A STUDY ON PROPAGATION MECHANISM OF THE VIBRATION DUE TO ROAD TRAFFIC

Norio TOKUNAGA
Hanshin Expressway Public Corporation
4-1-3 Kyutarocho, Chuo-ku, Osaka, 541 Japan
Fax: +81-6-252-4583

Hirokazu IEMURA
Professor
Kyoto University
Yoshidahoncho, Sakyo-ku, Kyoto, 606-101 Japan
Fax: +81-75-753-5926
iemura@catfish.kuciv.kyoto-u.ac.jp

Takashi NISHIMURA
Professor
Osaka City University
Sugimoto, Sumiyoshi-ku, Osaka, 558 Japan
Fax: +81-6-605-2731

Kiyoshi HAYAKAWA
Professor
Ritsumeikan University
Nojicho, Kusatsu, Shiga
558 Japan
Fax: +81-775-61-2789
kiyoshi@bkc.ritsumei.ac.jp

Atsushi MORI
Ph.D
Japan Engineering Consultants Co.
5-33-11 Honcho, Nakano-ku, Tokyo, 164 Japan
Fax: +81-3-5385-8525
moria@jecc.co.jp

abstract: Traffic vibration is one of the most important issues in the consideration of roadside environments. Of the conceivable traffic vibration countermeasures at different points of view, namely source of vibration, propagation of vibration and receiving end of vibration, few discussions have been made as to the control of vibration during its propagation. This is because vibration mechanism is too complicated to be identified as it is substantially influenced by vibration components and ground properties like layer composition. This paper presents the results of an approach using a proposed method which is based on more precise back analyses as an alternative to obtain input load waves made with a view to identify the mechanism of vibration caused by traffic on a viaduct in an urban area, and to review the effectiveness of vibration reduction countermeasures using vibration-proof walls.

1. INTRODUCTION

In response to traffic vibration on viaducts in urban areas, some organizations in Japan have taken countermeasures mainly for superstructures, which were considered effective based on the actual situation identified by observation of traffic vibration. Traffic vibration, one of the crucial issues to be considered in relation to roadside environments, poses problems in the form of a complex mixture of three key factors - (1) source of vibration, (2) propagation of vibration, and (3) receiving end of vibration (Adachi et al, 1989). In the case where vibration is mainly caused by the superstructure of the viaduct or at the source of vibration, countermeasures like jointless superstructures can produce certain effects, but fall short of eliminating complaints from roadside residents. Vibration caused by vehicles running on the viaducts spreads to roadsides via girder, bridge pier, foundation and surrounding ground. Vibration typically goes through the ground as a surface wave and/or a body wave. The current technology to insulate the vibration has only a limited effect to control vibration from various sources including vehicles. If the existing countermeasures cannot be applicable to reduce vibration in roadside areas satisfactorily, it is necessary to adopt countermeasures effective in the propagation path, at receiving end or a coupling of both. From a viewpoint of road management, however, it is difficult to quantitatively evaluate the scope and level of countermeasures required because roadside structures which are subject to vibration at its destination are widely different in type. In this connection, sheltering by vibration-proof walls or other countermeasures in roadside green belts established as buffer zones between the viaduct and the roadside residential areas is considered one of the effective vibration insulation means in the vibration propagation path. If such a countermeasure proves to be effective, it will help to obtain acceleration distribution effect in distance in addition to that expected by the width of the existing roadside green belt which generally measures from 10 to 20m. Then land in urban areas will be put to more effective use, and roadside environments greatly enhanced.

It is difficult to apply distinguishable countermeasures against traffic vibration in its propagation path because the frequency band and the magnitude of the vibration in question are complicated under a heavy influence of layer composition and stiffness of surrounding ground. This is the reason that only a few cases of small-scale vibration countermeasures in the propagation path have been taken for surface roads (Hayakawa et al, 1990 and 1991). At the present, there is hardly any works done for viaducts.

In view of the influence of ground properties on vibration as described above, numerical analyses are frequently conducted, when taking vibration control measures in the propagation path, for a model consisting of a substructure of viaduct, the foundation and the ground in order to determine an effective method and design. While few cases of analytical review of vibration control measures in propagation path have been reported among the studies on traffic vibration, Japan's Hanshin Expressway Public Corporation has been continuing studies in this field as shown in Table 1. In the analyses by the Corporation, a model consisting of the ground, bridge piers and foundations has been developed by a two-dimensional or three-dimensional finite element method (FEM), and vibration reduction effects have been reviewed for different methods by applying an acceleration wave at mass points on the bridge pier or by inputting steady-state vibration (Matsuura et al, 1990).

Simulation analyses based on the vibration data observed in the manner described above provide significant knowledge indeed, but involve the following problems as well.

① Observed data has been obtained at a limited number of points on the structure and the ground, and, as such, is insufficient for reviewing vibration propagation characteristics. Data observed underground is considered also necessary.

② Results of simulation analyses do not always show a good agreement with observed data. One reason may be the difficulty of setting a load wave to be input to the model.

③ Traffic vibration occurs as a result of such a complex mixture of characteristics at the source, in the propagation path and at the receiving end of vibration that numerical analyses are required as a tool for grasping the entire picture of the problem, but a model having

Table 1 Review of Analytical Approaches in Relation to Vibration Control in Propagation Path

Document	1988 Study Report on Road Traffic Vibration Control Countermeasures (March 1989)			1989 Study Report on Road Traffic Vibration Control Countermeasures (March 1990)
Analysis methods and cases	Two-dimensional dynamic finite element method for ground- structure coupled vibration	Finite element method for vibration-proof wall (two and three dimensional models were made for ground and structure.)	Two-dimensional dynamic finite element method for ground-structure coupled vibration	A three-dimensional model of a viaduct was made, and a model of the ground made by two-dimensional finite element method
	Model: Modelling of viaduct and ground by using plane strain elements	Model: Modelling of a vibration-proof wall and the ground by using plane strain element	Model: Modelling of an viaduct, a vibration-proof wall and the ground by using plane strain elements.	Model: Results of analyses of superstructure and substructure were input for analyses of substructure, foundation and ground.
	Case: 8 cases 1. Analysis for current status 2. Embankment 3. Poor-mix concrete used as embankment material 4. Stiffness of column material increased 5. Low-strength material used for the top ground of footing(type A) 6. Low-strength material used for the top ground of footing(type B) 7. Poor-mix concrete used on the ground surface 8. Poor-mix concrete used on the ground surface in the vicinity of limited area	Case: Whether one- or five-element per an urethane layer was used. Whether lightweight cellular materials were used or not in the pore space in the vibration-proof panel. When the Young's modulus of the cellular material was 10.0tf/m2 or below, there was no effect on vibration control.	Case: Excitation by steady-state vibration: 1. No vibration-proof wall was installed. 2. A Vibration-proof wall was installed on one side of the bridge pier. 3. When the distance from the center of the foundation varied in case 2. (in cases of 9m, and 16m) 4. When the depth of the vibration-proof wall varied in case 2. (in cases of 20m and 30m in depth) 5. Vibration-proof walls were installed on both sides of the bridge pier. 6. When the distance from the center of the foundation varied in case 5. (in cases of 9m, and 16m) Excitation by transient vibration: 1. No vibration-proof wall was installed. 2. A 30-m deep vibration-proof wall was installed on one side of the bridge pier at the point 9m from the center of the foundation.	Case: Whether concrete was placed on ground surface or not Whether there was continuous girders supported by structural bearing support or not
Input load	Input a measured wave at the center of the beam	Steady-state vibration of 10Hz was applied to the ground in the direction of y axis.	Steady-state and transient vibrations were applied at the center of the top of bridge pier	Analysis results obtained as running vehicles, and response of superstructure and substructure
Analysis results	In cases 7. and 8., there were substantial decreases of acceleration, velocity and VL amplitude on a wide scale.	Same results were obtained regardless of whether one- or five-element per an urethane layer was used.	Excitation by steady-state vibration: When the vibration-proof wall had a depth of 20m, diffraction from the bottom was so large that no effects were obtained. Greater vibration control was achieved when the distance from the center of the foundation was 16m than when the distance was 9m. Excitation by transient vibration: The vibration-proof wall reduced both acceleration and velocity in a rate of 55%.	Superstructure and substructure: Maximum acceleration of superstructure and displacement in the horizontal direction were reduced 40 to 60% by using continuous girders. substructure, piles and the ground: The difference between continuous girder and simple girder in response acceleration at the position of horizontal beam on the viaduct was horizontally small but vertically large. Also the difference between them in maximum response values on the ground surface (acceleration, velocity and vibration level) was small within the range of 20m from the center of the viaduct, but none outside that range.

excessively complicated dimensions may make interpretation of results difficult.

In view of the above, this paper presents a new method using two-dimensional finite element analysis based on the data observed for real structures, the results of an attempt by such a new procedure to grasp difficult-to-explain propagation mechanism at a certain point, and the effects of countermeasures for reducing vibration in its propagation path using vibration-proof walls. Not many studies have yet been made about vibration reduction in propagation path.

2. VIBRATION CHARACTERISTICS AT A STUDY SITE

The traffic vibration to be studied in this paper is caused by vehicles running on the viaducts in urban areas.

The viaduct in this study has I-beams, T single-pile bridge piers, and cast-in-place piles as generally used in the city area. The ground had an alternation of relatively soft alluvial sandy soil and alluvial cohesive soil. The piles were embedded into the diluvial deposit. Figure 1 shows a schematic structure of the bridge substructure for the study, and a diagram of the layers. The dominant frequencies and the causes of vibration on the structure and the ground were obtained (as shown in Table 2) by observing vibration due to test vehicles running on the viaduct. Vibration was observed using a testing vehicle (25tons) for 8 running patterns (for example, up-line and down-line directions with driving lane and passing lane by traveling speeds of 60 km/h and 80 km/h. Amplitude characteristics of vibration components in the vicinity of 2.9Hz, 3.8Hz, 9.5 to 10.5Hz and 13.4Hz shown in Table 2 were predominant though there was a slight variation of prevailing vibration components. As a result, such frequency figures were considered to represent the noteworthy characteristics of vibration components at the location for the study.

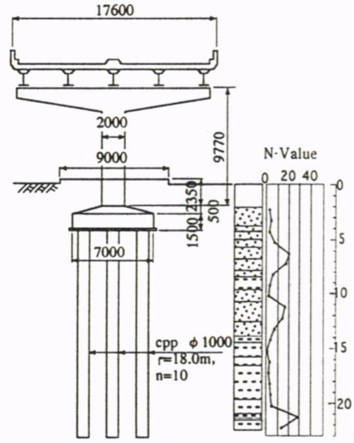


Figure 1 Schematic Structure of the Bridge Pier for the Study

Table 2 Dominant Frequency and Causes of Vibration

Dominant frequency (Hz)	Assumed cause of vibration
2.6	Bridge pier rocking perpendicular to bridge axis
2.9	Vehicle vibration of sprung mass, value unique to the ground
3.8	Deck deflection (primary), bridge axis bending on the bridge pier
8.9	Bridge pier torsional vibration
9.5～10.5	Vehicle vibration of un-sprung mass, bridge pier torsional vibration
13.4	Deck deflection (secondary)
19.7～	Shock-induced local vibration at the end of the Deck

3. DESCRIPTION OF ANALYSIS AND AN OUTLINE OF THE PROCEDURE

3.1 Description of Analysis

In this study, the finite element method (here after referred to as FEM) was used as a means of analysis of a combination of bridge piers, foundation and the ground. Of the two conceivable models for finite element method, two- and three-dimensional methods, the two-dimensional model was used as in preceding studies on the ground that a relatively simple analysis model should be used here and that it was easier to handle boundary conditions for having an underground wave propagation infinitely. The analysis program FLUSH was used which well serves general purposes and is highly reliable in FEM analyses in the frequency domain (Lysmer et al, 1975).

The input loading wave (equivalent to the ground excitation wave or acceleration wave in traditional studies) applied for simulation analysis in this study was calculated inversely, as shown in Section 3.2, using the transfer function obtained by two-dimensional FEM model and the acceleration wave at given observing points on the ground or underground. So that the input loading wave in this study was based on a different concept than that used in models in the traditional analyses. It could basically contain influences of vibration propagation from other bridge piers, vibration of decks caused by running vehicles and mobility of the dynamic and static loads themselves producing by running vehicle because it was computed based on the actual data observed on the ground or underground.

It was decided to set an input loading wave in a back analysis as described above because values obtained in analyses and those actually observed did not match well in the past studies and because the use of observed acceleration as an input loading wave as currently practiced would make it difficult to study the vibration propagation mechanism of the ground in a precise manner. The discrepancies between the values obtained in analyses and those observed were considered attributable to lack of proportionality between vibration load applied from vehicles or decks to the bridge pier, and the acceleration response on the bridge pier.

3.2 Analysis Procedure

(1) Calculation of a loading wave

A flowchart of calculating loading wave is shown in Figure 2.

① Transfer function T between the input force point (top of the bridge pier) and the reference point on the ground (representative vibration observing point) is obtained by the point excitation method using a FEM model.

② Loading spectrum A' is obtained by dividing Fourier spectrum B' for acceleration wave B at a representative vibration observing point, by T (A'=B'/T). Then loading wave A at the point where a force was applied is calculated through inverse Fourier analysis for A'. Loading wave A is to be used in this study as an input loading wave.

(2)Establishment of an analytical model focused on vibration characteristics in propagation path

① The spectrum at the reference point (Bi') is computed by multiplying the loading spectrum A' obtained above, by transfer function Ti between the input force point and the reference point (Bi'=A' x Ti).

② The response value (acceleration wave) at the reference point is computed by inverse Fourier transform for Bi' obtained above.

③ A comparison is made in terms of the acceleration values at the reference points between the analysis and the observation. If no correlation is found, the FEM model constants (e.g. a damping ratio) are modified and analyses are repeated until a certain level of correlation is obtained.

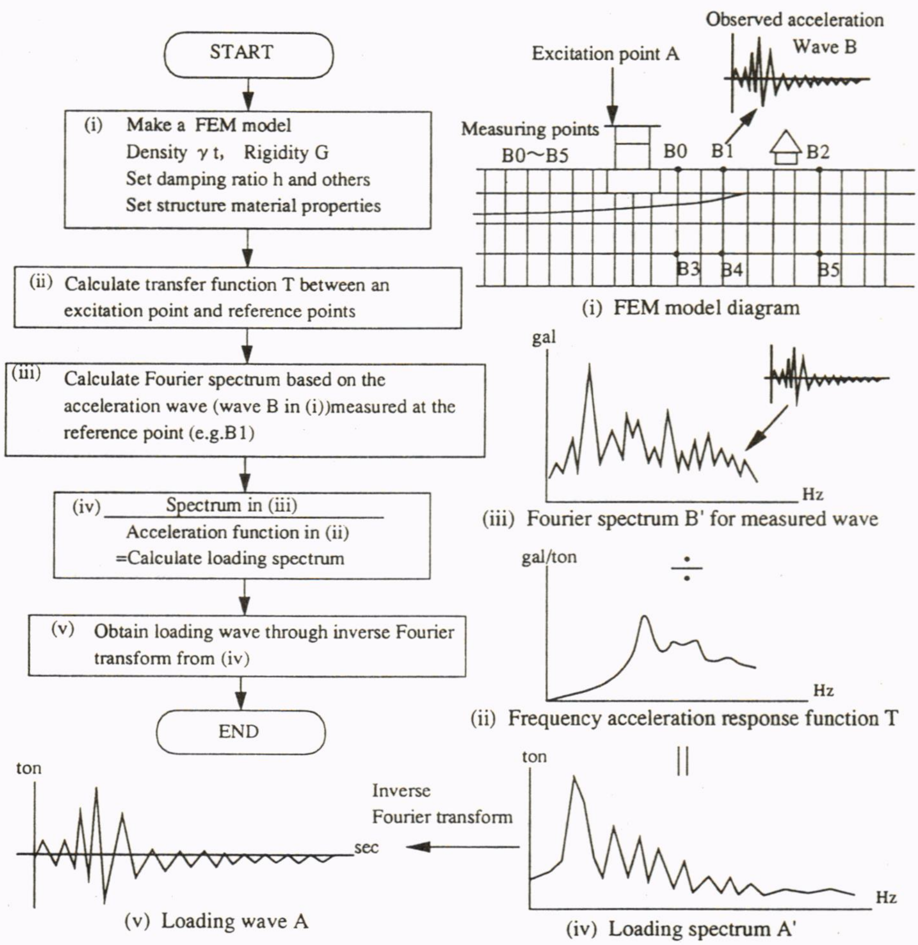


Figure 2 A Flowchart of Processing Input Loading Wave

4. CONDITIONS OF ANALYSIS

4.1 Ground Model

Figure 3 shows the structure/ground model used for the FEM analysis in this study. The analytical model was established based on the ground rigidity, considering irregular factors such as the layer condition identified by boring and the velocity (Vs) classification resulting from a seismic velocity logging test. A transmitting boundary at both ends of the analytical model was set in order to take into account semi-infinite continuity of the ground. Vertically, the model was supposed to reach the layer which was recognized by boring survey as having a certain level of hardness. In order to take into consideration downward semi-infinite continuity of the ground, a viscous boundary was established at the bottom of the model with a view to reducing the influence of wave reflection on the bottom boundary surface.

In setting the maximum thickness of elements, efforts were made so as not to influence propagation of a wave of up to 30Hz, the maximum frequency in the analysis (the significant maximum frequency obtained as a result of observation). It was found unnecessary to consider strain dependent soil properties because the analysis focused on traffic vibration level. The damping ratio h of a slight 2% was initially applied to all layers. The plane strain elements applied to the ground here were basically four-node isoparametric elements (Zienkiewicz and Taylor, 1989).

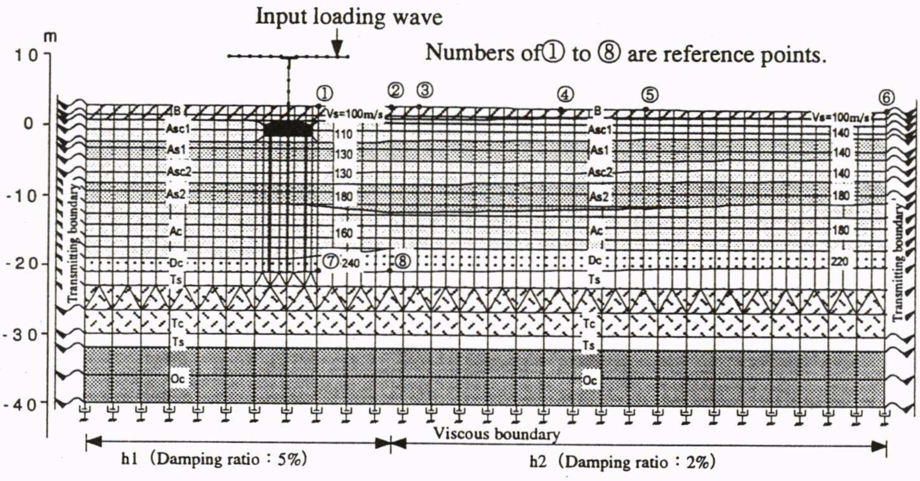


Figure 3 FEM Model of the Structure/Ground for the Study

4.2 Bridge Pier Model

The bridge pier model consisted of members of four-node elements in the footing part as in the ground, and had beam members for the piles and a pier. In view of the difference in continuity with the ground in terms of depth between actual and model members, the equivalent value equal to the depth of footing was used so that the balance between actual and model values in terms of correlation between stiffness and inertial force may be maintained to a certain degree. The reaction force of superstructure was applied to the top of the bridge pier as additional masses.

5. IDENTIFICATION OF VIBRATION PROPAGATION CHARACTERISTICS BASED ON NUMERICAL ANALYSES

5.1 Establishment of an Analytical Model

(1) Setting a loading wave
A loading wave at the excitation point on top of the bridge pier was obtained by the method proposed in this study. An input loading wave is obtained through inverse calculation based on the data observed on the ground (acceleration wave here). That means different input loading waves are obtained from different data observed at several points. Input loading waves should essentially be one and the same regardless of the point where the data was obtained as a basis for setting the loading wave. Observed data, however, is so much influenced by interference of vibration from other bridge piers that the two-dimensional FEM model as used here cannot fully evaluate three-dimensional effects. However, if such an input loading wave can be established as has the dominant frequency component contained in the acceleration wave actually observed on the bridge pier in this study, it seems possible to make a rough estimate of vibration characteristics propagating in the ground.

Based on the above assumption, the input loading wave was obtained by inverse calculation based on the acceleration wave observed at several points. As a result, it was found that dominant frequency components on top of the bridge pier (3 to 4Hz and 10 to 14Hz) matched that for observed acceleration better in the case where the wave observed at point ② (about 15m from the bridge pier) was used as a basis than in any other case. At point ②, the observed maximum acceleration value was largest, and the acceleration calculated using the input loading wave was almost identical to that actually observed. In view of the above, the input loading wave obtained based on the value observed at point ② was used for the model of this study.

Figure 4 shows the process of establishing the input loading wave described above. Loading waves obtained based on the acceleration waves observed at points other than point ② had

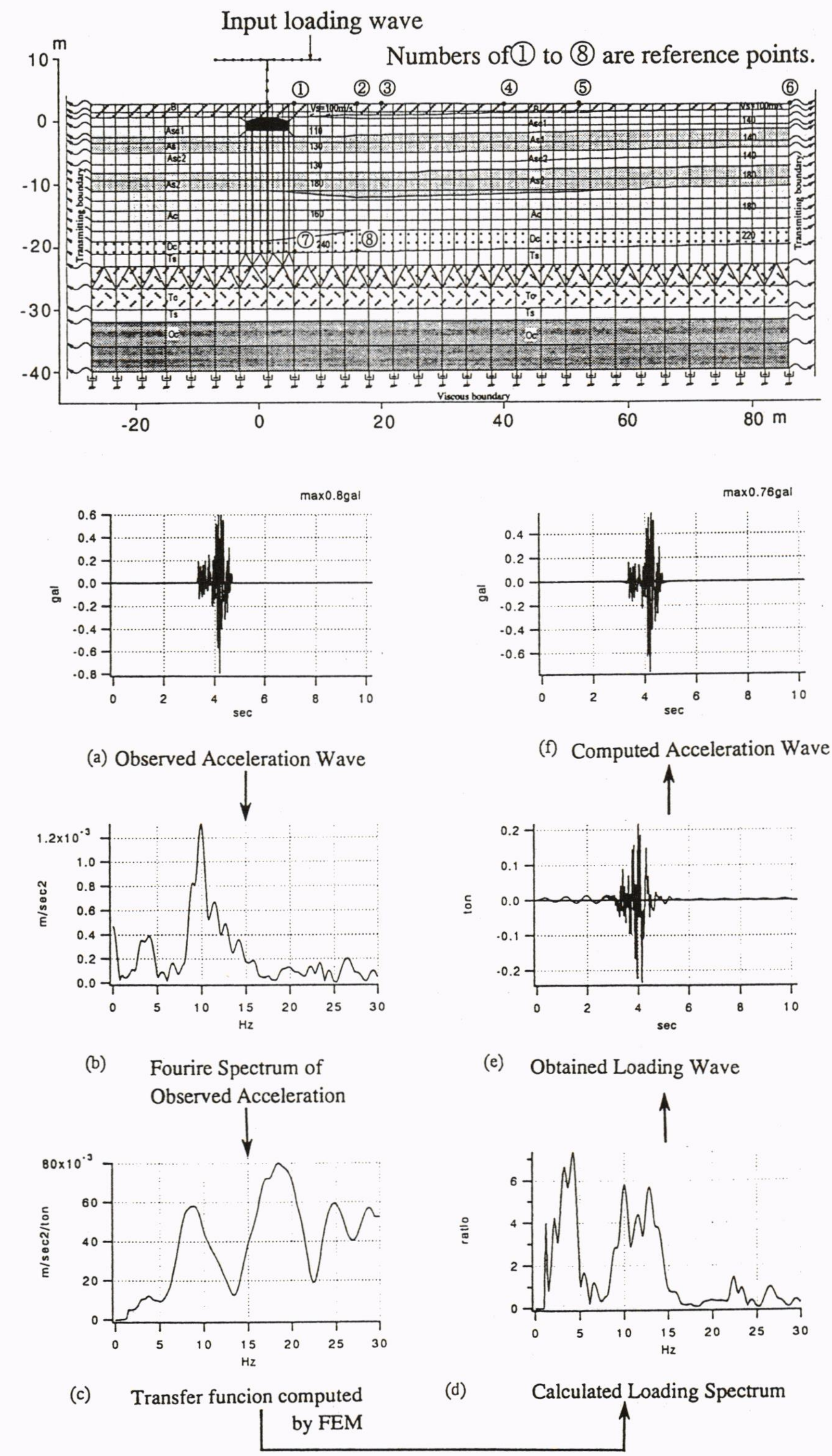


Figure 4 Computed Input Load Wave by the Proposed Producer at Point ②

different maximum values and dominant frequency characteristics, but related envelope looked similar.

(2)Comparison in terms of acceleration at the reference points between observed and calculated data

A comparison between calculated and observed data in terms of acceleration distribution in distance on the ground surface (at points ①, ②, ④, ⑤ and ⑥) is shown in Figure 5. The calculated data was obtained by an analysis using the input loading wave described above. A ground damping ratio of 5% was set for the range h1 and 2% for h2 as shown in the figure. These values and ranges were considered appropriate because damping effect was found large near the bridge pier based on the observed acceleration distribution in distance. Figure 6 shows acceleration Fourier spectrum obtained in the analysis at points ① through ⑥. Figure 5 also presents characteristics of surface wave acceleration distribution in distance obtained by the formula shown below as the equation (1) (Yamahara, 1974). A comparison among the value obtained by the formula, that obtained in the analysis and the observed value indicates that surface acceleration remained almost the same in the range from the bridge pier to point ④, that acceleration distribution in distance in the range was greatly influenced by surface wave and that there was little amplifying effect due to the ground.

$$A = Ao \times r^{-n} \times \exp(-\alpha \times r) \tag{1}$$

Where A is the acceleration at a distance of r from the source of vibration, Ao is the acceleration at the source of vibration, n is a constant number of 0.5 (surface wave) and α is a coefficient for acceleration distribution in distance.

At point ⑤, the values obtained by both observation and analysis were greater than those obtained by the theoretical formula shown above. It was assumed from the acceleration Fourier spectrum that ground amplification in the frequency bandwidth of 4Hz and 13 to 14Hz, which were major vibration components of the input loading wave, caused the surface acceleration to increase. The difference between observed and calculated values was greater at point ⑤ than at other points probably because the value set for physical properties of the ground having particularly soft surface, used in the analytical model, was inappropriate. Again at point ⑥, values obtained in both observation and analysis were larger than those calculated by theoretical vibration formula for surface wave. This may have been caused by amplification effect due to the ground. However, as shown in Figure 6, vibration at point ⑥, unlike at point ⑤, was influenced more by amplification in the bandwidth near 4Hz than by that in the 10 to 14Hz bandwidth.

5.2 Identification of Vibration Propagation Characteristics

The maximum acceleration distribution contour obtained in the analysis are shown in Figure 7.

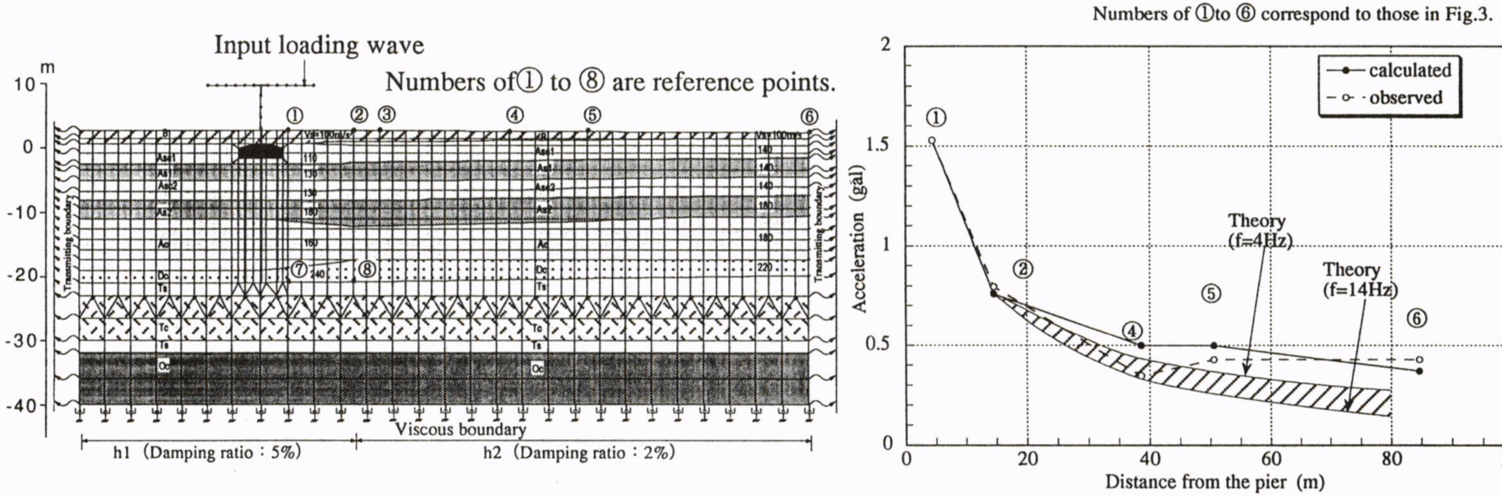


Figure 5 Analytically Obtained Acceleration Distribution in Distance in Comparison with Observed one

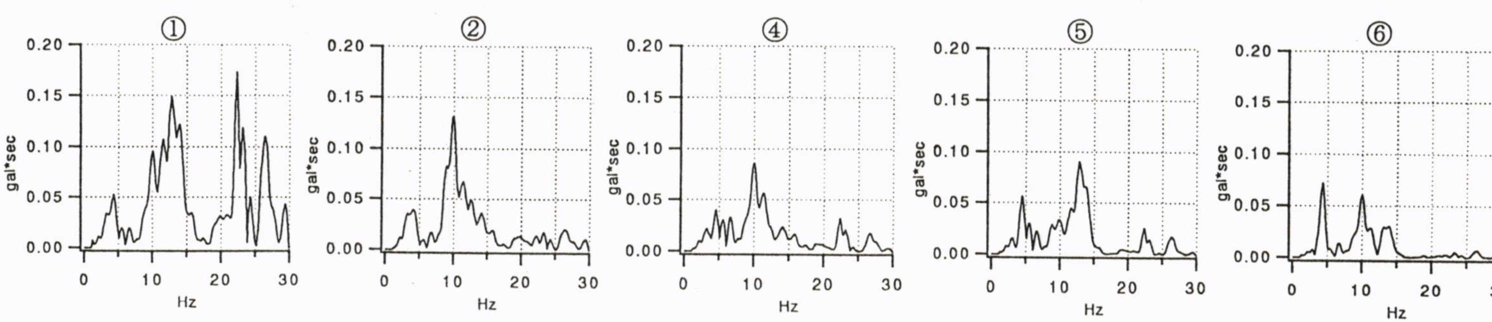


Figure 6 Acceleration Fourier Spectram Obtained by Analysis

The figure shows observed values as well. Following findings are obtained from the analysis.

1) The acceleration value was greater in the earth filling portion on the footing. This is attributable to resonance as a result of the fact that a theoretical predominant frequency of 12.5Hz for the earth filling portion on the footing [=Vs/4H=100(m/s)/4 x 2 (m), where Vs: shear wave velocity, and H: thickness of the layer] was identical to the peak frequency (10 to 14Hz) for the input loading wave shown in Figure 4. Judging from the contour diagram near the bridge pier, vibration transmitted from the bridge pier to surrounding ground seems to have come mainly from either the earth filling portion on the footing described above and the footing as represented by changes in contours near the footing. It also seems that only a small vibratory force was transmitted from piles.

2) The further from the source (footing and the earth filling portion on the footing), the smaller the magnitude of vibration becomes. Acceleration distribution in distance was, however, small on the B layer (Vs=100m/s), A_{sc1} layer (Vs=110m/s) and A_{s1} layer/A_{sc1} layer (Vs=130m/s for both), all of which were surface layer having a small velocity of S-wave. This indicates that vibration propagated mainly through this soft surface soil.

3) In deeper soil layer, vibration was greater at point ⑦ which was near the bridge pier than at point ⑧ located slightly further. This may be because vibration propagating from the footing (the source) mentioned in (1) above was great while piles worked to counter the vibration of the ground, especially at the end. There seems to have been a phenomenon as shown in the schematic diagram given in Figure 8.

4) The acceleration wave of the ground obtained in the analysis had frequencies of 4Hz and 10 to 14Hz, which was identical to the peak frequencies of the input loading wave shown in Figure 4. Judging from such frequency properties and the contour diagram shown in Figures 7 and 8, it seemed inconceivable that in relation to the traffic vibration in this study site the ground (especially the surface stratum) worked as a filter to amplify vibration in the range between the bridge pier to point ④ except the earth filling portion immediately over the footing. It was assumed that frequency components of vibration due to running vehicles and decks propagated without any amplification. At point ⑤ or at further points, amplification by soft ground particularly in the upper part was expected. Amplification of frequencies of 4Hz and 10 to 14Hz, the predominant frequencies for the input loading wave, seems to have resulted in a considerably loose contour as shown in Figure 7.

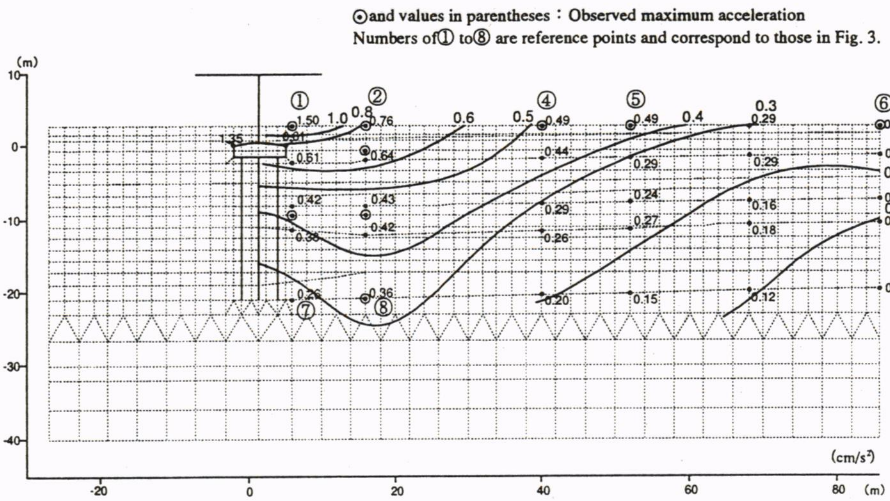
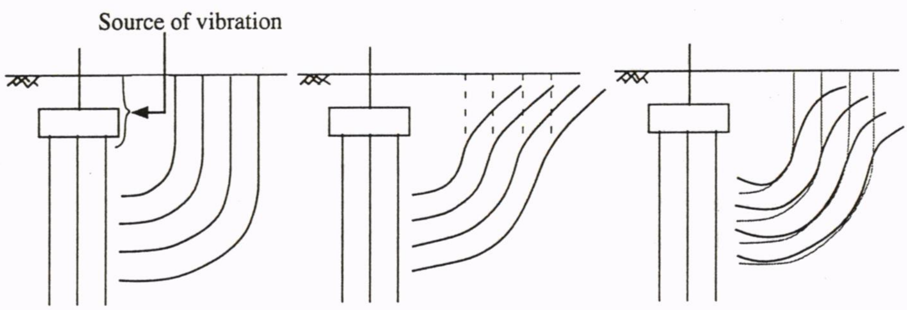


Figure 7 Maximum Acceleration Distribution Contour Obtained in the Analysis



1. Contour diagram showing acceleration distribution in distance from the source (in the case of a body wave)
2. Countour diagram where acceleration distribution in distance for surface soil was small
3. Countour diagram where piles restrain surrounding ground (including case 2.) (used in the calculation this time)

Figure 8 Schematic Contour Diagram of Vibration Propagation

6. ESTIMATING EFFECTS OF VIBRATION REDUCTION WORK

6.1 Overview

As there are few analytical studies about the effects of reduction of traffic vibration on viaducts in its propagation path, analyses were carried out here by the method proposed in this paper, in order to identity a qualitative tendency. Generally conceivable vibration countermeasures in the propagation path includes ground improvement, underground vibration-proof walls, vibration-proof hollow trenches, isolation of footing from surrounding ground, and buffer zones. The study here focused on underground vibration-proof wall for which few analytical approaches have been done, because buffer zones could be utilized effectively to set the wall and FEM could deal with a model for this study aim. Vibration-proof hollow trenches were considered most effective as they would guarantee the greatest vibration reduction effect (Yoshioka and Ishizaki, 1980), but was finally eliminated from the analyses in this study for the following reasons. In the specific place where the analyses were carried out, a relatively low frequency component of 3 to 4Hz was prevalent because of vibration characteristics for viaducts. It is generally stated that a trench must be substantially deeper than what is required to control relatively high frequency components involved in traffic vibration on surface roads, if it is to work effectively against vibration (Woods,1968). Specifically, in view of the theoretical vibration isolation effect of a hollow trench (Yamahara, 1974), it needs the depth larger than the length of the propagating wave by a factor of 0.3 if it is to reduce frequency and amplitude of vibration by half. This means a trench having a depth of 10m will be required in the place under review based on the ground properties described earlier (wave length: 130/4=32.5m where, propagation speed for the surface stratum:130m/s, and dominant frequency:4Hz). Judging from its sustainability and problems involved in its maintenance, use of a vibration-proof hollow trench having a depth of about 10m was considered impractical.

Table 3 shows the dimensions of vibration-proof walls used in the analyses of their effects.

Table 3 Dimensions of Vibration-proof Work (Underground Vibration-proof Walls)

Case	Type	Width (m)	Length (m)	G (tf/m2)	γ t (tf/m2)	Poisson's ratio	Damping ratio
A	Rigid wall (benchmark model)	0.5	23	820,000	1.600	0.167	0.02
B	Less rigid wall (rigidity of 1/120000 of Case A)	0.5	23	6.8	0.015	0.400	0.02
C	Less rigid wall (rigidity of 1/10000 of Case A)	0.5	23	82	0.015	0.400	0.02
D	Less rigid wall (rigidity of 1/1000 of Case A)	0.5	23	820	0.015	0.400	0.02

Two types of walls, stiff walls made from concrete and those built with less rigid materials like urethane inserted between a pair of steel sheet panel, were used. The vibration-proof wall was built at point ② in the analysis model shown in Figure 3 as an element model with a width of 50cm to the depth where the bridge pier piles were embedded into the ground. Similar approaches using EPS block in field have been conducted by Hayakawa and Matsui (1996) for surface roads.

6.2 Comparison of Effects

Figure 9 is a comparison in acceleration distribution in distance of the vibration between cases for which vibration-proof walls were installed and the case without such countermeasure. In relation to the ground composition and frequency properties of the traffic vibration at this study site, it was found that the less rigid wall in Case B was more effective than any other type in reducing vibration. It was also confirmed, however, that the rigid wall in Case A and the less rigid wall in Case C could also reduce vibration. Figure 10 presents a comparison in transfer function between Case B, as a typical case, and the ground surface for which no vibration-proof countermeasures were taken. It shows remarkable reduction of vibration in the frequency range exceeding 4 to 5Hz. In addition, similar studies of transfer function in Cases C and D have revealed that lowering of rigidity of the wall lead to a lower transfer level of vibration having relatively high frequency, at the back of the vibration-proof wall. In front of the wall (on the bridge pier side), in contrast, lower rigidity resulted in greater effect of vibration and higher frequency level after countermeasures were taken (refer to Figure 10). Acceleration distribution in distance as shown in Figure 9 suggests that the nearer to the source (the bridge pier in this particular case) the vibration-proof wall is installed, the greater acceleration decrease effect can generally be obtained.

The contour diagram for the maximum acceleration before and after vibration-reduction work (Case B is given as a typical example in Figure 11) reveals that the lower the rigidity of the vibration-proof wall, the smaller the vibration transmitted to the rear of the wall. As a result, acceleration distribution in distance as shown in Figure 9 becomes remarkable in the ground (surface and underground) at the rear of the wall or further points. The results of the analysis in Case D show no improvement in acceleration decrease, transfer function and contour of maximum acceleration from the status where no vibration-reduction works were taken. It was thus found that the wall with such rigidity could provide no satisfactory effects against the ground properties of this study site.

Judging from the discussions so far, the vibration-proof wall could substantially reduce vibration if it had lower level of rigidity. On the other hand, a certain degree of rigidity (rigid enough to resist the soil and to remain un-defected or unchanged semi-eternally) will be required for installing and maintaining the wall and for reducing vibration continuously. Location of the wall can be built also has an influence on the degree and scope of vibration

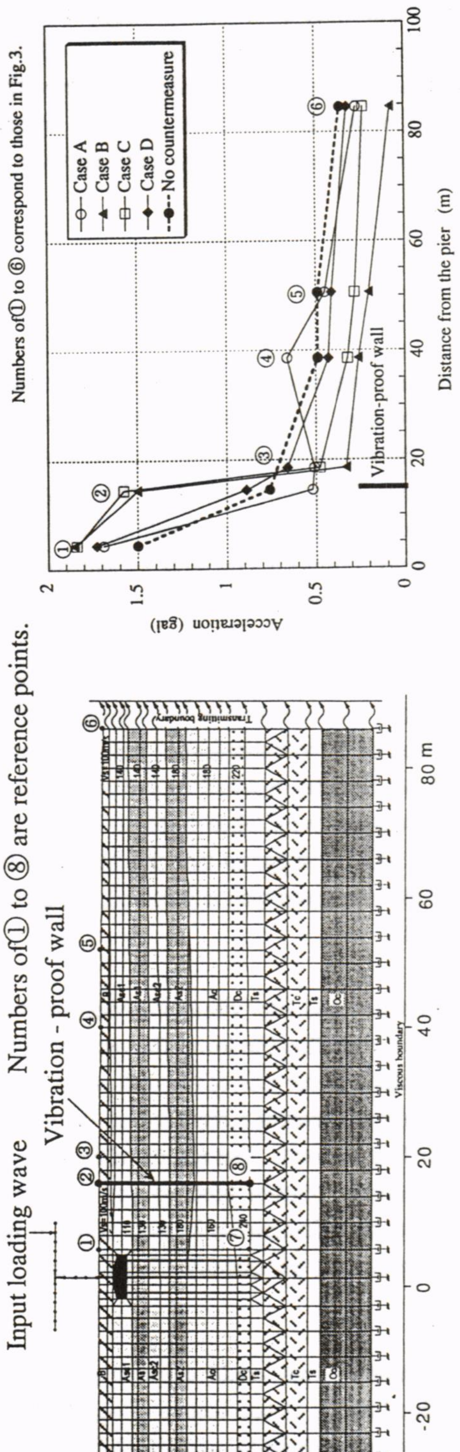


Figure 9 Acceleration Distributions in Distance among Different Types of Countermeasure

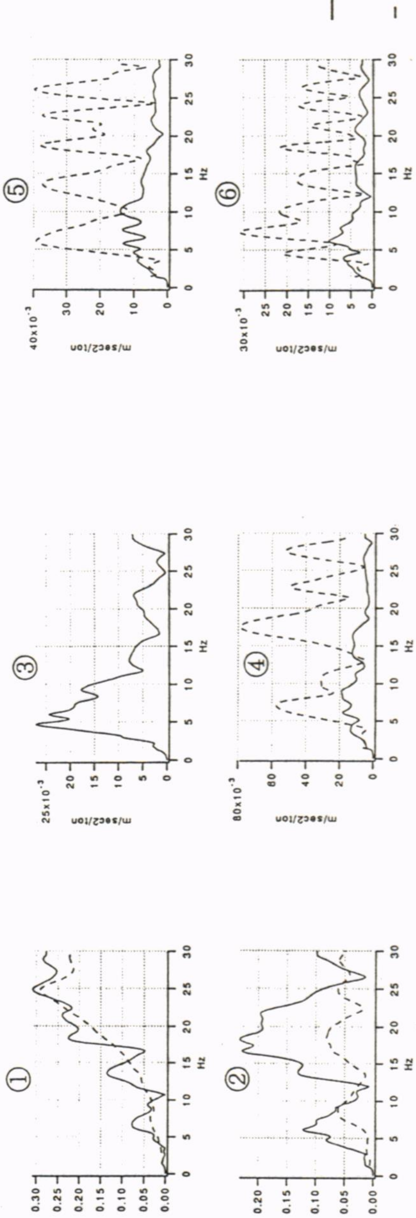


Figure 10 Comparsion of Transfer Function between Case B and No Countermeasure

reduction. In actually implementing vibration reduction work, therefore, an appropriate vibration control method should be selected, considering not only guaranteed effect but also economy and ease of construction.

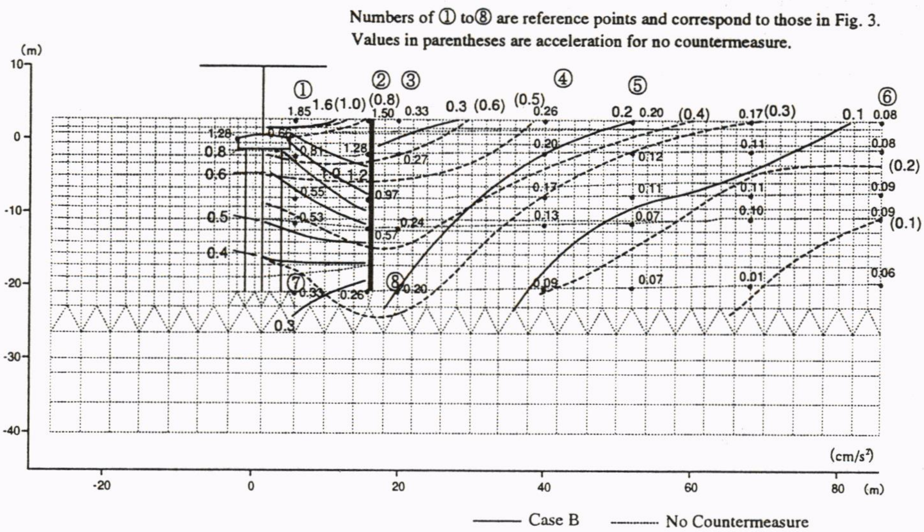


Figure 11 Maximum Acceleration Distribution Contour for Case B and No Countermeasure Obtained in the Analysis

7. CLOSING REMARKS

This paper, in an attempt to identify the difficult-to-quantify problem of traffic vibration propagation to the ground, has presented a new means of effectively defining input loading wave in the two-dimensional finite element method based on the data observed for real bridge structures, on the basis of the results of existing experiments and analytical studies. Then it has shown the results of an attempt by such a new method to grasp the propagation mechanism at certain points. The paper also provided the effects of a countermeasure for reducing vibration in its propagation path using vibration-proof walls as no specific study has yet been made about vibration reduction in propagation path.

Road traffic vibration caused by overcrowding of cities affects human living. Therefore, fundamental solutions of the problem should incorporate a viewpoint of roadside residents suffering from the trouble. Countermeasures applied jointless deck for superstructure alone cannot fully reduce vibration, though they may have certain effect as a vibration reducing method at the source. A combination of such countermeasures with vibration reduction works in propagation path as described in this paper is also considered useful. In order to grasp, characteristics of vibration propagation in the ground, and effects of vibration-proof countermeasures, it is necessary to accumulate observed vibration data on a wide scale and, then it can be understood based on such data, with considering dimensions of structures and the ground at a certain site. In reality, however, road traffic vibration is such a complicated mixture of properties of vibration at the source, of the structure which transmits vibration, and of the ground that evaluation or assessment based on the existing data is not necessarily provide appropriate conclusions. In this connection, the selection of an effective method based on numerical analyses as described in this paper provides a useful method. In the future, more cases will be studied based on the concept introduced in this paper, and a more reliable way of selecting the method for controlling vibration in the propagation path will be pursued. Few studies have been made about traffic vibration propagation mechanism on viaducts and about effects of vibration-proof countermeasures in propagation path, the authors would therefore appreciate this paper will provide useful data for further studies.

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