11 WCEE
Copyright © 1996 Elsevier Science Ltd
Paper No. 523. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

EFFECT OF MULTIPLE SEISMIC INPUT ON THE RESPONSE OF LONG MULTI-SPAN BRIDGES

OGUZHAN BAYRAK

Department of Civil Engineering, University of Toronto,
35 St. George Street, M5S 1A4, Toronto, Ontario, Canada.

ABSTRACT

Rigorous seismic analyses of bridges and elevated viaducts require rational predictions of free-field ground motions at all the structure-ground interface points. Loss of coherency of seismic waves due to reflections and refractions in the non-homogenous half-space, spatially varying soil conditions and their influence on the frequency content and amplitude of the bedrock motion, decay of wave amplitudes with distance due to energy dissipation and filtration of high frequency components, geometric spreading in the half-space, and difference in the arrival times of the seismic waves at separate locations give rise to spatial variability of earthquake ground motions. The present study addresses itself to phenomena described above, and focuses upon the effect of the difference in the arrival times of the seismic waves at different stations along the length of a bridge. Since the attenuation effect is normally insignificant for typical sizes of structures, this effect os not considered here. Furthermore, in order to concentrate on the wave passage effect, the local soil conditions are considered to be uniform and hence the effects of wave reflection and refraction are assumed to be negligible. A computer program developed during the course of the current study is presented. Selected results from parametric studies performed on two bridges using three different earthquake records and six different wave propagation velocities are illustrated.

KEYWORDS

Bridges, wave passage effect, multiple seismic input, local soil conditions, dynamic analyses.

INTRODUCTION

"*...Complete seismic analysis of critical structures, particularly with long spans such as bridges and elevated viaducts, requires realistic predictions of free-field ground motions at all the interface points on the supporting foundation under design earthquake conditions...*" (Bolt 1991)
"*...Empirical studies with array recordings have shed light on the nature and magnitude of the wave passage and incoherence effects...*" (Kiureghian 1994).

High vulnerability of bridges to strong ground motions necessitates precision in the analysis and design procedures as suggested by Bolt (1991) and Kiureghian (1994). The objective of the present study is to evaluate the importance of the phenomena expressed by the aforementioned researchers. After the recognition of the importance of wave passage and incoherence effects, some researchers conducted analytical studies to evaluate the effect of these factors on the seismic response of bridge structures. Due to space limitations, only two of the studies are briefly summarized here. Dumanoglu et al. (1986) conducted an analytical study to evaluate the importance of the wave passage effect on the response of Bosphorus and Humber suspension bridges. The ground motion at one support point was assumed to propagate with finite speed and arrive at other support points without any change in shape and amplitude. A parametric study was conducted to evaluate the influence of wave propagation velocities on the seismic response of the two bridges. S16E component of San Fernando (Pacoima Dam) record was used for analysis purposes. It was concluded that the conventional method which is based on common ground motion assumption underestimated the displacements and internal forces often by appreciable amounts in comparison with those given by asynchronous motion at sensible input speeds. Kang and Wieland (1988) studied the seismic behavior of a four-span continuous girder railway bridge subjected to multiple support excitations. As well as performing a time history analysis in which conventional and asynchronized application of ground motion were both considered, researchers performed a response spectrum analysis on the bridge. Different combination rules for the superposition of modal maxima such as square-root-of-sum-squares, double sum, etc. were employed. With the application of asynchronized ground motion, considerably smaller absolute maximum dynamic responses (bending, shear, deflection) were predicted as opposed to conventional/uniform application of ground motion. It should be noted that this conclusion opposes with that of Dumanoglu et al. (1986). As the absolute maximum errors of different combination rules varied from 10.2% to 141.1%, for the bridge analyzed the response spectrum analysis results were far from being realistic. In other studies reviewed in the earlier stages of the ongoing research, some results supported Dumanoglu et al.'s (1986) conclusion and some supported Kang and Wieland's (1988) conclusion; and it was decided to perform parametric studies on several bridges to further investigate this phenomenon.

The following four distinct phenomena give rise to spatial variability of earthquake ground motions (Kiureghian 1994): (1) loss of coherency of seismic waves due to reflections and refractions in the heterogeneous and non-homogeneous medium of the ground and due to the difference in the manner of superposition of waves arriving from an extended source; (2) difference in the arrival times of seismic waves at separate stations -*wave passage effect*-; (3) spatially varying soil conditions and the manner in which they influence the amplitude and frequency content of the bedrock motion; (4) gradual decay of wave amplitudes with distance due to geometric spreading and energy dissipation in the half space. The present study addresses itself to the second phenomena; the focus being on the effect of the difference in the arrival times of the seismic waves at different degrees of freedom of a bridge. Since the attenuation effect is normally insignificant for typical sizes of man-made structures, this effect is not considered here. Furthermore, in order to concentrate on the wave passage effect, the local soil conditions are considered to be uniform and hence the effects of wave reflection and refraction are assumed to be negligible. However, it should be noted that in the analysis and design of multiply supported structures in regions with rapidly varying soil conditions, site response effect should be taken into consideration (Kiureghian 1994).

THEORY

<u>*Governing Equations*</u> : In matrix form, the equation of motion of a dynamic system with N degrees of freedom is;

$$[M]\{\ddot{v}^t\} + [C]\{\dot{v}^t\} + [K]\{v^t\} = \{P\} \tag{1}$$

where, M, C, K, are NxN mass, damping, stiffness matrices respectively. v^t is a Nx1 vector of displacements. P is a vector of input forces, in this case, it is a null vector. Equation 1 is partitioned for ground degrees of freedom, denoted as v_g^t, where the structure is subjected to ground motion; and for response degrees of freedom, denoted as v_r^t.

$$\begin{bmatrix} M_{rr} & M_{rg} \\ M_{gr} & M_{gg} \end{bmatrix} \cdot \begin{bmatrix} \ddot{v}_r^t \\ \ddot{v}_g^t \end{bmatrix} + \begin{bmatrix} C_{rr} & C_{rg} \\ C_{gr} & C_{gg} \end{bmatrix} \cdot \begin{bmatrix} \dot{v}_r^t \\ \dot{v}_g^t \end{bmatrix} + \begin{bmatrix} K_{rr} & K_{rg} \\ K_{gr} & K_{gg} \end{bmatrix} \cdot \begin{bmatrix} v_r^t \\ v_g^t \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \tag{2}$$

In Equation 2 total displacements, v^t, can be expressed as the sum of relative displacements or dynamic displacements, v^d, and ground displacements or pseudo-static displacements, v^s.

$$\begin{bmatrix} v_r^t \\ v_g^t \end{bmatrix} = \begin{bmatrix} v_r^s \\ v_g^s \end{bmatrix} + \begin{bmatrix} v_r^d \\ v_g^d \end{bmatrix} \tag{3}$$

By substituting Equation 3 into Equation 2 following equation is obtained in terms of relative displacements.

$$M_{rr}\ddot{v}_r^d + C_{rr}\dot{v}_r^d + K_{rr}v_r^d = P_{eff}(t) \tag{4}$$

It may also be shown that the load vector, $P_{eff}(t)$ in Equation 4, can be expressed in terms of ground accelerations, as shown in Equation 5.

$$P_{eff}(t) = -M_{rr}R\ddot{v}_g^s \quad ; \quad R = -K_{rr}^{-1}K_{rg} \tag{5}$$

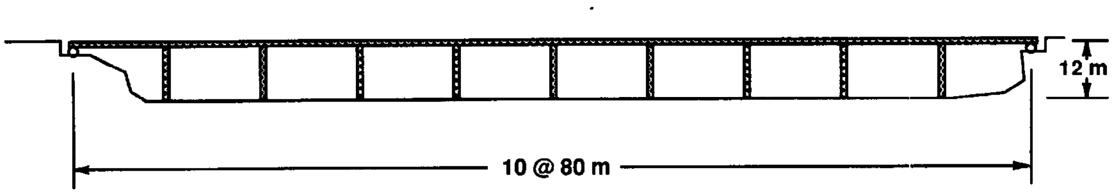
Numerical Evaluation of the Dynamic Response : In the present study, Newmark $\beta=^1/_4$ method, i.e. constant acceleration method is used, although many procedures are available for performing numerical integration (e.g. Newmark 1962, Bathe and Wilson 1973, Hilber et al. 1977). A preliminary study is conducted on SDOF systems to compare the results obtained using the Newmark $\beta=^1/_4$ method and linear acceleration method (Clough and Penzien 1975), and it is observed that for the practical range of stiffness, mass and damping values both methods' predictions of displacement histories are extremely close to each other. Finally, the differential equation of motion, presented in Equation 2.4, is solved using the Newmark $\beta=^1/_4$ method. Details of these numerical methods are available elsewhere (e.g. Newmark 1962, Craig 1981).

During an earthquake adjacently located structures often vibrate out of phase due to their different dynamic characteristics and due to spatial variability of the earthquake ground motions. Earthquake reconnaissance reports (e.g. San Fernando 1971, Loma Prieta 1990, Costa Rica 1991, Kobe 1995) indicate that pounding at movement joints and damage associated with it is still an issue to be considered. Multi-Span bridges with geometric nonlinearities such as, construction gaps, expansion joints, etc., can behave as two different structures before the gaps close and they behave as a single structure once the gaps are closed. This phenomenon further complicates the problem. In other words, it involves dynamic, mass and stiffness matrices. This phenomenon is also implemented in the computer program PHASE, and tested for correctness and accuracy, meaning, there is a control routine which detects movements around the gaps at each time step and accounts for the interaction at the gap. On the other hand, difficulties are encountered handling the impact problem and related energy losses. First the energy losses are neglected to test the accuracy of the numerical algorithm employed to model the physical problem, and without any problems the accuracy is tested. However, to model the energy losses caused by the impact, no rational method is encountered in the literature, and it is concluded that without incorporating a model for energy losses it is not feasible to use that feature of the program. Maragakis et al. (1991) studied the impact between the bridge decks and the abutments during earthquakes, and reported that an equivalent viscous dampers could be used to model the energy losses; and for the evaluation of the damping coefficient case studies were recommended. Jeng and Kasai (1994) performed an analytical study on the out-of-phase vibration of adjacently located bridge structures and they proposed a spectral difference method for the calculation of relative displacements. The conclusions drawn are particularly useful for relative motion design of adjacent structures and no method to evaluate the energy dissipation during impact is suggested. As there is no means to evaluate the energy losses due to impact physically, and then calibrate viscous dampers accordingly, geometric nonlinearities are considered to be beyond the scope of the current study. However, the importance of geometric nonlinearities on the dynamic response of bridges is recognized.

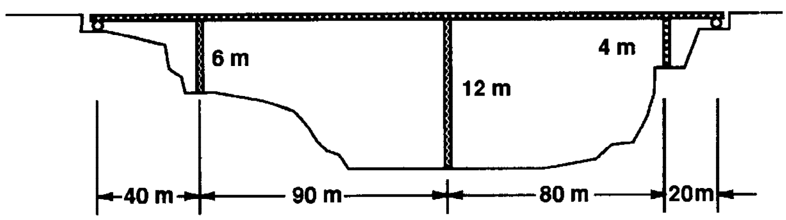
Computer program PHASE is developed for the solution of the physical problem described above. To simplify the physical problem by adding some constraint equations, the deck is assumed to be axially infinitely rigid, and this assumption is checked in every time step by calculating the axial strains in the deck and comparing these strains with the cracking strain of the concrete. In horizontal earthquake analysis, it is highly unlikely that the contribution of the transverse vibrations of the girders to the relative column displacement response will be pronounced; because most of the time deck-pier connections are not rigid and axial deformation of girders due to their transverse vibration are negligible. PHASE reads the structural system definition data from a user-prepared input file and depending on the wave propagation velocity the time lags in the application of the ground motion to different ground degrees of freedom are calculated and after the generation of lagged input vectors, analysis is performed. As an output the program stores the displacement histories of each degree of freedom in a file. Similarly velocity, acceleration, strain energy, kinetic energy, shear force and bending moment histories are stored in separate files. Due to space limitations computer program PHASE will not be discussed further. However, detailed explanations are included in its user's manual (Bayrak 1995).

CASE STUDIES

Case studies on two different bridges using three different earthquake records and six different wave propagation velocities are presented here. The elevations of the two bridges are illustrated in Figure 1. Bridge A has 27 piers -3 per row / 9 rows. 12 m long piers of bridge A are 0.6x1.2m in cross-section. They are spaced evenly and the spacing is 80 meters. Bridge B has 6 piers -2 per row / 3 rows- each of which has the same cross-sectional dimensions (0.6x1.2 m). The heights of the piers, on the other hand, are different from each other. From left to right, they have heights of 6m, 12m, 4m respectively (Figure 1b). The distance between the abutment at the left and the first pier set is 40 m, the next span is 90 m, and following that there are two spans first of which is 80 m long and the second one is 20 m long.



(a) Bridge A



(b) Bridge B

Figure 1. Bridges on which parametric studies are conducted.

All the three earthquake records -El-Centro NS 1940, Santa Barbara S48E 1952, Castaic N21E 1971- are normalized such that the peak ground acceleration is 1.0 x g. Furthermore, the wave propagation velocities used are selected such that they cover a wide range from soft soil (V_{wave} = 200m/s) to stiff soil (V_{wave} = 500m/s-800m/s) and rock (V_{wave} = 1000m/s-2000m/s). Figures 2-3 illustrate the relative displacement histories of the middle piers and the time variation of the total energy in the same piers, for both bridges. It should be recognized that these figures are selected representative results from the parametric study performed; and Table 1 contains the maximum relative displacements of the middle piers.

The maximum displacements depend on the geometry of the structure, the propagation velocity of the travelling waves and the structural damping ratio which is kept constant at 5% of the critical value for all analyses. The analyses performed on the Bridge A under Santa Barbara S48E 1952 earthquake record shows

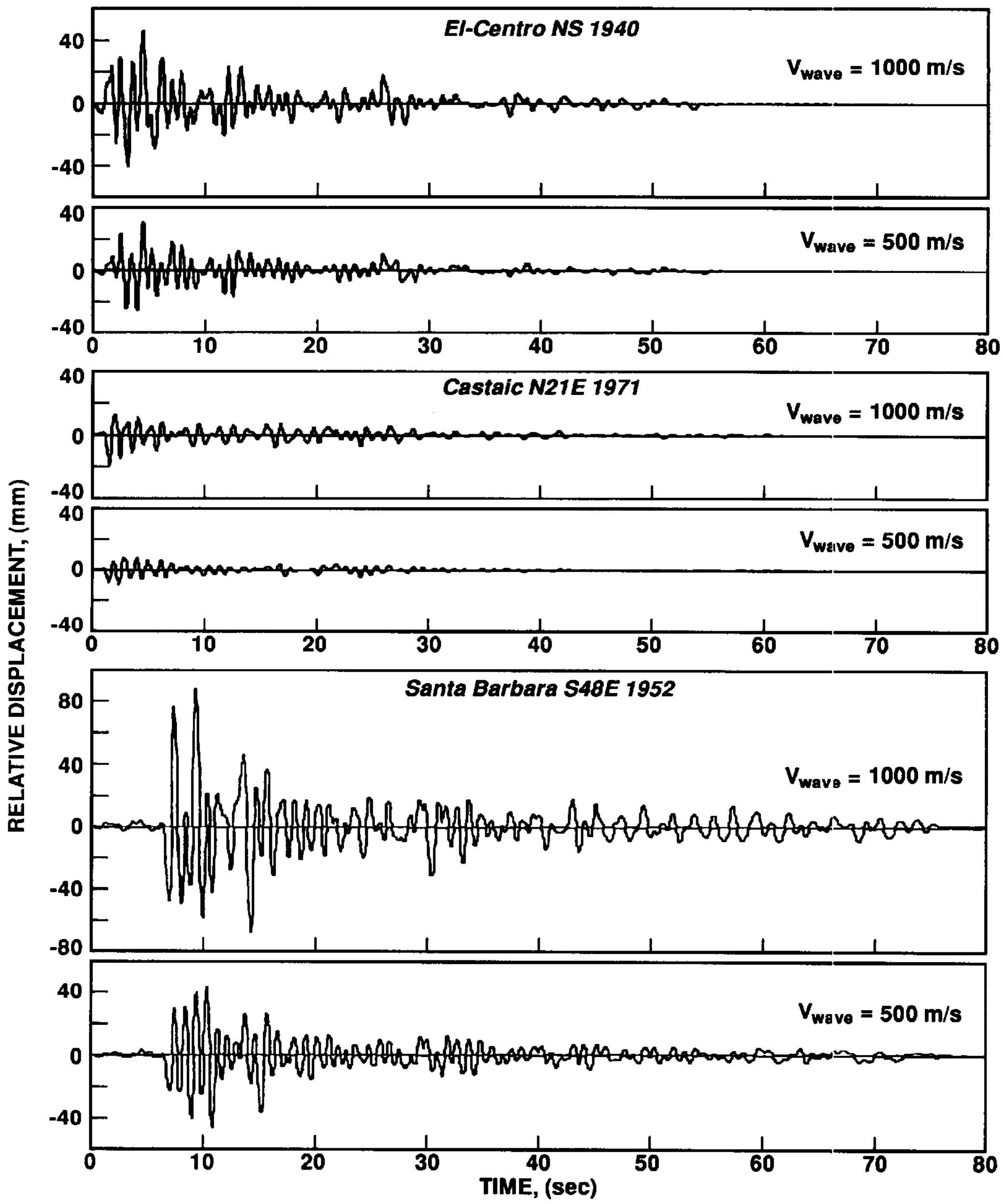


Figure 2a. Relative Displacement History of Bridge A, Pier #5

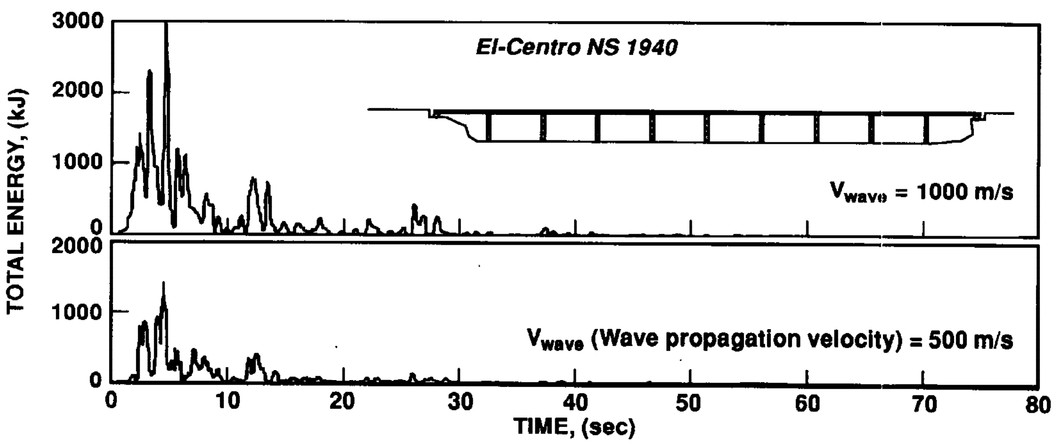


Figure 2b. Total Energy History of Bridge A, Pier #5

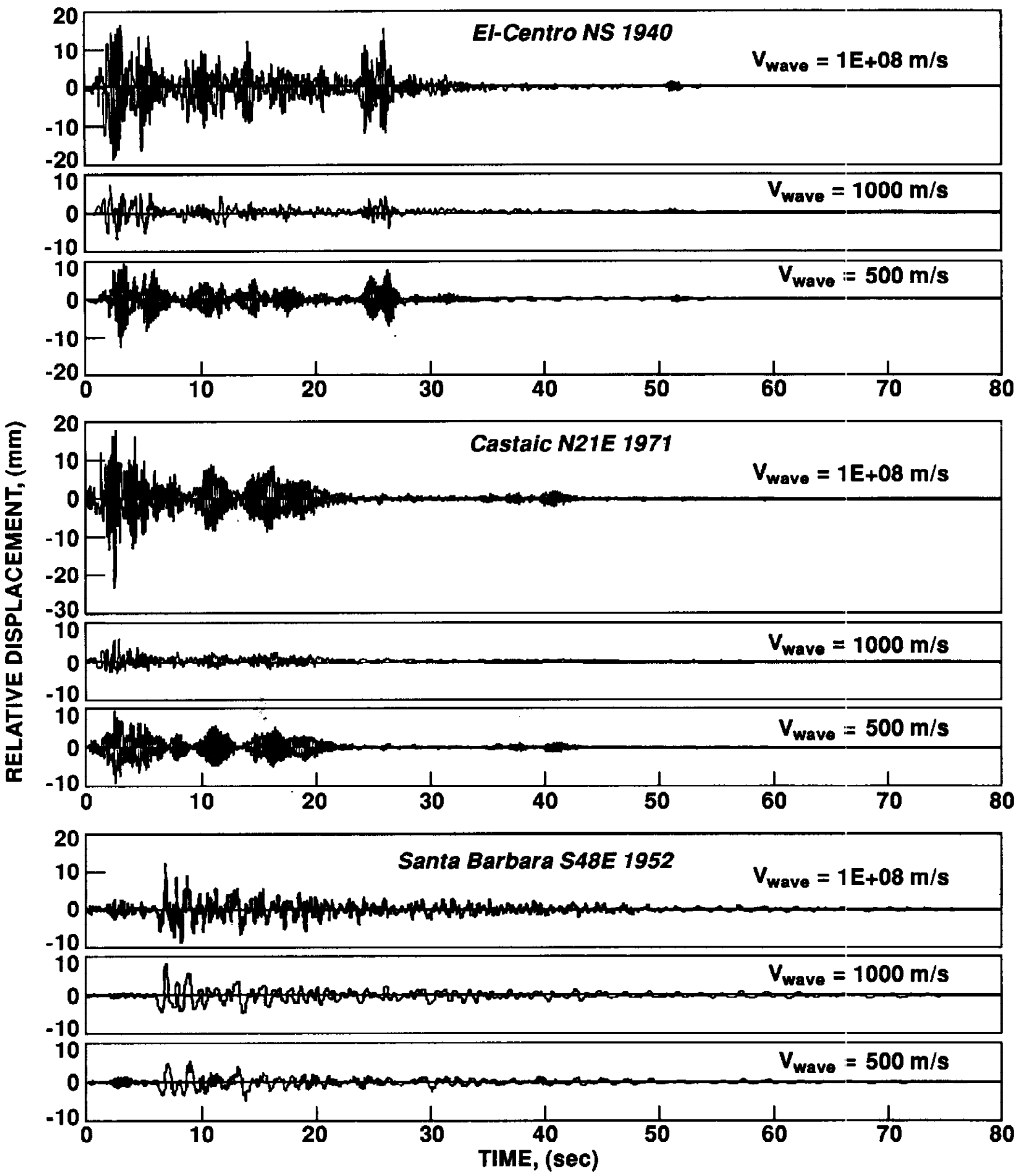


Figure 3a. Relative Displacement History of Bridge B, Pier #2

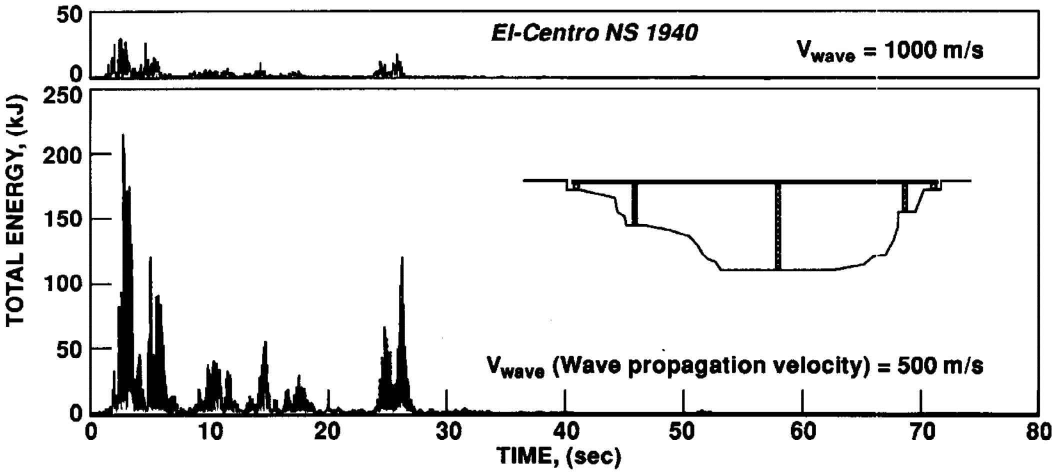


Figure 3b. Total Energy History of Bridge B, Pier #2

that if the soil supporting that bridge is soft (V_{wave}=200 m/s) the maximum displacement is 25.0% of the maximum displacement obtained from the conventional analyses where ground motion is applied to all ground degrees of freedom without phase differences. Bridge A is also analyzed using Castaic N21E 1971 earthquake record and results show that, if the soil supporting Bridge A is rock (V_{wave}= 2000 m/s) the maximum displacement is 215.0 % of the maximum displacement obtained from the conventional analysis. Kang and Wieland (1988) reported that the maximum displacement reached, when the wave passage effect was considered, was about 48% of that obtained by disregarding the wave passage effect. It should be noted that the wave propagation velocity used is not reported in the paper and only one wave propagation velocity was used. In the study performed by Dumanoglu et. al (1986) a change of the wave propagation velocity from 250m/s to 500m/s resulted in a decrease of maximum displacements by 380% at some degrees of freedom along the length of the bridge. A comparison of maximum displacements reached in 36 analyses performed on two model bridges shows that to define a relationship between the wave propagation velocity and the maximum displacements & internal forces is not straightforward and perhaps should not be done.

Because an increase of the wave propagation velocity does not necessarily mean an increase (or a decrease) of the maximum displacement. The results reported by previous researchers (Kang and Wieland 1980, Dumanoglu et al. 1986) support this argument in the sense that for some cases conventional method underestimated the response variables and for others it overestimated them. However a parametric study has been conducted recently to evaluate the influence of wave passage effect on the response of a 100m. long bridge which had 60 piers in 15 rows of 4 piers each. In all the analyses conducted conventional method yielded in conservative results. Hence, it is believed that when the ground degrees of freedom exceeds a certain limit due to erratic characteristic of the ground motion conventional analysis would be conservative in most cases. This observation is currently being investigated on other bridge types.

Table 1. Maximum Relative Displacements

Wave Propagation Velocity [m/s]	Maximum Relative Displacement of the Middle Pier [mm]		
	El-Centro NS 1940	Santa Barbara S48E 1952	Castaic N21E 1971
	Bridge A		
200	11.5	10.1	8.6
500	31.4	46.6	10.3
800	55.8	60.6	15.5
1000	46.1	87.3	19.8
2000	19.7	30.8	28.6
Conventional	29.6	42.1	13.3
	Bridge B		
200	12.0	4.3	13.8
500	12.5	5.4	9.3
800	6.0	7.1	5.1
1000	7.0	8.2	5.6
2000	18.9	12.2	23.5
Conventional	10.2	10.8	14.4

The number of displacement excursions at large peak displacements is another important characteristic that should be considered to estimate the possible damage that can be caused by an earthquake (Newmark and Rosenblueth 1971). Bearing that in mind when Figures 2-3 are analyzed it can be observed that the shape of the displacement cycles and the number of large displacement excursions is influenced by the velocity of the travelling waves. For instance, it can be observed that when wave propagation velocity is 500 m/s, the middle pier of Bridge B -subjected to El-Centro NS 1940 earthquake record- experiences severe forcing and stores an appreciable amount of energy in the first 27 seconds, whereas when the wave propagation velocity is 1000m/s (implying stiffer supporting soil condition) first 7 seconds of the earthquake is the most critical time period for the same pier. Furthermore the maximum energy stored in the same pier is 216 kJ when V_{wave}=500 m/s and it is 31 kJ when V_{wave}=1000 m/s. Therefore comparison of maximum displacements alone may be misleading and comparisons of energies and number of large displacement excursions are useful to evaluate pier performance.

CONCLUDING REMARKS

The problems associated with the rigorous seismic analysis bridges are traced. The computer program "PHASE", developed for the seismic analyses of bridges, is briefly presented and the inclusion of the wave passage effect to the classical problem is illustrated. Case studies performed to evaluate the significance of the wave passage effect are presented. Analyses results reported by previous researchers are compared with current results. It is concluded that in the complete seismic analysis of bridges and elevated viaducts the spatial variability of earthquake ground motions should be considered. Otherwise the design forces may be conservative or uncoservative depending on the geometrical properties of the bridge, soil type and design earthquake. Wave passage effect was pronounced for all the 36 analyses conducted in this study.

REFERENCES

Bathe, K.J., and Wilson, E.L., "Stability and Accuracy Analysis of Direct Integration Methods", Earthquake Engineering and Structural Dynamics, 1973, V.1, pp.283-291.

Bayrak, O., "PHASE", a Computer Program for Dynamic Analysis of Bridges, Version 1.0, April 1995.

Bolt, B.A., "Development of Phased Strong-Motion Time-Histories for Structures with Multiple Supports", Proceedings of the 6th Canadian Conference on Earthquake Engineering, Toronto, Ontario, 1991, pp.35-45.

Clough, R.W., and Penzien, J., "Dynamics of Structures", McGraw-Hill Book Co., New York, NY, 1975, 634p.

Craig, R.R.Jr., "Structural Dynamics", John Wiley & Sons, Inc., New York, NY, 1981, 527p.

Costa Rica Earthquake Reconnaissance Report, Earthquake Spectra, EERI, Supplement B to V.7, 1991, pp.59-92

Dumanoglu, A.A.; Severn, R.T.; BrownJohn, J.M.W., "Asynchronous Seismic Analysis of Bosphorus and Humber Suspension Bridges", Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, 1986, pp.6.9/1-6.9/8.

Hilber, H.M.; Hughes, T.J.R.; Taylor, R.L., "Improved Numerical Dissipation of Time Integration Algorithms in Structural Dynamics", Earthquake Engineering and Structural Dynamics, 1977, V.5, pp.283-292.

Jeng, V., and Kasai K., "Spectral Relative Displacement Between Adjacent Bridge Structures". Proceedings of the 5th U.S. National Conference on Earthquake Engineering, Chicago, Illinois, 1994, V.I, pp.531-538.

Kang, K.D., and Wieland M., "Application of Response Spectrum Method To A Bridge Subjected to Multiple Support Excitation", Proceedings of 9th World Conference on Earthquake Engineering, 1988, Tokyo-Kyoto, Japan, V.VI, pp.VI.531-VI.536.

Kiureghian, A.D., "Model for Coherency Function For Spatially Varying Ground Motion", Proceedings of the 5th U.S. National Conference on Earthquake Engineering, Chicago, Illinois, 1994, V.I, pp.541-550.

Loma Prieta Earthquake Reconnaissance Report, Earthquake Spectra, EERI, Supplement to V.6, 1990, pp.151-187.

Maragakis, E.; Douglas, B.; Vrontinos, S., "Classical Formulation of the Impact between Bridge Deck and Abutments During Strong Earthquakes", Proceedings of the 6th Canadian Conference on Earthquake Engineering, Toronto, Ontario, 1991, pp.205-212.

Newmark, N.M., "A Method of Computation for Structural Dynamics", Trans. ASCE, 1962, V.127, pt.1, pp.1406-1435.

Newmark, N.M., and Rosenblueth E., "Fundamentals of Earthquake Engineering", Prentice Hall, Eaglewood Cliffs, N.J., 1971, 640p.

Park et al., "The Hyogo-ken Nanbu Earthquake of 17 January 1995", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 28, No.1, March 1995, pp.1-98.

Paz, M., "Structural Dynamics Theory and Computation", Van Nostrand Reinhold, 1991, 627p.

San Fernando, California, Earthquake of February 9, 1971, U.S. Department of Commerce, Washington, D.C., 1973, III.V.