



Wave-passage effect on the seismic response of long bridges

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ABSTRACT: The effect of the spatial variation of seismic ground motions on the response of bridge structures has been of concern for decades. However most of these studies focus on the elastic behaviour of the structure. In this paper, a study of the inelastic response of an irregular bridge with piers of different heights subjected to travelling seismic motion is presented. The time-history responses of bridge models were produced using a 3D computer program. Parametric analyses using different propagation velocities of the seismic waves and different natural accelerograms were conducted to assess the wave-passage effect on the response of a bridge. The response of the bridge to asynchronous input motions at different pier bases consists of two components: a dynamic component induced by the inertia forces and a so-called pseudo-static component, due to the difference between the adjacent support displacements. It was observed that the propagation velocity of the seismic waves had a significant effect on the response of the bridge. When the travelling wave velocity was low, the response was dominated by the pseudo-static component. As the travelling wave velocity increased, the pseudo-static component reduced considerably and the dynamic component increased rapidly. When the travelling wave velocity was high, the response was dominated by the dynamic component.

1 INTRODUCTION

Extended structures such as long bridges can be subjected to very differing motions along their length due to the spatial variability of the input seismic motion. The effect of the spatial variation of the seismic ground motions on the response of bridge structures has been of concern for a number of decades. The spatial variation of seismic ground motion may be thought of as the result of the combination of three different phenomena: (1) the so-called wave-passage effect, which is the difference in the arrival times of seismic waves at different locations along the bridge; (2) the incoherence effect, resulting from reflections and refraction of waves through the soil during their propagation, as well as the difference in the manner of superposition of waves arriving from an extended source at various locations; (3) the local effect, due to the difference in local soil conditions at each location. Because of insufficient knowledge of the mechanisms underlying the spatial variability of the motion, it is often assumed that the spatial variability is attributed only to the wave-passage effect.

The influences of the wave-passage effect on the responses of bridges have been investigated by several researchers, for example, Bogdanoff et al. (1965), Vanmarcke (1977), Werner et al. (1979), Somaini (1987), Bayrak (1996), Monti et al. (1996), and Tzanetos et al. (1998). However, most of these studies focus on the elastic behaviour of the structure with very few of those studies dealing with the nonlinear responses of bridges. Monti et al. (1996) studied the nonlinear responses of symmetric bridges with symmetric boundary conditions. They have found that the travelling wave effect on the response consists essentially in a reduction of the dynamic part due to the incomplete synchronisation of the excitation for the range of apparent velocities in excess of 500 m/s. Tzanetos et al. (1998) studied the wave-passage effect on the inelastic responses of symmetric bridge structures with symmetric or asymmetric boundary conditions. They concluded that special consideration should be

given to bridges with asymmetric boundary conditions since they may respond in higher modes more readily than symmetric structures and the demands are likely to increase on the bridges. In this paper, a study of the inelastic seismic responses of a bridge with an irregular configuration was carried out in the transverse direction by using the program RUAUMOKO-3D (Carr 2001). Parametric analyses using different propagation velocities of the seismic waves and different natural accelerograms were conducted to assess the wave-passage effect on the response of the bridge. The response of the bridge to asynchronous input motions at different pier bases consists of two components (Clough and Penzien 1993), a dynamic component induced by the inertia forces and a so-called pseudo-static component, due to the difference between the adjacent support displacements. It was observed that the propagation velocity of seismic waves had a significant effect on the response of the bridge. When the travelling wave velocity was lower than 150 ~ 250m/s, the response was dominated by the pseudo-static component. As the travelling wave velocity increased, the pseudo-static component reduced considerably and the dynamic component increased rapidly. When the travelling wave velocity was higher than 150 ~ 250m/s, the response was dominated by the dynamic component.

2 DESCRIPTION OF THE PROTOTYPE BRIDGE

The prototype bridge was a nine-span straight bridge continuous between abutments with a total length of 344m. The spans between the piers were 40m long while the end spans between the abutments and the adjacent pier are 32m long as shown in Figure 1. The deck was comprised of twin prestressed concrete box girders. These were supported through sliding bearings on reinforced concrete piers comprised of a single 1.5m diameter circular column with a 2.5m deep ‘hammerhead’ cap beam. The bearings permitted longitudinal movement of the girder relative to the cap beam. The lateral movement of the girder was restrained by shear keys. Abutment 1 was constructed monolithically with the end diaphragm in the girder, and abutment 10 supported the girder-end through sliding bearings with freedom of movement longitudinally, transversely and rotationally. The heights of piers 2 to 9 were 6m, 8m, 5m, 5m, 5m, 11m, 11m and 5m respectively. The longitudinal reinforcement consisted of 48D32 bars in pairs running the entire height of the pier. The transverse reinforcement consisted of D12 bars at 75mm centres for the bottom 20% of the pier height and 140mm centres for the remainder of the height. The piers were supported by a 4.5m by 4.5m by 1.5m deep pile caps and four 1m-diameter piles. The design concrete cylinder strength was 35 MPa for the piers, and 45MPa for the prestressed girder. Reinforcement nominal yield strength was 430MPa. The site had uniform soil conditions, consisting of cohesionless soils.

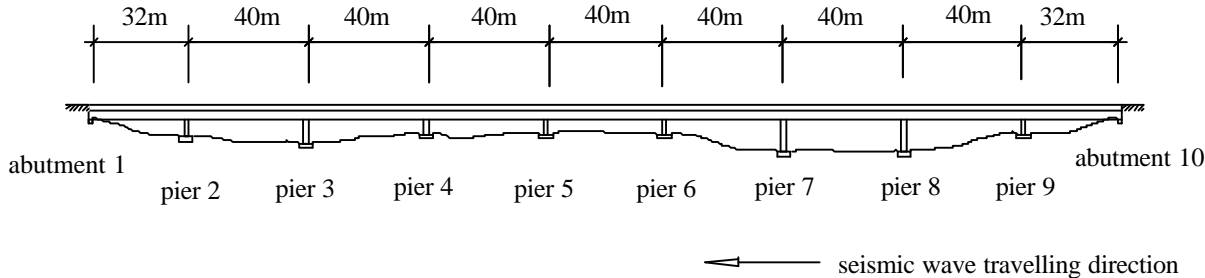


Figure 1. Bridge Elevation

3 STRUCTURAL MODELING AND SEISMIC INPUT

The girder was represented by 3D linear elastic beam members placed at the geometric centroid of the cross section, having the following characteristics: section area = 6.93m², moment of inertia I_{max}=86.25m⁴, I_{min}=3.16m⁴, torsional moment of inertia J=6.97m⁴, member length = 8m, and mass = 200KN/m..

The piers were modeled as 3D concrete beam-column members using a one-component model (Carr 2001), which idealised a reinforced concrete beam or column member as a perfectly elastic mass-less line element with non-linear rotational springs at the two ends to model the potential plastic hinges. The bi-linear hysteresis rule was employed for the hinge spring to represent the elastic and inelastic behaviour of the member. The plastic hinge length $L = D$ ($D =$ the diameter of the piers) was assumed. The effective member properties, which reflected the extent of concrete cracking and reinforcement yielding, were taken as follows: effective moment of inertia $I_e=0.124m^4$, effective torsional moment of inertia $J_e=0.15m^4$, and effective shear area $A_{ve}=0.88m^2$.

The effective stiffness EI_e was determined from section moment-curvature analyses as (Priestley 1996)

$$EI_e = \frac{M_y}{\Phi_y} \quad (1)$$

where M_y and Φ_y represent the ideal yield moment and curvature for a bilinear moment-curvature approximation. The result of the section moment-curvature analysis of the piers under a static axial load of 8000KN is shown in Figure 2. The effective stiffness reduction in shear was considered proportional to the effective stiffness reduction in flexure (Priestley 1996). The torsional moment of inertia was multiplied by a factor of 0.3 to get the effective torsional moment of inertia for these prototype bridge piers after Singh and Fenves (1994). Piers were assumed to be fixed at the top of pile cap level.

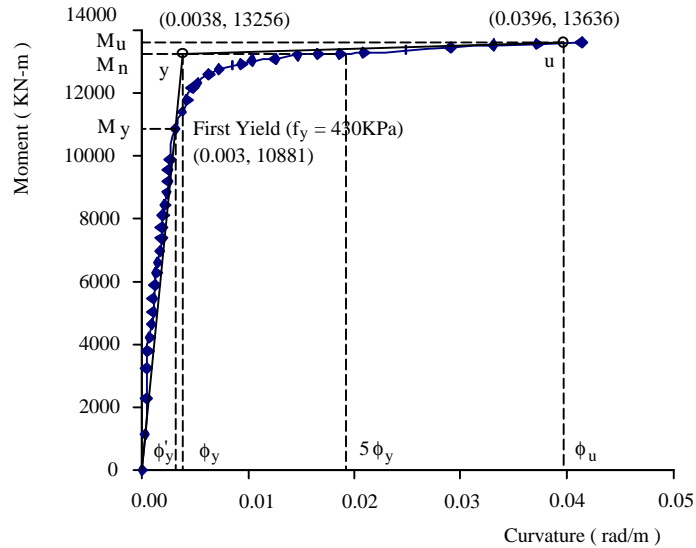


Figure 2 Moment-Curvature Relationship for Pier Section

Sliding bearings were modeled by 3D spring elements. The spring stiffness in the longitudinal direction was based on the idealized shearing deformation of the bearing pads given by $G_{elast}A/h$ where $G_{elast}=1.0MPa$ was the assumed shear modulus for the elastomer, $h=50mm$ was the height of the bearing pads, and $A=0.34m^2$ was the plan area of the bearing pads. The yield force was equal to the constant vertical reaction of the bearing pads from gravity loads multiplied by the dynamic friction coefficient ($= 0.12$). For the bearings installed on the piers, the spring stiffness in the transverse direction was set as large as possible because the shear key will provide a rigid restraint in this direction. The bearings on the abutment were given the same stiffness value in both the longitudinal and transverse directions.

Masses were lumped on the ends of each member. The Rayleigh damping model was used to model the damping exhibited by the structure in which the fractions of critical damping were assumed to be five percent in modes 1 and 2.

It was assumed that the seismic wave travelled along the longitudinal direction of bridge from abutment 10 to abutment 1. The El Centro 1940 earthquake NS component and the Northridge 1994 earthquake NS component (recorded at the ground floor of the 6-story county hospital, Sylmar) were used. These were applied at abutment 10 and there were time lags between the ground motions at other pier supports. The time lag was calculated from $\Delta t_i = r_i/v_s$, where r_i was the separation between abutment 10 and pier i and v_s was the wave propagation velocity that was selected to cover a wide range from soft soil ($v_s= 100, 125, 150, 200\text{m/s}$), to stiff soil ($v_s= 300, 400, 500\text{m/s}$), and rock ($v_s=1000, 1500, 2000\text{m/s}$). In the description that follows, the term “asynchronous” refers to the travelling wave situation whereas the term “synchronous” refers to the case where all the piers undergo the same, simultaneous motion.

4 EARTHQUAKE RESPONSES OF THE BRIDGE

The parameters investigated for the inelastic response were the maximum pier drift, the maximum shear force in the pier, and the maximum section curvature ratio of the pier. The maximum pier drift (column top to column base) provided a measure of the flexural response of the pier. The maximum shear force in the pier was an indication of the shear demand at the pier. The maximum section curvature ratio was the maximum section curvature of the pier reached in the analysis divided by yield section curvature of the pier and it indicated the curvature ductility demand on the pier.

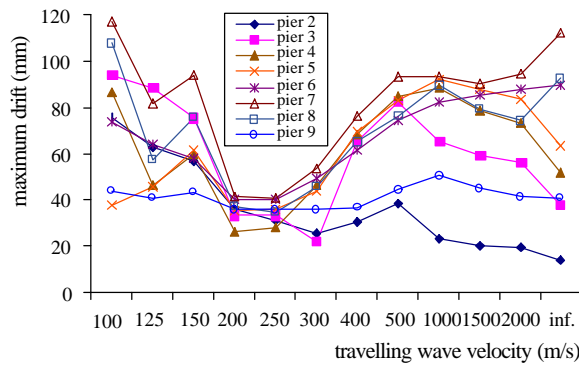


Figure 3. The variations of the maximum pier drifts with travelling wave velocity under El Centro 1940 earthquake record.

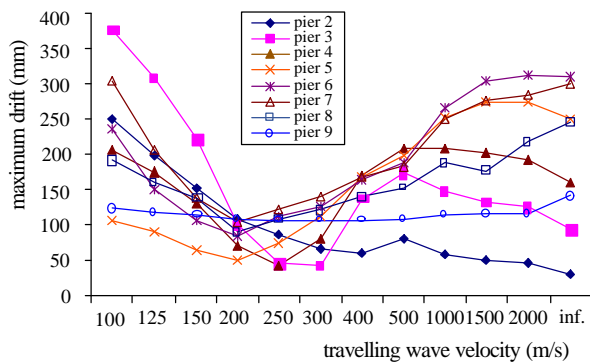


Figure 4. The variations of the maximum pier drifts with travelling wave velocity under Northridge 1994 earthquake record.

The maximum pier drift response to the North-South components of the El Centro (May 18, 1940) earthquake and Northridge (January 17, 1994) earthquake for the travelling motion cases are shown in Figures 3 and 4. Generally, the maximum pier drifts decreased as the wave travelling velocity increased from 100m/s to between 200 and 250m/s where they had minimum responses; after that the pier drifts increased with the increase of the travelling wave velocity, although there were some local variations. Pier 2 was an exception; its maximum pier drift decreased as the travelling wave velocity increased over the range of travelling wave velocities used in the analyses.

The response of the bridge to asynchronous input motions at different pier bases consists of two components (Clough and Penzien 1993), a dynamic component induced by the inertia forces and a so-called pseudo-static component, due to the difference between the adjacent support displacements. As the travelling wave velocity increased, the pseudo-static component decreased considerably because the differences between the adjacent support displacements reduced sharply with the increase of travelling wave velocity. Figure 5 shows the variations of the maximum differential displacements between the adjacent pier bases with the travelling wave velocity under the NS component of El Centro earthquake. It showed that when the travelling wave velocity was 100 m/s, the maximum differential displacement between two adjacent supports (span = 40m) was 107.4 mm. When the travelling wave velocity was increased to 300m/s and 500m/s, the maximum differential displacement dropped to 40.3% and 26.4% of the value when the travelling wave velocity was 100 m/s, respectively. For the synchronous input case (velocity = infinity) the maximum differential displacements between two adjacent supports were nil.

In order to investigate the variation of the dynamic component of the response, several synchronous analyses were carried out. The time-histories used in these synchronous cases were the averages of the ten travelling wave time-histories that had been applied to the ten bridge supports in each of the previous asynchronous cases. The responses of these synchronous cases were considered to represent the dynamic components of the response of asynchronous cases. The responses of these synchronous cases to the NS component of the El Centro 1940 earthquake record are shown in Figure 6. It can be seen that when the time-histories used are combined from the asynchronous cases with travelling wave velocities higher than 1000m/s, the maximum pier drifts are almost the same as the synchronous case under the natural earthquake record (compare Figure 3 with Figure 6). For the other cases in which the time-histories were combined from the asynchronous cases with travelling velocities lower than 1000 m/s, the maximum pier drifts decreased as the travelling velocity was decreased. When the travelling wave velocity was decreased to 300/m/s, the maximum pier drifts dropped to 20% of the value of the synchronous case under the natural earthquake record.

A clear trend was observed when comparing the maximum pier drifts in the travelling wave cases (Figures 3), the maximum pier drifts in the synchronous cases (velocity = infinity) in which the combined earthquake records were used (Figure 6) and the maximum differential displacements between two adjacent pier bases (Figure 5). When the travelling wave velocity was lower than 200 ~ 250m/s, the response of the bridge structure to asynchronous motion was dominated by the pseudo-static component and the dynamic component was much smaller than the pseudo-static component. When the travelling wave velocity greater than 200 ~ 250m/s, the pseudo-static component was reduced considerably and the dynamic component increased rapidly, and the response of the structure to asynchronous motion became more dependent on its dynamic component.

The drift of pier 2 was dominated by the pseudo-static effect because pier 2 was the closest pier to the fixed end of the girder where the structural stiffness was very large and its dynamic component was small. This probably explained why the maximum drifts of pier 2 decreased as the travelling wave velocity increased in Figures 3 and 4.

The total response of the bridge structure to asynchronous motion also could be affected by the fact that the frequency spectrum of the excitation to which the whole bridge was subjected was different from that of the synchronous case. The frequency spectrum of the excitation changed with the travelling wave velocity, although the time-histories of the seismic motion did not change in shape at the various supports on the ground surface. This could be seen from the Fourier spectra of the pier drift of pier 9 (Figure 7), which showed that the relative significance of the first mode changed with the travelling wave velocity. For the synchronous case ($v = \text{infinity}$), the response mainly depended on the first mode, but for the asynchronous cases (travelling wave velocity $v < \text{infinity}$) the responses depended not only on the first mode but also on the higher modes as indicated by other researchers (Tzanetos et al. 1998). Any local variation of pier drift could be attributed to the change in the frequency spectrum of the asynchronous motion consistent with the travelling wave velocity.

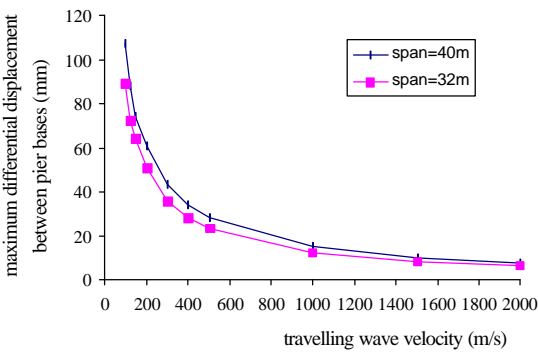


Figure 5. The variations of the maximum differential displacement between pier bases with travelling wave velocity under El Centro 1940 earthquake record.

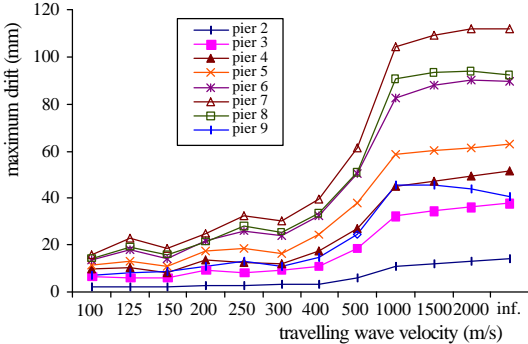


Figure 6. The variations of the maximum pier drifts with travelling wave velocity for the synchronous cases under the averaged time-histories.

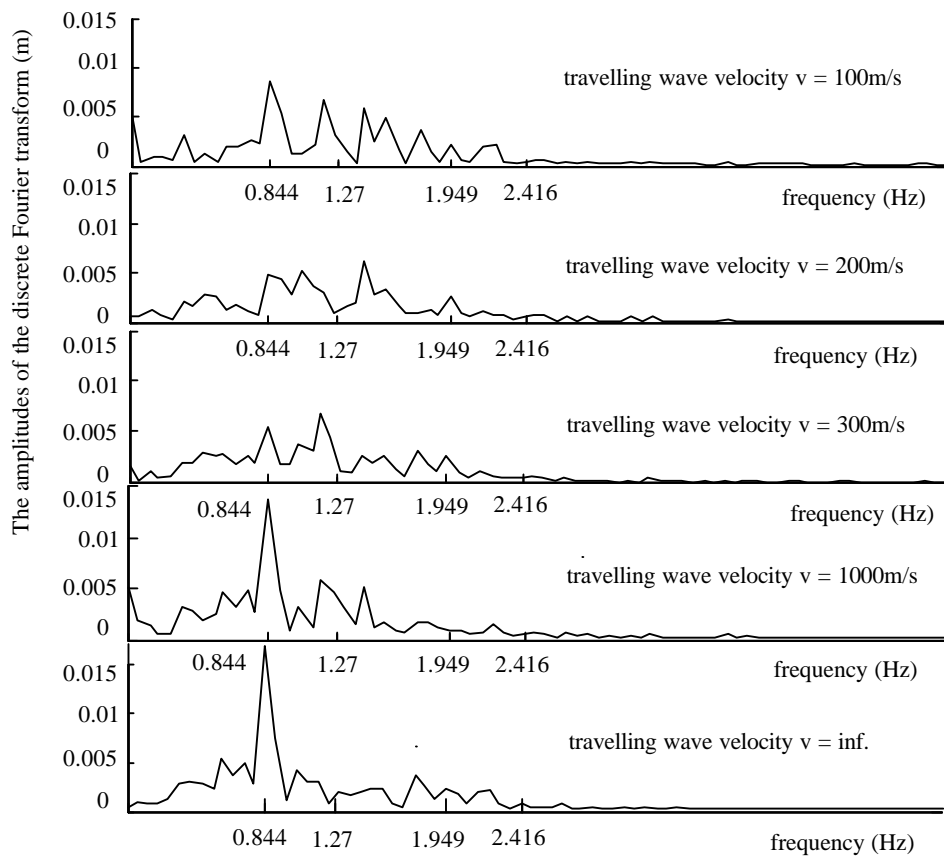


Figure 7. Fourier spectra of the displacement at the top of pier 9.

The combination of these three aspects of the pseudo-static effect, the dynamic effect, and the change in the magnitude of the frequency spectrum of excitation with the travelling wave velocity could make the responses of the bridge to travelling waves greater than those to the synchronous motion.

The maximum shear forces in the piers and the maximum section curvature ratios of the piers varied with the travelling wave velocity in approximately the same way as the pier drifts for both the earthquake records that were used. Figures 8 and 9 show the results from the responses to the NS component of the El Centro 1940 earthquake record for the asynchronous motions.

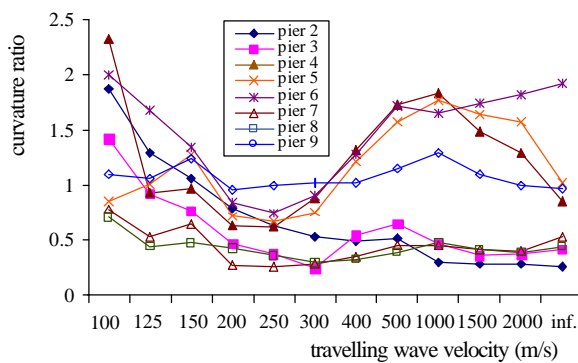


Figure 8. The variations of the maximum curvature ratios with travelling wave velocity under El Centro 1940 earthquake record.

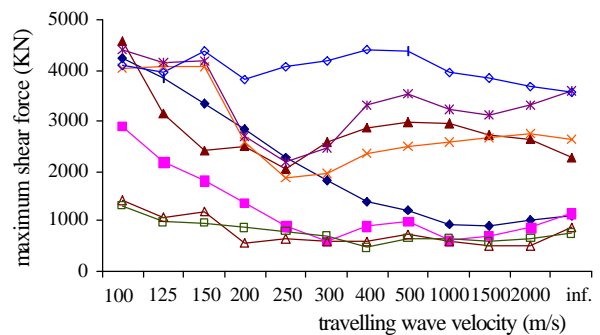


Figure 9. The variations of the maximum shear forces in piers with travelling wave velocity under El Centro 1940 earthquake record.

5 EARTHQUAKE RESPONSES OF BRIDGES WITH DIFFERENT CONFIGURATIONS

In order to investigate the wave-passage effect on the responses of bridges with different configurations, three additional bridge models were employed, in which the structural details were the same as the prototype bridge except for the boundary conditions and the pier heights. Model 2 was identical with the original model of Figure 1 except that abutment 1 was also supported on sliding bearings like those at abutment 10. In Model 3, all piers had the same height of 13.5 m. Abutments 1 and 10 were supported through sliding bearings. Model 4 had all piers of 13.5 m height with abutment 1 being fixed while abutment 10 was supported through sliding bearings as in the original model. The maximum pier drifts of the three models to the NS component of El Centro 1940 earthquake are shown in Figures 10, 11 and 12. Similar trends could be observed in the variation of maximum pier drift with the propagation velocity of seismic motions.

Comparing Figure 3 with Figure 10, it could be seen that with the modified Abutment 1 the maximum drift of pier 2 in Model 2 increased a lot when the travelling wave velocity was higher than 250m/s, but the maximum drifts of other piers were not significantly affected. The increase of the maximum drift of pier 2 was due to the increase of its dynamic component, i.e. the increase of the transverse stiffness of Abutment 1. Hence the variation of the maximum drift of pier 2 with the travelling wave velocity also followed the same form as other piers.

Model 3 represented a symmetric structure. From the responses of Model 3 (in Figure 11), it could be seen that the shape of maximum pier drifts was symmetric and similar to the shape of the first mode of vibration of the bridge in the synchronous case ($v = \text{infinity}$), but the shape of maximum pier drifts were asymmetric and tended to become flat in travelling motion cases as indicated by other researchers (Monti et al. 1996, Tzanetos et al. 1998). This suggested that the first mode of vibration of bridge dominated the response in the synchronous case, and higher modes were excited by travelling waves in asynchronous cases.

The responses of Model 4 showed again that the variation of maximum pier drift with the travelling wave velocity followed the same way as those mentioned before for other bridge models even though the bridge configurations were quite different between bridge models.

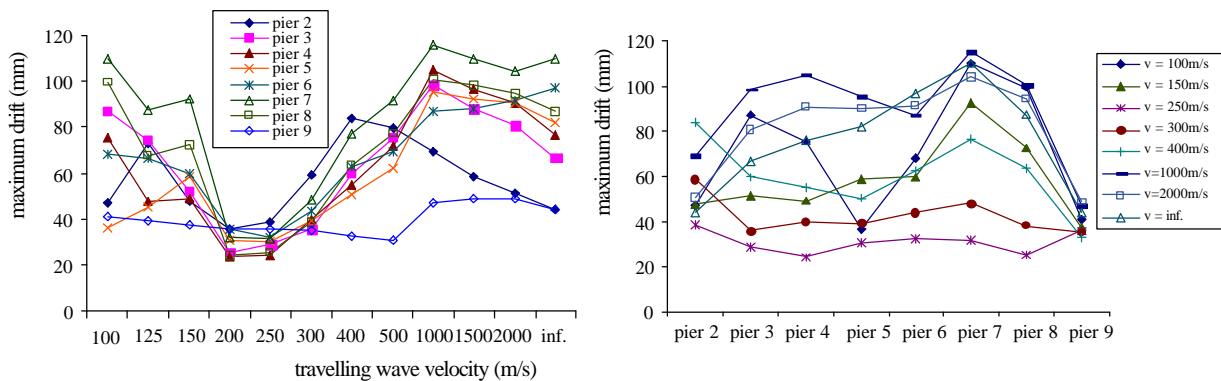


Figure 10 The responses of model 2

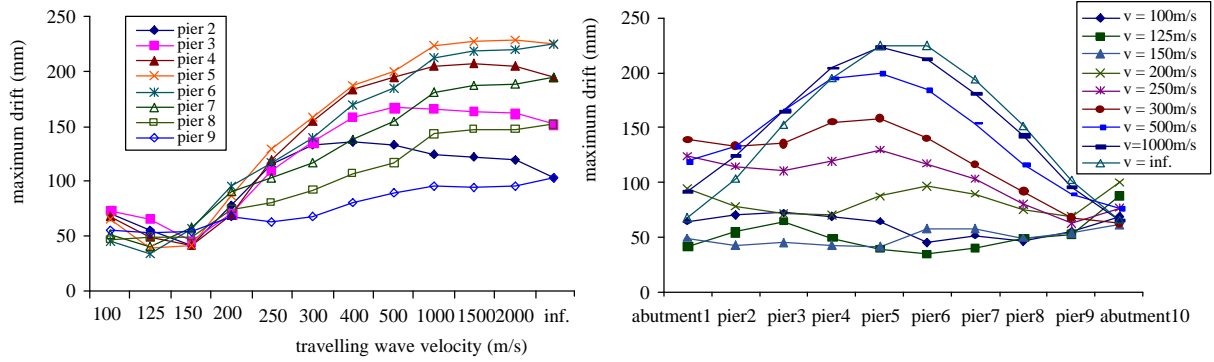


Figure 11. The responses of model 3.

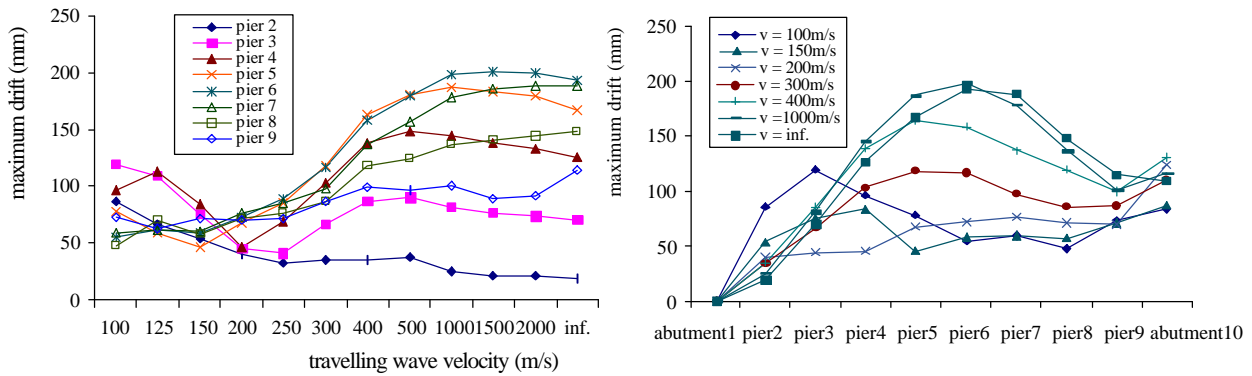


Figure 12. The responses of model 4.

6 CONCLUSIONS

- 1) From response analyses conducted in this paper, it was found that when the travelling wave velocity was lower than 150 ~ 250m/s, the response was dominated by the pseudo-static component; as the travelling wave velocity increased, the pseudo-static component reduced considerably and dynamic component increased quickly; and when the travelling wave velocity was higher than 150 ~ 250m/s, the response was more dominated by the dynamic component.
- 2) The first mode contribution to the response varied with the travelling wave velocity. The higher modes were excited by asynchronous motions.
- 3) Several cases were observed where the travelling motion response was more critical than the synchronous response.

7 ACKNOWLEDGEMENT

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