Effects of ground motions from high-speed trains on structures, instruments, and humans

Charles H Dowding
Professor, Civil Engineering, Northwestern University, Evanston, IL USA

ABSTRACT: This paper describes effects of ground motions generated by high-speed trains on near-by structures, instruments and humans. Both model generated and field measured motions are employed to estimate these effects. Measured train-induced motions are compared with motions generated by typical construction and building occupant activities. Response spectra of the train-induced ground motions are calculated for comparison with those that have produced cosmetic cracking as well as to illustrate the importance of dynamic building response characteristics. Finally, instrument and human response is assessed based upon guidelines provided by instrument manufacturers and American standards organizations. Special emphasis is placed on motions generated by trains traveling faster than the surface wave propagation velocities of soils on which the tracks rest.

1. MODEL STUDY TO PROVIDE BACKGROUND

Amplification effects of soft foundation soils were estimated with a boundary element program (Pflanz, 2000) that was employed to calculate the ground motions generated by trains moving at 300 km/hr or at 1.04 and 0.69 times the shear wave propagation velocities of 80 m/s (288 km/hr) and 120 m/s. Figure 1 is the plan view of this model that shows the locations 8, 16 and 32 m from the track center-line, where ground motion time histories were calculated. In order to simplify the comparison, an unlayered profile was employed where the soil was modeled as an elastic solid with a Poissons ratio of 0.33, density of 2000 kg/m^3, and shear wave velocities of 80 and 120 m/s. The total train load from an engine and a car of 960 kN (wt = 98 tons) was distributed on 16 wheels over a distance of 40 m. Full model details can be found in (Pflanz, 2000)

Peak particle velocity time histories in the vertical direction produced by this model 32 m from the track center-line are shown in Figure 2. Amplification of the ground motions produced by breaking the “propagation velocity barrier” easily can be seen by the comparison of the motions produced by train speeds of 1.04 and 0.69 time the shear wave propagation velocity. These computed motions are typical of field measurements as will be discussed later. While at 32 m the larger motions are just at the threshold of cracking, they do exceed human sensitivity guidelines for continuous motions as do smaller motions produced by the lower train speeds. The implications of both ranges of

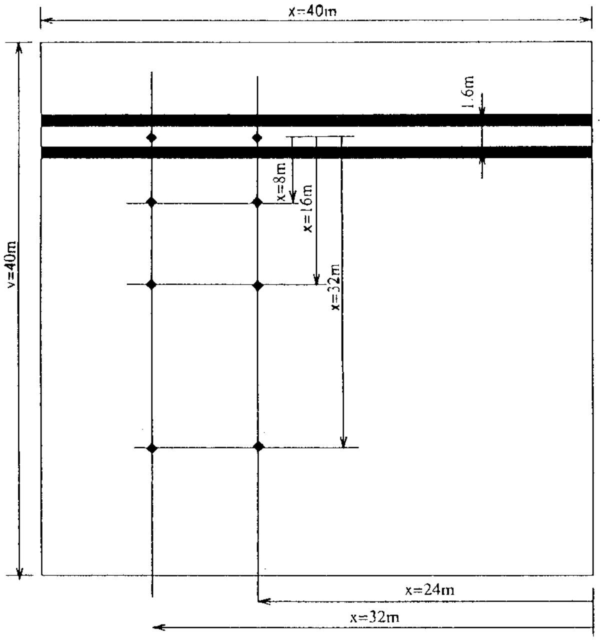

Figure 1. Plan view of boundary element model showing track and points at which ground motions were captured (from Pflanz, 2000)

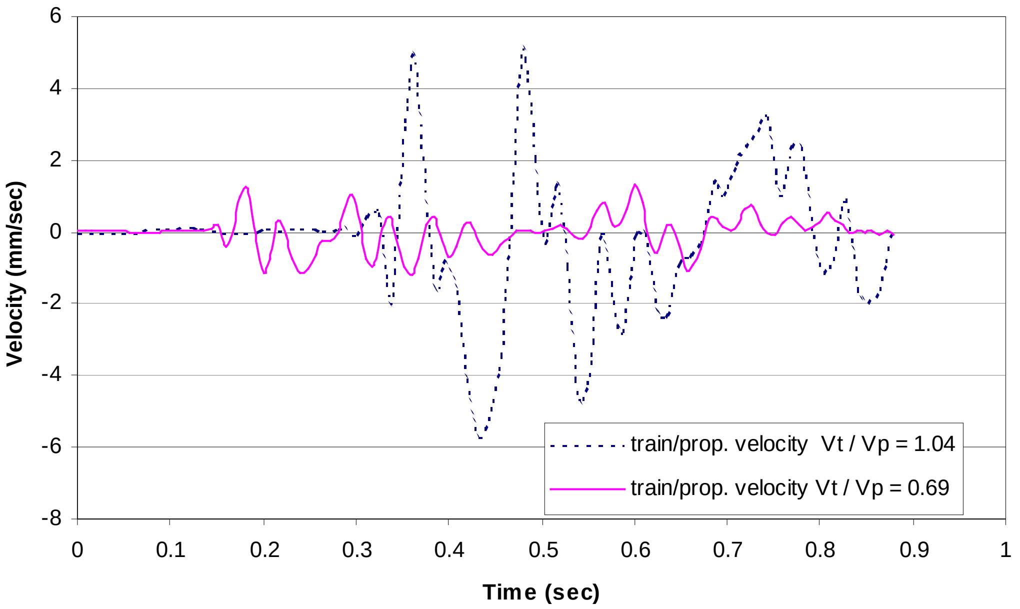

Figure 2. Particle velocity time histories at a distance of 32 m from the track centerline generated by trains traveling 1.04 and 0.69 time the propagation velocity of the surface wave

motions will be discussed in detail in later sections

Normally concern for the train response leads to a focus on the displacement time histories at the track center-line. However since the focus of this paper is structural and

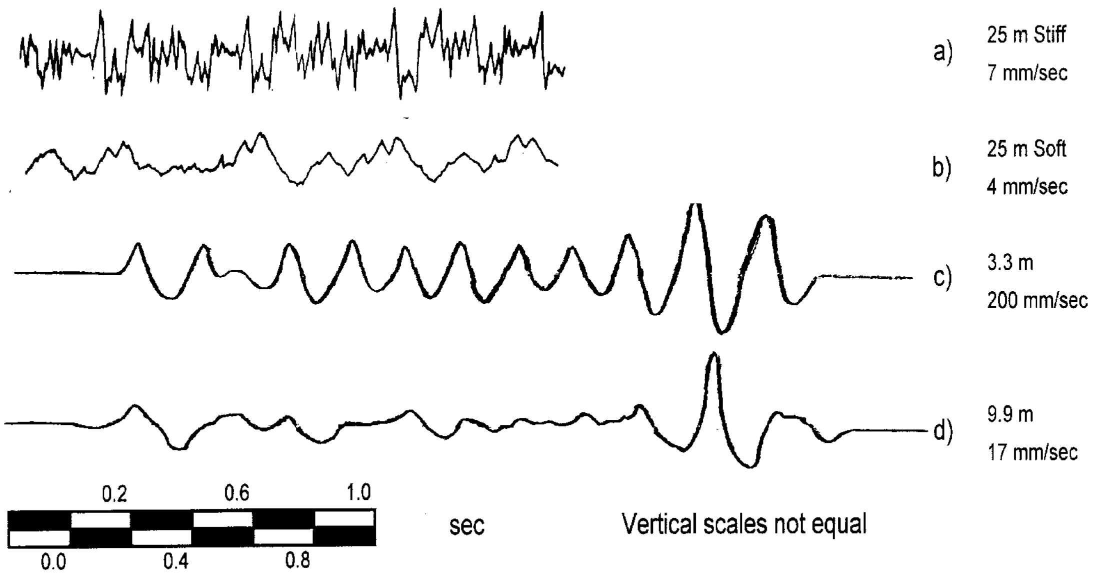

Figure 3. Comparison of particle velocity ground motion time histories generated by high speed trains traversing differing geologies. Time histories a) and b) are 25 m from Japanese trains traveling slower than the propagation velocity, whereas c) and d) were recorded only 3 and 9 m from Swedish train traveling at speed greater than the propagation velocity

human response, the generated motions were evaluated in terms of the zero to peak PPV, which is customary for vibration concerns. While this measure is customary in the United States, other measures such as dB, RMS, VL values may be employed elsewhere. These distinctions will be discussed later.

As has been shown by structural dynamics and earthquake engineering, the actual ground motion time history is important to investigate specific instances of response. Dominant frequency, duration, and transient shape (number of principal pulses) affect structural and human response. Figure 3 illustrates the wide difference between train induced ground motion time histories. Effects of all of these factors can be incorporated by the response spectrum of the ground motions. The response spectrum is a collection of the maximum responses of single degree of freedom systems with varying natural frequencies but constant damping to the same ground motion. Pseudo velocity response spectra of the calculated motions at 8, 16, and 32 m produced by the model high speed trains are shown in Figure 4. The Appendix contains a detailed discussion of the response spectrum as well as the “tripartite” paper on which it is presented.

Response spectra of modeled motions produced by high speed trains in Figure 4 show that the dominant frequency decreases with distance. Dominant frequencies are those with the greatest amplitude. For instance, close-in at 8 m, the dominant frequency is 25 Hz, while at 32 m the dominant frequency is 7 Hz. Even close-in there is still a significant secondary peak at 9 Hz. Field measurements of actual motions, discussed later show that the actual dominant frequencies are below 5 Hz. However the model correctly shows the importance of the low frequency waves. These low frequency waves are most likely surface waves. Response spectra of the ground motions produced by the slower trains traversing stiffer (higher propagation soils) in Figure 5 show that low frequency wave is not dominant at 32 m.

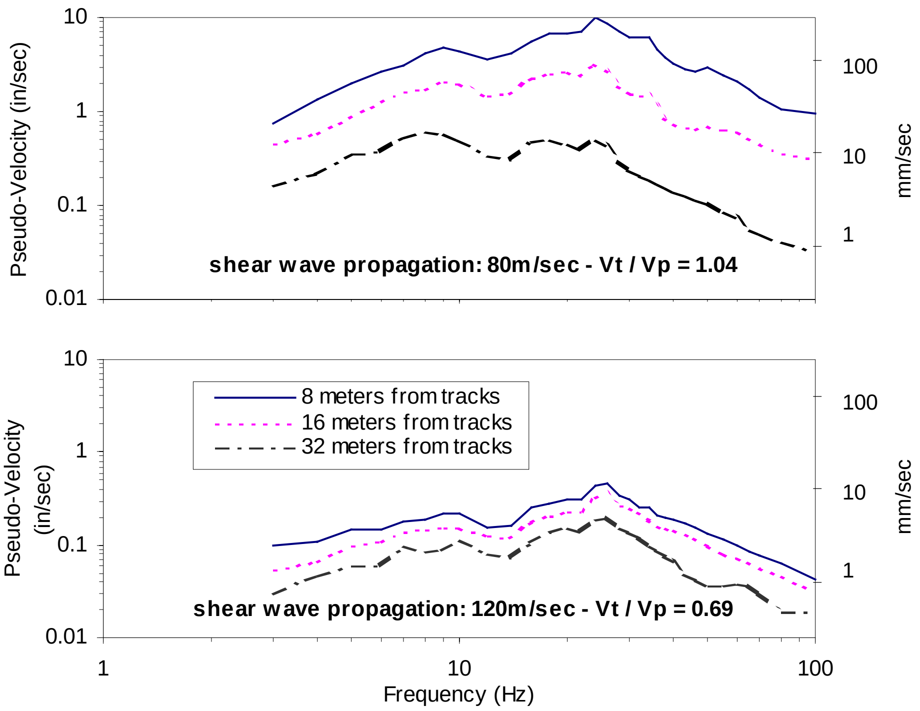

Figure 4. Response spectra (damping = 5%) of model ground motions showing greater amplitudes for $V_t/V_p > 1$ and declining dominant frequency with increasing distance from tracks and smaller amplitudes for $V_t/V_p < 1$ and relatively constant dominant frequency with increasing distance.

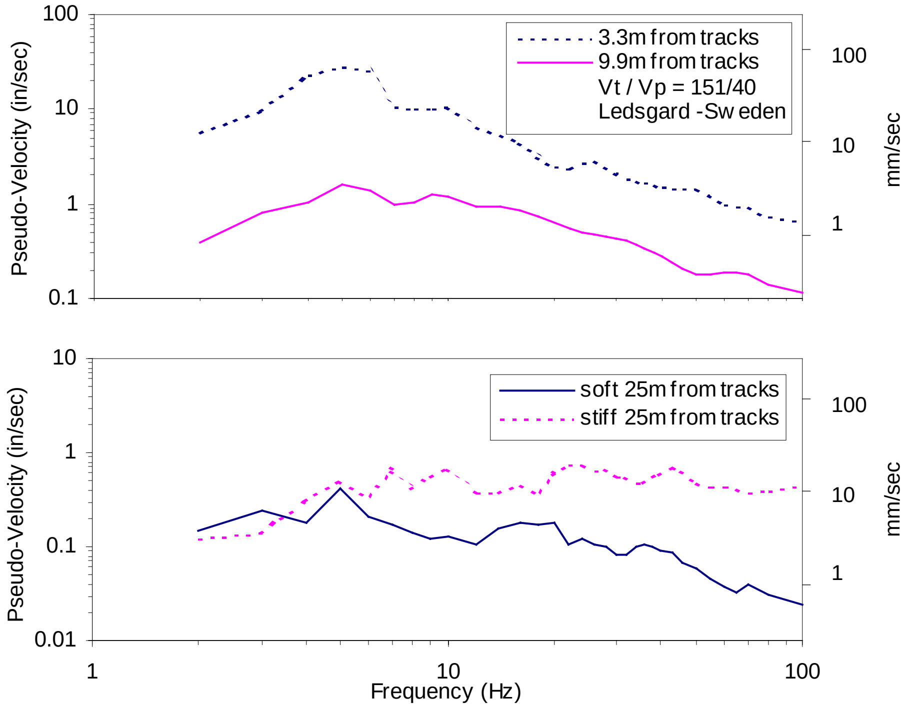

Figure 5. Response spectra (damping = 5%) of measured ground motions at Ledsgard ($V_t/V_p > 1$) and for Shinkansen ($V_t/V_p < 1$) for soft and stiff soils

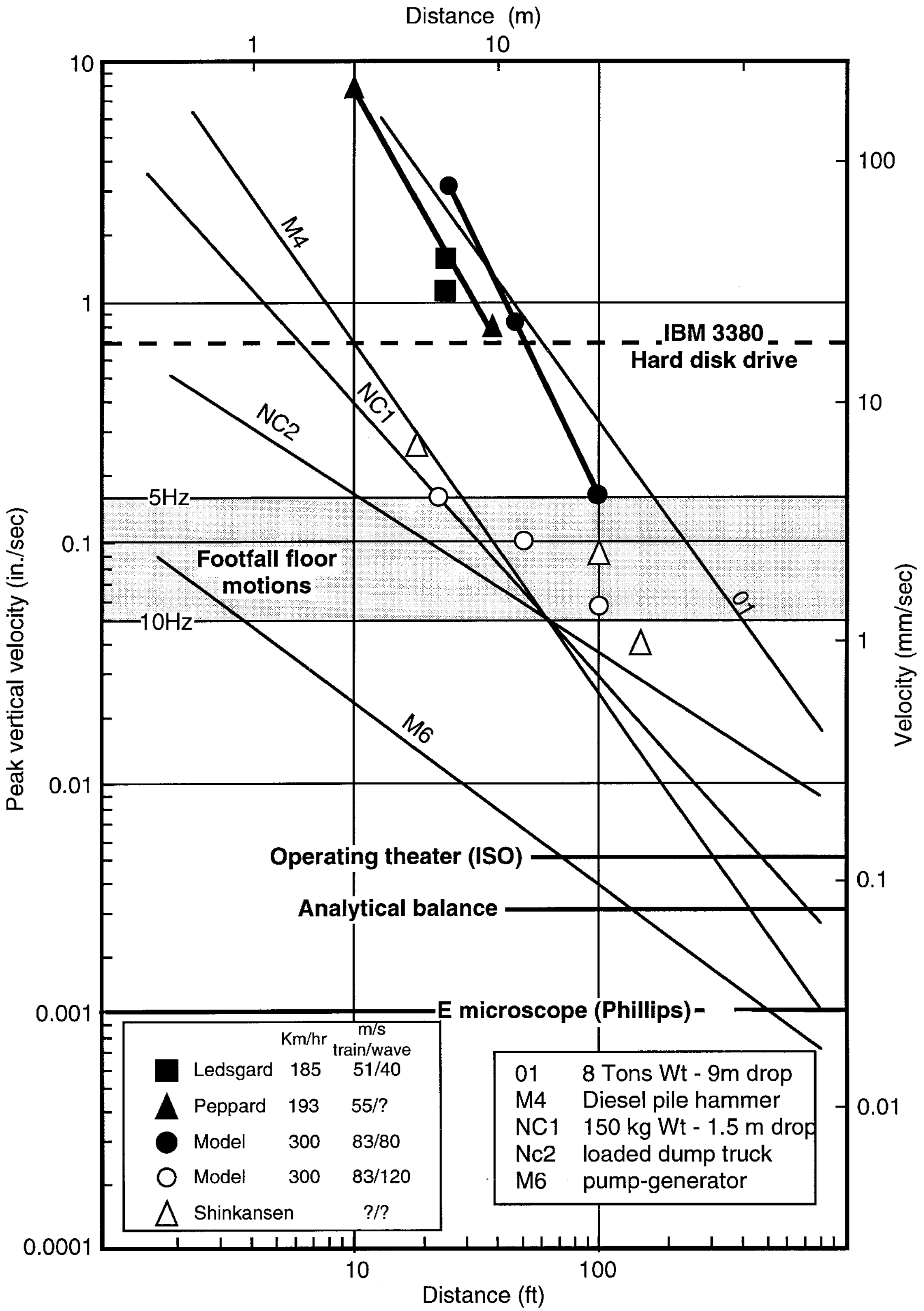

Figure 6: Attenuation of measured and modeled motions compared with common construction activities, floor motions produced by habitation, and instrumentation sensitivity

2. CONFIRMATION OF MODELED WITH FIELD MEASURED MOTIONS

As shown in Figure 6, field measurements (Adolfsson et al, 1999a and Adolfsson et al, 1999b) at Peppard and Ledsgard in Sweden show trends similar to those calculated with the model. The Swedish site is composed of a 2 m stiff crust that over lays 5 m of soft marine clay, which in turn over lays a stiff clay. As shown by the loading diagram and resulting displacement time histories in Figure 7, the test train is approximately 4 times longer than that employed in the model. Trains were operated much more slowly than the model in terms of absolute velocity: 185 km/hr (51 m/s) at Ledsgard vs. 300 km/hr (83 m/s) for the model. However, since the shear wave velocity of the soft layer at Ledsgard is some 40 m/s, the ratios of train to propagation velocity (V_t/Vp)for the high speeds are similar: 51/40 at Ledsgard and 83/80 for the model.

The modeled and measured Peak Particle Velocities (PPV's) produced at high speeds are compared in Figure 6: thick lines with triangles and squares for the measured values and circles for the modeled values. The calculated values are approximately 1.5 times those measured, which may be a result of the higher absolute velocities of the model. However the attenuation (decay) with distance is the same for both measured and calculated PPV's. These PPV's are compared with other common vibration producers (Dowding 1996 and Blais, 1994) in the figure. The implications of these comparisons will be discussed in detail in a later section.

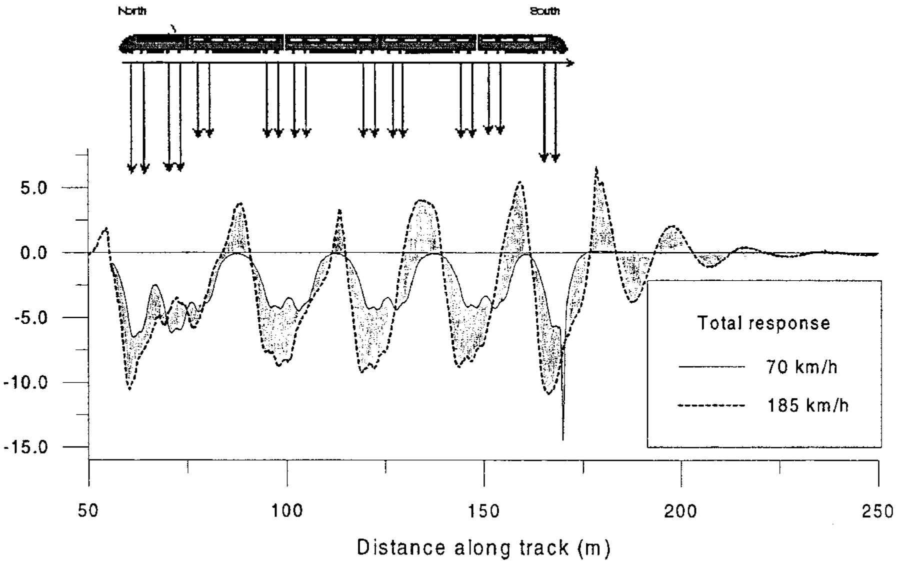

Figure 7. Loading and measured displacement diagrams for high speed trains on soft soils at Ledsgard (from Adolfsson et al, 1999 b).

Measured and modeled PPV's for lower train velocities are also compared in Figure 6: open triangles for measured and open circles for calculated values. The calculated PPV's for slower speed trains decay more slowly than those for high speed trains. However they are still within the expected range as shown by their comparison with case NC 2, PPV's of motions produced by loaded dump trucks traversing bumps. The measured values are PPV's of ground motions produced by the Shinkansen crossing elevated bridges (Kobyashi, 1974). These motions are not associated with $V_t/V_p > 1$.

Response spectra of ground motions, produced by Japanese trains with low V_t/V_p are compared in Figure 5 very with those from the high speed trains over very soft soil (high V_t/V_p). Time histories employed to compute these response spectra were recorded 25 m from the tracks on soft (standard penetration resistance, spt = 5) and stiff (spt = 30) soils. Since PPV's were not reported, only the frequency content can be deduced form the spectra. Even at slow speeds the dominant frequency of motions produced by traversing soft soils is low, 2 Hz. It increases to over 30 Hz for the stiff soils.

3. DIFFERENCES IN CONTINUOUS AND IMPULSIVE TERMINOLOGY

As seen by the presentation in the first section, this paper presents and discusses ground motion in its raw, unprocessed format as is customary in structural dynamics for short duration, impulsive motions. Maximum amplitudes are expressed in zero to peak values of particle velocity. Particle velocity has been employed rather than displacement (train response) or acceleration (earthquake engineering and some human response) as it is customarily employed in the United States to describe low-level motions where there may be a potential for cosmetic cracking. Whenever possible actual time histories were displayed. The remainder of this section presents a comparison of zero to peak particle velocities with other common measures of vibration.

Measures of vibration at low amplitudes are widely variable throughout the world. Most of this variability has its origin in the difference between the relatively continuous nature of sound and industrial vibration and the impulsive character of transient vibration. To further complicate approaches, mathematical descriptors of low amplitude processes, such as root mean square amplitude, have been borrowed from electrical engineering. While such descriptors may be appropriate for describing human response for continuous traffic, they tend to mask important effects of singular events such as the passage of a single high-speed train.

Unlike impulsive motions (with limited numbers of peaks such as a high speed train), peaks of continuous motion (usually associated with reciprocating machinery), are normally described in terms of an average measure. The preferred average measure of continuous motion is the root mean square (rms). This is the average value of the square of the parameter being measured. Also many times peak to peak (pp) amplitudes, rather than the zero to peak (p) amplitudes, are reported by instrument makers. Amplitude conversions between these three systems are as follows:

$$p = 1/2 \text{ pp} = 0.50 \text{ pp}$$

$$p = \sqrt{2} \text{ rms} = 1.41 \text{ rms}$$

Great care should be exercised to ensure that the zero to peak particle velocities, normally recorded by digital instruments, are compared to criteria expressed in the same terms. As can be seen by the above conversions, the differences are as large as 40 to 50%.

To further complicate matters peak values, a, are often described by a decibel, dB, level relative to some base level, a_0, where

$$dB = 20 \log_{10}(a/a_0)$$

In Japan the base, a_0, is 10^{-5} m/s^2 and in Europe the base is 10^{-6} m/s^2. The dB scale is not a standard reporting method in the United States for vibration; however it is standard for reporting sound.

Another difference in reporting involves the use of frequency weighting factors. Weighting was originally employed for sound measurement to record pressures sensed by the human ear at its sensitive frequencies. Frequency weighting was especially troublesome when the U. S. Bureau of Mines developed its standard for blast induced sound pressures because structures responded to pressure waves at frequencies far lower than those sensed by the human ear. As a result, the sound guidelines must now be expressed as a function of the frequency weighting of the pressure transducer, A , C, C-slow, etc.

This custom of frequency weighting has now begun to be adopted for human vibration response. Unfortunately data that are recorded with instruments that filter data by frequency cannot be employed for structural response. The full, unfiltered wave form is necessary for structural response. For this reason, the American National Standard Institute (ANSI) recommends that filtering be undertaken after recording so that any weighting scheme might be applied at a later date .

Relative amplitude values often are reported in octave bands as a result of the use of Fourier analysis of the motions and the observation of differences in narrow bands of frequencies in a broad band of possible frequencies. Instead of the logarithmic power of ten progression (1, 10, 100, etc.) normally reported in structural dynamics, octave intervals with a power of two progression (2, 4, 8, etc.) are sometimes employed. The 1/3 octave indicates that between major divisions, values are computed as frequencies of 2^{n+1} and $2^{n+2.}$ For instance between 8 Hz (= 2^3)and 16 Hz (=2^4), two intermediate values are considered, namely $2^{3+1/3}$ (10.1 Hz) and $2^{3+2/3}$ (12.7 Hz). The concept of center frequency probably means that the average amplitude between 8 and 10.1 Hz is plotted at 9 Hz.

More importantly, differences in the response of a structure , instrument, or a human to 1) a small number of impulses or 2) a continuous excitation may be very large. As discussed in detail in Dowding (1996), differences in responses to two identical excitations, that differ only in number of significant pulses, should be scaled by a response factor. Response of an instrument or human idealized as a single degree of freedom system is a factor of 2 less for a 4 pulse ($n = 4$) event than for a continuous ($n > 15$) event. In other words a single train passage might produce only 50% of the response produced by continuous, reciprocating machine-induced, excitation of the same particle velocity. Excitation at the instrument's natural frequency is the most critical as it produces the greatest response.

4. COMPARISON OF TRAIN AND OCCUPANT INDUCED MOTIONS WITH MICROVIBRATION CRITERIA

It is important to compare serviceability criteria for various types of electronic instrumentation with train and occupant induced vibrations. To this end, vibration attenuation relations for train and typical construction-induced motions are compared with these criteria in Figure 6. Details for these specific cases were given in (Dowding, 1996). A number of important observations can be made from this comparison.

Probably the most important observation is that occupant induced floor vibrations are large in comparison with microvibration serviceability requirements. The stipled band of foot fall motions in Figure 6 were found by assuming a smooth foot fall loading for 5 and 10 Hz floor systems and is a lower bound of expected occupant induced motion in modern, wide span, office floors. On the other hand, this band would be an upper bound of expected occupant induced motions for stiffer, older and/or industrial grade floors (Ungar, 1979).

Because these occupant-induced motions are so large, most especially sensitive instruments are individually isolated or their floors are specifically designed. For instance, laser measurement systems are normally founded upon soft isolation springs (similar to under pressurized small tire inner tubes). An operating room (theater) would have to be designed with an industrially stiff floor to avoid normal assistant walking from interfering with microsurgery.

On the other hand there are electronic instruments that are surprisingly robust and have serviceability requirements that are less restrictive than foot-fall induced motions. Computer disk drives are an example: soft errors for the IBM 3380 did not begin until floor velocities exceeded even those measured on modern, flexible floors.

Another important observation to make is that construction and train-induced motions in Figure 6 have been recorded on the ground outside of the structures, and the serviceability guides are generally for the motions of the instrument itself. These ground motions must be transmitted through the structure where they are normally reduced through both attenuation and deamplification resulting from frequency mismatch of excitation and response natural frequencies. However, secondary systems on these floors – such as tables -- can reamplify the deamplified motions if their frequencies are similar to the floor’s.

Despite the probable deamplification of the ground motions by structures, conclusions regarding proximity will be made with the train and construction ground motions as though they were transmitted directly to the instruments. This comparison with non deamplified motions provides a conservative view point from which to observe the significance of ground motions from high speed trains.

High speed trains will have to be separated significant distances from critical facilities if ground motions are transmitted by low propagation velocity soils. As shown by the extension of the thick lines on Figure 6, under these special conditions high speed trains would have to be separated by some 20 to 30 m from structures not to exceed the occupant induced floor motions. As shown by Ramirez, (1986), heel drops on these same floors produce motions 4 times greater (20 mm/s) than smooth foot falls (5 mm/s). Typical low speed trains or high speed trains traversing stiff soils would have to be separated by only 10 to 20 m not to exceed occupant induced floor motions. As shown by the Cases M4, and NC1&2, these separation distances are only slightly greater than required by typical construction near critical facilities.

Construction traffic motions as represented by case NC2 show that for distances greater than 15 m (50 ft), the ground motions are below floor motions induced by smooth foot falls on typical (10Hz) floors. Thus construction traffic would have to come within 3 m (10 ft) of a structure to produce motions that would exceed atypical, but not unlikely, occupant induced motions. Measurements in the New York City library showed atypical occupant-induced motions to be as high as 8 mm/s on a concrete floor housing their computer facilities

More vibrant construction activities will require greater stand off distances to reduce motions below those produced by smooth footfall induced floor motions. For instance heavy dynamic compaction activities as represented by Case 01, would require separation of over 100 meters to reduce ground motions to that lower than produced by smooth foot fall, and 60 m to fall below those produced by heel drops. However pile driving, represented by case M4, could be conduced within 6 m without producing ground motions that exceeded the floor motions produced by heel drops on a 10 Hz floor.

Most importantly of all, differences in train-induced ground motions, deamplification of motions through structural transmission, existing machine isolation, and differences in vibration sensitivity, require that each instance be investigated specifically. Full time histories of motions are required to design systems to avoid reamplification of the train-induced ground motions The above standoff distance comparisons were conservatively made for typical situations.

5. RESPONSE SPECTRUM ANALYSIS OF HIGH SPEED TRAIN GROUND MOTIONS TO DETERMINE POTENTIAL FOR COSMETIC CRACKING

Response spectra in Figures 4 & 5 are employed to predict structural response and thus best describe potential adverse response of structures near high-speed rail lines. This response approach is based upon principles that have been developed in structural dynamics and earthquake engineering. As described earlier and in the Appendix, the response spectrum allows the simultaneous incorporation of the excitation time history (

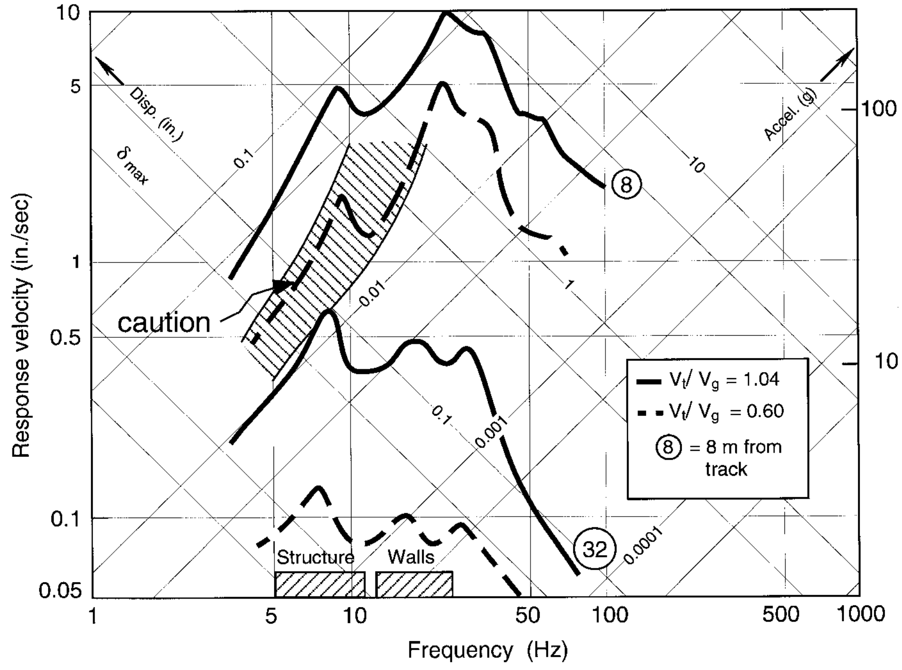

Figure 8: Response spectra of modeled motions at 8 and 32 m for $V_t/V_p > 1$ & <1 compared to precautionary bounds associated with cosmetic cracking of plaster wall materials

including its dominant excitation frequency and pulse sequence) and the dynamic response properties of the structure(s) of concern.

A response spectrum curve in Figure 8 is the representation of the responses of structures (distinguished by their differing natural frequencies) to the same ground motion. For instance the highest, thick solid line is the spectrum of responses to ground motions 8 m from the model of a 300 km/hr train on soft ground ($V_t/V_g > 1.0$) The most critical frequencies are those walls (and floors) and superstructures (1 and 2 storied) shown by the stippled regions along the frequency axis.

Spectra of ground motions associated with observation of vibratorialy induced, cosmetic, hair thickness cracks fall within the cross hatched “caution” zone. This region contains spectra of motions with the lowest peak particle velocity observed by the US Bureau of Mines to cause hairline cosmetic cracks (Siskind, 1980). Such cosmetic cracks are the size of those caused by natural, environmental effects and represent the threshold of vibration induced cracking.

The above discussion has established a the foundation for assessing the cracking potential of ground motions produced by high speed rail traffic over unmodified soft soils. Four spectra calculated from the model ground motions introduced at the beginning of this article are compared in Figure 8. Thick solid lines represent those of motions measured at 8 and 32 m from 300 km/hr trains outrunning the induced ground motions (V_t/V_g =1.04). Thick dashed lines are those of the same trains on soft soil with slightly higher propagation velocities (V_t/V_g =0.69). Trains outrunning their ground motions would have to be more than 32 m from the nearest structure to prevent cosmetic cracking. Trains on only slightly (50%) stiffer soils would require less (say16 m) separation from the nearest structure. These comparisons were produced with motions calculated with the model introduced at the beginning of this paper and can be made for actual conditions from ground motions collected in the field.

Dominant excitation frequencies decline with distance as expected. Near the track (8 m) the ground motions are maximize at 30 Hz. At 32 m the dominant frequencies have dropped to 7 to 8, which is in the range of the natural frequencies of superstructures and some modern floors. There continues to be significant motion in the range of natural frequencies (12-25 Hz) for wall and older and stiffer floor natural frequencies,.

6. HUMAN RESPONSE TO FLOOR MOTION PRODUCED BY HIGH-SPEED TRAINS

Siskind's (1980) description of the challenges in understanding human response to vibration set the stage well for the following discussion about high-speed train induced impulsive motion. He writes:

> "Most studies of human tolerance to vibrations have been of steady-state sources or those of relatively longer duration than typical mine, quarry, and construction blasting (lasting less than a second). In the absence of data on tolerance to impulsive vibrations, these results have been assumed to be applicable to blasting. Additionally, most useful data are from tests involving human subjects directly, when not in their homes. The duration and frequency of occurrence of the events are obviously critical.
>
> The vibration limits required for reasonable comfort from a long-term vibration [or noise] source (e.g., air conditioning, machinery, building

> elevators, and vehicle traffic) are certainly more restrictive than for sources of short duration and infrequent occurrence."

Frequency dependency between responding floor velocities and excitation ground motions also greatly complicates description of levels of ground motion for given levels of human response. Studies of human response employ floor velocity or acceleration for the measure of excitation. This floor vibration is not that of the ground and is significantly affected by the difference between the dominant frequency of excitation and the natural response frequency of the floor. As shown by the response spectrum of the 30 Hz dominant frequency ground motion in Figure 8 from the 8 m distant high speed train, a 5 Hz floor would respond less than a 12 Hz floor because 12 Hz is closer to the dominant excitation frequency. Thus in this two floor scenario there would have been <u>two</u> peak velocities of floor motion and <u>two</u> dominant floor motion frequencies to the same train!! Thus it is quite likely that the reaction of the same person to that ground motion would have depended upon which floor they were standing.

Present standards for tolerable levels for whole body motion are based on exposure to <u>continuous</u> sources lasting 60 seconds or longer (Ashley,1976, proposed amendment to International Standards Organization STD 2631). Therefore they are most applicable to situations involving continuous sources of construction vibration such as vibratory pile driving, rollers, and vibrofloatation. Since train induced ground motions are transient (not continuous) and last less than 1 to 2 seconds, these standards cannot be applied directly. Most importantly these standards are based upon floor response, and thus require a transformation from ground motions to floor motions regardless whether the motions are continuous or transient.

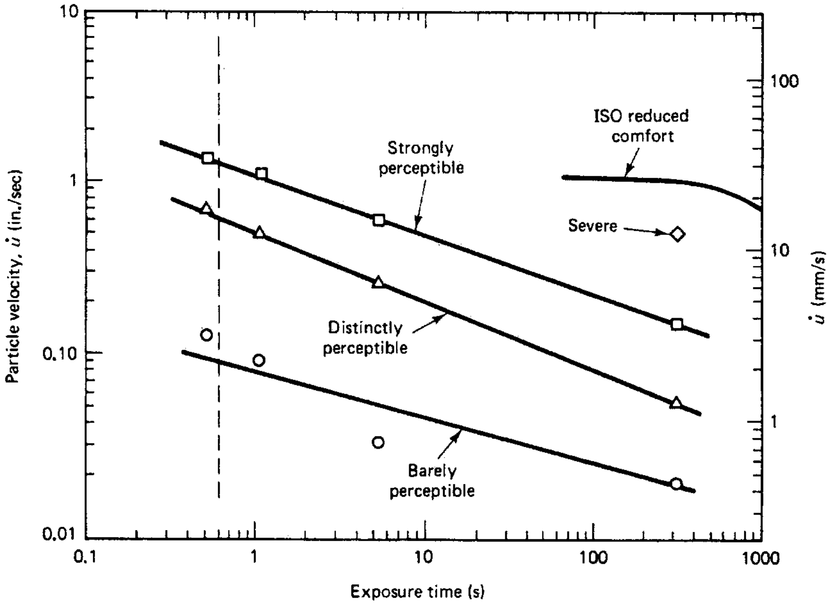

Figure 9. Experimental results showing increasing peak particle velocity for the threshold of human perception with decreasing duration of floor motions for short

Shorter transient motions are less perceptible and thus potentially less annoying than longer transient and continuous motions. This logical assumption was validated by Wiss and Parmelee (1974), and Siskind et al. (1980) have recast Wiss's results in a plot of exposure time and particle velocity, as shown in Figure 9. Responses are presented in terms of the mean responses. Thus, if "strongly perceptible" in the Wiss and Parmelee study is associated with "more than moderately annoyed" from Siskind's work, then a 1 second transient *floor* motion could be as high as 25 mm/s before producing that response in 50% of the exposed population. The particle velocity at which only 5% of those exposed would have thought the motion to be strongly perceptible can then be found from the standard deviation of Wiss's original data, and is 12 mm/s. The same technique will show that some 16% of Wiss's respondents had described a 1 second transient motion with a peak particle velocity of 11 mm/s as strongly perceptible. On the same chart the ISO standards (International Standards Organization, 1978) for 1 minute exposure are plotted for comparison.

7. PERCEPTION, ANNOYANCE, COMFORT, AND PROFICIENCY

Vibration guidelines established by standards organizations for avoiding adverse human response involve several differing thresholds and unfortunately (for impulsive event analysis) still rely upon continuous motion test results. It should be remembered that even though called standards, these are guides for the evaluation of existing or planned conditions. They are not prohibitions, but are guidelines. Consider the two American National Standards Institute (ANSI) guides for human exposure to vibration. (ANSI S3.18-1979 and ANSI S3.29-1983). The 79 guide addresses worker proficiency and passenger comfort during transport. In other words those who have chosen to participate and are exposed to relatively continuous motions. Whereas the 83 guide addresses those exposed to vibrations inside of buildings. In other words those who may not have chosen to be exposed. Unfortunately the 83 study continues to rely upon continuous motion data, but does allow an alternative interpretation for 3 or fewer impulsive events per day.

The 1983 guideline includes a residential site multiplier of 90 for impulsive motions which occur three or fewer times per day. The office site multiplier is 128. This allowance was included specifically in recognition of the difference between human response to continuous and singular, separated events that are associated with construction blasting. However, it would apply to any impulsive source lasting less than several seconds such as high speed trains.

The four descriptions of thresholds of human response, 1)perception, 2)annoyance, 3)comfort, and 4)proficiency require careful distinction least they be confused. The 83 guide bases annoyance or the threshold of complaint at the level of perception. The 79 guide focuses upon work related proficiency. Where upon there are two main descriptions of response. The threshold of inefficiency is defined as "fatigue decreased proficiency" (FDP) and the less severe response is reduced comfort (RC). FDP is defined as the loss of efficiency in the operation of machinery and RC as difficulties in eating, reading, writing as used in the transport industry.

As expected, human response is a function of the frequency of excitation, length of exposure, and environment as shown in Figure 10 and in Table 1 in the guideline. The frequency dependency of continuous vibration levels that trigger annoyance/perception is compared to fatigue decreased proficiency (FDP/min) and reduced comfort (RC/min) in Figure 10. As expected FDP and RC for short exposure times (1 minute in the figure)is reached at vibration levels some 100 times that of perception. This is approximately the same vibration level as the residential guide for 3 or fewer separate impulses (

BASE(1/0.7)(90)). Day long (24 hr in the figure) exposure significantly reduces the threshold of FDP. The threshold of the avoidance of annoyance (perception in these guidelines) as shown by the lowest two lines in the plot does not consider a minimum number of events except in the case of impulsive events for which there are 3 or fewer. The guides are so focused upon continuous motions that the thresholds are described in terms of root mean square (RMS) amplitudes that are most useful in describing harmonically, continuous excitation. The adjustment for zero to peak amplitudes (RMS x 1/ 0.7) normally measured in construction is also shown by the thin dotted line. Table 1 gives the weighting factors to adjust the suggested threshold levels for building types, time of day, and impulsive events.

Unfortunately intermittent vibrations and even more than 3 separate events per day are treated as the same as continuous vibrations in the 83 guide.

> "Intermittent vibration is a string of vibration incidents, each of short duration, of the order of two seconds or less, separated by intervals of much lower vibration magnitudes. They may result from sources that are regular (e.g. pile drivers, forging presses) or irregular (e.g. traffic, intermittent machinery, elevators). Repeated intermittent vibration should be treated as continuous vibration or the

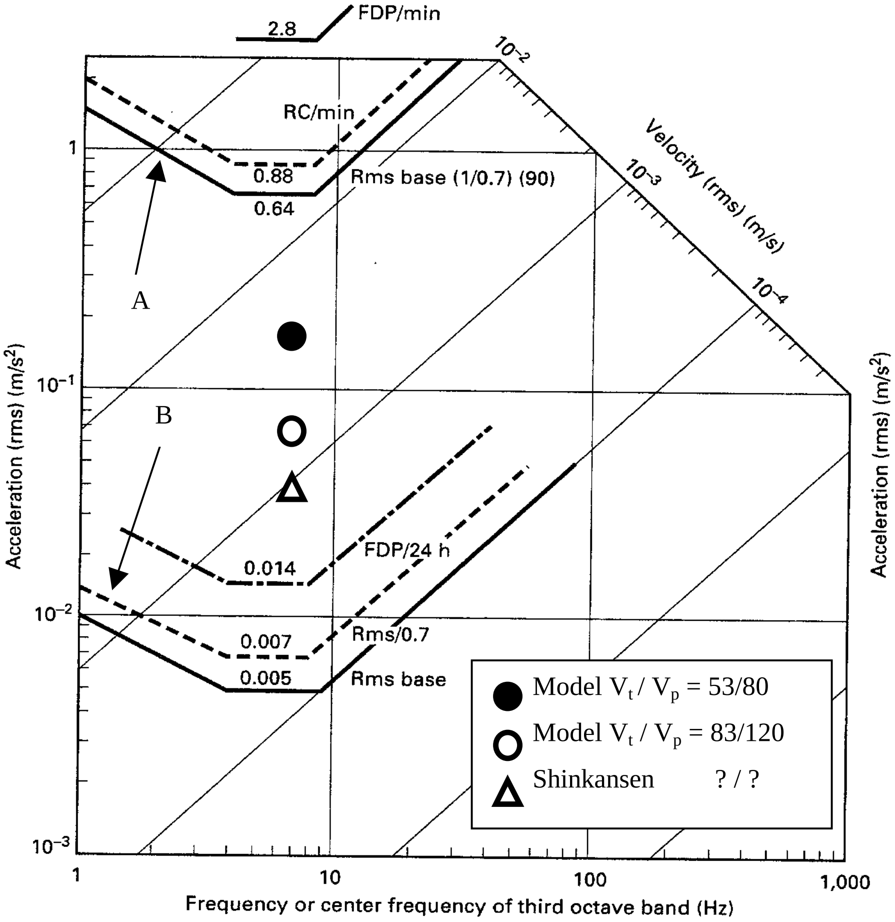

Figure 10. ANSI acceleration-velocity -frequency guidelines for acceptable floor vibration (based upon perception) compared to calculated and measured ground motions produced by high speed trains.

> purpose of this standard" "Continuous vibration is one which remains uninterrupted over a period of one minute or more."

More work seems to be needed to establish a scientific basis for the applicability of these standards for situations where vibration events are separated but may occur more than three times per day.

8. COMPARISON OF HIGH SPEED TRAIN INDUCED MOTIONS TO ANSI STANDARDS.

Peak frequency content and particle velocities for high speed train motions presented in Figures 4, 5 and 6 are compared with the ANSI standards in Figure 10. The ANSI standard is frequency sensitive as were the response spectrum bounds in Figure 8. The allowable zero to peak values for continuous motions (Rms/0.7 curve) is 0.1 mm/s for frequencies greater than 10 Hz. For impulsive motions this guide is multiplied by 90 for three or less events per day [Rms Base(1/0.7)(90) curve] is 12 mm/s.

The separation distance for high speed trains outrunning their ground motions ($V_t/V_g > 1.0$) can be found as the distance the 12 mm/s level intersects the Ledsgard field data in Figure 6. Thus these high speed trains would have to be separated by some 20 m from responding structures to avoid exceeding the standards – IF structural amplification is not included. Remember the field data in Figure 6 were measured on the GROUND but the standards are based upon floor motions, which may be amplified. Floors will amplify ground motion the most when excitation occurs at their natural frequency, 5 to 15 hz. Response spectra in Figures 4 & 5 indicate that dominant frequencies are in the 7 to 10 Hz range for ground motions in soft soils at 32 m from the tracks.

Expected structural amplification is needed where dominant frequencies of ground motions are near the natural frequencies of floors. Figure 4 shows a pseudo velocity response of 16.5 mm/s for motions at 32 m at 7Hz . The peak ground motions at this distance are 5.5 mm/s. Thus the amplification of these motions would be on the order of 3.3. Therefore at 32 m ground motions need to be multiplied by 3.3 to to compare with the guidelines. Consider the Ledsgard data in Figure 6, which show approximately 3 mm/s at 32 m. They could produce 10 mm/s floor motions.

9. CONCLUSIONS

Site specific time histories of ground motions generated by high-speed trains traversing soft soils are necessary both for validating models as well as investigating strategies for reduction of the motions and mitigating their effects. These motions can then be employed with models of responding structures to determine the range of possible effects as well as strategies for reduction of external effects. Models of the train-soil interaction are helpful in calculating resulting motions at both the track and at near-by structures. It appears that high-speed train generated motions can and will be perceived and may become annoying under special circumstances. Furthermore under special circumstances these motions may hinder operation of sensitive, research grade instrumentation. As with structural concerns, actual time histories of the ground motions are necessary to study and mitigate the effects of human and instrument response.

10. ACKNOWLEDGEMENTS

This article is truly an international effort and illustrates the degree to which electronic communication has shrunk the world. First of all I'd like to thank Prof Schmidt of the Ruhr University in Bochum, Germany for inviting me to give this paper, and have written it through the lense of American dynamic design guidelines. Special thanks are due to Gero Pflanz of the Dynamics of Structures lab at Ruhr University for development of the boundary element model employed in this paper. All of our data have been exchanged electronically over the Internet. Response spectra have been generated by Michael Louis of France and JJ Lee of South Korea (geotechnical graduate students at Northwestern University). I'd also like to thank Drs Osamu Yoshioka and Sunao Kunimatsu of the Japanese Railway Technical Research Institute, and National Institute for Resources and Environment for providing background for the Japanese Velocity Level (VL) measure. Finally special thanks are due to Svahn Björn and the library staff at Sweden's Banverket for supplying raw time histories of ground motions measured at Ledsgard.

11 REFERENCES

Adolfsson, K, Andreasson, B, Bengtsson, P & Zackrisson, P (1999a) High speed train X2000 on soft organic clay- measurements in Sweden, *Geotechnical Engineering for Transportation Infrastructure, Barands et al (eds), Balkema, Roterdam*

Adolfsson, K, Andreasson, B, Bengtsson, P, Bodare, A, Madshus, C, Massarch, R, wallmark, G & Zackrisson, P (1999b) Evaulation and analyses of measurements from the west coast line: Appendices 1-8, available from library of Banverket, Gothenberg, Sweden.

ANSI -American National Standards Institute- (1979) "ANSI S3.18-1979: Guide for Evaluation of Human Exposure to Whole-Body Vibrations," American Standards Institute; Acoustical Society of America, New York, NY, Secretariat of Committees 51, 52 and 53.

ANSI -American National Standards Institute- (1983) "ANSI S3.29-1983: Guide for Evaluation of Human Exposure to Whole-Body Vibrations," American Standards Institute; Acoustical Society of America, New York, NY, Secretariat of Committees 51, 52 and 53.

Ashley (1971), "Proposed International Standards Concerning Vibrations in Buildings," *Proceedings of Symposium on Instrumentation for ground Vibration in Earthquakes,* The Institution of Civil Engineers, UK

Blais, F. (1993), "Frequency and Energy Considerations for Construction Vibrations," M.S. thesis, Department of Civil Engineering, Northwestern University, Evanston, IL.

Dowding, C. H. (1996), Construction Vibrations, Prentice Hall, Upper Saddle River, New Jersey.

ISO - International Standards Institute (1978), "Guide for Evaluation of Human Exposure to Whole Body Vibrations," ISO 2631-1978 (E), 15 pp.

Pflanz, G (2000) Boundary element model to generate surface ground motions from high speed train loading, Personal communication, Ruhr University, Bochum, Germany

Kobayashi, Y, (1974) Vibration nuisance caused by running trains and its countermeasures, *Quarterly Report Railway Technical Research Institute,* Tokyo, Japan, Vol 15 No1.

Ramirez, M. R. (1986), "Analysis and Design of Floors to Prevent Objectional Vibrations," M.S. thesis, Department of Civil Engineering, Northwestern University, Evanston, IL.

Siskind, D. E., Stagg, M. S., Kopp, J. W., and Dowding, C. H. (1980b), "Structure Response and Damage Produced by Ground Vibrations from Surface Blasting," *Report of Investigations 8507,* US Bureau of Mines, Whashington, DC.

Ungar, E. E. and White, R. W. (1979), "Footfall-Induced Vibrations of Floors Supporting Sensitive Equipment," *Sound and Vibration*, Vol. 13, No. 10, pp. 10-13.

Veletsos, A & Newmark, N (1964), *Design Procedures for Shock Isolation Systems of Underground Protective Structures*, Vol III, *Response Spectra of Single Degree-of-Freedom Elastic and Unelastic Systems*, Technical Documentary Report RTD TDR-63-3096 Ad44989, Volume 111, Air Force Weapons Laboratory, Kirkland, AFB.

Wiss, J, & Parmelee, R (1974), "Human Perception of Transient Vibrations," *Journal of the Structural Division*, ASCE, Vol. 100, No. S74, pp.773-787.

Yoshioka, O, (2000) Personal Communication, Railway Technical Research Institute, Tokyo, Japan

APPENDIX: SINGLE DEGREE OF FREEDOM MODEL & RESPONSE SPECTRUM

I-1. STRUCTURAL ANALOGY

The cracking potential of ground borne vibrations can be discussed most accurately in terms of the response of structures to the passing of the vibration wave form. One of the critical response factors is the amount of differential movement that occurs between structural members or between different points on the same structural member because it causes strains which, in turn, cause cracking.

To compute the differential displacements that may occur in an actual structure or structural component, it is necessary to simplify a structure so that computations are practical. The simplest model that accounts for the dynamic interaction of the three simplified characteristics is the single degree of freedom (SDF) system shown in Figure I-1. The concentrated mass is analogous to the masses of the main components (floors); the spring represents the stiffness of the main components (walls), and the dashpot, through viscous resistance, models the dissipation of energy (connections). The differential movement, δ, is the difference between the absolute displacement of the mass, x, and the absolute displacement of the ground, u. Even multiple degree of freedom systems such as multiple storied structures may be idealized as a single-degree-of-freedom system if one is interested in the dominant or fundamental mode of response. Greater detail on the response of multiple degree of freedom systems can be found in tests on structural dynamics and earthquake engineering

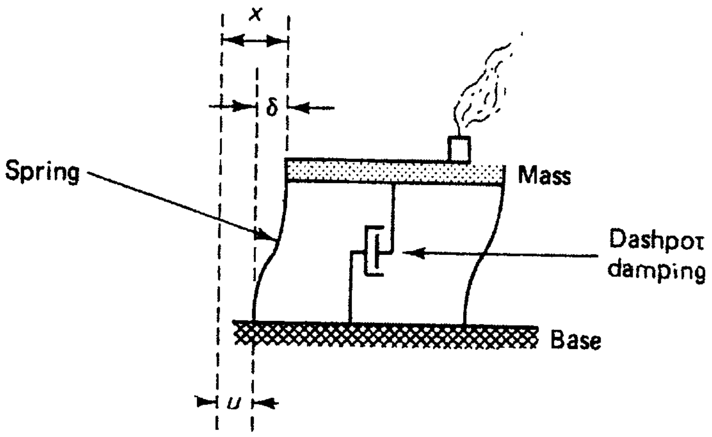

Figure I-1. Single degree of freedom model showing structural and model analogy.

I-2. MATHEMATICS OF THE SDF MODEL

The equation of motion for the SDF system in Figure I-1, when subjected to ground excitation, is

$$m\ddot{x} + c_1\dot{\delta} + k\delta = 0 \qquad \text{(I-1)}$$

where x is the absolute acceleration of the mass, m; c_1 the damping coefficient; the velocity of the mass relative to the ground; k the linear spring constant; and δ the relative displacement between the ground and the mass. Using the relationship for the relative displacement ($\delta = x - u$, shown in Figure I-1), Equation I-1 becomes

$$m\ddot{\delta} + c_1\dot{\delta} + k\delta = - m\ddot{u} \qquad \text{(I-2)}$$

The circular natural frequency of the undamped spring-mass system, p, is equal to $\sqrt{k/m}$. The fraction of critical damping, β, is equal to $\frac{c_1}{2\sqrt{mk}}$. If the mass is displaced from its equilibrium position, it will not oscillate when released but will simply return to its equilibrium position when c_1 is equal to $2\sqrt{mk}$. Under this condition the system is said to be critically damped. The circular natural frequency of the damped system, p_d, is equal to $p\sqrt{1 - \beta^2}$. Equation I-2 can be recast as

$$\ddot{\delta} + 2\beta p\dot{\delta} + p^2\delta = \ddot{u} \qquad \text{(I-3)}$$

in terms of percentage critical damping, β, and circular natural frequency, p. The ground-acceleration time history, which is to be integrated from time zero to time t, is represented by u (t).

Thus if a structure's undamped natural frequency, p, and its fraction of critical damping, β, are known, it is not necessary to define particular values of m, k, and c_1 in order to model the structure accurately. Furthermore, dynamic properties, p and β, can be more accurately measured from a free vibration time history of the building response than calculated from estimates of m, k, and c_1. These measured parameters automatically account for the factors that are difficult to quantify, such as the degree of fixity of the columns (which affects k) and the damping coefficient, c_1.

The preceding discussion dealt with the response of a particular structure to a particular ground motion. However, to distinguish different types of ground motions and their differing cracking potentials, it is necessary to compare the effect of the wave on a wide variety of structures. The response spectrum, which can be calculated from solutions to Equation I-3, provides a mechanism for this comparison.

Solution to Equation I-3 for relative displacements at any time may be expressed in terms of the Duhamel integral of the absolute ground acceleration time history as

$$\delta(t) = -\frac{1}{p\sqrt{1-\beta^2}}\int_0^t (\tau)\, e^{\beta p(t-\tau)} \sin[\,p_d(t-\tau)]\, d\tau \qquad \text{(I-4)}$$

where δ and are zero at t_0 (Veletsos and Newmark, 1964).

Equation I-4 yields the relative displacement response of an SDF system from a ground-acceleration time history. If a velocity time history is used as the input time history, the relationship between u and δ can be found by integrating Equation I-4 by parts and combining terms (Veletsos and Newmark, 1964). The resulting equation can be expressed as

$$\delta(t) = - \int_0^t \dot{u}(\tau) e^{\beta p(t-\tau)} \left[\cos\,[\,p_d(t-\tau)] - \frac{\beta}{\sqrt{1-\beta^2}} \sin\,[\,p_d\,(t-\tau)] \right] d\tau \qquad \text{(I-5)}$$

when δ and as well as *displacement, velocity, and acceleration* are zero at t_0.

I-3. CONSTRUCTION OF THE RESPONSE SPECTRUM

When a particle velocity time history such as that of the radial ground motions shown in Figure I-2b is processed by computer with Equation I-5, a relative displacement, “δ”, time history is calculated. In the calculated relative displacement time history there will be a maximum, $\delta_{\max}$.

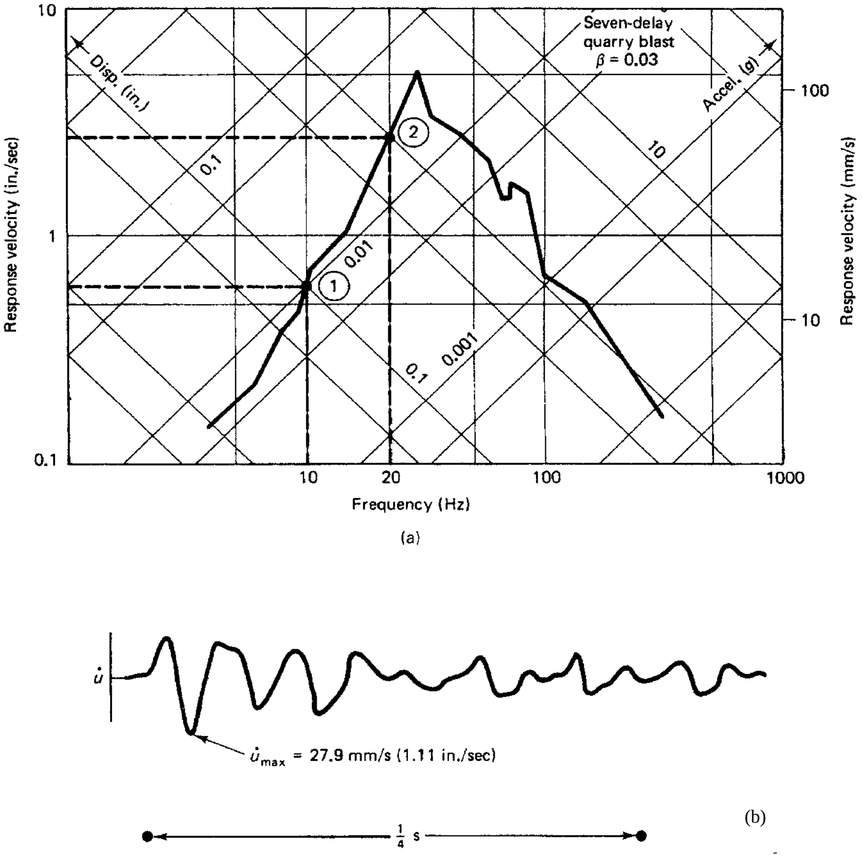

Figure I-2. Response spectrum (a) calculated from excitation ground motions (b) showing responses of systems with natural (fundamental) frequencies of 10 and 20 Hz

If that maximum relative displacement is multiplied by p, the structure's circular natural frequency (or $2\pi\ f_s = 2\pi\ (1/T)$),

$$PV = 2\pi\ f_s \cdot \delta_{\max} = p \cdot \delta_{\max}$$

is called the pseudo velocity (PV). This pseudo velocity is a close approximation of the relative velocity, , if the pulse associated with $\delta_{\max}$ is approximately sinusoidal

The pseudo velocity response spectrum of a single ground motion, such as that of the seven pulse motion in Figure I-2a, is generated from the $\delta_{\max}$ values of a number of different SDF systems when excited by that motion. Consider two different components of the same structure, a l0-Hz superstructure and the 20-Hz floor. If the ground motions, u *(t)*, of the seven pulse motion are processed twice by Equation I-5 with β, damping, held constant at 3%, and $f_s = 10$ and 20 Hz, two $\delta_{\max}$ values will result.

The first computation is made with the l0-Hz system, which has a circular natural frequency of

$$p = 2\pi(10)$$

and results in

$$\delta_{\max} = 0.25 \text{ mm } (0.01 \text{ in.})$$

This $\delta_{\max}$ is then converted to PV as

$$PV_{10} = p\delta_{\max} = 2\pi\,(10)(0.25) = 15.7 \text{ mm/s } (0.62 \text{ in./sec})$$

and is plotted as point 1 in Figure I-2a. The same computation is then repeated for the 20-Hz system.

$$p = 2\pi(20)$$

$$\delta_{\max} = 0.5 \text{ mm } \quad (0.02 \text{ in.})$$

$$PV_{20} = 2\pi\,(20)(0.5) = 63.5 \text{ mm/s } \quad (2.5 \text{ in./sec})$$

and PV_{20} is plotted as point 2 in Figure I-2a. If the ground motions are processed a number of times for a variety of f_s's with β constant, the resulting pseudo velocities will form the solid line in Figure I-2a.

The response spectrum in Figure I-2b is plotted on four-axis tripartite paper. These four axes take advantage of the sinusoidal approximation involved in calculating a pseudo velocity. The axis of the maximum relative displacement, δ, is inclined upward to the left and is the pseudo velocity (PV) divided by $2\pi\phi_\sigma$. The pseudo acceleration (PA) axis is inclined upward to the right and is PV times $2\pi\phi_\sigma$. PA and PV are called pseudo acceleration and pseudo velocity because they are sinusoidal approximations. However, these simplifications closely approximate the absolute acceleration of the mass and the relative velocity for systems with small β values (Veletsos and Newmark, 1964).