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Vibration Annoyance Assessment of Train Induced Excitations from Tunnels Embedded in Rock

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Abstract

Train movements generate oscillations that are transmitted as waves through the track support system into its surroundings. The vibration waves propagate through the soil layers and reach to nearby buildings creating distractions for human activities and causing equipment malfunctioning. Not only the train components and the rails, but also the surrounding tunnel, soil and rock strata have dynamic characteristics that play significant roles in the vibration levels felt in a nearby structure. This paper presents a finite element study conducted to investigate the vibrations resulting from train movements in nearby subway tunnels. The subway line is located at an average horizontal distance of 50 ft (15.2 m) from the structure in assessment, which is a six-story office building. The main goal of the work is to assess the train-induced vibrations at the ground level of the building through a case study and sensitivity analysis. A plane strain finite element model is built to represent the railroad tunnel embedded in the rock and the soil stratum above it. The one train loading function is applied to the model as a point source at the track level and compared to the two-train scenario. Other simulations are undertaken for sensitivity analysis involving increased loading, decreased damping and decreased distance to tunnels. Even though there are several numerical studies on the propagation of train induced vibrations in the literature; a finite element model accompanied with a sensitivity analysis has not been discussed in detail in a technical publication before. The paper not only presents the finite element modeling but also compares the results with the criteria of *Transit Noise and Vibration Impact Assessment Manual*, which was published by the Federal Transit Administration (FTA) of the U.S. Department of Transportation.

Keywords: Vibrations disturbance; train vibrations; ground-borne vibrations; railroad tunnels; tunnels embedded in rock; ANSYS; finite element modelling

1. Introduction

Occupants in buildings are sensitive to vibrations. While most perceptible indoor vibrations are originated from sources within the buildings (human movement, mechanical units, etc.), there are also outdoor sources of vibrations (trains, construction equipment, vehicle traffic, etc.) propagating through the ground, penetrating the building and creating vibratory distractions for building occupants (Sanayei et al. 2014; Xia et al. 2010). The vibrations generated due to vehicles on tires are generally observed to be less than the vibrations caused by train excitations. Among the outdoor sources of vibrations, even though construction blasting and pile-driving (Grizi et al. 2016) have been reported to result in damage on buildings, the train vibrations seldom result in building damage. However, there are cases where excessive train vibrations cause extensive distractions to building occupants such as decreasing work focus in office environments during day time and negatively affecting sleep quality during night time. In addition, such vibrations might result in malfunctioning of vibration sensitive equipment. Meanwhile, the train vibrations can be felt by people standing outdoors, however it is seldom that people that are outside complain about outdoor vibrations.

Train induced noise and vibration have been an area of research over the decades (Degrande and Schillemans 2001; Kephalopoulos et al. 2014; Yang et al. 2019; Zou et al. 2017). Regardless of the train being at the ground level or in a tunnel, ground-borne vibration is a predominant concern for the building occupants especially in the close vicinity of transit system routes (Kouroussis et al. 2016; Licitra et al. 2016; Ngamkhanong and Kaewunruen 2018; Peris et al. 2014, 2016). Vibration levels generated by trains are dependent on several factors (Connolly et al. 2016; Lee et al. 2014; Zou et al. 2015). While the material and dynamic properties of the train components, tunnels and surrounding geological conditions are one set of factors (Cai et al. 2010; Galvín and Domínguez 2007; Sheng et al. 2006); wheel smoothness, rail smoothness, vehicle suspension system, track support system are another set (Fiala et al. 2007; Metrikine and Vrouwenvelder 2000; Sheng et al. 2004; Triepaischajonsak et al. 2011). For instance, dynamic characteristics of the soil stratum and the depth of the bedrock are important parameters affecting wave propagation (Avillez et al. 2013; Yang and Hsu 2006). While the wave propagation is more effective in stiff clays, when the depth to bedrock is 30ft (9.1 m) or less, it is known that vibration energy is more pronounced near at-grade track. This would most probably result in vibration problems even at large distances from the railway tracks (Bian et al. 2015; Hanson et al. 2018). Also, for the tunnels embedded in rock the train vibrations do not attenuate as rapidly as in soils. It is also observed that the buildings with heavier foundation and superstructure tend to experience relatively smaller vibrations than the lighter structures. The maximum response of the structure will mainly depend on the level of vibrations reaching the building foundations, modal characteristics of the building, and the coupling of the building foundation to the soil (soil-structure interaction).

As the steel wheels of the trains roll on the rails, vibratory forces are generated. As the wheel meets a discontinuity like a joint, a reactive force is exerted on the wheel and hence on the train. When the vibration of the transit structure generates oscillatory waves propagating away within the soil layers, vibrational energy moves through the surrounding media in a variety of wave forms (shear, compression and Rayleigh waves). The Rayleigh waves carry considerable amount of vibrational energy. In the presence of multi-layered soils, the mathematical modeling of vibration is even more complicated; therefore, FE modeling appears as a reasonable way to model such conditions in a relatively easier and practical way (Andersen and Jones 2006). Yet, vibration propagation from the railways to the structure foundations is complex and requires extensive FE modeling. On a different note, the interaction between the soil and the foundation is significantly important for vibration assessment with FE models (Galvín et al. 2010; Galvín and Domínguez 2009; Hall 2003; Ju and Lin 2004; Yaseri et al. 2014). Once the oscillations propagate through the structure, based on the intensity levels, the vibration can be felt by occupants and/or cause malfunctioning on sensitive equipment. The propagation of oscillations from the foundation to the upper levels of the structure is also complex and has been researched in detail. The propagation within the building depends on the type of the building and the structural design.

Since railway induced vibration propagation involves many parameters, it is difficult to estimate the resulting vibrations at a specific location (Park et al. 2008a; b). Therefore, the majority of assessment techniques utilized for transit projects are based on empirical data. For more project specific estimations, where vibration impact is probable, detailed FE models are utilized by researchers and engineers for estimating the vibration levels. While the DVA method typically assumes a steel-wheel/rail system, it is based on frequency domain information typically in terms of one-third octave-band spectrum.

2. Vibration analysis levels per the Transit Noise and Vibration Impact Assessment Manual

The United States Department of Transportation's Federal Transit Administration (FTA) released an updated version of the Transit Noise and Vibration Impact Assessment Manual (Hanson et al. 2018), which outlines means and methods for prediction and assessment of vibration (and noise) impacts of transit projects as part of FTA's environmental review process. The manual includes clarifications to existing policy and updates to outdated references, where applicable, but does not change the existing assessment procedures of previous versions of the document.

In the FTA Transit Noise and Vibration Impact Assessment Manual (Hanson et al. 2018), there are three levels of procedure presented to assess the potential ground-borne vibration impacts resulting from a train line. The appropriate level of analysis is a function of the project specifics such as environmental settings, scale and type of the transit project. According to the FTA manual, there are three levels of analysis:

- Vibration Screening Procedure (VSP)
- General Vibration Assessment (GVA)
- Detailed Vibration Analysis (DVA).

In order to determine which level of procedure to apply, the primary item to consider is the presence of any vibration sensitive land use through the VSP. This screening procedure determines the work area of any vibration assessment process. In other words, if there is not any vibration-sensitive equipment present within the defined screening distance, then there is no need for further vibration assessment. When the major project details are known relatively early in the project development stage, the GVA procedure is implemented. The GVA procedure is conducted if there is any potential for considerable vibration levels. For this, vibration levels at receiving locations are determined by predicting the overall vibration velocity level as a function of distance from the rail-tracks. This process also includes adjustments to consider factors like the building type, track and wheel conditions, vehicle speed and track support systems. The GVA is adequate for the environmental review of standard projects where transit modal alternatives are considered for a potential relocation or any other project modification. The GVA is considered adequate when there is a commitment to mitigate vibration impacts, such as a change in transit mode or alignment. However, if the impact is identified through the GVA procedures and not mitigated, a DVA must be conducted. The DVA is conducted to determine the severity and extent of the impact especially for sensitive buildings, i.e., in the close vicinity of train routes. If needed, vibration mitigation measures would also be implemented after a DVA procedure. On another note, a DVA may be warranted earlier in the environmental review process if there are impact indications regarding the closeness of vibration-sensitive structures. This type of assessment task requires experienced professionals to perform tests and post-process data.

On another note, a DVA may not be necessary for all segments of a project. Generalized prediction curves from the GVA procedures may be sufficient for most of the alignment, and the DVA procedure may only need to be applied to particularly sensitive receivers. Furthermore, a DVA is typically required for special type of track-support systems (e.g., ballast mats, floating slabs). Costly vibration mitigation techniques can only be called following a DVA in the design stage.

The type of work conducted in this paper is a typical example of a DVA procedure where the vibration impact of twin tunnels embedded in rock are researched in detail to make sure that the occupants of an existing nearby six story office building do not get affected by the vibrations generated by the trains passing by. The railway line is an additional line to an existing subway system of a relatively large city located in North America. The proposed alignment of the railway tunnels is passing adjacent to a six story office building and right underneath a six story parking garage. A detailed finite element (FE) model is built and train excitations are simulated to observe the vibration responses at the base of the existing building.

The criteria for the DVA and the interpretation of vibration criteria are summarized in Figure 1 and Table 1 which are both adapted from the FTA Assessment Manual (Hanson et al. 2018). On Figure 1, the x-axis is one-third octave band center frequency while the y-axis is the vibration velocity level. The DVA criteria of Figure 1 and Table 1 are based on international standards for the effects of vibration on people related to annoyance and interference with activities in buildings (Hanson et al. 2005). The criteria also covers the extensively used standards for vibration-sensitive equipment (Nelson et al. 1982). Specifically, these criteria define limits for acceptable vibration velocity levels with a one-second averaging time at the floor of the building under consideration. For this, the x-axis representation requires to be based on one-third octave band frequency spectrum. For the band levels exceeding a particular criterion curve, mitigation options should be introduced considering the specific range of frequency where the mitigation is expected to be the most efficient. Interpretations of each criterion are defined in Table 1. It must be emphasized that both criteria are on a frequency spectrum since vibration-related problems are dependent on the resonant conditions of the structure and the sensitive equipment. With that, a DVA run is expected to provide an assessment on the criterion limits. The DVA criterion is based on generic cases when people are standing or equipment is mounted on the building floor (Hanson et al. 2018). Therefore, the criterion is less stringent at relatively lower frequencies (less than 8 Hz) as observed in Figure 1. It should be noted that the first bending mode of elevated building floors are almost always less than 8 Hz. For the special case of vibration isolation, the Figure 1 curves may be considered as flat also at lower frequencies.

For the detailed vibration analysis of this paper, the VC-A category curve is appropriate for the vibration impact criterion of the six story office building. The VC-A curve is defined applicable for healthy functioning of optical balances, microbalances, medium-to high-power optical microscopes (400X) and similar specialized equipment. Therefore, the DVA results of this study are to be compared to this curve.

3. Assessment of the office building

The building in assessment is the office building right next to the parking garage. The proposed alignment of the new railway tunnels passing adjacent to the six story office building and right underneath the six story parking garage is shown in Figure 2. The train line is an additional line to an existing subway system of a relatively large city located in North America. The closest edge of the building is about 25 ft (7.6 m) away from the East Bound (EB) tunnel horizontally and the farthest horizontal distance from the building to the tunnel is about 90 ft (27.4 m), as shown in Figure 3.

The office building has a pile foundation system where the piles are bearing on bedrock at 70 ft (21.3 m) below the ground level. It has been reported in the literature that the piles tend to attenuate the motion at the base of the superstructure as shown in seismic problems (Anoyatis et al. 2013; Di Laora and de Sanctis 2013). The tunnel is located within the bedrock at about 97 ft (29.6 m) below the ground. Figure 4 shows the geologic profile in the vicinity of the building in assessment.

The main goal of the project is to assess the vibrations that are be transmitted from the rail tracks and the tunnel walls, through the bedrock and soil layers, then to the piles and the ground level. The approach adapted for the assessment is to build a FE model to assess the vibrations generated from the trains in the twin tunnels. The primary assumption in the modeling is to use a plane strain approach and treat the train loading as a harmonic line source.

Many problems in elasticity have been treated satisfactorily by the two-dimensional plane theory. In plane strain approach, the out of plane strain is assumed to be zero and the plane strain model is applicable for the cases where the strain state of a point has non-zero components lying in one plane only. As will be discussed in the next section, the plane strain approach is efficient since the simulations are run in a single plane and the plane strain refers to the type of element used to discretize the soil domain.

4. Finite element model

A multi-purpose FE software, ANSYS (ANSYS 2009) is used for the study. Train vibrations and wave propagation have been effectively analyzed and studied with FE models in similar studies (Guo et al. 2018; Wang et al. 2005). For the vibration nuisance assessment presented in this paper, the major features of the FE model are shown in Figure 5. The ground surface vibrations at the foundation level of the building are analyzed in the FE model (the elevated floors of the building are not modeled). The soil fill extends to a depth of 70 ft (21.3 m) and the tunnels are located in the rock layer where the top of tunnel is located at 97 ft (29.6 m) from the ground surface. A plane strain FE model is built to represent the soil and the rock layers. The foundation slabs of the office building are not included in the FE model. The piles however, are included in the model as discrete beam elements extending from the ground surface down to the bed rock. For the 2D model, it is assumed that the next row of piles is far away to affect the results at this 2D section. The foundation slab and the piles of the parking garage are not included in the model.

In the FE model, the boundaries are placed far away from the excitation source to avoid corner/ 3D effects (Figure 5). Also, absorbing boundary conditions based on Lysmer dampers (Lysmer and Kuhlemeyer 1969) are placed on the model boundaries to avoid reflections. These boundaries have been used by various researchers in FE models for eliminating the waves reflecting from the boundaries (Bao et al. 2012; Mott and Wang 2011; Nielsen 2006; Shen and Giurgiutiu 2015; Volpini et al. 2019; Zhang and Tang 2007).

Rayleigh damping model is used in the ANSYS model for the analysis, in which the damping ratio is frequency dependent (Figure 6). Since the frequency range of 2 Hz - 30 Hz was observed to be more critical than higher frequencies in this study, to be conservative in the FE model analysis, smaller damping values are chosen for this critical range of frequency. The smaller damping ratios would result in higher vibration levels, which is basically being on the safe side for an assessment study like this. It can also be argued that reducing the damping ratio compensates for the linearity in the surrounding soil. As such, based on the frequency dependent characteristics of the Rayleigh damping model, the damping ratio varied from 3% to 5% for the 2 Hz - 30 Hz range. For higher frequencies, the damping is higher than 5% based on the curve trend shown in Figure 6. On a related note, linear material properties are used for the analysis. For the soil, the elastic properties are based on shear wave velocity of about 400 ft/s (122 m/s). For the rock, a shear wave velocity of about 13000 ft/s (3963 m/s) is used (Table 2).

The train loading is introduced as a harmonic line source in the FE model. Properties of Flexity Swift M5000 LRV train car are used for the analysis; elevations and plan view of which are shown in Figure 7. The empty train car weight is about 80,000 lbs (36,288 kg) and the car length is about 95 ft (29 m). Assuming a total of 100 people with an average weight of about 150 lb (68 kg), the total load is about 95000 lbs (43,091 kg). The load is spread over the train length of 95 ft (29 m), i.e., a uniform load of about 1kip/ft (14.6 kN/m). The load is distributed on the two rail tracks and each track has a load of about 0.5 kip/ft (7.3 kN/m). Based on this, the source of vibration is a line harmonic source with frequency varying from 0 to 200 Hz and amplitude of 0.5 kip/ft (7.3 kN/m) for each rail track. One train loading condition assumes that there is a train only in the East Bound (EB) tunnel and two train loading condition assumes trains in both EB and WB tunnels at the same time.

As mentioned earlier, a plane strain FE model is used for the vibration assessment work in this paper. The plane strain model is applicable for the cases where the strain state of a point has non-zero components lying in one plane only (Boresi et al. 1993). This approach of using a plane strain model with harmonic line load to assess ground vibration due to underground tunnels is consistent with the work by Yang et al. (Yang et al. 2007). The ANSYS simulations run in a single plane and the plane strain refers to the type of element used to discretize the soil domain. As such, based on the standard 2-D plane strain assumption, the strains in the direction normal to the plane of Figure 5 are considered to be zero. In two-dimensional elasticity, the governing partial differential equations for the plane stress model incorporating body forces and inertia forces can be written as (Przemieniecki, J. 1968; Sokolnikoff 1956):

$$G\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\frac{G(1+\nu)}{(1-\nu)}\frac{\partial}{\partial x}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+X=\rho\frac{\partial^2 u}{\partial t^2} \tag{1}$$

$$G\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{G(1+\nu)}{(1-\nu)}\frac{\partial}{\partial y}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+Y=\rho\frac{\partial^2 v}{\partial t^2} \tag{2}$$

$$G=\frac{E}{2(1+\nu)} \tag{3}$$

Where,

ρ = density of the material

X, Y = body forces in x and y directions respectively, per unit volume

E = modulus of elasticity

ν = Poisson's ratio

G = shear modulus

Similarly, the governing partial differential equations for the plane strain model incorporating body forces and inertia forces can be simplified as:

$$G\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\frac{G}{(1-2\nu)}\frac{\partial}{\partial x}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+X=\rho\frac{\partial^2 u}{\partial t^2} \tag{4}$$

$$G\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{G}{(1-2\nu)}\frac{\partial}{\partial y}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+Y=\rho\frac{\partial^2 v}{\partial t^2} \tag{5}$$

4.1 Generalized Loading

From pure analytical approach, the loading function created by a train can be presented as a moving harmonic load with frequency, f_0. As shown by Yang et al. (Yang and Hung 2008), for the train travelling with speed c on the z-axis with the coordinate origin at the train railway contact point, the loading function is:

$$f(x, y, z, t) = \psi(x, y)\phi(z - ct)e^{-i2\pi f_0 t} \tag{6}$$

On the x-y plane, the influence function of moving loads is written as:

$$\psi(x, y) = \delta(x)\delta(y) \tag{7}$$

Where;

$\phi(z)$= load distribution function for the train travelling on the z-axis.

$\delta(x)$= influence function component of the moving load in x-direction

$\delta(y)$= influence function component of the moving load in y-direction

$\psi(x, y)$= the influence function of the moving loads on the tunnel cross sectional plane (x-y plane)

f_0= moving load excitation frequency

c= train speed

The external loading in time domain $f(x, y, z, t)$ is represented as the component summation of a series of harmonic loads (Yang and Hung 2008):

$$f(x, y, z, t) = \int_{-\infty}^{\infty} \frac{1}{c}\delta(x)\delta(y)\tilde{\phi}(-k)e^{-ikz}e^{-i\omega t}d\omega \tag{8}$$

Where;

$\tilde{\phi}(k)$= Fourier transform of the function $\phi(z)$

$k = (\omega - 2\pi f_0)/c$

The steady-state response in the time domain for a linear system is determined by the superposition of responses created by the series of harmonic load components. For the location (x', y') assuming the response created by a harmonic load, $\delta(x)\delta(y)e^{-ikz}e^{-i\omega t}$ is represented as $H(i\omega)$. With that, again in the time domain, the final response is presented as (Yang et al. 2003; Yang and Hung 2008):

$$d(x', y', z, t) = \int_{-\infty}^{\infty} \frac{1}{c}\tilde{\phi}(-k)H(i\omega)e^{-ikz}e^{-i\omega t}d\omega \tag{9}$$

Meanwhile, in the frequency domain, the equation of motion is:

$$\left([K] - \omega^2[M] + i\omega[C]\right)\{D\} = \{F\} \tag{10}$$

Where $[K]$ is the global stiffness matrix; $[M]$ is the global mass matrix; $\{D\}$ is the vector of nodal displacements; and $\{F\}$ is the vector of applied loads. The displacements $\{D\}$ in the frequency domain become:

$$\{D\} = ([K] - \omega^2[M] + i\omega[C])^{-1}\{F\} \quad (11)$$

When $\{D\}$ of Eq.(12) is inserted for representing the function $H(i\omega)$, a time domain response can be achieved (Yang and Hung 2008).

However, the various unknowns regarding the design of the train tunnels and the surrounding media are only a few of many set of factors making the use of Equations (9-10) unreasonable at instances. The tunnels mentioned in this paper are still in the design stage which means the material and geometrical properties are always subject to change. In addition, due to the complex dynamic interaction among the subgrades, train car, bogie, wheelset, rail-pads, rails, ballast, sleepers and unevenness in the soil layers, Equations (9-10) acquire a further degree of uncertainty, hence cannot be used with full confidence. On top of all this, the modal properties like stiffness and inherent damping are also unknown with potential nonlinearity (Avci 2016, 2017).

4.2 Adapted Loading Function

The uncertainties in the complex response function $H(i\omega)$; the oscillation of train car suspension system; unevenness of the rails; the complicated interaction between the rails and wheels; geometric and material irregularities of the soil layers and radiation damping phenomenon are other important factors resulting in a need to create a FE model to determine the vibrations at the surface. A detailed FE model would result in vibration levels to be compared to the FTA criterion. For the FE analysis, a model is developed in ANSYS. The train load is simplified to a sinusoidal excitation:

$Psin(\omega t)$ = harmonic force function generated by the train pass

P = harmonic force amplitude

ω = frequency range 0-200 Hz

Vibration velocity level in decibels is formulated as:

$$L_{vel} = 20\,log\left(\frac{vel}{vel_{ref}}\right) \quad (12)$$

where:

L_{vel} = velocity level, VdB

vel = RMS velocity amplitude

vel_{ref} = 10^{-6} in/sec in the USA

= 10^{-8} m/sec internationally

Since the reference quantities vary in the literature (Hanson et al. 2018), it is crucial to report the reference quantity when specifying velocity levels. In this paper, all the vibration levels are referenced to $1x10^{-6}$ inches/second. A sinusoidal excitation is applied at the tunnel location of the 2D model, and the response at the bottom of the building at 50 ft (15.2 m) horizontal distance from the train excitation location is calculated. The ANSYS run is based on the sinusoidal excitations being applied with the harmonic sweep (sine sweep) function which cycles through a range of frequencies in a single run. The results are post-processed at specific frequency values. These specific frequencies at which results are extracted are based on the FTA manual criteria (One-Third Octave Band Frequency). The peak responses at the critical frequencies are automatically recorded and printed out at each of the corresponding frequencies. The results of the harmonic sweep analysis are discussed in the following section.

5. Analysis results and sensitivity analysis

The surface vibrations at a horizontal distance of about 50 ft (15.2 m) from the tunnel are shown in Figure 8. The Federal Transit Authority (FTA) VC-A criterion is also plotted in the figure. The vibrations from the railway tunnel peak in the frequency ranges of 8 Hz to 12 Hz and 20 Hz to 30 Hz. In these ranges the peak RMS VdB for the one train loading condition is about 60 VdB and for the two train loading condition is about 64 VdB. These peaks are lower than the VC-A criteria therefore no adverse impact is expected on the building.

When there is uncertainty in the output of a task, the input items (independent variables) of the same task can be sub-grouped and/or assigned to various sources of uncertainty within reasonable boundaries. This is the simplistic description of the sensitivity analysis which is also used in the context of this paper. The harmonic loading of the one-train condition, damping ratios and the horizontal distance from the harmonic load to the tunnel are the sensitivity analysis items focused on within this study. While increasing the loading, decreasing the damping ratio and decreasing the distance to harmonic loads are all independent inputs, they would all result in an increased vibration response at the building foundations. This makes the analyses conservative which serves the purpose of the sensitivity analyses.

In order to assess the sensitivity of the results three additional sets of analyses are conducted with the following variations from the above baseline model:

1. One train loading condition with 20% load amplitude increase.
2. Lower the damping ratio from 3% - 5% range (see Figure 6) to 1% - 2% range.

3. Decrease the horizontal distance from the loading to the building from 50 ft (15.2 m) to 25 ft (7.6 m).

The analysis result for one train condition with 20% load amplitude increase is shown in Figure 9. For 20% increase in the load the VdB values are found to increase by about 1.6 VdB. Therefore, for the most critical frequency of one train condition, the peak increases from 60 VdB to about 61.6 VdB (an increase of 2.7%), still remaining below the FTA VC-A limit (66 VdB).

Considering the frequency dependency of the damping ratios in Rayleigh damping model (Figure 6), the second run to check for the sensitivity analysis was conducted by using lower damping ratios than the original range of 3% to 5% in the FE model runs. The damping ratios were lowered to 1% to 2% in the critical frequency range of 2 Hz - 30 Hz considering the fact that lower damping ratios would result in higher vibration peaks. In other words, lowering the damping ratios makes the analysis more conservative, hence complying with the sensitivity analysis. With that, the analysis result for one train condition with lower damping is shown in Figure 10. For lower damping, VdB values are found to increase by about 1.6 VdB. Therefore for one train condition, the peak increases from 60 VdB to about 61.6 VdB (an increase of 2.7%), staying below the VC-A criteria limit of FTA (66 VdB).

For the third sensitivity analysis, the horizontal distance from the load to the building is decreased from 50 ft (15.2 m) to 25 ft (7.6 m). The analysis results are shown in Figure 11. The results show that for relatively low frequencies (less than 50 Hz) the ground surface vibrations are not sensitive to the tunnel location, at least for the two locations compared against each other. For regions closer to the tunnel, the higher frequency (greater than 50 Hz) vibrations are not attenuated, and consequently at higher frequencies the vibration levels are higher as shown in the figure. For 25 ft (7.6 m) distance, at the most critical frequency value of 25 Hz, the vibrations are found to increase by about 1.0 VdB. The peak increases from 60 VdB to about 61 VdB (an increase of 1.7%), staying below the VC-A criteria limit of FTA (66 VdB).

6. Other components of nuisance

6.1 Vibration nuisance in the existing building

Floor vibrations are unwanted in a building since the oscillations irritate the occupants and cause discomfort. In the last couple of decades, slender structural members and modern construction technologies have resulted in more vulnerable floor systems against vibrations (Avci 2015; Bhargava et al. 2013; Muhammad et al. 2018; Younis et al. 2017). As such, the vibration serviceability of the floors has become a standard design parameter that needs to be checked by the designers to ensure human comfort on building floors (Avci 2005). The vibrations on building floors can be caused not only by human activities such as walking, running, dancing, and aerobics; but also by the train traffic nearby the building (Alabbasi et al. 2019).

There are several methods for the assessment of floor vibrations serviceability, ranging from probabilistic methods to simplified methods (Abdeljaber et al. 2018, 2019). There are four well established floor vibration evaluation methods extensively used over the world. These are the American Institute of Steel Construction (AISC) Design Guide 11 Chapter 4 method (Murray et al. 2016); Steel Construction Institute (SCI) P354 Simplified Method (Smith et al. 2009); Steel Construction Institute (SCI) P354 Vibration Dose Values Method (Smith et al. 2009); and Human Induced Vibration of Steel Structures (HIVOSS) Method (RFCS 2009). While all of these methods are very useful for human-induced excitations, none of them are covering the vibrations serviceability and human comfort levels due to train-induced excitations.

Transmission of train vibrations from the ground into the building and propagation within the building towards the upper floors have been researched by various researchers. It has been reported that developing a methodology to predict vibration levels at the upper floors based on the vibrations at the foundation level is not very straightforward (Sanayei et al. 2014). For human response to vibrations, the FTA Assessment Manual (Hanson et al. 2018) recommends an attenuation of -2 dB per floor for the first five stories of the building; and a -1 dB per floor for the next five stories of the building (floor five to floor ten). However, there is also an amplification component due to the resonances of walls, floors and ceilings which may potentially increase the vibration levels +6 db for light-weight and timber-frame structures. Since the existing building of this study is a steel-framed building, this amplification is not applicable (summarized in Table 3).

Moreover, for the detailed vibration analysis of this paper, the VC-A category curve of the FTA Assessment Manual is used for the vibration impact criterion of the six story office building which is applicable for healthy functioning of optical balances, microbalances, medium-to high-power optical microscopes (400X) and similar specialized equipment. This criterion is compared with the ground surface vibrations (at the foundation level of the building) with the results of detailed FE model investigation and the accompanying sensitivity analysis. As discussed earlier, the peak RMS VdB for the one train loading condition is about 60 VdB and for the two train loading condition is about 64 VdB. Similarly, as the one train loading amplitude is increased 20%, the RMS peak was found to be 61.6 VdB;

and with lower damping ratio, the RMS peak was found to be 61.6 VdB. Even with the decreased distance to the train line, the RMS peak was at 61 VdB. While all these peak values are below the 66 VdB limit of the VC-A criteria, they are all calculated at the foundation level of the existing building which is fairly conservative. This conservatism is due to the fact that the vibrations will attenuate as the waves are propagating upwards within the building. The FTS suggested attenuation of -2 dB per floor for the first five stories of the building; and a -1 dB per floor for the next five stories of the building would further decrease the already acceptable RMS VdB peaks. As a result of this, no adverse impact is expected for occupants located on the elevated floors of the existing building.

On another note, there are simplified impedance-based analytical models available in the literature for train-induced vibration predictions (Sanayei et al. 2011). In these models, the vibrations propagate through the axial waves in the columns of the building. The impedance of columns and slabs representing the stiffness, mass and damping properties are the predominant components of the model where each finite portion of the column is modeled according to the impedances at the top and bottom of each column. With that, the displacements and forces at the ends of the columns are represented in the dynamic stiffness matrices.

6.2 Noise nuisance

Like vibration, noise may also cause annoyance to building occupants. Not only trains but also other transit sources create noise in various levels based on the type of the transit and operating conditions. Between the source and the receiver, the level of noise can be attenuated along the path depending on the obstacles in the way, ground type, damping components and several other factors. Generally, the noise metrics are expressed in terms of “A-Weighted Decibels” (dBA) which is the basic noise unit in transit noise terminology. Also known as “A-Weighted Sound Level”, this term represents the overall noise at a receiver location that is adjusted in frequency to approximate typical human hearing sensitivity. The letter "A" indicates that the sound has been filtered to reduce the strength of very low and very high-frequency content to represent the human response to sound levels (Hanson et al. 2018).

In transit noise impact assessment, another important parameter is the “Sound Exposure Level” (SEL). This term is defined as the cumulative noise exposure from a single noise event, normalized to one second. The louder events have a larger SEL value than the quitter events. Similarly, the shorter events have a smaller SEL value than the longer events. Basically, the SEL value represents the identical overall sound energy as the actual varying sound energy during the single noise event. Being a primary metric for the measurement of transit vehicle noise emissions, it is also an "A-Weighted" cumulative measure.

In transit noise terminology, another common term is the "Maximum Sound Level" (L_{max}) which is also known as the "Maximum Noise Level". This measure represents the maximum level of sound generated during a single noise event, and it is considered as "A-Weighted Maximum Noise Level" in various references. This metric is predominantly used in vehicle noise specifications and typically calculated for individual vehicles.

The equation for computing L_{max} for a single locomotive pass-by per FTA Assessment Manual (Hanson et al. 2018) is:

$$L_{max} = SEL_{ref} - 10log\left(\frac{L_{meas}}{50}\right) - 10log\left(\frac{D_{meas}}{50}\right) + 10log(2\alpha) - C_{consist} - C_{emissions} - 3.3 \quad (13)$$

Where;

SEL_{ref}= Source Reference Level (Reference SEL)

L_{meas} = total length of measured group of locomotives or rail cars (ft)

D_{meas} = closest distance between measurement position and source (ft)

$\alpha = \arctan\left(\frac{L_{meas}}{2D_{meas}}\right)$ in radians

$C_{consist} = -10\log(N_{cars})$ for locomotives and rail cars where N_{cars} is the number of locomotives in the measured group

$C_{emissions} = 0$ for T<6 for locomotives

$C_{emissions} = -2(T-5)$ for T≥6 for locomotives where T is average throttle setting of the locomotives.

While the sound nuisance of the train excitations was not intended to be discussed in this manuscript, the train noise calculations are briefly shown here for the sake of completeness. The train-induced noise calculations were also not run nor discussed as the presented FE model was used only to investigate the vibrations resulting from the train movements, at the foundation level of the existing building.

7. Conclusions

In this paper, ground surface vibrations are investigated due to trains passing through proposed underground tunnels in the vicinity of an office building. A 2D finite element model is created reflecting the railroad tunnels embedded in the rock and the soil stratum above it. Also, a train loading function is applied to the model as a point source at the track level. Various simulations are undertaken followed by a sensitivity analysis. Taking the plane strain assumption within the finite element model simulations, the vibrations from the railway tunnel were determined to have peaks in the frequency ranges of 8 Hz to 12 Hz and 20 Hz to 30 Hz. In these ranges the peak RMS VdB for one train loading condition is about 60 VdB and for two train loading condition the peak RMS is about 64 VdB. Meanwhile, for the sensitivity analyses, when the one train loading amplitude is increased 20%, the RMS peak was found to be 61.6 VdB. On another run, when lower damping values are used, the RMS peak was found to be 61.6 VdB. For the last sensitivity analysis run, the distance from the building to the loading is decreased to 25 ft (7.6 m) which resulted in an RMS peak of 61 VdB. The results are compared to the criteria of Transit Noise and Vibration Impact Assessment Manual, which was published by Federal Transit Administration (FTA) of the U.S. Department of Transportation. Since all of the peaks are observed to be lower than the FTA VC-A criteria (66 VdB); no adverse impact is expected on the building.

The predominant limitations of the study presented in this paper could be listed as the use of 2D analysis and material linearity. While a 2D model is relatively easier to build and processed faster, a 3D model would have allowed a more realistic modeling of the harmonic force function and would have produced more information on wave propagation. Yet, a 3D model would also have the same material and geometrical unknowns as a 2D model for the tunnels, surrounding media, and complex dynamic interaction among the train car and wheel components. In addition, while the material and dynamic properties of the structural members and the soil layers are known to be non-linear in reality, the model used in the study assumes a linear behavior.

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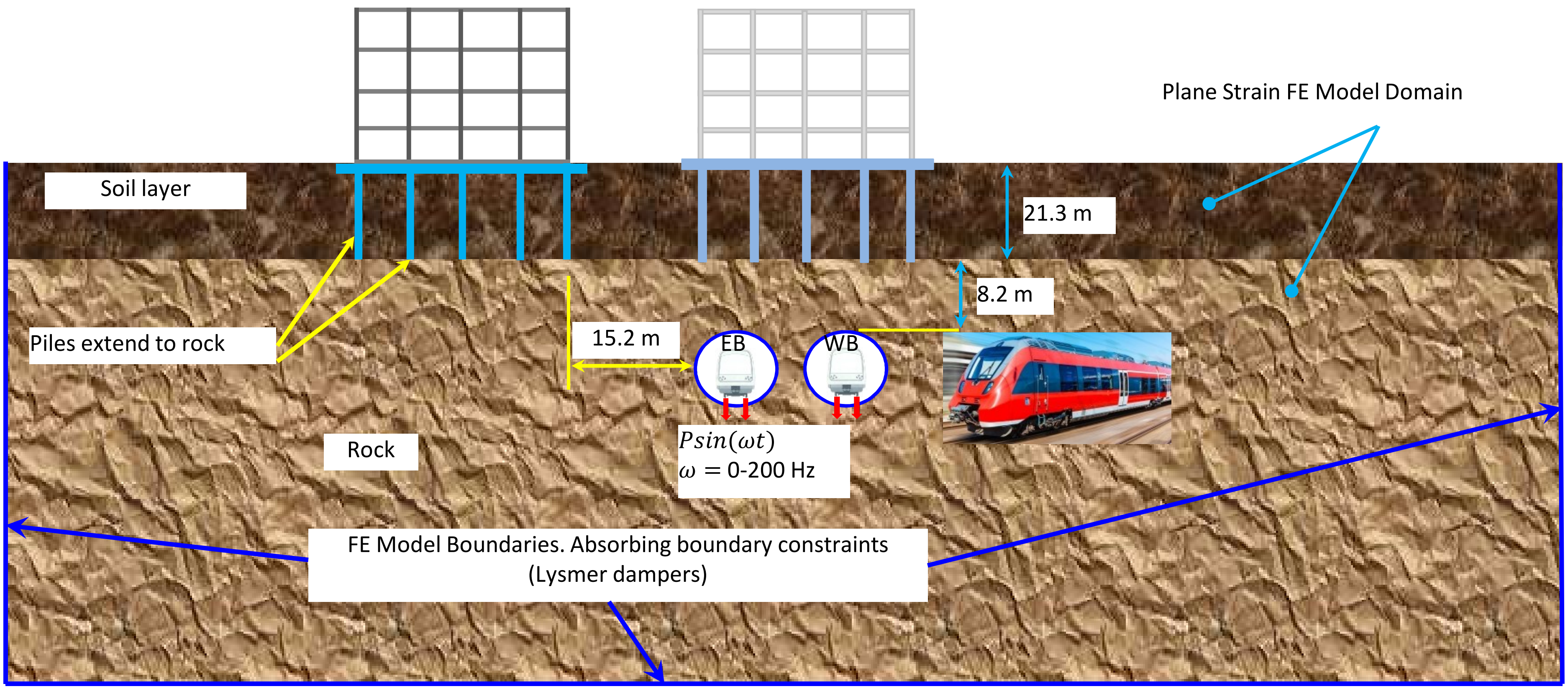
*Graphical Abstract



***Highlights (for review : 3 to 5 bullet points (maximum 85 characters including spaces per bullet point)**

Vibration Annoyance Assessment of Train Induced Excitations from Tunnels Embedded in Rock

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Highlights

- This paper presents a study conducted to assess vibrations of trains to nearby buildings through a case study.
- A plane strain finite element model is built for the railroad tunnel and the surrounding media.
- Various simulations are undertaken for sensitivity analysis.
- The results are compared to the *Transit Noise and Vibration Impact Assessment Manual*.
- All peaks are found to be lower than the criteria, no adverse impact is expected.

*Manuscript (double-spaced and continuously LINE and PAGE numbered)-for final publication
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Vibration Annoyance Assessment of Train Induced Excitations from Tunnels Embedded in Rock

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Abstract

Train movements generate oscillations that are transmitted as waves through the track support system into its surroundings. The vibration waves propagate through the soil layers and reach to nearby buildings creating distractions for human activities and causing equipment malfunctioning. Not only the train components and the rails, but also the surrounding tunnel, soil and rock strata have dynamic characteristics that play significant roles in the vibration levels felt in a nearby structure. This paper presents a finite element study conducted to investigate the vibrations resulting from train movements in nearby subway tunnels. The subway line is located at an average horizontal distance of 50 ft (15.2 m) from the structure in assessment, which is a six-story office building. The main goal of the work is to assess the train-induced vibrations at the ground level of the building through a case study and sensitivity analysis. A plane strain finite element model is built to represent the railroad tunnel embedded in the rock and the soil stratum above it. The one train loading function is applied to the model as a point source at the track level and compared to the two-train scenario. Other simulations are undertaken for sensitivity analysis involving increased loading, decreased damping and decreased distance to tunnels. Even though there are several numerical studies on the propagation of train induced vibrations in the literature; a finite element model accompanied with a sensitivity analysis has not been discussed in detail in a technical publication before. The paper not only presents the finite element modeling but also compares the results with the criteria of *Transit Noise and Vibration Impact Assessment Manual*, which was published by the Federal Transit Administration (FTA) of the U.S. Department of Transportation.

Keywords: Vibrations disturbance; train vibrations; ground-borne vibrations; railroad tunnels; tunnels embedded in rock; ANSYS; finite element modelling

1. Introduction

Occupants in buildings are sensitive to vibrations. While most perceptible indoor vibrations are originated from sources within the buildings (human movement, mechanical units, etc.), there are also outdoor sources of vibrations (trains, construction equipment, vehicle traffic, etc.) propagating through the ground, penetrating the building and creating vibratory distractions for building occupants (Sanayei et al. 2014; Xia et al. 2010). The vibrations generated due to vehicles on tires are generally observed to be less than the vibrations caused by train excitations. Among the outdoor sources of vibrations, even though construction blasting and pile-driving (Grizi et al. 2016) have been reported to result in damage on buildings, the train vibrations seldom result in building damage. However, there are cases where excessive train vibrations cause extensive distractions to building occupants such as decreasing work focus in office environments during day time and negatively affecting sleep quality during night time. In addition, such vibrations might result in malfunctioning of vibration sensitive equipment. Meanwhile, the train vibrations can be felt by people standing outdoors, however it is seldom that people that are outside complain about outdoor vibrations.

Train induced noise and vibration have been an area of research over the decades (Degrande and Schillemans 2001; Kephalopoulos et al. 2014; Yang et al. 2019; Zou et al. 2017). Regardless of the train being at the ground level or in a tunnel, ground-borne vibration is a predominant concern for the building occupants especially in the close vicinity of transit system routes (Kouroussis et al. 2016; Licitra et al. 2016; Ngamkhanong and Kaewunruen 2018; Peris et al. 2014, 2016). Vibration levels generated by trains are dependent on several factors (Connolly et al. 2016; Lee et al. 2014; Zou et al. 2015). While the material and dynamic properties of the train components, tunnels and surrounding geological conditions are one set of factors (Cai et al. 2010; Galvín and Domínguez 2007; Sheng et al. 2006); wheel smoothness, rail smoothness, vehicle suspension system, track support system are another set (Fiala et al. 2007; Metrikine and Vrouwenvelder 2000; Sheng et al. 2004; Triepaischajonsak et al. 2011). For instance, dynamic characteristics of the soil stratum and the depth of the bedrock are important parameters affecting wave propagation (Avillez et al. 2013; Yang and Hsu 2006). While the wave propagation is more effective in stiff clays, when the depth to bedrock is 30ft (9.1 m) or less, it is known that vibration energy is more pronounced near at-grade track. This would most probably result in vibration problems even at large distances from the railway tracks (Bian et al. 2015; Hanson et al. 2018). Also, for the tunnels embedded in rock the train vibrations do not attenuate as rapidly as in soils. It is also observed that the buildings with heavier foundation and superstructure tend to experience relatively smaller vibrations than the lighter structures. The maximum response of the structure will mainly depend on the level of vibrations reaching the building foundations, modal characteristics of the building, and the coupling of the building foundation to the soil (soil-structure interaction).

As the steel wheels of the trains roll on the rails, vibratory forces are generated. As the wheel meets a discontinuity like a joint, a reactive force is exerted on the wheel and hence on the train. When the vibration of the transit structure generates oscillatory waves propagating away within the soil layers, vibrational energy moves through the surrounding media in a variety of wave forms (shear, compression and Rayleigh waves). The Rayleigh waves carry considerable amount of vibrational energy. In the presence of multi-layered soils, the mathematical modeling of vibration is even more complicated; therefore, FE modeling appears as a reasonable way to model such conditions in a relatively easier and practical way (Andersen and Jones 2006). Yet, vibration propagation from the railways to the structure foundations is complex and requires extensive FE modeling. On a different note, the interaction between the soil and the foundation is significantly important for vibration assessment with FE models (Galvín et al. 2010; Galvín and Domínguez 2009; Hall 2003; Ju and Lin 2004; Yaseri et al. 2014). Once the oscillations propagate through the structure, based on the intensity levels, the vibration can be felt by occupants and/or cause malfunctioning on sensitive equipment. The propagation of oscillations from the foundation to the upper levels of the structure is also complex and has been researched in detail. The propagation within the building depends on the type of the building and the structural design.

Since railway induced vibration propagation involves many parameters, it is difficult to estimate the resulting vibrations at a specific location (Park et al. 2008a; b). Therefore, the majority of assessment techniques utilized for transit projects are based on empirical data. For more project specific estimations, where vibration impact is probable, detailed FE models are utilized by researchers and engineers for estimating the vibration levels. While the DVA method typically assumes a steel-wheel/rail system, it is based on frequency domain information typically in terms of one-third octave-band spectrum.

2. Vibration analysis levels per the Transit Noise and Vibration Impact Assessment Manual

The United States Department of Transportation's Federal Transit Administration (FTA) released an updated version of the Transit Noise and Vibration Impact Assessment Manual (Hanson et al. 2018), which outlines means and methods for prediction and assessment of vibration (and noise) impacts of transit projects as part of FTA's environmental review process. The manual includes clarifications to existing policy and updates to outdated references, where applicable, but does not change the existing assessment procedures of previous versions of the document.

In the FTA Transit Noise and Vibration Impact Assessment Manual (Hanson et al. 2018), there are three levels of procedure presented to assess the potential ground-borne vibration impacts resulting from a train line. The appropriate level of analysis is a function of the project specifics such as environmental settings, scale and type of the transit project. According to the FTA manual, there are three levels of analysis:

- Vibration Screening Procedure (VSP)
- General Vibration Assessment (GVA)
- Detailed Vibration Analysis (DVA).

In order to determine which level of procedure to apply, the primary item to consider is the presence of any vibration sensitive land use through the VSP. This screening procedure determines the work area of any vibration assessment process. In other words, if there is not any vibration-sensitive equipment present within the defined screening distance, then there is no need for further vibration assessment. When the major project details are known relatively early in the project development stage, the GVA procedure is implemented. The GVA procedure is conducted if there is any potential for considerable vibration levels. For this, vibration levels at receiving locations are determined by predicting the overall vibration velocity level as a function of distance from the rail-tracks. This process also includes adjustments to consider factors like the building type, track and wheel conditions, vehicle speed and track support systems. The GVA is adequate for the environmental review of standard projects where transit modal alternatives are considered for a potential relocation or any other project modification. The GVA is considered adequate when there is a commitment to mitigate vibration impacts, such as a change in transit mode or alignment. However, if the impact is identified through the GVA procedures and not mitigated, a DVA must be conducted. The DVA is conducted to determine the severity and extent of the impact especially for sensitive buildings, i.e., in the close vicinity of train routes. If needed, vibration mitigation measures would also be implemented after a DVA procedure. On another note, a DVA may be warranted earlier in the environmental review process if there are impact indications regarding the closeness of vibration-sensitive structures. This type of assessment task requires experienced professionals to perform tests and post-process data.

On another note, a DVA may not be necessary for all segments of a project. Generalized prediction curves from the GVA procedures may be sufficient for most of the alignment, and the DVA procedure may only need to be applied to particularly sensitive receivers. Furthermore, a DVA is typically required for special type of track-support systems (e.g., ballast mats, floating slabs). Costly vibration mitigation techniques can only be called following a DVA in the design stage.

The type of work conducted in this paper is a typical example of a DVA procedure where the vibration impact of twin tunnels embedded in rock are researched in detail to make sure that the occupants of an existing nearby six story office building do not get affected by the vibrations generated by the trains passing by. The railway line is an additional line to an existing subway system of a relatively large city located in North America. The proposed alignment of the railway tunnels is passing adjacent to a six story office building and right underneath a six story parking garage. A detailed finite element (FE) model is built and train excitations are simulated to observe the vibration responses at the base of the existing building.

The criteria for the DVA and the interpretation of vibration criteria are summarized in Figure 1 and Table 1 which are both adapted from the FTA Assessment Manual (Hanson et al. 2018). On Figure 1, the x-axis is one-third octave band center frequency while the y-axis is the vibration velocity level. The DVA criteria of Figure 1 and Table 1 are based on international standards for the effects of vibration on people related to annoyance and interference with activities in buildings (Hanson et al. 2005). The criteria also covers the extensively used standards for vibration-sensitive equipment (Nelson et al. 1982). Specifically, these criteria define limits for acceptable vibration velocity levels with a one-second averaging time at the floor of the building under consideration. For this, the x-axis representation requires to be based on one-third octave band frequency spectrum. For the band levels exceeding a particular criterion curve, mitigation options should be introduced considering the specific range of frequency where the mitigation is expected to be the most efficient. Interpretations of each criterion are defined in Table 1. It must be emphasized that both criteria are on a frequency spectrum since vibration-related problems are dependent on the resonant conditions of the structure and the sensitive equipment. With that, a DVA run is expected to provide an assessment on the criterion limits. The DVA criterion is based on generic cases when people are standing or equipment is mounted on the building floor (Hanson et al. 2018). Therefore, the criterion is less stringent at relatively lower frequencies (less than 8 Hz) as observed in Figure 1. It should be noted that the first bending mode of elevated building floors are almost always less than 8 Hz. For the special case of vibration isolation, the Figure 1 curves may be considered as flat also at lower frequencies.

For the detailed vibration analysis of this paper, the VC-A category curve is appropriate for the vibration impact criterion of the six story office building. The VC-A curve is defined applicable for healthy functioning of optical balances, microbalances, medium-to high-power optical microscopes (400X) and similar specialized equipment. Therefore, the DVA results of this study are to be compared to this curve.

3. Assessment of the office building

The building in assessment is the office building right next to the parking garage. The proposed alignment of the new railway tunnels passing adjacent to the six story office building and right underneath the six story parking garage is shown in Figure 2. The train line is an additional line to an existing subway system of a relatively large city located in North America. The closest edge of the building is about 25 ft (7.6 m) away from the East Bound (EB) tunnel horizontally and the farthest horizontal distance from the building to the tunnel is about 90 ft (27.4 m), as shown in Figure 3.

The office building has a pile foundation system where the piles are bearing on bedrock at 70 ft (21.3 m) below the ground level. It has been reported in the literature that the piles tend to attenuate the motion at the base of the superstructure as shown in seismic problems (Anoyatis et al. 2013; Di Laora and de Sanctis 2013). The tunnel is located within the bedrock at about 97 ft (29.6 m) below the ground. Figure 4 shows the geologic profile in the vicinity of the building in assessment.

The main goal of the project is to assess the vibrations that are be transmitted from the rail tracks and the tunnel walls, through the bedrock and soil layers, then to the piles and the ground level. The approach adapted for the assessment is to build a FE model to assess the vibrations generated from the trains in the twin tunnels. The primary assumption in the modeling is to use a plane strain approach and treat the train loading as a harmonic line source.

Many problems in elasticity have been treated satisfactorily by the two-dimensional plane theory. In plane strain approach, the out of plane strain is assumed to be zero and the plane strain model is applicable for the cases where the strain state of a point has non-zero components lying in one plane only. As will be discussed in the next section, the plane strain approach is efficient since the simulations are run in a single plane and the plane strain refers to the type of element used to discretize the soil domain.

4. Finite element model

A multi-purpose FE software, ANSYS (ANSYS 2009) is used for the study. Train vibrations and wave propagation have been effectively analyzed and studied with FE models in similar studies (Guo et al. 2018; Wang et al. 2005). For the vibration nuisance assessment presented in this paper, the major features of the FE model are shown in Figure 5. The ground surface vibrations at the foundation level of the building are analyzed in the FE model (the elevated floors of the building are not modeled). The soil fill extends to a depth of 70 ft (21.3 m) and the tunnels are located in the rock layer where the top of tunnel is located at 97 ft (29.6 m) from the ground surface. A plane strain FE model is built to represent the soil and the rock layers. The foundation slabs of the office building are not included in the FE model. The piles however, are included in the model as discrete beam elements extending from the ground surface down to the bed rock. For the 2D model, it is assumed that the next row of piles is far away to affect the results at this 2D section. The foundation slab and the piles of the parking garage are not included in the model.

In the FE model, the boundaries are placed far away from the excitation source to avoid corner/ 3D effects (Figure 5). Also, absorbing boundary conditions based on Lysmer dampers (Lysmer and Kuhlemeyer 1969) are placed on the model boundaries to avoid reflections. These boundaries have been used by various researchers in FE models for eliminating the waves reflecting from the boundaries (Bao et al. 2012; Mott and Wang 2011; Nielsen 2006; Shen and Giurgiutiu 2015; Volpini et al. 2019; Zhang and Tang 2007).

Rayleigh damping model is used in the ANSYS model for the analysis, in which the damping ratio is frequency dependent (Figure 6). Since the frequency range of 2 Hz - 30 Hz was observed to be more critical than higher frequencies in this study, to be conservative in the FE model analysis, smaller damping values are chosen for this critical range of frequency. The smaller damping ratios would result in higher vibration levels, which is basically being on the safe side for an assessment study like this. It can also be argued that reducing the damping ratio compensates for the linearity in the surrounding soil. As such, based on the frequency dependent characteristics of the Rayleigh damping model, the damping ratio varied from 3% to 5% for the 2 Hz - 30 Hz range. For higher frequencies, the damping is higher than 5% based on the curve trend shown in Figure 6. On a related note, linear material properties are used for the analysis. For the soil, the elastic properties are based on shear wave velocity of about 400 ft/s (122 m/s). For the rock, a shear wave velocity of about 13000 ft/s (3963 m/s) is used (Table 2).

The train loading is introduced as a harmonic line source in the FE model. Properties of Flexity Swift M5000 LRV train car are used for the analysis; elevations and plan view of which are shown in Figure 7. The empty train car weight is about 80,000 lbs (36,288 kg) and the car length is about 95 ft (29 m). Assuming a total of 100 people with an average weight of about 150 lb (68 kg), the total load is about 95000 lbs (43,091 kg). The load is spread over the train length of 95 ft (29 m), i.e., a uniform load of about 1kip/ft (14.6 kN/m). The load is distributed on the two rail tracks and each track has a load of about 0.5 kip/ft (7.3 kN/m). Based on this, the source of vibration is a line harmonic source with frequency varying from 0 to 200 Hz and amplitude of 0.5 kip/ft (7.3 kN/m) for each rail track. One train loading condition assumes that there is a train only in the East Bound (EB) tunnel and two train loading condition assumes trains in both EB and WB tunnels at the same time.

As mentioned earlier, a plane strain FE model is used for the vibration assessment work in this paper. The plane strain model is applicable for the cases where the strain state of a point has non-zero components lying in one plane only (Boresi et al. 1993). This approach of using a plane strain model with harmonic line load to assess ground vibration due to underground tunnels is consistent with the work by Yang et al. (Yang et al. 2007). The ANSYS simulations run in a single plane and the plane strain refers to the type of element used to discretize the soil domain. As such, based on the standard 2-D plane strain assumption, the strains in the direction normal to the plane of Figure 5 are considered to be zero. In two-dimensional elasticity, the governing partial differential equations for the plane stress model incorporating body forces and inertia forces can be written as (Przemieniecki, J. 1968; Sokolnikoff 1956):

$$G\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\frac{G(1+\nu)}{(1-\nu)}\frac{\partial}{\partial x}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+X=\rho\frac{\partial^2 u}{\partial t^2} \quad (1)$$

$$G\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{G(1+\nu)}{(1-\nu)}\frac{\partial}{\partial y}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+Y=\rho\frac{\partial^2 v}{\partial t^2} \quad (2)$$

$$G=\frac{E}{2(1+\nu)} \quad (3)$$

Where,

ρ = density of the material

X, Y = body forces in x and y directions respectively, per unit volume

E = modulus of elasticity

ν = Poisson's ratio

G = shear modulus

Similarly, the governing partial differential equations for the plane strain model incorporating body forces and inertia forces can be simplified as:

$$G\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\frac{G}{(1-2\nu)}\frac{\partial}{\partial x}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+X=\rho\frac{\partial^2 u}{\partial t^2} \quad (4)$$

$$G\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{G}{(1-2\nu)}\frac{\partial}{\partial y}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+Y=\rho\frac{\partial^2 v}{\partial t^2} \quad (5)$$

4.1 Generalized Loading

From pure analytical approach, the loading function created by a train can be presented as a moving harmonic load with frequency, f_0. As shown by Yang et al. (Yang and Hung 2008), for the train travelling with speed c on the z-axis with the coordinate origin at the train railway contact point, the loading function is:

$$f(x, y, z, t) = \psi(x, y)\phi(z - ct)e^{-i2\pi f_0 t} \quad (6)$$

On the x-y plane, the influence function of moving loads is written as:

$$\psi(x, y) = \delta(x)\delta(y) \quad (7)$$

Where;

$\phi(z)$= load distribution function for the train travelling on the z-axis.

$\delta(x)$= influence function component of the moving load in x-direction

$\delta(y)$= influence function component of the moving load in y-direction

$\psi(x, y)$= the influence function of the moving loads on the tunnel cross sectional plane (x-y plane)

f_0= moving load excitation frequency

c= train speed

The external loading in time domain $f(x, y, z, t)$ is represented as the component summation of a series of harmonic loads (Yang and Hung 2008):

$$f(x, y, z, t) = \int_{-\infty}^{\infty} \frac{1}{c}\delta(x)\delta(y)\tilde{\phi}(-k)e^{-ikz}e^{-i\omega t}d\omega \quad (8)$$

Where;

$\tilde{\phi}(k)$= Fourier transform of the function $\phi(z)$

$k = (\omega - 2\pi f_0)/c$

The steady-state response in the time domain for a linear system is determined by the superposition of responses created by the series of harmonic load components. For the location (x', y') assuming the response created by a harmonic load, $\delta(x)\delta(y)e^{-ikz}e^{-i\omega t}$ is represented as $H(i\omega)$. With that, again in the time domain, the final response is presented as (Yang et al. 2003; Yang and Hung 2008):

$$d(x', y', z, t) = \int_{-\infty}^{\infty} \frac{1}{c}\tilde{\phi}(-k)H(i\omega)e^{-ikz}e^{-i\omega t}d\omega \quad (9)$$

Meanwhile, in the frequency domain, the equation of motion is:

$$\big([K] - \omega^2[M] + i\omega[C]\big)\{D\} = \{F\} \quad (10)$$

Where $[K]$ is the global stiffness matrix; $[M]$ is the global mass matrix; $\{D\}$ is the vector of nodal displacements; and $\{F\}$ is the vector of applied loads. The displacements $\{D\}$ in the frequency domain become:

$$\{D\} = ([K] - \omega^2[M] + i\omega[C])^{-1}\{F\} \tag{11}$$

When $\{D\}$ of Eq.(12) is inserted for representing the function $H(i\omega)$, a time domain response can be achieved (Yang and Hung 2008).

However, the various unknowns regarding the design of the train tunnels and the surrounding media are only a few of many set of factors making the use of Equations (9-10) unreasonable at instances. The tunnels mentioned in this paper are still in the design stage which means the material and geometrical properties are always subject to change. In addition, due to the complex dynamic interaction among the subgrades, train car, bogie, wheelset, rail-pads, rails, ballast, sleepers and unevenness in the soil layers, Equations (9-10) acquire a further degree of uncertainty, hence cannot be used with full confidence. On top of all this, the modal properties like stiffness and inherent damping are also unknown with potential nonlinearity (Avci 2016, 2017).

4.2 Adapted Loading Function

The uncertainties in the complex response function $H(i\omega)$; the oscillation of train car suspension system; unevenness of the rails; the complicated interaction between the rails and wheels; geometric and material irregularities of the soil layers and radiation damping phenomenon are other important factors resulting in a need to create a FE model to determine the vibrations at the surface. A detailed FE model would result in vibration levels to be compared to the FTA criterion. For the FE analysis, a model is developed in ANSYS. The train load is simplified to a sinusoidal excitation:

$Psin(\omega t)$ = harmonic force function generated by the train pass

P = harmonic force amplitude

ω = frequency range 0-200 Hz

Vibration velocity level in decibels is formulated as:

$$L_{vel} = 20\,log\left(\frac{vel}{vel_{ref}}\right) \tag{12}$$

where:

L_{vel} = velocity level, VdB

vel = RMS velocity amplitude

vel_{ref} = 10^{-6} in/sec in the USA

= 10^{-8} m/sec internationally

Since the reference quantities vary in the literature (Hanson et al. 2018), it is crucial to report the reference quantity when specifying velocity levels. In this paper, all the vibration levels are referenced to $1x10^{-6}$ inches/second. A sinusoidal excitation is applied at the tunnel location of the 2D model, and the response at the bottom of the building at 50 ft (15.2 m) horizontal distance from the train excitation location is calculated. The ANSYS run is based on the sinusoidal excitations being applied with the harmonic sweep (sine sweep) function which cycles through a range of frequencies in a single run. The results are post-processed at specific frequency values. These specific frequencies at which results are extracted are based on the FTA manual criteria (One-Third Octave Band Frequency). The peak responses at the critical frequencies are automatically recorded and printed out at each of the corresponding frequencies. The results of the harmonic sweep analysis are discussed in the following section.

5. Analysis results and sensitivity analysis

The surface vibrations at a horizontal distance of about 50 ft (15.2 m) from the tunnel are shown in Figure 8. The Federal Transit Authority (FTA) VC-A criterion is also plotted in the figure. The vibrations from the railway tunnel peak in the frequency ranges of 8 Hz to 12 Hz and 20 Hz to 30 Hz. In these ranges the peak RMS VdB for the one train loading condition is about 60 VdB and for the two train loading condition is about 64 VdB. These peaks are lower than the VC-A criteria therefore no adverse impact is expected on the building.

When there is uncertainty in the output of a task, the input items (independent variables) of the same task can be sub-grouped and/or assigned to various sources of uncertainty within reasonable boundaries. This is the simplistic description of the sensitivity analysis which is also used in the context of this paper. The harmonic loading of the one-train condition, damping ratios and the horizontal distance from the harmonic load to the tunnel are the sensitivity analysis items focused on within this study. While increasing the loading, decreasing the damping ratio and decreasing the distance to harmonic loads are all independent inputs, they would all result in an increased vibration response at the building foundations. This makes the analyses conservative which serves the purpose of the sensitivity analyses.

In order to assess the sensitivity of the results three additional sets of analyses are conducted with the following variations from the above baseline model:

1. One train loading condition with 20% load amplitude increase.
2. Lower the damping ratio from 3% - 5% range (see Figure 6) to 1% - 2% range.

3. Decrease the horizontal distance from the loading to the building from 50 ft (15.2 m) to 25 ft (7.6 m).

The analysis result for one train condition with 20% load amplitude increase is shown in Figure 9. For 20% increase in the load the VdB values are found to increase by about 1.6 VdB. Therefore, for the most critical frequency of one train condition, the peak increases from 60 VdB to about 61.6 VdB (an increase of 2.7%), still remaining below the FTA VC-A limit (66 VdB).

Considering the frequency dependency of the damping ratios in Rayleigh damping model (Figure 6), the second run to check for the sensitivity analysis was conducted by using lower damping ratios than the original range of 3% to 5% in the FE model runs. The damping ratios were lowered to 1% to 2% in the critical frequency range of 2 Hz - 30 Hz considering the fact that lower damping ratios would result in higher vibration peaks. In other words, lowering the damping ratios makes the analysis more conservative, hence complying with the sensitivity analysis. With that, the analysis result for one train condition with lower damping is shown in Figure 10. For lower damping, VdB values are found to increase by about 1.6 VdB. Therefore for one train condition, the peak increases from 60 VdB to about 61.6 VdB (an increase of 2.7%), staying below the VC-A criteria limit of FTA (66 VdB).

For the third sensitivity analysis, the horizontal distance from the load to the building is decreased from 50 ft (15.2 m) to 25 ft (7.6 m). The analysis results are shown in Figure 11. The results show that for relatively low frequencies (less than 50 Hz) the ground surface vibrations are not sensitive to the tunnel location, at least for the two locations compared against each other. For regions closer to the tunnel, the higher frequency (greater than 50 Hz) vibrations are not attenuated, and consequently at higher frequencies the vibration levels are higher as shown in the figure. For 25 ft (7.6 m) distance, at the most critical frequency value of 25 Hz, the vibrations are found to increase by about 1.0 VdB. The peak increases from 60 VdB to about 61 VdB (an increase of 1.7%), staying below the VC-A criteria limit of FTA (66 VdB).

6. Other components of nuisance

6.1 Vibration nuisance in the existing building

Floor vibrations are unwanted in a building since the oscillations irritate the occupants and cause discomfort. In the last couple of decades, slender structural members and modern construction technologies have resulted in more vulnerable floor systems against vibrations (Avci 2015; Bhargava et al. 2013; Muhammad et al. 2018; Younis et al. 2017). As such, the vibration serviceability of the floors has become a standard design parameter that needs to be checked by the designers to ensure human comfort on building floors (Avci 2005). The vibrations on building floors can be caused not only by human activities such as walking, running, dancing, and aerobics; but also by the train traffic nearby the building (Alabbasi et al. 2019).

There are several methods for the assessment of floor vibrations serviceability, ranging from probabilistic methods to simplified methods (Abdeljaber et al. 2018, 2019). There are four well established floor vibration evaluation methods extensively used over the world. These are the American Institute of Steel Construction (AISC) Design Guide 11 Chapter 4 method (Murray et al. 2016); Steel Construction Institute (SCI) P354 Simplified Method (Smith et al. 2009); Steel Construction Institute (SCI) P354 Vibration Dose Values Method (Smith et al. 2009); and Human Induced Vibration of Steel Structures (HIVOSS) Method (RFCS 2009). While all of these methods are very useful for human-induced excitations, none of them are covering the vibrations serviceability and human comfort levels due to train-induced excitations.

Transmission of train vibrations from the ground into the building and propagation within the building towards the upper floors have been researched by various researchers. It has been reported that developing a methodology to predict vibration levels at the upper floors based on the vibrations at the foundation level is not very straightforward (Sanayei et al. 2014). For human response to vibrations, the FTA Assessment Manual (Hanson et al. 2018) recommends an attenuation of -2 dB per floor for the first five stories of the building; and a -1 dB per floor for the next five stories of the building (floor five to floor ten). However, there is also an amplification component due to the resonances of walls, floors and ceilings which may potentially increase the vibration levels +6 db for light-weight and timber-frame structures. Since the existing building of this study is a steel-framed building, this amplification is not applicable (summarized in Table 3).

Moreover, for the detailed vibration analysis of this paper, the VC-A category curve of the FTA Assessment Manual is used for the vibration impact criterion of the six story office building which is applicable for healthy functioning of optical balances, microbalances, medium-to high-power optical microscopes (400X) and similar specialized equipment. This criterion is compared with the ground surface vibrations (at the foundation level of the building) with the results of detailed FE model investigation and the accompanying sensitivity analysis. As discussed earlier, the peak RMS VdB for the one train loading condition is about 60 VdB and for the two train loading condition is about 64 VdB. Similarly, as the one train loading amplitude is increased 20%, the RMS peak was found to be 61.6 VdB;

and with lower damping ratio, the RMS peak was found to be 61.6 VdB. Even with the decreased distance to the train line, the RMS peak was at 61 VdB. While all these peak values are below the 66 VdB limit of the VC-A criteria, they are all calculated at the foundation level of the existing building which is fairly conservative. This conservatism is due to the fact that the vibrations will attenuate as the waves are propagating upwards within the building. The FTS suggested attenuation of -2 dB per floor for the first five stories of the building; and a -1 dB per floor for the next five stories of the building would further decrease the already acceptable RMS VdB peaks. As a result of this, no adverse impact is expected for occupants located on the elevated floors of the existing building.

On another note, there are simplified impedance-based analytical models available in the literature for train-induced vibration predictions (Sanayei et al. 2011). In these models, the vibrations propagate through the axial waves in the columns of the building. The impedance of columns and slabs representing the stiffness, mass and damping properties are the predominant components of the model where each finite portion of the column is modeled according to the impedances at the top and bottom of each column. With that, the displacements and forces at the ends of the columns are represented in the dynamic stiffness matrices.

6.2 Noise nuisance

Like vibration, noise may also cause annoyance to building occupants. Not only trains but also other transit sources create noise in various levels based on the type of the transit and operating conditions. Between the source and the receiver, the level of noise can be attenuated along the path depending on the obstacles in the way, ground type, damping components and several other factors. Generally, the noise metrics are expressed in terms of “A-Weighted Decibels” (dBA) which is the basic noise unit in transit noise terminology. Also known as “A-Weighted Sound Level”, this term represents the overall noise at a receiver location that is adjusted in frequency to approximate typical human hearing sensitivity. The letter "A" indicates that the sound has been filtered to reduce the strength of very low and very high-frequency content to represent the human response to sound levels (Hanson et al. 2018).

In transit noise impact assessment, another important parameter is the “Sound Exposure Level” (SEL). This term is defined as the cumulative noise exposure from a single noise event, normalized to one second. The louder events have a larger SEL value than the quitter events. Similarly, the shorter events have a smaller SEL value than the longer events. Basically, the SEL value represents the identical overall sound energy as the actual varying sound energy during the single noise event. Being a primary metric for the measurement of transit vehicle noise emissions, it is also an "A-Weighted" cumulative measure.

In transit noise terminology, another common term is the "Maximum Sound Level" (L_{max}) which is also known as the "Maximum Noise Level". This measure represents the maximum level of sound generated during a single noise event, and it is considered as "A-Weighted Maximum Noise Level" in various references. This metric is predominantly used in vehicle noise specifications and typically calculated for individual vehicles.

The equation for computing L_{max} for a single locomotive pass-by per FTA Assessment Manual (Hanson et al. 2018) is:

$$L_{max} = SEL_{ref} - 10log\left(\frac{L_{meas}}{50}\right) - 10log\left(\frac{D_{meas}}{50}\right) + 10log(2\alpha) - C_{consist} - C_{emissions} - 3.3 \qquad (13)$$

Where;

SEL_{ref}= Source Reference Level (Reference SEL)

L_{meas} = total length of measured group of locomotives or rail cars (ft)

D_{meas} = closest distance between measurement position and source (ft)

$\alpha = \arctan\left(\frac{L_{meas}}{2D_{meas}}\right)$ in radians

$C_{consist} = -10\log(N_{cars})$ for locomotives and rail cars where N_{cars} is the number of locomotives in the measured group

$C_{emissions} = 0$ for T<6 for locomotives

$C_{emissions} = -2(T-5)$ for T≥6 for locomotives where T is average throttle setting of the locomotives.

While the sound nuisance of the train excitations was not intended to be discussed in this manuscript, the train noise calculations are briefly shown here for the sake of completeness. The train-induced noise calculations were also not run nor discussed as the presented FE model was used only to investigate the vibrations resulting from the train movements, at the foundation level of the existing building.

7. Conclusions

In this paper, ground surface vibrations are investigated due to trains passing through proposed underground tunnels in the vicinity of an office building. A 2D finite element model is created reflecting the railroad tunnels embedded in the rock and the soil stratum above it. Also, a train loading function is applied to the model as a point source at the track level. Various simulations are undertaken followed by a sensitivity analysis. Taking the plane strain assumption within the finite element model simulations, the vibrations from the railway tunnel were determined to have peaks in the frequency ranges of 8 Hz to 12 Hz and 20 Hz to 30 Hz. In these ranges the peak RMS VdB for one train loading condition is about 60 VdB and for two train loading condition the peak RMS is about 64 VdB. Meanwhile, for the sensitivity analyses, when the one train loading amplitude is increased 20%, the RMS peak was found to be 61.6 VdB. On another run, when lower damping values are used, the RMS peak was found to be 61.6 VdB. For the last sensitivity analysis run, the distance from the building to the loading is decreased to 25 ft (7.6 m) which resulted in an RMS peak of 61 VdB. The results are compared to the criteria of Transit Noise and Vibration Impact Assessment Manual, which was published by Federal Transit Administration (FTA) of the U.S. Department of Transportation. Since all of the peaks are observed to be lower than the FTA VC-A criteria (66 VdB); no adverse impact is expected on the building.

The predominant limitations of the study presented in this paper could be listed as the use of 2D analysis and material linearity. While a 2D model is relatively easier to build and processed faster, a 3D model would have allowed a more realistic modeling of the harmonic force function and would have produced more information on wave propagation. Yet, a 3D model would also have the same material and geometrical unknowns as a 2D model for the tunnels, surrounding media, and complex dynamic interaction among the train car and wheel components. In addition, while the material and dynamic properties of the structural members and the soil layers are known to be non-linear in reality, the model used in the study assumes a linear behavior.

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Table 1 - Interpretation of Vibration Criteria for Detailed Vibration Analysis (adapted from Table 6-6 of the FTA Assessment Manual)

Criterion Curve	Max Lv, VdB	Description
Workshop (ISO)	90	Vibration that is distinctly felt. Appropriate for workshops and similar areas not as sensitive to vibration.
Office (ISO)	84	Vibration that can be felt. Appropriate for offices and similar areas not as sensitive to vibration.
Residential Day (ISO)	78	Vibration that is barely felt. Adequate for computer equipment and low-power optical microscopes (up to 20X).
Residential Night, Operating Rooms (ISO)	72	Vibration is not felt, but ground-borne noise may be audible inside quiet rooms. Suitable for medium-power optical microscopes (100X) and other equipment of low sensitivity.
VC-A	66	Adequate for medium-to high-power optical microscopes (400X), microbalances, optical balances, and similar specialized equipment.
VC-B	60	Adequate for high-power optical microscopes (1000X) and inspection and lithography equipment to 3-micron line widths.
VC-C	54	Appropriate for most lithography and inspection equipment to 1-micron detail size.
VC-D	48	Suitable in most instances for the most demanding equipment, including electron microscopes operating to the limits of their capabilities.
VC-E	42	The most demanding criterion for extremely vibration-sensitive equipment.

Table 2
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Table 2 – Shear wave velocities used in the FE model

Material	Property	Symbol	US Customary units	SI units
Soil	Shear Wave Velocity	V_s	400 ft/s	122 m/s
Rock	Shear Wave Velocity	V_s	13,000 ft/s	3963 m/s

Table 3
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Table 3 – Receiver Adjustment Factors for Ground Borne Vibration Propagation (adapted from Table 6-13 of the FTA Assessment Manual)

Receiver Factor	Adjustment to Propagation Curve		Description
Floor to floor attenuation	1 to 5 stories above grade	-2 dB per story	This adjustment is intended to address the dispersion and attenuation of vibration energy as it propagates through a structure with the first starting with elevated floor.
	5 to 10 stories above grade	-1 dB per story	
Amplification due to resonances of floors, walls, and ceilings.	+6 dB		The amplification will depend on the type of the existing structure (wood frame, masonry, reinforced concrete, steel).

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List of Figures

Figure 1

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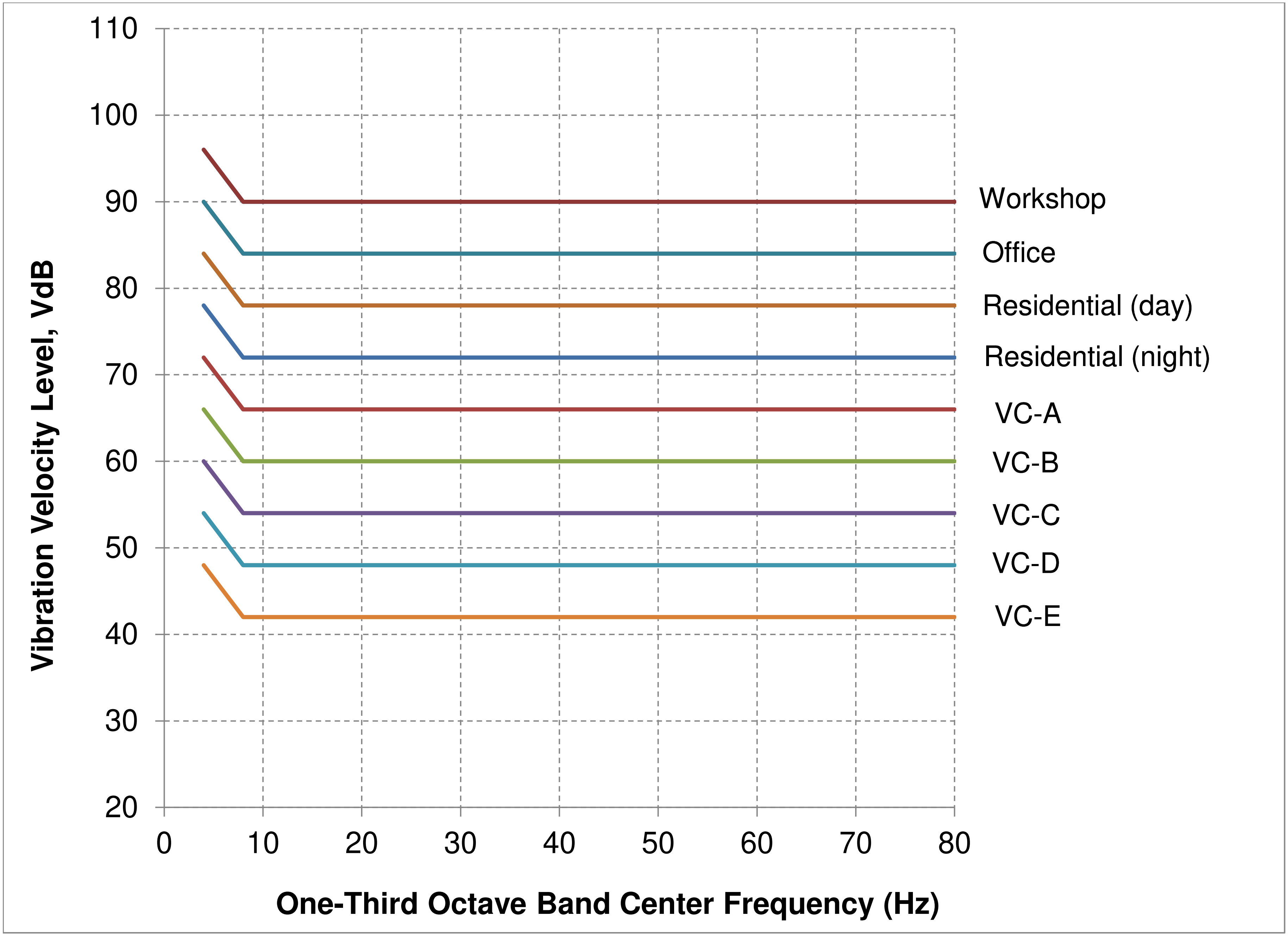

Figure 2a

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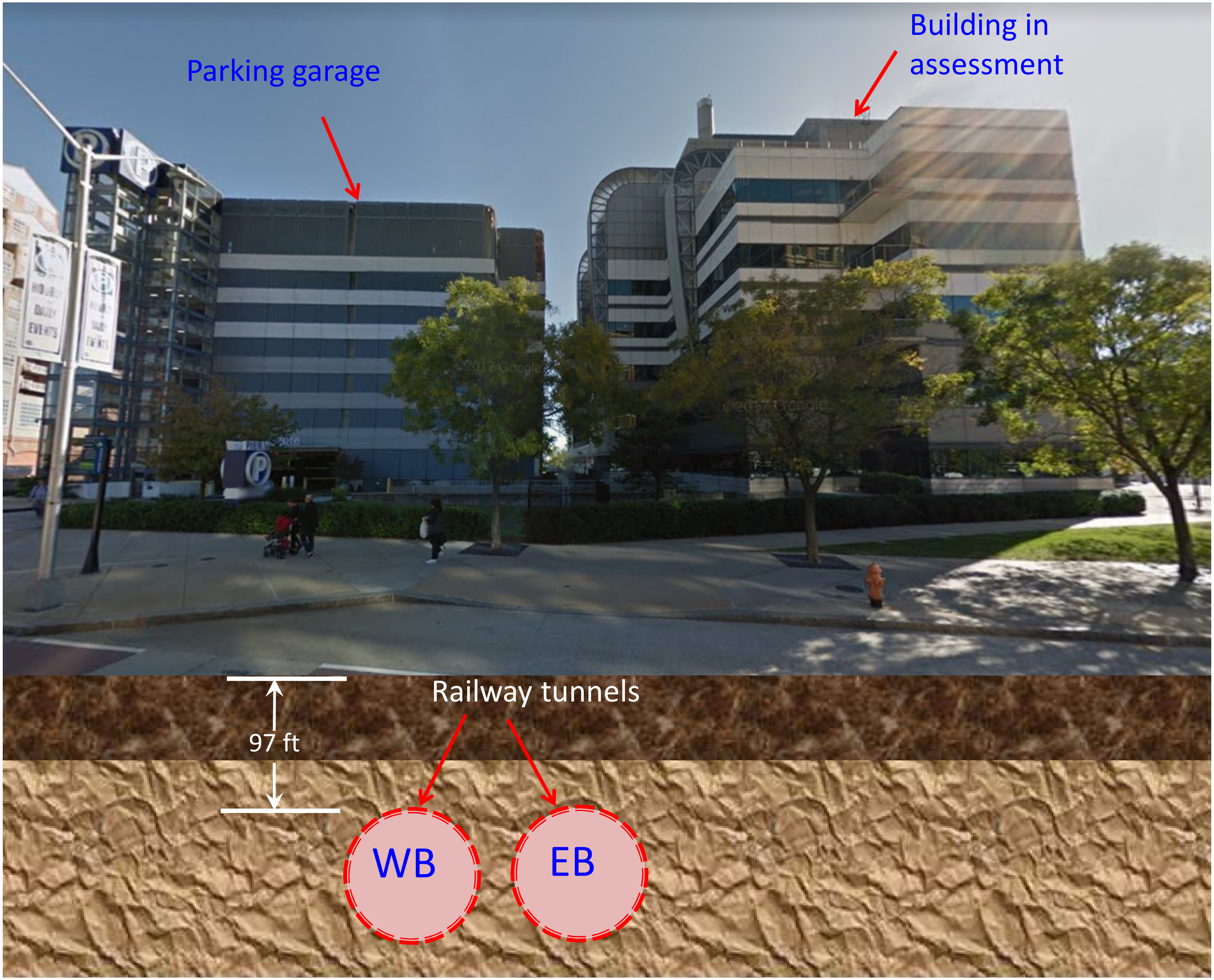

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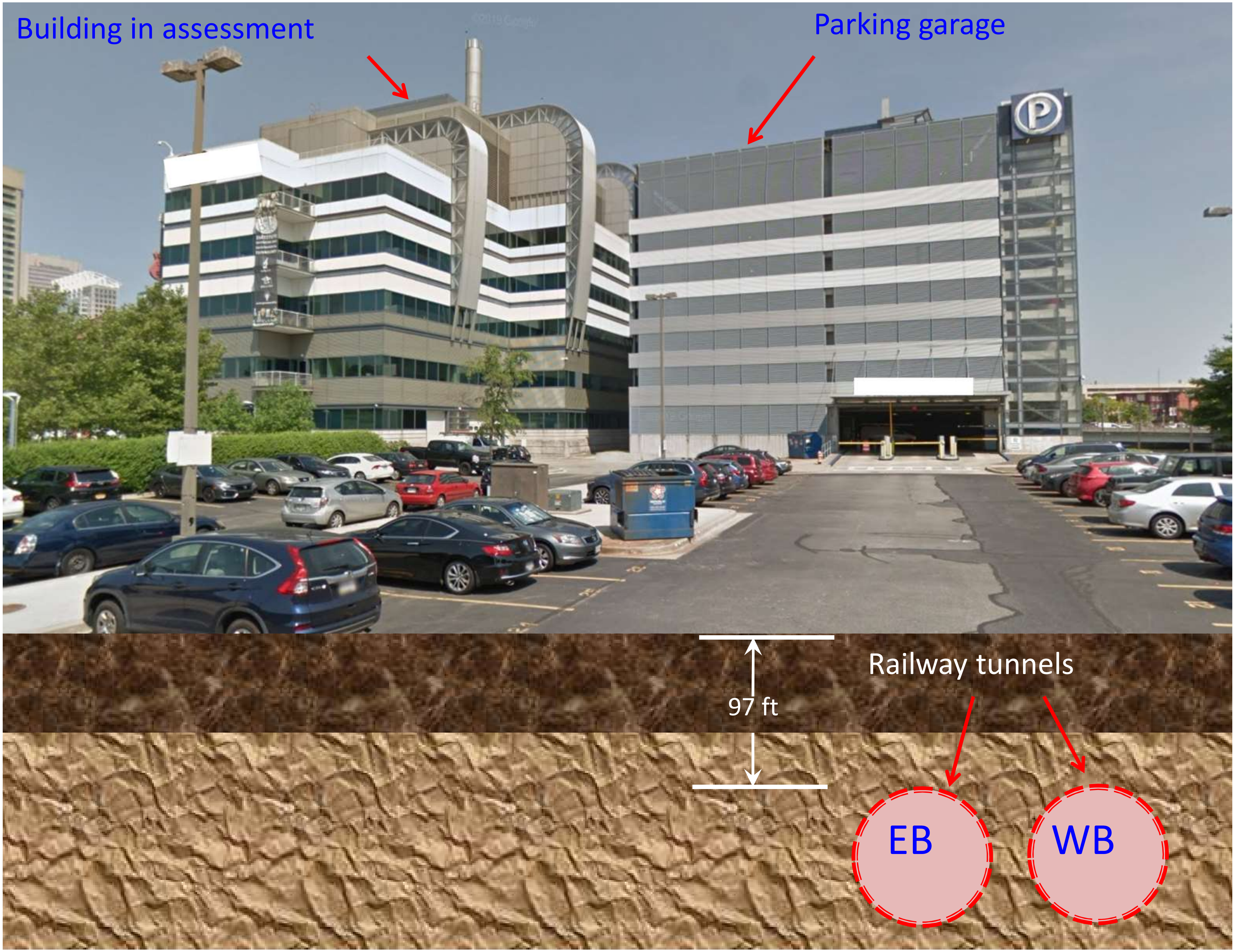

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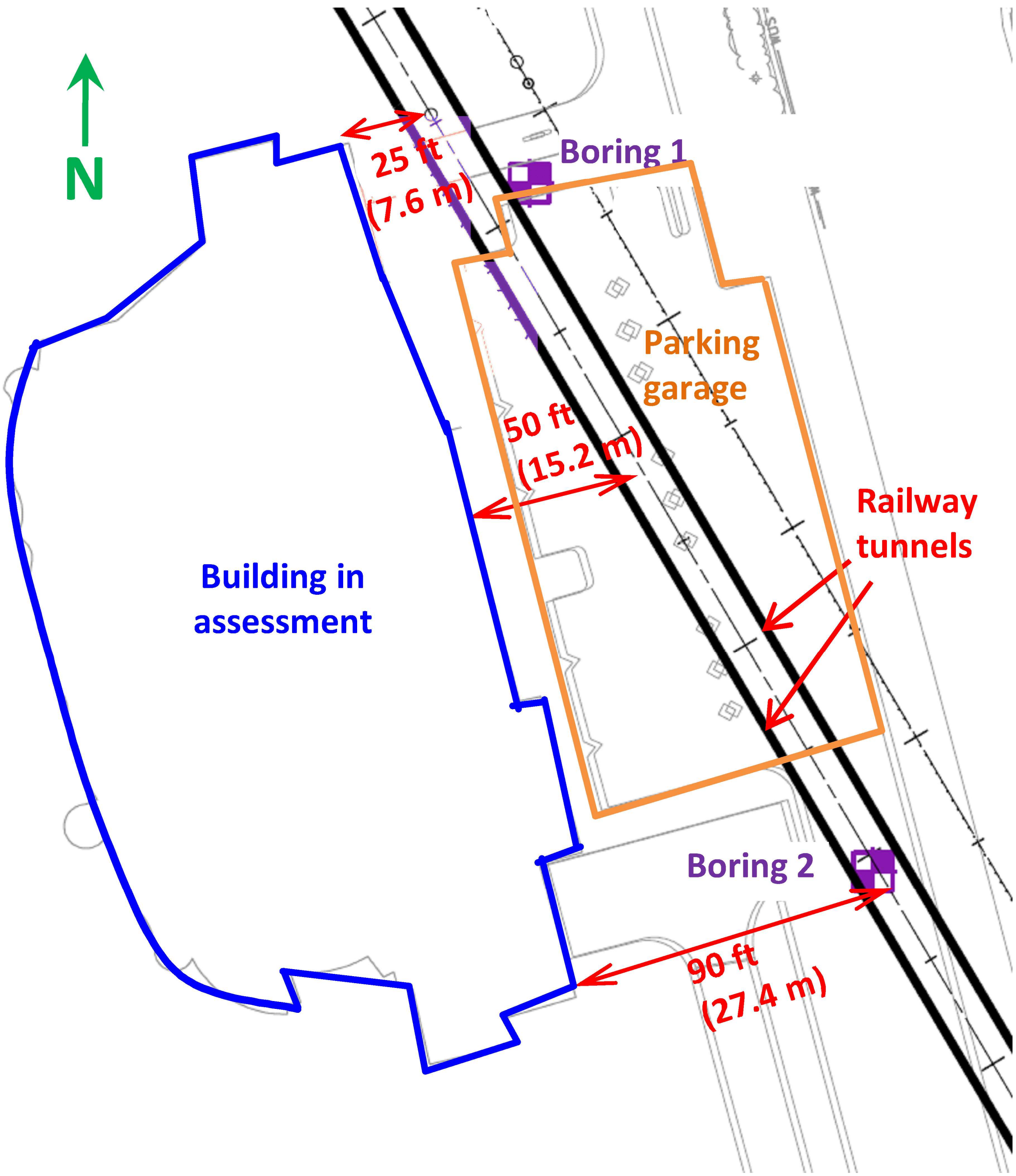

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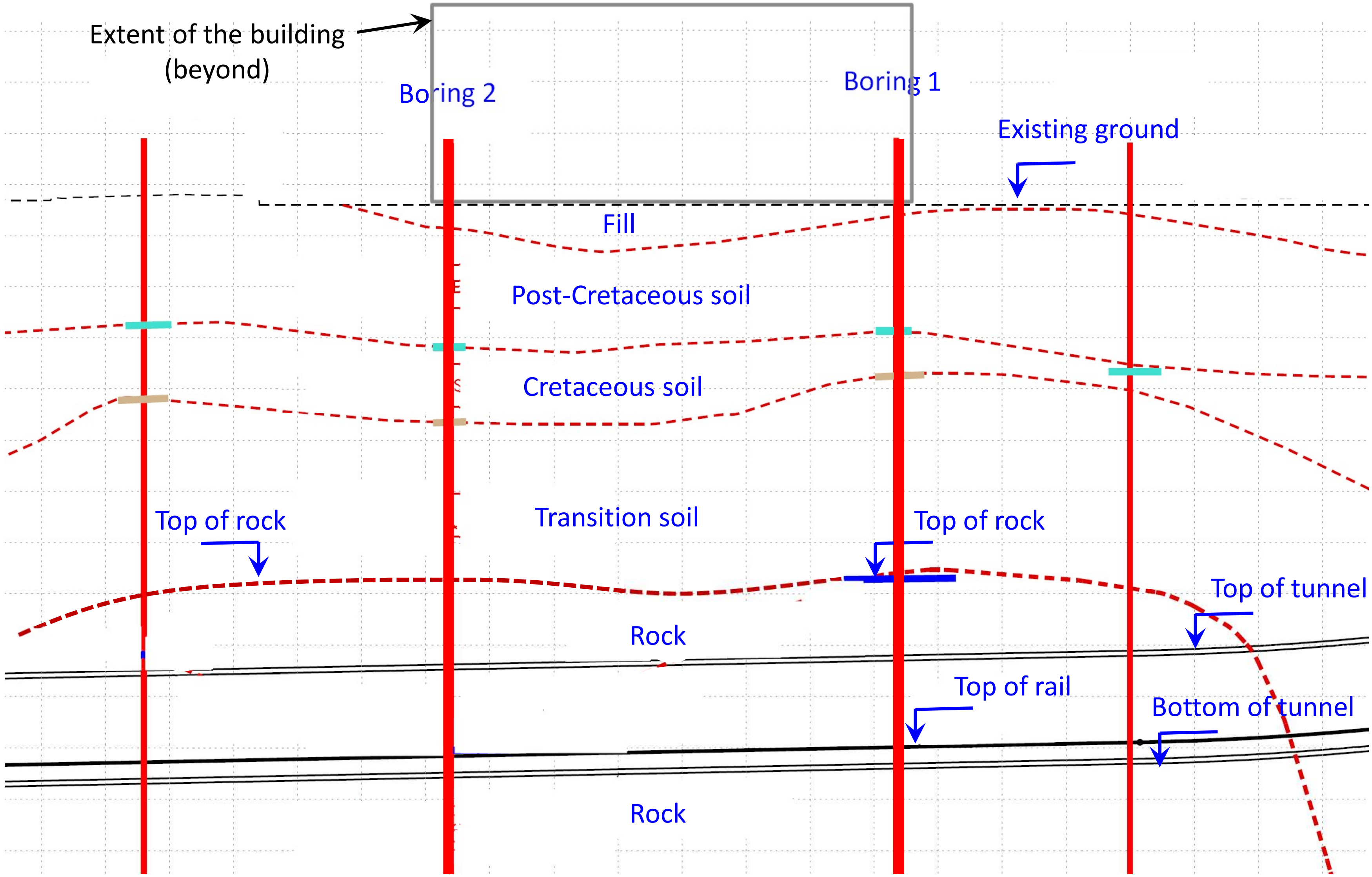

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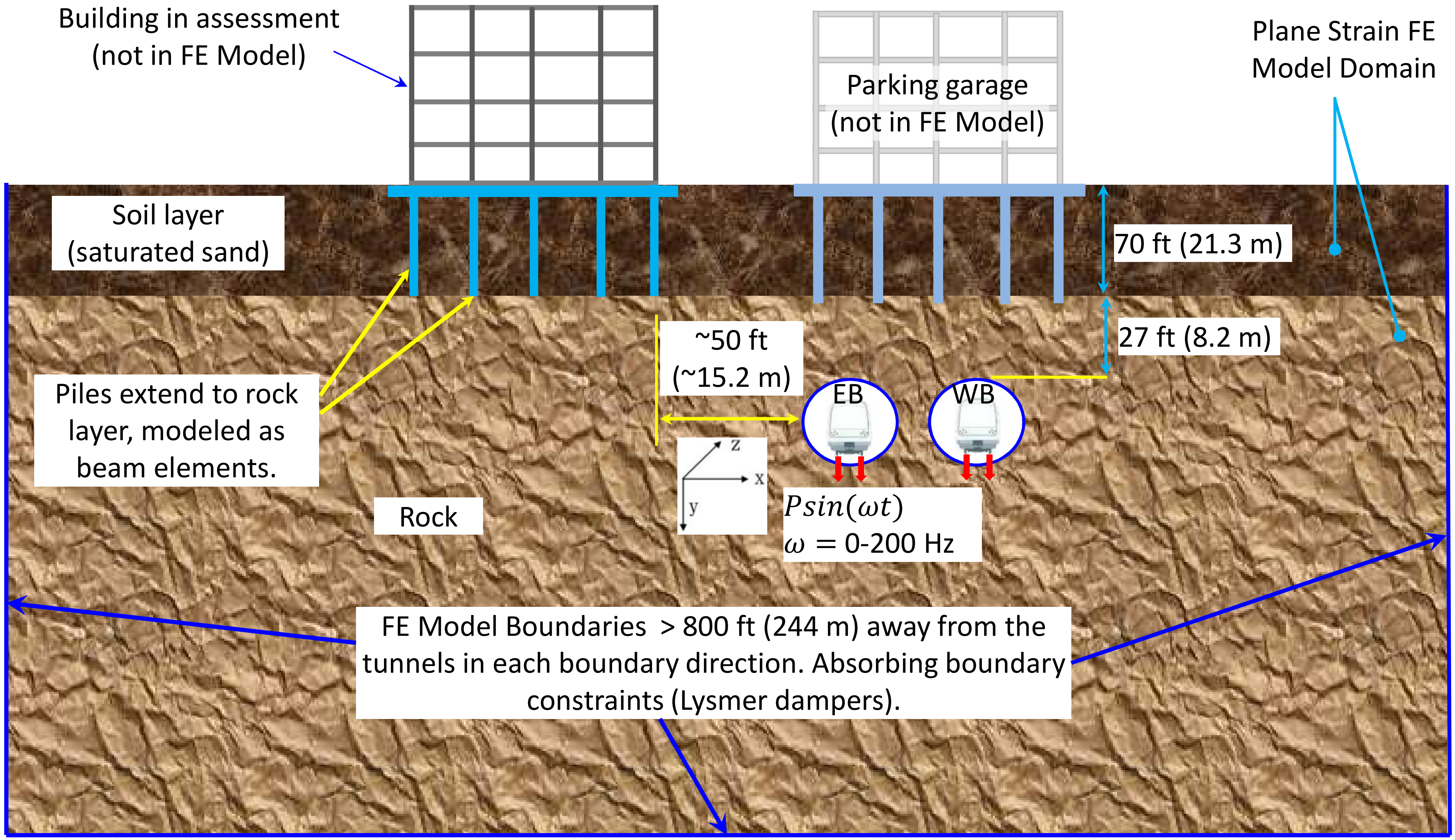

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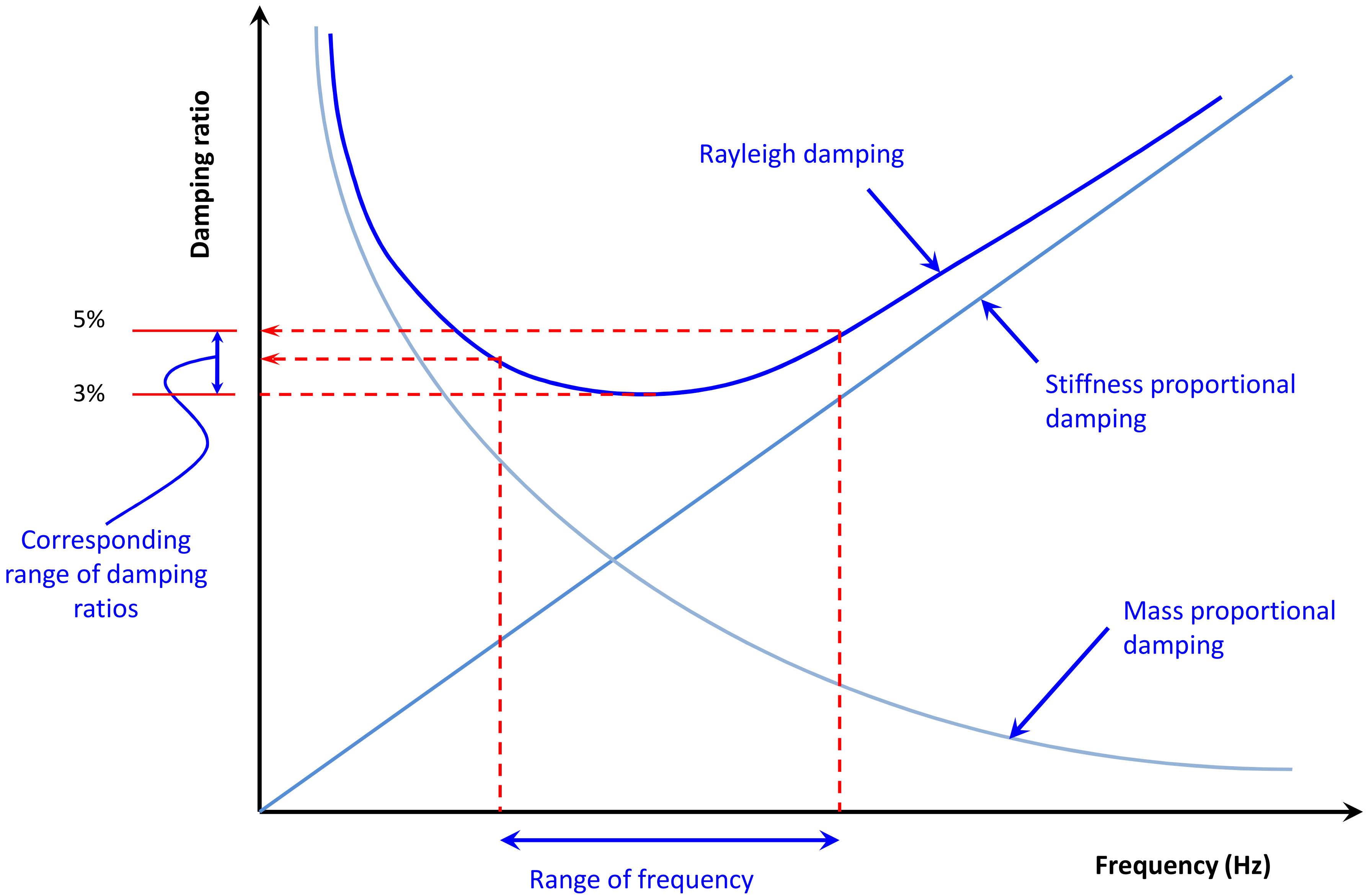

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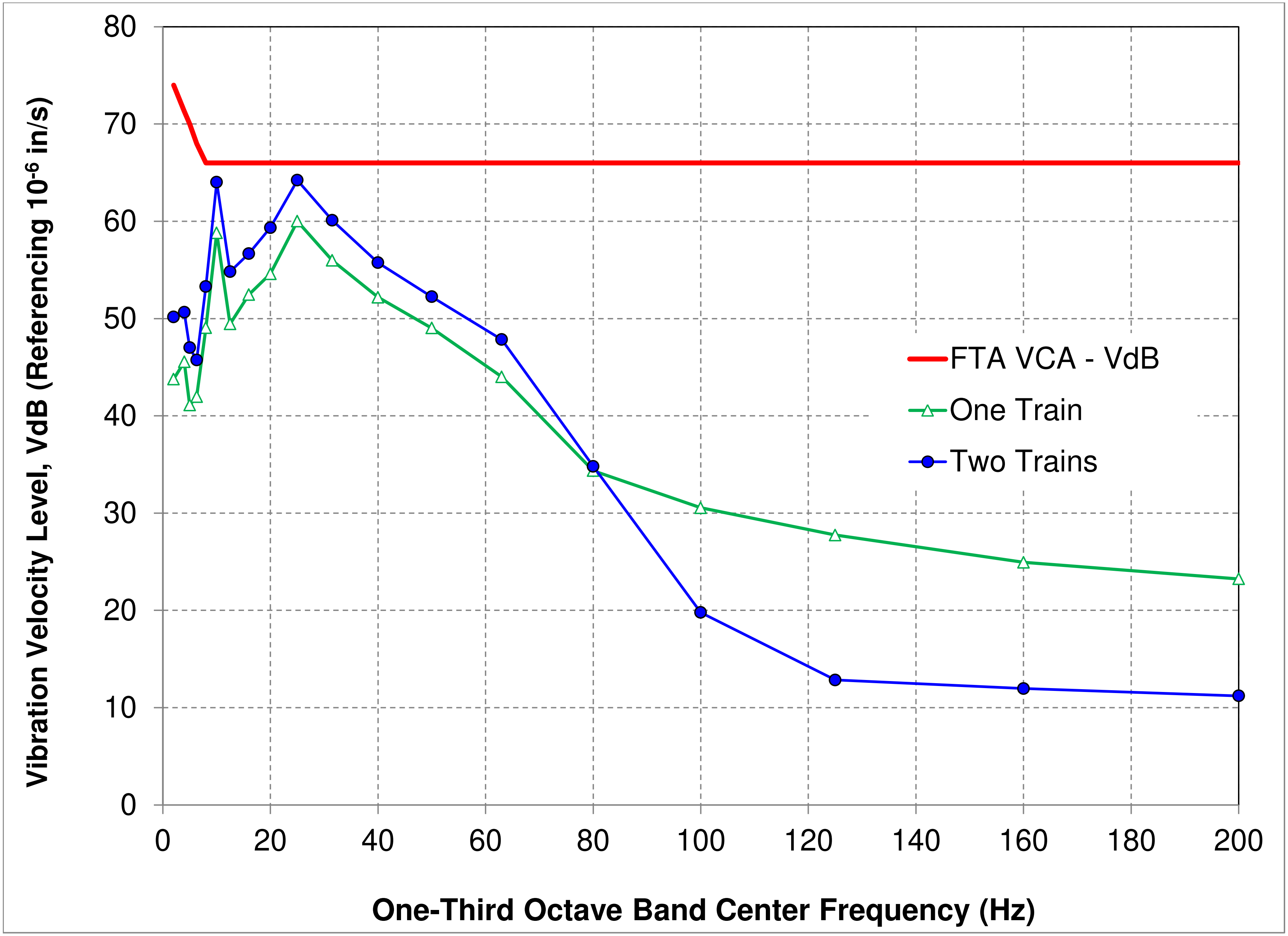

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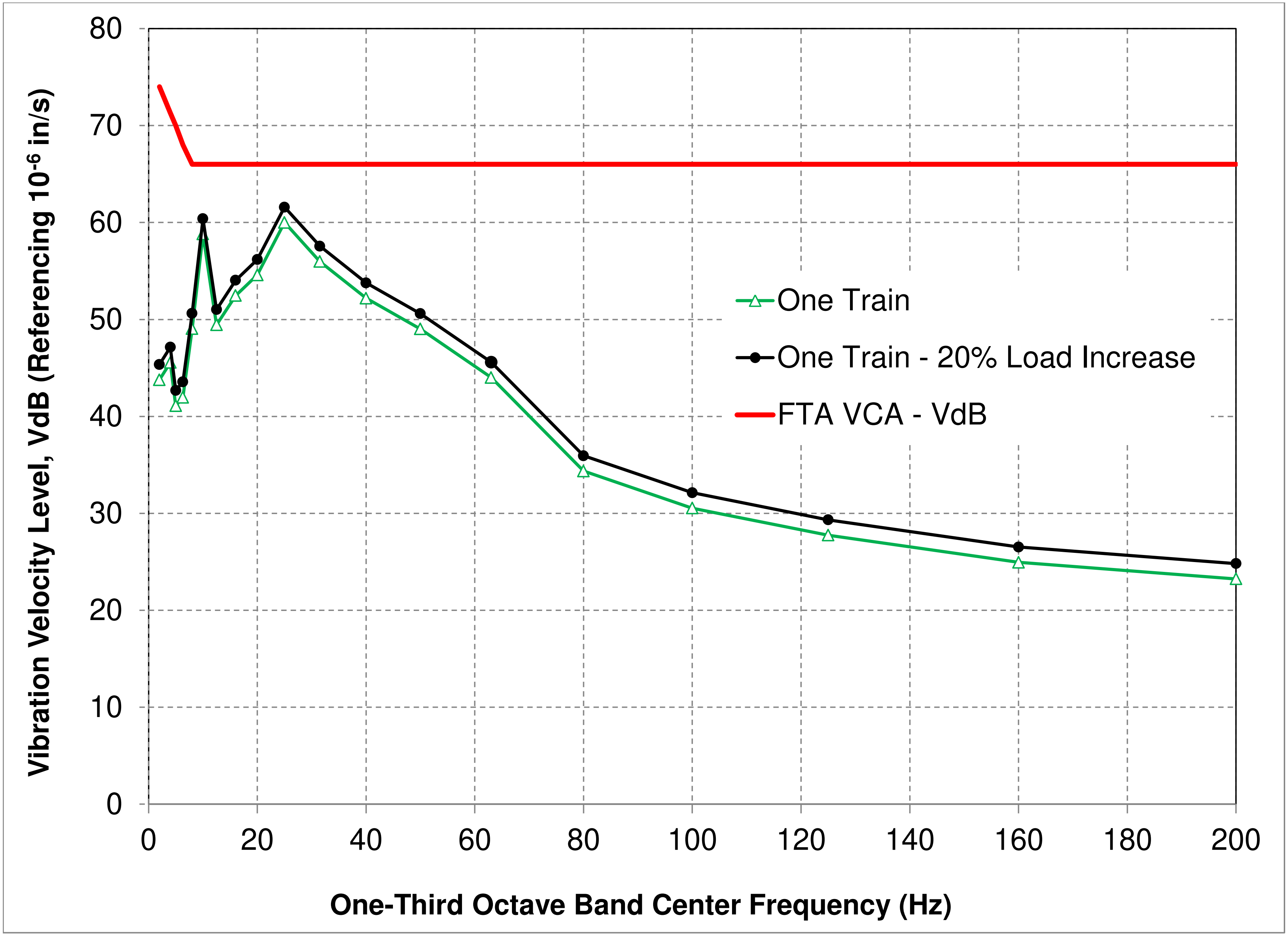

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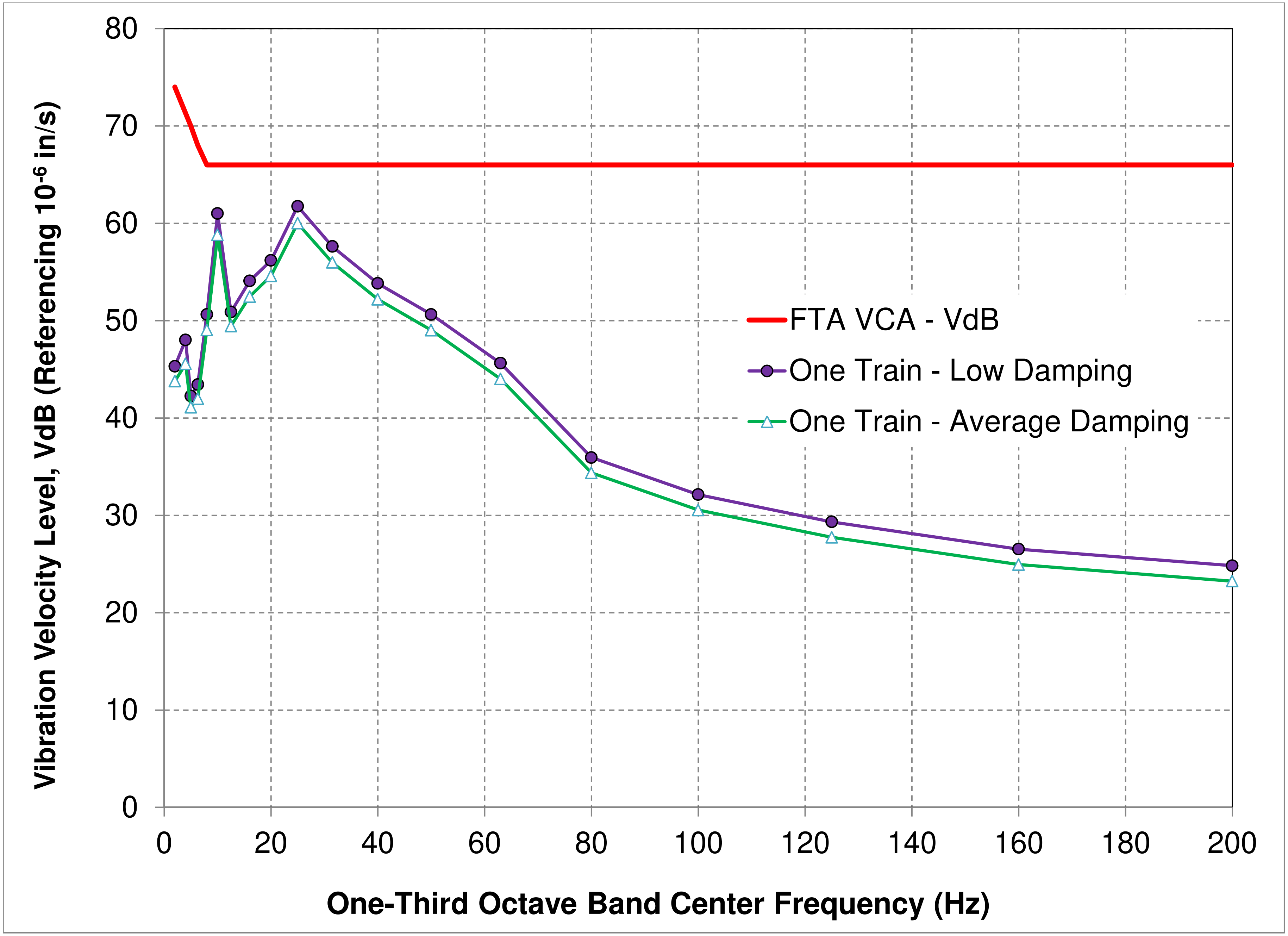

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