

# EFFECTS OF SPATIAL VARIATION OF SEISMIC INPUTS ON BRIDGE LONGITUDINAL RESPONSE

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# SUMMARY

Unseating of bridge spans is a common type of seismic failure in bridges. Many effects contribute to this type of failure with the spatial variation of the seismic ground motions being one effect that may play an important role. This paper deals with the effects of the asynchronous seismic inputs on the relative movement of the bridge deck across joints in the longitudinal direction. The asynchronous seismic inputs were generated using the conditional simulation method with a natural earthquake record specified at one pier support. Two different bridge models and three different earthquake records were employed to carry out parameter studies. It was found that the relative displacements of the bridge deck across the joints were much greater in asynchronous motion cases than those in the synchronous motion case. For the type of bridge used in this study, the differences of these relative displacements between the synchronous case and the asynchronous cases were mainly caused by the wave-passage effect of asynchronous seismic inputs.

# INTRODUCTION

Unseating of bridge spans have been observed in most major earthquakes and it is a common type of seismic failure in bridges. The bridge girders move off their supports because the relative movement of the spans in the longitudinal direction exceeds the seating widths. The structural differences of sections separated by movement joints and the local soil conditions can increase the relative movements across the movement joints. It also has been observed that skewed spans develop larger displacements than right spans. However, asynchronous ground motions may play a more important role in this type of failure.

In this paper, the seismic responses of bridges with movement joints subjected to asynchronous input motions are presented in order to gain an insight into the effect of asynchronous inputs on the relative movements across joints. Time-history analyses of long bridges were carried out using seismic motions acting in the longitudinal direction of the bridge, and traveling along the bridge in the same direction. The asynchronous seismic inputs were generated using the conditional simulation method with a natural earthquake record specified at one pier support.

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Two different bridge models and three different earthquake records were employed to carry out the parameter studies. The two response parameters investigated are the maximum relative displacement of the bridge deck across the joints and the maximum relative displacement between the girder end and the top of the abutment. If these displacements are large enough and seats with sufficient width or joint restrainers are not provided, these displacements may result in unseating of the girder and collapse.

It was found that the relative displacements of the bridge deck across the movement joints and the relative displacements between the girder end and the top of the abutments were much greater in asynchronous motion cases than those in the synchronous motion case. This could be one important reason why many bridges have collapsed during past earthquakes because of inadequate seating widths designed using the synchronous motion. For the type of bridge used in this study, the differences of these relative displacements between the synchronous case and the asynchronous cases were mainly caused by the phase shifts between the vibrations of the different segments of the bridge separated by the movement joints.

## DESCRIPTION OF THE PROTOTYPE BRIDGE

As shown in Figure 1, the prototype bridge was a nine-span straight bridge with a total length of 344m. The spans between the piers were 40m while the end spans between the abutments and the adjacent pier were 32m long. There were two movement joints in spans 3-4 and 7-8 at 7.5m from the nearest bent centerline. A typical movement joint is shown in Figure 2, but in this study the joints had no restrainers and large enough seat widths and initial openings were provided to ensure that the two parts of the joints were free to move without collapse due to unseating of the girder or collision.

The deck was a twin box prestressed concrete girder and was supported on a single circular pier through sliding bearings and a cap beam. The bearings permitted the longitudinal movement of the girder relative to the cap beam and transverse 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 end of the girder on sliding bearings with freedom of movement longitudinally, transversely and rotationally. The circular piers were of reinforced concrete and 1.5m-diameter. A 2.5m deep cap beam was monolithically connected to the top of each pier that had free heights of 6m, 8m, 5m, 5m, 5m, 11m, 11m and 5m for piers 2 to 9, respectively. The longitudinal pier reinforcement consisted of 48-32mm deformed bars in pairs running the entire height of the pier. The transverse reinforcement consisted of 12mm deformed bars at 75mm centers for the bottom 20% of the pier height and 140mm centers for the remainder of the height. Each pier was supported by a 4.5m by 4.5m by 1.5m deep pile cap and four 1m-diameter piles. The design concrete cylinder strength was 35 MPa for the piers, and 45MPa for the prestressed girder and the nominal yield strength of the reinforcement was 430MPa. The site had uniform soil conditions, consisting of cohesionless soils.



Figure 1 Bridge plan



#### STRUCTURAL MODELLING AND SEISMIC INPUT

The girder was represented by three dimensional (3D) linear elastic beam members (see Figure 3) placed at the geometric centroid of the girder cross section, having the following characteristics:section area =  $6.93m^2$ ; moment of inertias,  $I_{max}=86.25m^4$  and  $I_{min}=3.16m^4$ ;

torsional moment of inertia  $J=6.97m^4$ ; member length = 8m; weight = 200KN/m.

The piers were modeled as 3D concrete beam-column members using a one-component model [1], which idealized a reinforced concrete beam or column member as a perfectly elastic massless line element with non-linear rotational springs at the two ends to model the potential plastic hinges. The bi-linear hysteretic rule was employed for the hinge spring to represent the elastic and inelastic behavior 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  $J_e=0.15m^4$ ; effective shear area  $A_{ve}=0.88m^2$  [2].



Figure 3 General layout of finite element model of bridge

The effective flexural rigidity  $EI_e$  (where E = elastic modulus and  $I_e = effective$  second moment of area) was determined from section moment-curvature analyses from Priestley [3] as

$$EI_e = \frac{M_y}{\Phi_y} \tag{1}$$

where  $M_y$  and  $\Phi_y$  represent the ideal yield moment and curvature for a bilinear moment-curvature approximation. The effective stiffness reduction in shear was considered proportional to the effective stiffness reduction in flexure [3]. 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 [4]. Piers were assumed to be fixed at ground level.

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  $G_{elast}=1.0$ MPa was the assumed shear modulus for the elastomer, h=50mm was the height of the bearing pads, and A=0.34m<sup>2</sup> 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 of 0.12. For the bearings installed on the piers, the spring stiffness in the transverse direction was set as large as possible because shear keys provided a rigid constraint in this direction. The bearings on the abutment 10 were given the same stiffness value in both the longitudinal and transverse directions.

The joint element model used in this study is shown in Figure 4. The joints were modeled with sets of slaved nodes that were rigidly constrained in a horizontal array of five nodes across the width of the superstructure. The nodes were located where the bearing pads were located and where the pounding could take place. Each set of five nodes represented one side of the joint and was connected to another set of five nodes via zero-length nonlinear spring elements. The bearing pads were modeled as elastic-perfectly plastic springs as mentioned before for sliding bearings.



Figure 4 Schematic of joint model

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.

For the asynchronous input motions, a natural earthquake record was specified at Abutment 10 and the conditionally simulated time-histories were used at pier supports and Abutment 1. It was assumed that the seismic input motions acted in the longitudinal direction of the bridge and propagated from Abutment 10 to Abutment 1 in the bridge longitudinal direction. The seismic time-histories were generated using the simple method for stochastic dispersion of earthquake waves [5]. In the following sections, the cases in which both the geometric incoherence effect and the wave-passage effect of the spatial variability of the ground motion were considered in the bridge seismic responses are referred to as wave dispersion cases. The other cases, in which only the wave-passage effect of the spatial variability of the seismic input motion was considered in the responses of the bridge models, are referred to as the wave passage cases. In this work, the dispersion introduced to the ground motion does not change the earthquake spectra. The East-West components of three natural earthquake records, the El Centro 1940, the Northridge 1994 and the Kobe 1995, were employed at Abutment 10 respectively as the specified earthquake motion.

In order to enable less structure-specific conclusions to be drawn, two bridge models with different configurations were analyzed. Table 1 lists the characteristics of the configurations of the two bridge models.

	The free heights of piers	Boundary conditions
Model 1	6m, 8m, 5m, 5m, 5m, 11m, 11m, 5m for piers 2 to 9	At abutment 1 the superstructure was completely fixed while at abutment 10 the superstructure was supported on the abutment structure through sliding bearings (vertical support only)
Model 2	11m for all piers	At abutment 1 and abutment 10 the superstructure was supported on the abutment structure through sliding bearings (vertical support only)

 Table 1 The description of bridge models - see Figure 1

## THE WAVE PASSAGE CASES

## Model 1 responses

The change in gap opening for Model 1 when subjected to the East-West component of the El Centro 1940 earthquake record are presented in Figure 5 where a positive displacement indicates opening of a joint and a negative value indicates closing. The positive maximum relative displacement of the bridge deck across the movement joints and the positive maximum relative displacement between the girder end and the top of the Abutment 10 increased with the decrease in the traveling wave velocity although there are some local variations. These positive maximum relative displacements in the wave passage cases reached 4 to 20 times those in the synchronous case (velocity = infinity). It can be seen that the wave-passage effect on the responses was significant.

In the asynchronous input cases, the relative displacement of the bridge deck across all the movement joints consist of two parts: the first is the dynamic component due to the inertia effects arising from the difference between the vibrations of the two frames separated by the movement joint, and the second is the pseudo-static component caused by the time delay between the vibrations of the separated frames.



Figure 5 The gap displacement responses of Model 1 to El Centro 1940 earthquake record

The dynamic component is affected by the stiffness of the two frames separated by the movement joint, the yield strengths of the frames, the frictional restraint of sliding, the impact on closing the joints, and the characteristics of restrainers connecting the frames [3]. The dynamic component is also affected by the changes in the response time-histories of the bridge with the traveling wave velocity [2]. The vibration amplitudes of the separated frames generally decrease as the traveling wave velocity decreases because of the non-synchronism that does not allow the bridge to resonate at its fundamental frequency.

The pseudo-static component is dominated by the fact that the wave-passage effect makes the separated frames vibrate out of phase with each other. In any bridge structure with dimensions greater than the characteristic length of the ground motion, different parts of the foundations can be out of phase with each other due to an asynchronous seismic input. The wave-passage effect (i.e. the phase shift of the seismic arrivals at the different parts of the structure) is sufficient to generate incoherent motion on a scale length of the order of one hundred meters [6]. Therefore the pseudo-static component should play an important role in these relative displacements. The lower the traveling wave velocity, the longer the phase shifts between the vibrations of the two frames and hence the pseudo-static component will change with the traveling wave velocity.

From the response displacement time-histories of the bridge deck at the two sides of the movement joints, the girder end at Abutment 10 and Abutment 10 (Figures 6(a) to 6(d)), it can be seen that the phase shift between the vibrations of the two frames increased with the decrease in traveling wave velocity. The relative displacement of the bridge deck across all the movement joints were the differences between these two displacements, so they were not only dependent on the phase shift, but also on the shapes of these displacement time-histories that changed with the traveling wave velocity. As shown in Figure 5, the positive maximum relative displacement of the bridge deck across all the movement joints increased as the traveling wave velocity decreased and hence the pseudo-static component increased with the decrease in the traveling wave velocity. The pseudo-static was also noticed that the rates of increase of these relative displacements with the traveling wave velocity were slightly different because the shapes of the displacement time-histories changed with the traveling wave velocity due to the variations in the spectrum, and hence the bridge response, to the seismic input motion.



(b) In the asynchronous case with travening wave velocity of 2000m/s





(d) in the asynchronous case with traveling wave velocity of 200m/s

Figure 6 The response displacement time-histories of Model 1 to EL40EWC

The maximum relative displacement responses of the Model 1 to the East-West components of the Kobe 1995 earthquake record and the Northridge 1994 earthquake record for the wave passage cases are presented in Figures 7 and 8. The positive maximum relative displacement between the bridge girder end and the top of the Abutment 10 increased with the decrease in the traveling wave velocity, their trends being similar to that for the El Centro earthquake record. However, the responses of the relative displacements of the bridge deck across the movement joints are not similar to those for the El Centro earthquake record. However it still can be seen that some responses of the wave passage cases were more critical than that of the synchronous case especially the closing of joint 2.



travelling wave velocity (m/s)

Figure 7 The responses of Model 1 to the Kobe 1995 earthquake record



Figure 8 The responses of Model 1 to the Northridge 1994 earthquake record

Figures 7 and 8 show that the variation of the relative displacements of the bridge deck across the movement joints with the traveling wave velocity, were not great when the traveling wave velocity was greater than 1000m/s. These variations could be greater when the traveling wave velocity was less than 1000m/s and this fact indicates that the pseudo-static component played an important role in these cases. Although one observes larger phase shifts with lower the traveling wave velocity, the pseudo-static

components do not simply increase as the traveling wave velocity decreases because the values of the relative displacements also depend on the displacement time-histories of the bridge deck at the corresponding points. Consequently, the variations of the relative displacements of the bridge deck across the movement joints with the traveling wave velocity followed different trends for different seismic input excitations.

#### Model 2 responses

In order to obtain more general trends followed by the relative displacement of the bridge deck across the movement joint, the responses of Model 2 to the E-W components of the El Centro 1940 earthquake, the Kobe 1995 earthquake and the Northridge 1994 earthquake were determined [7]. The variations of the relative displacements of the bridge deck across the movement joints with the traveling wave velocity are shown in Figures 9, 10 and 11 for the cases using the three earthquakes.

Figure 9 shows that the maximum positive relative displacements of the bridge deck across all the joints increased with the decrease in traveling wave velocity in a similar manner to that for Model 1. However, the positive maximum relative displacement between the girder end and the top of Abutment 1 decreased as the traveling wave velocity decreased from infinity to 500 m/s, and when the traveling wave velocity was less than 500 m/s the displacement remained almost constant. This was because the difference between the displacements of the girder end and Abutment 1 decreased first as the phase shifts increased, and then increased as the phase shifts increased. Therefore, it could be concluded that the relative displacement response of the bridge deck across the movement joints and the relative displacement between the girder end and the top of the abutment of Model 2 also followed the same patterns as that for Model 1 in which the pseudo-static component played an important role. For the response of Model 2, it also is true that some relative displacements in the wave passage cases were larger than those in the synchronous case.

Figures 10 and 11 also showed increases in gap displacements similar to the El Centro responses, but most of the responses were less or approximately the same magnitudes as the synchronous cases. The greatest maxima are approximately 310mm at 100m/s for El Centro earthquake record, 230mm at 150m/s for Kobe and 380mm at 1000m/s for the Northridge earthquake, compared with associated values from the synchronous analyses of 70mm, 150mm and 350mm respectively.



Figure 9 The responses of Model 2 to the El Centro 1940 earthquake record



Figure 10 The responses of Model 2 to the Kobe 1995 earthquake



Figure 11 The responses of Model 2 to the Northridge 1994 earthquake

### THE WAVE DISPERSION CASES

The responses of the Model 1 subjected to the generated time-histories conditioned by the East-West component of the El Centro 1940 earthquake record at Abutment 10 with dispersion factors of 100, 10 and 1 are presented in Figures 12 to 14.

In the wave dispersion case, the variation of the ground motion at different bridge supports is not only due to the difference in the arrival time of seismic waves but also is attributed to the change in shape of the seismic motions. This means that the differential displacement between pier supports in these cases could be greater than that in the wave passage cases. The differential displacements between pier supports for the wave passage cases and the wave dispersion cases were completely different. However, the positive maximum relative displacements of the bridge deck across the movement joint 1 and the relative displacement between the girder end and the top of Abutment 10 in the wave dispersion cases and wave passage cases were similar to each other (compare Figure 5 with Figures 12 to 14). This indicates that the pseudo-static components of the relative displacements were still controlled by the phase shifts between the vibrations of the two frames separated by the movement joints, and the differential displacements

between pier supports had little effect on the relative displacements because the sliding bearings separated the bridge girder and the piers in the longitudinal direction. The differences in the relative displacements between the wave dispersion cases and the wave passage cases increased as the wave dispersion factor was reduced. These were caused by the changes of their dynamic components due to the changes of the input acceleration spectra resulting from the wave dispersion.

The maximum relative displacements of the bridge deck across movement joint 2 in the wave dispersion cases showed greater differences than those in the wave passage cases and the differences increased as the dispersion factor was reduced. This is because the displacement of the bridge deck at the left side of the movement joint 2 was almost the same as the displacement of Abutment 1. In Model 1, the Abutment 1 was fixed and its displacement changed with the change of the asynchronous input motion that was directly affected by the geometric effect of the variability of the seismic motion. The more the input motion changed, the smaller the dispersion factor.



Figure 12 The gap responses of Model 1 to EL40EWC in wave dispersion cases (d = 100)



Figure 13 The gap responses of Model 1 to EL40EWC in the wave dispersion cases (d = 10)



Figure 14 The gap responses of Model 1 to EL40EWC in the wave dispersion cases (d = 1)

The responses of Model 2 for the wave dispersion cases, subjected to the East-West component of the El Centro 1940 earthquake record, with dispersion factors of 100, 10 and 1 respectively showed similar trends to those of Model 1 in the corresponding cases [7].

## CONCLUSIONS

The longitudinal relative displacement of the bridge deck across the movement joints and the longitudinal relative displacement between the girder end and the top of the abutment were generally much greater in asynchronous motion cases than those in the synchronous motion case. This could be the main reason why many bridges collapsed in the past earthquakes because of inadequate seating widths.

In the case of asynchronous motion, when the traveling wave velocity was low, these relative displacements were dominated by the pseudo-static component, resulting from the phase shifts between the vibrations of the two adjacent separated frames.

The differential displacements between the pier supports had little effect on the investigated parameters even in the wave dispersion cases because the bridge deck was separated from the piers by sliding bearings.

#### ACKNOWLEDGEMENT

The doctoral scholarship provided by University of Canterbury to the first author is gratefully acknowledged.

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