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SEISMIC RESPONSE OF SHINKAN-SEN/EXPRESS HIGHWAY VIADUCTS DURING HYOGO-KEN NANBU EARTHQUAKE, 1995

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ABSTRACT

The Hyogo-ken Nanbu Earthquake, occurred on January 17, 1995 caused the heaviest structural damages to the Sanyo Shinkan-sen viaducts at the Itami-Nishinomiya-Amagasaki sections and also to the Hanshin Express Highway viaducts in Kobe. Not only partial failures of those structures but also the total collapse occurred at places. On site observation implied that the structural failures were mostly due to the application of excessive bending and/or shear force at supporting columns. The railway/highway viaducts are characterized by their structural configuration as extended top heavy structures. This paper investigates the failure mechanism from the viewpoint of the vibration characteristics that include the subgrade foundation impedance.

KEYWORDS

Railway/highway viaducts, Extended top-heavy structures, Vibration characteristic, Structural failure, Hyogo-ken Nanbu Earthquake, Dynamic analysis

INTRODUCTION

The Hyogo-ken Nanbu Earthquake, occurred on January 17, 1995 caused the heaviest structural damages. Among them, those of the Sanyo Shinkan-sen viaducts at the Itami-Nishinomiya-Amagasaki sections and also of the Hanshin Express Highway viaducts in Kobe should be pointed out. Not only partial failures of those structures but also the total collapse occurred at places, in contrast to the nearby residential houses which lucky to say survived the strong earthquake shaking with modest damages. The railway/highway viaducts are characterized as extended top heavy structures by their high elevated heavy girders along the track.

Fig. 1 is a photo that shows the total fall of Hanshin Express Highway girder over 600 m at Fukae section. This viaducts were made of prestressed concrete girder spanning 18 spans continuously on single slender circular cross-section columns. At the design stage all central hanged portions of the girder were assumed hinged-type connection with the built-in side portions to the pier; however, at the construction stage their connection were changed into rigid by inserting tie-bars and gluing by cements. See the drawing in Fig. 3. The statement of witnesses at the moment of the collapse was that the overturning started with the eastside spans followed by other spans successively one by one and proceeded toward the west, like a standing screen fell down on to the ground from one end to the other end. The piers were broken near the feet with the inside reinforcing steel bars coming outside after torn off or buckled. At the design stage the soil condition at the site

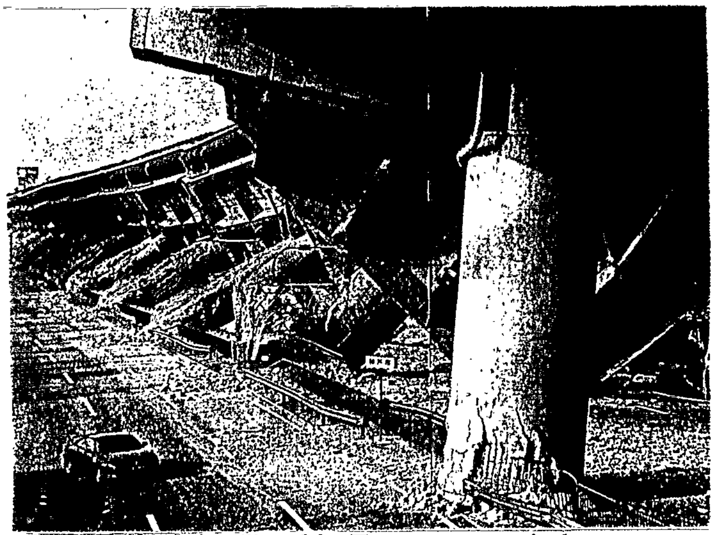


Fig. 1 Fall of the Hanshin Expressway viaduct

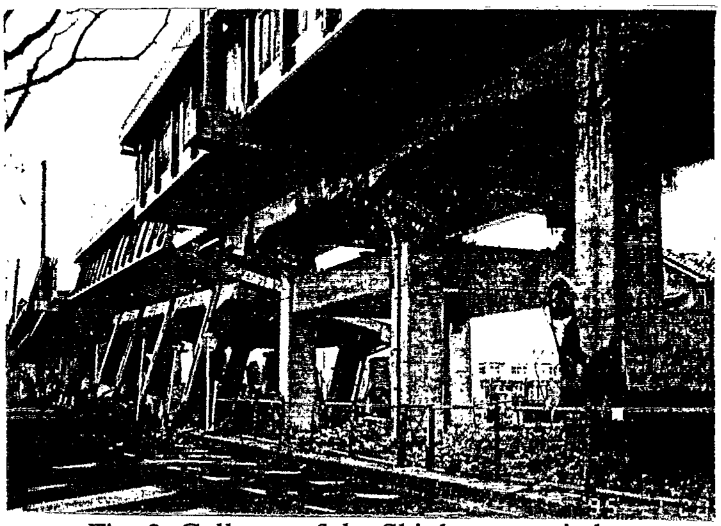


Fig. 2 Collapse of the Shinkan-sen viaduct

was classified as the second class soil of sandy soils and gravels based on the Japan Seismic Design Code. However the geology survey indicated soft soils are irregularly distributed at the surface.

Fig. 2 is a photo that shows the damages of the Shinkan-sen viaducts. The viaducts were mostly comprised of the three-span continuous units as illustrated in Fig. 8. The fall-down of the Shinkan-sen viaducts occurred due to the damages of the supporting columns at the top or the connection with the horizontal beams in the frame system. The sight inspection implied that the shear failure at columns was the main cause. Interesting to note is the fact that these structural damages centered at the sites of soft shallow alluvia. Therefore, the site condition is supposed to be closely concerned with them. In this study, the authors investigate the seismic behavior of the top-heavy viaduct structures from the viewpoint of the vibration characteristics in soil-structure interaction and clarify the reason why the excessive bending moment and/or shearing force were generated at the supporting columns of them.

HANSHIN EXPRESS HIGHWAY VIADUCTS

The continuous eighteen-span model was considered here for the seismic analysis of the Hanashin Express Highway at Fukae section. Fig. 3 gives the dimensions of the viaduct of one span. The three dimensional frame analysis was conducted with consideration of the pile head impedance. The girder is modeled by a grid system of beam elements and the pier also by beams. The soil-pile interaction was performed based on the ring pile modeling and assumption (Takemiya and Katayama, 1994). The material damping of 5 % of the critical value is introduced in the Rayleigh type assumption. The input motion was prepared through the surface soil amplification at the site.

Fig. 4 is the vibration modes of the total 18-spans of the viaducts. It is observed that they are characterized by the localized torsional behavior over several spans which is caused by the horizontal and vertical motions of the

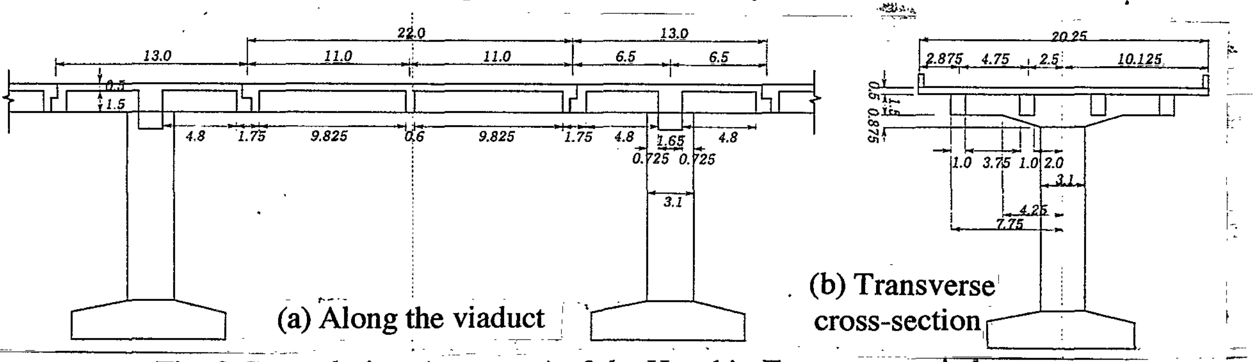


Fig. 3 General view (one-span) of the Hanshin Expressway viaducts

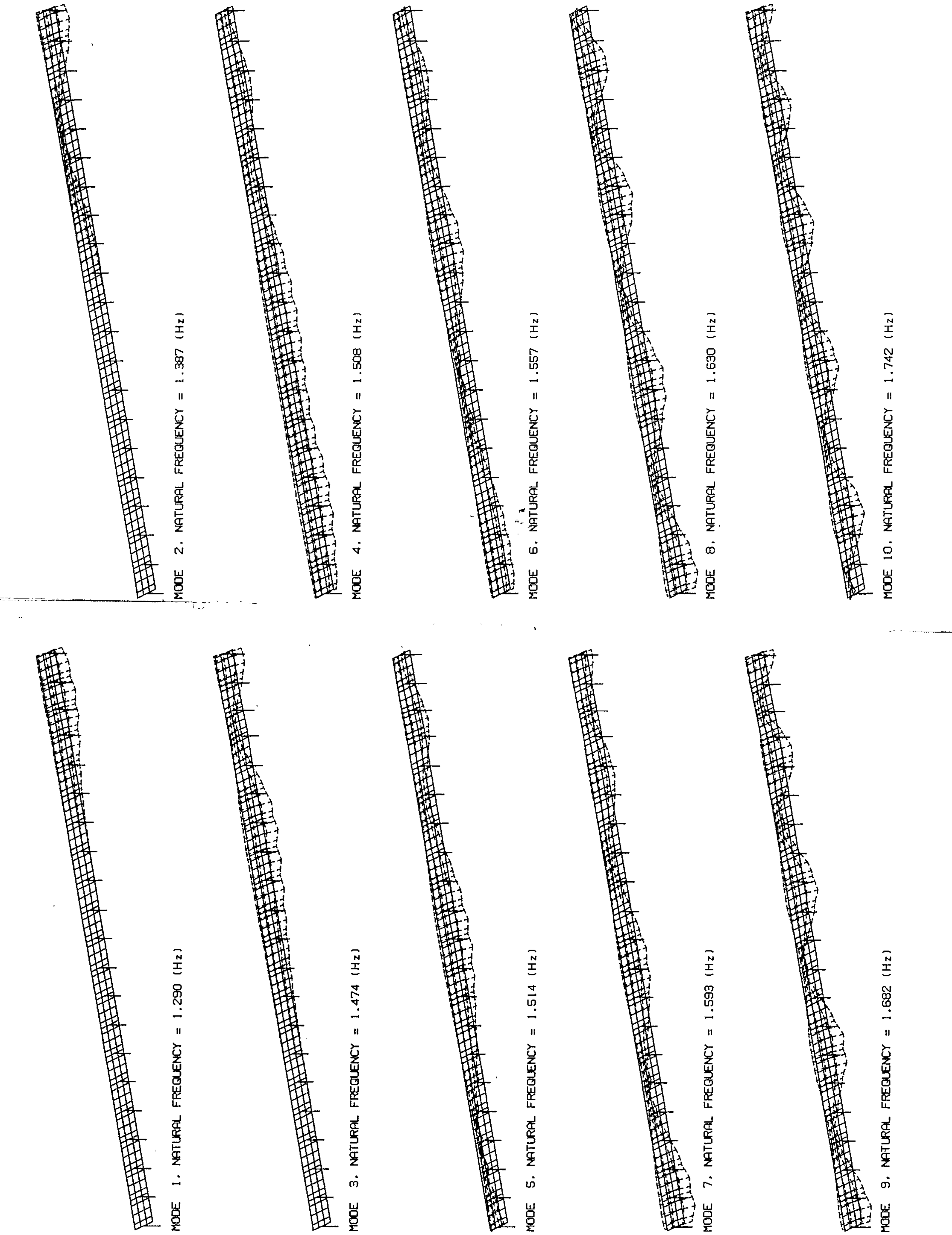
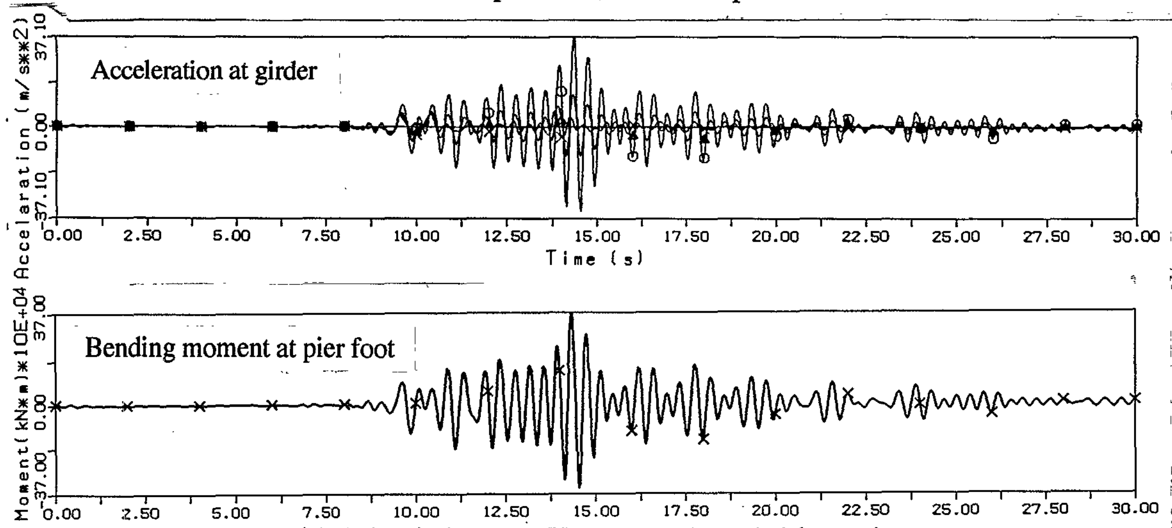


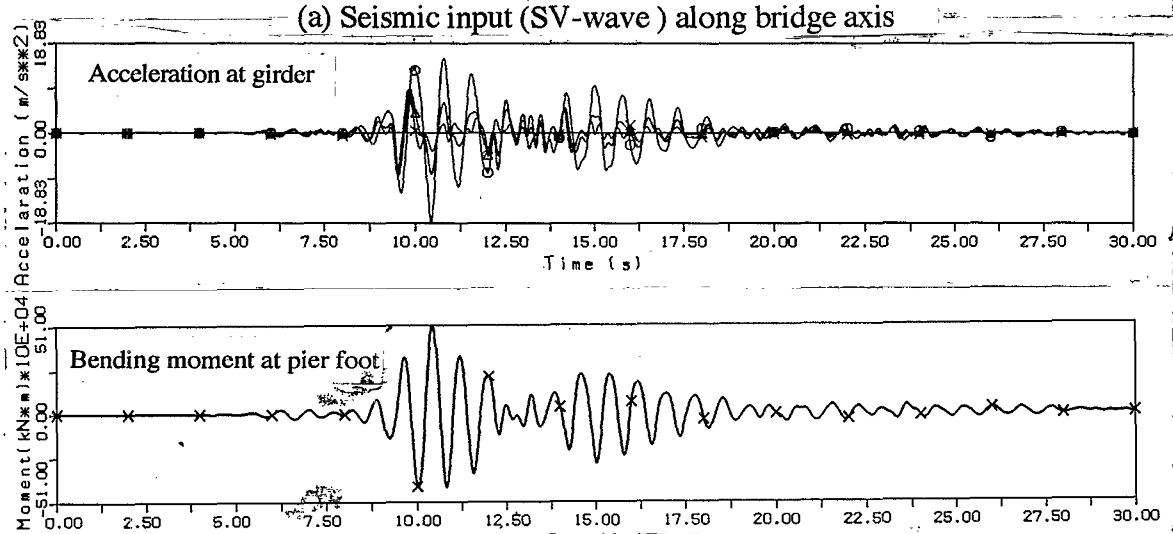
Fig. 4 Vibration modes of the Hanshin Expressway viaducts

sections perpendicular to the bridge axis. The location shifts according to the focused period range. Specifically, the first three modes are concerned with the torsional motions at the east-side several spans, the 4th mode with those from the center to the west-side spans, and the 5th and higher with those of the total spans. These vibration modes are generated in a narrow period range from 0.5 to 0.8 seconds. The predominance of the east-side spans may be caused by the fact that the pier height becomes higher and the surface soft soil becomes deeper toward the east-end pier. A significant resonance of these vibration modes with the amplified surface soil motion is very possible. The vibration modes which are related to the longitudinal motions appear in the higher frequency range.

The SV and P waves field is assumed for the site response computation in the plane of the bridge axis (in-plane motions) and the two-dimensional amplification is considered for the soft surface layers (Takemiya and Wang, 1994). The SH wave field is assumed for the response computation perpendicular to the viaduct axis and the one-dimensional amplification (SHAKE, 1972) is employed for the soft layers because of less topography effect compared to the in-plane motions. The frequency domain formulation is taken for convenience in incorporating the frequency-dependent soil effect. The Fourier transform method is employed for the response evaluation in time domain. Since the Hanshin Express Highway runs almost in the East-West direction, the EW component of the modified JMA-Kobe earthquake record is used for the seismic simulation along the bridge axis and the NS component for that perpendicular to it. Fig. 5 shows the response time histories of the pier at which the maximum bending moment was attained at the foot. The out-of-plane response gains the larger maximum response that comes earlier than that in in-plane motions that comes later. The relative displacement in the perpendicular direction to the bridge axis is more than 20 cm between the pier top and the bottom. The acceleration near 2, 000 gal is attained at the pier top. The bending stress at the bottom is over 1,700 kg/cm^2 when the total cross-sectional are is used for computation, which implies the failure of RC members.



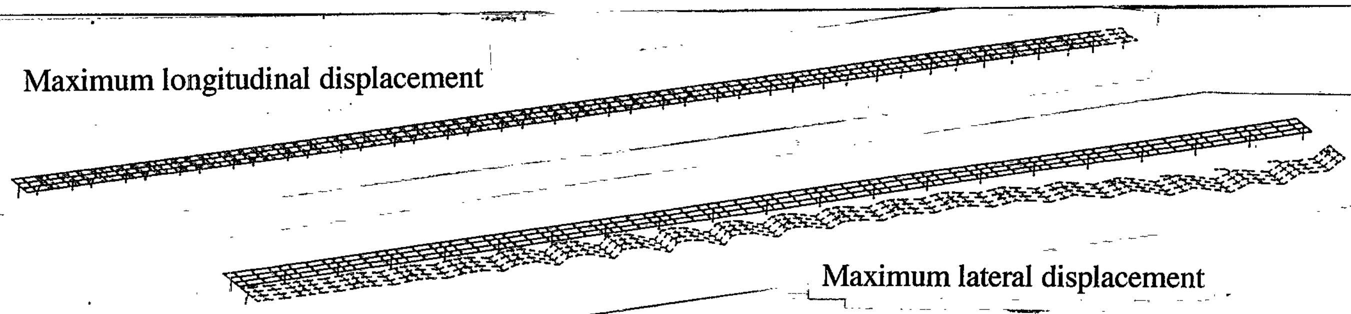
(a) Seismic input (SV-wave) along bridge axis



(b) Seismic input (SH-wave) perpendicular to bridge axis

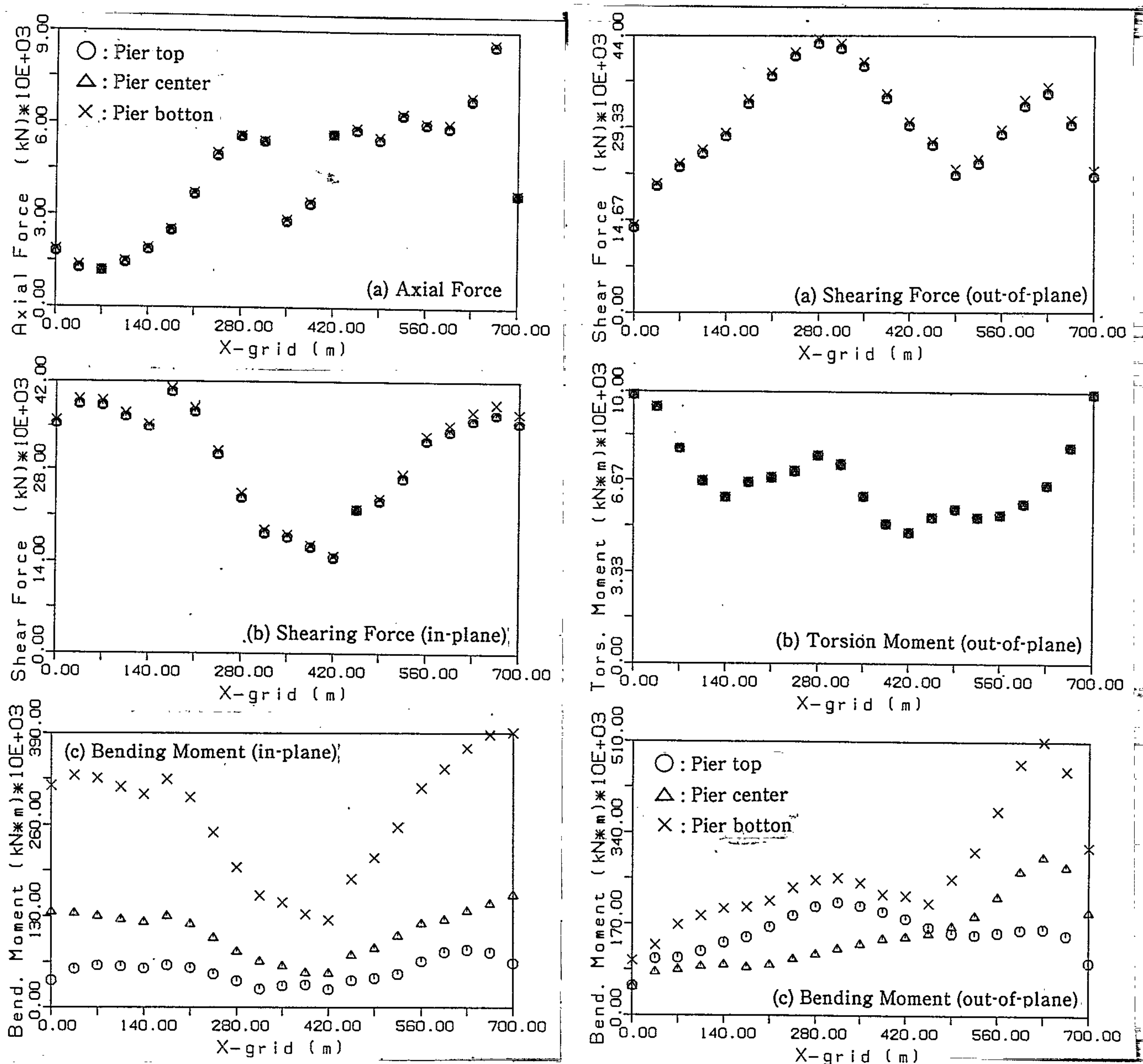
Fig. 5 Response time histories

Fig.6 is the maximum displacement responses of the girder in which the predominant motions of the east-side spans is noted. Fig.7 gives the maximum internal forces for the seismic input along the bridge axis and those perpendicular to it. The variation of internal forces along the bridge axis is caused by the vibration modes. It is clear that the bigger pier foot bending moment resulted in the case of out-of-plane motions for the SH wave incidence than in the case of in-plane motions for the SV wave incidence.



(a) Seismic input along bridge axis (b) Seismic input perpendicular to bridge axis

Fig. 6 Maximum displacement



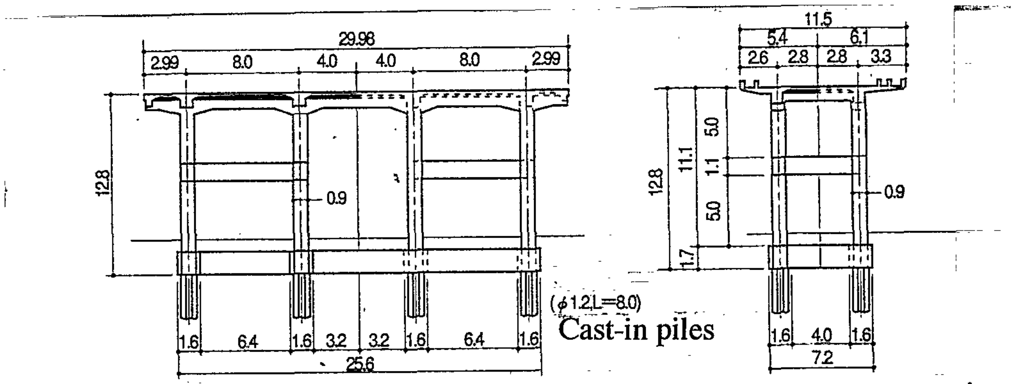
(a) Seismic input (SV-wave) along bridge axis

(b) Seismic input (SH-wave) perpendicular to bridge axis

Fig. 7 Maximum internal forces

SHINKAN-SEN VIADUCTS

The shinkan-sen viaducts extend across different soil conditions of soft shallow alluvium deposits and stiff diluvium. Fig. 8 illustrates one unit of three-span frame structure out of them. For the dynamic analysis the three dimensional beam model is used. The foundation effects are incorporated in terms of the spring constants. The damping is assumed as the Rayleigh type of 5 % critical value. Fig. 9 shows the vibration modes which reflect the heavy girder motions. The first three vibration modes are characterized by the girder motions as a rigid body, giving the motions along the track, those perpendicular to it, and the twisting motions around the vertical. Note that those vibration modes have very close natural period around from 0.54 to 0.52 seconds. The modes from 4th to the 6th are concerned with the girder deformations. Those higher than the 7th are caused by the pier deformation.



(a) Along the viaduct (b) Transverse cross-section

Fig. 8 General view of a 3-span Shinkan-sen viaduct

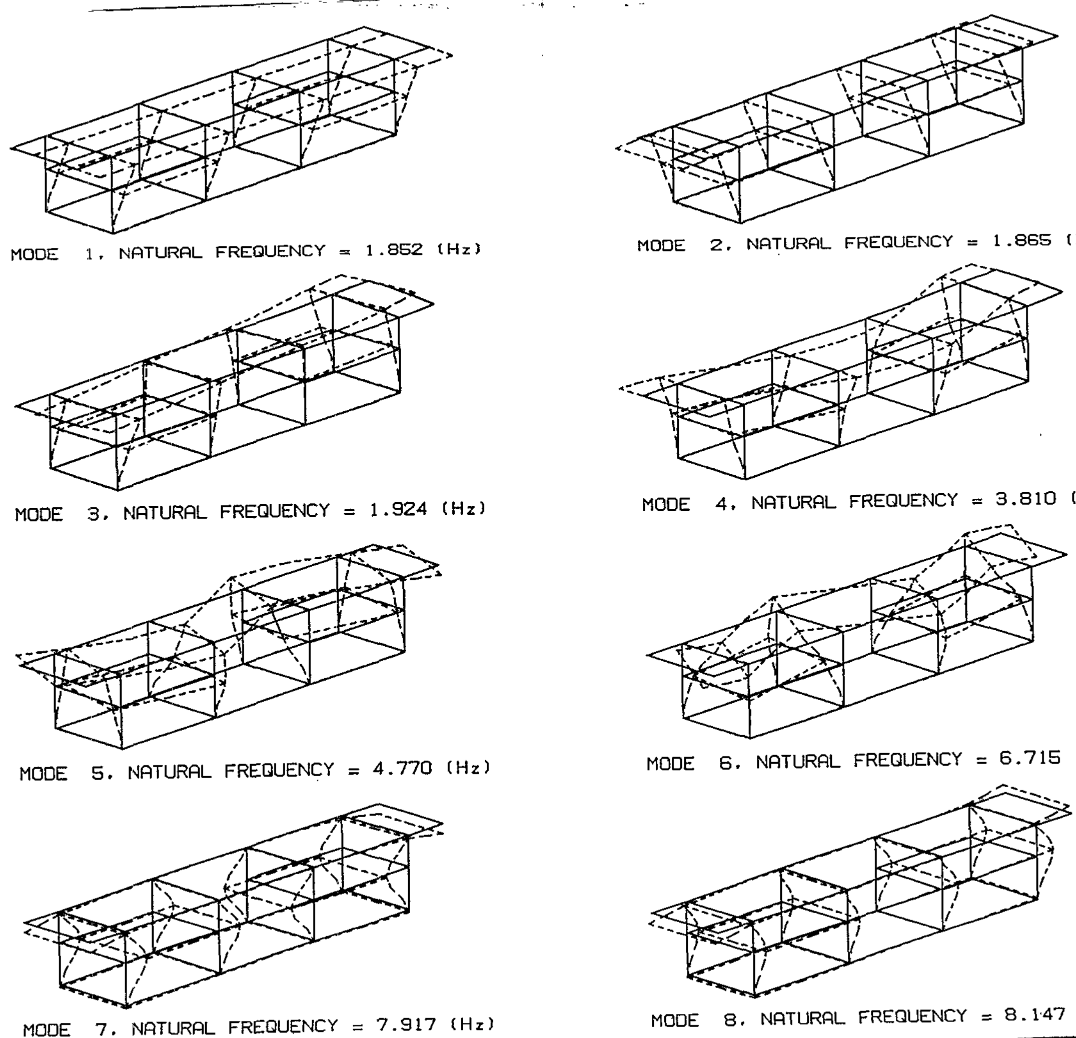
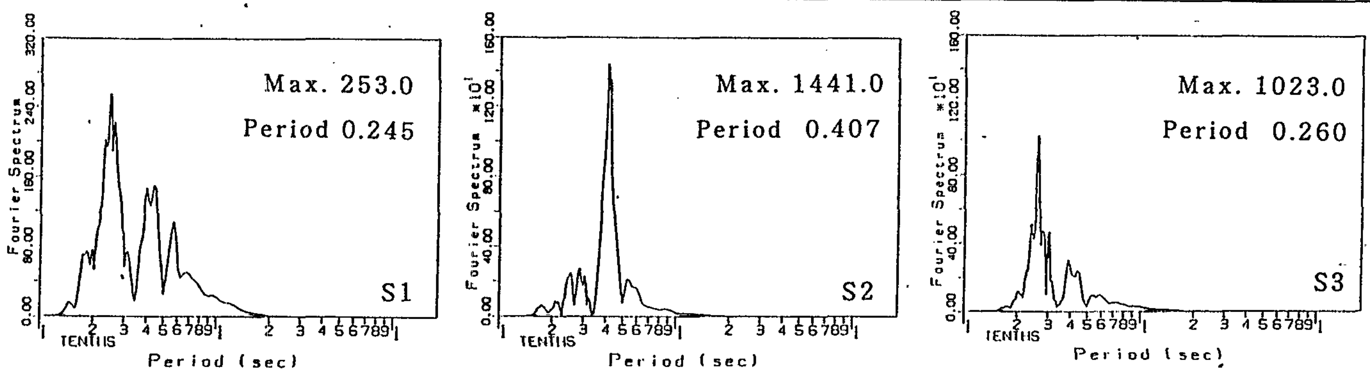


Fig. 9 Vibration modes of a 3-span Shinkan-sen viaduct

The earthquake response is computed at the site that encompasses the area where heavy damages occurred. An artificial motions which include the short period components but exclude the long period components is used in view of the wave lengths to be involved for shallow soft amplification. This motion is comparable with the JMA-Kobe motion (Takemiya and Adam, 1995) in terms of acceleration at soft shallow surface. The ground surface motions at different site, whose frequency characteristics are depicted in Fig. 10, are taken as the input motions to the viaduct. The heavy damage occurred at the site designated as S2.



S1: Stiff soil near rock base,, S2: Soft shallow soil (Shinkan-sen viaducts collapsed), S3: Stiff sandy soil and gravel

Fig. 10 Fourier spectral densities of the seismic input motions at different sites

Fig. 11 gives the maximum internal force diagrams at the inside and outside columns of the three-span viaduct. Note that the response values at S2 location in the SV wave incidence along the track axis indicate twice larger values than those at S1 and S3 locations. This is due to the amplified soil motions at the S2 site from the topography. The maximum stresses are computed, based on the total cross sectional area, as around 800 kg/cm^2 for the bending at the base of the columns, and the half at the connection with the horizontal beam and at the column tops. The shearing force in the columns is almost the same from top to base, indicating around 30 kg/cm^2, and axial force is stepwise increased at the connection with the horizontal beam, showing around 75 kg/cm^2 at the bottom. These bending and shearing force imply a possibility of sever column damages of the viaducts. The response features due to seismic motions perpendicular to the track axis is very similar to that along the track axis. However, at girder there is a significant difference between response values by the direction of seismic input. The seismic input along track lead substantial girder vibrations and the seismic input perpendicular to it affected the torsional vibrations at the end spans.

CONCLUSION

Through the seismic simulation on the Shinkan-sen viaducts and the Hanshin Express Expressway viaducts the resonance between the vibration modes and seismic waves are pointed out for the large amplified responses. The seismic behavior of those viaducts are interpreted in view of the predominant vibration modes. In the case of the Hanshin Express Expressway viaducts, due to the torsional behavior at the heavy girder, the collapse seems to started with the east-side higher piers at deeper soft soil by the excessive overturning stresses and it proceeded toward the west-side piers. In the case of the Shinkan-sen viaducts the sever damages at the column top and column bottom and the connection with the horizontal beam are substantiated from the high stress generation due to the girder motions along the track and perpendicular to it in close proximity of natural periods.

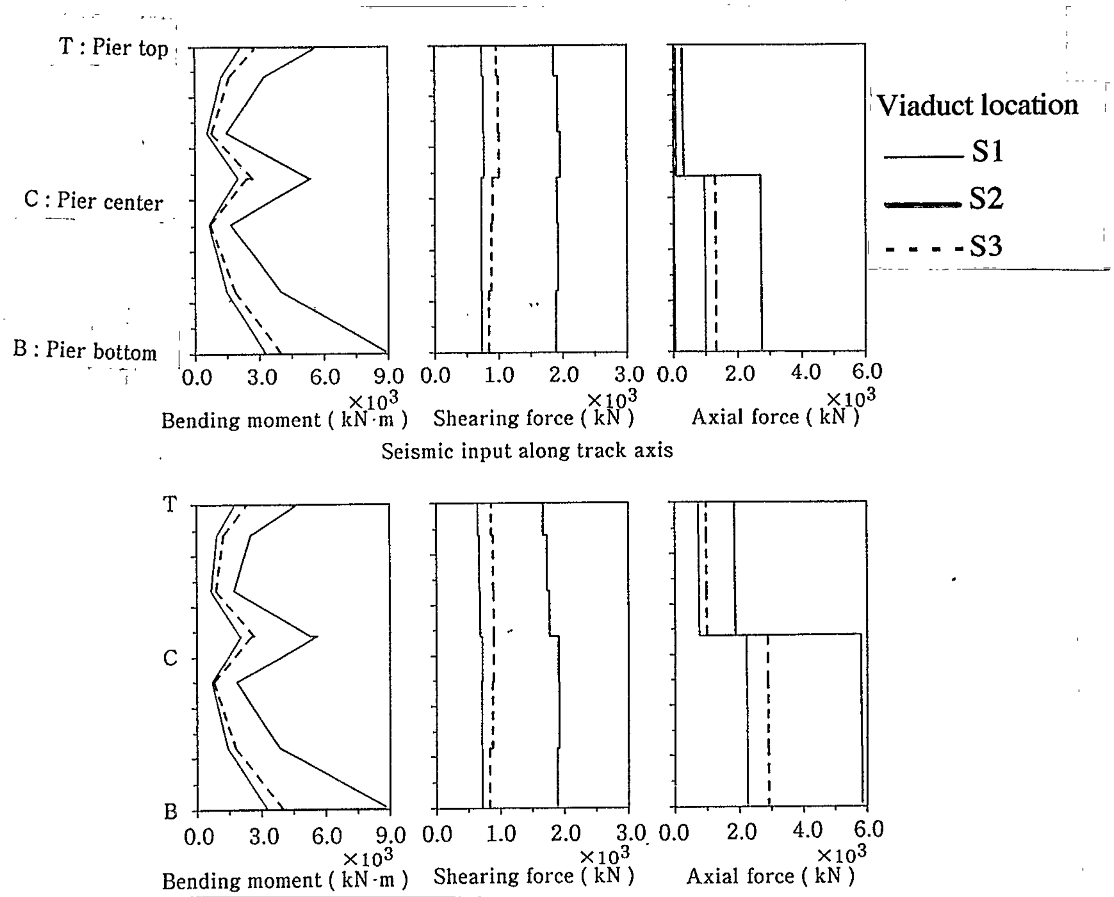
REFERENCES

Schnable, P.B. Lysmer,J. and Seed, H.B. (1972). SHAKE, A computer program for earthquake response analysis of horizontally layered sites, ERRC-72, UC Berkeley

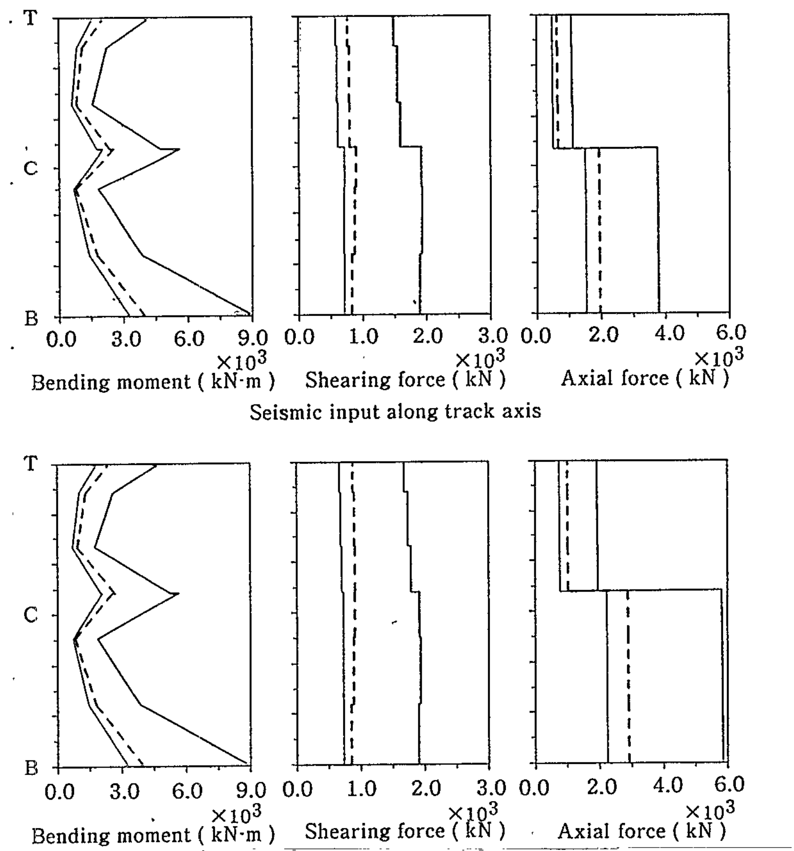
Takemiya, H. and Katayam, Y.(1994). Flexibility approach for dynamic analysis of grouped-pile foundation and design proposal, Proc. JSCE, 489/I-27, 207-215.

Takemiya, H. and Wang, H.(1994). Amplification and phase characteristics of seismic waves by soft alluvium, Proc. Symp. Soil amplification at soft soil, JSSMF, 275-282.

Takemiya, H. and Adam, M. (1995). Why the heaviest damages occurred in Kobe during the Hyogo-ken Nanbu Earthquake, Japan 1995. The Kobe Earthquake, Comp. Mech. Publ., Southampton, UK., 39-58.



(a) Internal forces at inside pier
due to seismic input perpendicular to bridge axis



(b) Internal forces at outside pier
due to seismic input perpendicular to bridge axis

Fig. 11 Internal forces diagram of 3-span continuous viaduct
for short-type artificial earthquake motion