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Reduction of footing response to man-made excitations by using a Wave Impeding Barrier

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Ground vibrations due to surface loads and incident waves are considered. The reduction of the vibration is achieved by using a wave impeding barrier. The considered structure is the foundation of a building. For incident waves from below the current investigation shows that a wave impeding barrier is not a proper reduction measure. Ground vibrations due to the surface load can be strongly reduced by the wave barrier. The effectiveness of the wave barrier can be improved by increasing its length and stiffness.

Key Words: Traffic induced vibrations, response reduction, wave impeding, BE-FE coupling

1. Introduction

Man-made vibrations in soil can be caused by many human activities, such as construction works, production activities in factories, or also by traffics. However, their effects are still not well investigated. Most of the investigations on the effects of ground motions put more emphasize on earthquakes as the source. The reason for this is that earthquakes frequently cause spectacular damages to structures and loss of human lives. Recently, man-made ground vibrations receive more attentions because people are more aware of their life qualities. The awareness becomes stronger due to the increase use of high-speed traffic, which may produce strong ground vibrations in its surrounding. A good collection of works on the effect of man-made vibrations is given in the proceedings of the recent WAVE 2000-meeting [1].

In order to reduce ground vibrations one can install wave barriers in the soil. The wave barriers change the propagation of the waves, and cause therefore a reduction in certain area. A trench can be used [2]. However, depending on its dimension it could be difficult in practice to keep it open. A row of piles or a wall can also be used as wave barrier [3), 4]. In order to increase the effectiveness one can use gas cushion screen as wall in soil [5].

In this work a wave impeding barrier is considered. It is based on the dynamic transmitting behavior of a soil layer over bedrock [6], which is characterized by the cut-off frequency of the layer between the bedrock and the soil surface. In order to create artificial bedrock a concrete block is installed horizontally at a certain depth location below the structure to be protected. The sources of the ground vibrations are surface and underground traffic.

2. Numerical modelling and analyzed reduction measure

In order to introduce the basic idea of the reduction approach used in this work, the wave propagation in a soil layer is considered. The source is a surface strip rigid foundation with a width of 3m, which is subjected by a vertical unit harmonic load with a frequency of 16Hz. The soil has a density of 1800 kg/m^3, a Poisson's ratio of 0.33 and a shear wave velocity of 172 m/s. It is assumed that the soil has no material damping. LR is the length of the Rayleigh wavelength and has the value of 10 m. The dimensionless calculation is performed in the frequency domain by using the boundary element method [7]. Fig. 1 shows the development of the steady-state response amplitude with the distance from the source for different layer thickness H. The reason for the different amplitude development (wave propagation) is that the vibration transmitting behaviour of a soil layer over bedrock depends not only on the soil material and the direction of the excitation, but also significantly on the natural frequencies of the soil layer or the corresponding critical thickness [8]. For a vertically acting source the critical thickness is determined by the propagation velocity of the compression waves c_p and the excitation frequency f. In the considered case the smallest critical thickness $H_{cr} = c_p/4f = 0.53$ LR. If the layer thickness is smaller than the critical thickness, there is no wave propagation and the soil behaves like a finite system (the case H = 0.1 LR in Fig. 1). Only, when the layer thickness is larger than the critical thickness, can the waves propagate laterally. In case of H = 0.6 LR the amplitude development shows a behaviour similar to resonance, because H is very closed to the critical thickness. With increasing thickness the layer behaviour become closer to the behaviour of a half-space.

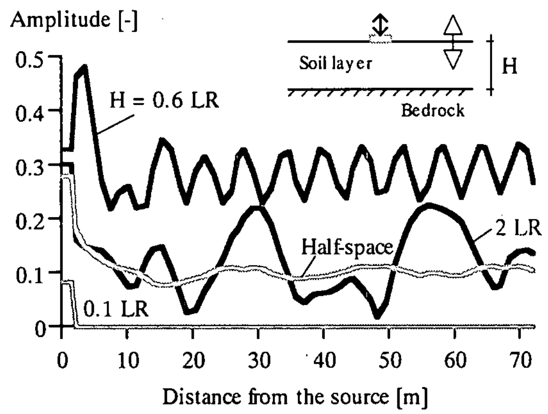

Fig. 1 Influence of the layer thickness H on the development of the surface amplitude

Since it is not always possible to have proper bedrock at certain location, one can create artificial bedrock by installing a stiff obstacle in the soil. The artificial bedrock can be placed at the proper depth below the vibration source or below the object, which is to be protected, as it is considered in this work. In the latest case the critical thickness is determined by the propagation velocity of the dominant incoming waves. In case of a transient source the activated waves are much complex, and the dominant frequency of the source can only be used as an orientation for the determination of the critical thickness. In order to clarify the frequency content of the dominant waves at certain location field test is necessary.

In the considered investigation a reinforced concrete strip footing of 5 m width is located at 22.5 m center-to-center distance from a single-track railway embankment as shown in Fig. 2. The footing thickness is assumed to be 0.80 m and embedded in a uniform half space with its top surface coincides with the ground surface. The embankment layer thickness is 1 m above the ground surface. It has a top surface width of 6 m and its bottom width is 8 m. A plain concrete Wave Impeding Barrier (WIB) is embedded inside the soil at a depth dw of 1.5 m from the ground surface. The material properties of the uniform half space, embankment layer, footing and WIB are given in Table 1.

The numerical analysis is performed using a time domain computer program that utilizes a coupled Boundary Element-Finite Element (BE-FE) method developed earlier by the first author [9]. In this method the problem in hand is divided into two parts. The uniform half-space is modelled by boundary elements. The non-homogenous part that can be an embankment, a dam, a footing or a structure is modelled by finite elements. The compatibility and equilibrium conditions are satisfied along the common interface between the two parts.

The governing equation of motion of the finite element domain due to a time dependant applied load P(t) can be expressed and partitioned for the considered time step N as:

$$\begin{bmatrix} M_{oo} & M_{oi} \\ M_{io} & M_{ii} \end{bmatrix} \begin{Bmatrix} \ddot{u}_o \\ \ddot{u}_i \end{Bmatrix}^N + \begin{bmatrix} C_{oo} & C_{oi} \\ C_{io} & C_{ii} \end{bmatrix} \begin{Bmatrix} \dot{u}_o \\ \dot{u}_i \end{Bmatrix}^N + \begin{bmatrix} K_{oo} & K_{oi} \\ K_{io} & K_{ii} \end{bmatrix} \begin{Bmatrix} u_o \\ u_i \end{Bmatrix}^N = \begin{Bmatrix} P_o \\ P_i \end{Bmatrix}^N \tag{1}$$

, where M, C, and K are the mass, damping and stiffness matrices, respectively. The vectors contain the nodal accelerations, velocities and displacements, respectively. The interface variables are denoted by the subscript i and the other variables are denoted by the subscript o.

Similarly, using the subscript i for the interface nodes and s for the surface nodes of the boundary element domain, the governing equation can be written as:

$$\begin{bmatrix} U^1_{ss} & U^1_{si} \\ U^1_{is} & U^1_{ii} \end{bmatrix} \begin{Bmatrix} t_s \\ t_i \end{Bmatrix}^N = \begin{bmatrix} T^1_{ss} & T^1_{si} \\ T^1_{is} & T^1_{ii} \end{bmatrix} \begin{Bmatrix} u_s \\ u_i \end{Bmatrix}^N + \begin{Bmatrix} E_s \\ E_i \end{Bmatrix}^N \tag{2}$$

, where $\mathbf{U}$ and $\mathbf{T}$ are the coefficient matrices of the system. For the time step N, all traction vectors $\mathbf{t}$, (step 1 to step N), and previous displacement vectors $\mathbf{u}$, (step1 to Step N-1), are known. Employing the traction- free condition on the surface, and solving for the interface node variables, the nodal traction along the interface can be expressed as:

$$\{t_i\}^N = [K_b]\{u_i\}^N - \{E_{bu}\}^N, \tag{3}$$

where $[K_b]$ and $\{E_{bu}\}$ are the results from the necessary matrix and vector operations. The traction in Eq. (3) is transformed into the nodal force vector $\{P_{bb}\}$ by applying the principle of virtual work and written as:

$$\{P_{bb}\}^N = [K_{bb}]\{u_i\}^N - \{F_b\}^N, \tag{4}$$

where $[K_{bb}]$ and $\{F_b\}$ represent $[K_b]$ and $\{E_{bu}\}$ after transformation, respectively. Then, the principle of weighted residual is applied along the interface to obtain the coupled equation of motion as:

$$\begin{bmatrix} K_{oo} & K_{oi} \\ K_{io} & K_{ii} + K_{bb} \end{bmatrix}^* \begin{Bmatrix} u_o \\ u_i \end{Bmatrix}^N = \begin{Bmatrix} P_o \\ P_{bb} \end{Bmatrix}^{N*} \tag{5}$$

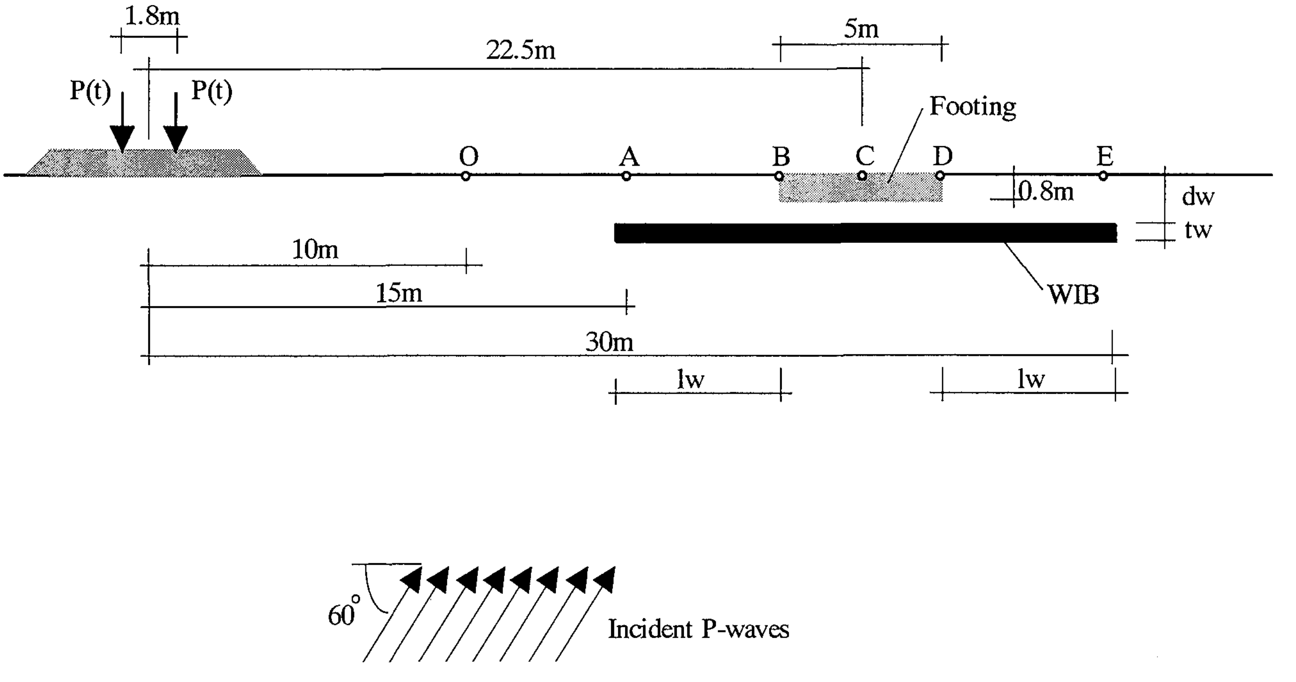

Fig. 2 Embankment-soil-footing system

, where the superscript * denotes the resulting coupled terms including the contributions of the boundary element domain as well as the mass and damping matrices of the finite element domain. More detailed formulation can be found elsewhere [9]. This program has been applied successfully for the investigation of wave propagation and traffic-induced vibrations for simple models and a practical case of railway embankment [9),10)].

In the modelling, the embankment-soil-footing system shown in Fig. 2 is divided into a finite element domain and a boundary element domain. The finite element domain is designed to include the embankment, footing, the WIB and enough depth inside the half-space. Therefore, the finite element discretized area had a horizontal width of 40 m and a vertical depth of 6 m inside the half-space. A total number of 343 FEM nodes are used, out of them 42 nodes on the free surface and 55 nodes on the interface between the finite element domain and the boundary element domain. The boundary element domain included the rest of the surrounding infinite uniform half-space. The BE discretization on the free surface is truncated at enough distance far from the finite domain to exclude any side effects on the wave propagation characteristics.

The traffic load is represented by two-concentrated line loads simultaneously applied on the embankment. The horizontal distance between the two loads is 1.8 m as shown in Fig. 2. The time history of each load consists of four consecutive impulses; each impulse has a time period of 0.02 seconds and 1 MN in amplitude (Fig. 3(a)). The separation time between each two consecutive impulses is 0.02 second. Consequently, the total time period of the applied load is 0.14 seconds.

Table 1 Material Properties and shear wave speeds

Material	Mass density ρ [t/m^3]	Shear wave speed Cs [m/s]	Poisson's ratio ν [-]
Half space	2.0	150	0.333
Embankment	2.0	250	0.333
Reinforced Concrete footing	2.50	2400	0.20
Wave Impeding Barrier (WIB)	2.40	2200	0.20

In order to present the characteristic of the vibration source and the effect of the ground vibrations due to the wave propagation from the source, the response of a single degree of freedom (SDOF) system with a fixed base is used. The maximum response of the SDOF system of certain natural frequency and damping ratio to the vibration source or the ground vibrations is displayed in a response spectrum as a function of the natural frequency of the system. Even though this response spectrum is not a real spectrum, like for example a Fourier spectrum, the term response spectrum is well established in Earthquake Engineering, and it is also adopted in this work.

The equation of motion of a SDOF system with ground excitation $a_g(t)$ as shown in Fig. 3(b) is

$$m\,a(t) + c\,v(t) + k\,u(t) = -m\,a_g(t) \qquad (6)$$

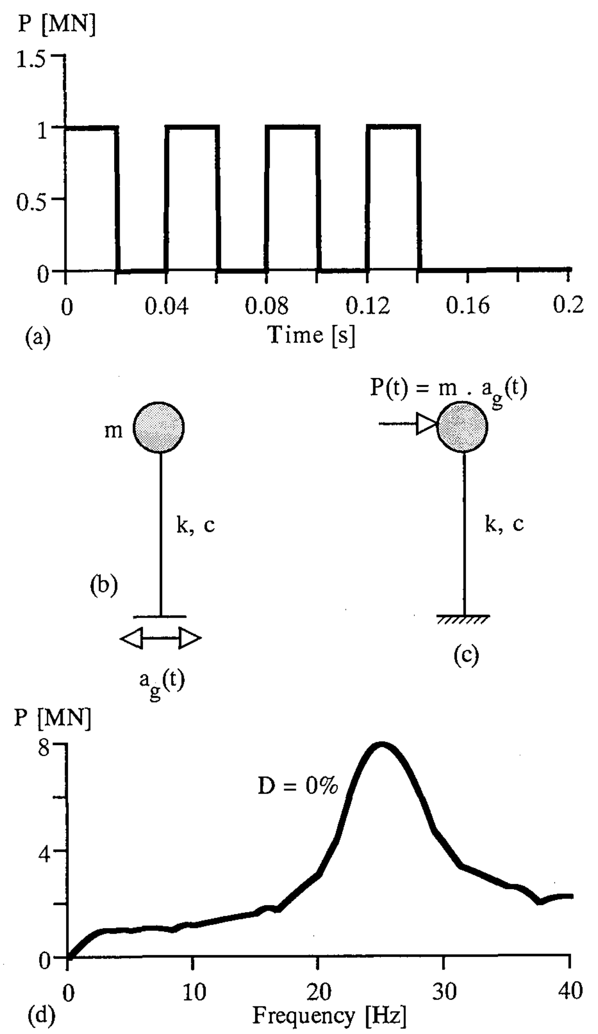

Fig 3 (a) The surface load; (b) and (c) SDOF system with ground and direct excitation; (d) the response spectrum of the surface load

, where m, c and k are the mass, the damping coefficient and the stiffness of the system, respectively. a(t), v(t) and u(t) are the relative acceleration, velocity and displacement due to the ground motions, respectively. By introducing the damping ratio D and the circular natural frequency ω of the system the equation becomes

$$a(t) + 2D\omega\, v(t) + \omega^2\, u(t) = -a_g(t) \tag{7}$$

If the system properties D and ω are known, one needs not to determine the values of m, c, and k, in order to be able to obtain the response acceleration a(t) of the system to the ground vibrations $a_g(t)$.

Fig. 3(c) shows that the ground motions $a_g(t)$ can also be considered as a direct load $P(t) = m \cdot a_g(t)$. With a unit mass the maximum response acceleration represents then the maximum occurring force in the system in the direction of the load. Since the response is determined by the relation between the system and load properties, the response spectrum of the applied load in Fig. 3(d) reflects the characteristics of the load. In order to exclude any additional influence on the response only an undamped SDOF system is considered (D = 0 %) in this work. The response spectrum shows that the load posses a wide range of frequency contents with a predominate range of 20-35 Hz, which covers the frequency range that is expected to be caused by the heavy axle train passage [11), 12)].

The investigation in this work aims at finding out the effect of footing itself on the soil surface response as well as the effect of the WIB on the surface and footing responses. The variable parameters are the WIB length lw on both sides of the footing and the WIB thickness tw as given in Fig. 2. The considered cases are given in Table 2 where Case P1 represents the case of uniform half-space subjected to the traffic load without footing and WIB inside it. Case P2 represents the case of half-space with footing and without WIB. Case P3 is the case when the WIB of lw = 5.0 m and tw = 0.50 m is used without footing. In Case P4, a WIB of the same size is used while the footing exists.

In Case P5, a row of vertical concrete column is assumed to be constructed under the footing edge on both sides to connect footing edges with the corresponding points on the WIB top surface. The columns are embedded (driven) at regular intervals of 1m in the normal direction to the 2D space in Figure 2. Each column has a square cross section of 0.20 m side length and is modeled as beam element with considering axial deformation. In Cases P6 and P7, the WIB is used with different values for lw and tw, respectively.

Table 2 Cases of analysis under dynamic loads on the track

Case No.	lw [m]	tw [m]	footing
P1	uniform half-space without WIB and footing		
P2	without WIB		exist
P3	5.0	0.50	not exist
P4	5.0	0.50	exist
P5	5.0	0.50	exist with columns
P6	7.0	0.50	exist
P7	5.0	1.50	exist

3. Results and discussion

3.1 Effects of WIB and footing

The surface response is picked up in terms of acceleration time history at certain selected locations at different distance from embankment central line as shown in Fig. 2 and given in Table 3. The vertical acceleration time histories at locations O, C and D are depicted in Fig. 4 for Cases P1, P3 and P4. At all locations, as expected the highest amplitude occurs in Case P1 (half-space

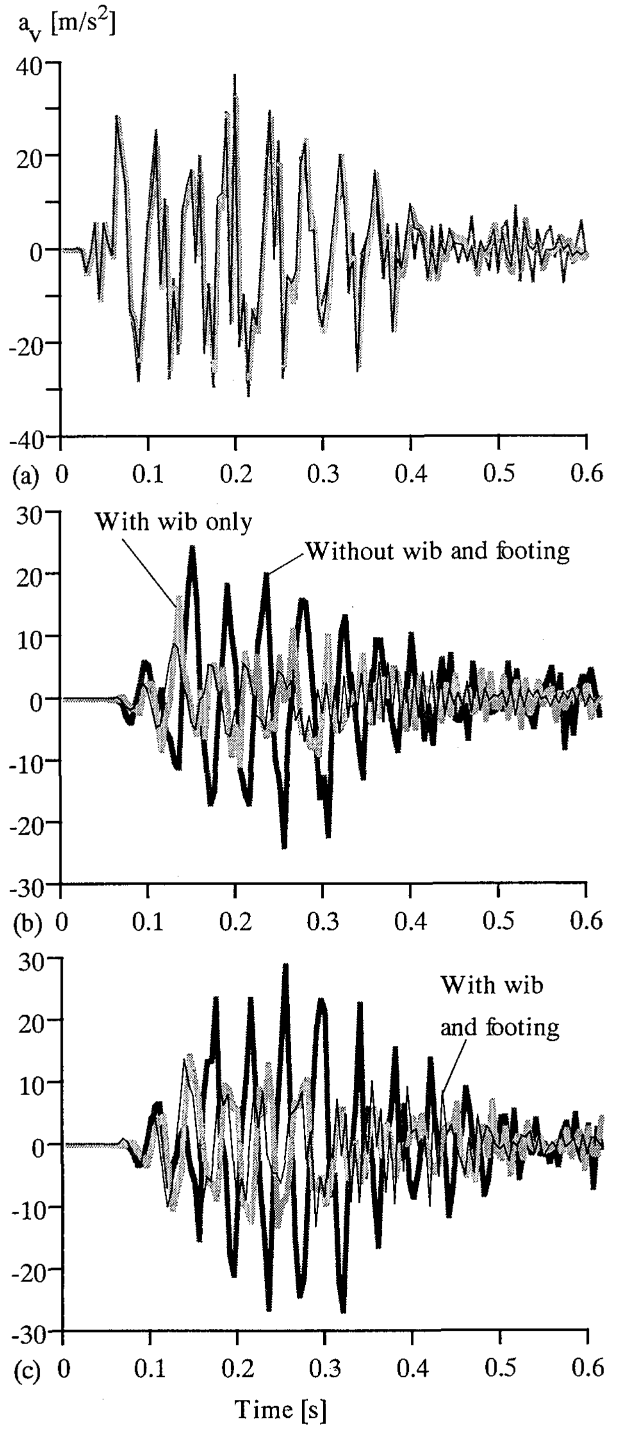

Fig. 4(a)-(c) Effect of the wib and footing on the vertical vibrations at the location of (a) 10m, (b) 22.5m and (c) 25m from the central line of the embankment

without WIB and footing). When the WIB of lw = 5 m is used in Case P3, the response amplitude is reduced at location C and D. In Case P4 the response amplitude is greatly reduced at location C and considerably reduced at location D when the WIB and footing are exist due to the additional effect from the kinematics interaction between the footing and the surrounding soil. At location O, the response time histories were almost identical in all three cases with very little reduction in Case P4.

The horizontal responses that result mainly due to the wave scattering show no much difference at all locations in all cases of analyses. Therefore, the horizontal responses will be omitted herein and only the vertical responses will be focused in this work. The above behavior indicates that the WIB and the footing have a local strong reduction effect on the vertical response amplitudes and the maximum reduction effect could be achieved at the center of the footing.

Table 3 Selected locations for response picked up

Location	Distance from Embankment central line [m]	Remarks
O	10.0	Intermediate location
A	15.0	5 m from footing left edge
B	20.0	At footing left edge
C	22.5	At center of footing
D	25.0	At Footing right edge
E	30.0	5 m from footing right edge

3.2 Effects of WIB without and with columns

The response spectra at locations C, D and E for the load cases P2, P4 and P5 are shown in Fig. 5. For Case P2 without WIB it can be remarked that predominant frequency amplitudes (about 25 Hz) at location D and E are twice as large as the amplitude at location C. This means that the amplitude increases as the distance from the source increases. This can be owed to the effect of the interference between the body waves and generated surface waves that propagate with low attenuation rates with distance. In Case P4 (with WIB), about 40 % reduction in the amplitude was achieved at location C and D. At location E the amplitude reduction was about 30 %. This reflects the effect of the wave barrier that locally increases the stiffness inside the soil and impedes the passing of the major parts of the waves.

In Case P5 (WIB with columns), about 65 % reduction in the central frequency amplitude was achieved at location C and D. About 53 % reduction at location E was resulted at the central frequency amplitude. Compared to the Case P4 the reduction effect in Case P5 was more pronounced in the frequency range from around 20Hz to around 30Hz. In addition, the case WIB with columns also caused a remarkable reduction in all considered higher frequency range above 30Hz.

This enhancement is due to the effect of the columns that connected the footing to the WIB. The WIB therefore becomes more stable with less vibration level and thus more effective. The former investigations [13] showed that the less the barrier vibrates the more effective it will be.

It should be mentioned here that the resulted axial and shear forces at each column were not large and could be safely resisted by the assumed concrete cross section. In addition, no more

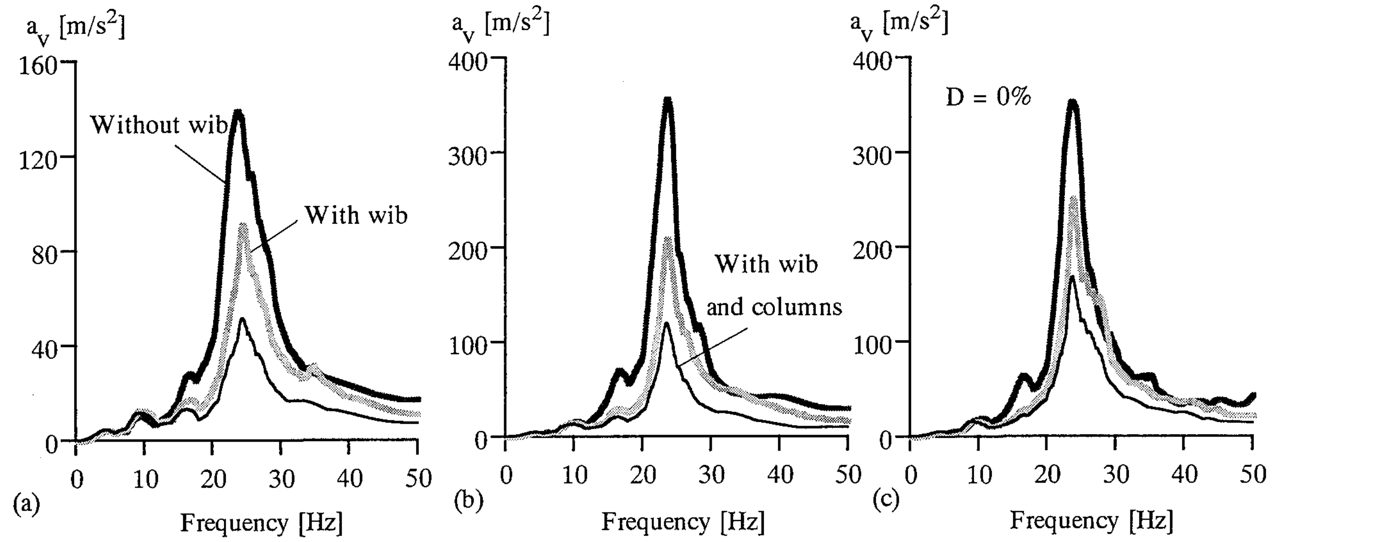

Fig. 5(a)-(c) Effect of the wib with columns on the vertical vibrations at the location (a) 22.5m, (b) 25m and (c) 30m from the central line of the embankment

enhancements could be achieved for larger size of the column (not shown here). Therefore, it could be stated that the connection between the footing and the WIB is the most effective factor, and the rigidity of the connecting columns has little influence.

3.3 Effects of WIB length and thickness

The response spectra at location C, D and E that resulted from the load Case P6 are compared to those resulted from cases P2 and P4 in Fig. 6. The comparison showed that the increase of WIB length in Case P6 (lw = 7 m) resulted in higher response reduction than Case P4 (lw = 5 m). The highest reduction resulted at location E and the reduction at location D was higher than at location C. The amplitudes at wider frequency range at location E (12-50 Hz) were drastically suppressed. It can be stated that the longer WIB length extended the reduction effect towards longer surface distance and suppressed wider frequency range.

When the WIB thickness tw was increased to be 1.5 m in Case P7, the resulted amplitude reduction was higher than in Case P4 (tw = 0.5 m) at all location as shown in Fig. 7. In addition, the predominant frequency was slightly shifted towards the lower frequency (about 24 Hz). This can be explained as follows: by increasing the WIB thickness it became very rigid layer compared to the surrounding soil and therefore, it could be assumed as rigid bedrock. Consequently, the soil layer above the WIB vibrated almost as a fixed base layer with lower amplitudes. The dominant frequency of the source can still be seen.

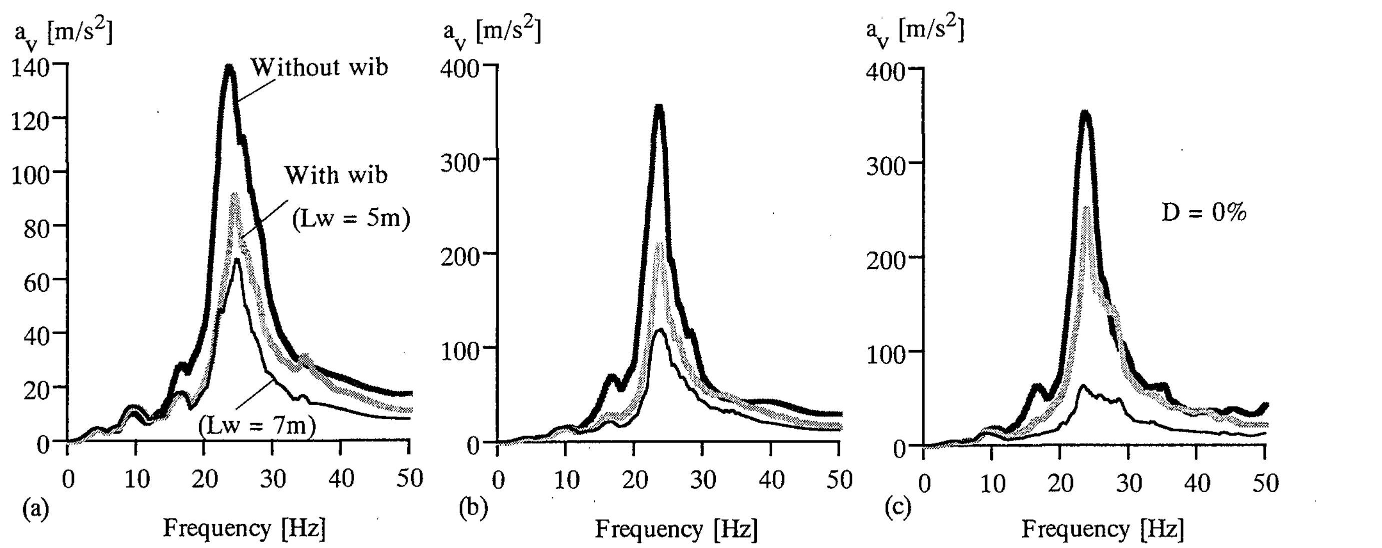

Fig. 6.(a)-(c) Effect of the length Lw of the wib on the vertical vibrations at the location (a) 22.5m, (b) 25m and (c) 30m from the central line of the embankment

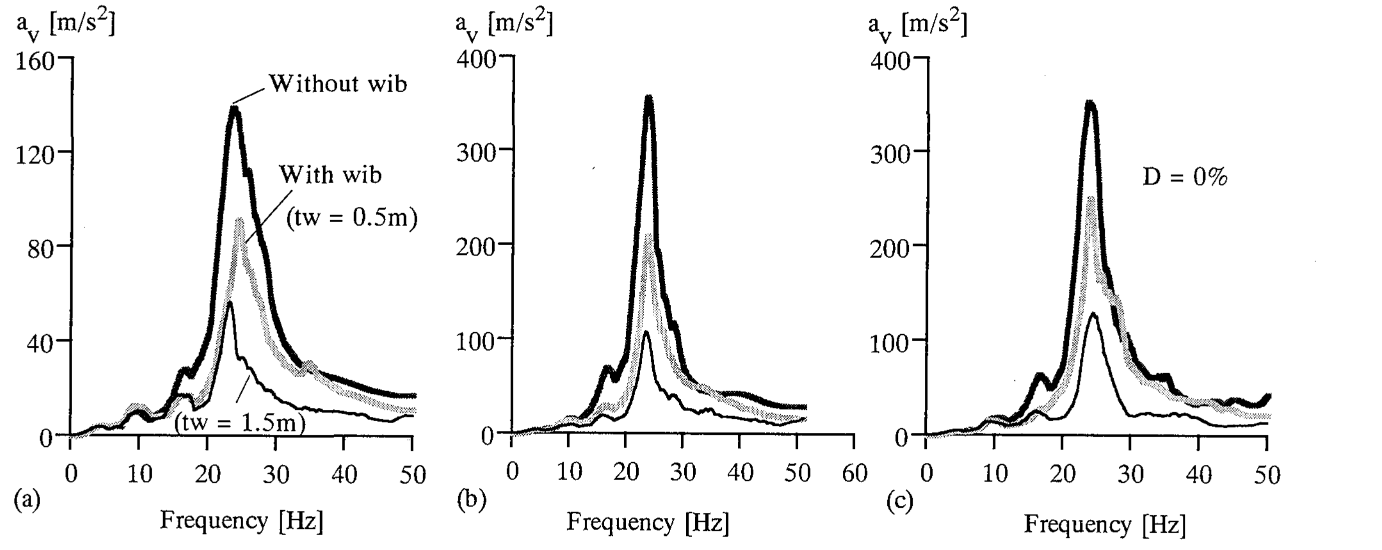

Fig. 7(a)-(c) Effect of the thickness tw of the wib on the vertical vibrations at the location (a) 22.5m, (b) 25m and (c) 30m from the central line of the embankment

It should be mentioned here that increasing the WIB length or thickness would lead to additional costs for the soil excavation and WIB construction. Therefore, a feasibility study should be performed for each practical case to obtain the optimum reduction effect with the lowest economical cost.

3.4 Response to incident waves

In addition to the above-mentioned cases of applied surface trainloads, the effectiveness of the WIB in case of wave propagation due to train passage in tunnels is studied. The effect of underground subway trains is represented by an artificial excitation as incident waves coming from the soil far field as shown in Fig. 2. This excitation should have a wider and higher frequency range than the surface applied loads [11), 12)]. The incident wave is simulated by a set of four consecutive P-wave Ricker wavelets [14), 15)]. Each of them has a certain predominant frequency and a specific time for maximum response as given in Table 4. The time history of the incident displacement and the response spectrum are shown in Fig. 8. The incident displacement at the far field is adjusted to be 0.5 cm and the angle of incidence is selected to be 60° with horizontal direction. In order to exclude the wave scattering effects the embankment layer is removed from the model during the analysis of the case of incident waves.

Table 4 Simulated Excitation by Ricker wavelets

Wavelet number	Predominant frequency f_o [Hz]	Time of maximum amplitude t_s [s]
1	50	0.05
2	40	0.06
3	33	0.08
4	20	0.10

The vertical acceleration time histories at location C and D are shown in Fig. 9(a), (b), respectively and the response spectra at locations C, D and E are depicted in Fig. 9(c), (d), (e), respectively. It is remarked that very small reduction in amplitude could be achieved by the WIB in the frequency range up to 50 HZ. For higher frequencies no reduction is remarked. This can be owed to the fact that the WIB itself is subjected to a direct motion due to

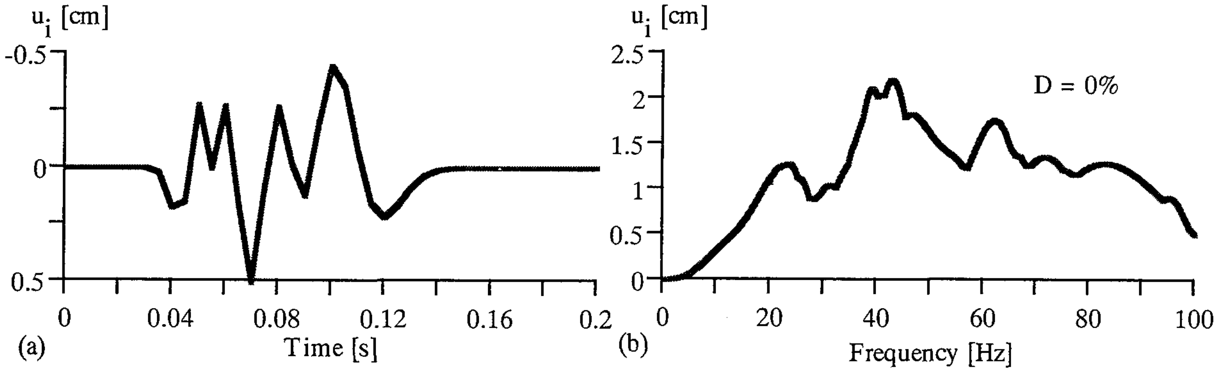

Fig. 8(a) and (b) The incident displacement; (a) its time history and (b) response spectrum

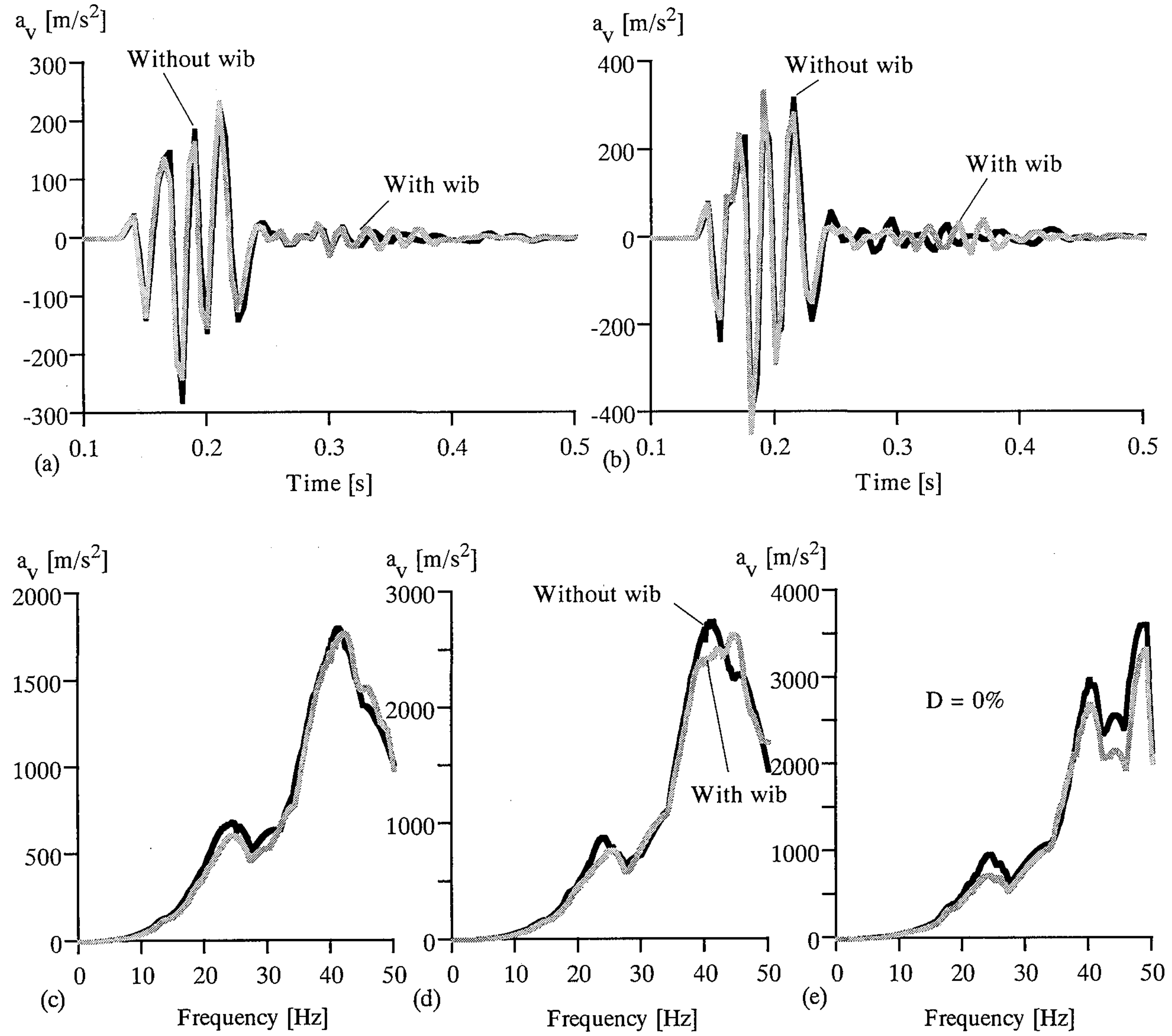

Fig. 9(a)-(e) Effect of the wib on the vertical vibrations at the location of (a) 22.5m, (b) 25m, (c) 22.5m, (d) 25m and (e) 30m from the central line of the embankment due to the incident compressive waves

the incident waves coming from below and is not stationary in place as the case of surface load. Therefore, the WIB had lost in this case the advantage of its rigidity and could not totally reflect back the incident wave. However, in order to clarify the effectiveness of WIB in case of incident waves further investigations are necessary.

4. Conclusions

A theoretical investigation on the response reduction of a footing subjected to a traffic induced vibration is presented. The cases of surface load due to train passage on the soil surface as well as the case of train in tunnels (incident waves) are studied. The reduction measure is a Wave Impeding Barrier (WIB) placed at a certain depth inside the soil below the footing. A time domain BE-FE coupling computer program is utilized for the analysis in two dimensions. Based on the obtained numerical results the following conclusions could be stated:

- The WIB and the footing have a local strong reduction effect on the vertical response amplitudes of the surface and the maximum reduction effect could be achieved at the center of the footing.
- A reduction up to 40% of the footing vertical response could be achieved by the WIB effect. When the WIB is connected to the footing by vertical columns, the footing response was reduced by about 65 %. The horizontal response showed no much difference.
- Increasing the WIB length led to a higher reduction effect and extended the WIB reduction effect to longer distance on the surface and widened the reduced frequency range. A reduction up to 80 % can be achieved.

- Increasing the WIB thickness resulted in higher reduction effect but this would increase the excavation and construction coast. In the considered cases a reduction up to 70 % can be obtained.

- The WIB is effective in cases of surface applied loads. In the case of incident waves other reduction measure should be used.

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