contour graph in Figure 9. The domain below the thick solid line is where the ratio exceeds 1.0. This separation is made nearly along the first mode frequency of the vehicle, below which we have soft bridges and stiff vehicles. This result is consistent with the above argument made regarding Figures 7 and 8.

6 EARTHQUAKE RESPONSE

Time history of earthquake responses of the structure-vehicle systems dealt with in the previous chapter was obtained. Two sets of artificial earthquake motions, one with narrow bands and the other with wide bands, generated with various values of the predominant period T_g (Kameda and Kita (1981) were used.

Figure 10 shows the time histories. (a) shows a case where the bridge and the vehicles undergo in-phase motions, and the response of the bridge with vehicles grows larger than the case of the bridge-alone. In contrast, (b) is a case where the bridge motion and the vehicle motion are out-of-phase, in which case the

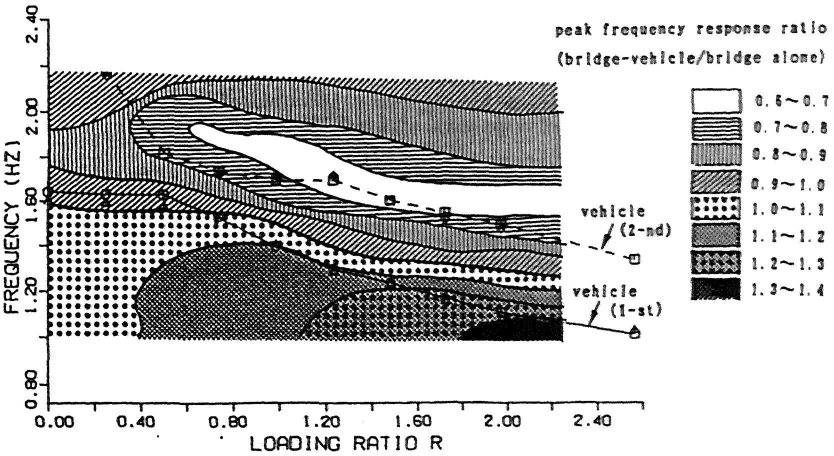
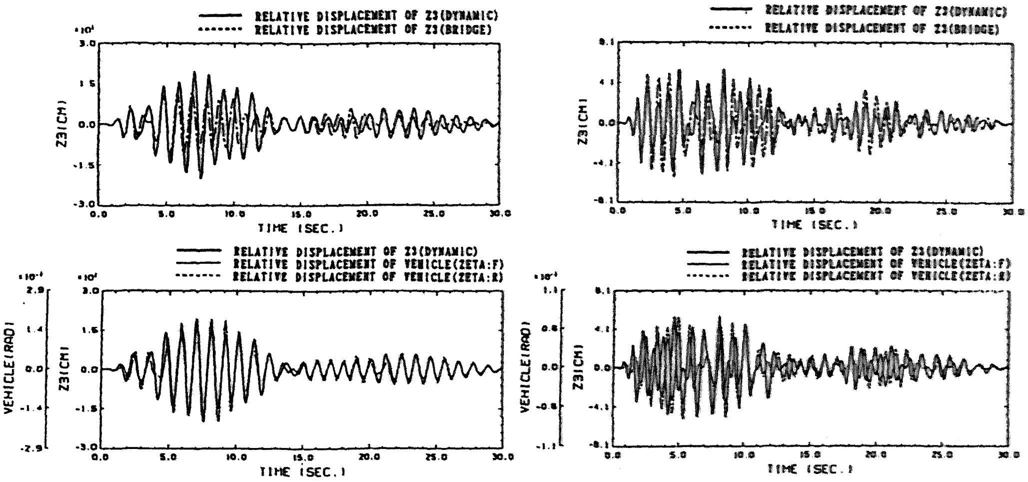


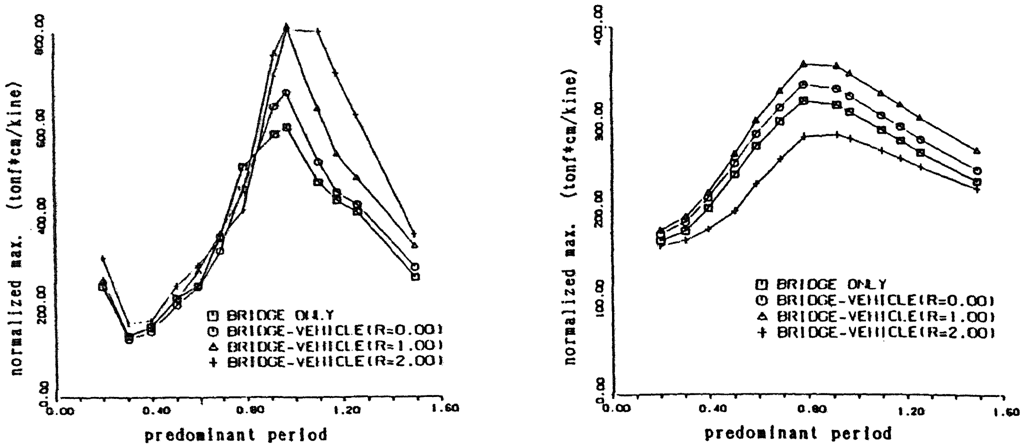
Figure 9. Contour graph of peak frequency response of bridge-vehicle models normalized to the case of bridge alone.



(a) T_g=1.14 (sec)

(b) T_g=0.70 (sec)

Figure 10. Time history of bridge-vehicle model response to narrow-band artificial earthquake motion (two cargo trucks with r=2.00).



(a) narrow band input

(b) wide band input

Figure 11. Peak response of bending moment at lower end of pier for artificial earthquake motion (two cargo trucks with r=2.00).

presence of vehicles makes the bridge response smaller than the case of the bridge-alone.

Figure 11 shows plots of the peak response of the bending moment at lower end of the pier. In the case of the narrow-band input motions, we observe the results to be similar to the frequency response curved shown in Figure 7. The case of wide-band input motions, on the other hand, show somewhat different trends. The curves are smoother than the case of narrow-band input. Moreover, the case of $r = 2.0$ consistently gives response values smaller than the case of the bridge-alone. Whereas narrow-band input motions activate single selected modes, wide-band input motions tend to activate more than one vibration modes simultaneously. The results in Figure 11(b) are understood as a consequence of combined mode participations.

7 CONCLUSIONS

The major conclusions derived from this study may be summarized as follows:

1. The problem of structure-vehicle interaction of highway bridges under seismic excitation was formulated and an analytical model was developed.
2. Full-scale testing was performed to verify the analytical model.
3. From the frequency response analysis, the details of the dynamic interaction between the bridge structures and vehicles were discussed, from which it was made clear that relation between their fundamental frequencies is a major factor to govern the bridge seismic response under the action of vehicles.
4. From the time history analysis of earthquake response, it was demonstrated that the structure-vehicle interaction tend to increase the bridge response when their in-phase mode is activated and decreases it when the out-of-phase mode is activated. The effects of band width of the input motions were also made clear.

REFERENCES

Kameda, H. & H. Kita 1989. Earthquake response of highway bridges under dynamic effects of vehicle loading, *Annuals, Disaster Prevention Res. Inst., Kyoto Univ.*, 32-B-2: 49-82, (in Japanese).

Sugiyama, I., H. Kameda, N. Sasaki & S. Kawakita 1990. *Proc. 6th U.S.-Japan Bridge Engineering Workshop:* 379-392.