16TH EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING
THESSALONIKI 18 - 21 JUNE 2018

OPTIMALLY LOCATED WAVE BARRIERS FOR REDUCING HORIZONTAL VIBRATIONS INDUCED BY EARTHQUAKE

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ABSTRACT

The present study proposes a new methodology to reduce free-field horizontal ground motions induced by earthquake loading. In this methodology, a specific volume of stiff wave barriers is placed optimally in a specific domain of the soil profile, namely design domain. It is aimed at mitigating the amount of earthquake induced energy reaching the soil surface, and consequently the structural vibrations .The optimal layout of the wave barriers is obtained considering the topology optimization concepts and utilizing genetic algorithm. The complex calculations for finding the fitness values of the optimal design procedure is performed using dynamic finite element analysis that is coupled efficiently to the genetic algorithm. Using the developed methodology, the effect of various parameters such as impedance mismatch between the soil and wave barriers and the frequency content of the seismic loadings is investigated. It is observed that the suggested methodology can find the effective wave barrier layouts with high potential capacity for mitigating the free-field accelerations, especially for the earthquake loadings that have high-frequency content.

Keywords: Wave Barrier; Vibration control; Wave propagation; Genetic algorithm; Finite element.

1. INTRODUCTION

As a way of reducing the soil surface vibration due to the surface waves mostly induced by machine foundation, pile driving and train, using wave barriers can be an effective solution. Woods (1968) and Richart et al. (1970) were among the scholars who first introduced the use of wave barriers (WBs) to control vibrations in the 1970s. Since then, there has been an extensive research investigating the impact of various parameters among which the position, shape, and material of WBs are the most important ones. Ahmad et al. (1996) proposed using circle trenches to mitigate ground disturbance due to surface waves generated from the machine foundation of medium to high frequency vibration. They found that the amplitude reduction ratio relies heavily on the distance of WBs to the source and the depth of the trenches. In another study, Al-Hussaini et al. (1996), by introducing some dimensionless parameters such as the ratio of the shear wave velocities of the barrier material and soil , pointed out that in-filled material, which were chosen to be concrete and soil-bentonite, can affect the reduction ratios significantly. As well as concrete and soil-bentonite, water and Geofoam are also studied as the potential materials that can be used in WBs (Alzawi and Hesham El Naggar 2011; Persson, Persson, and Sandberg 2016). Additionally, some other research has recently been focusing on the effect of WBs on mitigating earthquake loads. Miniaci et al. (2016) employed large scale metamaterial within

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the ground to protect structures from the bulk and surface waves due to earthquakes. Flora et al. (2016) and Lombardi et al. (2015) utilized soft WBs under a foundation to screen the vibrations. They made the case that by increase of impedance mismatch between the soft WBs and surrounding soil medium, higher reduction ratios could be achieved.

Finding the optimum position layout of WBs within the soil domain could be categorized in the field of topology optimization. More and more studies are being conducted to find the optimum layout of some stiff or soft material in the design domain. Van hoorickx et al. (2016) obtained different patterns of stiff inclusions within the soft domain under the variety of frequencies. They first defined the insertion loss expression and then by gradient base approach, found the optimum patterns. In another study, Jensen (2007) investigated the efficacy of replacing some parts of the design domain with specific materials in order to scatter and absorb the incoming waves. Furthermore, as a way of conducting topology optimization, Genetic Algorithm optimization (GA) can be utilized (Hajela, Lee, and Lin 1993). In this context, Rafiee-Dehkharghani et al. (2015) found the optimum positions and diameters of some circular holes in a rectangular plate, which was loaded at one side and clamped at the other side. Rafiee-Dehkharghani et al. (2014) proposed the optimum multi-layered stress wave attenuators under impulse loading. They developed a coupled GA-FE procedure to obtain the optimal layer thickness and material properties.

In the present study, the capability of optimally located concrete WBs is explored for the purpose of reducing the vibrations induced by body waves generated due to earthquakes. The soil profile, which at the specific presumed depth rested on the bedrock, is taken as the design domain in which the concrete blocks are to be injected. The major aim of the investigation is reducing the mean horizontal acceleration of a specific region/zone at top of the soil surface. The soil deposit and the WBs are discretized into a finite mesh of elements using finite element (FE) software ABAQUS, which is coupled with MATLAB to employ GA toolbox. The optimum layouts are first found in the frequency domain and then the efficiency of obtained patterns is verified in the time domain using Ricker wavelet (Ricker 1943, 1944).

2. FINITE ELEMENT MODELING

As illustrated in Figure. 1, the soil medium is presumed to be an infinite horizontal deposit resting on a rigid bedrock at the given depth of 50 m; and the design domain along with the protected area are determined. The soil medium is assumed to be an elastic matrix with the constant damping ratio of two percent, which is the damping ratio of most various kinds of soil at lower level of shear strain (Towhata 2008). In other words, variable damping ratio of the soil with respect to the shear strain has not been taken into account, for the primary objective of the paper is to explore the mitigating capacity of the wave barriers by their presence in optimal location and making high impedance mismatch with the surrounding soil medium. Furthermore, shear modulus of the soil deposit is supposed to change in terms of depth according to Gazetas (1982). Since the matrix is elastic, the shear modulus and Young's modulus at every depth can be specified. Table. 1 shows the corresponding Young's modulus, mass density, and Poisson's ratio of each sub-layer of 5 meters depth. Concrete, with $E=2e10$ Pa, $\rho=2400$ kg/m^3 and $\upsilon=0.2$ is utilized as the material properties of the inclusions (wave barriers). Both matrix and inclusions are modeled using four-node plane strain elements with reduced integration (element CPE4R from ABAQUS element library).

Table 1. Mechanical Properties of Sub-layers

z (m)	E (Mpa)
0-5	42.4
5-10	59.1
10-15	78.8
15-20	101.1
20-25	126.4
25-30	154.4
30-35	185.2
35-40	218.8
40-45	255.1
45-50	294.4

As the incoming waves impinge upon the wave barriers, some portion of the wave is either refracted or reflected. Thus, Lysmer-Kuhlemeyer type of boundary conditions (Lysmer and R. L Kuhlemeyer 1969) should be used in both sides of the model. Moreover, these boundaries must be placed at a specific distance from the middle of the soil medium (Roesset and Ettouney 1977). In the present paper, the horizontal dimension of the soil profile is 20 times of the depth of the bedrock, and in ABAQUS, CINPE4 elements, four-node continuum plane strain infinite element, are used to absorb the outgoing P- and S-waves. The damping constants of Lysmer-Kuhlemeyer boundaries are determined as follows (Equations 1 and 2):

$$\mathrm{C_P} = \rho c_p A \tag{1}$$

$$\mathrm{C_S} = \rho c_s A \tag{2}$$

in which A is the corresponding area of each node and compressional and shear wave velocities are $c_p = \sqrt{E(1-\upsilon)/(\rho(1+\upsilon)(1-2\upsilon))}$ and $c_s = \sqrt{E/(2\rho(1+\nu))}$, respectively.

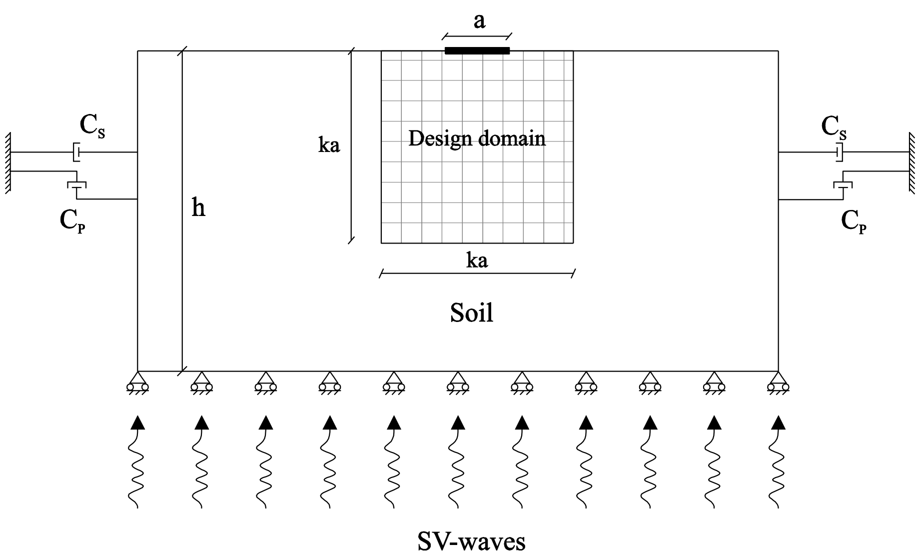


Figure 1. FE Details

2.1 Model verification

Gazetas (1982) has solved the one dimensional horizontal equation of motion analytically with variable shear modulus (Equation 3):

$$\rho \frac{\partial^2 u}{\partial t^2} = \frac{\partial}{\partial z}\left(G \frac{\partial u}{\partial z}\right) \tag{3}$$

In a soil profile rested on a rigid bedrock, the shear stress at the free surface and the horizontal displacement at the base is zero (Equations 4 and 5):

$$u(H,t) = 0 \tag{4}$$

$$G(z) \frac{\partial u(z,t)}{\partial z}\bigg|_{z=0} = 0 \tag{5}$$

The variation of the shear wave velocity with respect to the depth is assumed to be linear, i.e. $c(z) = c_{s0}(1 + bz)$ in which c_{s0} is shear wave velocity at the surface of the soil, the transfer function relating the horizontal displacement of the bottom and top of the soil deposit thus can be obtained by (Equation 6):

$$TF = \frac{2q}{(-0.5 + q)(1 + bH)^{-0.5-q} + (0.5 + q)(1 + bH)^{-0.5+q}} \tag{6}$$

in which $q = \sqrt{o.25 - \omega^2 / c_{0s}{}^2 b^2 (1 + 2i\xi_s)}$, where ξ_s is damping ratio and ω is circular frequency of the harmonic excitation . A good agreement between the results of FE simulation and the solution proposed by Gazetas (1982) can be seen in Figure 2.

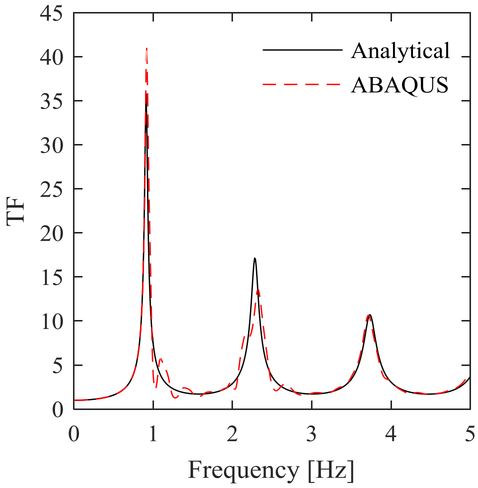


Figure 2. Comparison between Gazetas solution (Gazetas 1982) and FE results.

3. METHODOLOGY

In the present study, the optimum pattern of WBs is first determined under harmonic excitation for a specific range of frequencies in the frequency domain, and the capability of the optimally located WBs is then explored in the time domain. As long as the predominant frequency of many earthquake loadings, specifically the horizontal component, is lower than 5 Hz, the optimization procedure has been conducted for the frequency range of 0.01-5 Hz.

As represented in Figure 1, the design domain in which the WBs are to be inserted is presumed to be a square part of the soil medium with the dimension of ka where k is a constant and a stands for the width of the area to be protected. Another part of an optimization problem is specifying the optimization function that here is defined as (Equation 7)::

$$F = TF_i \qquad \text{for} \quad i = 1, n \tag{7}$$

where, TF is transfer function at the natural frequencies of the soil deposit. The main reason behind considering such a definition for optimization function is that at the natural frequencies of the soil deposit, maximum amplification occurs, thus it is of value to reduce the amplifications at these specific frequencies.

Other parameters that must be defined are the solution space and constraints. In this paper, the design domain is considered to be a square with the dimensions of $ka \times ka$, where $k = 3$ and $a = 10\,m$. Moreover, the maximum number of WB blocks of the dimensions $2 \times 2\,m^2$ is supposed to be 20. Since the problem is symmetric with respect to the middle vertical axis (Figure 3), only half of the solution space is considered in the optimum design procedure, and then the obtained layout of WBs is mirrored with respect to the vertical axis of symmetry.

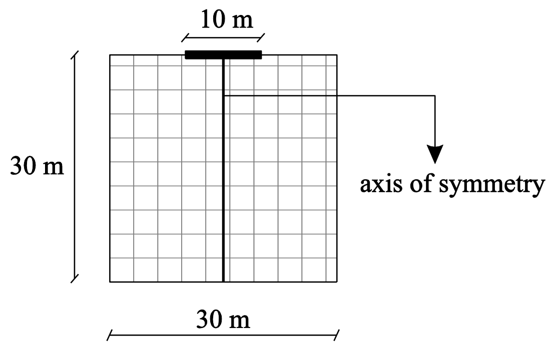


Figure 3. Solution Space

Scripting with Python language in ABAQUS (2014) is employed to assign the WBs material to the proposed area by GA during each generation. Besides, the python script runs the model and reads the results from the FE output file (with .odb extension) and send them to the GA code. At the final stage, when the convergence criteria is met, the optimum pattern of WBs is then reported by GA.

After finding the layout of WBs at each frequency, it is necessary to explore the obtained solution efficacy in the time domain. To this end, the excitation is presumed to be Ricker wavelet with various predominant frequency. The Ricker wavelet is the solution of Stokes differential equation (Ricker 1943, 1944) by considering the effect of Newtonian viscosity, thus it could be indicative of seismic waves (Wang 2015). The Ricker wavelet in the time domain is written as (Equation 8):

$$r(t) = \left(1 - \frac{1}{2}\omega_p^2(t-t_0)^2\right)\exp[-\frac{1}{4}\omega_p^2(t-t_0)^2] \tag{8}$$

where ω_p is the predominant circular frequency of the excitation and t_0 is the time when the maximum normalized amplitude occurs. Figure 4 illustrates a typical Riker wavelet with the $\omega_p = 4\pi$ rad/s and $t_0 = 1$ s.

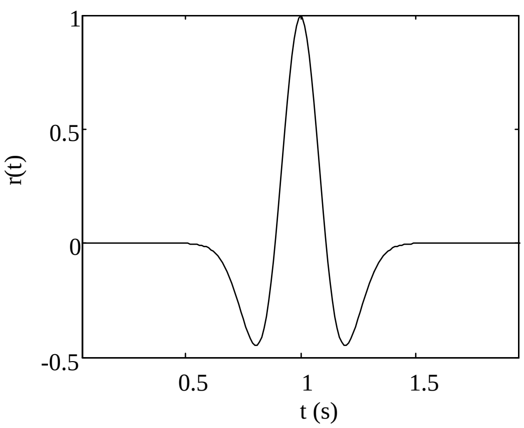


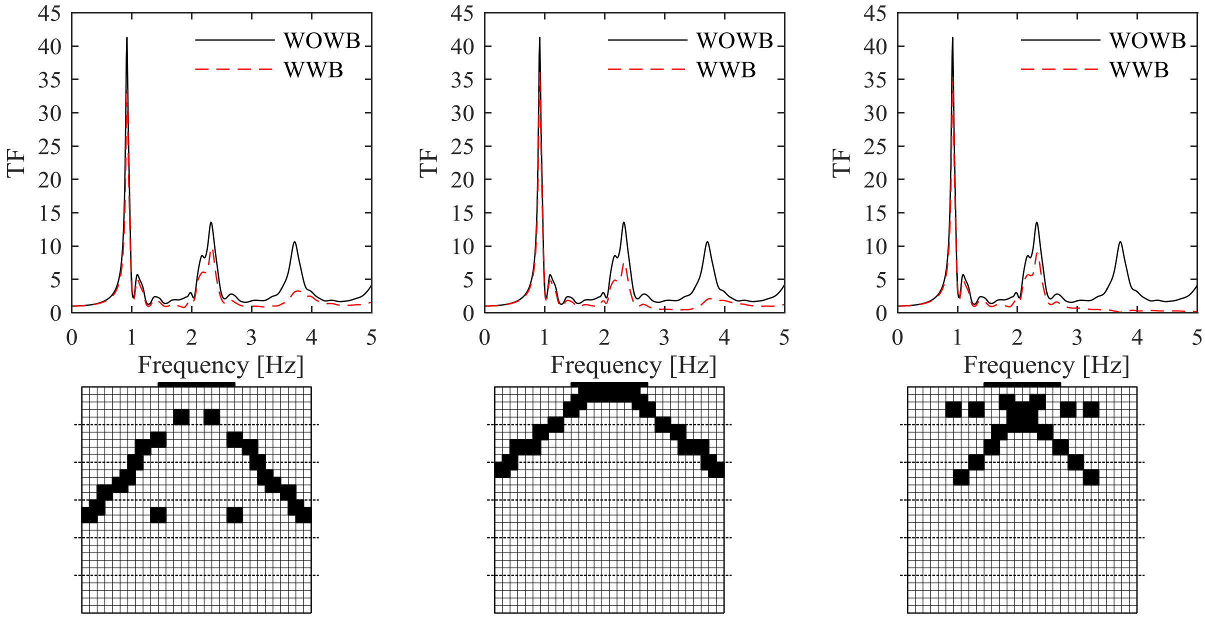
Figure 4. Ricker Wavelet (with the predominant frequency of 2 Hz)

4. RESULTS AND DISCUSSIONS

In what follows, the results in the frequency domain are firstly pointed out and then the verification of the obtained patterns is investigated in the time domain. The natural frequencies of the soil deposit with the heterogeneity rate of 2, those of which fall into the range of 0.01-5Hz, are 0.92 Hz, 2.32 Hz and 3.72Hz. Thus, three optimization problems are to be solved to determine the optimum layouts of WBs according to each natural frequency. Figure 1 presents the position of WBs along with the analogy between the results of transfer functions in the models with (WWB) and without WBs (WOWB). Also, Table 2 summarizes the percentage of the transfer function reductions. It can be seen that by increase of the frequency of the load, the reduction ratios increase. This is due in part to the fact that by increase of the frequency of excitation the wavelengths become shorter and the ratios of WBs dimensions to the wavelengths of incoming waves are then larger, which in turn increases the efficiency of the WBs to mitigate and scatter the waves. In lower frequencies, the branched-shaped part of WBs tends to create 45 degree angle, which provides the maximum stiffness, with respect to the horizontal line. However, as the frequency of excitation increases, the wavelength decreases and more WBs are placed beneath the foundation to reflect the incoming waves and the other remained WBs are branched-shape with the angle of almost 45 degree.

Table 2. Transfer function reduction

Frequency (Hz)	**Transfer function**		**Reduction (%)**
	WOWB	**WWB**	
0.92	41.3	33.8	18
2.32	13.6	7.5	45
3.72	10.6	0.1	99



(a) (b) (c)

Figure 5. Treansfer function reduction: a) 0.92 Hz b) 2.32 Hz c) 3.72 Hz

The same trend of reduction is also achieved in the time domain with the form of Ricker wavelet of various predominant frequencies.

Figure. 6 depicts the mean horizontal acceleration of protected nodes on the soil surface. The maximum acceleration has decreased by 21%, 60% and 87% under predominant frequency of 0.92 Hz, 2.32 Hz and 3.72 Hz, respectively.

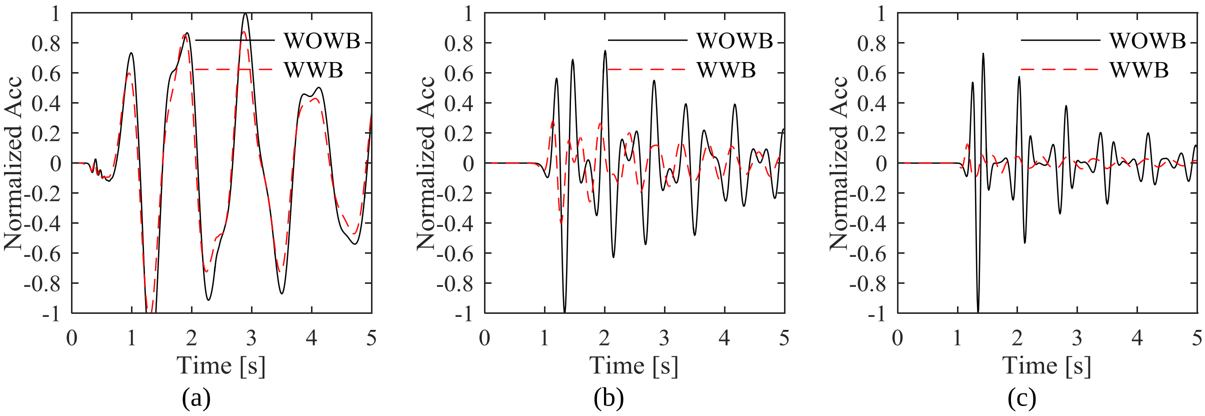


Figure. 6. Mean horizontal acceleration at soil sutface under Ricker wavelet with the predominant frequncy of a) 0.92 Hz b) 2.32 Hz c) 3.72 Hz

5. CONCLUSIONS

To take the maximum advantage of WBs in decreasing the mean horizontal acceleration of a specific area on the soil surface under vertically propagating shear waves, GA optimization is utilized to obtain the optimum position of WBs within the medium. By applying the steady state harmonic excitation at the base of the soil deposit, the transfer function is thus determined. Therefore, the frequencies in which the resonance occurs are defined. These frequencies are chosen as the frequency of the optimization function, since as a result of amplifications, the amplitude of earthquake load of frequency content close to these natural frequencies are intensified. After determining the optimum layouts of WBs in the frequency domain, their influence on vibration reduction in the time domain under various Riker wave lets is examined. As the frequency content of excitation increases, the high level of vibration reduction can be achieved using optimally located WBs. Moreover, higher frequency of excitation leads to the condition that the WBs tends to locate beneath the foundation to reflect more portion of incoming waves. The optimum position of WBs in lower frequency, which is obtained by coupled FE-GA, is gained such as two branches with almost 45 degree with respect to the horizontal axis, which in turn provide the maximum stiffness.

In order to make conclusive results, it is of importance to investigate the effect of vertical vibration on the WBs position along with horizontal one in the future studies.

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