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Structural Response Spectra under Passing Underground Trains

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The vibrations due to passing of underground trains can harm nonstructural elements and disturb the occupants of nearby buildings. Importance of the problems raised are more highlighted for buildings like hospitals, laboratories, museums, etc., and their assessments is an important design need. As a response to this necessity, in this paper response spectra for buildings due to passing trains moving with different velocities are calculated. These spectra are very convenient substitutes for the time consuming and costly analysis of a tunnel-soil system under moving loads, as they give the maximum structural responses rapidly. For derivation of the spectra, the soil-tunnel system is modeled using a 3D finite element mesh and the standard moving train loads are applied to the system. The ground surface vibration time histories are calculated at different distances from the tunnel axis. The peak acceleration, velocity, and displacement responses are determined for an SDF dynamical system under the same ground motions. Different standard trains, train velocities, tunnel depths, distances from tunnel, and soil types are taken into account and the results of the analysis are presented as acceleration, velocity, and displacement spectra.

1. Introduction

In the last decades, there have been many research works in the field of noise and vibrations due to passing trains, each trying to increase the accuracy of results in addition to developing simplified models. Summary of these cases can be found in refs. [1, 2]. In general, research conducted in this regard can be divided into three groups. The first group includes researches that consider methods of exact simulation of train movement and interaction of track and soil, e.g., those by V. Krylov [3], A. M. Kaynia [4], H. Takemiya [5], Lombaert et al [6], Cheblia et al [7], Gopta et al [8], Galvin et al [9], and many others. The second group includes various numerical, analytical and empirical methods that predict train-induced vibrations (along open track as well as in tunnels), e.g., G. Degrande [10], L. Hall [11], Hussein et al [12], Pakbaz et al [13] and so on. The last group includes methods of control and reduction of ground-borne vibrations from railway traffic, e.g., G. M. Adam [14], Lombaert et al [15], Andersen et al [16], Y.L. Xu and A.X. Guo [17]. In this work, a general consideration in metro train-induced vibration has been performed and the results are exhibited in the form of practical graphs. This research belongs to the first and second groups above modeling the train loading and interaction of tunnel and soil using a 3D finite difference scheme. Then dynamic analysis of the system results in ground surface

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vibration time histories at different distances from the tunnel axis. With these time histories, the response spectra are calculated for an SDF dynamical system representative of the modal response of surface buildings. The results are in the form of velocity spectra, what humans and nonstructural elements are more sensitive to. This relieves the need for doing complex interaction analysis for buildings nearby a railway.

2. Problem Outline

In this research induced vibrations due to passage of trains inside an underground subway are evaluated using numerical modeling in FLAC3D V3.0. The FLAC3D finite difference program has been extensively used in similar studies. The finite difference simulation of trains-induced ground vibrations includes a three-dimensional model that is influenced by means of a certain moving load. All the dynamic analyses in this research were performed in the time domain using an explicit algorithm. The optimum time step of the analyses was found to be $\Delta t = 2.2\times10^{-5}$ s and the computations totaled 251440 steps taking about 18 hours for each run on a personal laptop. After accomplishing the finite difference analysis of the tunnel-soil interaction, time histories and maxima of velocity of an SDF system at different distances from the tunnel, are calculated.

3. Finite Differences Models

The three-dimensional model investigated is shown in Figure 1. The model possesses a vertical plane of symmetry removing the need to model the whole system. This model was set to 110 m in length (along railway line, to include the full train), 45 m in width and 50 m in height. The average element size in the ballast part and where the load was applied was about 0.6*0.625*0.5 m; in the central part around the tunnel was about 0.6*0.725*1.75 m and in the other zones was about 3*2.36*2.5 m. The element sizes are taken as small enough to allow waves at the input frequency to propagate accurately in the vertical direction. For determining the optimum size of elements in order to get a reasonable accuracy in a minimized time, different meshing patterns under a single moving load were analyzed resulting in the pattern exhibited in Figure 1. In the FLAC3D code, 8-node brick elements were used to simulate the soil and ballast and shell elements to simulate the concrete tunnel lining. The static boundaries of the model (which do not exist in reality) are taken sufficiently far away to avoid direct influence of the boundary conditions and also viscous boundaries are specified at the vertical boundaries at both ends [18].

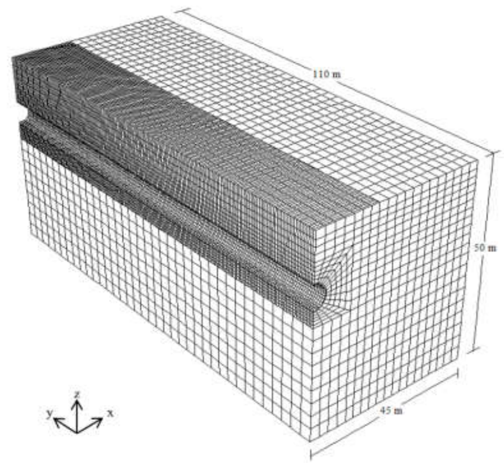

Figure 1. Three-dimensional finite difference mesh model

3.1. Loading scheme

Train load is considered as a series of moving loads applied on the nodes of the sleeper elements. The load is assumed to be according to the load distribution proposed by Krylov being transmitted to the soil by each couple of wheels through the sleepers [2] considering the amendments offered by Galvin et al for other mesh models [9]. Krylov obtained a quasi-static load distribution pattern among several sleepers that transmit the load of an axle. In This model, the track is represented as a beam on a Winkler foundation. The model does not incorporate some sources of vibration such as rail roughness, wheel flats and parametric excitation. Ground vibration induced by moving axle loads is independent of the dynamics of vehicles and of track quality. In fact, Lai et al [19] showed that consideration of only quasi-static loads underestimates the actual response level, especially for higher excitation frequencies. But when load speed is less than the Rayleigh-wave velocity of the ground, the ground response due to a moving load is essentially quasi-static. That is, the displacements and stress fields are essentially the static fields under the load simply moving with it [20]. Hence according to Table 1 and Rayleigh-wave velocity for each soil, representing only moving quasi-static axle loads seems to be reasonable.

Table 1. Characteristics of materials

Material	Shear wave velocity	Mass density	Poisson ratio
	C_s [m/s]	[kg/m^3]	ν
Soil I	750	2000	0.30
Soil II	550	1900	0.35
Ballast	650	1800	0.30

3.2. System characteristics

The soil is modeled as an elastic medium, which is characterized by the shear modulus, Poisson's ratio, the density and the material damping ratio. In this study, two types of soils were utilized with the specifications contained in Table 1, representing hard and medium soils. In all soil models, the medium is considered to be a half-space with 2% material damping. The tunnel has an internal radius of 3m and a concrete wall with a thickness of 0.35m (Figure 1). Concrete lining properties used in the modeling are E = 24,860 MPa and the Poisson ratio = 0.20. To investigate effect of tunnel depth (distance from tunnel center to surface) on surface vibration, two different depths of 10 and 15 m are considered.

The track is a classical ballasted track with UIC60 rails supported every 0.60 m by grooved rubber pads on monobloc concrete sleepers. Continuously welded rails with a mass per unit length of 60 kg/m and a moment of inertia $I = 0.3038 * 10^{-4}\ m^4$ are fixed with a pandroll E2039 rail fixing system on precast, pre-stressed concrete monoblock sleepers of length 2.5 m, width 0.235 m, height 0.205 m and mass 300 kg, under the rail. The values for the track have been taken from Ref. [10]. Thickness of ballast at the tunnel floor is 0.60 m with the specifications contained in Table 1. Standard train model AVE-Alstom with two velocities 180 and 300 km/h are considered in this paper. The length of the train is 222 m. Its profile and shape is given in Figure 2. The specifications for the AVE-Alstom high-speed train is provided in Table 2 [9].

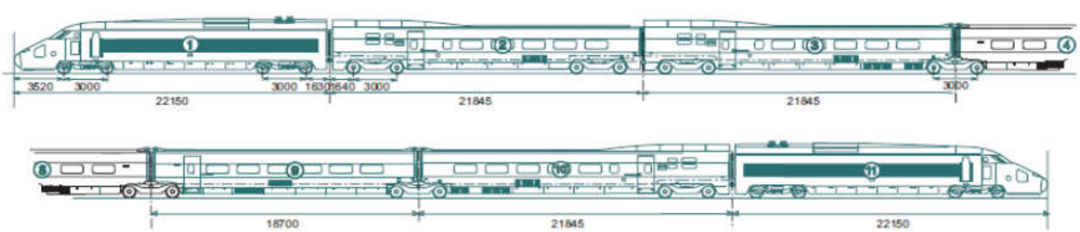

Figure 2. Configuration of the AVE-Alstom high-speed train [9]

Table 2. Geometrical and mass characteristics of the AVE-Alstom high-speed train [9]

	No. of wagons	No. of axles	length of wagon [m]	Inter-bogie spacing [m]	Inter-axle spacing [m]	weight per axle [kg]
Locomotive	2	4	22.15	14.00	3.00	17000
End wagon	2	3	21.84	18.70	3.00	14500
Laboratory wagon	1	4	21.84	15.56	3.00	10875
Middle wagon	6	2	18.70	18.70	3.00	17000

4. Analysis Results

In this section, the results for the maximum velocity are presented. Each set of the results are drawn for 0, 2 and 5% structural damping. The results for other damping ratios in between can be derived by linear interpolation. Therefore, knowing the damping value and the periods of the (governing) modes of the building under consideration, the maximum response at each level under passage of a train can be calculated using the conventional modal procedure. Factors considered in calculating a spectrum include the train type, train speed, tunnel depth, soil type and distance from the tunnel.

According to Figure 3, the distance from the tunnel can be computed as follows. If H is depth of the centerline of tunnel, h = H+2.4 m will be load depth and distance from the tunnel axis in P1, P2 and P3 points are zero, h and 2h, respectively.

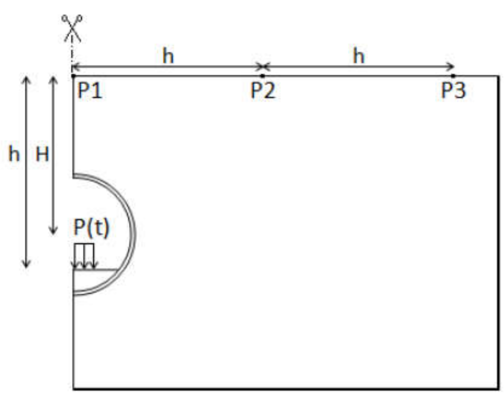

Figure 3. Tunnel cross-section

The response spectra are given in the following order. Figures 4 to 7 display the effect of tunnel depth, soil type, and distance for train velocity 180 km/h, and Figures 8 to 11 the same effects for train velocity 300 km/h. It is to be mentioned that for a new railway and substantially altered railways the highest vibration level in a bedroom at night should not exceed 0.4 mm/s, while for the existing railways the limit is set to 1.0 mm/s [21].

As is seen, at the velocity of 180 km/h there are two clear peaks in the response, one at very small periods (around 0.01 sec) and the other at a larger period of around 0.03 sec. At 300 km/h, the first peak practically disappears and only the second one remains. This range of periods belongs to rigid equipment inside buildings. The effect of structural damping is tangible comparing zero damping and 2% damping cases where the response at the latter case decreases by several times. The values of the peaks are essentially independent of tunnel depth and distance for the range of values examined but they are sensitive to the soil type as the response at small periods decreases in soil type II. The peaks of response for no damping just touch the vibration limits cited. For building periods more than 0.2 sec, passage of trains seems not to be felt by the structure in its fundamental mode, but it still affects the structure in its higher modes of vibration.

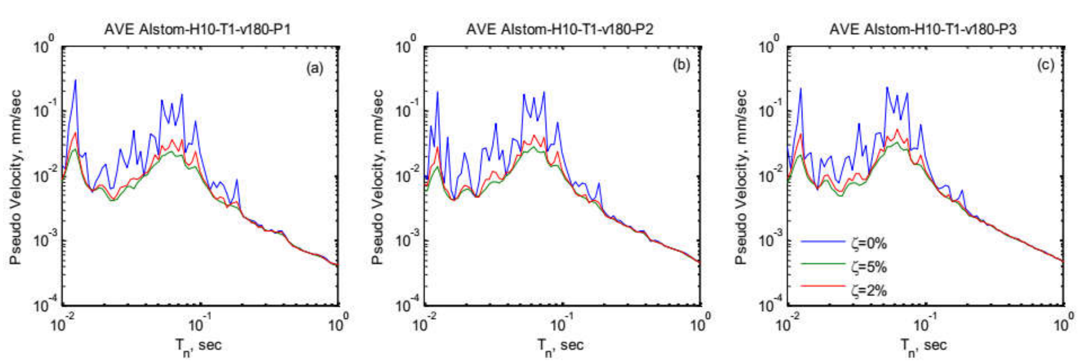

Figure 4. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 180 km/h, tunnel depth 10 m and soil type I, (a) at the point P1, (b) at the point P2 (c) at the point P3

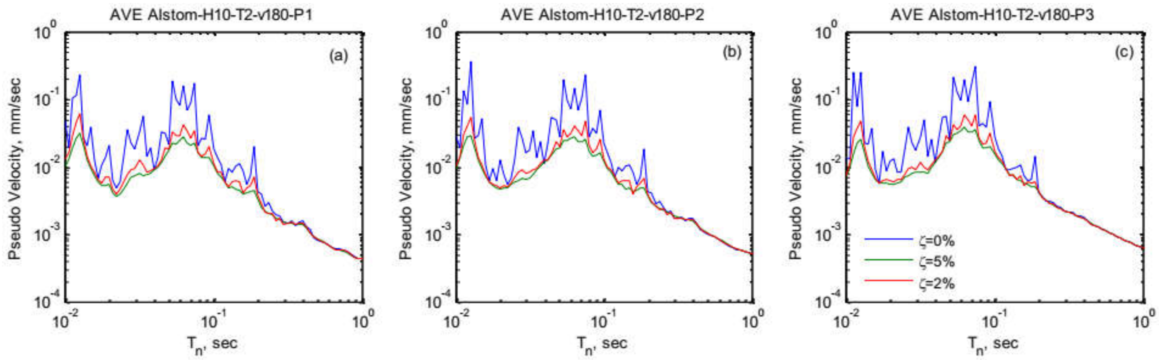

Figure 5. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 180 km/h, tunnel depth 10 m and soil type II, (a) at the point P1, (b) at the point P2 (c) at the point P3

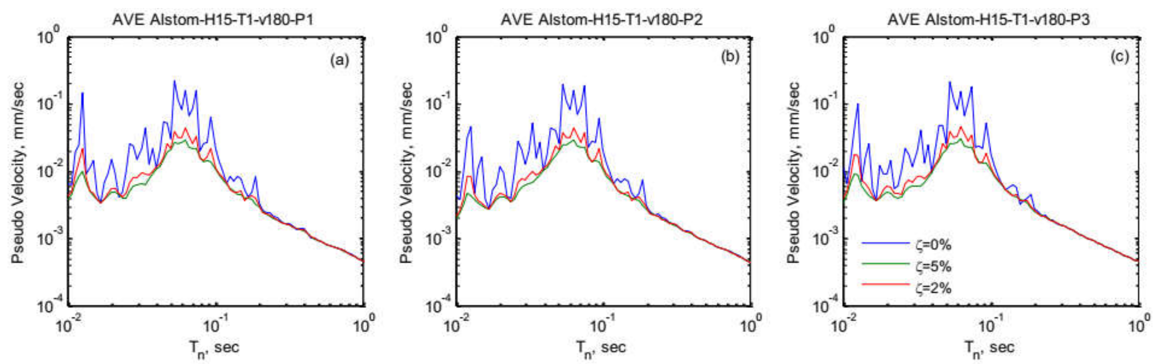

Figure 6. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 180 km/h, tunnel depth 15 m and soil type I, (a) at the point P1, (b) at the point P2, (c) at the point P3

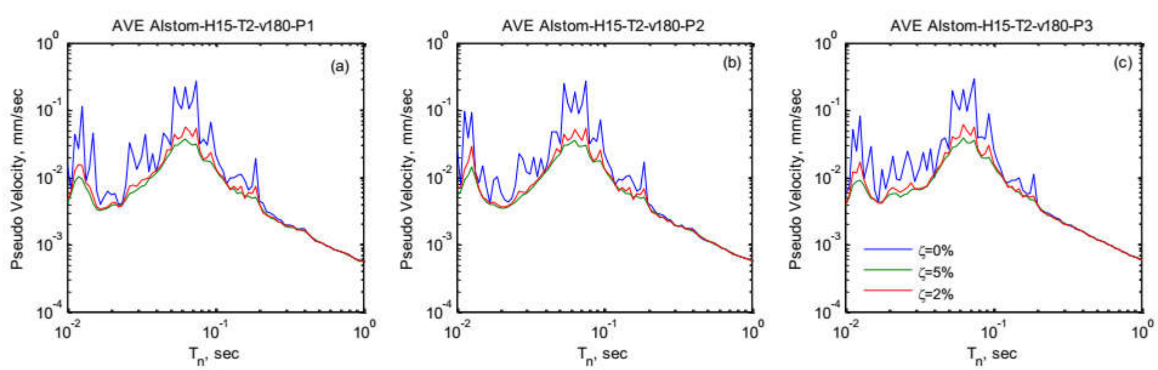

Figure 7. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 180 km/h, tunnel depth 15 m and soil type II, (a) at the point P1, (b) at the point P2, (c) at the point P3

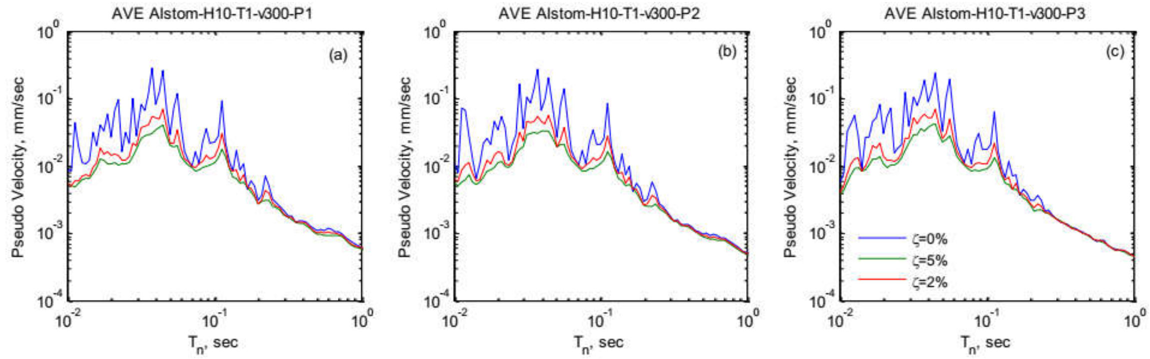

Figure 8. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 300 km/h, tunnel depth 10 m and soil type I, (a) at the point P1, (b) at the point P2, (c) at the point P3

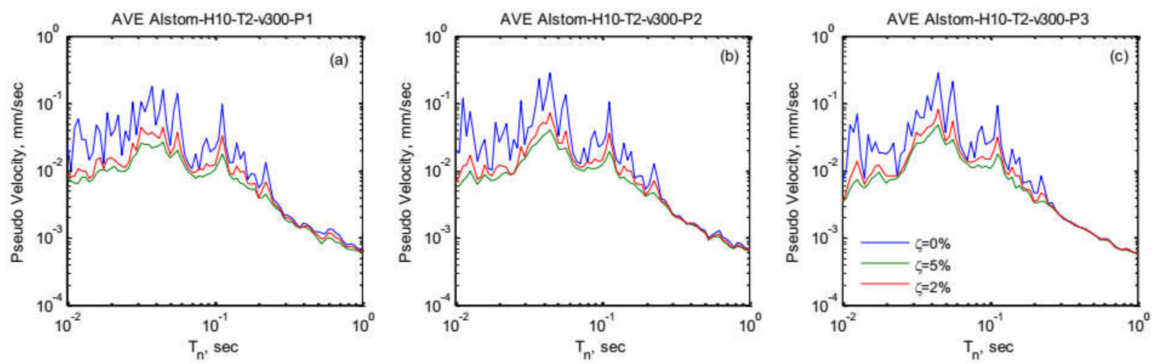

Figure 9. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 300 km/h, tunnel depth 10 m and soil type II, (a) at the point P1, (b) at the point P2, (c) at the point P3

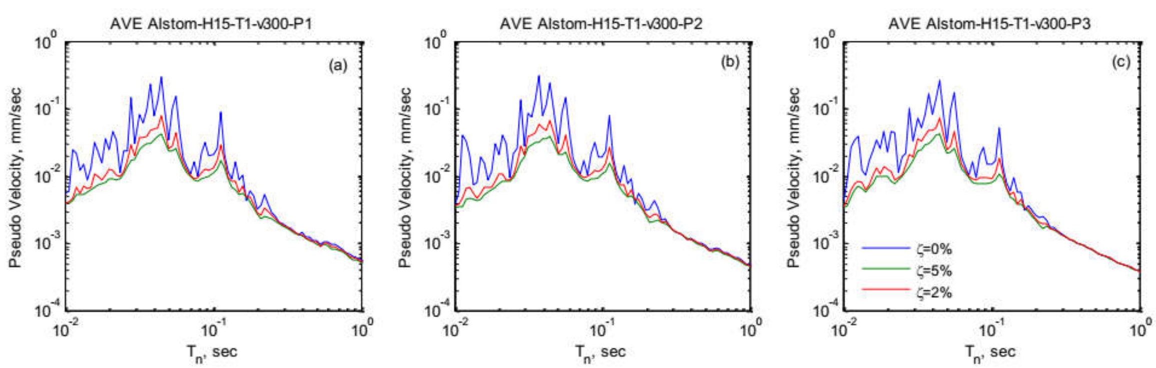

Figure 10. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 300 km/h, tunnel depth 15 m and soil type I, (a) at the point P1, (b) at the point P2, (c) at the point P3

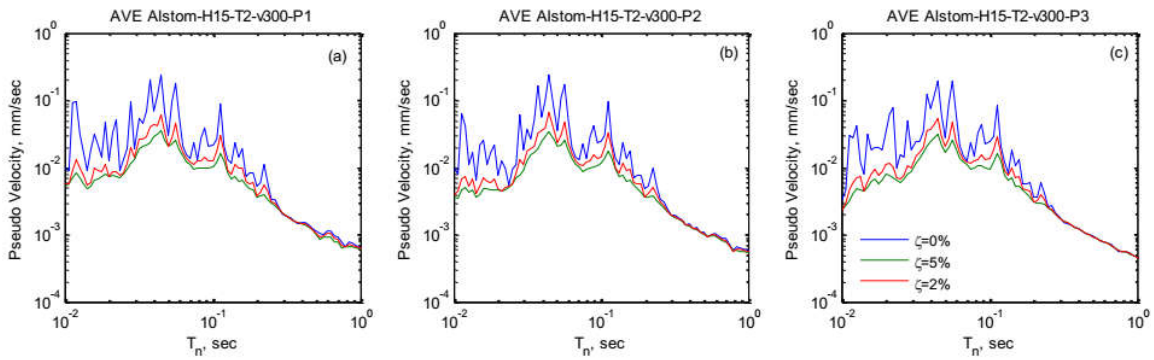

Figure 11. Pseudo-velocity spectra of a SDF system under passage of AVE Alstom train with velocity 300 km/h, tunnel depth 15 m and soil type II, (a) at the point P1, (b) at the point P2, (c) at the point P3

5. Conclusions

To predict and assess the vibrations induced by underground train, in this research 72 cases were investigated. The cases differed in train velocity, tunnel depth, soil type and distance from tunnel axis. Time histories of the velocity of the ground surface at certain distances were calculated. These were then input to SDF systems with different damping ratios and natural periods. From the cases presented in this paper it can be concluded that:

- Response of the SDF system decreases with increasing depth of tunnel. However, this effect is not overwhelming for depths up to 15 m.

-With increasing train speed, response of very small period SDF systems decreases.

- Putting the SDF system farther from the tunnel results in a reduction in the system response.

- For the analyses cases performed, the peaks of the responses were always less than the threshold causing disturbance for human.

- Overall, the system response is more sensitive to the train speed and soil type and less sensitive to distance and depth.

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