

Analysis of Vibrations and Infrastructure Deterioration Caused by High-Speed Rail Transit

Final Report
Metrans Project 01-3

December 2005

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Abstract

The effects of a heavy travelling load on the soil medium and on the adjacent structures were studied. Five soil profiles were analyzed, three from sites in Southern California and two from well documented sites in Europe. It was found that the material damping in the soil layers does not affect attenuation, but rather radiation damping is the most significant. Four of the Five soil profiles studied showed a decay of vibration of 20dB within a distance of 10 meters. Only the Swedish soil profile at Ledsgard showed a long-distance propagation tendency; that peculiar site has a soft marine layer between two stiff layers and the vibrational energy was retained near the surface. Flexible foundation soil-structure interaction analyses were performed and found that the damping values used for the building vibration modes to be the key factor. Some ground vibration amplitudes were diminished by the kinematic interaction with the rigid footings and by the large inertia of the building. vibrations, however, can be amplified by resonance within the structure.

Keywords: high-speed trains, wave propoagation, soil-structure interaction, structural vibrations

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Disclosure

Project was funded in entirety under this contract to California Department of Transportation.

Acknowledgements

This work was supported by a grant from METTRANS,a U.S. Department of Transportation (DOT) designated University Transportation Center (Grant No. 01-3). The author is appreciative of the guidance provided by Professor J.E. Luco of the University of California, San Diego. A significant part of this research was performed using computer software developed by Dr. Luco and his former student, Randy Apsel.

Introduction

The research devoted to high-speed rail transportation is extensive; especially in Europe and Japan where that mode of transportation is vital. Detailed studies have been made on nearly every aspects which concern rail transit: from the design of mechanical components, to rail infrastructures and to environmental impact caused by noise and vibration. This research project is aimed at the effects of ground-borne vibration on the surrounding buildings.

Dowding (2000) described how transit or construction vibration can generate ground waves which interact with a nearby structure in a manner similar to that of seismic waves, although with a much smaller amplitude. A response spectrum approach, an earthquake engineering tool, was actually used to determine whether the vibration could resonate with certain modes of a floor, or a wall. Direct acoustic waves can penetrate a wall if the frequency of the sound wave coincides with a natural frequency of the wall and the interior of the wall can serve as a giant loud speaker, generating acoustic waves at an intensity far above the incident wave. Train noise has a lower frequency content than other modes of transportation, it is from about 8 to 100 Hz. The frequency below 20 Hz is known as Infrasound and they will not affect the human ear but vibration for sensitive equipment could cause a challenge. The higher frequency ground waves generally attenuate quickly in the soil medium, therefore, the audible range of concern results from the above ground mechanical noise.

An exciting area of research in recent years in the area of ground borne vibration is the special case when the train speed exceeds the shear wave velocity of the soil medium. The outgoing wave from the track is that of a shock wave because the train arrives at a specific location before it can be notified through a propagated P-wave or S-wave. The shock wave behaves as a plane wave and it can travel a much farther distance without attenuation. The controversial THSRC, the Taiwan High Speed Railway Corp project, has a planned route

which goes within 200 to 300 meters of a large cluster of High Technology buildings in Southern Taiwan. Besides the political and contractual problems of that project, it is a major concern that the small vibration, undetectable by human, could affect the sensitive instruments of the semiconductor industry. Since a railway is extended over a long distance, widely varied soil conditions can exist on different section of the same route.

Madshus and Kaynia (2000) noted the general consensus of the research community that rail infrastructures deteriorate quickly if the speed of the train is increased. Although no specific reference was made regarding the soil properties underneath the rail track, it is safe to assume that high stresses exerted on the soil medium and on the rail supporting structure play a major role in their rapid deterioration. Kim and Drabkin (1994) recorded soil settlement in an experiment even with low-level vibrations.

DeGrande and Schillemans (2001) used as an example a soil condition on an European HST track between Paris and Brussels, having a shear wave velocity of 80 m/sec for the top 1.4m, 133 m/sec for the next 1.9m and 226 m/sec for the assumed half space below. The 80 m/sec value is below some of the HST's operating velocities which range from 220 to 315 km/hr, the latter converts to 88 m/sec. Dowding (2000) reported that the Swedish site is composed of a 2 m stiff crust that overlays 5 m of soft marine clay, which in turn overlays a half space of stiff clay. The soft layer at Ledsgard has a shear wave velocity of only 40 m/sec.

In Los Angeles and in most areas in California, the soil condition is quite firm and the top-layer shear wave velocity is mostly 150 m/sec or far above. According to the ROSRINE geological website, there are only 3 sites with top-layer shear wave velocity less than 150 m/sec, with another 4 other sites that was measured at 150 m/sec. A summary of the idealized soil profile is given in Appendix A. There are areas, However, that the soil profile is extremely soft. For example, a section of the Alameda Corridor near the city of Long

Beach has soil types SM (Silty Sand), ML (Inorganic Silt and very fine Sand), and CL (Inorganic Clays of low to medium plasticity). These soil types have Young's Moduli of 4000, 12000 and 18000 kg/m², respectively, and a specific weight of 17.2 kN/m³. Using the theory of elasticity, one can estimate the compressional wave velocity to be approximately 65, 83 and 101 m/sec, respectively. Since the Poisson's ratio was estimated as 0.2 by the geological report, the shear wave velocities are respectively, 40, 51 and 62 m/sec. Hence, over long distances, there would be spots in every route that require special attention.

Methodology

Wave propagation has always been one of the main research topics in seismology and earthquake engineering. Long period waves are of interest in exploration seismology; medium to short period (high frequency) waves are important to earthquake engineering. There are two major types of numerical methods available for the study of waves: the finite element method and the continuum mechanics method. The finite element method has the flexibility to analyze models with difficult geometrical shapes and nonlinear material properties; but its formulation requires a volume integral over the model and that limits the method's effectiveness for the study of waves in a model with large dimensions. The continuum mechanics method models the earth as a layered semi-infinite space and its solution can be written as a superposition of the fundamental solution, known as Green's functions. The radiating boundary condition for the far field would be satisfied automatically.

The Green's functions for a load traveling at a constant velocity along a straight line were developed by deBarros and Luco (1992) and have been applied to a class of diffraction problems for canyons and tunnels (Luco et al 1990). These functions are complex and a significant programming effort was required to implement them. They can be adapted and applied to the transportation problem at hand because it can readily model the properties of geological layers, for example, the properties of wave velocities, mass density, layer thicknesses and damping values, etc. The restriction of the above Green's function is that the velocity of the point load must be constant and that it moves along a straight line.

Although the traveling load Green's functions represent a good fit for this project. It was deemed not flexible enough to consider the effects of speed changes and curved track geometries. As substitute, the Green's functions for a stationary load, developed by Apsel (1979), is selected. Phase shifts, according to the load velocity, are applied to the stationary

loads to simulate a moving load. The approach is able to model most situations with linear soil profiles, but the computational effort is also increased, perhaps by an order of magnitude, because it would require an infinite integral for each observer and for each frequency in the spectrum.

Working with the Green's Functions in the frequency domain, real-time solutions can be obtained using Digital Fourier Transform for the motion in a soil medium subjected to time-dependent loads caused by a passing train. Once the frequency dependent transfer function is determined for an observation station, solution for different time dependent loading histories could be evaluated without extra effort. Using this approach, it is straightforward to do an extensive numerical investigation of the effect of travelling waves. One key parameter is to increase train speed for different ratios with the shear wave velocity of the top layer, covering subsonic, transonic and supersonic ranges. If the traveling speed exceeds the wave velocity of the top soil layer, then shock waves would be generated. The behavior of a shock wave is that of a plane wave and it would not attenuate quickly over distance. An estimate of that amplitude attenuation pattern can be evaluated numerically.

With the generation of ground waves accomplished, the output data of the previous phase can be used as the input excitation for a building model. The method to be used for this analysis is commonly known as the CLASSI approach. CLASSI is acronym for a Continuum Linear Analysis for Soil-Structure Interaction, a general approach to a large class of soil-structure interaction problems. The foundation analysis, including the determination of the soil impedance matrix and the kinematic interaction of the footing with the incident ground motion, is to be done using the boundary integral equation method. The superstructure will be modeled by a conventional finite element program such as COSMOS or ADINA.

After the ground waves are analyzed, they can be used as input excitation to nearby structures in a soil-structure interaction analysis. Different parameters can be used to represent structures of different type, size, weight, height, steel or concrete construction. The filtering properties of the foundation and structure can be analyzed. This methodology for calculating response to a structure subjected to incoming waves is quite common in earthquake engineering and can be applied directly to ground waves generated by rail vibration.

The proposal to use the continuum method over a discrete method is necessary. Private communication with a consulting firm participating in the Taiwan THSRC project indicates that their finite element models required many weeks of computation and some details have to be sacrificed. Also, the discrete method, used with a larger element size, will filter out some vital wave frequencies of interest for the rail vibration problem. One well-known problem for the finite element method is that an artificial boundary must be created for the model to limit the volume to a finite size. When the wave strikes that artificial boundary, the reflected waves would contaminate the solution.

Research and Analysis

The Moving Load Parametric Study

This research project was fortunate to have access to the Green's functions program developed by Apsel (1979), otherwise, several years of development would have been required. Dr. Apsel's program can analyze a stack of viscoelastic layers over a half space with the freedom of choice for a wide range of soil properties, the major requirement is that the materials are linear. The Green's functions were derived in the cylindrical coordinates for displacements, in the form of

$$U(R, \psi, z_s, z_o, \vec{\rho}, \vec{\beta}, \vec{\nu}, \vec{\xi}_\beta, \vec{\xi}_\alpha, \vec{D})e^{i\omega t}/R \quad , \quad (1)$$

and for stresses or tractions, in the form of

$$T(R, \psi, z_s, z_o, \vec{\rho}, \vec{\beta}, \vec{\nu}, \vec{\xi}_\beta, \vec{\xi}_\alpha, \vec{D})e^{i\omega t}/R^2 \quad , \quad (2)$$

in which R is the radial distance from the point source, ψ is the azimuthal angle, z_s is the depth of the source point, z_o is the depth of the observation point, $\vec{\beta}$ is the array of S-wave velocities, $\vec{\rho}$ is the array of mass densities, $\vec{\nu}$ is the array of Poisson ratios, $\vec{\xi}_\beta$ is the array of S-wave damping ratios, $\vec{\xi}_\alpha$ is the array of P-wave damping ratios, and $\vec{D}$ is the array of layer depths. For the present application, $z_s = z_o = 0$, because the load and the observer are both on the ground surface.

Consider now the coordinate systems illustrated in Fig. 1. The cylindrical coordinate system is defined with the z' -axis pointing downward, a coordinate system used in most classical geophysical problems. The origin of the polar coordinate system is defined at the source point, $\vec{r}_s$, therefore, the observation point, $\vec{r}_o$, is located at the coordinates, (R, ψ) , where $R = |\vec{r}_o - \vec{r}_s|$, and

$$\psi = \arg(\vec{r}_o - \vec{r}_s) = -\tan^{-1} \left(\frac{y_o - y_s}{x_o - x_s} \right) \quad . \quad (3)$$

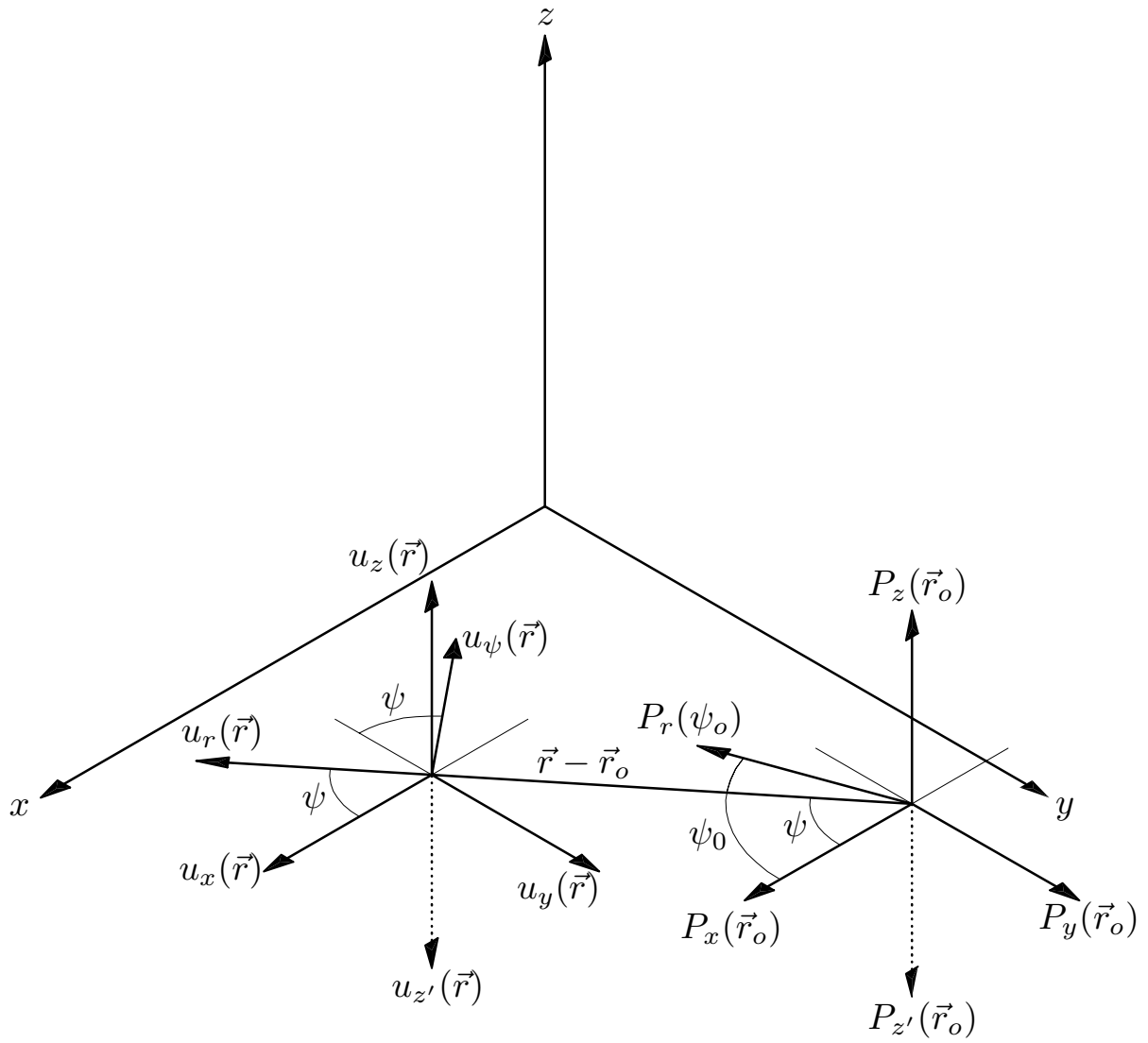


Figure 1 – Transformation of Green's Functions in Cylindrical Coordinates to Green's Functions in Cartesian Coordinates.

Using a coordinate transformation, the relation between the harmonic response, $\vec{u}$ at $\vec{r}_o$, and the load, $\vec{P}$ at $\vec{r}_s$, can be written in the form

$$\begin{Bmatrix} u_x(\vec{r}_o) \\ u_y(\vec{r}_o) \\ u_z(\vec{r}_o) \end{Bmatrix} = \frac{1}{\mu R} [G] \begin{Bmatrix} P_x(\vec{r}_s) \\ P_y(\vec{r}_s) \\ P_z(\vec{r}_s) \end{Bmatrix} \quad , \quad (4)$$

with the time factor, $e^{i\omega t}$, implied and the matrix $[G]$ defined as

$$[G] = \begin{bmatrix} f_{rr} \cos^2 \psi - f_{\psi r} \sin^2 \psi & -(f_{rr} + f_{\psi r}) \sin \psi \cos \psi & -f_{rz} \cos \psi \\ -(f_{rr} + f_{\psi r}) \sin \psi \cos \psi & f_{rr} \sin^2 \psi - f_{\psi r} \cos^2 \psi & f_{rz} \sin \psi \\ f_{rz} \cos \psi & -f_{rz} \sin \psi & f_{zz} \end{bmatrix} . \quad (5)$$

The complex scalar functions, f_{rr} , $f_{\psi r}$, f_{rz} , and f_{zz} are the 4 displacement Green's functions derived in cylindrical coordinates. The factors, $\sin \psi$ and $\cos \psi$ in eqn. (5), can be evaluated simply as $\sin \psi = -(y_o - y_s)/R$ and $\cos \psi = (x_o - x_s)/R$.

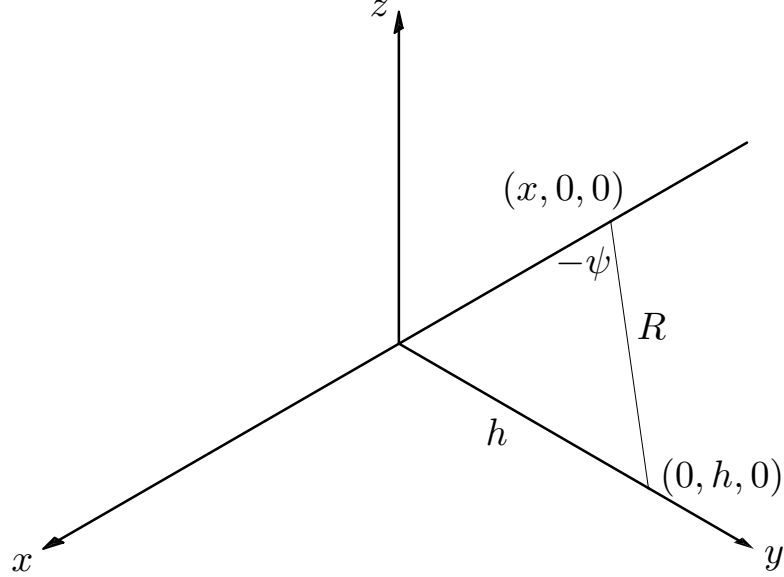


Figure 2 – Schematic of a moving load at $(x, 0)$ and an observer at $(0, h)$.

Using the stationary harmonic point load described above, a moving point load can be simulated by changing the location of the source point as a function of time, t . A time shift factor, $\exp(i\omega(t - x/c))$, represents a load moving in the positive x direction while the factor, $\exp(i\omega(t + x/c))$, represents a load moving in the negative x direction. The parameter, c , is the speed of the load and the quotient, x/c , has the unit of time.

In the schematic depicted in Fig. 2, a moving vertical point load, P_z , is at location $(x, 0)$ and the observation point is defined at $(0, h)$. The vertical displacement at that instant shown can be expressed as

$$u_z(0, h, x, \omega) = \frac{f_{zz}(\omega\sqrt{x^2 + h^2})}{\sqrt{x^2 + h^2}} e^{i\omega t} e^{-i\omega x/c} \quad . \quad (6)$$

Accumulating the effects of the moving load over the entire x -axis, integrate eqn. (6) over x as

$$u_z(0, h, \omega) = \left[\int_{-\infty}^{\infty} \frac{f_{zz}(\omega\sqrt{x^2 + h^2})}{\sqrt{x^2 + h^2}} e^{-i\omega x/c} dx \right] e^{i\omega t} \quad . \quad (7)$$

In the above equation, the phase factor, $\exp(-i\omega x/c)$, accounts for the position of the load as a function of time.

Using the third column of the matrix $[G]$ as defined in eqn. (5) and substituting $\sin \psi = -h/R$ and $\cos \psi = -x/R$, the x and y components of the displacement can also be written as

$$u_x(0, h, \omega) = \left[\int_{-\infty}^{\infty} \frac{x f_{rz}(\omega\sqrt{x^2 + h^2})}{x^2 + h^2} e^{-i\omega x/c} dx \right] e^{i\omega t} \quad , \quad (8)$$

and

$$u_y(0, h, \omega) = \left[-h \int_{-\infty}^{\infty} \frac{f_{rz}(\omega\sqrt{x^2 + h^2})}{x^2 + h^2} e^{-i\omega x/c} dx \right] e^{i\omega t} \quad . \quad (9)$$

The complex functions, u_x , u_y and u_z , of eqns. (7), (8), and (9) represents the fundamental solutions that can be used to form solution for more specific geometries. The train loads are

distributed dynamically from the axles of the train onto a pair of deformable tracks; and then through the sleepers onto the ballast overlying the soil medium. The loads of the train can be approximated by a uniform distribution over the ballast in the direction perpendicular to the track while the load parallel to the track is a function of time and is dependent of the train speed, c .

In the frequency domain, the displacements as frequency dependent functions can be expressed as linear combination of the load function, $L(\omega)$, as

$$u_x(0, h, \omega) = U_x(0, h, \omega)L(\omega) \quad , \quad (10)$$

$$u_y(0, h, \omega) = U_y(0, h, \omega)L(\omega) \quad , \quad (11)$$

$$u_z(0, h, \omega) = U_z(0, h, \omega)L(\omega) \quad , \quad (12)$$

in which U_x , U_y , and U_z are the transfer functions between the horizontal displacements, u_x , u_y , the vertical displacement, u_z , respectively, and the load function $L(\omega)$. The transfer functions, U , are the integrals, over area, of the fundamental solutions, or the Green's Functions.

The process for calculating transient displacement response is to first find the Fourier Transform of the load function, $L(t)$, as

$$L(\omega) = \int_{-\infty}^{\infty} L(t)e^{-i\omega t} dt \quad , \quad (13)$$

and then obtain the displacements using the Inverse Fourier Transformation as

$$u_x(0, h, t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} U_x(0, h, \omega)L(\omega)e^{i\omega t} d\omega \quad , \quad (14)$$

$$u_y(0, h, t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} U_y(0, h, \omega)L(\omega)e^{i\omega t} d\omega \quad , \quad (15)$$

$$u_z(0, h, t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} U_z(0, h, \omega)L(\omega)e^{i\omega t} d\omega \quad . \quad (16)$$

In the frequency domain, it is easy to find the transfer functions for velocity as $\dot{U} = i\omega U$, and that for acceleration as $\ddot{U} = -\omega^2 U$.

The Loading Time Function

Since the problem at hand is a linear approximation, the superposition of solutions is allowable. A finite element model was made for the track over the sleepers and ballast for one single train axle and the load distribution for the one axle is obtained a function of $L(|x - x_s|)$ with x_s being the location of the axle load. Since the observation point was defined at $x = 0$, the function as a function of time is $L(c|t - t_s|)$, in which t_s is the time of arrival of that particular axle at $x = 0$.

The configuration of the Thalys HST model (DeGrande and Schillemans, 2001) will be used in the present analysis. The axle loads are given as 17,000 kg for the Locomotives and the Central Carriages; and as 14,500 kg for the Outer Carriages. The 4 axles of the Locomotive are located with separations of 3m, 11m, and 3m, respectively. The 3 axles of the Outer Carriages are located with separation of 3m, 15.7m, respectively. Finally, the 2 axles of the Central Carriages are located 15.7m apart. The locations of the front and rear axles are about 1.5m from the extreme ends of the cars.

With spacing defined as above, the train load for the examples were created with the array of one Locomotive, followed by an Outer Carriage; then 4 consecutive Central Carriages, followed by an Outer Carriage and another Locomotive. The total length of the train is about 140 meters and the time for it to pass a reference point is about 1.75, 2.34, and 3.51 seconds for train speeds of 80, 60 and 40 m/sec, respectively. With each axle load represented approximately by pulses, the faster train speed will cause the loads to be spaced closely in time and the effective load would be higher, but for a shorter time. The total weight of the train is 4.19 MN.

The Soil Profiles of Interest

Five soil profiles are selected for this research, three of which are typical of soil conditions in Southern California and two from sites of interest in Europe. The soil properties of the sites are list in Table 1, they are idealized from the borehole data as a set of uniform layers. The wave speeds are measured in m/sec, the depths are in meters, and the mass densities are in units of kg/m^3 .

Table 1 – Idealized Soil Profiles for High-Speed Train Analysis

Profile #1	Profile #2	Profile #3	Profile #4	Profile #5
$\beta_1 = 1000$ $\alpha_1 = 1700$ $\rho_1 = 1400$ $D_1 = \infty$	$\beta_1 = 150$ $\alpha_1 = 1700$ $\rho_1 = 1750$ $D_1 = 25$	$\beta_1 = 80$ $\alpha_1 = 160$ $\rho_1 = 1500$ $D_1 = 1.4$	$\beta_1 = 200$ $\alpha_1 = 400$ $\rho_1 = 2000$ $D_1 = 2$	$\beta_1 = 100$ $\alpha_1 = 180$ $\rho_1 = 1500$ $D_1 = 2$
	$\beta_2 = 350$ $\alpha_2 = 1700$ $\rho_2 = 1750$ $D_2 = \infty$	$\beta_2 = 133$ $\alpha_2 = 266$ $\rho_2 = 1500$ $D_2 = 1.9$	$\beta_2 = 40$ $\alpha_2 = 80$ $\rho_2 = 2000$ $D_2 = 5$	$\beta_2 = 180$ $\alpha_2 = 320$ $\rho_2 = 1500$ $D_2 = 4$
		$\beta_3 = 266$ $\alpha_3 = 452$ $\rho_3 = 1500$ $D_3 = \infty$	$\beta_3 = 250$ $\alpha_3 = 500$ $\rho_3 = 2000$ $D_3 = \infty$	$\beta_3 = 300$ $\alpha_3 = 800$ $\rho_3 = 2000$ $D_3 = \infty$

Soil Profile #1 is a rock site, consistent with the Southern California sites of Pacoima, ETEC RD-7 and Mira Catalina School. It is approximated very well by a uniform half space with a shear velocity of about 1000 m/sec. It is highly unlikely that a high speed train would come close to the wave speeds of the soil medium, the analysis of this site would determine whether the small vibration would travel a long distance.

Soil Profile #2 is a weathered soil layer overlaying a stiffer half space. The top layer has a shear velocity of 150 m/sec and has a relatively thick alluvial layer of 25m, consistent with some valley locations. In particular, it is similar to the Pier F Long Beach, Arleta and Portrero SMA sites.

Soil Profile #3 is that used by DeGrande and Schillemans (2001), and DeWuff et al (1990), for the HST track between Brussels, Belgium and Paris, France. With a top-layer shear velocity of 80 m/sec, the top speed of the HST at 88 m/sec can exceed the shear wave velocity in the soil medium and some phenomena of shock waves may appear.

Soil Profile #4 is the Swedish site report by Dowding (2000), it is composed of a 2 m stiff crust that overlays 5 m of soft marine clay, which in turn overlays a half space of stiff clay. The soft layer at Ledsgard has a shear wave velocity of only 40 m/sec and it is used here as an example. The top stiff layer is assumed to have a shear wave velocity of 200 m/sec while the half space below is assumed to have a shear wave velocity of 250 m/sec.

Soil Profile #5 is the famous Rinaldi site in Southern California that recorded the highest velocity of 183 cm/sec during the 1994 Northridge Earthquake. It has a soft, 2m thick, top layer with a shear wave velocity of 100 m/sec. Its second layer has a shear wave velocity of 180 m/sec and is about 4m thick. The lower layers can be approximated as a half space. The intent of this soil profile is to compare the peak velocity caused by a hypothetical high speed train as compared to a large earthquake of magnitude 6.5.

Analysis of Ground-Borne Vibration

Using soil profile #1, the train speed of a very high 100 m/sec and a moderate 50 m/sec were used. Even with the higher speed value, the ratio of the train speed to the shear wave speed is 0.10, a near static value. In this example, the damping ratio is varied from 1% to 4% to determine if the damping value of the soil medium affects the transmission of ground waves.

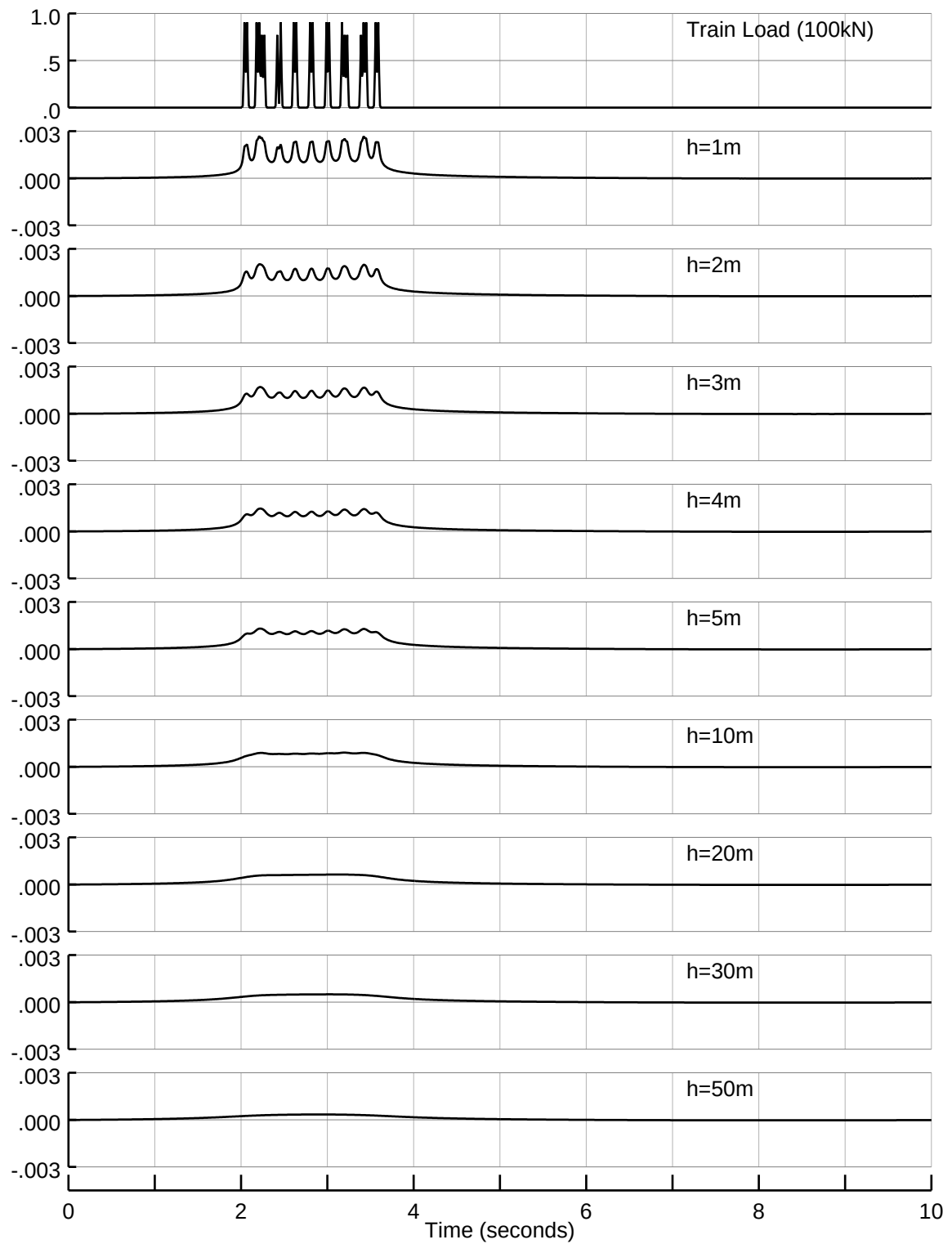


Figure 3 – Displacements (cm) induced by a High-Speed Train with a speed of 100 m/sec on Soil Profile #1.

But the results show there is nearly no effect caused by the choice of damping values. In fact, the peak acceleration is less than 5 cm/sec^2 at a distance of 1m from the track and it decreases to near zero at the distance of 5m. The peak velocity at 5 meters is less than 0.5 mm/sec, it was determined that a walker with hard sole shoes on a floor would cause a disturbance of about 20 mm/sec. It is therefore safe to conclude that with a strong rock site, the vibrations caused by a high-speed train is negligible at any reasonable distance from the track.

One interesting result to indicate from the analyses of soil profile #1 is shown in Fig. 3. The displacement as a function of time is shown for various distances from the track. Although they are very small, the shape of the response functions show that the high frequency vibration attenuates much faster than the low frequency counterpart. Furthermore, the displacement response at the farther distances show that the vibrations started before the train arrives and that the ground waves propagates to the site well ahead of the train. As will be shown later, this effect would not be duplicated when the train is fast compared to the shear wave velocity.

Soil profile #2 is significantly softer than soil profile #1. But a high train speed of 90 m/sec is still well below the top-layer shear wave velocity of 150 m/sec. The peak acceleration right next to the track is about 120 cm/sec^2 , slightly over 0.1g. But again the decay is rapid and the vibration disappears at distances greater than 10m. With the train speed decreased to 60 m/sec or 30 m/sec, the peak accelerations are respectively, 24 cm/sec^2 and 4 cm/sec^2 . At the same train speeds, the peak velocity at the distance of 1m are 20 mm/sec, 6 mm/sec and 2 mm/sec, respectively. It is therefore safe to state that subsonic train speeds with Mach numbers of less than 0.5, the vibration level is not of concern to the environment. Shown in Fig. 4 is the acceleration as a function of time for various distances from the track. Because

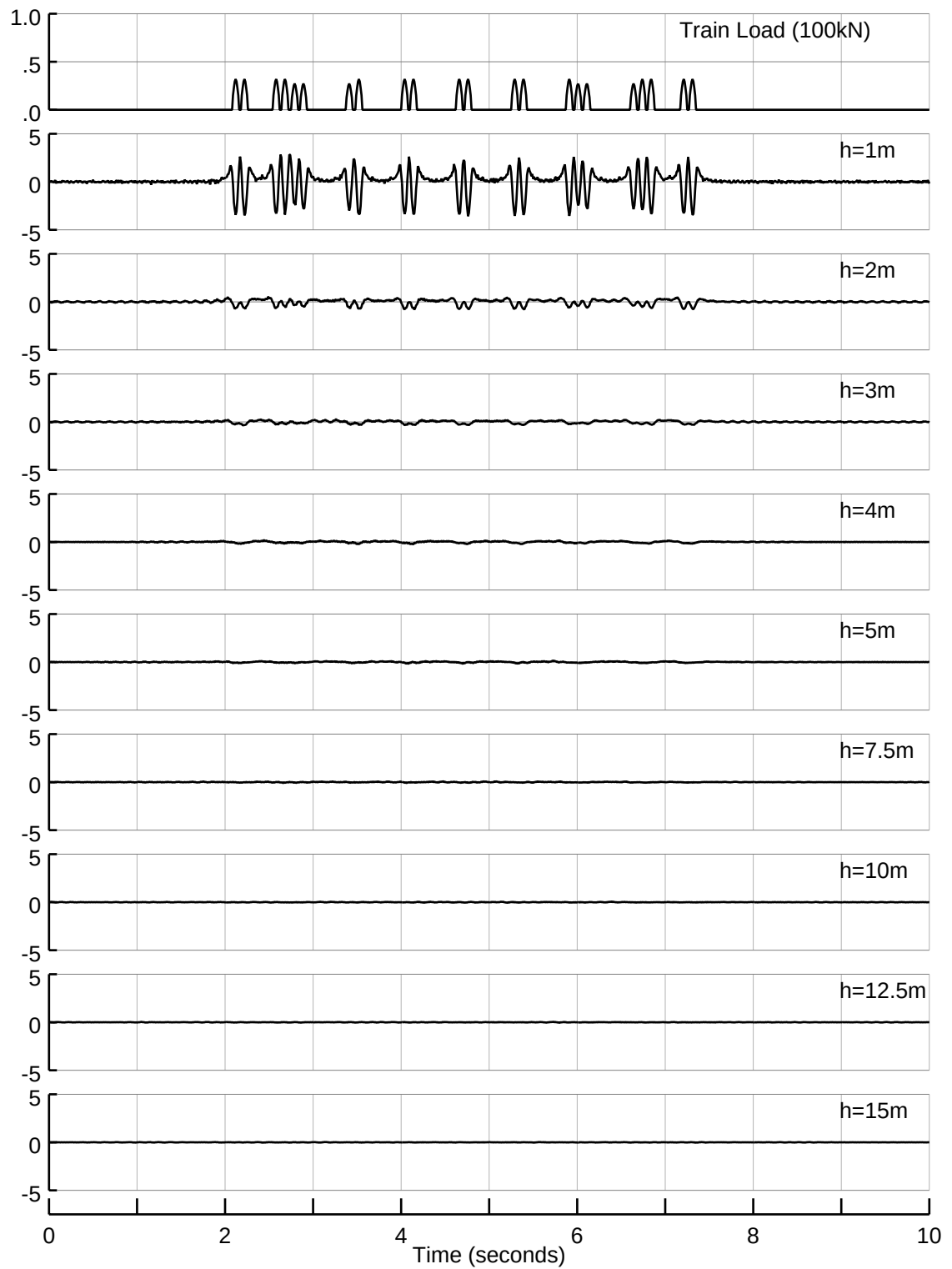


Figure 4 – Accelerations (cm/sec^2) induced by a High-Speed Train with a speed of 30 m/sec on Soil Profile #2.

the train speed is slow at 30 m/sec, the loads are spread out over time and the effective amplitudes of the load is much smaller than those of the higher train speeds.

Soil profile #3 is an actual site along the HST route from Brussels to Paris with documented data. The top-layer shear wave velocity is 80 m/sec and it is in the range of the top speed HST train of about 88 m/sec. In this numerical experiment, the train speeds of 100, 80, 60 and 40 m/sec will be used. The 100 m/sec is faster than the existing trains but it is used here to study the effects of wave propagation if the Mach number is 1.25 for the soil. The 80 m/sec train speed is right at the critical speed of the soil medium.

The peak acceleration when the train speed is 100 m/sec is uncommonly high: it is about 3 to 4g at the distance of 3 to 4 meters from the track. At 10 meters, it is still 1.5g and it decays to 0.3g at 20 meters. The high values are likely incorrect because the soil behavior would become nonlinear at that high level of excitation and the assumption of the present theory is inadequate. But nevertheless, this analysis shows that the level of vibration is very high when the train load is faster than the shear wave velocity. At the critical train speed of 80 m/sec, the values are more reasonable: 1g at 3 to 4 meters and 0.05g at 10 meters. For subsonic speeds of 60 and 40 m/sec, the peak accelerations at the distance of 3 to 4 meters are 12 cm/sec^2 and 2 cm/sec^2 , respectively.

The peak velocities for train speeds of 100, 80, 60 and 40 m/sec are respectively, 30, 6, 2, and 0.1 cm/sec. The value of 30 is near an earthquake value but the train vibration decays far more rapidly than the earthquake waves because the size of the wave source is much smaller. Shown in Fig. 5 is the variation of velocity time histories as a function of distance for a supersonic train speed of 100 m/sec. The figure shows that the response at distanced locations do not start until the train has gone by, a clear indication that a shock wave has developed.

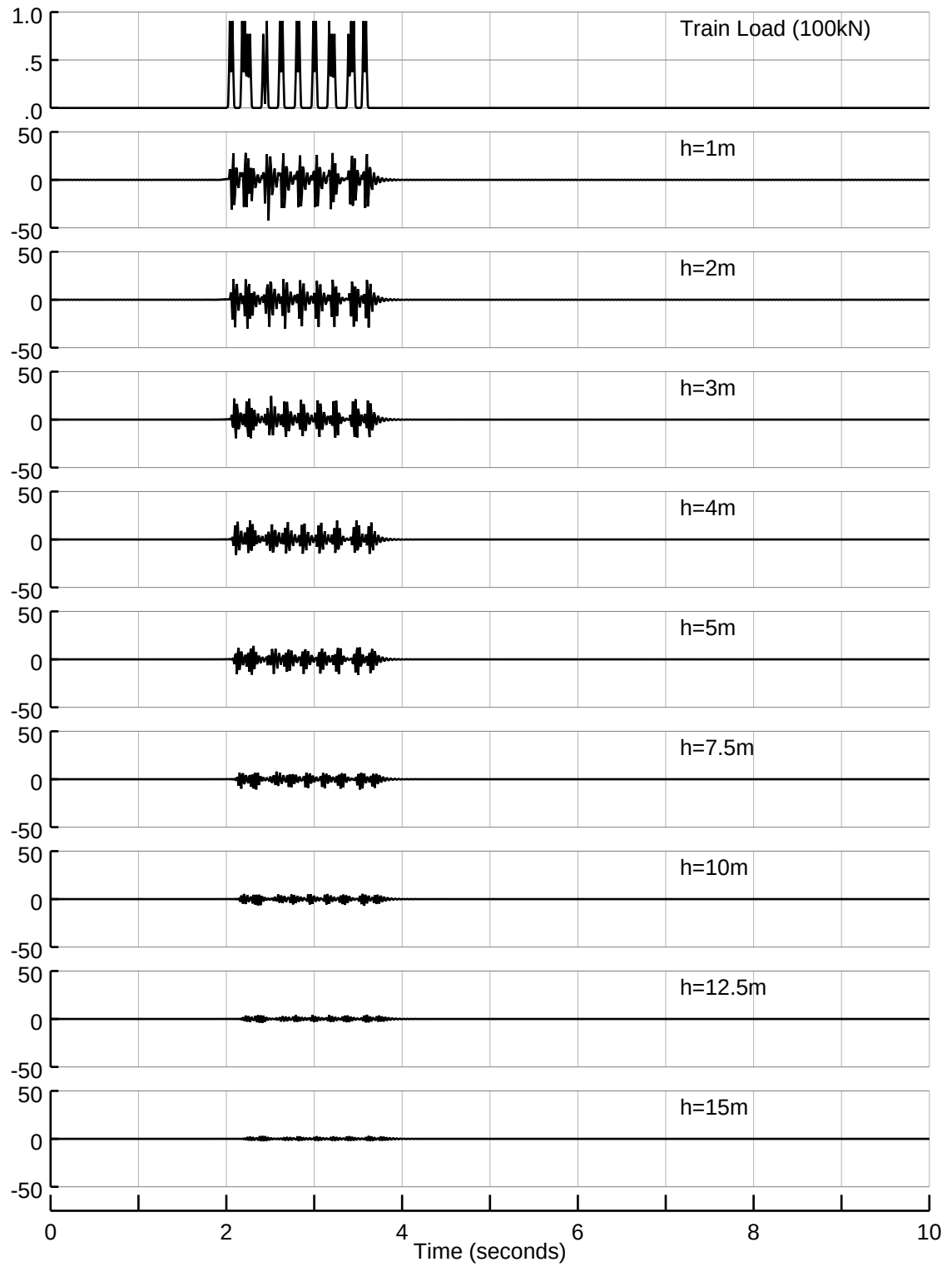


Figure 5 – Velocities (cm/sec) induced by a High-Speed Train with a speed of 100 m/sec on Soil Profile #3.

Soil profile #4 is an approximate replica of a Swedish site (Dowding 2000) that is composed of a 2 m stiff crust that overlays 5 m of soft marine clay, which in turn overlays a half space of stiff clay. Even though the top layer shear wave velocity is assumed to be 200 m/sec, well above the high speed train speed, the second layer is a soft marine deposit with a shear wave velocity of only 40 m/sec. The analysis of this soil profile is the most interesting of all the examples, the wave energy is trapped between the stiffer layers and the ground waves travel much greater distances than the other soil profiles considered.

Shown in Fig. 6 is the variation of accelerations results from a train speed of 80 m/sec. The peak acceleration is roughly 200 cm/sec^2 ($0.2g$) near the track but it is still 40 cm/sec^2 at 50 meters. The attenuation rate is much less than the other soil profiles considered. one interesting effect to consider is that the free oscillations continued for a long time after the train has gone by, indicating that energy is trapped between the layers. The velocities for the same train speed is shown in Fig. 7, the same trend exists but more prominent than the accelerations because of the lower frequency content. At the distance of 50 meters, the peak velocity is about 30% of that near the track. It is probable that the vibration would be felt at much greater distances, depending on the level that is tolerable for the particular purpose of the building.

As an academic exercise, a hypothetical train speed of 100 m/sec is used and the velocity time histories are shown in Fig. 8. Again, the attenuation is gradual over distance but it is clear that the high frequency content is filtered out rather quickly. The free oscillation after the load has gone by indicates a strong resonance frequency of the layered medium of about 2.5 cycles per second. A frequency of 2.5 Hz, or a period of 0.4 seconds, is approximately the resonance frequency of a typical 4-storey building, as estimated by an engineering formula that roughly predicts the period of a building as the number of storeys divided by 10.

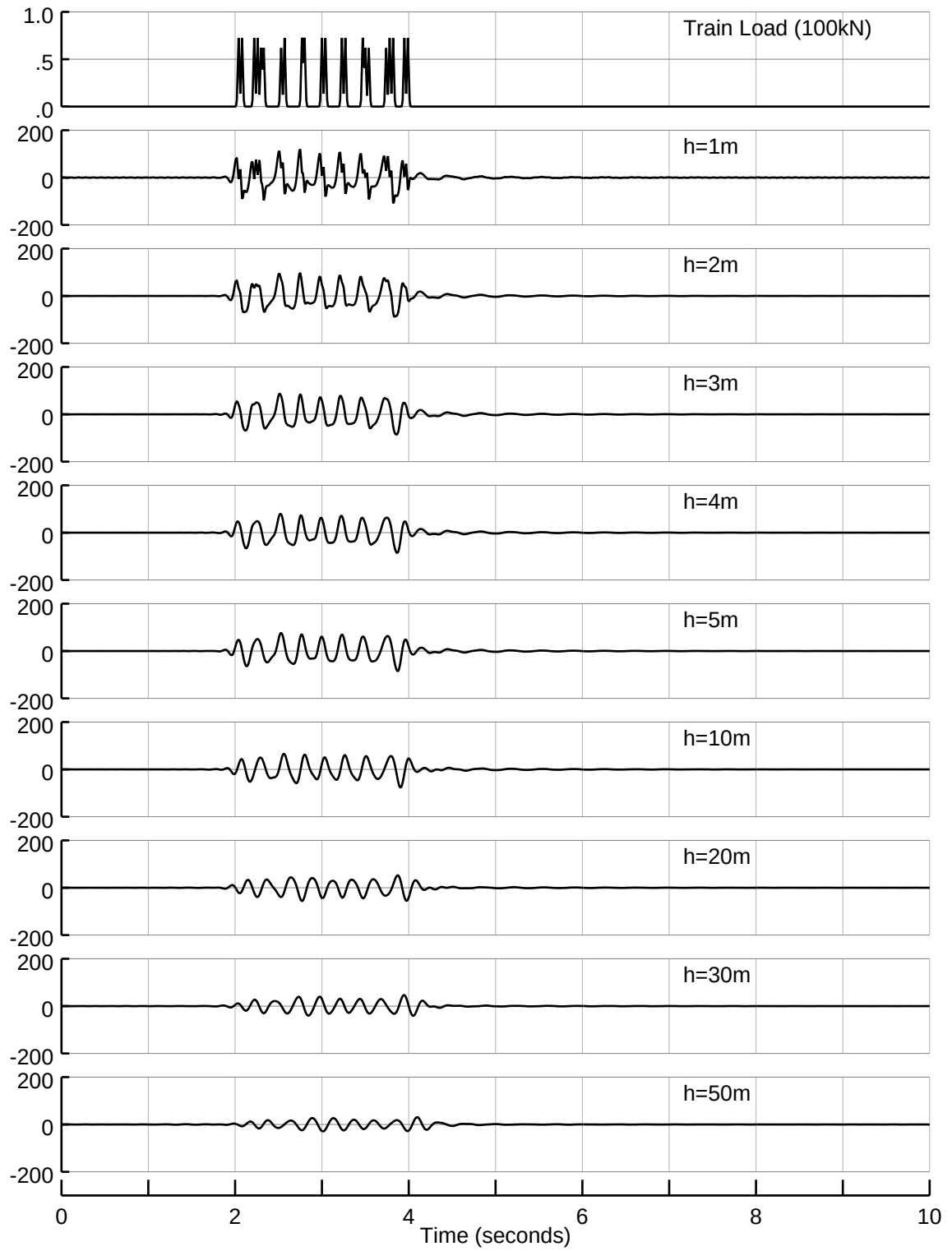


Figure 6 – Accelerations (cm/sec) induced by a High-Speed Train with a speed of 80 m/sec on Soil Profile #4.

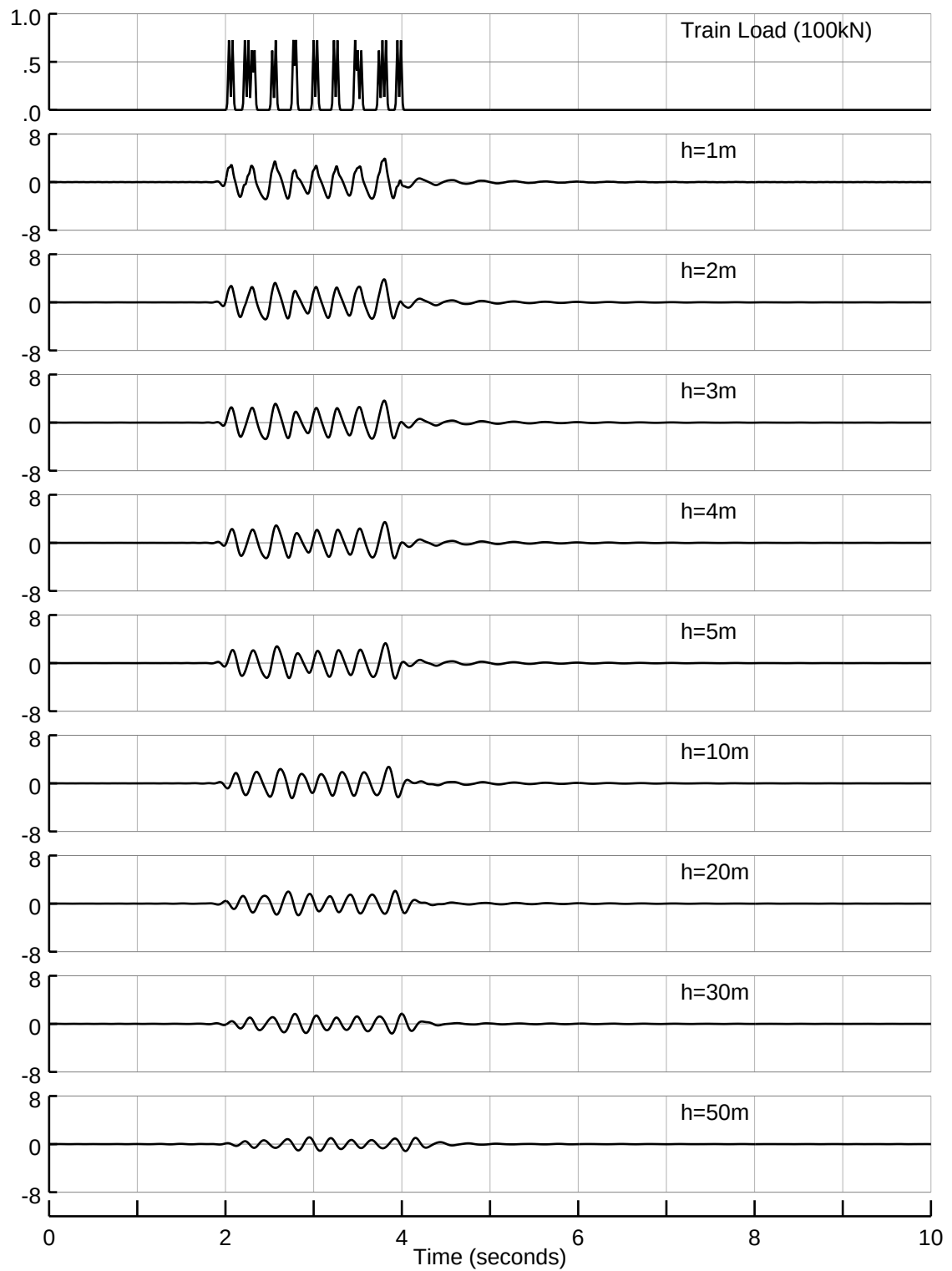


Figure 7 – Velocities (cm/sec) induced by a High-Speed Train with a speed of 80 m/sec on Soil Profile #4.

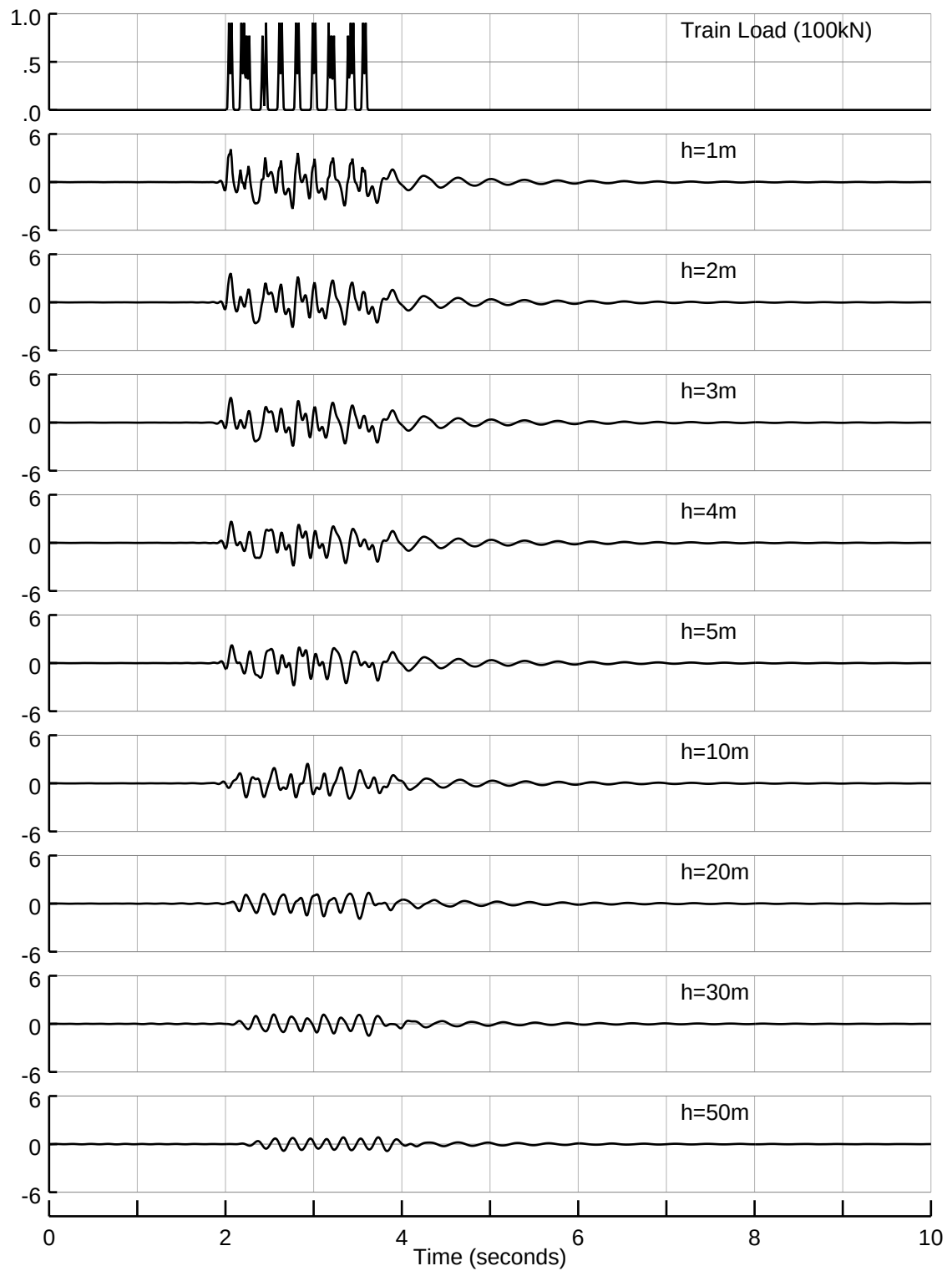


Figure 8 – Velocities (cm/sec) induced by a High-Speed Train with a speed of 100 m/sec on Soil Profile #4.

Soil profile #5 is a simplified soil profile of the Rinaldi Site in Southern California. The shear wave velocity of the top layer is 100 m/sec, with another soft layer directly below. The Rinaldi site recorded the highest velocity in recent history, 183 cm/sec, during the Northridge, California earthquake of 1994. Shown in Fig. 9 is the velocity time histories excited by a train load at a speed of 80 m/sec. The peak velocity is about 30 cm/sec, far below that of an earthquake. At the distance of 5m, the peak velocity decreased to 10 cm/sec and at 20m, the peak velocity is down to 0.2 cm/sec. The rapid attenuation is not due to material damping as numerical experiments in this project have shown; the material damping is not an important factor. Radiation damping caused the majority of the attenuation as the outgoing waves spread geometrically through a vast soil medium. The radiation effects are modelled well by the present Continuum Mechanics approach, something not done particularly well by a finite elements approach.

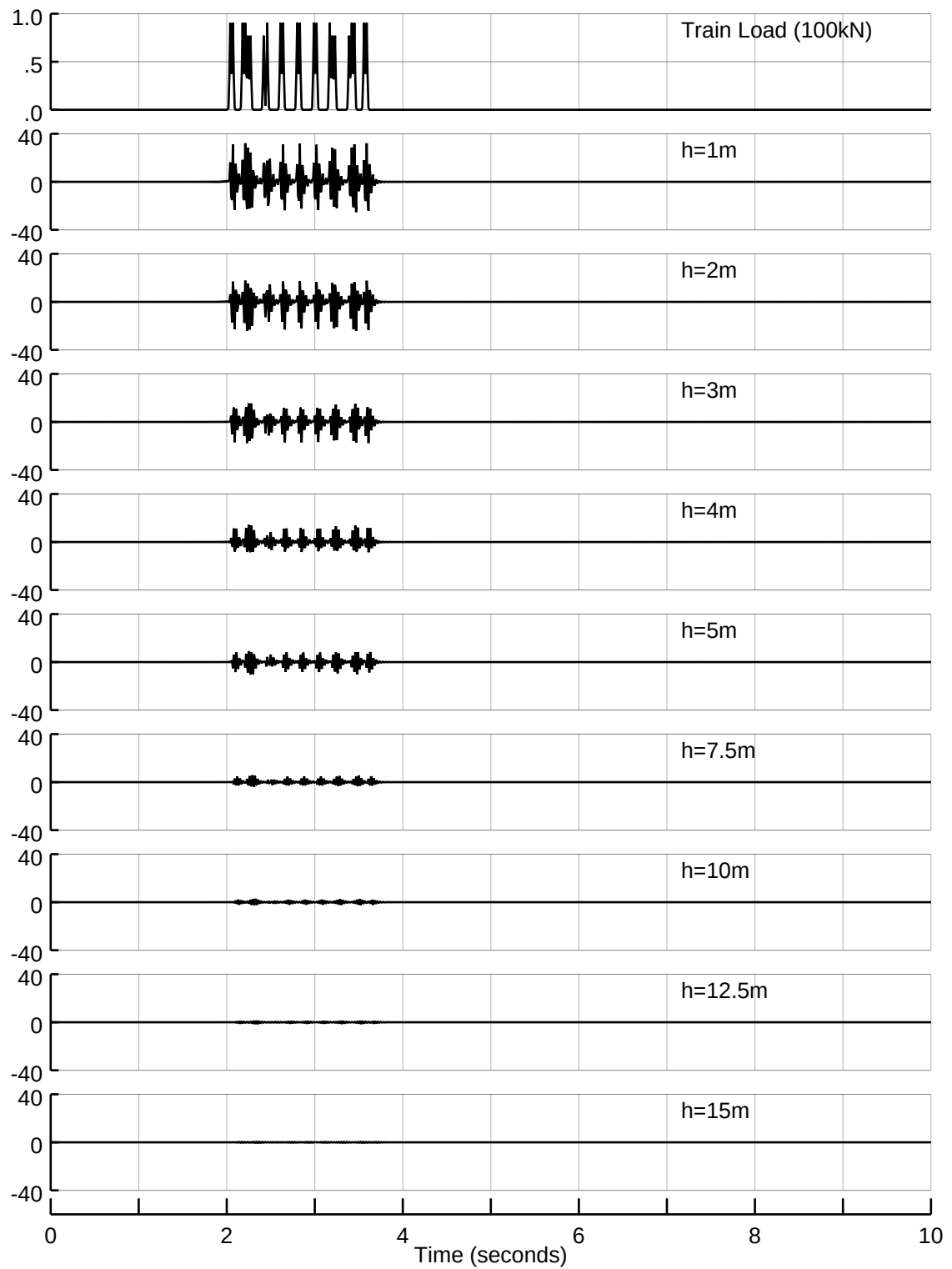


Figure 9 – Velocities (cm/sec) induced by a High-Speed Train with a speed of 80 m/sec on Soil Profile #5.

The effects of Soil-Structure Interaction

The effect of ground-borne vibration can affect the occupants of nearby structures. When the apparent velocity of the incoming wave is low, the variation of the ground motion within the area of the foundation is significant. Foundation footings, which can be approximated as rigid plates on the surface of the soil medium, as shown in Fig. 10, will move relative to each other, generating unwanted vibrations in the building, at least in the lower floors. Although the vibration level is quite low compared to earthquakes, structures with large horizontal dimensions such as a large warehouse, or an industrial complex, can have uncomfortably high level of vibrations if certain resonant modes of the structure are excited.

In seismic analyses, the effect of soil-structure interaction is often neglected for convenience even though after-earthquake damage reports clearly showed the importance of that effect. For those analyses which considered the effects of soil-structure interaction, the foundation is often assumed to move as a rigid body, with three degrees of freedom for translation and another 3 for rotation. But all of the above assumptions would not be valid for cases in this research project because a rigid foundation would filter out most of the frequencies of interest and thereby defeating the purpose of the analysis.

Another possible analysis is to constraint the building columns to move with incident ground motion, ignore the soil-structure interaction effects. But that would over-estimate the vibration levels because the massive building would certainly scatter away a large portion of the wave energy. The only reasonable assumption to be made in this research to analyze the structure including soil-structure interaction with a flexible foundation assembly.

Given a building supported on a flexible foundation subjected to travelling waves, the equations of motion for the structure-foundation system can be written in matrix form,

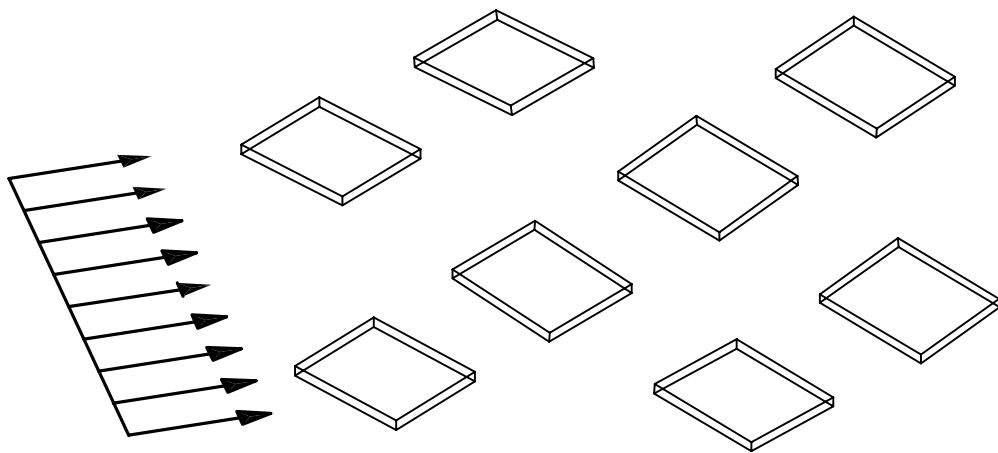


Figure 10 – Kinematic Interaction of Foundation Plates with an Incident Wave

in the frequency domain, as

$$\left(-\omega^2 \begin{bmatrix} M_{bb} & 0 \\ 0 & M_{oo} \end{bmatrix} + i \begin{bmatrix} \omega C_{bb} & 2\xi K_{bo} \\ 2\xi K_{ob} & 2\xi K_{oo} \end{bmatrix} + \begin{bmatrix} K_{bb} & K_{bo} \\ K_{ob} & K_{oo} \end{bmatrix} \right) \begin{Bmatrix} U_b \\ U_o \end{Bmatrix} = \begin{Bmatrix} 0 \\ -F_o^s \end{Bmatrix} \quad (17)$$

in which subscripts b and o refer to nodes on the superstructure and on the foundation, respectively. The displacement vectors, $\{U_b\}$ and $\{U_o\}$, correspond to generalized displacements, 3 translations and 3 rotations, with respect to a fixed frame of reference. The force vector, $-\{F_o^s\}$, corresponds to the generalized forces that the soil exerts on the foundation. In writing Eqn.(17) it has been assumed that the damping associated with foundation nodes is of the hysteretic type, while that associated with the superstructure is of the viscous type.

If the structure-foundation system is isolated, i.e., in absence of other structures, the force vector, $\{F_o^s\}$, can be written in the form

$$\{F_o^s\} = [K_{ss}] (\{U_o\} - \{U_o^*\}) \quad , \quad (18)$$

in which $[K_{ss}]$ is the impedance matrix for the soil and $\{U_o^*\}$ is the foundation input motion. In some formulations, the notation of the driving force, $\{F_o^*\}$, is also use; it corresponds physically to the foundation footings being held fixed while subjected to ground vibration, mathematically, it is defined as

$$\{F_o^*\} = [K_{ss}] \{U_o^*\} \quad . \quad (19)$$

The partitioned matrix equation in eqn. (1) can be separated into two matrix equations as follows:

$$(-\omega^2 M_{bb} + i\omega C_{bb} + K_{bb}) U_b = -(1 + 2i\xi) K_{bo} U_o \quad , \quad (20)$$

$$(-\omega^2 M_{oo} + (1 + 2i\xi) K_{oo} + K_{ss}) U_o = F_o^* - (1 + 2i\xi) K_{ob} U_b \quad . \quad (21)$$

Eqn. (20) is basically the equation of motion for a structure with its base nodes fixed and the effects of a flexible base motion have been distributed to the lower part of the superstructure as external forces by virtue of the coupling matrix, K_{bo} . Since the model of the superstructure is now one with a fixed base, it can be solved by a modal method. Let the matrix, Φ , be one which contains the mode shapes of the superstructure as column vectors and that the columns are normalized such that, $\Phi^T M_{bb} \Phi = I$, and that the operation, $\Phi^T K_{bb} \Phi$ yields a diagonal matrix containing the terms, ω_r^2 , where ω_r is the natural frequency of the r -th structural mode. After several steps of algebraic manipulations. the response of the foundation nodes can be obtained as

$$U_o = \left(Z(\omega) - \frac{(1 + 2i\xi)^2}{\omega^2} \beta_f^T D_1(\omega) \beta_f \right)^{-1} F_o^* \quad , \quad (22)$$

in which the modal participation factor matrix can be defined as

$$\beta_f = \Phi^T K_{bo} \quad . \quad (23)$$

After U_o is obtained as the solution of the soil-structure interaction problem, the response of the superstructure can then be calculated using

$$U_b = -\frac{(1 + 2i\xi)}{\omega^2} \Phi D_1(\omega) \beta_f U_o \quad . \quad (24)$$

In this formulation, the most difficult part of the analysis is the generation of the foundation impedance matrix, K_{ss} , which represents all the through-soil effects between the foundation footings, and the driving forces vectors, F_o^* . Using the methodology of CLASSI, an acronym for the Continuum Linear Analysis for Soil-Structure Interaction, the impedance matrix and driving forces for multiple foundation plates can be obtained as a function of frequency.

As part of this research project, a new iterative algorithm based on the classical Jacobi or Gauss-Siedel methods was developed because of the large number of foundation footings

required for a reasonable model. Typical analyses of this magnitude, i.e., with 10 or more footings and a layered soil medium, would require several days of computation on a relatively fast computer. But the newly developed algorithm, doing the iterations matrix block by matrix block, can obtain a numerical solution an order of magnitude faster if the typical separation distances between the rigid plates are used. With the soil impedances and driving forces calculated using CLASSI, the modelling of the superstructure was performed using an existing finite element program, yielding the mode frequencies and mode shapes, the stiffness and mass matrices of the superstructure including the coupling matrix, K_{bo} , for the base nodes.

Several building models were analyzed and the results were the same as what is obvious, that the damping of the building modes is the major parameter. If the building is highly damped, the response to ground-borne vibration is minimal. But if the building is lightly damped, the building modes might actually amplify the incoming waves. More analyses are needed to pinpoint a more effective way to isolate the vibrations.

Conclusion and Recommendations

The effects of a moving load from a modern high speed train was analyzed using five soil profiles representative of the soil conditions in Southern California and in Europe. The results show that for most cases the attenuation of waves is rapid and that a hundred-fold decrease in amplitudes for accelerations, velocities or displacements is likely within about 10 meters. The assumed material damping value of the soil medium was found to be a minor factor, the attenuation is caused mostly by radiation damping resulted from the geometric spreading of ground waves. Only one soil profile, that which approximate the soil medium in Ledsgard, Sweden, has a slow attenuation factor through distance. In that particular site, a soft layer is sandwiched between two stiff layers and the wave energy was confined to the surface. Since a train track usually covers a long distance, some peculiar soil condition is likely to occur at sections of the route, a careful soil analysis should be performed with environmental impact in mind.

Soil-structure interaction analyses for buildings with flexible foundations were also performed and found that the modal damping values are the most important factors in the analysis. Although some ground vibration amplitudes were diminished by the kinematic interaction between the rigid footings and the incident ground waves, the vibration can also be amplified by resonance within the structure. It is recommended that for buildings where sound and vibration isolation is critical, the modern methods for increasing the damping of the structure should be considered.

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Appendix A

Southern California Soil Properties

The soil properties in Southern California are well known because of the many years of seismic research in the area. The ROSRINE (acronym for Resolution Of Site Response Issues from the Northridge Earthquake) website provides data for many sites.

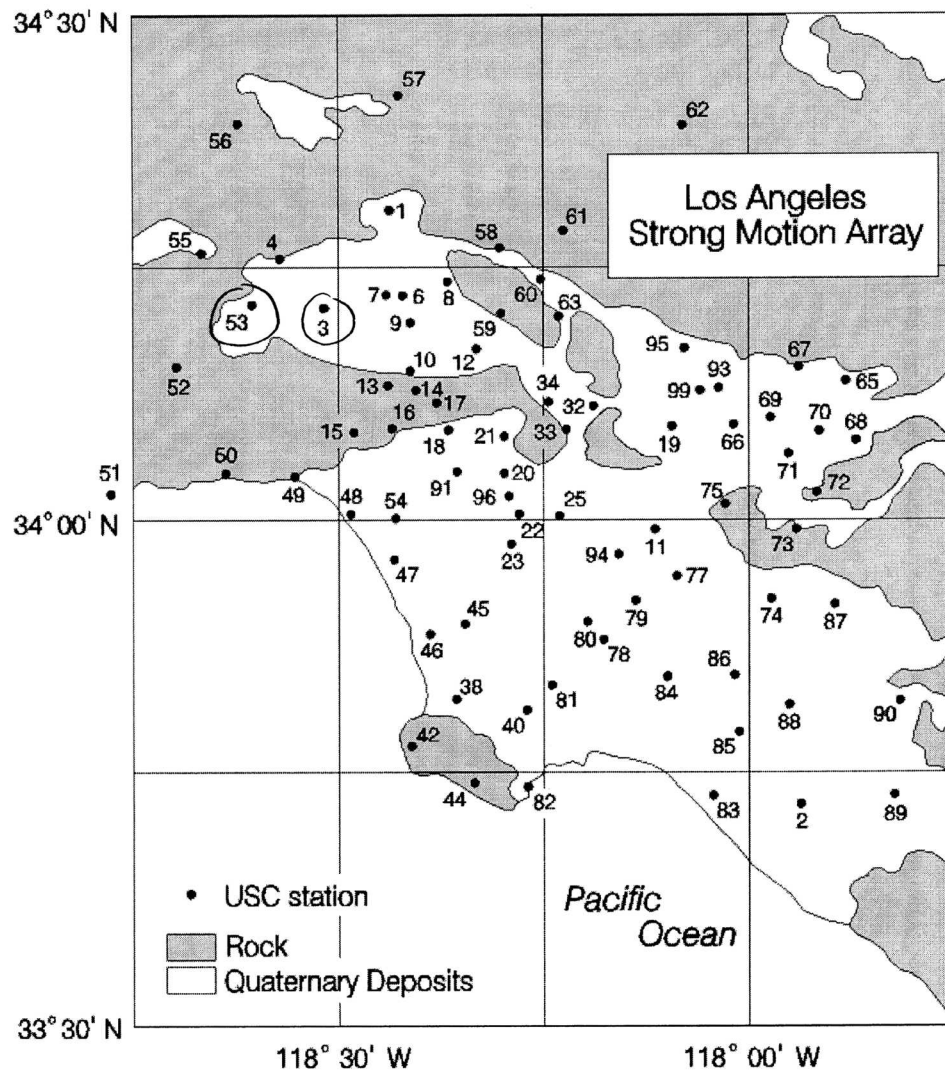


Figure A1 – Los Angeles Area Strong Motion Array of the University of Southern California

Table A – Simplified Soil Profiles in Southern California
ROSRINE Data Set
[Layer Thickness, S-Wave Velocity, P-Wave Velocity]

Pacoima Downstream (PAC)	[∞,1200,2750]		
Newhall Fire Station (NWH)	[30,180,300]	[∞,700,1200]	
Arleta (ARL)	[18,170,300]	[∞,500,1200]	
Kagel Canyon (KAG)	[8,220,450]	[∞,500,950]	
La Cienega (Multiple Holes) (LCN)	[38,300,1800]	[∞,800,1800]	
Sepulveda VA #5 B-2 (SPV2)	[22,300,?]	[60,500,?]	[∞,600,?]
Tarzana (TAR)	[20,200,400]	[∞,400,1000]	
Baldwin Hills (BLD)	[10,161,350]	[∞,240,800]	
Portrero 1 (SMA Site) (PIC)	[15,150,250]	[∞,600,1300]	
Portrero 2 (Valley Edge) (PI2)	[8,250,400]	[∞,600,1200]	
Portrero 3 (Valley Center) (PI3)	[10,110,190]	[20,200,1200]	[∞,600,2300]
Rinaldi 2 (RRS2)	[2,100,180]	[4,180,320]	[∞,300,800]
Lake Hughes #9 (LH9)	[8.5,225,500]	[∞,1100,2400]	
Meloland 1 (EMO)	[60,150,1600]	[∞,350,1800]	
Dayton Heights Elementary (WST)	[3,120,500]	[11,270,850]	[∞,500,1800]
Saturn Elementary (SAT)	[10,200,200]	[∞,350,800]	
Bell-LA Bulk Mail (LBM)	[3,180,300]	[19,250,600]	[∞,500,1700]
Yermo (YRM)	[15,310,450]	[25,400,700]	[∞,600,1800]
Joshua Tree (JST)	[5,190,400]	[40,400,700]	[∞,700,1200]
Halls Valley (HLV)	[20,240,1500]	[20,350,1700]	[∞,800,2000]
Gilroy 3 (GR3)	[5,120,240]	[15,200,1000]	[∞,500,1800]
Superstition Mountain Top (SMT)	[10,200,620]	[∞,350,1200]	
IBM, Santa Teresa Hills (IBM)	[2,200,420]	[5,400,1000]	[∞,1000,2000]

Table A (continued)

Jensen Generator Bldg (JEN)	[6,450,600]	[∞,600,1150]	
Jensen Main Building (JEM)	[18,310,800]	[∞,456,1930]	
Olive View Hospital (SYL)	[17,200,385]	[∞,350,1740]	
Sherman Oaks Park (SOP)	[17,200,385]	[∞,350,1700]	
Sepulveda VA B-1 (SPV1)	[25,320,360]	[∞,400,500]	
Downey South Middle School (BIR)	[23,220,400]	[∞,380,1800]	
Willowbrook Park / Gardena (116)	[60,320,320]	[∞,500,1700]	
Dolphin Park / Del Amo (WAT)	[40,200,1200]	[∞,400,1600]	
Pico Rivera #2 (PR2)	[20,200,280]	[50,450,1500]	[∞,700,2000]
Sylmar Converter E	[8,250,820]	[12,400,1000]	[∞,500,1800]
Sylmar Converter E#2	[20,250,400]	[∞,450,1800]	
Wadsworth VA Hospital North (WAN)	[20,380,600]	[∞,500,1700]	
Wadsworth VA Hospital South (WAS)	[3,150,500]	[∞,400,1700]	
Brentwood Va Hospital (BVA)	[30,400,800]	[∞,700,1700]	
ETEC RD-7 (SSU)	[∞,1000,1700]		
ETEC RD-20 (SS2)	[8,1000,2000]	[∞,1250,2800]	
LADWP Receiving Station E (LRS)	[15,200,200]	[∞,400,500]	
Parachute Test Site Control Bldg.	[3,200,380]	[∞,400,850]	
Pier F Long Beach (PRF)	[25,150,1700]	[∞,350,1700]	
Wonderland Elementary (WON)	[4,400,700]	[∞,1000,2000]	
Obregon Park (OBR)	[5,450,800]	[∞,450,1800]	
Mira Catalina School (LUC)	[∞,1200,2200]		
Griffith Observatory (GPK)	[10,700,1300]	[∞,1400,2600]	
La00-Stone Canyon Reservoir (LA0)	[10,500,1200]	[∞,1000,2700]	