EFFECTS OF RAILROAD TRACK STRUCTURAL COMPONENTS AND SUBGRADE ON DAMPING AND DISSIPATION OF TRAIN INDUCED VIBRATION

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SUBGRADE ON DAMPING AND DISSIPATION OF TRAIN INDUCED
VIBRATION

DISSERTATION

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in the College of Engineering at the University of Kentucky

By
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EFFECTS OF RAILROAD TRACK STRUCTURAL COMPONENTS AND SUBGRADE ON DAMPING AND DISSIPATION OF TRAIN INDUCED VIBRATION

A method for numerical simulation of train induced track vibration and wave propagation in subgrade has been proposed. The method uses a mass to simulate the bogie of a train and considers the effect of rail roughness. For this method, rail roughness is considered as a randomly generated signal and a filter is used to block the undesired components. The method predicts the particle velocity around the track and can be applied to many kinds of railroad trackbeds including traditional ballast trackbed and modern Hot mix asphalt (HMA) trackbed. Results from ballast and HMA trackbeds are compared and effects of HMA layer on damping track vibration and dissipating wave propagation are presented. To verify the credibility of the method, in-track measurements were also conducted. Site measurements included performing geophysical tests such as spectral analysis of surface wave test and seismic refraction test to determine the subsurface conditions at the test site. Ballast and HMA samples were tested in the laboratory by resonant column test to obtain the material properties. Particle velocities were measured and analyzed in the frequency domain. Results from in-track tests confirm the applicability of the numerical method. The findings and conclusions are summarized and future research topics are suggested.

KEYWORDS: Shear Wave Velocity, Railroad Track Vibration, Rail Roughness, Rayleigh Wave Propagation, Peak Particle Velocity
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To my parents
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Chapter One

Introduction

Railroads have been used for more than a century as a very important transportation mode. Currently, two trends involving axle load and train speed are focused as railroads continue to develop and expand. These are accommodating heavy axle loads for freight trains and high-speeds for passenger trains. However, train induced vibration and noise have been recently recognized as major environmental issues. Therefore, various new materials have been applied to the railroad tracks for minimizing the environmental pollution associated with heavy axle load freight lines, and high-speed passenger lines.

On United States railroads, heavy axle loads for freight railroad lines have increased dramatically in recent years. For example, according to the Association of American Railroads (AAR) data, U.S. Class I railroads set another traffic record in 2003 despite sluggish growth in the industrial economy and a flat coal market (AAR, 2004). At the same time, statistical data confirms that the cost can be reduced by increasing the axle load so that the profits can be guaranteed for freight railroad companies (Armstrong, 1998). For instance, Class I operating revenue grew 3.7 percent for 2003 to a record $36.6 billion, driven by higher traffic volumes and slightly higher nominal revenue per ton-mile (AAR, 2004). As a result of efforts to improve the track performance under heavy axle loads, Hot Mixed Asphalt (HMA) trackbed has been tested, evaluated and applied since the early 1980s (Rose, Brown, and Osborne, 2000). Previous studies showed
advantages of HMA trackbed for improving the ride quality of the train and simplifying track maintenance activities (Su, 2002). Furthermore, another interesting finding was that HMA has a potential benefit to minimize ground vibrations when incorporated as a sublayer in the trackbed. If this potential benefit can be confirmed, there is a very extensive range for its application. It will not only be limited to open track, but it will also be particularly applicable for track locations where abnormal dynamic forces are frequently generated such as rail joints, turnouts, bridge approaches and crossing diamonds. Therefore, testing of HMA performance in reducing track vibration is performed in this research.

Passenger transportation services via high-speed railroads have been very popular in Europe and East Asia for many years. In Europe, it was found to be an effective solution for public transportation due to its advantages in economic and environmental aspects. Based on the previous study (Übleis, 1992), several items such as energy consumption per traveler, noise emission, and air pollution were investigated between railroad, highway, and aviation systems. The results arguably showed that high-speed rail systems were superior to aviation and highway systems. However, a rail system is not perfect and has its own defects. One of the problems for a high-speed passenger rail system is the vibration due to corrugated rail. Generally, the negative effects associated with vibration include excessive train body acceleration, which causes passenger discomfort, and ground borne propagation, which causes failure of soft subgrade.
The discomfort of passengers due to excessive train to body vibrations can be overcome by adjusting the train’s suspension system to reduce the train to body vibration at high speeds. The ground borne propagation problem is expected to be solved by decreasing the vibration of trackbed, which is the source of ground borne noise and vibration. When reviewing the energy dissipation in a trackbed, it can be found that it is the kinetic energy, which is originally created from the train vibration that causes the trackbed (ballast) vibration. Some energy is dissipated due to the particle friction from ballast vibration, while the other energy is radiating to the subgrade in form of waves. The percentage of energy dissipated by the trackbed is dependent on the stiffness, damping capacity, and vibration frequency of the trackbed. A previous study (Zeng, Rose and Rice, 2001) showed that asphalt related materials such as HMA and Rubber Modified Hot-mixed Asphalt (RMHA) have a potential success in the use as a sublayer of trackbeds for high-speed rail lines. To further investigate the effect of applying new material in a railroad track for reducing vibration, a method for simulating ground vibration and wave propagation around the track should be developed.

The above description of testing new material performances in both freight and passenger lines reveals that it is necessary to develop a method which is able to simulate train induced ground vibration and wave propagation phenomena. By using this new proposed method, the effect in vibration reduction of new material such as HMA can be evaluated.
Chapter Two

Methodology

Previous research has been conducted to evaluate soil response under moving loads (Cole and Huth, 1958; Eason, 1965; Payton, 1967; Fryba, 1972; Balendra, Chua, Lo and Lee, 1989; Duffy, 1990; de Barros and Luco, 1994; Krylov, 1995; Dieterman and Metrikine, 1997; Kim and Roesset, 1998; Takemiya, 1998; and Grundmann, Lieb and Trommer, 1999). Their studies can be considered an analytical approach to an idealized case. In this “idealized” type of analysis category, a train load was simplified to a concentrated constant load. The trackbed was neglected and subgrade soil was idealized as a uniform continuous medium composed of one or more layers. Generally, the exact solution for this type of problem is very difficult to achieve. Also, the model used in this type of analysis has been highly simplified, thus its result does not truly representative of the actual case. Therefore, it is an ideal method for qualitative analysis of train induced vibrations, but is not a good solution for a practical engineering problem.

For other analytical approaches, field in-situ measurement is the most direct method to measure train induced ground vibration. It records the data directly or indirectly, and the results are generally accepted as the “true” value. However, the railroad extends several hundred miles and the properties of the track, trackbed, and subgrade vary greatly. It is unrealistic to make measurements for each case, to draw a generalized conclusion, especially for railroad track vibration, because too many primary factors are involved such as
train type, train speed, axle load, trackbed type, trackbed thickness, subgrade soil properties, etc.

To overcome disadvantages of field in-situ measurements, another method should be adopted. We know that numerical simulation would be a good supplement to in-situ measurements. Therefore, once the numerical model has been setup, changing a primary factor is easy so it can cope with different cases. The disadvantage of numerical analysis is that in a numerical simulation, some idealized materials with some behaviors such as perfect uniform soil, are employed and cannot precisely represent the real material behavior. Sometimes the numerical result may not be accurate even if a satisfied numerical technique is applied due to the idealization of material properties. The numerical model is trustworthy only when its results can be confirmed by other methods.

It is obvious that field in-situ measurements and numerical simulation are complimentary with respect to each other. Therefore, in this research, both in-track and numerical modeling methods will be used and their results will be compared to verify each other.

The steps performed in this research included the following:

- A test site was selected. The ideal place should include both HMA and ballast trackbed. A CSX Transportation revenue mainline at Conway, Kentucky met this requirement. Another advantage of choosing this place is that ample trains can be guaranteed since it is a revenue line. Also, the types and speeds of trains on this line each day are consistent and the axle loads only change slightly.
- Pressure distribution under the ties would be determined based on some in-track pressure tests and results would be verified by predicted values from theoretical analysis. The acquired pressure distribution would be used as inputting loads (force boundary) for numerical analysis.

- Ballast and HMA properties would be determined by the free-free resonant column test based on the samples collected from the test site. Several soil parameters such as shear modulus, Poisson’s ratio and damping ratio would be determined and those values would be utilized as the material parameters for numerical simulations.

- The in-situ field test includes the Spectral Analysis of Surface Wave (SASW) test, seismic refraction test, and subgrade vibration recording. It is very important to determine the structure of the subsurface. This includes the number of layers contained in the subgrade, the thickness of each layer, and the primary and shear wave velocities of each layer. The in-situ tests could be completed by applying SASW and seismic refraction tests with the results from the two methods used to confirm the results of each test. Results obtained for number of layers, layer thickness, and shear wave velocities etc., would be used as required information for numerical simulation.

- Subgrade vibration was measured directly by accelerometers and geophones. These return surface soil particle acceleration and velocity in the time domain respectively. Post data processing was carried out based on a spectral analysis method such as Fourier transform and data was reviewed in the frequency domain.
Numerical simulation was done using the Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC$^{3D}$) program. Boundary forces used in the numerical simulation were generated by Matlab script and it is described in Chapter 6. In this force generation procedure, two factors that affect ground vibration were considered. They are bogie mass and rail roughness. Model parameters used in FLAC$^{3D}$ were acquired from the laboratory and in-track test results. The simulation result was the soil particle vibration velocity, which is the same motion as the field measurement. Therefore, direct comparisons were made. By using the benefits of numerical simulation previously described, effects of different types of trackbed materials on the ground vibration could be investigated by only adjusting the material parameter's value. This is significant for a better understanding of HMA trackbed performances in different circumstances.

By going through the above steps, HMA and ballast trackbed results under several different conditions have been evaluated and compared.
Chapter Three

Pressure Determinations in Track

In order to better understand the performance of a track under train loadings, it is necessary to determine the pressure distribution in the track. Previous research studies in this area have been conducted using both theoretical analyses and field tests. For the pressure distribution in the rail track structure, beam on elastic foundation theory has been utilized for over a hundred years for calculating the rail bending stress (Winkler, 1867; Zimmermann, 1887). Also, based on laboratory tests and measurements, pressure distribution under the tie and bending stress of tie was calculated (Talbot, 1918; Clark, 1957; and Schramm, 1961). Relative to the pressure distribution in the trackbed and subgrade, both theoretical and laboratory studies have been performed. In theoretical analysis, Boussinesq elastic layer theory was frequently used, which assumed that the trackbed or subgrade was composed of a semi-infinite, homogenous idealized material (Boussinesq, 1885). Laboratory test work was done by inserting the pressure cells in to the trackbed (Talbot, 1918). Although a portion of both results is still used today, it is only considered when an approximate value is desired. As technology improved, advanced testing equipment has been made available such that more accurate test procedures can be used. At the same time, empirical equations have been replaced by using some advanced simulation techniques such as finite element method, finite difference method, etc. This involves setting up a track model and assigning corresponding material properties so that accurate pressures can be calculated.
**Test site description – Conway, Kentucky**

Conway, Kentucky is located on CSX Transportation’s mainline between Cincinnati and Atlanta. The test site parallels highways US 25 and I-75 just a few miles south of Berea, Kentucky. This heavy-tonnage, high-speed, double track mainline railroad is owned and operated by CSX Transportation. The eastern track (#2) is the mainline track with an HMA underlayment and receives the majority of traffic. The western track (#1) is used primarily as a passing siding and is composed of conventionally ballasted track design. This portion of CSX track experiences an annual tonnage of approximately 50 million gross tons.

The selected track was originally constructed between 1881-1883, at which time it was called the Kentucky Central. In June 1983, it was reconstructed and two 1000-foot sections of HMA underlayment were placed under the wood tie mainline track for testing and evaluation. One 350-m long section is composed of a 13-cm thick HMA underlayment and the other 350-m long section has an 20-cm thick HMA underlayment. Thirteen to twenty center-meter of new ballast was placed on top of the asphalt after construction. All measurements for this research were conducted on the #2 mainline track, with pressure cells placed above the HMA underlayment and below the HMA underlayment. The Tekscan sensors were placed between the rail base and tie plate.

Figure 3.1 shows a longitudinal view of the current track structure at Conway. Figure 3.2 is a current view of the track.
In-track pressure measurements by Tekscan sensors

In the past, direct pressure measurements in the rail base/tie plate, tie plate/tie interfaces have been difficult to measure. However, newly obtained Tekscan sensors have been utilized. Tekscan sensors (Tekscan Inc., 2003) are
made by a thin (0.1mm thick) matrix-based sensor consisting of two flexible polyester sheets with silver conductive electrodes printed on them. One sheet has a semi-conductive “ink” printed in rows while the other sheet has the “ink” printed in perpendicular columns. These two sheets of polyester are glued together at the edges. The “ink” is pressure sensitive and its conductivity varies with the force applied to it, similar to a strain gauge. By exciting one row and one column at a time the system isolates the location where the row and column meet which completes the circuit. The force applied is determined by measuring the change in resistivity through the circuit. The progress is repeated for all the rows and columns and the distribution of force over the active area is thus determined (Rose and Stith, 2004).

Tests were conducted on an open track at CSX mainline at Conway. A six-axle engine was selected which has a wheel load of 16.33 metric ton (18 short ton). Figure 3.3 shows the dead load measurement using a Tekscan sensor. Figure 3.4 illustrates one of the engine bogie positions on the track. Test results are shown in Table 3.1. Results are given in form of force, which was calculated from the measured pressure between the rail and rail plate interface.
Figure 3.3 Dead Load Measurement by Using Tekscan Sensor

A set of wheels for a bogie of 6-axle engine

Figure 3.4 Bogie Position along the Track
Table 3.1 Load Distribution Measured by Using Tekscan Sensors

<table>
<thead>
<tr>
<th>Tie Number</th>
<th>Force (kN)</th>
<th>Tie Number</th>
<th>Force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>54.42</td>
<td>12</td>
<td>85.28</td>
</tr>
<tr>
<td>2</td>
<td>86.67</td>
<td>13</td>
<td>62.88</td>
</tr>
<tr>
<td>3</td>
<td>112.60</td>
<td>14</td>
<td>41.99</td>
</tr>
<tr>
<td>4</td>
<td>118.88</td>
<td>15</td>
<td>26.11</td>
</tr>
<tr>
<td>5</td>
<td>116.45</td>
<td>16</td>
<td>21.48</td>
</tr>
<tr>
<td>6</td>
<td>120.81</td>
<td>17</td>
<td>17.97</td>
</tr>
<tr>
<td>7</td>
<td>123.35</td>
<td>18</td>
<td>15.09</td>
</tr>
<tr>
<td>8</td>
<td>120.11</td>
<td>19</td>
<td>12.26</td>
</tr>
<tr>
<td>9</td>
<td>115.50</td>
<td>20</td>
<td>11.13</td>
</tr>
<tr>
<td>10</td>
<td>113.19</td>
<td>21</td>
<td>9.78</td>
</tr>
<tr>
<td>11</td>
<td>112.86</td>
<td></td>
<td>1.0 kN = 225 lbs</td>
</tr>
</tbody>
</table>

It should be noted in Table 3.1 that the peak value occurs just under the bogie as expected. The maximum peak value is 123.35 kN at tie number 7. Values between tie number 3 and 11 are higher compared with the other ties. This is reasonable because the reaction forces in this range are either just under a wheel load or between two wheel loads where superposition had a greater effect. Also the reaction force is about 9.8 kN (2200 lbs) at tie number 21, and this value is approximately the same as the unloaded condition. So it could be
found that the effect of a bogie force at this position is very small. Since the tie spacing is 0.51 meter the distance between tie number 7 and 21 is 7.1 m. Then, the effective range of a 3-axle bogie loading is 14 m. The center wheel load was considered a symmetrical axis where maximum values were observed.

For the numerical analysis portion of this project, force boundary conditions are necessary, which are obtained from the actual measurements. Therefore results from numerical analysis and from actual measurements are comparable. To find the force boundary condition, a function describing the loading distribution along the rail was made by performing the following steps: 1) Determine the function shape - by taking the observation of measured data, a sinusoidal function was chosen. 2) Normalize load amplitudes - divide every value by the maximum value, which gives values between 0 and 1. 3) Normalize the load distribution distance - since a sine or cosine function was chosen as the target function, the load distribution distance should be normalized between 0 and 2π. 4) Derive load distribution function - the proposed load distribution function was given in equation 3.1,

\[ q_i = A \sum_{i=0}^{14} (0.5\cos\left(\frac{i\pi}{7}\right) - 0.5) \quad (3.1) \]

where, \( q_i \) is the calculated force at point \( i \); \( A \) is the amplitude (maximum force); and \( i \) is the nodal point number.

To verify the assumption on the load distribution of equation 3.1, the normalized data is plotted in Figure 3.5 together with a standard cosine function. Note that although the two curves do not match perfectly, they are still acceptable.
Note that the load distribution function given by equation 3.1 is based on a bogie containing three axles. However, the bogie used in numerical analysis is composed of two axles. Therefore, equation 3.1 cannot be applied to this case. It was necessary to find the load distribution function for each wheel load, so every bogie containing a different number of wheels could be simulated by applying the superposition principle.

By observing equation 3.1 and the bogie it represents, it could be found that this cosine curve was caused by three equal loads with an equal spacing of 2 meters. Previous research showed that the single wheel load distribution curve was a cosine curve (Kalinski, 1999). By using this finding, the original load distribution curve given by equation 3.1 could be decomposed to three same
cosine curves except each curve had a unique phase angle. The expression for a single wheel load distribution curve is given in equation 3.2.

\[ q_i = \frac{A}{2.31} \sum_{i=0}^{10} (0.5\cos(\frac{i\pi}{5}) - 0.5) \]  

(3.2)

where, variables have the equivalent meaning as in equation 3.1.

Figure 3.6 shows the superposition of three single-wheel loads. As shown in this figure, three red solid cosine curves represent the single wheel load distribution curve corresponding to each wheel load in a bogie containing three axles and the blue solid curve represents the superposition of three red curves. It can be seen that superposition produced a load distribution that compare very well to the bogie load distribution curve shown as a black dashed curve in this figure.

![Figure 3.6 Superposition of Single Wheel Load](image-url)
In-track pressure measurements by hydraulic earth pressure cells

Pressure measurements under the tie were also conducted by using hydraulic earth pressure cells. A recent study revealed that the ratio between the diameter of the hydraulic earth pressure cell and a maximum granular particle size should be 5 or greater to get an accurate result (Miura, Otsuka, Kohama, Supachawarote and Hirabayashi, 2003). Since this requirement was satisfied, the results are acceptable. Similar measurements were performed previously for determining the stress distribution in HMA trackbeds and results regarding HMA trackbed performance were derived (Rose, Li and Walker, 2002). However, previous measurements were mainly focused on the top of the HMA layer whereas, pressure distribution and amplitude under the tie were favored in current projects since it would be used as the inputting stress functions for numerical simulation. Measurements were made at Conway (the same railroad track as the Tekscan measurements were made) and at the Transportation Technology Center Inc. (TTCI) test track. Test results are shown in Figure 3.7 and Figures 3.8 and 3.9, respectively.

Predicted pressures by KENTRACK

KENTRACK (Huang, Lin, Deng and Rose, 1984), a computer program designed for determining the pressure in a railroad track under the static axle loads and predicting the service life of HMA and subgrade in terms of annual gross tons, was utilized to confirm the pressure measured under the tie with hydraulic earth pressure cells.
Figure 3.7 Dynamic Compressive Stress Measurement at Conway, Kentucky
Figure 3.8 Dynamic Compressive Stress Measurement at TTCI

Figure 3.9 Subgrade Stress Comparison for Different Trackbeds Measured at TTCI
In previous research, the validity of KENTRACK computer program was proven by two methods. One way was to compare results with values generated by other similar computer programs such as GEOTRACK (Chang, Adegoke, and Selig, 1980). The other way was to compare KENTRACK results with in-track test measurements. Most recent work was done at TTCI (Li, Rose and Lopresti, 2001).

Both HMA and ballast trackbeds were considered when using KENTRACK computer program for predicting the pressures. The results are shown in Figure 3.10. The figure shows the peak pressure was beneath the rail seat for both HMA and ballast trackbed, but a larger peak value was found in HMA trackbed (438.2 kPa versus 317.2 kPa). Relative to the calculation of peak pressure under the tie, an equation was suggested by American Railway Engineering Association (AREA, 1975), as follows:

\[ P_a = \frac{2 \times P \times (1 + IF) \times (DF/100)}{A} \]  \hspace{1cm} (3.3)

where, \( P_a \) is the pressure at base of tie; IF is the impact factor in percent; DF is the distribution factor in percent; A is the area of tie and P is wheel loading in pound force. The value given by AREA equation was favorable to the result from KENTRACK based on ballast trackbed. Moreover, details about the comparison between in-track measurement results and KENTRACK predictive values were recorded in related reference (Rose, Su and Twehues, 2004).
In-track measurements provide the information about understanding the stress distribution in the track structure and they are particularly important for numerical analysis. Inputting stress functions were determined based on in-situ pressure test and computer prediction results.
Chapter Four

In Situ Tests

In situ geophysical tests were performed at the Conway, Kentucky site in order to determine the railroad subgrade properties, such as number of layers, layer thickness, shear modulus of soil, p-wave velocity and shear wave velocity of soil. The site properties and material properties provided model parameter values for the numerical simulation of ground vibration. There are two categories of in situ methods for measurement of dynamic soil properties: one category is toward measurement of low-strain properties and the other category is toward measurement of high-strain properties. For most railroads, stress waves generated due to train-track vibration propagate and radiate in the soil layers. Wave propagation generally belongs in the low-strain case. This has been proven by previous research work on testing the railroad subgrade soil (Rose, Brown, and Osborne, 2000). A subgrade composed of very soft soil would produce high-strain deformation in localized areas near the rail structure. The remaining subgrade would still respond in the low-strain range. Therefore, only low strain tests were employed in this project. They included the Spectral Analysis of Surface Waves (SASW) method and the seismic refraction method.

*Spectral Analysis of Surface Waves*

SASW (Heisey, Stokoe and Meyer, 1982; Nazarian and Stokoe, 1983; Stokoe, Wright, Bay and Roesset, 1994) has been frequently used for characterizing dynamic soil properties. The method applies an impulsive load to
the surface and monitors the propagation of the wave at two vertical receivers located at the ground surface, then a spectral analysis of the two acquired signals is used to find the variation of the surface wave velocity with frequency or wavelength.

In this test, the system consisted of two 1-Hz geophones as receivers and a 5.5 kg (12 lbs) X-KS sledgehammer as well as a 27 kg (60 lbs) mass as the source generators. Geophones were set into two half-inch deep holes and were wrapped by some crushed rocks to provide horizontal restraint. Since the substructure of the subgrade under the track is primarily concerned, geophones were laid just next to the track. Two geophones were attached to a dynamic signal analyzer with disk drive by wires. Acquired data can be either processed by the analyzer or saved to the disk. Figure 4.1 illustrates the equipment layout for the SASW test.

\[ d = 2m, 4m, 8m \text{ respectively} \]

**Figure 4.1 Equipment Arrangement for SASW Test**
Two geophones were equally spaced on the one side of the source as shown in Figure 4.1. The spacing d is determined by two factors: signal strength and recognition distance. Smaller d is preferred for better signal for the same input energy because the energy is dissipating with the increasing of travel distance. However, using larger d increases the investigation area, which requires more input energy for keeping the same signal strength. In this research, three spacings were chosen and they were 2, 4 and 8 meters. For 2 and 4-meter spacing cases, a sledgehammer was used as the inputting source and for the 8-meter spacing case, a weight drop was used to provide a quality signal. Soil particle velocities were measured in terms of amplitude (mV) versus time (s). A Fourier transform (Bracewell, 2000) was next used to convert the measured data into the frequency domain in order to separate the wave amplitude and the phase angle. The phase angle difference was calculated based on the transfer function which is defined as:

\[ H(f) = \frac{Y(f)}{X(f)} \]  \hspace{1cm} (4.1)

where, H(f) is transfer function, X(f) is the response function acquired by the first geophone after Fourier transform, and Y(f) is the response function acquired by the second geophone after Fourier transform. Then, knowing the distance between the source and the receiver, surface wave velocity or wavelength could be calculated using equation (4.2),

\[ V_R = \frac{2\pi}{\Delta \Phi} fR \]  \hspace{1cm} (4.2)
where, $V_R$ is Rayleigh wave phase velocity, $\Delta \Phi$ is the unwrapped phase difference, $R$ is the distance between two receivers and $f$ is the frequency.

SASW testing has an objective to evaluate the shear wave velocity profile of subsurface because Rayleigh wave, an attribute of SASW measurement, whose velocity is primarily affected by shear wave velocity and only slightly affected by compression wave velocity and Poisson’s ratio. Field acquired data were interpreted and analyzed in three procedures: 1) interactive masking; 2) determination of the representative experimental dispersion curve; and 3) forward modeling (or inversion analysis). During these three processing steps, a software program “WinSASW” was used (Joh, 1992).

Interactive masking is one of the most important steps in interpretation and analysis of raw data. It eliminates the undesirable data and assigns a jump number of the wrapped phase spectrum. The masked phase spectrum is unwrapped to calculate the phase velocities. Figure 4.2 shows the calculated phase angle difference that resulted from the forward 4-m spacing tests. It should be noted that the phase spectrum was not smooth at some frequency ranges and this is because of the non-uniformity of the ground layers, which reduces the collected data quality. Therefore, these low-quality data should be masked out. However, it should be noted that the interactive masking step is human experience based operation. Different people may have different judgement. In WinSASW program, only fundamental mode of Rayleigh wave data was used and higher modes were dropped.
After finishing the interactive masking step, phase velocities for wavelength or frequency, called phase velocity dispersion curve, can be calculated by applying equation 4.2. Since the dispersion curve was determined experimentally in the field, it is called experimental dispersion curve. When a series of SASW tests were performed at several different receiver spacings, the dispersion curve for each individual receiver spacing can be combined to form a complete dispersion curve for the test site. This complete dispersion curve is called experimental composite dispersion curve. In this research, the experimental composite dispersion curve was derived from 2, 4 and 8-m spacing tests and is shown in Figure 4.3 as the dashed line.

To determine the structure of subgrade, it is necessary to assign some initial values to the program for getting a theoretical dispersion curve. If the theoretical dispersion curve matches the experimental dispersion curve, then the input earth model is a reasonable solution. However, if the theoretical dispersion curve does not match the experimental curve, initial values should be adjusted until the two dispersion curves converge. Inputting parameters include shear wave velocity, Poisson’s ratio, total number of layers, layer thickness and soil unit
weight. The method used in WinSASW for generating a theoretical dispersion curve from a given site is termed forward modeling. This method consists of solving wave propagation equations in a two-dimensional multi-layer continua by employing dynamic stiffness matrix proposed by Kausel and Roesset (1981), and Kausel and Peek (1982). The results include a site soil profile given in Table 4.1 and a theoretical dispersion curve resulted from this soil profile shown in Fig. 4.3 as dotted line in terms of wave velocity versus wavelength.

Figure 4.3 Wave Velocity versus Wavelength for Site at Conway, KY

(1 fps = 0.3048 m/s, 1 ft = 0.3048 m)
Table 4.1 Site Soil Profile at Conway, Kentucky

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>Thickness (m)</th>
<th>Shear wave velocity (m/s)</th>
<th>Poisson’s ratio</th>
<th>Unit Weight (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.61</td>
<td>91</td>
<td>0.25</td>
<td>1922</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>335</td>
<td>0.25</td>
<td>2002</td>
</tr>
<tr>
<td>3</td>
<td>Half-space</td>
<td>2000</td>
<td>0.25</td>
<td>2002</td>
</tr>
</tbody>
</table>

The advantages and disadvantages of SASW test can be summarized as follows:

Advantages: 1) SASW is an in situ test method that can be performed quickly. 2) SASW is a non-destructive test method. 3) Compared to borehole test, SASW is an inexpensive test method. 4) The change in effective stress due to ground water fluctuations may be detected by repeating SASW tests at different times since shear modulus is stress dependent.

Disadvantages: 1) The determination of subsurface in SASW is by comparing theoretical dispersion curve to experimental dispersion curve. Therefore, the solution may not be unique from this inverse solution scheme. 2) Many SASW solutions are based on two-dimensional forward modeling, which only considers the fundamental mode shape of Rayleigh wave, drops off the higher mode shapes of Rayleigh wave. It is not accurate at the frequency range above “cut-off” frequency. Three-dimensional modeling is time consuming.
Seismic Refraction Test

A seismic refraction test is frequently used in earthquake engineering application and it measures the travel time of compressional wave (p-wave) or shear wave (s-wave) to determine the p-wave or s-wave velocity in terms of the distance between sources and geophones. This test is effective for test sites where wave propagation velocity increases with depth.

In any seismic refraction investigation, energy is artificially created via hammer impact, shotgun, weight drop, explosives, or any source that is able to produce any amount of energy at or near the surface, which then propagates through the different subsurface layers as body waves (Sheriff and Geldhart, 1989). There are two types of body waves: p-waves and s-waves. Traditionally, p-waves have been exclusively used in seismic investigations because of their high resolving abilities at depth, and because these high frequency waves are easily generated. However, p-waves can attenuate very rapidly depending on the medium of propagation, resulting in poor seismic coherency (Burger, 1992).

Another choice for this research is measuring shear wave, which is far superior for imaging near-surface structures because it is a “framework wave” (i.e., not affected by the degree of water saturation) as opposed to the fluid sensitive p-waves. There are two main types of shear wave, the SH (horizontally polarized shear wave) and the SV (vertically polarized shear wave). The SH-wave is the preferred phase because it is easier to identify, unlike the SV-wave, because of the lack of mode conversion at the refraction boundary.
The seismic refraction test was conducted at Conway, Kentucky, near the SASW test location. Data acquisition equipment included a 48-channel Geometrics StrataVisor seismograph and 48 Mark products, and 30-Hz, horizontally-polarized geophones with 7.5-cm spikes. The seismograph is a 24-bit system with an instantaneous dynamic range of 115 db that stores data on an internal hard drive. Experience suggests a spacing of 2-m should be used. The power source used in this study was a section of steel H-pile beam struck horizontally with a 1.4 kg hammer shown in Fig. 4.4. This is the most effective power source for imaging near surface structures. The steel H-pile has a hold-down weight of approximately 70 to 80 kg, including the weight of the hammer swinger and the H-pile section. The flanges of the H-pile are placed and struck perpendicular to the geophone spread for SH-mode generation. Polarity reversals and impacts of the sledgehammer on both sides of the energy source are recorded to ensure the correct identification of the SH-wave. The total length of the spread is 94 meters, with geophones in two inline-spreads spaced at 2-m intervals. The hammer blows were stacked seven times per shot point. This is done to capture desirable amplitudes at the site. The configuration is shown in Fig. 4.5. An on site view of testing is shown in Fig. 4.6.
Figure 4.4 Some Equipment Used for Seismic Refraction Test (From left to right:

4.5 kg hammer, Steel H-pile, and Horizontally polarized geophone)
Figure 4.5 Configuration of Seismic Refraction Test (Relative orientation of spread, sensor, and hammer swing. Steel H-pile and Horizontally-polarized geophone are located bottom right)
Figure 4.6 On Site View of Generating Inputting Source

Totally, three locations were selected as the location for the source generation and they are marked as shot location A, B, and C as shown in Figure 4.7. Two of three shot positions (marked as Shot A and Shot C) were placed at a 2-m near offset from the geophone array, and the third position (marked as Shot B) was at the mid point of the geophone array. Raw data for shot A, B and C are shown in Figures 4.8a, b, and c, respectively. To have a better view, a signal trace balancing termed “Automatic Gain Control” (AGC) was applied. AGC modified data for shots A, B, and C are shown in Figures 4.9a, b, and c respectively.

A software program named “SIP” was utilized to process the data to determine the number of layers, layer thickness and the shear wave velocities for each layer. Results are printed and shown in Table 4.2 for summarizing layer number and shear wave velocities, and Figures 4.10 for showing arrival time and 4.11 for showing depth model. The principal of analyzing data in “SIP” can be
found in Kramer (1996). For the generation of site depth model, the method used in SIP is generalized reciprocal method. This method was given by Palmer (1980 and 1981). For utilizing generalized reciprocal method, the following information should be provided (USACE, 1995):

1) An arrival time from the same refractor from both directions at each geophone.
2) A reciprocal time for energy traveling on that refractor.
3) A closely spaced set of geophones.
Figure 4.7 Three Shot Positions in Seismic Refraction Test
Figure 4.8a Raw Data Collected from Shot A
Figure 4.8b Raw Data Collected from Shot B
Figure 4.8c Raw Data Collected from Shot C
Figure 4.9a AGC Modified Data Collected from Shot A
Figure 4.9b AGC Modified Data Collected from Shot B
Figure 4.9c AGC Modified Data Collected from Shot C
Table 4.2 Shear Wave Velocity Analysis for Site at Conway, KY

<table>
<thead>
<tr>
<th>Shot Position</th>
<th>S-wave velocity for Soil</th>
<th>Average S-wave velocity for Soil</th>
<th>S-wave velocity for Bedrock</th>
<th>Average S-wave velocity for Bedrock</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>417 m/s</td>
<td>337 m/s</td>
<td>2125 m/s</td>
<td>2126 m/s</td>
</tr>
<tr>
<td>B</td>
<td>313 m/s</td>
<td></td>
<td>1720 m/s</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>323 m/s</td>
<td></td>
<td>2820 m/s</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.10 Modeling Result for Arrival Time of All Geophones
Summary and Results Comparison

Results from the seismic refraction test show that there are two layers at this site: a soil layer on the top and a bedrock layer on the bottom. The average shear wave velocities are 337 m/s for soil and 2126 m/s for bedrock. Furthermore, from the depth model of this site, it can be found that the soil layer thickness is approximately 10 meters. Also, it should be noted that the boundary of soil and bedrock is nearly flat (with 1.5-m variance) over the 94-m test range (from position 2-m to 96-m in Figure 4.11). Thus, it will be reasonable to assume a flat boundary between soil and bedrock in the numerical analyses.
A comparison of SASW and seismic refraction test results shows that there are three layers in the SASW test result, one layer less than the seismic refraction test. However, it should be noted that when making the SASW test, geophones were set on the ground just next to the track, which is about 2 to 3 meters beside the track. For the seismic refraction test, the geophone spread was set about 15 to 20 meters away the track as shown in Figure 4.6. The location selected for making SASW test was covered by a thin layer of loose ballast. Therefore, the results from the SASW test were reasonable since it contains this thin layer relative to the lack of ballast underlying the seismic refraction test. Also, observing the layer thickness, SASW gives a 10-meter thickness for the second layer that corresponds to the first layer in the seismic refraction test.
Chapter Five

Laboratory Tests

Introduction

In order to characterize the response of a railroad track site under dynamic train loading, measurement of the dynamic soil properties is necessary. Also, for numerical analysis, material properties are required as input parameters. Therefore, soil properties, including Young's modulus (E), shear modulus (G), density (p), damping ratio (D), and Poisson's ratio (ν), should be determined. This work is commonly done in the laboratory. For laboratory tests, small strain (strain below 0.001%) tests include resonant column (ASTM, 2003; Drnevich, Hardin and Shippy, 1978), ultrasonic pulse (Lawrence, 1963; Nacci and Taylor, 1967), and piezoelectric bender element (Dyvik and Madshus, 1985) methods. In this project, the free-free resonant column test method was used.

Since a train induces small strains in most cases, only small-strain tests were conducted. In this small-strain range, the behavior of soil is generally considered as elastic, so constitutive relationships based on elastic behavior can be applied. For the typical case, five dynamic soil parameters are of concern (E, G, p, ν, and shear wave velocity (v_s)). Knowing three of these, the other two can be calculated.

The device used to perform free-free resonant column testing is illustrated in Fig. 5.1. For each specimen, two types of tests were conducted by doing torsional and longitudinal tests for obtaining v_s and compression wave velocity (v_p), respectively. For the torsional test, two accelerometers were fixed in the
same circumferential direction (either both clockwise or both counterclockwise) at one end cap near the edge. At the other end cap, a Ledex H-1079-032 rotary solenoid was fixed at the center. It was controlled by a function generator and an amplifier to generate torsional excitation. The mass of the solenoid is 65.5 grams. For the longitudinal test, one accelerometer was fixed to the center of one end cap and oriented axially. A Ledex 195207-224 tubular solenoid was used instead of the rotary solenoid to generate longitudinal excitation. The mass of this solenoid (core and coil) is 241.6 grams. The detailed parameters used for the HP3314 function generator and the HP 6825A power supply (amplifier) are listed in Table 5. Details of principles of free-free resonant column test and soil property calculation based on test results can be found in related references (Drnevich, Hardin and Shippy, 1978).

![Figure 5.1 Device Used for free-free Resonant Column Test](Kalinski and Thummaluru, 2005)
Table 5.1 Parameters used for generating voltage pulse for solenoids

a) For HP3314 Function Generator

<table>
<thead>
<tr>
<th></th>
<th>N cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>N cycle</td>
</tr>
<tr>
<td>Frequency</td>
<td>20 Hz</td>
</tr>
<tr>
<td>Amplitude</td>
<td>1.00 V</td>
</tr>
<tr>
<td>Offset</td>
<td>-0.52 V</td>
</tr>
<tr>
<td>Symmetry</td>
<td>50%</td>
</tr>
<tr>
<td>Phase</td>
<td>90 degrees</td>
</tr>
<tr>
<td>N</td>
<td>1</td>
</tr>
<tr>
<td>Function (sine, square, triangle)</td>
<td>Sine wave</td>
</tr>
<tr>
<td>Trigger</td>
<td>Manual</td>
</tr>
</tbody>
</table>

b) For HP 6852A Power Supply/Amplitude

<table>
<thead>
<tr>
<th></th>
<th>Variable Gain Amplifier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode</td>
<td>Variable Gain Amplifier</td>
</tr>
<tr>
<td>Amplification</td>
<td>X4</td>
</tr>
<tr>
<td>Voltage Dial Position</td>
<td>Full Clockwise</td>
</tr>
<tr>
<td>Current Dial Position</td>
<td>Full Clockwise</td>
</tr>
</tbody>
</table>

Generally, there are two methods for determining the material damping ratio (D). One of them is the free vibration decay method and the other one is half
power bandwidth method (Richart, Hall and Woods, 1970). For the first method, D is calculated based on the amplitude logarithmic decrement. For the second method, D is estimated using the amplitude spectrum of the free vibration decay record as:

$$D \approx \frac{f_2 - f_1}{2f_n}, \quad (5.1)$$

where, $f_1$ and $f_2$ are frequencies at an amplitude corresponding to 0.707 times the maximum amplitude, and $f_n$ is the frequency at the maximum amplitude. In this research, the half power bandwidth method was used.

**Testing of Ballast Specimen**

Ballast was collected from the railroad site and transported to the laboratory for testing. Since ballast consists of cohesionless gravel, it has no strength until confined. A cylindrical mold with a length of 24.76 cm and diameter of 10.2 cm was used to construct reconstituted specimens. Specimens were reconstituted inside a latex membrane, and pore vacuum was used for effective confinement. The confining stress was set as 37 kPa and 64 kPa to simulate different trackbed stiffness. Above values can be considered representative of a stiff trackbed. However, it should be noted that due to the variation of ballast trackbed, the highest confining stress may be several times higher, such as in cemented ballast trackbed, and the lowest confining stress may only be a few kPa, such as newly placed, uncompacted ballast. It is expected that stiffness
should increase with increasing confining stress. Test results are summarized in Table 5.2.

Table 5.2 Resonant Column Test Results for Ballast

<table>
<thead>
<tr>
<th></th>
<th>37</th>
<th>64</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vacuum Level (kPa)</td>
<td>37</td>
<td>64</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>1750</td>
<td>1750</td>
</tr>
<tr>
<td>Longitudinal Resonant Frequency (Hz)</td>
<td>487</td>
<td>500</td>
</tr>
<tr>
<td>Compressional Wave Velocity (m/s)</td>
<td>253.1</td>
<td>259.8</td>
</tr>
<tr>
<td>Torsional Resonant Frequency (Hz)</td>
<td>300</td>
<td>325</td>
</tr>
<tr>
<td>Shear Wave Velocity (m/s)</td>
<td>156.4</td>
<td>169.4</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.31</td>
<td>0.18</td>
</tr>
<tr>
<td>Shear Modulus (MPa)</td>
<td>42.8</td>
<td>50.2</td>
</tr>
<tr>
<td>Average Damping</td>
<td>2%</td>
<td></td>
</tr>
</tbody>
</table>

Testing of Hot Mixed Asphalt Specimen

Hot Mixed Asphalt (HMA) was used as a trackbed sublayer material in the railroad at Conway, Kentucky. A HMA core was recovered and sealed in a plastic bag at the site. Since HMA is very stiff and confining stress has little effect on stiffness, the HMA core was tested without confinement. The rotary solenoid and accelerometers were glued to the HMA core directly. The dimension of the tested HMA core was 19.1 cm for length, and 10.2 cm for diameter. Note that HMA properties such as elastic modulus, viscosity, damping etc., will change based on asphalt content and aggregate gradation. Also, some external factors like
temperature, oxygen and age will affect HMA properties. However, due to the cover of ballast, the environment of HMA is superior to the one of highway pavement with respect to degrading factors. This has been confirmed by a previous study based on predicted HMA service life and actual observations by Rose, Su and Long (2003). In a separate study by Li, Rose and Lopresti (2001), the HMA and air temperatures were monitored simultaneously. Results, shown in Fig. 5.2, indicate that the HMA temperature fluctuated less than air temperature.

![Figure 5.2 Variations of Air and HMA Temperatures (Li, Rose and Lopresti, 2001)](image)

Free-free resonant column testing of HMA was performed in the laboratory with a measured temperature of 15 degrees (celsius), which approximates the annual average temperature for the HMA layer of railroad track in Conway, Kentucky. Test results are summarized in Table 5.3.
Table 5.3 Resonant Column Test Results for HMA

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2407</td>
</tr>
<tr>
<td>Longitudinal Resonant Frequency (Hz)</td>
<td>7325</td>
</tr>
<tr>
<td>Compressional Wave Velocity (m/s)</td>
<td>2793.3</td>
</tr>
<tr>
<td>Torsional Resonant Frequency (Hz)</td>
<td>4625</td>
</tr>
<tr>
<td>Shear Wave Velocity (m/s)</td>
<td>1766.8</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
</tr>
<tr>
<td>Shear Modulus (MPa)</td>
<td>7514</td>
</tr>
<tr>
<td>Average Damping</td>
<td>4.2%</td>
</tr>
</tbody>
</table>

**Summary**

The free-free resonant column test method described herein provides an alternative for measurement of geologic material properties such as G and D. The half power bandwidth method was used instead of the tree vibration decay method for calculating the damping (D) since a reliable result can be expected when the quality of the time-domain records is poor.

According to test results of ballast (Table 5.2), vacuum level (confining stress) has a minor effect on the damping and shear wave velocity within the stress range tested. The average shear wave velocity of two vacuum levels will be used in the numerical analysis.
Chapter Six

Numerical Simulation of Ground Vibration

Introduction

Numerical simulation was carried out in this project for two reasons: 1) comparison with the in-track test results, and 2) further simulating the ground vibration under train loads as different types of material were used in trackbed. The Fast Lagrangian Analysis of Continua in three Dimensions (FLAC\textsuperscript{3D}) software was used (FLAC\textsuperscript{3D}, 2002). This finite difference program has been used for similar studies and proved as an effective key to understand and predict wave propagation in soils induced by moving trains (Andreasson, 1998; Kalinski, 1999). MATLAB software was used for generating the input stress functions for FLAC\textsuperscript{3D}, and for reducing the output data from the numerical simulation. Details regarding the track model, rail roughness simulation, and forcing function generation are discussed in this chapter.

FLAC\textsuperscript{3D} Model Description

The overall FLAC\textsuperscript{3D} model was constructed in two portions: 1) track model on the top and 2) ground model on the bottom. Several factors determined the construction of the FLAC\textsuperscript{3D} model. First, the railroad track is symmetrical with respect to its centerline, which means only half of the track needs to be modeled. Second, the length of the model should approach or exceed the length of a passing train. As shown in Fig. 6.1, the total length of the train, composed of two
engines and one coal hopper, was 63 m. However, considering the computer memory limit, 60 m was selected as the model length. The width of ground model was selected as 30 m; this dimension was large enough to monitor wave propagation consistent with the in-track measurements. Total depth of ground model was chosen in terms of the substructure of railroad track site. Based on results from SASW and seismic refraction tests, there were two layers at the Conway site, excluding the shallow ballast and soil mixture layer. The thickness of first layer soil is 10 m and the thickness of second layer (bedrock), consider as half-space. In the numerical model, bedrock is modeled by using fixed boundary condition. Also, the ballast layer is incorporated into the embankment, which sits on the shallow layer in the numerical model. Thus, the depth of model was set as 10 m. The embankment model was 60 m long, 3 m wide, and 0.5 m thick, and was constructed over the ground model.

![Figure 6.1 Wheel Positions for Running Train](image)

The model mesh size was selected based on the balance of accuracy and computation time. A finer grid size is desirable for improved accuracy, but a coarser grid may be necessary to reduce computation time. As a common criterion, one-tenth to one-eighth of the minimum wavelength should be used for
dynamic wave propagating modeling (FLAC\textsuperscript{3D}, 2002). Based on the experimental SASW test results, a dispersion plot based on the profile at Conway is shown in Fig. 6.2 in terms of wavelength versus wave frequency.

A smooth rail surface without any roughness or corrugation is assumed. The train speed used for the simulation was 22.4 m/s (80.5 km/h). Given the relationship between loading frequency and train speed in equation 6.1, the loading frequency can be obtained by taking the wavelength, $\lambda$, as 10 m, as discussed in Chapter 3. In this case, the calculated loading frequency, $f$, was 2.24 Hz according to equation 6.1.

$$V = \lambda f \quad (6.1)$$

where, $V$ is the train speed, $f$ is the loading frequency, and $\lambda$ is the wavelength.
As shown in Figure 6.2, the wavelength corresponding to a load frequency of 2.24 Hz is approximately 110 m, so the maximum grid spacing should be approximately 13.7 m. Therefore, choosing a grid spacing of 0.5 m is adequate to simulate the wave propagation in soil. However, since this analysis is based on the smooth rail case without any rail roughness, and rail roughness affects the grid spacing, the grid spacing must also be checked later to make sure it meets the requirement caused by rail roughness.

Numerical simulation was performed in two steps: 1) achieving static equilibrium and 2) measuring the response of the model to dynamic excitation. Different types of boundaries were modeled for each phase. For the static analysis, only gravity forces were involved, and roller boundaries were imposed at the four sides of the ground model. At the bottom of the model, fixed boundary conditions were used to simulate bedrock. The nodes representing the track were fixed laterally and allowed to move in the vertical direction only. For the dynamic analysis, forces generated by the passing train would be added to the model. In reality, the ground is an unbounded half-space. However, it is impossible to use an infinitely wide, infinitely long model in a numerical simulation. To simulate the unbounded case, quiet boundaries were applied at the sides of the model with the exception of the symmetric plane, which had roller boundaries. To use quiet boundaries, dashpots were set at normal and tangent directions of the boundaries to absorb the energy carried by waves and to minimize reflections. Details about this viscous dashpot based absorbing boundary are given by
Lysmer and Kuhlemeyer (1969). Figures 6.3a and 6.3b illustrate the details of the model.

Material properties were assigned to the model, and varied at different regions. According to SASW and seismic refraction test results, there was only one soil layer at the Conway site. Thus, the ground model was composed only by one soil layer. Shear modulus was determined from the SASW test.
For the track model, two types of trackbed were simulated: HMA or ballast. Corresponding properties were determined by laboratory testing. It should be noted that isotropic linear elastic behavior was assumed for all materials.

Numerical simulation in FLAC\textsuperscript{3D} consisted of two steps. Establishing static equilibrium was the first step. After static analysis, forces generated by the train were imparted to the model for the dynamic analysis. The constitutive relationship for soil is nonlinear rather than linear. To achieve a reasonable result, an equivalent linear iterative approach was adopted. By applying this method, maximum shear modulus and minimum damping ratio were used for the initial analysis. Shear strain was calculated based on the particle displacement obtained from the initial analysis. According to the shear strain, shear modulus and damping were revised based on related shear modulus reduction and shear strain relationships (Vucetic and Dobry, 1991). The revised shear modulus and damping ratios were applied to the model for a second analysis. The model was
iteratively repeated until differences between computed shear modulus and damping ratio in successive iterations fell below a predetermined value. However, as discussed in Chapter 7, the calculated shear strain in most of the soil model is below 0.001%. Therefore, the model is not actually updated and the soil deformation is assumed as being in its elastic range. A summary of the parameters used to model the track and ground in the analysis are listed in Tables 6.1a and 6.1b, respectively.

Table 6.1a Parameters Used for Track Model

<table>
<thead>
<tr>
<th>Trackbed Type</th>
<th>Item</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Density (kg/m³)</td>
<td>1750</td>
</tr>
<tr>
<td>Ballast</td>
<td>Shear Modulus (Mpa)</td>
<td>50.2</td>
</tr>
<tr>
<td></td>
<td>Bulk Modulus (MPa)</td>
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</tr>
<tr>
<td></td>
<td>Thickness (m)</td>
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</tr>
<tr>
<td></td>
<td>Damping Ratio</td>
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</tr>
<tr>
<td>HMA</td>
<td>Density (kg/m³)</td>
<td>2407</td>
</tr>
<tr>
<td></td>
<td>Shear Modulus (MPa)</td>
<td>7513</td>
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<tr>
<td></td>
<td>Bulk Modulus (MPa)</td>
<td>12522</td>
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<tr>
<td></td>
<td>Thickness (m)</td>
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<tr>
<td></td>
<td>Damping Ratio</td>
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</table>
### Table 6.1b Soil Parameters Used for Initial Simulation

<table>
<thead>
<tr>
<th>Layer Name</th>
<th>Item</th>
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<tbody>
<tr>
<td>Soil Layer</td>
<td>Density (kg/m³)</td>
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</tr>
<tr>
<td></td>
<td>Shear Modulus (MPa)</td>
<td>225</td>
</tr>
<tr>
<td></td>
<td>Bulk Modulus (MPa)</td>
<td>375</td>
</tr>
<tr>
<td></td>
<td>Thickness (m)</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Damping Ratio</td>
<td>2%</td>
</tr>
</tbody>
</table>

**Simulation of Rail Roughness**

Rail roughness is one of the major track problems causing excessive vibration and noise. Much research has been carried out in addressing rail roughness (Smith, 1967; and Stone, Marich, and Rimnac, 1982), and many conclusions have been drawn, although the mechanism about the initiation and development of rail roughness is still unclear. Generally, two observations have been made regarding rail roughness. First, rail roughness can be described by a mathematical function. For a specific railroad line, especially for a track section that has only one type of train operating at a constant speed, this result is very credible. However, for most cases, a track is shared by different trains, all of which have different masses and speeds. In this case, the measured rail roughness is very difficult to characterize mathematically. Therefore, a second conclusion was drawn that rail roughness is a randomly generated signal that can be statistically regulated.
Regarding the shape of commonly encountered rail roughness, there are two types, referred to as short pitched and long pitched corrugation (Hay, 1982). Short pitched corrugation, with periods around 50 to 90 mm long or even longer or shorter, is often found in high-speed passenger lines. This kind of track is also named “washboard” or “roaring” track. For the long pitched corrugation, periods are around 200 to 600 mm long, and can be up to 3 m long. Long pitch corrugation is frequently found in freight railroad lines. Herein, only long pitched corrugation will be investigated.

After rail roughness values are determined, they can be used to develop the roughness induced acceleration, which is described in the previous section and shown in equation 6.1.

In this research, calculation of rail roughness induced forcing functions can be divided into five steps:

- Step 1: a random number generator was used to generate rail roughness amplitudes at 0.5-m increments along the rail. The rail roughness amplitude was limited to ±1 mm.

- Step 2: Fourier analysis was used to transform the randomly generated signals from the spatial domain to the wavenumber (spatial frequency) domain. In the wavenumber domain, an amplitude boost was made to increase amplitudes for all waves to the maximum amplitude. Then, a half-wavelength cosine curve was applied to cut the amplitude at different wavenumbers. The purpose of this is to limit the amplitudes at some undesired wavenumbers that
correspond to very long wavelengths. Phase angles are preserved without any change during this process.

- Step 3: inverse Fourier analysis was used to convert all the signals (roughness waves) back to the spatial domain. During the inverse Fourier transform process, phase angles for all waves were preserved without any change.

- Step 4: a zero-phase signal filter was set up to limit the passing band of waves in the spatial domain. Filtering criteria were expressed as functions of Nyquist wavenumbers. More details can be found in related references (Matlab, 2002).

- Step 5: amplitudes of filtered waves were scaled so that the absolute value of maximum amplitude was 1 mm.

During the above sequence, it is important to determine the passing band for the filter. Herein, a study regarding the statistical analysis of rail corrugation (Mair, Jupp, and Groenhout, 1982) was used as the reference for rail roughness selection. According to this paper, the wavenumber range of interest was from 0.5 to 2.5 cycles/m. As to the model used in this research, the grid spacing in the longitudinal direction was 0.5 m. Therefore, the Nyquist wavenumber can be calculated as:

$$k_{ny} = \frac{1}{2\Delta x}$$  \hspace{1cm} (6.2)

where, $k_{ny}$ is the Nyquist wavenumber, and $\Delta x$ is the grid spacing. The calculated Nyquist wavenumber corresponding to a grid spacing of 0.5-m is 1 cycle/m, so the wavenumber range that this model can simulate is from 0 - 1 cycle/m, where
0 cycle/m represents a “stationary shift”. It can be found that the wavenumber range simulated by this model is not identical to the one given by the above mentioned rail roughness study. However, to simulate a wavenumber of 2.5 cycle/m requires a grid spacing of 0.2 m. Higher wavenumbers correspond to shorter wavelengths, which are normally generated by high-speed trains. So in this research, only the low wavenumber range is of interest since only heavy axle, low speed trains travel on the railroad line at Conway.

Filter passing band was set as 0.3 to 0.9 cycle/m, which corresponds to a wavelength range from 1.1 to 3.3 m. Knowing train speed of 22.4 m/s and using equation 6.1, calculated loading frequencies are 6.71 and 20.12 Hz, respectively. From Figure 6.2, the wavelengths corresponding to these frequencies are 42 and 11 m, respectively. Therefore, a grid spacing of 0.5 meter should be adequate to simulate wave propagation. The rail roughness results are shown in Figure 6.4.
Figure 6.4 Generated Rail Roughness Function Curves

Figure 6.4 illustrates the rail roughness function in the spatial domain (upper plot), and in the wavenumber domain (lower plot). The red curve shown in Figure 6.4 represents the original data generated by the random number generator. As described previously, a half-wavelength cosine curve with maximum amplitude equaling the maximum amplitude of the original signal (the green curve) is applied in the wavenumber domain, and its result is referred to as whitened signal, which is shown in green. After applying inverse Fourier analysis, the whitened signal is transferred back to the spatial domain. Observing the whitened spatial-domain signal (green curve), it can be found that amplitudes at low wavenumbers are low. The reason for this is that a long wavelength signal is
not commonly encountered since generally maximum wavelength of observed roughness is about 3 m. Therefore, limiting amplitudes at low wavenumbers is necessary. Finally, a filter was applied in the spatial domain to limit the signal passing band between 0.9 to 1 cycles/m to minimize distortion and aliasing. The resulting signal is shown as a blue curve in both plots. From the wavenumber domain data, it can be found that the amplitude is reduced by about 60 dB at wavenumbers greater than 0.9 cycles/m.

**Generation of Forcing Function**

During the dynamic analysis, forces generated by forcing function to simulate the wheel load are necessary as the force boundary condition. The force generated by the train could be considered as the superposition of: 1) the static force due to the self-weight of train, and 2) the dynamic force due to the rail roughness. Rail roughness was considered as randomly generated with an amplitude limit between ±1 mm, and sampled at 0.5-m increment to match the grid spacing as previously described.

Since rail roughness value, y, was varied along the rail longitudinal distance, x, rail roughness could be expressed as a function of distance along rail, written as y(x). Then, taking the first and second derivatives with respect to x, y′(x) and y″(x) could be calculated. Assuming the bogie mass, m, had a perfect isolation from train body and was applied to the rail, the dynamic force could be expressed as a product of bogie mass and the acceleration of bogie mass, which could be expressed by the second derivative of rail roughness with respect to
time, \( t \), shown as \( y''(t) \). By applying the chain rule, dynamic force, \( F_{\text{dyn}} \), could be expressed as a function of mass, \( m \), and horizontal pulling force, \( f \):

\[
F_{\text{dyn}} = m\left(\frac{\partial^2 y}{\partial t^2}\right) = m\left(\frac{\partial^2 y}{\partial x^2}\right)\left(\frac{\partial^2 x}{\partial t^2}\right) = my''(x) = fy''(x)
\]  \hspace{1cm} (6.3)

After calculating the dynamic forces, the total forces generated by the train could be obtained by summing the dynamic and static forces. However, in FLAC\(^3\text{D}\), the required inputting force parameter is stress. So calculated forces must be converted into stress. Knowing the area, \( A \), and calculated static and dynamic forces, \( F_{\text{stat}} \) and \( F_{\text{dyn}} \), static and dynamic stresses, \( \sigma_{\text{stat}} \) and \( \sigma_{\text{dyn}} \), can be expressed as:

\[
\sigma_{\text{stat}} = \frac{F_{\text{stat}}}{A} \hspace{1cm} (6.4a)
\]

and

\[
\sigma_{\text{dyn}} = \frac{F_{\text{dyn}}}{A}. \hspace{1cm} (6.4b)
\]

Thus, total stress, \( \sigma_{\text{total}} \), can be expressed as:

\[
\sigma_{\text{total}} = \sigma_{\text{stat}} + \sigma_{\text{dyn}} = \frac{F_{\text{stat}}}{A} + \frac{F_{\text{dyn}}}{A}. \hspace{1cm} (6.5)
\]

Dividing equation 6.5 by equation 6.4a,

\[
\sigma_{\text{total}} = \sigma_{\text{stat}} \left(\frac{F_{\text{stat}} + F_{\text{dyn}}}{A}\right) = \sigma_{\text{stat}} \left(1 + \frac{F_{\text{dyn}}}{F_{\text{stat}}}\right). \hspace{1cm} (6.6)
\]

Equation 6.6 illustrates that total applied stress to the model is a function of static stress, static force, and dynamic force. Note that the term \( 1 + \frac{F_{\text{dyn}}}{F_{\text{stat}}} \)
equation 6.6 represents the dynamic effect of the moving train. Equations 6.4a and 6.6 reveal that only three parameters: $\sigma_{\text{stat}}$, $F_{\text{dyn}}$, and $A$, are needed to obtain $\sigma_{\text{total}}$. In Chapter 3, pressures had been tested in the field and calculated by computer program. Herein, the results obtained in Chapter 3 are utilized. Along the rail, the load distribution curve was given in equation 3.2. For the lateral direction (parallel to the tie), it was assumed that the load distribution was uniform, which was found to be 200 kPa. Note that this number was obtained based on the actual contact area between the tie and the underlying ballast. However, the grid in the FLAC$^3$D model cannot be that fine due to limitations of computation time and available memory. Thus, a simplification was adopted that each wheel load was distributed over a 15 m$^2$ area (1.5 m wide, 10 m long).

The generated stresses were applied to the FLAC$^3$D model at different grid nodes at different time to simulate the moving train. Figures 6.5 to 6.7 depict the loading history for either a constant point at various time or a constant time at various points under static, dynamic and total (static + dynamic) load.
Figure 6.5 Applied Static Stresses

a) At a Fixed Time \( t = 3.02 \) s

b) At a Fixed Location \( x = 30 \) m
a) At a Fixed Time \( t = 3.02 \text{ s} \)

b) At a Fixed Location \( x = 30 \text{ m} \)

Figure 6.6 Applied Dynamic Stresses
Figure 6.7 Applied Total (Static + Dynamic) Stresses

a) At a Fixed Time (t = 3.02 s)

b) At a Fixed Location (x = 30 m)
Finally, discussion of the damping system used for dynamic analysis in FLAC\textsuperscript{3D} is warranted. Rayleigh damping was used, and the critical damping ratio can be calculated based on any angular frequency in a multiple degrees-of-freedom system (Bathe and Wilson, 1976). The most important point in using Rayleigh damping in FLAC\textsuperscript{3D} is proper choice of predominant (center) frequency. For this case, the average wavenumber in rail roughness function is 0.75 cycle/m. Therefore, considering the train speed of 22.4 m/s, the predominant frequency for Rayleigh damping is 16.8 Hz.

FLAC\textsuperscript{3D} is a time explicit scheme based program and it uses finite difference method for solving the equation of motion. After obtaining new velocity from equation of motion, strain rate can be calculated and new stress can also be obtained by using material constitutive model. Obtained new stress will be cause the unbalanced force at the node thus new equation of motion should be solved. By repeating above procedure, problem can be solved. More information about this solving technique can be found in FLAC\textsuperscript{3D} (2002).
Chapter Seven
Analyses, Discussions, and Comparisons of In Situ Tests and Numerical Simulation Results

Introduction

Field tests and numerical simulation were discussed in the previous chapters. As previously described, test results were vertical particle velocity and acceleration in the time domain. To better understand the ground vibration, it is necessary to analyze the spectral content of the data. Fourier analysis was employed to view the data in the frequency domain.

Vibration Results from In Situ Tests

Numerous in situ tests were performed at different locations. However, only a major in situ test performed on August 7, 2003 at Conway is illustrated here. Details about the other tests and their results are presented in the Appendix. For the test at Conway, the layout of test equipment is shown in Figure 7.1. Two geophones were set on the ground surface beyond the ballast trackbed section and HMA trackbed section, respectively. Vertical particle velocity was recorded at the ground surface in the time domain, and converted into the frequency domain. Figure 7.2 shows the measured particle velocity in time-domain for ballast trackbed under the passing train speed of 64.4 km/h.
Figures 7.3 and 7.4 show the measured results for ballast trackbed and HMA trackbed under the passing train speed of approximately 77 km/h (21.5 m/s or 48 mph). Figures 7.5-7.8 show the same cases, except the passing train speeds were 64.4 km/h (17.9 m/s or 40 mph) and 48.3 km/h (13.4 m/s or 30 mph).
As illustrated in Figure 7.2, recorded vertical particle velocity is generally less than 0.2 cm/s. However, there are a few values that are up to 0.6 cm/s. The reason of this is due to a defective (flatspot) wheel, which causes abnormal acceleration and cannot represent the normal case.
Figure 7.3 Particle Velocity for Ballast Trackbed under Train Speed of 21.5 m/s

Figure 7.4 Particle Velocity for HMA Trackbed under Train Speed of 21.5 m/s
Figure 7.5 Particle Velocity for ballast Trackbed under Train Speed of 17.9 m/s

Figure 7.6 Particle Velocity for HMA Trackbed under Train Speed of 17.9 m/s
Figure 7.7 Particle Velocity for Ballast Trackbed under Train Speed of 13.4 m/s

Figure 7.8 Particle Velocity for HMA Trackbed under Train Speed of 13.4 m/s
Observing Figures 7.3-7.8, it is found that there were peak values at certain frequencies on these measured curves. Assessing the effect of factors like train speed, wheel spacing and tie spacing, some interesting conclusions can be drawn, which are described in the following paragraphs.

The excitation frequency related to the train speed and tie spacing can be expressed as:

$$f_t = \frac{c}{l_t}$$  \hspace{1cm} (7.1)

where, \(f_t\) is the tie spacing effect frequency, \(c\) is the train speed and \(l_t\) is the tie spacing. Generally, for a given track, tie spacing ranges from approximately 0.5 to 0.65 m. Thus, the tie spacing effect frequency is proportional to the passing train speed. The importance of tie spacing frequency is that there will be a peak particle velocity observed at this frequency due to resonance. For example, in this case, the standard value for tie spacing was 0.51 m (20 in.). Therefore, for a passing train speed of 21.5 m/s (48 mph), 17.9 m/s (40 mph) and 13.4 m/s (30 mph), calculated tie spacing effect frequencies are 42, 35, and 26 Hz, respectively. Note that in reality, the tie spacing varies, although it has an average value of 0.51 m (20 in.). According to the field measurement, most tie spacing deviates by less than 2.5 cm (1 in.) from this value, and the maximum error was approximately 0.05 m (2 in.). In the field, it was observed that there was a local peak particle velocity value at a frequency approximately equal to the calculated tie spacing frequencies as shown in Figures 7.3 through 7.8.
Another interesting observation was that of the wheel spacing frequency, which had a similar effect as the tie spacing frequency. The wheel spacing effect frequency is calculated as:

\[
f_w = \frac{c}{l_w}
\]  

(7.2)

where, \(f_w\) is the wheel spacing effect frequency, \(c\) is train speed, and \(l_w\) is the spacing between wheels. The effect of wheel spacing effect frequency is similar to that of tie spacing effect frequency, where peak values could be observed. As shown in Figure 6.1, for a passing train, the spacing between wheels varied. Thus, for a given train speed, several wheel spacing frequencies exist. Table 7.1 lists the wheel spacing effect frequencies for the train shown in Figure 6.1 at different train speeds.

<table>
<thead>
<tr>
<th>Wheel Spacing (m)</th>
<th>21.5 m/s</th>
<th>17.9 m/s</th>
<th>13.4 m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>10.75 Hz</td>
<td>8.95 Hz</td>
<td>6.70 Hz</td>
</tr>
<tr>
<td>4</td>
<td>5.38 Hz</td>
<td>4.48 Hz</td>
<td>3.35 Hz</td>
</tr>
<tr>
<td>5</td>
<td>4.30 Hz</td>
<td>3.58 Hz</td>
<td>2.68 Hz</td>
</tr>
<tr>
<td>11</td>
<td>1.95 Hz</td>
<td>1.63 Hz</td>
<td>1.22 Hz</td>
</tr>
<tr>
<td>12</td>
<td>1.79 Hz</td>
<td>1.49 Hz</td>
<td>1.12 Hz</td>
</tr>
</tbody>
</table>

Table 7.1 Wheel Spacing Effect Frequencies
It should be noted that due to the different types of trains, the wheel spacing varied correspondingly. The wheel spacings listed in Table 7.1 are approximate values, and calculated wheel spacing effect frequencies may not exactly correspond to measured values. The data shown in Figures 7.3 through 7.8 indicate that peak values appear at most calculated frequencies. Compared to the particle velocity at tie spacing effect frequency, the particle velocity amplitude at wheel spacing effect frequency is smaller. Therefore, tie spacing has a larger effect on the energy distribution in the frequency domain than does wheel spacing. Similar work regarding tie and wheel spacing effect has been reported by Dawn and Stanworkth (1979), Melke and Kraemer (1983), and Heckl, Hauck, and Wettschureck (1996). Other factors that may affect the vibration may be related to the roughness of rail, the train’s natural vibration frequency (Krylov and Ferguson, 1994), and the bogie mass.

Vibration energy distribution is apparent in Figures 7.3 through 7.8. For the surface ground vibration, the frequency range of interest is below 50 Hz, as confirmed by Hall (2003). Note that the peak vibration particle velocity in three different train speed cases were nearly the same, which indicates that train speed does not have a large effect on ground vibration. This was also observed in previous research by Okumura and Kuno (1991). However, it should be noted that this is only true when the train speed is significantly less than the fundamental mode Rayleigh wave speed. When the train speed approaches the Rayleigh-mode speed, excessive ground vibration would be observed (Madshus and Kaynia, 1998a, 1998b).
Generally, a railroad track consists of several layers of different materials. Thus, the wave propagation speed in different layers will be variable. When train speed approaches the lowest wave propagation (shear or surface wave) speed in those layers, excessive track deformation is expected to occur. Herein, a simplified model shown in Figure 7.9 is used to illustrate this point.

As shown in Figure 7.9, a finite beam with a length of $L$ was used to simulate the rail. This is reasonable since under a single axle load, the observed rail deflection is negligible at some distance from the load. The beam was placed over a continuous foundation, which consisted of springs with stiffness $k$. This stiffness value can be easily determined by using Kerr’s method (Kerr, 2000).

For study of the dynamic vibration of this system, it is necessary to find the characteristic property of the vibration system, which is referred to as system free vibration, with an associated natural frequency. The equilibrium equation for the beam can be expressed as:
\[ EI \frac{\partial^4 y(x,t)}{\partial x^4} + m \frac{\partial^2 y(x,t)}{\partial t^2} + kx = 0 \]  

(7.3)

where, \( E \) is the elastic (Young's) modulus of the beam, \( I \) is the moment inertia of the beam, \( t \) is time, \( k \) is spring stiffness, and \( m \) is unit mass per length of the beam.

The solution to equation 7.3 is:

\[ y = Ae^{(px-\omega t)}, \]  

(7.4)

where, \( A \) is the maximum bending wave amplitude, \( p \) is the wavenumber, \( p = \frac{2\pi}{\lambda} \), \( \lambda \) is wavelength, and \( \omega \) is the circular frequency. Substituting equation 7.4 into equation 7.3, the circular frequency can be expressed as a function of wave number as:

\[ \omega = \sqrt{\frac{(k + Elp^4)}{m}} \]  

(7.5)

The wave propagation speed can be expressed as natural frequency divided by wavenumber:

\[ c = \frac{\omega}{p}, \]  

(7.6)

where, \( c \) is the bending wave propagation speed. Substituting equation 7.5 into equation 7.6, wave propagation can be expressed as:

\[ c = \sqrt{\frac{k}{p^2 + Elp^2}} \frac{1}{m} \]  

(7.7)

All the variables except wavenumber are constant. Thus, wave propagation speed is a function of wavenumber and a minimum wave propagation speed, \( c_{\text{min}} \),
which can be expressed as:

\[
c_{\min} = \left( \frac{4kE}{m^2} \right)^{1/4} \quad (7.8a)
\]

\[
p = \left( \frac{k}{EI} \right)^{1/4} \quad (7.8b)
\]

The minimum wave propagation speed describes the wave propagating in the trackbed layer, and is different than Rayleigh wave propagation speed, which is considered as the train critical speed for causing excessive vibrations due to Mach-like effect. When train speed achieves the track wave speed, a similar phenomenon occurs, and large rail deflections occur, which increases the possibility of derailment. For example, for a typical ballast railroad trackbed, the minimum wave velocity in the trackbed is on the order of several hundred meters per second, which is typically higher than the Rayleigh wave speed. Therefore, excessive vibration problems more typically result from train speed approaching the Rayleigh wave speed. However, for some cases, especially when the railroad is elevated by structures such as beams, the minimum velocity for wave propagated in track will control the highest service train speed due to the high stiffness of structural materials (e.g. concrete, steel etc.).

**Vibration Results from Numerical Simulation**

As described in Chapter 6, a numerical simulation method was proposed in this research to simulate ground vibration induced by rail roughness. Herein, vibration results are given in terms of vertical particle velocity. Figures 7.10 through 7.13 illustrate the ground surface particle velocities for a ballast trackbed
for x=10 m at y=1, 5, 10 and 20 m. Note that x, y and z coordinate system is following the right hand rule and x direction is parallel to the rail, z direction is perpendicular to the ground surface. Figures 7.14 through 7.17 show similar cases for a HMA trackbed type.

Each of these figures consists of two subplots. The top subplot illustrates the particle vibration history in the time domain, and the bottom subplot illustrates the corresponding spectral content of the particle vibration.

By observing the particle vibration history in time domain, it is interesting to find that at the initial time, the velocity is zero. This is reasonable because before the train bogie enters the model, there is no external dynamic force disturbing the model. Once the train enters the model, non-zero particle velocities are observed as the model is perturbed. Note that the small particle velocity during the first two seconds of the simulation are caused by the p-wave mode due to its high propagation speed.

According to in situ test results, high particle velocities are observed when the train passes the position where the receiver is deployed. In the numerical simulation, the observation position shown is at x=52 m. Then, considering the stress bowl length of 10 m, the total distance for the first bogie moving to the observation point is 57 m (moving distance in model of 52 m plus half stress bowl length of 5 m). The train speed used in the simulation is 22.4 m/s. Therefore, the time for the first bogie moving to the observation point can be calculated as 2.55 s. By inspecting Figure 7.10, it can be observed that at 2.5 s, particle velocity begins to increase dramatically.
Figure 7.10 Particle Vibration of Ballast Trackbed
From Numerical Simulation at x=52 m, y=1 m

Figure 7.11 Particle Vibration of Ballast Trackbed
From Numerical Simulation at x=52 m, y=5 m
Figure 7.12 Particle Velocity of Ballast Trackbed
From Numerical Simulation at x=52 m, y=10 m

Figure 7.13 Particle Velocity of Ballast Trackbed
From Numerical Simulation at x=52 m, y=20 m
Figure 7.14 Particle Vibration of HMA Trackbed
From Numerical Simulation at x=52 m, y=1 m

Figure 7.15 Particle Vibration of HMA Trackbed
From Numerical Simulation at x=52 m, y=5 m
Figure 7.16 Particle Vibration of HMA Trackbed
From Numerical Simulation at x=52 m, y=10 m

Figure 7.17 Particle Vibration of HMA Trackbed
From Numerical Simulation at x=52 m, y=20 m
As shown in Figures 7.10 through 7.17, particle velocity decreases with increasing of distance from center of railroad line as expected. At y=1 m, which is within the track, high particle velocity was recorded in the time and frequency domains. This is obvious because the forces were directly added on it.

As seen in Figures 7.9 and 7.13, several peaks can be found in the frequency domain. The first one, which contains the highest amplitude values, is located at a frequency range of 1 to 3 Hz. This indicates that the trackbed has a low frequency vibration character in its vertical direction. Other peaks are mainly concentrated in the frequency range of 10 to 40 Hz. It is interesting to find that the velocity amplitudes at high frequencies (greater than 60 Hz) are relatively small. Therefore, it can be concluded that the trackbed generally vibrates at a very low frequency. Reviewing the generated forcing functions in Chapter 6, a conclusion can be drawn that the trackbed vibration is a result of a combination of static forces generated by a moving mass, which contributes to the low frequency components in vibration of track panel bending, and dynamic forces caused by rail roughness, which contributes to the high frequency components that forces track components such as rail, fastener and tie to shake quickly. For the other positions (y=5, 10, and 20 m), it is interesting to see that low frequency components dissipate much faster than high frequency ones. Typically, high frequency components dissipate faster than low frequency components for propagation over the same distance in a homogenous system because high frequency components correspond to more hysteric damping cycles. To explain
this counterintuitive phenomena, two factors were considered as described in the following paragraph.

The major reason is due to the geometry of model. As stated previously, the soil layer at Conway is very thin (about 10 m), and is underlain by bedrock. Therefore, when there is a source on the ground generating excitations, the maximum properly propagating Rayleigh wave length in this soil site is approximately 10 m. Note the soil has a measured shear wave velocity of 335 m/s, thus the Rayleigh wave velocity for the soil should be around 295 m/s. The low frequency component, such as 2 Hz Rayleigh wave that has a wavelength of 148 meters, cannot propagate properly in this shallow layer of soil. However, some certain high frequency components, which have a wavelength less than 10 m, can propagate properly in this thin soil layer. Once the long wavelength Rayleigh wave hits the boundary (the interface of soil and bedrock), some of its energy then radiates in the form of body waves such as p-wave and s-wave. According to the relationship between particle motion attenuation and propagation distance for body wave and surface wave shown in equations 7.9a and 7.9b respectively, for the same material (damping ratio), initial amplitude, frequency and traveling distance, body wave has a much higher energy loss than surface wave.

\[
A_R = \frac{A_0}{R} \exp(-\alpha R) \quad (7.9a)
\]

\[
A_R = \frac{A_0}{\sqrt{R}} \exp(-\alpha R) \quad (7.9b)
\]
where, \( A_0 \) is initial source amplitude, \( A_R \) is receiver amplitude, \( R \) is traveling distance and \( \alpha \) is attenuation coefficient, which can be determined from:

\[
\alpha = \frac{2\pi f D}{V}
\]  

(7.10)

where, \( D \) is material damping ratio, \( V \) is velocity, and \( f \) is frequency.

The other factor is due to Rayleigh damping scheme used in numerical analysis. The Rayleigh damping scheme is based on a frequency dependent calculation method, which defines it as having a minimum damping ratio at its center frequency, and damping ratio gets larger as frequency values far away from center frequency. However, for geological material, damping ratio is one of its properties and is not associated with the propagating wave frequency.

Figures 7.18 and 7.19 show the synthetic peak particle velocities for ballast and HMA trackbeds over the surface of the model. Peak Particle Velocity (PPV) is defined as the maximum absolute amplitude during a recording of particle vibration velocity versus time. This peak particle velocity can be either positive (upward) or negative (downward). For both figures, all the nodes on the surface of model were monitored during passage of the train, and maximum amplitude was identified at each node for contouring.
Figure 7.18 Peak Particle Velocity Contour for Ballast Trackbed

Figure 7.19 Peak Particle Velocity Contour for HMA Trackbed
Comparing Figures 7.18 and 7.19, it can be observed that the maximum PPV occurs at the same position for both ballast and HMA trackbed. This is reasonable because the same rail roughness profile was used. Higher velocities were calculated closer to the track, where forces were directly applied. However, due to the high damping ratio of the HMA layer, PPVs in the HMA trackbed are lower than those in the ballast trackbed, which indicates that use of material with a higher damping ratio helps reduce track vibrations by absorbing more energy. By reviewing the particle vibration in the frequency domain for ballast and HMA trackbed at the same position (Figure 7.10 versus 7.14, Figure 7.11 versus 7.15 etc.), the shapes of these figures are almost identical except for their amplitudes. Thus, the damping effect of HMA is identical in all frequency levels.

Figure 7.20 illustrates a PPV cross-section at x = 5 m. PPV at 23 m from center of rail has a higher value than at 10 m from center of rail. This figure indicates that PPV does not necessarily decrease with increasing distance from the rail. This can be explained by the constructive and destructive interference of propagating waves. The amplitude of a particular particle vibration depends on the incoming wave properties such as amplitude, phase angle, frequency; vibration source type such as number of bogies, bogie distance and rail roughness; and soil site profile such as number of layers, layer thickness and soil properties. All of these factors will affect the wave energy dispersion.
Another interesting finding was determined by plotting the shear strain of one soil cross section located at $x = 52$ m as shown in Figure 7.21. From this figure, it can be found that the maximum shear strain occurs at the foot of embankment. Therefore, subgrade problem due to the shear failure is most likely at the side of embankment.

Figure 7.20 PPV for Surface Particles at Cross Section of $x = 5$ m
Results Summary

As described in the previous two sections, results from in situ tests agree reasonably well with results from numerical analysis. Therefore, actual measurements confirm the credibility of the numerical analysis method used in this research. However, it should be noticed that there are two differences between in-situ and numerical results, which can be observed by inspecting the particle motion graphs in the frequency domain such as Figures 7.3 versus 7.11, etc. For the in situ tests, the wheel and tie spacing effects are evident. But for the numerical results, these effects are not apparent. The reason for the absence of the wheel spacing effect in the numerical data is because wheel forces are
converted to the stress bowls. The tie spacing effect is absent because of the method of applying stresses, which assumes stresses were uniformly applied on the top of ballast (model), whereas in reality, stresses were only applied to the ballast at points where ties contact the ballast. To solve this problem, finer grid spacing should be chosen in the numerical model so that stresses can be applied to the model separately rather than continuously.
Chapter Eight

Summary of Conclusions

The findings and conclusions emanating from the in situ tests and the numerical simulation are contained in the following discussions.

The train induced ground vibration phenomena has been investigated, measured and simulated in this research study. A numerical simulation method that considers the rail roughness was developed. It was determined to be applicable for predicting the ground vibration and monitoring the surface wave propagation. The credibility of the method has been confirmed by comparing the results from numerical simulation and in situ tests. The method is applicable for evaluating both conventional ballast trackbeds and HMA trackbeds.

The effects of numerous variables on trackbed design and evaluations, as simulated and measured in this project, are presented in particular detail. The incorporation of an asphalt-bound layer (HMA underlayment) in the track structure, in place of a granular layer, has a particular effect of reducing both trackbed and vicinity ground vibration. This increases the service life of the track structure and subgrade, thus reducing maintenance and rehabilitation costs and improving operating efficiencies. Also implied is that the application of rubber modified hot-mixed asphalt as a trackbed sublayer material will further reduce vibration.

One of the benefits of the proposed simulation method is that this method is able to simulate corrugated rail. Roughness values used in this project were generated based on a function generator, which generates the roughness
numbers randomly. Then the roughness function was modified based on some real statistical regression results. It should be noted that it does not perfectly match the actual rail roughness case at Conway, where measurements were taken. However, it has been proven by in situ test results that this is an acceptable method for simulating the rail roughness, and it can yield a reliable solution. The other benefits of this simulation method is that the generation of rail roughness values can be separated from the numerical simulation, which means the rail roughness data can be either generated by using the method provided in this method or from other sources. For example, if rail roughness values can be obtained based on the actual measurements, then those values can be used directly by dropping the roughness generator.

HMA effect on the vibration reduction was investigated and compared with that of ballast trackbed. Results, from both numerical simulation and in situ tests, indicate that the HMA layer contributes to damping energy due to its relative higher damping ratio. However, it should be noted that in this study the HMA damping ratio was set at 5%, which is near the upper limit of common HMA damping values. The reason for doing so was to find the potential application of RMA, which has a damping ratio from 7% to 10%, since it has similar material properties as HMA except it possesses a higher damping ratio.

The ground vibration caused by a moving train on a corrugated rail has been simulated and tested in this research and the spectral contents of ground vibration have also been investigated. The moving train on a smooth rail only causes low frequency vibration whereas high frequency contents have been
generated due to the rail roughness. The dissipation of energy carried by waves is not only associated with geologic material damping capacity, but also related to the geometry of the subsurface. The boundary effect assists in converting surface waves into body waves, which has a higher energy loss for propagating the same distance.

A significant outcome is the synthetic peak particle velocity generation. This indicates that the peak particle velocity amplitude is not only a function of distance between measuring position and rail. It will be affected by many factors including distance, amplitude, phase angle etc. This is because of patterns of wave construction and destruction, which can interfere with each other. Therefore, using distance as an indicator could lead to incorrect estimation. Actually, PPV can increase as the distance increases from the rail.
Chapter Nine

Future Research Suggestions

Based on current research achievements, it is recommended that future research should be concentrated in following areas:

1. Using a finer model for numerical simulation. The current model has a minimum grid spacing of 0.5 meter. However, a finer grid spacing is desired for several reasons. Firstly, it can improve the accuracy of numerical simulation results. Secondly, it allows the application of discontinuous stresses to the track model. It should be recognized that in the actual case, pressures applied to the ballast are not continuous. Thirdly, a finer model enables the application of discrete element in track model (embankment), which can simulate rail, tie, ballast and sublayer. This gives great improvement of simulating the real track structure. Also, the entire train should be used in the simulation progress, which represents the true case. However, the limitation of using a finer model and an entire train is the increasing of computation time and the requirement of larger computation memory.

2. Due to the soil stiffness, the soil constitutive model used in this research is linear elastic. However, other soil constitutive models should be considered such as Modified Cam Clay (which is good for clay) and hyperbolic (non-linear elastic) model (which is good for sand). Also, pore water pressure distribution should be considered in the model. This is very critical especially when the train transverses an area composed of sand. When the wave generated by the train
propagates through the soil, it may cause the pore water pressure redistribution thus leading to liquefaction.

3. CSX Transportation’s freight railroad line at Conway has been used in this dissertation as the test line. However, this may not be the ideal indicator for passenger line due to the low train speed. Another location should be selected for study of the ground vibration caused by high-speed passenger trains. The high speeds of passenger trains will cause excessive vibrations when the train’s speed approaches the Rayleigh wave propagating speed in soil. This will be an interesting area of research as high-speed rail lines are developed.
Appendix

A folder, labeled “Appendix,” containing all field data and numerical scripts for this research is attached with this dissertation. There are three folders in the main folder and they are “field test,” “simulation,” and “results.”

There are two folders named “test1” and “test2” respectively in the folder “field test.” All the field test results from the test performed on June 27, 2003 are stored in folder “test1.” All the field test results from the test performed on August 7, 2003 are stored in folder “test2.”

There are numerous files in the folder “simulation” and their names and explanations of their functions are listed as follows:

“dyncwbald.txt”: a FLAC$^{3D}$ file for dynamic analysis of ballast track model by recording the particle displacement.

“dyncwbalv.txt”: a FLAC$^{3D}$ file for dynamic analysis of ballast track model by recording the particle velocity.

“dyncwhmav.txt”: a FLAC$^{3D}$ file for dynamic analysis of HMA track model by recording the particle velocity.

“fftpostprocessconway.m”: a MATLAB script used for performing FFT on the data generated from FLAC$^{3D}$.

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“stacwbal.txt”: a FLAC$^{3D}$ file for static analysis of ballast track model.

“stacwhma.txt”: a FLAC$^{3D}$ file for static analysis of HMA track model.

Note that the computation time of FLAC3D depends on the computer hardware, especially the Central Process Unit (CPU) type, speed etc. In this research, a Pentium 4 2.0-GHz CPU is used and the computation time is about 2 days for static analysis, 6 days for dynamic analysis of ballast track model, and 25 days for dynamic analysis of HMA track model. Results obtained from numerical simulation in this research are provided by storing them in the folder “results.” The file named “cwbalvel.exe” contains results from dynamic analysis of ballast track model. The file named “hmavel.exe” contains results from dynamic analysis of HMA track model. Both of them are executable self-extracted files.
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