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SCIENCE APPLICATIONS, INC.

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ENGINEERING GUIDELINES
FOR THE ANALYSIS OF
TRAFFIC-INDUCED VIBRATIONS

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16. Abstract

These guidelines present a methodology for assessing the potential for adverse environmental impact resulting from seismic vibrations induced by traffic operating on highways. Vehicle/pavement interaction, propagation of vibration, response of building structure, and evaluation criteria are presented in quantitative engineering terms. Abatement strategies, measurement, and analysis of traffic-induced vibration are discussed.

Pavement surface roughness, vehicle speed, and vehicle gross weight are the important parameters describing traffic-induced vibration. Example problems and analyses are presented.

Highway engineers and planners familiar with techniques used to evaluate traffic noise impact should easily understand and use these guidelines.

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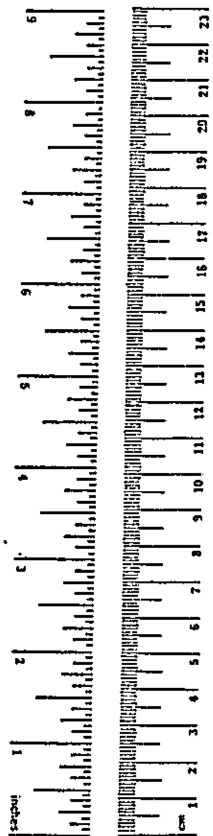
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

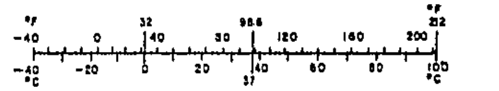
Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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1.0 INTRODUCTION

These engineering guidelines present recommended procedures for the prediction, measurement, and analysis of ground vibration related to highway traffic operations. The guidelines are concerned with highway pavements in direct contact with the subgrade. Engineering parameters, familiar to highway design engineers, are utilized. The procedures for the prediction, measurement, and analysis of traffic-induced vibration are an exact parallel to the consideration of highway traffic noise.

1.1 Objective of the Guidelines

These guidelines present a step-by-step description of the procedures recommended to assess the potential for adverse environmental impact from ground vibration generated by highway traffic operations.

1.2 Scope of the Guidelines

These guidelines present the primary considerations required to evaluate highway traffic-induced vibrations. The details upon which the guidelines are based are presented in Reference 1. All procedures and supporting data required to utilize the guidelines are presented in this report.

1.3 Brief History of Traffic-Induced Vibrations

Environmental vibration resulting from highway traffic has been investigated and reported in the technical literature for the past 30 years. The earliest reference to traffic induced vibration appears to be Barnhard in 1941.* Southerland (1950) investigated

* Appendix I of Reference 1 contains an annotated bibliography of these references.

vibrations produced in residential structures from buses. Southerland's test data was based upon driving buses over a short ramp to induce "controlled" impact loading on the pavement. (See Sections 2.3.3 and 3.4 of these guidelines). Southerland developed, perhaps the first defensive abatement strategy for traffic-induced vibration complaints — good public relations. Additionally, Southerland utilized building inspections as a basis for evaluating complaints of alledged damage from traffic-induced vibration. By comparing structural conditions of buildings of similar construction and age, he was able — through public relations — to convince a majority of the complaining population that traffic-induced vibrations were not the cause of alledged building damage.

Steffens (1952) presented test data results in his summary of various proposed methods for assessing vibration intensity as related to criteria. This test data presents results of highway traffic-induced ground vibration measurements conducted in England as far back as 1929.

In the past four years, most of the technical research relating to traffic-induced vibrations has been conducted in Europe and Japan. Bata (1971) reported extensive studies conducted in Czechoslovakia concerning traffic-induced vibrations. In England, Whiffin and Leonard (1971) of the Transport and Road Research Laboratory reported the results of a survey project on traffic-induced vibration. House (1973) presented a discussion of the possible factors defining excitation, propagation, and building response as related to traffic-induced vibration. In addition, House presented a rather extensive survey of building damage data as related to vibration. In 1973, Tokita presented data ranking various vibration sources as to their magnitude and distance attenuation effects. Tokita described a frequency weighting characteristics for acceleration that is analogous to the A-weighting characteristic used in environmental noise.

In 1975, Tokita again described the results of experimental work conducted in Japan related to ground vibration generated by highway traffic. This result is, apparently, the first reference in the open technical literature that reported a quantitative vibration emission level prediction. Tokita identified the pavement surface roughness, vehicle weight and vehicle speed as the relevant parameters.

Although not directly related to traffic induced vibration, the United States National Academy of Sciences published (1977) "Guidelines for Preparing Environmental Impact Statements on Noise". The National Academy of Sciences (NAS) Guidelines contain criteria levels for human perception and annoyance and potential for building damage as related to environmental seismic vibrations. The criteria presented in this report are consistent with the NAS proposed criteria.

Today, the general topic of environmental seismic vibrations is void of standardization. Neither the metric(s) describing the amplitude and frequency content or a measurement methodology are internationally accepted.

This report identifies specific engineering parameters describing excitation, propagation, and building response for traffic-induced vibration. Mixed traffic flows are considered. The assessment of traffic-induced vibration is a very site specific problem as described in the following sections.

1.4 Historical Bibliography

The references cited in Section 1.3 are listed in chronological order as follows:

Barnhard, R.K.: "Noise Tremor Due To Traffic", Journal of the Acoustical Society of America, Vol.12, January 1941, pp. 338-347.

Southerland, H.B.: "A Study of the Vibration Produced in Structures by Heavy Vehicles", Highway Research Board, Proceedings of the 30th Annual Meeting, 1950.

Steffens, R.J.: "The Assessment of Vibration Intensity and Its Application to the Study of Building Vibration", National Building Studies Special Report No. 19, Department of Scientific and Industrial Research, Building Research Station, London, 1952.

Bata, M.: "Effects on Buildings of Vibrations Caused by Traffic", Building Science, Vol. 6, 1971, pp. 221 - 246.

Whiffin, A.C. and Leonard, D.R.: "A Survey of Traffic-Induced Vibrations", Road Research Laboratory Report, LR 418, 1971.

House, M.E.: "Traffic-Induced Vibrations in Buildings", Journal of the Institute of Highway Engineering, Vol. 20, No. 2, February 1973, pp. 6-16.

Tokita, Y.: "Ground Vibrations Generated by Factorie's Machine and Vehicles", Inter-noise '73, Tech. Univ. of Denmark, Copenhagen, Aug. 22-24, 1973, pp. 85-89.

Tokita, Y. and Oda, A.: "On the Characteristics of Ground Vibration Generated by Traffic", Inter-noise '75, Tohoku University, Sendai 980, Japan, August 27-29, 1975.

Anon: "Guidelines for Preparing Environmental Impact Statements on Noise", National Academy of Sciences, Washington, D.C., 1977.

2.0 CHARACTERIZATION OF TRAFFIC-INDUCED VIBRATIONS

The characterization of highway traffic-induced vibration is analogous to the characterization of highway traffic noise. Both traffic noise and traffic-induced vibration comprise a source-path-receiver scenario. For both traffic noise and traffic-induced vibration, each vehicle appears experimentally as a moving point source on the roadway and is modelled as such (1).*

For highway traffic noise, the noise source is defined by the traffic flow and the highway alignment relative to the receiver. For highway traffic-induced vibration, the vibration source is defined by the traffic flow, the pavement surface roughness, details of the pavement/subgrade structure, and the highway alignment relative to the receiver. For both highway traffic noise the traffic-induced vibration, the more significant traffic flow parameters are the vehicle speed and weight. Levels of traffic noise and vibration both increase with increasing vehicle speed and weight.

Whereas highway traffic noise analysis identifies the vehicle as the primary noise source, highway traffic-induced vibration must consider the vehicle-pavement system as the primary vibration source. The pavement surface roughness is the primary highway design (or condition) parameter affecting traffic-induced vibration.

Away from the highway alignment, both traffic noise and vibration decrease in amplitude with increasing distance. Generally, traffic noise is not an environmental consideration beyond 1000 feet (305 meters) from a roadway. It appears that traffic induced vibration is not an environmental consideration beyond 200 to 300 feet (61 to 90 meters) from the roadway.

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* Numbers in ( ) denote references listed at the end of the report.

Considering the receiver to be an occupant or an activity in a building adjacent to a roadway, the main difference between traffic noise and vibration becomes evident. Building structure attenuates or decreases the amplitude of traffic noise from the exterior to the interior. Highway traffic-induced vibration, as received at the building foundation, may cuase the building structure to amplify the vibration. Depending upon the vibration amplitude received at the foundation and the amplification characteristics of the building structure, the floor or wall vibration inside a building may be perceptible to an occupant. Whereas traffic noise may be perceptible but not annoying to a building occupant, it appears that perception of traffic-induced vibration may result in complaints, concerted public action, and the potential for litigation.

The general public may well confuse the separate issues of traffic noise and traffic-induced vibration. On a single event basis, noise from a vehicle may induce perceptible exterior wall vibration and resulting "rattles" inside a building. Similarly, traffic induced vibration may be imperceptible to an occupant but generate annoying "rattles" inside a building. Sound levels great enough to result in perceptible building vibration are so loud that the noise is the dominant annoyance factor. Perception of building vibration via the rattling noises generated in a building, however, may be extremely annoying to an occupant.

Figure 2-1 presents an outline of the traffic-induced vibration problem illustrating the interrelationship between source, path, and receiver parameters. Figure 2-2 presents a graphic comparison of highway traffic noise and highway traffic-induced vibration.

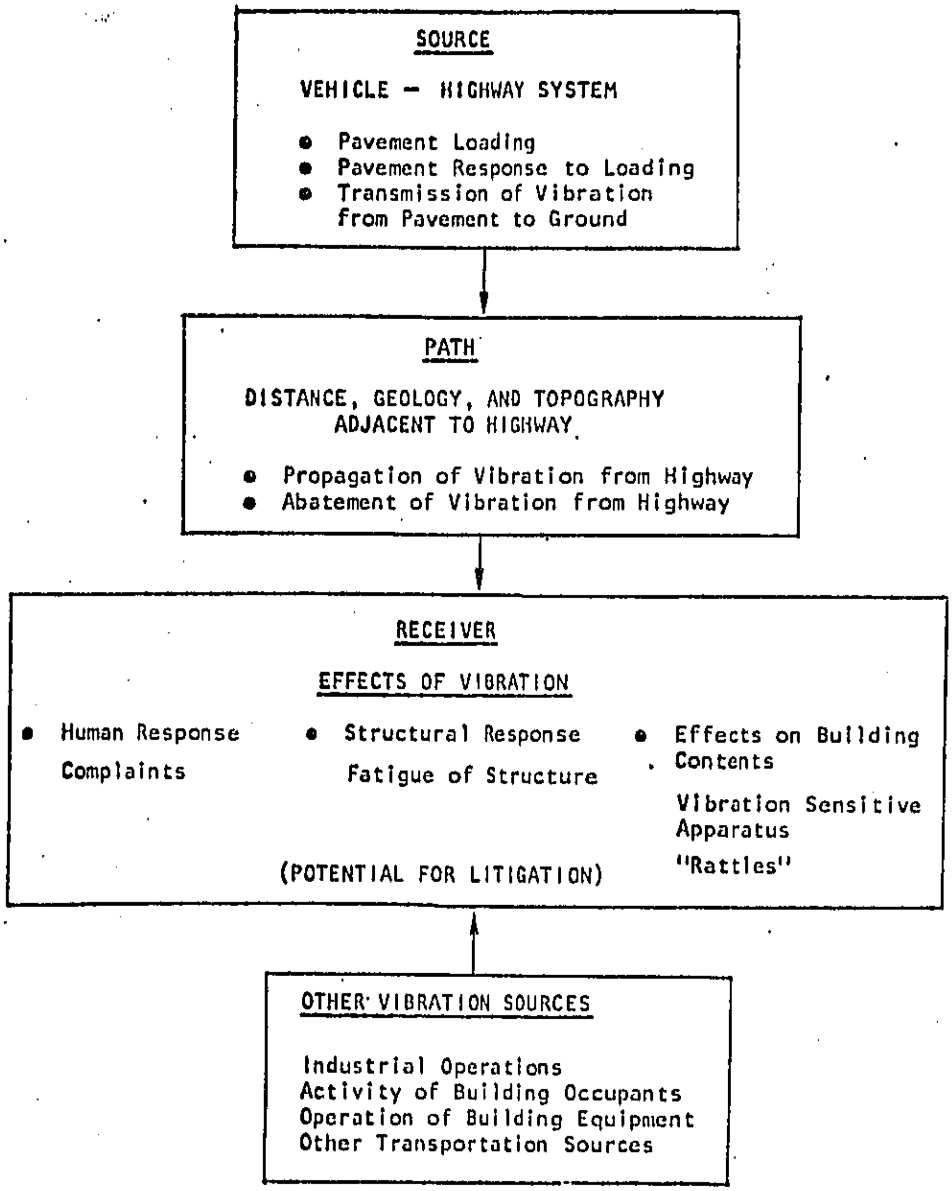


Figure 2-1 TRAFFIC-INDUCED VIBRATIONS: OUTLINE OF PROBLEM

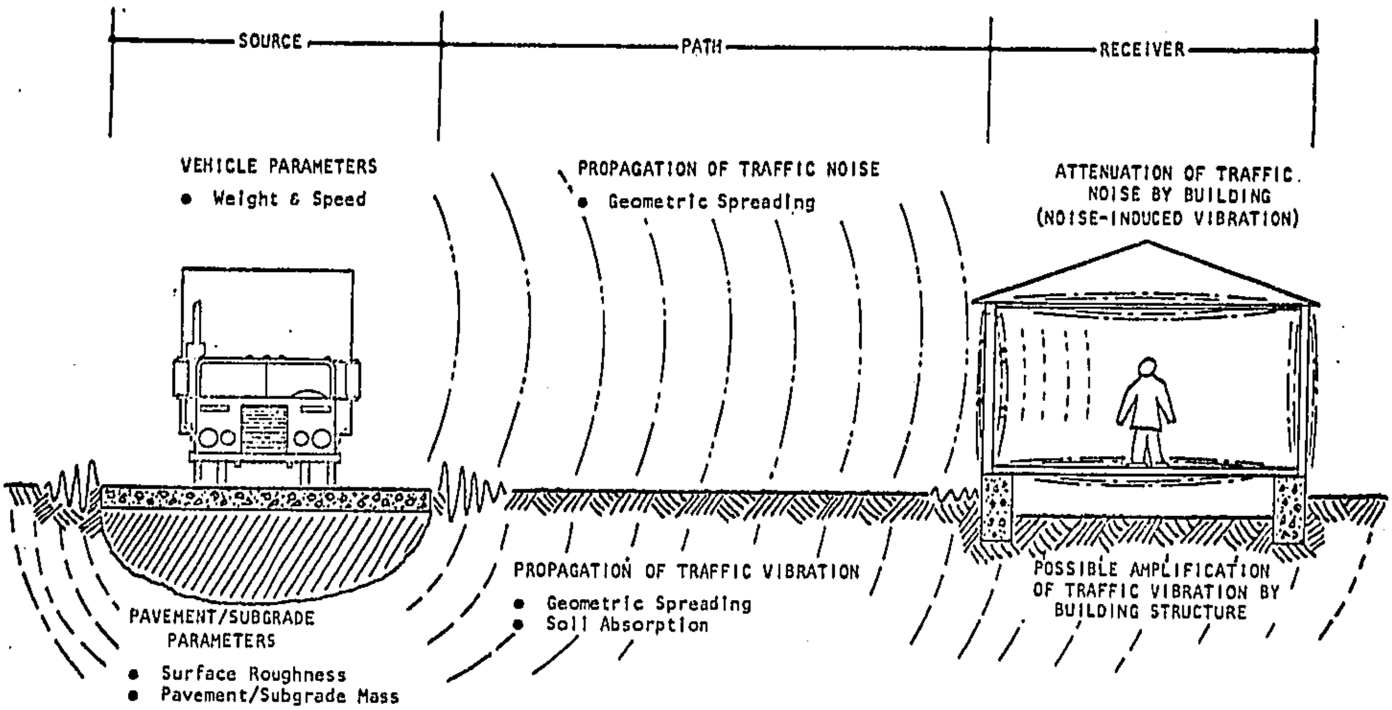


FIGURE 2-2 - SOURCE-PATH-RECEIVER SCENARIO FOR TRAFFIC NOISE AND VIBRATION

2.1 Sound Level and Vibration Level

The decibel measure is conveniently used for both traffic noise and traffic-induced vibration. The concept of an equivalent (energy mean) level, expressed in dB, may also be used to characterize both traffic noise and traffic-induced vibration.

For atmospheric noise, the physical quantity that is conveniently measured for amplitude description is acoustic pressure. For vibration, the physical quantities that are conveniently measured for amplitude description are displacement, velocity, or acceleration. If the frequency content of the vibration signal is known, displacement, velocity and acceleration are all related.

Acceleration is used as the vibration amplitude measure in these guidelines. The acceleration levels, expressed in dB, are referenced to an accleration of "1g" or 9.807 meters/(second)^2. Peak acceleration amplitudes associated with traffic-induced vibration are on the order of 0.001g to 0.030g. Hence, peak acceleration levels are on the order of -60 dB (re. 1g) to -30 dB (re. 1g). Using this convention, acceleration levels cannot be confused with noise levels from traffic.

The definition of the acceleration level used in these guidelines is:

$$L = 10 \cdot \log \left(a^2/a_o^2\right) = 20 \cdot \log \left(a/a_o\right), \text{ dB} \qquad (2\text{-}1)$$

where a is the acceleration measured
a_o is the reference acceleration (taken as 1g or 9.807 meters/second^2).

Acoustic pressure characterizes the sound level at a point. For vibration, however, a complete description of the motion at a point requires six amplitudes corresponding to three longitudinal components and three rotations. From a practical standpoint, one need not be generally concerned with a complete description. For traffic-induced vibration rotational motion at a point may be ignored. However, one must always note and report the directions in which vibration measurements are taken. Usually, only one component or direction of the vibration need be estimated or measured. For traffic-induced vibration it appears that it is sufficiently accurate to use the vertical component of ground motion or the component normal to the line or plane of least stiffness of structures.

2.2 Criteria for the Evaluation of Vibration Impact

Traffic noise is well defined with respect to both a level metric and associated criteria. Traffic noise levels are expressed in terms of the A-weighted sound level. The A-weighted sound level is a single number based upon frequency weighting of the sound pressure. Accepted values of the A-weighted sound level have been developed over the past several years that relate sound level to effects of noise on people and structures.

Environmental vibration is not so well defined as environmental noise with respect to either a level metric or the associated criteria. Standardized frequency weighting for vibration, such as the A-weighted sound level for noise, is not available at the present time. Standardization of frequency weighting for acceleration does appear to be possible in the near future (2).

Criteria relating vibration amplitude to effects on people and structures is available. Table 2-1 presents a criteria description and related acceleration levels. For comparison, sound levels corresponding to the similar criteria description for noise are also presented.

The listing in Table 2-1 is important in quantifying and understanding highway traffic-induced vibration. The levels presented in Table 2-1 indicate that from perception to a high probability of structural damage, acceleration levels cover a 50 dB range and sound levels cover a 140 dB range. For both noise and vibration, the threshold for structural damage is a higher level than the level for extreme annoyance. For noise, the range between extreme annoyance and structural damage threshold is approximately 30 dB (a factor of 32 in pressure). For vibration, this range is approximately 10 dB (a factor of 3.2 in acceleration). For noise, the range between perception and annoyance is approximately 60 dB (a factor of one thousand in pressure). For vibration, the range between perception and annoyance is about 5 dB (a factor of 1.8 in acceleration).

Hence, for evaluating the effects of environmental vibration, the significance of a change in acceleration level is much greater than an identical change in sound level. Whereas, a 1 dB change in sound level either in measurement or prediction is generally considered insignificant, 1 dB change in vibration level may be very significant.

Figure 2-3 presents criteria curves for both human response and building response as a function of frequency. The vibration metric used in Figure 2-3 is the acceleration level expressed in dB (re: 1g). The use of Figure 2-3 requires that the acceleration spectrum level be used (i.e., the level in a frequency bandwidth 1 Hz wide). The conversion of either octave band or one-third octave band levels to spectrum levels is described in Section 4.0 of these guidelines.

Table 2-1

COMPARISON OF NOISE AND VIBRATION LEVELS AND CRITERIA

NOISE Sound Level dB (re. 2×10^{-5} N/m^2)	Criteria Description	VIBRATION Acceleration Level dB (re. $1g_{rms} = 9.8m/s^2$) (Approximate)
0	Threshold of Perception	-65
55-65	Annoying	-60
90	Extremely Annoying	-45
120	Threshold of Structural Damage*	-35
130	Structural Damage of Concern*	-25
>140	Structural Damage Highly Probable*	>-15

* The nature of structural damage is generally a fatigue effect. That is, cracks and damage slowly progress over a period of time.

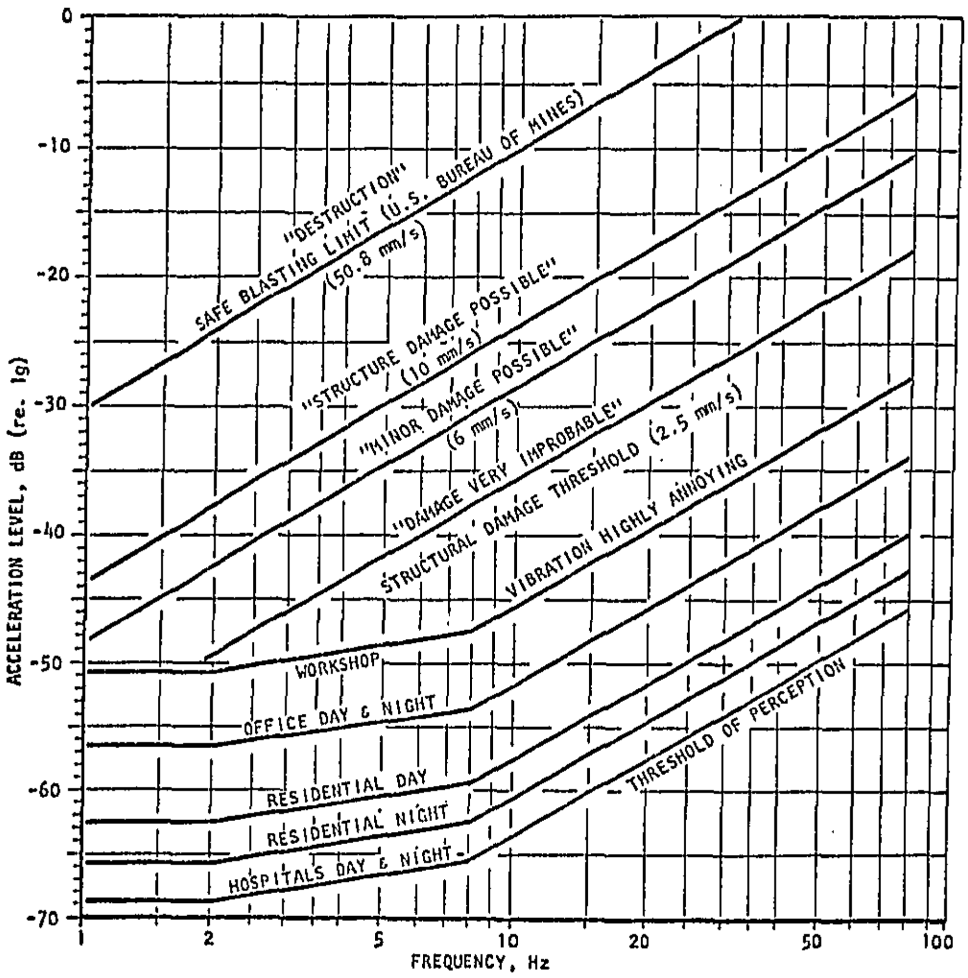


FIGURE 2-3 - CRITERIA CURVES FOR ENVIRONMENTAL VIBRATION:
ACCELERATION LEVEL VERSUS FREQUENCY

Traffic-induced vibration is generally a transient discrete frequency signal. Hence, the peak overall acceleration level will be quite close to the spectrum level at the discrete frequency.

Traffic-induced ground vibration and building response field test data generally exhibit maximum levels in the frequency range of 5 Hz to 30 Hz. From Figure 2-3 it is seen that the criteria curves all increase approximately 16 dB between 5 Hz and 30 Hz. That is, for both human response and building response criteria, higher acceleration levels are permitted at higher frequencies.

The discrete frequency excitation that generally characterizes traffic-induced vibration appears to be related to the vehicle tire/suspension system characteristics. This excitation occurs, generally, in the range of 5 Hz to 20 Hz. The ground motion appears to be forced non-resonant vibration of the pavement/subgrade system. Generally, the pavement/subgrade fundamental natural frequency appears to be in the range of 25 Hz or greater.

Acoustic noise from the vehicle source may induce both ground vibration and excitation of building structure. This excitation occurs generally in the frequency range above 40 Hz. The source of this excitation is the vehicle exhaust and other discrete frequency components related to the engine cooling system. Generally, vibrations generated by the airborne path are measurable but are too low in level at their excitation frequency to be perceptible. (See Section 3.6 and 4.0 of these guidelines.)

2.2.1 Human Response Criteria

The criteria presented in Figure 2-3 for human response to vibration are those recommended by the Reference 2 study. They have been compared and agree with criteria proposed by other researchers as reported in Reference 1. These criteria are generally based upon laboratory tests aimed at defining comfort and functional agility boundaries for vehicle passengers. Most of the data, indeed, is based upon subjective reactions of young male military aircraft pilots. Many of the psychological factors required to translate the criteria to an occupant in a house are described in Reference 1. The criteria presented in Figure 2-3 for human response to environmental vibration is believed adequate for the purposes of evaluating the environmental impact of traffic-induced vibration.

Two important points must be remembered, however. First, the perception criteria level and the high annoyance criteria level are separated by 20 dB. This level range is easily within the range of vibration levels generated by various traffic operating conditions. Secondly, since the vibration criteria levels are based, in general, upon subjective reactions of vehicle passengers, vehicle design and vehicle operation on the highways utilize the same criteria. As described in Section 2.3, pavement surface roughness is a primary variable defining traffic-induced vibration. For example, high speed travel on a very rough road may result in an estimate of perceptible vibration to a highway neighbor, but the vehicle generally would not be operated at this condition since the passengers would perceive a very "rough" ride and slow down the vehicle.

Also, the user of these guidelines should recognize that the 5 Hz to 20 Hz frequency range associated with traffic-induced vibration is not the result of chance. Vehicle suspension systems are designed to resonate at frequencies above the most sensitive frequency range for passenger comfort (i.e., above 5 Hz).

2.2.2 Building Response Criteria

The criteria presented in Figure 2-3 for building response to vibration are those recommended by the Reference 2 study. They have been compared and agree with criteria proposed by other researchers as reported in Reference 1. These criteria are levels of constant vibration velocity. Dynamic stresses induced in structures are proportional to the velocity amplitude (3). The nature of the building damage from man-made environmental vibration is generally of the form of a fatigue failure over a long time period. The criteria levels presented in Figure 2-3 reflect this consideration.

Traffic-induced vibration is not the only source of environmental vibration potentially damaging to building structure. The highway engineer faced with evaluating alleged building damage must consider all vibration sources both external and internal to the building.

One aspect of traffic-induced vibration that may be a unique consideration for alledged building damage is the settlement of footings or foundations. Low level ground vibration may result in irregular foundation settlement over a long time span (perhaps years). The evaluation of this aspect of alleged building damage is a specialized soil mechanics problem. As such it can be evaluated only on the basis of detail analysis of the conditions at each site. Data presented in Reference 4 may help in providing guidance in this area.

2.2.3 Multiple Intrusions

The detail discussion of criteria levels for human response and building response presented above relates to a single event. For multiple events occurring during a daily (24 hour) period, the permissible limits for vibration decrease with increasing number of occurences. Additionally, lower criteria levels are recommended during nighttime (2200 to 0700 hours) than during the daytime (0700 to 2200 hours). Criteria acceleration levels for building interiors in residential areas are presented in Figure 2-4 in terms of the number of occurrences per day. As described in Reference 1, the complaint assessment conducted related to traffic-induced vibrations indicated that 90% of the complaints registered were associated with individual owner-occupied residences.

2.3 Parameters Governing Traffic-Induced Vibration

The most difficult task in evaluating the potential for adverse environmental impact from traffic-induced vibration is understanding the interrelationship between the significant parameters. The criteria levels presented in Section 2.2 indicate a range of approximately 20 dB between the perception level and the level of high annoyance. Hence, the prediction accuracy and the measurement accuracy associated with the analysis of traffic-induced vibration is rather stringent as compared to traffic noise analysis.

2.3.1 Traffic Parameters

Gross vehicles weight and vehicles speed are the primary traffic parameters affecting ground vibration from highway operations. Ground vibration levels increase approximately 3 dB for each doubling of gross vehicle weight. The effect of vehicle speed depends upon the pavement roughness. Increasing vehicle speed increases ground vibration levels within the limits of 3 dB to 6 dB per doubling of speed. An appropriate design value appears to be 5.2 dB per speed doubling. Since speed effects are dependent upon surface roughness, the evaluation of the relationship between vehicle speed and ground vibration level must be conducted on a site specific basis.

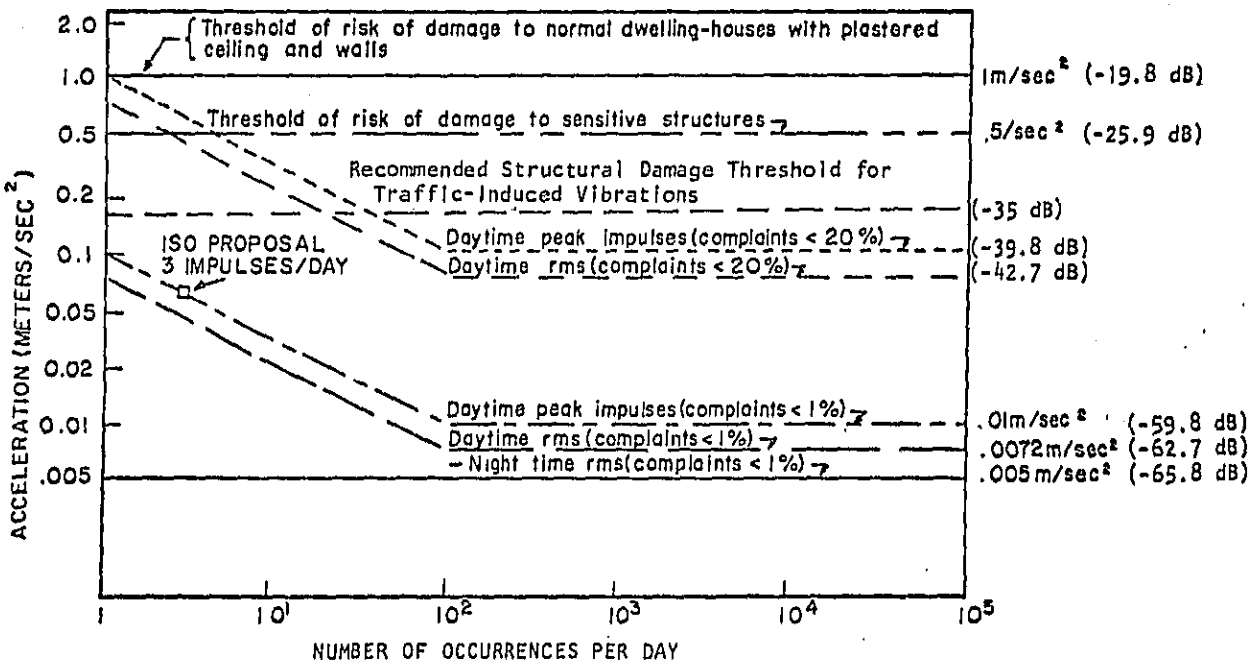


Figure 2-4. VIBRATION CRITERIA FOR BUILDING INTERIORS IN RESIDENTIAL AREAS (REF. 2 WITH ANNOTATION)

As indicated in Figure 2-4, the criteria levels associated with evaluation of traffic-induced vibration are related to a number of occurrences of a level during 24 hours. Since heavy vehicles will define the maximum ground vibration levels of the traffic flow, the criteria of Figure 2-4 place a constraint on the number of heavy vehicles in the traffic flow. This consideration is described in Section 3.6 of the guidelines under abatement strategies.

2.3.2 Pavement/Subgrade Parameters

Pavement surface roughness is the primary factor affecting ground vibration generated by highway traffic. Almost all measures of pavement surface roughness currently used by highway engineers are not applicable for a quantitative description of traffic-induced vibration. However, as reported in Reference 1, it appears possible to relate the Present Serviceability Rating (PSR) index of a pavement to the quantitative description of traffic-induced vibration. This approximation involves relating the PSR index to a pavement roughness power spectral density description. The details are presented in Reference 1. Other references describing the concepts of pavement roughness power spectral density are available in the open literature (5), (6).

As an approximation, traffic-induced vibration increases 4.2 dB with each unit decrease in the Present Serviceability Rating (PSR) index. For new pavement, an appropriate PSR value is 4.5 with a design life, for example, of 20 years to reach a PSR of 2.0. Thus, all traffic parameters held constant, traffic-induced ground vibration would increase about 10.5 dB over the design life of the pavement. On this point, however, one must always remember that the relation between speed and vibration level appears to depend upon surface roughness. The present availability of field test data does not allow this point to be further quantified.

Considering a typical suspension system natural frequency of 12 Hz and vehicle operating speeds between 15 to 60 mph (24 - 97 km/h) typical pavement roughness wavelengths on the order of 2 to 8 ft/cycle (0.6 to 2.4 m/cycle) appear to be most significant for the traffic-induced vibration problem. Roughness amplitudes on the order of 0.25 inches (6 mm) appear to be significant enough to cause perceptable ground vibration at the edge of the pavement.

Figure 2-5 presents a comparison of relationships between the PSR surface roughness index, as used in these guidelines, and the pavement roughness power spectral density (vertical axis). The horizontal axis is the reciprocal wavelength of the roughness. For comparison, representative values of the pavement roughness power spectral density functions for gravel roads and off-highway terrain are presented. Details are discussed in Reference 1.

A secondary parameter relating the pavement/subgrade system to traffic-induced vibrations is the pavement/subgrade mass. This mass parameter is a function of densities of the pavement material and subgrade material, the pavement width, the soil support value, and pavement thickness. Considering typical values of pavement and subgrade parameters, it appears that optimum compaction (density increase) of the subgrade/base materials is the only parameter controllable on an economic basis.

Pavement surface roughness and pavement/subgrade parameters are two aspects of traffic-induced vibrations that render the problem a site-specific consideration.

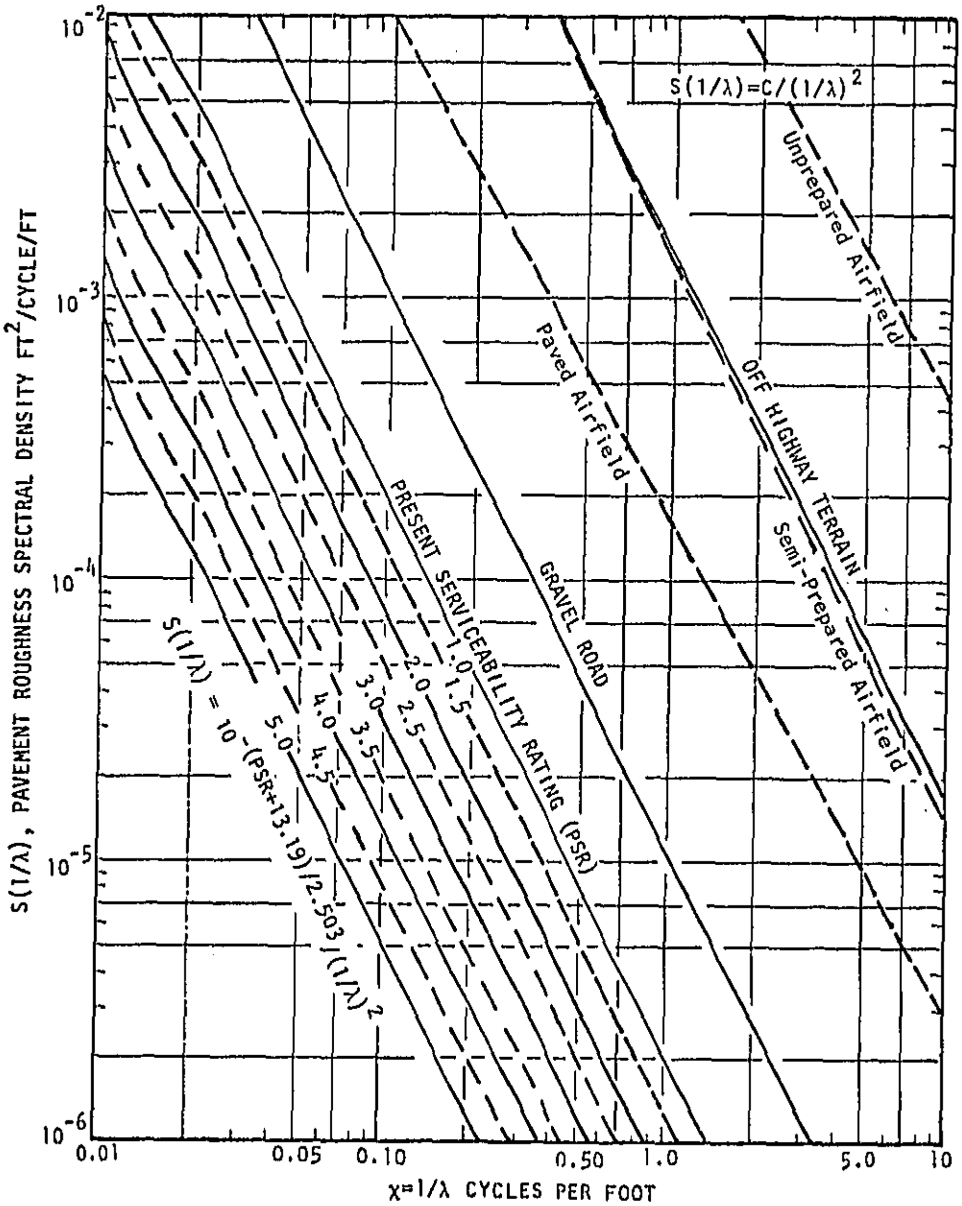


Figure 2-5. PAVEMENT ROUGHNESS POWER SPECTRAL DENSITY FUNCTIONS

2.3.3 Pavement Loading

In developing a quantitative description of traffic-induced vibration, effects of vehicle-pavement interaction and the resulting pavement dynamic loading were obtained. These results are presented in Reference 1. Since dynamic loading of the pavement is proportional to the acceleration of the pavement/subgrade mass, comments concerning vehicle speed and pavement surface roughness apply equally to pavement dynamic loading and acceleration level.

2.3.4 Potholes and Other Discrete Bumps

The pavement surface roughness discussed in Sections 2.3.2 and 2.3.3 corresponds to a random roughness distributed over a length of roadway. Potholes, manhole covers, rumble strips, etc. are forms of highway surface roughness that are discrete in character. A generally smooth highway may exhibit isolated potholes etc. that represent isolated sources of traffic-induced vibrations. Since source-receiver distances are constant for this type of surface discontinuity, the consideration of potholes is different from that of random surface roughness. Abatement of traffic-induced vibrations for a "pothole" source is obvious - repair the localized discontinuity.

Discrete surface irregularities are a completely different consideration than the consideration of random surface roughness. First, dynamic pavement loading resulting from a tire contacting a pothole or a ramp may approach 180% of the static wheel load. For random pavement roughness the dynamic pavement loading is no more than 10 to 20% of the static wheel load. Secondly, increasing vehicle speed may result in decreasing dynamic pavement loading for a pothole or ramp. This is contrary to the corresponding situation for random surface roughness.

These points are emphasized because many researchers have utilized ramps or planks secured to pavements to generate and to report ground vibration data related to traffic-induced vibration. Unless the ramp or bump configuration is typical of the highway design characteristics, such as an expansion joint, the consideration of such irregularities should be avoided in evaluating the potential for adverse impact from traffic-induced vibration.

Section 3.4 of these guidelines is devoted to the quantitative description of pothole effects.

2.3.5 Propagation Parameters

The propagation of traffic-induced vibration away from the highway depends upon the soil characteristics and conditions between the pavement and the receiver. Ground vibration from highway traffic decreases with distance away from the highway much more rapidly than traffic noise. In general, it appears that distances beyond 200 to 300 feet (62 to 91 meters) from a highway need not be considered for adverse impact from traffic-induced vibration.

In general, traffic noise attenuates over open terrain at a rate of 3 dB per distance doubling (geometric spreading from a line source) with a possible 1.5 dB per distance doubling excess attenuation for "soft site" absorption. Distance attenuation for traffic-induced ground vibration is totally absorptive. Hence, distance attenuation effects cannot, in general, be quoted in dB per distance doubling.

Additionally, the absorptive effects of soils in attenuating ground vibration are highly frequency dependent. The higher frequency components of the ground vibration attenuate much more rapidly with distance than the low frequency components.

Figure 2-6 presents four acceleration power spectra related to a bus passing along a two lane residential street. Rough pavement had resulted from repair of a sewer line underneath the near lane. Each spectra is identified as to distance from the edge of the road. This illustration clearly indicates a more rapid distance attenuation for the high frequency vibration.

At 12 Hz, the roadside acceleration level is approximately -55.9 dB (re. 1g) ($10 \cdot \log(2.6 \times 10^{-6})$) and -64.6 dB (re 1g) ($10 \cdot \log(3.5 \times 10^{-7})$) at the footing. At 43 Hz, the roadside acceleration level is -65.5 dB and at the footing is -90 dB.

Compared to the vibration criteria levels presented in Table 2-1 and Figures 2-3 or 2-4 it is seen that the -54.2 dB peak at 11 Hz and the -52.2 dB peak at 16 Hz for the bedroom floor response would be considered to exceed limits for residential annoyance and is approximately 15 dB below the threshold limit for structural damage. (The owner-occupants had recently purchased and redecorated the house. The house was approximately 40 years old. Their vigorous complaints centered upon the six buses that passed their house each week day. Their complaints were describable as extreme annoyance, rattles, and cracks in plaster, tile and masonry. Details of this and other cases are presented in Reference 1.)

2.3.6 Building Parameters

Ground vibration received at a building foundation may cause an amplification of vibration on the building interior. The degree of amplification depends upon the details of the building construction. For traffic-induced vibration, the building component vibration appears to be the most significant consideration. That is, floor and wall vibration is more significant than total building vibration.

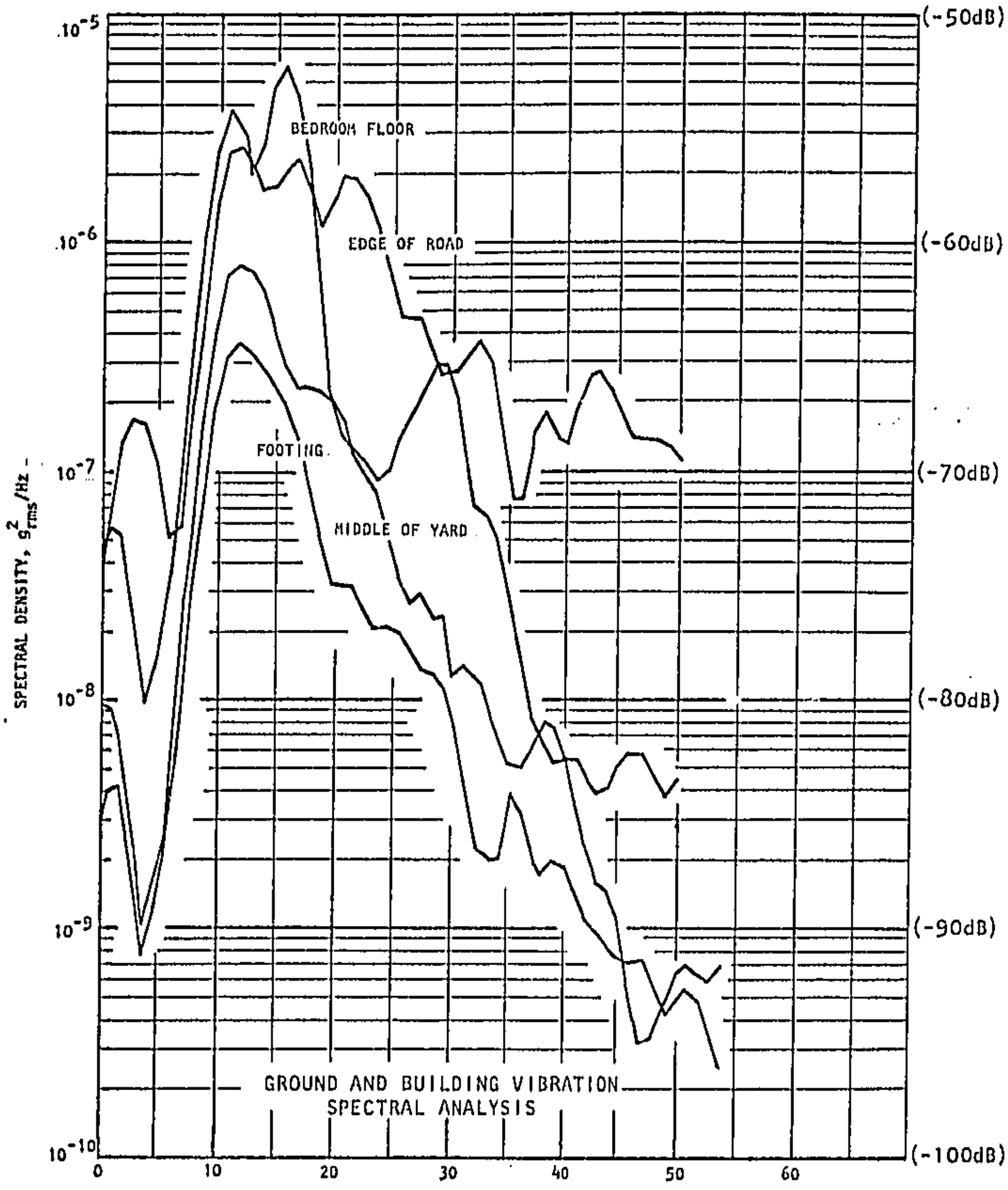


Figure 2-6. GROUND AND BUILDING VIBRATION SPECTRA FOR A BUS PASSING A RESIDENCE

It is extremely difficult to estimate accurately either the natural frequencies or response amplitudes of building construction. Typically, transverse vibration (motion out of the plane) of building floors and walls will occur in the frequency range of 20 Hz to 40 Hz. These vibration modes represent the fundamental and the first few harmonics of the floor or wall vibration. Above 40 Hz, vibration of floors and walls generally comprises motion of the structure between supports such as joists, etc. These vibration modes play a dominant role in the transmission loss or noise reduction of the structure and are generally excited by noise impinging on the structure.

As seen in Figure 2-6, the bedroom floor peak response occurs at 11 Hz (-54.2 dB), 16 Hz (-52.2 dB), and 28 Hz (-65.2 dB). The response spectra at the footing for these frequencies is: 11 Hz (-64.6 dB), 16 Hz (-67 dB), 28 Hz (-78.5 dB). Hence, the bedroom floor amplifications relative to the acceleration levels at the footing are: 11 Hz (12.4 dB), 16 Hz (14.8 dB), 28 Hz (13.3 dB). Since the peak bedroom floor response in the frequency range below 50 Hz is generally the most important for vibration perception (See Figures 2-3 and 2-4), it is usually necessary only to consider this frequency range if one must perform a spectral analysis of the data.

If the structure is relatively close to the roadway, the possibility of acoustic excitation of the building structure exists. In particular, if heavy vehicles are accelerating away from a stop, high level (80 dBA or greater) traffic noise may impinge on the structure. The building response to the combined traffic noise and vibration environment is different from that indicated in Figure 2-6. Figures 2-7 and 2-8 present building and ground motion acceleration spectra, respectively, for a combined environment of traffic noise and vibration. The low frequency peak corresponds to the ground excitation and the high frequency spectra corresponds to the atmospheric excitation. The interpretation of these two results is that

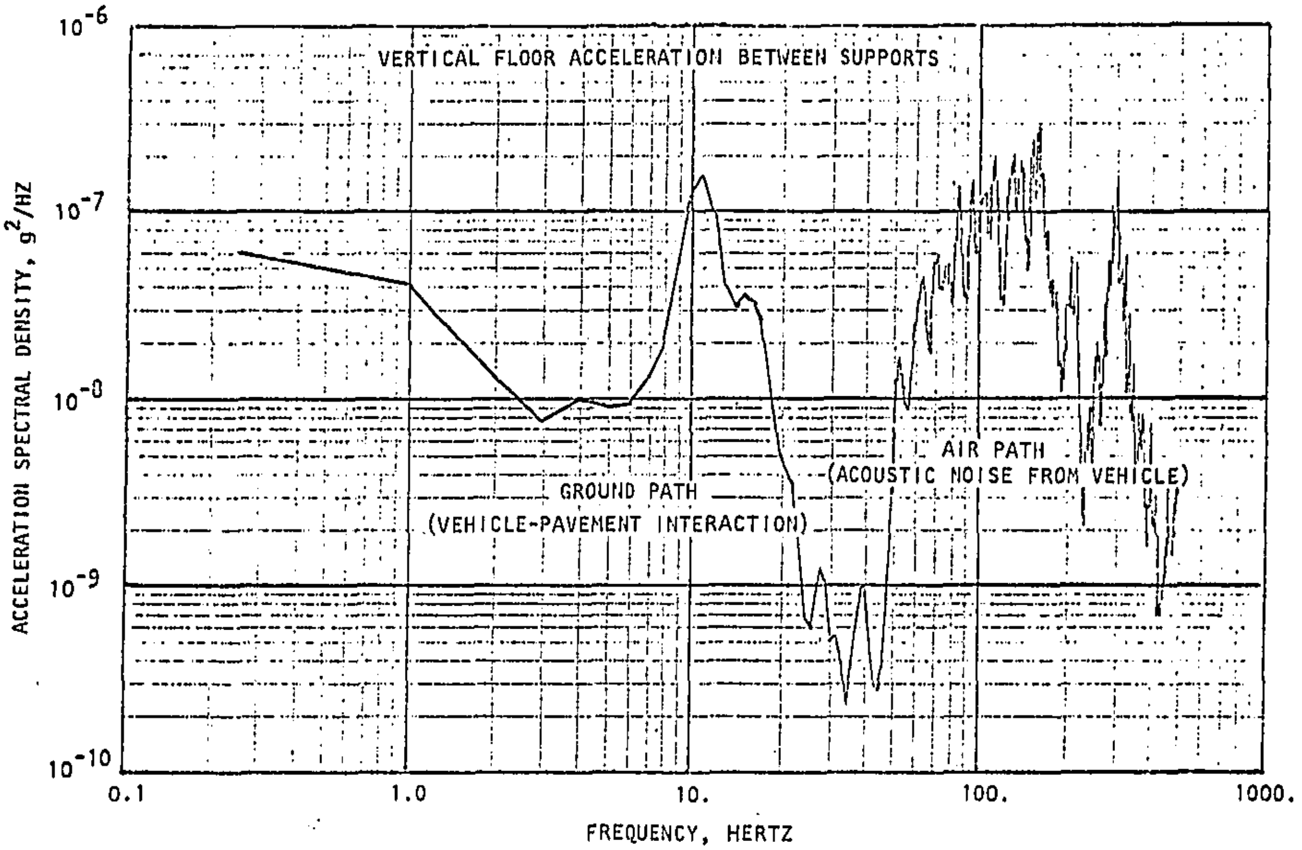


Figure 2-7. BUILDING RESPONSE TO COMBINED TRAFFIC NOISE AND VIBRATION

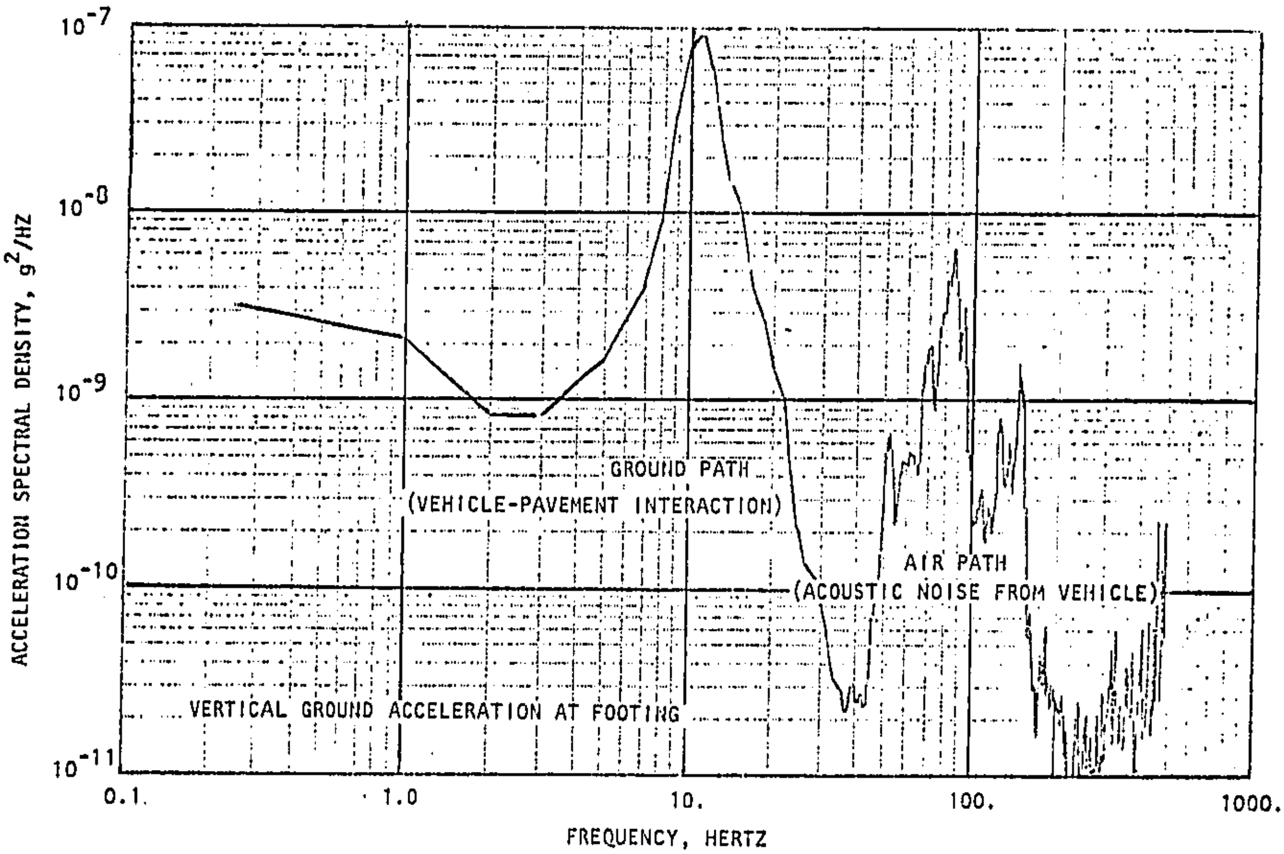


Figure 2-8. GROUND MOTION RESPONSE TO COMBINED TRAFFIC NOISE AND VIBRATION

the low frequency ground motion (Figure 2-8) resulted in a non-resonant forced vibration of the building as indicated by the low frequency peak in Figure 2-7. The high frequency noise-induced vibration of the building structure is indicated in Figure 2-7 by the acceleration spectra above 40 Hz. In Figure 2-8, the high frequency acceleration spectra above 40 Hz is the vibrating building shaking the ground!

The discussion of combined traffic noise and vibration excitation of building structures is not an academic exercise. The purpose of the discussion is to emphasize the importance of spectral analysis in evaluating complaints of traffic-induced vibration. The presence or absence of noise-induced vibration must always be determined. In particular, field measurement data must always indicate this consideration. The aspect is discussed in Section 4.

In the absence of field measured data, it is possible to estimate the expected amplification of ground vibration by building structure. Generally, floor vibration increases with increasing building storeys. For design use, the building amplification appears to be from -5 dB to +10 dB for ground level floors. Second storey floors appear to amplify ground motion from -5 dB to +15 dB. Specific guidance is presented in Section 3.5 of these guidelines.

The levels of vibration induced in a building by traffic operations are on the order of magnitude of vibrations caused by other household activities such as footsteps, closing doors, playing a loud recording, etc. Natural phenomena such as thunder, high winds, and minor earth tremors may also induce building vibration that exceed levels generated by traffic. Hence, on a long term basis, the possibility of identifying traffic-induced vibration as a single source of building damage is quite difficult to establish.

Considering all aspects, traffic-induced vibration appears to be an annoyance problem. Although it is an annoyance problem, the nature of complaints is more closely associated with public reaction to aircraft noise than public reaction to highway traffic noise. That is, when people complain about traffic-induced vibration the complaints are vigorous and public officials can expect continued action on the part of the people annoyed until the problem is resolved.

3.0 ESTIMATION OF TRAFFIC-INDUCED VIBRATION

3.1 Outline of Estimation Procedure

The procedures for estimating the effects of traffic-induced vibration are identical to the procedures used to evaluate traffic noise. First, a vehicle reference emission level is estimated for a location adjacent to the roadway. The reference emission level is then adjusted for distance attenuation from the roadway to the receiver. An appropriate building amplification is selected and the receiver level is adjusted to obtain the vibration level estimate on the building interior. The building interior vibration level is then compared to the appropriate criteria to estimate the impact from vibrations induced by highway traffic.

3.2 Vibration Reference Emission Level

The vibration reference emission level is the basic variable quantifying the vibration generation resulting from vehicle/pavement interaction. The vibration reference emission level is denoted by the symbol, L_o. This level, as used in these guidelines, is the peak vertical component acceleration level measured on the ground surface at the edge of the roadway.

The vibration reference emission level depends upon the following basic parameters:

- pavement surface roughness
- vehicle speed
- gross vehicle weight
- vehicle suspension stiffness
- pavement/subgrade mass

The present availability of data does not allow for a complete resolution of the vehicle suspension stiffness and the pavement/ subgrade mass effects. It appears, however, that these terms do not vary significantly between vehicles of the same gross weight category or between various sites. Hence, these two parameters are currently grouped as a "site constant".

The vibration reference emission level at the edge of a roadway may be qualitatively grouped in the form:

$$L_o = A \cdot \log\text{ (Surface Roughness Parameter)} + B \cdot \log\text{ (Speed} + C \cdot \log\text{ (Gross Vehicle Weight)} + \text{"site constant"} \quad (3\text{-}1)$$

where A, B, C, and the "site constants" are empirical constants.

Based upon the results of Reference 1, it appears that the vibration reference emission level at a point on the ground surface 6.5 feet (2 meters) from the edge of the pavement may be expressed as

$$L_o = -4.155(\text{PSR}) + 17.2 \cdot \log(V) + 10 \cdot \log(W_G) - 87.7,\ \text{dB (re. } 1g_{rms}) \quad (3\text{-}2a)$$

or

$$L_o = -4.155(\text{PSR}) + 17.2 \cdot \log(S) + 10 \cdot \log(\overline{W}_G) - 87.8,\ \text{dB (re. } 1g_{rms}) \quad (3\text{-}2b)$$

where PSR is the Present Serviceability Rating of the pavement roughness
V is the vehicle speed in miles per hour
S is the vehicle speed in kilometers per hour
W_G is the gross vehicle weight in thousands of pounds
$\overline{W}_G$ is the gross vehicle weight in thousands of kilograms

The use of Equations (3-2) to estimate ground vibration at the edge of a pavement does not distinguish between lanes of travel on the same pavement/subgrade structure. If surface roughness varies between lanes on the same pavement/subgrade structure, use the appropriate PSR index for the estimating procedure. Equations (3-2) assume a vehicle cruising past the observer on a roadway generally characterized by random surface roughness. Discrete roughness such as potholes, etc., are considered in Section 3.4.

Figure 3-1 presents a nomograph for estimating the vibration reference emission level based upon Equations (3-2). The similarity between Equations (3-2a) and (3-2b) allow the nomograph of Figure 3-1 to be used either for English or metric units. That is, the gross vehicle weight scale is entered <u>numerically</u> in either thousands of pounds or thousands of kilograms. Similarly, the vehicle speed may be used <u>numerically</u> in either miles per hour or in kilometers per hour. The PSR index is, of course, a pure subjective number and requires no conversion.

As an example, a 45 thousand pound (20.4 thousand kg) vehicle cruising along a roadway at 35 mph (56.3 kmph) with an estimated surface roughness with a PSR rating of 3.0 would cause an acceleration level of -57.1 dB (re. $1g_{rms}$) at 6.5 ft. (2m) from the edge of the roadway. The use of the nomograph of Figure 3-1 is indicated by the dashed path for this example problem.

The nomograph of Figure 3-1 may be used to determine any one of the variables in Equations (3-2) if all of the other variables are defined. For example, if one desired to limit vibration to a maximum single event level of -60 dB (re. $1g_{rms}$) at 6.5 ft. (2m) from the edge of a roadway with a posted speed of 35 mph (56.3 kmph) and an estimated PSR of 2.5, then the maximum gross vehicle weight allowable on the roadway is 15 thousand pounds (6.8 thousand kg).

Figure 3-2 presents axle arrangements and code designations for typical vehicles and vehicle combinations. Table 3-1 presents typical maximum gross vehicle weights based upon ranges of maximum axle weight limit (7, 8, 9). These values may be used for guidance in the absence of local data.

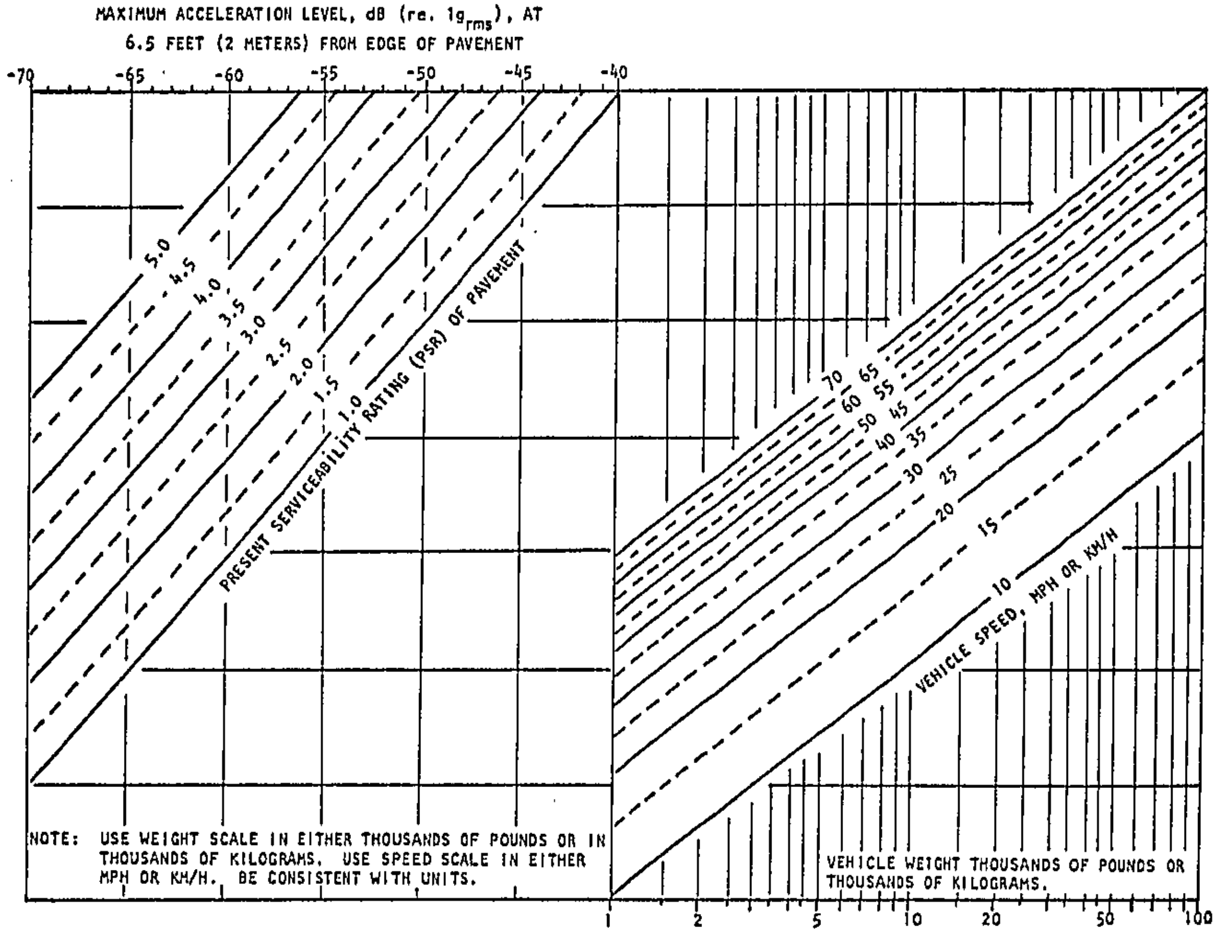


FIGURE 3-1 - DESIGN NUMOGRAPH FOR ESTIMATING THE VIBRATION REFERENCE EMISSION LEVEL

TABLE 3-1

MAXIMUM GROSS VEHICLE WEIGHTS FOR RANGES OF MAXIMUM AXLE WEIGHTS

Vehicle and Axle Code Designations (See Figure 3-2)	Single/Tandem Axle Weight Limit Thousands of Pounds (Thousands of Kilograms)				
	18/32 (8.2/14.5)	20/35 (9.1/15.9)	22/38 (10.0/17.2)	24/41 (10.9/18.6)	26/44 (11.8/20.0)
2D	25.4 (11.5)	28.2 (12.8)	31.0 (14.1)	33.8 (15.3)	36.6 (16.6)
3A	41.6 (18.9)	45.2 (20.5)	48.8 (22.1)	52.4 (23.8)	56.0 (25.4)
2-S1	43.6 (19.8)	48.0 (21.8)	52.3 (23.7)	56.5 (25.6)	60.6 (27.5)
2S-2	58.4 (26.5)	63.7 (28.9)	69.0 (31.3)	74.3 (33.7)	79.6 (36.1)
3-S2	73.7 (33.4)	80.0 (36.3)	86.3 (39.1)	92.6 (42.0)	98.9 (44.9)
3-2	77.8 (35.3)	85.2 (38.6)	92.6 (42.0)	100.0 (45.4)	107.4 (48.7)
2-S1-2	80.7 (36.6)	88.9 (40.3)	97.1 (44.0)	105.3 (47.8)	113.5 (51.5)

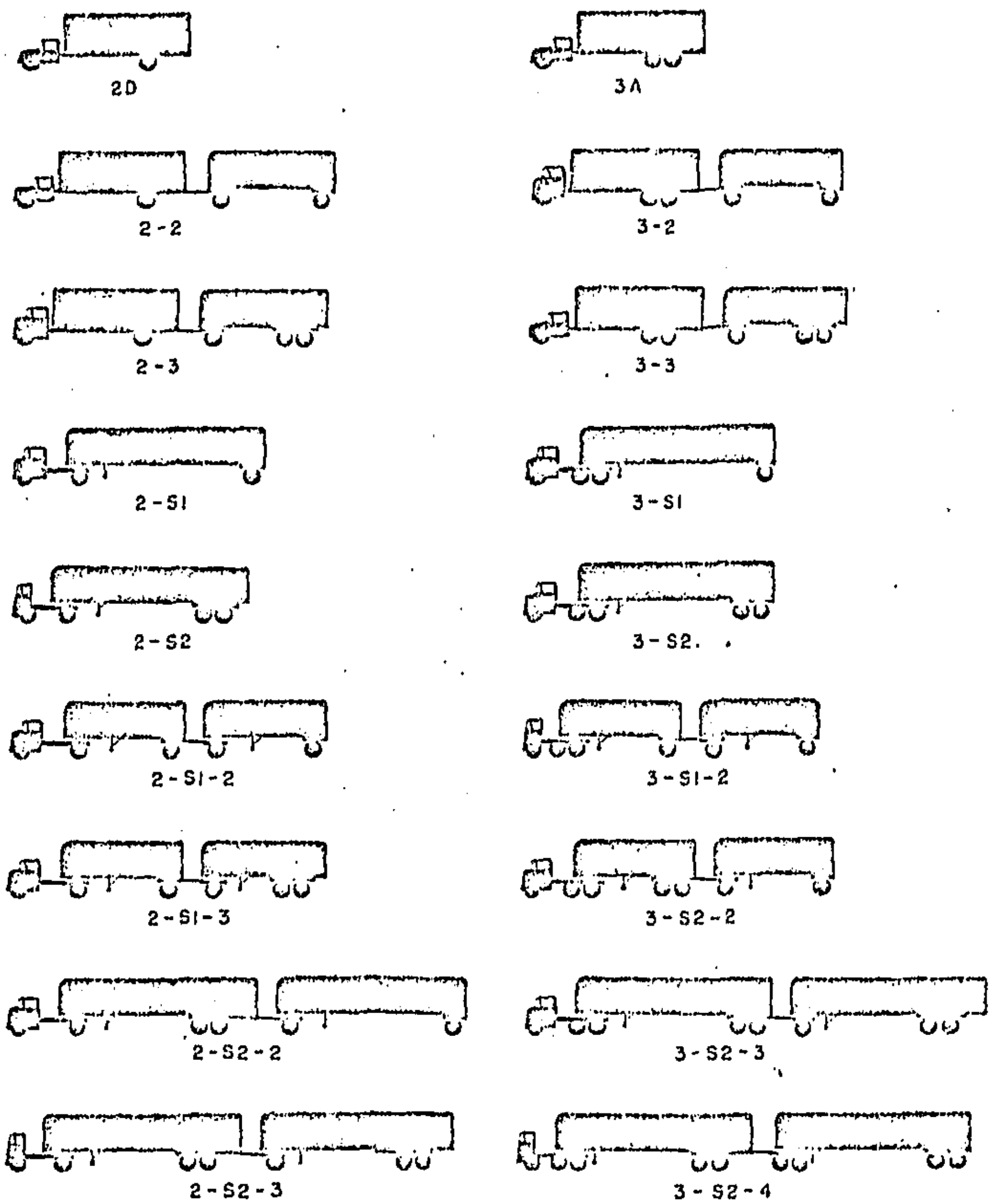


Figure 3-2. AXLE ARRANGEMENTS AND CODE DESIGNATIONS FOR TYPICAL VEHICLES AND VEHICLE COMBINATIONS

3.3 Propagation and the Equivalent Vibration Level

The results of Section 3.2 allow the estimation of the peak acceleration level at a point 6.5 feet (2 meters) from the edge of a roadway. The parameters required for this estimation are: PSR index of pavement, vehicle speed, and gross vehicle weight. The next step in the estimation procedure is to adjust the vibration reference emission level for distance attenuation.

3.3.1 Propagation of Vibration

In estimating traffic noise at a distance from the roadway, it is required to consider finite roadway segments to model the roadway-receiver geometry (10). The traffic-induced vibration problem does not require this complication. The reason for this is that traffic-induced vibrations attenuate very rapidly with distance away from the highway. Typically, the total time required for an acceleration level to rise from and return to a level 20 dB below the peak level is on the order of two seconds for high-speed traffic and seven seconds for low speed traffic. During this time interval, the vehicle is within 75 to 150 feet (23 to 46 meters) of the closest passby location as measured along the roadway. The comparable situation for traffic noise is a roadway distance of approximately 4950 feet (1509 meters) for a "hard" site and 1940 feet (591 meters) for a "soft" site. Hence, from a practical standpoint, all roadways are "infinite" for traffic-induced vibration problems.

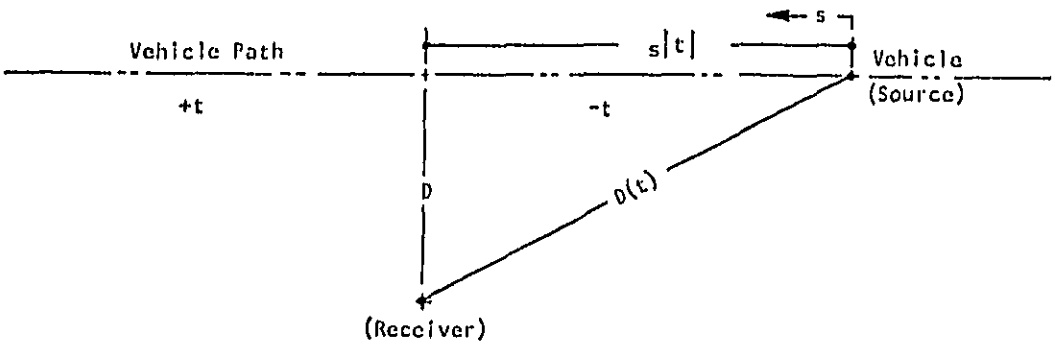
Figure 3-3 presents the source-path-receiver relationship for traffic-induced vibration. The distance between the point vehicle source and the receiver varies with time as indicated. Assuming that the vehicle generates a random vibration as it travels along the roadway, the vibration level at the receiver at any instant is expressed as:

$$L(t) = L_o + 10\ \log(D_o/R(t)) - 20 \cdot \log(e)\alpha(R(t) - D_o),\ \text{dB} \qquad (3\text{-}3)$$

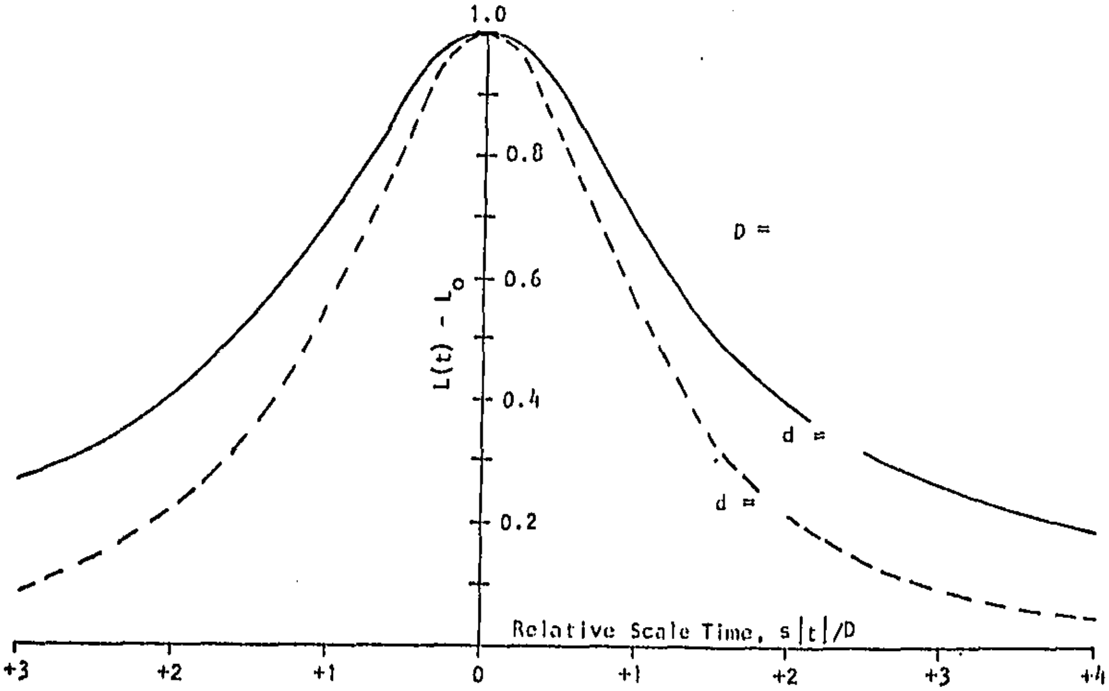
$$R^2(t) = D^2 + (Vt)^2 = D^2[\,1 + (st/D)^2]$$

= Vehicle Speed, Constant

t = Time Relative to $L(0) = D$



(a) Source-Receiver Geometry: Straight Highway Segment



(b) Normalized Attenuation Versus Relative Time

FIGURE 3-3 ROADWAY/RECEIVER GEOMETRY

where L_o is the reference vibration emission level presented (acceleration level from Eqn (3-2))

D_o is the reference distance at which L_o is defined

$R(t) = \sqrt{(D^2+(st)^2)}$ is the time varying source-receiver distance

D is the receiver distance from the roadway

s is the vehicle speed (ft/secon m/sec)

t is time (seconds, sec)

α is the distance attenuation constant for the soil between the source and the receiver

Equation (3-3) is totally analogous to the comparable result for traffic noise. The first term is the reference emission level. The second term is the geometric spreading of surface waves across the ground away from the source. The point vibration source geometric spreading varies inversely as the square-root of the distance. The third term is the distance attenuation of the vibration due to absorptive losses in the soil. The absorptive losses in the soil are directly proportional to the absolute distance between the source and the receiver. This term is analogous to the "soft site" attenuation used in traffic noise analysis (10).

Values of α are presented in Table 3-2 for various soils. As used in Equation (3-3), the absorption coefficient α is independent of frequency. The values of α presented in Table 3-2 were derived from data presented in References 11 and 12 in terms of the "loss factor" for the soil. The assumptions used to obtain the values of α are indicated in the footnotes to Table 3-2.

The maximum vibration level at the receiver is obtained from Equation (3-3) for the time $t = 0$.

3.3.2 Single Event Vehicle Passby

As shown in Reference 1, the equivalent (energy mean) vibration level is obtained from Equation (3-3) by integrating over

TABLE 3-2

SOIL ABSORPTION COEFFIECIENTS

SOIL DESCRIPTION	TRANSVERSE WAVE SPEED METERS/SECOND	α PER METER	TRANSVERSE WAVE SPEED FEET/SECOND	α PER FOOT
Moist Clay, Clayey Soil	152	0.025-0.25	500	0.008-0.08
Silty Clay	152	0.019-0.43	500	0.06-0.13
Wet Clay	152	0.31-0.50	500	0.10-0.15
Loess at Natural Moisture	259	0.04-0.13	850	0.01-0.004
Dry Sand	152-396	0.007-0.070	500-1300	0.002-0.023
Dense Sand and Gravel	250	0.015-0.045	820	0.005-0.014
Gravel (30-60%) plus Sand & Silt	250	0.023-0.053	820	0.007-0.016
Fine Grained Sand				
Water Saturated	110	0.09-0.300	360	0.026-0.091
Water Saturated, Frozen	110	0.050-0.170	360	0.016-0.052

Derived from Reference 11 as follows: $\alpha=2\pi f\eta/c$ $f=15Hz$ $\alpha=30\pi\eta/c$
where η is the soil loss factor and c is the transverse wave speed.

a passby time period, $-T/2 \leq t \leq T/2$. The resulting expression for the energy mean vibration level is:

$$L_e = L_o + 10 \log(D_o/sT) - 5 \cdot \log(\alpha D) - 20 \cdot \log(e)\alpha(D-D_o) + 5 \log(\pi) \qquad (3\text{-}4)$$

where T is the time period for the passby.

Any set of consistent units may be used in Equation (3-4). This result applies to a specific vehicle/roadway combination defined by the vibration reference emission level, L_o.

For a continuous flow of N vehicles of the same gross weight over a time period, T, at a constant speed, s, the energy mean vibration level for a given roadway is:

$$L_e = L_o + 10 \cdot \log(ND_o/sT) - 5 \cdot \log(\alpha D) - 20 \cdot \log(e)\alpha(D-D_o) + 5 \cdot \log(\pi), \text{dB} \qquad (3\text{-}5)$$

Any set of consistent units may be used in Equation (3-5).

For distances expressed in feet, speed in miles per hour, and time in hours, the energy mean vibration level is:

$$L_e = L_o + 10 \cdot \log(ND_o/VT) - 5 \cdot \log(\alpha D) - 8.686\alpha(D-D_o) - 16.1, \text{dB} \qquad (3\text{-}6a)$$

where $20 \cdot \log(e) = 8.686$

$5 \cdot \log(\pi/5280) = -16.1$

α in units of $(\text{feet})^{-1}$

For distances expressed in meters, speed in kilometers per hour, and time in hours, the energy mean vibration level is:

$$L_e = L_o + 10 \cdot \log(ND_o/sT) - 5 \cdot \log(\alpha D) - 8.686\alpha(D-D_o) - 12.5, \text{ dB} \qquad (3\text{-}6b)$$

where $20 \cdot \log(e) = 8.686$

$5 \cdot \log(\pi/1000) = -12.5$

α in units of $(\text{meter})^{-1}$

3.3.3 Mixed Vehicle Traffic Flow

For a mixture of vehicles of different gross weight categories travelling along a roadway at a constant speed, s, the energy mean vibration level at a receiver located at distance D from the roadway is:

$$L_e = 10\ \log(\sum_i p_i 10^{L_{oi}/10}) + 10 \cdot \log(\overline{N}D_o/sT) - 5 \cdot \log(\alpha D) - 20 \cdot \log(e)\alpha(D-D_o) + 5 \cdot \log(\pi),\ \text{dB} \qquad (3\text{-}7)$$

where $\overline{N}$ is the total vehicle count for the time period T

s is the traffic speed (ft or m per unit of T)

$p_i = N_i/\overline{N}$, the percentage of the total traffic count of vehicles in the i^{th} weight category

L_{oi} is the vibration reference emission level for vehicles of the i^{th} weight category travelling at the constant traffic speed s

D_o is the reference distance at which L_o is monitored

Σ is the sum over vehicle weight groups

Any consistent set of units may be used in Equation (3-7). Vehicles must be grouped by weight categories representative of the vehicle type. Table 3-1 may help in providing guidance as to representative gross vehicle weights by vehicle classification and axle weight limit.

For distances expressed in feet, speed in miles per hour, and time in hours, the energy mean vibration level for a traffic flow comprising vehicles of mixed gross weights is:

$$L_e = 10 \cdot \log(\sum_i p_i 10^{L_{oi}/10}) + 10 \cdot \log(\overline{N}D_o/VT) - 5 \cdot \log(\alpha D) - 8.686\alpha(D-D_o) - 16.1,\ \text{dB} \qquad (3\text{-}8a)$$

where α is in units of $(\text{feet})^{-1}$

For distances expressed in meters, speed in kilometers per hour, and time in hours, the energy mean vibration level for a traffic flow comprising vehicles of mixed gross weights is:

$$L_e = 10\cdot\log(\sum_i p_i 10^{L_{oi}/10}) + 10\cdot\log(\bar{N}D_o/ST) - 5\cdot\log(\alpha D)$$
$$-8.686\alpha(D-D_o) - 12.5,\ \text{dB} \qquad (3\text{-}8b)$$

where α is in units of $(\text{meters})^{-1}$

The results presented in Equations (3-8) are general in that the vibration reference emission level is not expressed in terms of a specific metric. That is, L_o could be expressed as a decibel level in either displacement, velocity, or acceleration and the equivalent levels will then be expressed in terms of the appropriate measure.

3.3.4 Difference Between Peak and Energy Mean Level

The determination of impact from traffic-induced vibration must be made in terms of available criteria. Figure 2-4 presents criteria in terms of the number of occurrences of peak acceleration level in a 24 hour period. In terms of a single event vehicle pass-by, the peak vibration level at a distance D from the roadway is given by Equation (3-3) with $t=0$ (i.e., $R(o)=D$). For a constant speed traffic flow on a roadway of specified roughness, the heaviest vehicle class will generate the maximum vibration level. Denoting the heaviest vehicle class by a subscript, I, the peak vibration level is (from Eqn. (3-3)):

$$L_{peak} = L_{oI} + 10\cdot\log(D_o/D) - 20\cdot\log(e)\alpha(D-D_o) \qquad (3\text{-}9)$$

where L_{oI} is the vibration reference emission level for the heaviest vehicle class in the traffic flow.

Subtracting Equation (3-7) from Equation (3-9), the difference between the single event peak vibration level and the energy mean vibration level is obtained. For a receiver at a distance D from the roadway the result is:

$$L_{peak}-L_e = 5\cdot\log(\alpha D)-10\cdot\log(D\overline{N}/sT)-10\cdot\log\{\sum_i p_i 10^{-\Delta_i/10}\}-5\cdot\log(\pi) \quad (3\text{-}10)$$

where $\Delta_i=L_{oI}-L_{oi}$

Any consistent set of units may be used in Equation (3-10).

This result is important in addressing complaints focusing upon a single vehicle type such as buses or other heavy vehicles passing on a roadway. This consideration will be discussed further under Abatement Strategies in Section 3.6.

For distances expressed in feet, speed in miles per hour, and time in hours, the difference between the single event peak vibration level and the energy mean vibration level for a traffic flow comprising vehicles of mixed gross weights is:

$$L_{peak}-L_e=5\cdot\log(\alpha D)-10\cdot\log(D\overline{N}/VT)-10\cdot\log\{\sum_i p_i 10^{-\Delta_i/10}\}-16.1 \quad (3\text{-}11a)$$

where α is in units of $(\text{feet})^{-1}$

$\Delta_i=L_{oI}-L_{oi}$

For distances expressed in meters, speed in kilometers per hour, and time in hours, the difference between the single event peak vibration level and the energy mean vibration level for a traffic flow comprising vehicles of mixed gross weights is:

$$L_{peak}-L_e=5\cdot\log(\alpha D)-10\cdot\log(D\overline{N}/ST)-10\cdot\log\{\sum_i p_i 10^{-\Delta_i/10}\}-12.5 \quad (3\text{-}11b)$$

where α is in units of $(\text{meters})^{-1}$

$\Delta_i=L_{oI}-L_{oi}$

3.3.5 Percentile Vibration Level

For traffic noise analysis, the equivalent sound level and the L_{10} sound level are common descriptors used to evaluate impact from traffic noise. Similarly, the determination of impact from traffic-induced vibration may be expressed in terms of percentile vibration levels. The approach is identical to that used for traffic noise analysis (12).

For dense traffic flows, it is assumed that the distribution of vibration amplitudes during a time period T is Gaussian. (The validity of this assumption appears to be as accurate for traffic-induced vibration as it is for traffic noise.) The vibration amplitude distribution is then completely defined by the mean vibration level, L_{50}, and the standard deviation of the vibration level, σ_L.

The mean vibration level is defined in terms of the energy mean vibration level and the standard deviation as

$$L_{50} = L_e - 0.115\sigma_L^2 \qquad (3\text{-}12)$$

$$\sigma_L = (10/\ln(10))\sqrt{\ln(1+\kappa_2)} = 4.343\sqrt{\ln(1+\kappa_2)}$$

where κ_2 is given by Equation (3-14)

In terms of either the mean vibration level, L_{50}, or the energy mean vibration level, L_e, the percentile vibration levels are:

$$\left.\begin{array}{ll} L_{10} = L_{50}+1.28\sigma_L & \text{(level exceed 10\% of time)} \\ L_{05} = L_{50}+1.648\sigma_L & \text{(level exceeded 5\% of time)} \\ L_{01} = L_{50}+2.33\sigma_L & \text{(level exceeded 1\% of time)} \\ L_{0.1} = L_{50}+3.09\sigma_L & \text{(level exceeded 0.1\% of time)} \end{array}\right\} \quad (3\text{-}13)$$

These results apply to a traffic flow comprising a mixture of vehicles of different gross weights.

For a constant speed traffic flow, the energy mean vibration level, L_e, is given by Equation (3-7) for any consistent set of units. As shown in Reference 1, the factor κ_2 (called a cumulant) is given by:

$$\kappa_2 = \frac{\alpha s T/\overline{N}}{\sqrt{2\pi\alpha D}} \left(\sum_i p_i 10^{\tilde{L}_{oi}/5}\right) / \left(\sum_i p_i 10^{L_{oi}/10}\right)^2 \qquad (3\text{-}14)$$

where p_i is the percentage of the total traffic flow comprising vehicles of the i^{th} weight category

L_{oi} is the reference vibration emission level for vehicles of the i^{th} weight category

$\tilde{L}_{oi}$ See Equation (3-16)

α is the soil absorption coefficient (Table 3-2)

$\overline{N}$ is the total traffic count during the time period T

s is the constant traffic speed

D is the distance of the receiver away from the roadway

Any consistent set of units may be used in Equation (3-14). (Vehicle speed, s, must be in feet per second or meters per second.) To account for variation in L_{oi} see Equation (3-16).

3.3.6 Variability of the Vibration Reference Emission Level

The vibration reference emission level, L_o must be established as the result of field tests. Section 3.2 describes an estimation of the acceleration reference emission level. This result exhibits a variation of approximtely 5 dB. That is, the estimation using either Equations (3-2) or the design chart of Figure 3-1, is the expected value of the reference emission level.

As shown in Reference 1 or in Appendix A of Reference (10), the variability of the reference emission level may be considered if one assumes that the distribution of values of L_o are Gaussian. For a regression analysis, the mean or expected value of the reference emission level, $\overline{L}_o$, is obtained along with the standard error associated with the regression. Thus, the value of the reference emission level is expressed as an energy average as

$$L_o = \bar{L}_o + 0.115\sigma_o^2 \qquad (3\text{-}15)$$

The value of L_o given by Equation (3-15) is the value to be used in Equations (3-3) through (3-11). In Equation (3-14), the following values must be used:

$$\tilde{L}_{oi} = \bar{L}_{oi} + 0.230\sigma_{oi}^2 \qquad \text{(numerator)} \qquad (3\text{-}16a)$$

$$L_{oi} = \bar{L}_{oi} + 0.115\sigma_{oi}^2 \qquad \text{(denominator)} \qquad (3\text{-}16b)$$

For an extimated value of σ_o=5, the reference emission level is given by

$$L_o = \bar{L}_o + 0.115(5)^2 = \bar{L}_o + 2.9 \text{ dB}$$

$$\tilde{L}_o = \bar{L}_o + 0.230(5)^2 = \bar{L}_o + 5.8 \text{ dB}$$

The value of $\bar{L}_o$ for traffic-induced acceleration level may be estimated using the results in Section 3.2 or may be based upon site measured field-test data.

3.4 Potholes and Impact Factors

The results presented in Section 3.3 apply to vehicles moving along a roadway characterized by a general random surface roughness. That is, the roadway surface does not contain any abrupt variations in surface roughness such as a pothole, etc. It is common practice in highway design to express the peak dynamic loading as an impact factor. The impact factor is defined as the ratio of the maximum peak dynamic load to the static load (13).

The estimation of traffic-induced vibration resulting from bumps is presented since such irregularities may result from an intentional design feature of the highway. Also, many investigators have used ramps on planks fixed to a roadway to induce and to report "traffic vibration" data.

For bumps or potholes in the roadway surface, the relation between vehicle speed and the resulting pavement loading is totally different than that described in Section 3.2. The prediction methods for random surface roughness are site specific in that pavement roughness must be estimated. Vehicle parameters required are the speed and the gross weight.

For the analysis of vibrations generated by vehicles striking bumps several specific vehicle parameters must be known or estimated as well as the bump geometry and pavement/subgrade structure. Details of the development of the results described in these guidelines are presented in Reference 1.

3.4.1 Bump, Vehicle, and Pavement/Subgrade Parameters

Bump Geometry: The significant bump parameters are the bump height, $\bar{h}$, and the bump length, ℓ. The height, $\bar{h}$, is measured normal to the local plane of the pavement surface. The length of the pavement bump, ℓ, is measured in the direction of travel. The methodology presented in these guidelines is limited to bump geometries that do not result in significant "tire enveloping" (14). Generally, the bump height must be less than 2 inches (50 mm) to satisfy this restriction.

Vehicle Parameters: The significant vehicle parameters are: the vehicle speed, S; the static tire load, W_o; the tire stiffness, k_t;

and the vehicle suspension system natural frequency, f_n. Many of these parameters can only be roughly estimated. Hence, one may have to work with typical values.

The vehicle suspension system natural frequency may be estimated using the following relationship:

$$f_n = (2\pi)^{-1}\sqrt{g(k_t+k_s)/W_s} \quad , \text{Hz.} \tag{3-17}$$

where k_t is the tire stiffness
k_s is the suspension stiffness
W_s is the suspension weight (axle, tire, rims, etc.).
g is the acceleration due to gravity (ie, 9.807m/s^2)

For design use the suspension system stiffness is approximately 17% of the tire stiffness. The suspension weight is approximately 12% to 18% of the static wheel load of an empty vehicle. For heavy vehicles, the suspension system natural frequency appears to be approximately 12 Hz (1), (14), (15).

Tire stiffness is generally considered to be a non-linear function of load. As an approximation, it appears that for surface roughness variations of 2 inches (50 mm) or less that linear tire stiffness is a reasonable assumption. There is little available information published concerning typical tire stiffnesses. Tire stiffness varies as a function of internal pressurization and tire geometry (16). In the absence of experimental data, the following result may be used to estimate tire stiffness (1):

$$k_t = 4\pi_o\sqrt{dD}\, P_o^{\ 2/3} \tag{3-18}$$

where P_o is the internal pressure
d is the tire width or minor diameter
D is the tire major diameter

The tire geometry is illustrated in the design nomograph of Figure 3-4. This design nomograph estimates the tire stiffness based upon the approximation of Equation (3-18). Neither Equation (3-18) nor Figure (3-4) should be used to estimate load-deflection characteristics for tires. The estimates do appear, however, to be sufficiently accurate for traffic-induced vibration calculations.

Pavement/Subgrade Parameters: The effect of a vehicle striking a bump is to cause an impulsive force on the pavement. The pavement/subgrade sturcture is a complex system. However, for dynamic loading generated by heavy vehicles it appears that somewhat simplified models are accurate (1), (17).

The basic pavement/subgrade parameters required to estimate response to loading are the natural frequency of the pavement/subgrade fundamental mode and the "effective" mass of the pavement/subgrade system.

For rigid pavements the natural frequency of the fundamental mode of a pavement/subgrade system may be estimated by the relation:

$$f_p^2 = (1/4\pi^2)\ (k_f/m_f)\cdot(1+\varepsilon)/(1+\mu) \qquad \text{, Hz.} \qquad (3\text{-}19)$$

where k_f is the modulus of subgrade reaction

m_f is the mass per unit area of the subgrade material

ε is a characteristic pavement/subgrade scale factor

μ is the ratio of the pavement mass per unit area to the subgrade mass per unit area.

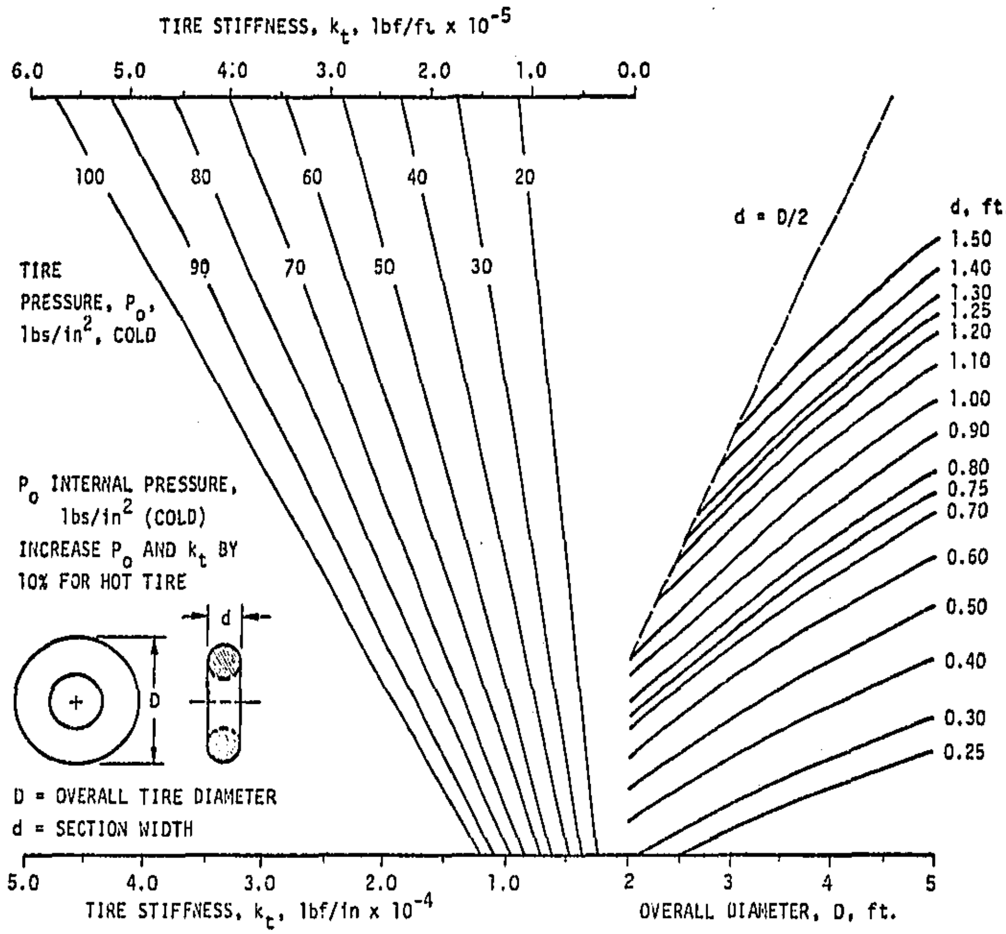


Figure 3-4. DESIGN NOMOGRAPH FOR TIME STIFFNESS

In terms of engineering constants, the parameters in Equation (3-19) may be estimated as:

$$k_f = E_f/H(1-\nu_f^2) \quad m_f = \gamma_f \; H/3g \quad m_p = \gamma_p h_p/g$$
$$\varepsilon = \sqrt{2(1-\nu_f)/3}\;(H/b) \qquad \mu = m_p/m_f \tag{3-20}$$

Where
- H is the subgrade depth* (units of length)
- h_p is the pavement thickness (units of length)
- b is the pavement width (units of length)
- γ_p is the density of the pavement material (force per unit volume)
- γ_f is the density of the subgrade material (force per unit volume)
- g standard value of the acceleration due to gravity ($9.807\ m/s^2 = 32.17\ ft./s^2 = 386.1\ in/s^2$)
- E_f is Young's modulus of the subgrade material (force per unit area)
- ν_f is Poisson's ratio for the subgrade material subscript f denotes foundation (subgrade) and p denotes pavement.

As for any soil mechanics problem, experimental results should be used if they are available. As an approximation, representative values of subgrade material properties are listed in Table 3-3. Table 3-4 lists representative values of k_f for regions of the United States (7).

The final pavement/subgrade parameter required is the effective weight (or mass) of the system. Based upon the model of Reference 1 and consistent with the above results, the effective weight of the pavement/subgrade system may be estimated by the relationship:

$$\overline{W} = 5.5\ \gamma_f\ b^2 H \cdot (1+\mu)/3 \tag{3-21}$$

* The theory upon which these results is based assumes that H is equal to or less than b/2. If H is greater than b/2 set it equal to b/2.

TABLE 3-3

ORDER-OF-MAGNITUDE VALUES FOR SOIL PROPERTIES
AND WAVE SPEEDS BY SOIL CLASSIFICATION SYSTEM

CLASSIFICATION SYSTEM		MODULUS OF ELASTICITY	DENSITY	POISSON'S	COMPRESSIONAL WAVE VELOCITY	TRANSVERSE WAVE VELOCITY
UNIFIED	AASHO	E, lbs/in.^{2}	γ lbs/ft^3	RATIO, ν	ft/sec	ft/sec
GW	A-1	15,000 - 30,000	130	0.30	848 - 1200	453 - 641
GP	A-1	15,000 - 30,000	120	0.31	896 - 1267	470 - 665
GM	A-1	15,000 - 30,000	127	0.32	896 - 1253	456 - 644
GC	A-1	15,000 - 30,000	123	0.33	915 - 1294	461 - 652
SW	A-2	7,500 - 12,000	120	0.34	668 - 844	329 - 416
SP	A-2	7,500 - 12,000	110	0.35	712 - 900	342 - 432
SM	A-2	1,500 - 3,000	117	0.36	316 - 447	148 - 209
SC	A-2	1,500 - 3,000	115	0.36	319 - 451	149 - 211
ML	A-3	1,000 - 2,000	107	0.40	305 - 431	124 - 176
CL	A-4	600 - 1,200	107	0.41	246 - 348	96 - 136
OL	A-5	200 - 600	90	0.42	183 - 281	85 - 104
MH	A-6	75 - 500	82	0.43	110 - 284	38 - 99
CH	A-7	75 - 500	92	0.44	111 - 286	36 - 94
OH	A-7	75	82	0.45	127	38

TABLE 3-4

REPRESENTATIVE AVERAGE SOIL SUPPORT
VALUES USED FOR THE DESIGN OF PAVEMENTS

(REFERENCE 7)

FHWA CENSUS DIVISION	RIGID PAVEMENT MODULUS OF SUBGRADE REACTION k_f
1. New England	150
2. Middle Atlantic	150
3. South Atlantic North	100
4. South Atlantic South	200
5. East North Central	100
6. East South Central	150
7. West North Central	100
8. West South Central	100
9. Mountain	250
10. Pacific	200

where $l^4 = D_p/k_f$ is the radius of relative stiffness of the pavement/subgrade system

$D_p = E_p h_p^3 / 12(1-\nu_p^2)$ is the bending rigidity of the pavement slab

E_p is Young's modulus of the pavement material

ν_p is Poisson's ratio of the pavement material

The effective weight (or mass) of the pavement/subgrade system is the representative "glob" of material forced into motion by the vehicle dynamic loading. The effective weight of the pavement/subgrade system is one of the characteristic parameters comprising the "site constant" of the vibration reference emission level given in Section 3.2.

To assist in evaluating the parameters used in Equations (3-19) through (3-21), design charts have been prepared. Figure 3-5 presents a nomograph for calculating the modulus of subgrade reaction based upon Equation (3-20). Figure 3-6 presents a design chart for calculating the radius of relative stiffness of the pavement slab resting on a subgrade. The result of Figure 3-6 is based upon the definitions of Equation (3-21).

3.4.2 Impact Factors and Impulse Loading

The impulse loading of a pavement following tire contact with a bump may be estimated based upon the concept of a shock spectrum (18). Based upon available experimental data (1), the peak dynamic loading is modelled as the response of a one degree-of-freedom vehicle suspension model to a half-cycle sine wave base displacement. The amplitude of the base displacement is taken as the bump height $\overline{h}$ and the duration of the forcing is taken as the time required for the vehicle to pass over the bump.

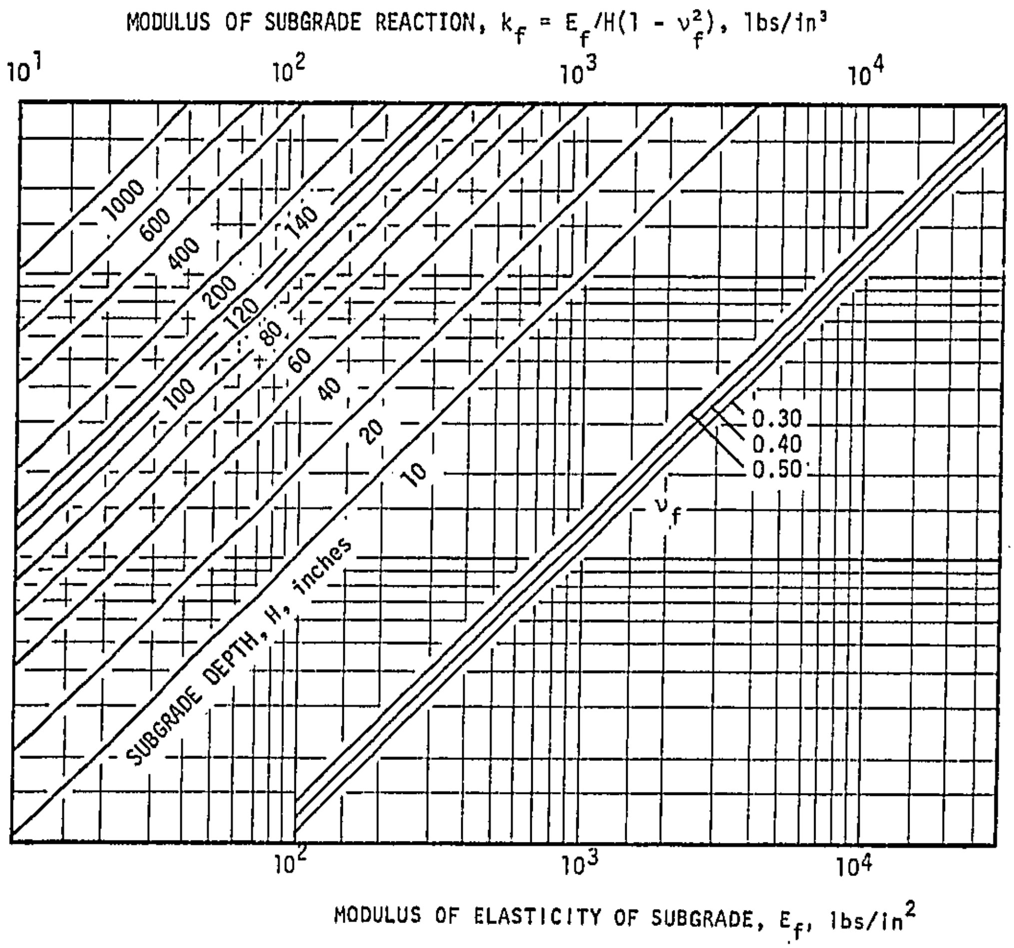


Figure 3-5. DESIGN NOMOGRAPH FOR MODULES OF SUBGRADE REACTION

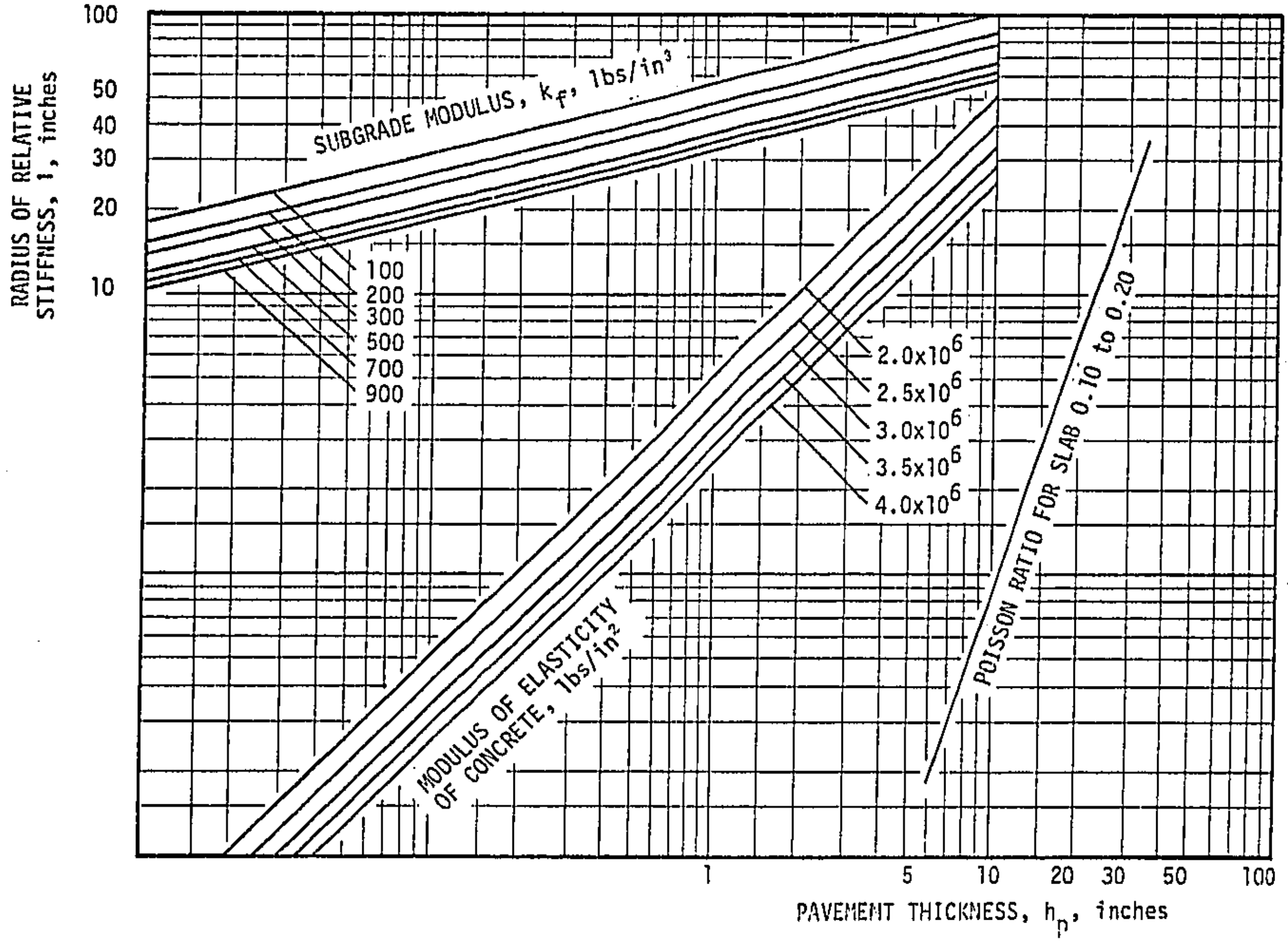


Figure 3-6. DESIGN NOMOGRAPH FOR RADIUS OF RELATIVE STIFFNESS

Using this approach, the peak pavement loading resulting from a vehicle traversing a bump of length ℓ and height $\bar{h}$ at a speed V is given by

$$\left.\begin{aligned} P_o &= k_t\bar{h} / (1-v) && 0 \le v \le 1/3 \\ P_o &= k_t\bar{h}\ \mathrm{SIN}(2\pi v/(1+v))\ /(1-v) && 1/3 \le v \le 1 \\ P_o &= 2k_t\bar{h}\, v\cdot \mathrm{COS}(\pi/2v)/(v^2-1) && v \ge 1 \end{aligned}\right\} \quad (3\text{-}22)$$

Where $v = V/\bar{V}$ is the ratio of the vehicle speed to the characteristic speed

$\bar{V} = 2\ell f_n$ is the characteristic speed

The parameters requried to evaluate Equation (3-22) are the vehicle suspension system natural frequency, f_n (Eqn (3-17)); the tire stiffeness, k_t (experimental values or Eqn (3-18)); the bump height, $\bar{h}$; and the bump length, ℓ. The characteristic speed $\bar{V}$ represents the vehicle speed at which the forcing time ℓ/V equals the natural period $T_n = 1/f_n$ of the suspension system. The impact factor is obtained by dividing both sides of Equation (3-21) by the static tire load. To assist in evaluating Equation (3-22) the design chart of Figure 3-7 has been prepared.

3.4.3 Pavement Response to Impulse Loading

The analysis of Reference 1 indicates that the pavement/subgrade system may be modelled as a multi-degree-of-freedon system. Further, based upon the range of engineering parameters normally

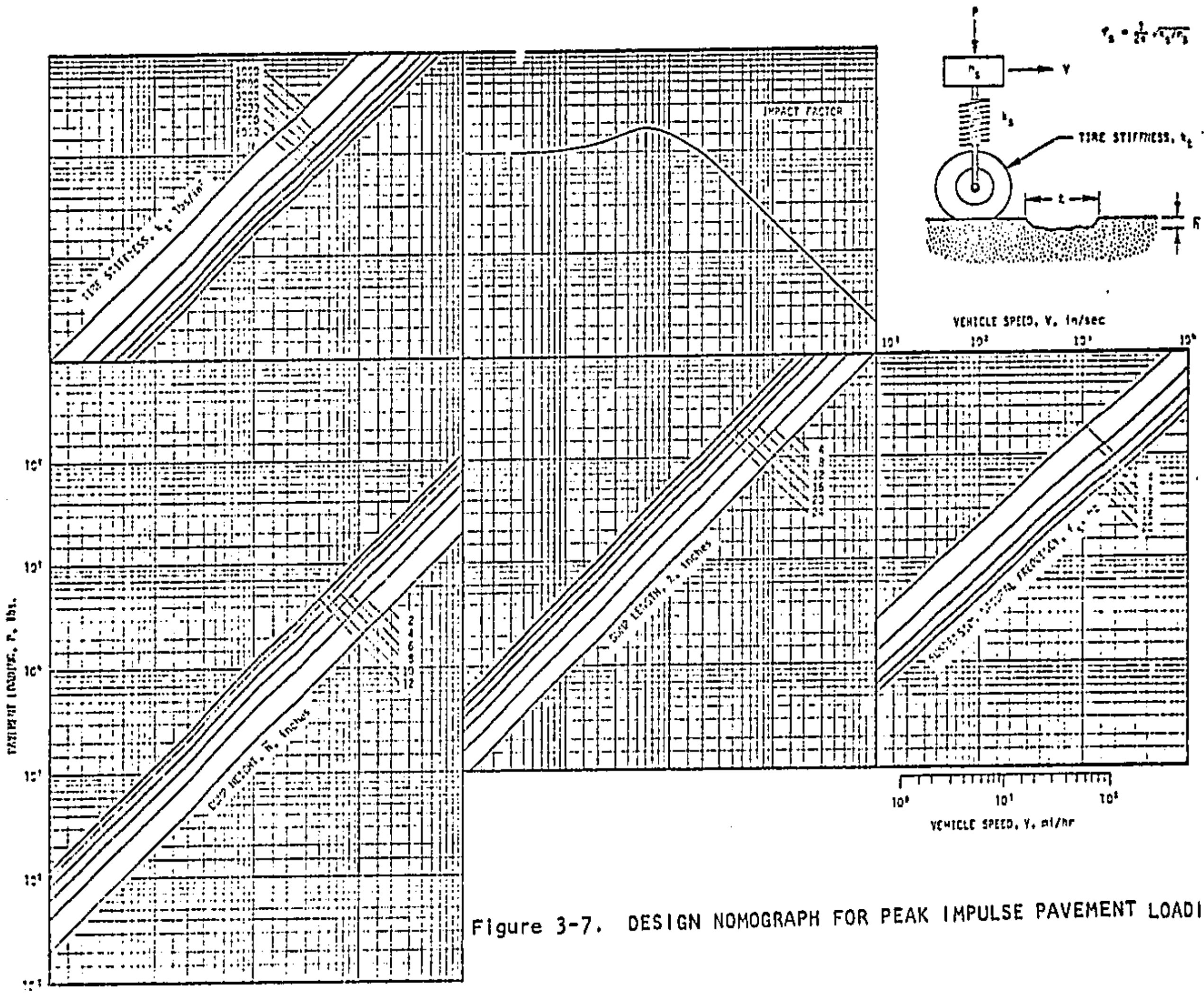


Figure 3-7. DESIGN NOMOGRAPH FOR PEAK IMPULSE PAVEMENT LOADING

encountered in rigid pavement design, it appears that the fundamental mode of the pavement/subgrade system may be considered separately from the higher frequency vibration modes. This modelling approach has been verified by field tests (17).

The peak dynamic pavement loading from Equation (3-22) is assumed to be constant during the time that the tire traverses the bump. That is, the impulse forcing of the pavement is assumed to be rectangular with amplitude P_o and a duration of ℓ/V.

The maximum pavement acceleration level resulting from this impulsive loading is denoted by L_{po} and is given by:

$$L_{po} = 20\log(P_o)-20\log(\overline{W})+20\log(|SIN(\pi/2\tilde{v})|)+6.0, dB(re.1g) \quad (3\text{-}23)$$

where P_o is the peak impulse load given by Equation (3-22)

$\overline{W}$ is the effective subgrade mass given by Equation (3-21)

$\tilde{v} = V/\tilde{V}$ is the ratio of the vehicle speed to the characteristic speed in the subgrade

$\tilde{V} = 2\ell f_p$

The acceleration level given by Equation (3-23) is an estimate of the pavement response. This estimate must be adjusted for distance between the source (bump) and the receiver location.

From Equation (3-3), the approximate receiver peak impulse level is estimated as:

$$L_{pr} = L_{po}-10\log(D)-20\log(e)\alpha D \quad (3\text{-}24)$$

where L_{pr} is the peak impulse level at the receiver

D is the distance between the bump and the receiver.

The frequency content of the impulse loading will be much lower than that generated by random surface roughness. Also, the values of α appropriate for use in Equation (3-23) are generally lower than the values given in Table 3-2. For guidance, the values for α given in Table 3-2 may be divided by 3 to indicate the low frequency content of the impulse excitation. That is, impulse ground vibrations generated by bumps will attenuate less rapidly with distance than random ground vibrations generated by rough pavement.

Considering the low frequency content of the impulse acceleration level, building response to the impulse is quite similar to a very short duration low level earth tremor. Due to the short duration of the impulse, building amplification of the vibration may be less than that resulting from traffic moving along rough pavement (19).

3.5 Building Amplification and Criteria Levels

The results of Section 3.3 and 3.4 provide guidelines for the prediction of traffic-induced vibration at a location away from the highway alignment. This location corresponds to the foundation of a building. The receiver or building occupant is assumed to be on the building interior. Hence, the levels predicted for the foundation excitation must be adjusted for building amplification.

3.5.1 Expected Levels for Building Amplification

In the absence of field measured data, it is possible to estimate the expected amplification of ground vibration by building structure. The estimate of building amplification of traffic-induced vibration is based upon the work of House (3), Tokita (20), and Rudder (1). The nature of the building amplification factor, as used in these guidelines, is a simple addition of dB levels. For example, if the acceleration level at the foundation of a building is estimated to be -70 dB (re.1g) with an amplification of +10 dB (re. ground vibration level) for floors, the estimated building floor acceleration level would be -60 dB (re.1g). Figure 3-8 presents a plot of building amplification versus probability of <u>not</u> exceeding the amplification. The type of building structure considered by the data of Figure 3-8 is a frame residential house (20). The shaded areas indicate the degree of data scatter. As indicated in Figure 3-8, negative amplification (attenuation) of the ground vibration is possible but not very probable.

The building amplification of traffic-induced vibration appears to increase with increasing building storeys. For a one storey house the floor amplification ranges from -5dB to +15dB. For a two storey house, the first floor amplification ranges from -5dB to +10dB. For the second floor, the amplification ranges from -5dB to +15dB. As a conservative limit, a +15 dB amplification may be assumed.

Wall vibration may be expected to be on the order of magnitude of the floor vibration. Considering the room to be a box, the wall vibration will be in the same ratio as floor vibration as the ceiling height is to the floor dimension normal to the wall. For example, a 12 ft. x 15 ft. (3.7 m x 4.8m) room with an 8 ft. (2.4 in) ceiling might be expected to have a wall amplification of 8/12 = 0.667 (-3.5 dB) for the 8 x 15 wall and an amplification of 8/15 = 0.533 (-5.5 dB) relative to the floor (21).

The results of Figure 3-8 are generally applicable to buildings adjacent to roads with randon surface roughness. For impulse excitation resulting from vehicles striking a pothole, it can be expected that the building amplification may be less than that indicated in Figure 3-8. Until additional field measurements can refine the data, it is recommended that the values of Figure 3-8 be used.

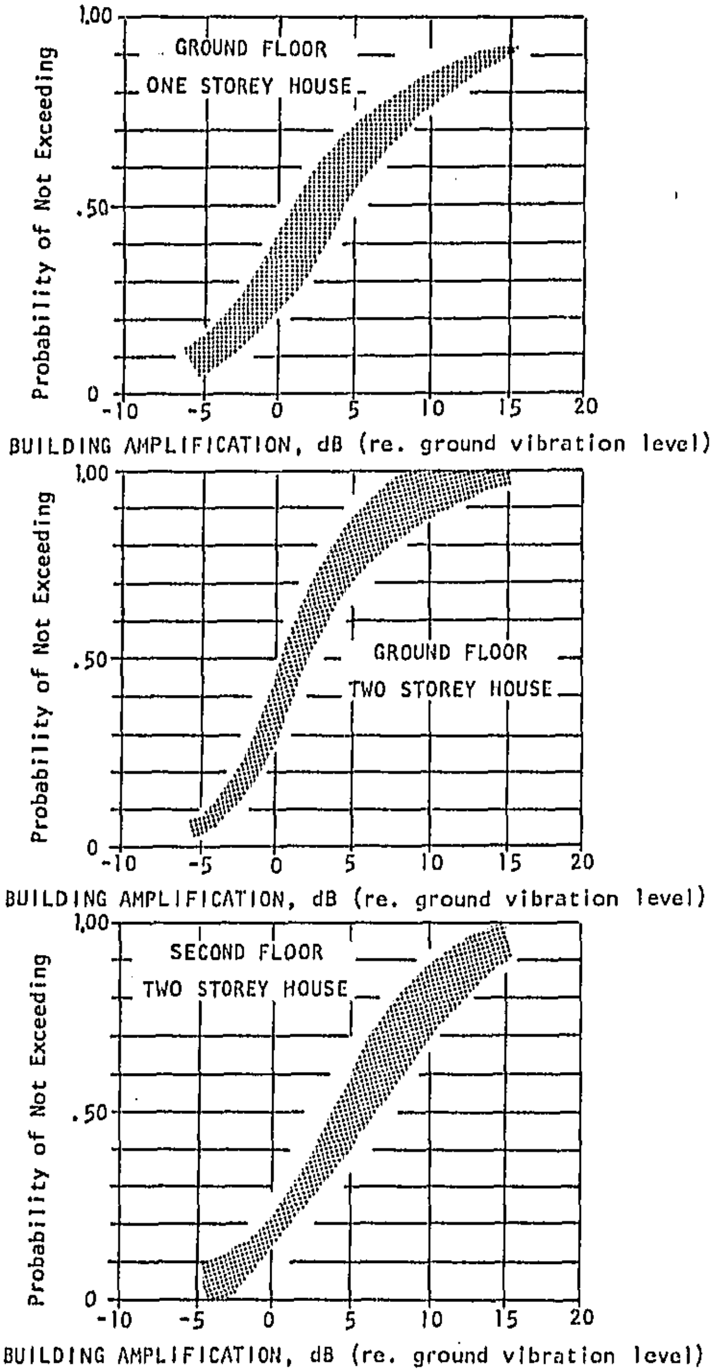


FIGURE 3-8. PROBABILITY OF NOT EXCEEDING BUILDING AMPLIFICATION

The application of the criteria presented in Figure 2-3 requires the estimation of the ground vibration spectra at the building foundation and the adjustment of the spectra based upon the dynamic characteristics of the building. Vibration analysis of extremely complex structures is possible using large computers. Bata (22) indicates such an approach. It is conceivable that formulating input data and conducting calculations that may be approximate - at best - could cost more than the value of a typical residence. Hence, the guidelines for building amplification factors appear both pracitcal and appropriate.

In general, building response to traffic-induced vibration appears to be non-resonant forced vibration. The ground vibration excitation appears generally as a transient harmonic oscillation for highway sources characterized by random surface roughness. Figures 2-6 through 2-8 are typical results (pages 25, 27, and 28). The excitation frequency of the traffic-induced vibration is generally less than the expected fundamental frequency for floor vibration. The building response is then controlled by the structural stiffness. This is, perhaps, why 2nd floor amplification factors indicated in Figure 3-8 are greater than values for the first floor.

3.5.2 Threshold Levels for Perception

The determination of the effects of traffic-induced vibration requires the application of criteria levels related to the measurement metric used. These guidelines use the acceleration level, expressed in dB (re. 1g).

The criteria curves presented in Figure 2-3 (page 13) indicate the frequency dependence of human response to vibration. Each curve describes an "acceptable" limit for the indicated activities. There can be little doubt that exceeding these limits will result in complaints.

The use of the criteria presented in Figure 2-3 indicates that a lower limiting value of -70dB (re.1g) is perhaps an appropriate threshold limit for design use. In the frequency range applicable to random roadway roughness (5Hz to 20 Hz) this design limit will allow a margin of safety of approximately 3 dB to 12 dB in evaluating impact.

The criteria curves in Figure 2-3 represent the "combined worst case x,y, or z axis" criteria of Reference 2. In the terminology of human response to vibration, the x-axis is the back-to-chest direction, the y-axis is the right-to-left side direction, and the z-axis is the foot (or buttocks)-to-head direction. That is, the criteria consider human postures of standing, sitting or lying down.

3.5.3 Threshold Levels for Potential Building Damage

The criteria curves presented in Figure 2-3 (page 13) indicate the frequency dependence of building structual damage potential. The criteria are based upon the recommendations of Reference (2). Each curve represents a curve of constant velocity.

The threshold for structural damage is taken as a constant velocity of 2.5 mm/s. This constant velocity corresponds to an acceleration level of

$$L_{so}(f) = -55.9 + 20\log(f) \qquad \text{dB(re.1g)} \qquad (3\text{-}25)$$

where f is the frequency in Hz.

The region in Figure 2-3 labeled "Damages Very Improbable" covers the velocity range between 2.5mm/s and 6mm/s. The limiting velocity of 6mm/s corresponds to an acceleration level of

$$L_{s1}(f) = -48.3 + 20\log(f) \qquad \text{dB(re.1g)} \qquad (3\text{-}26)$$

where f is the frequency in Hz.

The region in Figure 2-3 labeled "Minor Damage Possible" covers the velocity range between 6mm/s and 10mm/s. The limiting velocity of 10mm/s corresponds to an acceleration level of

$$L_{s2}(f) = -43.9 + 20\log(f) \qquad \text{dB(re.1g)} \qquad (3\text{-}27)$$

where f is the frequency in Hz.

The region in Figure 2-3 labeled "Structure Damage Possible" covers the velocity range from 10mm/s to 50.8 mm/s. The 50.8 mm/s limit is the 2.0 in./s "Safe Blasting" limit used by the Bureau of Mines U. S. Department of the Interior. The limiting value of 50.8 mm/s corresponds to an acceleration level of

$$L_{s3}(f) = -29.7 + 20\log(f) \qquad \text{dB(re.1g)} \qquad (3\text{-}28)$$

where f is the frequency in Hz.

Based upon complaint data, the nature of alledged building damage resulting from traffic-induced vibration is generally related to cracks in plaster, wall board, and separated grout around ceramic tiles. More serious complaints have alledged cracking of block and brick walls. Traffic-induced vibration has even been blamed for broken water pipes in residential yards (opinion of the plumber) and for broken dishes in a china cabinet (opinion of police investigating a vandalism call).

Except for vehicles leaving the roadway and striking a building, it does not appear that traffic-induced vibration can cause building damage on a single event basis. Any potential for building damage, it at all possible, can result only in a long-term exposure to repeated vibration excitation from both within and without the building.

The criteria curves of Figure 2-4 (page 18) indicate constant acceleration levels for the threshold of building damage. These criteria are based upon the number of intrusions of vibration per day. In Figure 2-4, the acceleration level of -26dB (re.1g) is taken as the "damage threshold for sensitive structures". This acceleration level corresponds to the 10mm/s upper limit of the "Minor Damage Possible" range of Figure 2-3 for frequencies around 10Hz. In Figure 2-4, the acceleration level of -20 dB (re.1g) is taken as the "damage threshold for normal residential structures with plastered ceilings and walls". This acceleration level corresponds to the lower range of the "Structure Damage Possible" range of Figure 2-3 for frequencies around 10Hz. Hence, the criteria of Figure 2-4 may be too lenient with respect to long-term building exposure to low level traffic-induced vibration.

Based upon the criteria of Figure 2-3, a lower limiting value of -35 dB (re.1g) has been indicated in Figure 2-4. The -35 dB limit is recommended as a threshold for traffic-induced vibration based on a structural damage potential for long-term exposure. This threshold level is an engineering judgement and indicates a limit above which the highway designer or planner should exercise caution.

The recommended threshold acceleration level of -35 dB (re.1g) is at least 35 dB above the lower limiting perception level for human ananoyance. The highway planner or designer can be assured that very strong complaints will be received before the -35 dB level is encountered. (The highest overall acceleration level recorded during the Reference 1 study was a peak of -24.4 dB with an rms level of -39.8 dB. This data point was for the same accelerometer and location as given in Figure 2-7. The building vibration was totally dominated by response to truck generated acoustic noise (airborne path) above 100 Hz. The building response to the traffic-induced ground vibration was -76 dB (spectrum level) at 5Hz.

3.6 Abatement Strategies for Traffic-Induced Vibration

There appears to be no ready solution that generally applies to the abatement of traffic-induced vibrations. Each situation must be treated as a special case. From a highway planning and design standpoint, these guidelines provide a rational methodology for assessing the potential for adverse impact from traffic-induced vibration. The highway designer can estimate the effects of traffic-induced vibration in the planning stage.

For engineers and public officials faced with immediate complaints, the best and most economical abatement strategy appears to be a rapid application of good public relations. As with any public relations approach, the complaint should be followed by visible action as soon as possible.

Public officials are apparently reluctant to admit that traffic-induced vibration problems exist (1). This is an understandable attitude since courts in the United States have awarded compensation to plantiffs alledging building damage resulting from traffic-induced vibration. (A legal summary is provided in Appendix III of Reference 1). Before 1976, there was almost no quantitive information concerning the characteristics of traffic-induced vibration. Hence, doubts conerning the appropriate strategy to use for the abatement of traffic-induced vibration are understandable. Hopefully, these guidelines will dispel many of the uncertainties associated with traffic-induced vibrations. That is, structural damages resulting from traffic-induced vibrations do not appear to be a highly probable situation.

For the purposes of these guidelines, abatement strategies are classified as Active Strategies, Passive Strategies, and Defensive Investigations. At some point, field measured data may be required. The topic of measurement of traffic-induced vibration is discussed in Section 4 of these guidelines.

3.6.1 Active Strategies

Active strategies for abatement of traffic-induced vibration are basically related to the engineering parameters characterizing the problem. As discussed in Section 2 of these guidelines, the engineering parameters characterizing the traffic-induced vibration problem are:

- Traffic parameters
- Pavement/subgrade parameters
- Propagation parameters
- Building parameters

The highway engineer or planner controls the traffic and the pavement/subgrade parameters. Building parameters can only be controlled via local building codes. There appears to be little benefit gained by attempting to alter the source-receiver propagation characteristics either on or off the highway right-of-way.

Pavement Smoothness: The first consideration for the abatement of traffic-induced vibration is the design, construction, and maintenance of smooth roadway surfaces. Potholes, bumps, ripples, etc. in the pavement surface can result in perceptable vibrations in building adjacent to the highway. Abrupt pavement discontinuities on the order of 1/2 inch (13mm) may be sufficient to generate perceptable vibration in buildings adjacent to the highway. These effects are described in Section 3.4.

Even smooth highways (PSR=3.0) may excite perceptable ground vibration at the edge of the roadway. (See Figure 3-1, Page 34). Based upon the estimate of Figure 3-1 on Equations (3-2), repairing a roadway surface from PSR=1.0 to PSR=3.0 would be expected to abate traffic-induced vibration approximately 8 dB for all vehicle weights and speeds.

Vehicle Speed and Weight Regulation: As described in Sections 3.2 through 3.4, vehicle speed and weight are primary variables for the traffic-induced vibration problem. Vehicles striking potholes or other types of bumps induce high impact loading on the pavement. This loading is very dependent upon vehicle speed, weight and suspension stiffness parameters. More important, however, is the fact that the high pavement loading results in a continued rapid deterioration of the pavement surface. Hence, a smooth roadway surface may rapidly become very rough and the potential for increased probability of traffic-induced vibration exists.

For random road roughness, the level of traffic-induced vibration appears to be a continuous function of both vehicle and vehicle weight. Vehicle speed appears to be a function of the pavement roughness. The results of Equation (3-2) indicate that decreasing posted speed limits by one-half may abate traffic-induced vibration approximately 5dB. This change in ground vibration level and the resulting decrease in building vibration levels may be quite significant. However, for roadways with posted speed limits of 40 miles per hour (64 km/h) or less, decreasing the speed limit may increase the traffic noise generated on the highway. As described in Sections 2.3 and 3.5, traffic-noise may induce high-frequency (above 50 Hz) building vibration. Heavy vehicles regulated to move at low speeds may aggravate a sensitive situation rather than resolve the problem.

Weight regulation of vehicles on the roadway is another potential abatement strategy for traffic-induced vibration. Heavy vehicles on a roadway are the focus of attention in traffic-induced vibration situations. Based upon the results of Equation (3-2), decreasing gross vehicle weight by factor of 2 would be expected to abate traffic-induced vibration 3dB. This may or may not be sufficient to resolve the problem. If heavy trucks of 50 thousand pounds (22.7 thousand kg) gross weight were prohibited from a roadway, it is possible that buses and medium trucks of approximately 25 thousand pounds (11.3 thousand kg) would be "identified" as the annoyance source since peak ground vibration levels would decrease only 3dB.

A combination of vehicle speed and weight regulation aimed at abating traffic-induced vibration may be possible. The particular combination of vehicle speed and weight regulation can only be assessed on a local basis. The methodology of these guidelines will hopefully assist in this respect.

Regulation of vehicle weight on a roadway implies a possible re-routing of traffic flow. If such re-routing is a practical alternative, based upon local conditions, the planner must ensure that he does not create or aggravate a problem along the alternate route.

<u>Trenches and Berms</u>: For highway traffic noise, one potential abatement method is the construction of barriers or berms along the highway. The effectiveness of a noise abatement barrier depends, in part, on the source height relative to the top of the barrier and on the source noise spectrum (wave length) (10).

A trench cut in the ground is the abatement analogue of a noise barrier for traffic-induced vibration. As is the situation for noise barriers, the effectiveness of trenches depends upon the source "elevation" relative to the bottom of the trench and the vibration source specturm (wave length). The design of trenches for vibration abatement is a highly specialized talent (4). First, trenches must be designed based upon the wave length of the incident vibration. The wave length is equal to the propagation speed of the vibration divided by the frequency of the vibration.

The vibration propagating away from a roadway is complex in nature, but it appears that the Rayleigh (R) wave is the more dominant wave associated with the traffic-induced vibration problem. The Rayleigh (R) wave essentially propagates at the transverse or shear wave speed of the soil (See Table 3-3, page 53). Typically, these propagation speeds are from 150 ft/s (46 m/s) to about 600 ft/s (183 m/s).

The general frequency range for traffic-induced vibration is from 5 Hz to 20 Hz with 10 Hz a typical number. Hence, the wave lengths for vibrations generated by highway traffic appear to be on the order of 15 feet (4.6 meters) to 60 feet (18.3 meters) based upon the 10 Hz excitation frequency.

Trenches are usually considered effective if the amplitude of the vertical surface motion is reduced to 25% of the no-trench condition (12 dB attenuation) within a semicircular area with a radius of one-half the trench length centered on the trench length. For example, a house with overall plan dimensions of 40 feet by 30 feet (12.2 m by 9.1 m) located 30 feet (9.1 m) from the roadway would require a trench approximately 150 feet (45.7 m) long to isolate the house. The depth of the trench would have to be scaled based upon the wave length of the Rayleigh (R) wave. Typically, the trench depth should be 1.2 to 1.5 wave lengths mininum (4). For traffic-induced vibrations, assuming a 15 foot (4.6 meter) wave length, it appears that the trench depth should be approximately 18 feet (5.5 m) to 22.5 feet (6.9 m). Hence, to isolate the house the trench would have to be approximately 150 feet (45.7 meters) long and about 20 feet (6.1 meters) deep. Trench width does not seem to be too important as related to attenuation.

Other than the obvious practical considerations of constructing and maintaining a deep trench, one must consider the potential for traffic noise to flank the trench and induce high frequency building vibration. The high-frequency building vibration may or may not be perceptable, but the resulting "rattles" may be annoying.

Berms, contrary to the analogy of barriers and trenches, present the possibility of attenuating both traffic noise and traffic-induced vibration (4). By reflecting and scattering incident Rayleigh (R) waves, berms may be used to decrease the amplitude of traffic-induced vibration. One propsoed use of berms as an abatement measure (23) indicates an approximate attenuation of 1 dB per each 1/4 wave length increase in topographic relief between the source and the receiver. For traffic-induced vibration, 1/4 wave length is approximately 4 feet (1.1 meter) to 15 feet (4.6 meter). That is, for terrain that is generally flat between the roadway and the receiver, a berm of approximately 20 feet (6.1 meter) in height with an included angle of approximately 60° might attenuate traffic-induced vibration about 3 to 4 dB. Depending upon the size of the building or buildings to be isolated, the length of the berm would have to effectively screen the building. As with any problem dealing with wave propagation in a real-world situation, extensive localized testing and geotechnical consulting would be required prior to initiating such a project.

3.6.2 Passive Strategies

Passive Strategies related to traffic-induced vibration are properly a form of public relations. The success of these strategies is based upon recognition that traffic-induced vibration is generally an annoyance problem. The extent to which a public agency may be able to address complaints is a local issue. However, some cities assign an individual as a public works ombudsman just to address issues such as traffic-induced vibration complaints. Public transit systems have uitlized retired drivers to visit residences to help identify situations in which speeding buses have generated complaints of traffic-induced vibration. An ombudsman can see that street repairs are expedited and improper operation of vehicles such as speeding can be minimized. The main point is that the complaint is addressed as quickly as possible and that the persons complaining know that something is being done.

3.6.3 Defensive Investigations

If a preliminary investigation indicates that normal road maintenance and/or other forms of public relations do not result in adequate abatement, defensive investigations may be in order. Alleged building damage usually is of the form of cracks and annoyance is usually related to "rattles" of building contents.

One need not use instruments to perceive traffic-induced vibration. By standing on the edge of the roadway as a vehicle passes by, one may be able to perceive ground vibration. Due to the low levels of ground vibration generated by traffic, it may be necessary to stand a long time so that several heavy vehicles pass. If ground vibration is felt at the edge of the roadway and the receiving building is within approximately 100 feet (30.5 meters), the vibration in the building may be great enough to be considered highly annoying.

If traffic-induced vibrations are not perceptable at the edge of the roadway and the receiver is within approximately 50 feet (15.2 meters), the vibration in the building may still be great enough to cause a high level of annoyance. (See Figure 3-8).

Eventually, one may have to enter the building either to observe or to measure traffic-induced vibrations. This is an absolute technique for assessing perception to traffic-induced vibration. As is the case with roadside observations, it may be required to observe a long time period to evalute the effects of several vehicles passing the building. Perception, if it occurs, will be quite similar to rattles and vibrations resulting from people walking about the house. As a practical matter, never take written notes during the observation period in a house. Always remain courteous and in all cases, inform local police of your activity prior to standing by a road for a long time in a residential neighborhood.

To evaluate the relative significance of alleged building damage, it is advisable to survey the building and to determine the general type of construction and building age. If complaints reach the point of potential litigation, survey buildings in the same area of similar construction and age that are not exposed to heavy vehicle traffic. Compile the data to determine any similar forms of damage or deterioration that may be related to building age and normal environmental factors.

One aspect of good public relations is the actual measurement of traffic-induced vibration in a building. There appears to be a positive attitude effect on the part of the complaining public in seeing technical measurements being taken. In particular, use instrumentation that has at least a direct indicating meter. Much to the surprise of the resident, you can show him that normal household activities will cause building vibration at least on the order of magnitude of traffic-induced vibration. This may appear to be a "snake oil" treatment; however, any data measured may well be available to the public in any event. Do nothing yourself to "induce" building vibration. Let the resident see what happens as a result of his normal daily activities. In particular, it may be necessary to ask a resident to turn off major appliances such as washers and dryers and airconditioners so that good traffic-induced vibration data may be obtained. Measurement and analysis of traffic-induced vibration is discussed in Section 4.

4.0 MEASUREMENT AND ANALYSIS OF TRAFFIC-INDUCED VIBRATION

The measurement and analysis of traffic-induced vibration is identical in concept to acoustic (airborne) traffic noise measurements. Traffic-generated noise and traffic-induced ground vibration both comprise a source-path-receiver scenario. The main differences between noise measurements and vibration measurements are related to test documentation and to detail instrumentation requirements. Test methodologies for traffic noise measurement are rather standardized (24). For traffic-induced vibration, however, standardization of detail test methodologies is non-existent.

The measurement and analysis of traffic-induced vibration must also recognize the purposes of the tests. Basically, the various types of tests that may be performed are categorized as:

- Source Emission tests: Verify the nature of the highway as a source of environmental vibration. The results are comparable to Equation (3-2) for traffic-induced vibration and to the results of References 24 and 25 for traffic noise.
- Propagation Tests: Verify the nature of the vibration propagation away from the highway. The results are comparable to Equations (3-3) and (3-4) for traffic-induced vibration and to the results of References 24 and 25 for traffic noise.
- Building Response: To verify the relative amplification of traffic-induced vibration by the building structure. The results are comparable to Figure 3-8 for traffic-induced vibration and of Reference 26 for traffic noise.
- Criteria Evaluation: To establish the validity of complaints and/or potential litigation related to traffic-induced vibration. The results would rely upon criteria such as presented in Figures 2-3 and 2-4. The comparable criteria for traffic noise are generally accepted.

Tests related to source emission, propagation, and building response are basically research-oriented in that the results would apply to refinement of prediction models. Tests for criteria evaluation are, however, the key issue and depend upon the engineer recognizing and quantifying the many aspects of traffic-induced vibration. The present lack of a documented and standardized data base for traffic-induced vibration is, perhaps, the most salient aspect of this problem. Accurate measurement and reporting of test data for complaint assessment and/or criteria evaluation is the issue. However, data collected and reported as a result of complaint assessment could form a basis for future refinement of prediction methods and criteria evaluation.

Traffic-induced vibration appears, at the present, to be a site specific problem. That is, the various parameters relating the source emissions, vibration propagation, building response and criteria evaluation are localized data. These basic parameters appear to be:

- SOURCE
  - (a) Vehicle Data - Gross Weight and Speed*
  - (b) Pavement Data - Surface Roughness and Structural Details*
  - (c) Subgrade Data - Density and Stiffness (See Tables 3-3 and 3-4)
- PROPAGATION
  - (a) Soil Data and/or "Loss Factors" (See Table 3-2)
  - (b) Site Topography (See Section 3.6.1)
- BUILDING RESPONSE
  - (a) Classification of Structure (Frame, Masonry, etc.)
  - (b) Age of Structure
  - (c) Measurement Location (Storey, floor, etc.)

The above is not necessarily a "shopping list" of everything that may or may not be very important at a site. However, the conduct of field tests of traffic-induced vibration may require a rather extensive and thorough site documentation. Available data, however, seem to indicate

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* The theory of Reference 1 indicates detail vehicle dynamic parameters required to evaluate vibration source emissions.

that large site-to-site variations in levels of traffic-induced are not readily apparent. Hence, the quantitative effect of varying a single parameter is not immediately evident in the form of potential abatement and/or relief from traffic-induced vibration (See Section 3.6). The combination of heavy vehicles, rough pavement, and residential structures close (100 feet or 30.5 m) to the pavement are the basic ingredient for a complaint.

These guidelines are prepared to provide general direction to the measurement and analysis of traffic-induced vibration data. The specific nature of traffic-induced vibration is addressed in describing instrumentation and appropriate methodology for data reduction.

4.1 Instrumentation Operating Envelope and Characteristics

The general characteristics of traffic-induced vibration are the form of a transient forced vibration. The duration of the signal is typically 5 seconds or less. The frequency content of the data is generally discrete — almost pure tone — in nature. For ground vibration induced by highway traffic, the frequency range containing significant data is 2 Hz to 50 Hz. The generation of traffic-induced vibration above 50 Hz appears to be related to acoustic (airborne path) noise from the traffic.

Typically, the amplitude range required for monitoring traffic-induced vibration will be from 10^{-4} g_{rms} to 10^{-1} g_{rms} (-80 to -20 dB (re. 1 g_{rms})). Since vibration instrumentation may be either displacement-sensitive, velocity-sensitive, or acceleration-sensitive, the instrumentation operating envelope for traffic-induced vibration measurements is presented in the format of Figure 4-1. In Figure 4-1, the horizontal axis is frequency and the vertical axis is velocity in meters/s. Axes for displacement in meters and acceleration in meters/s^2 are indicated. The shaded area is a recommended operating

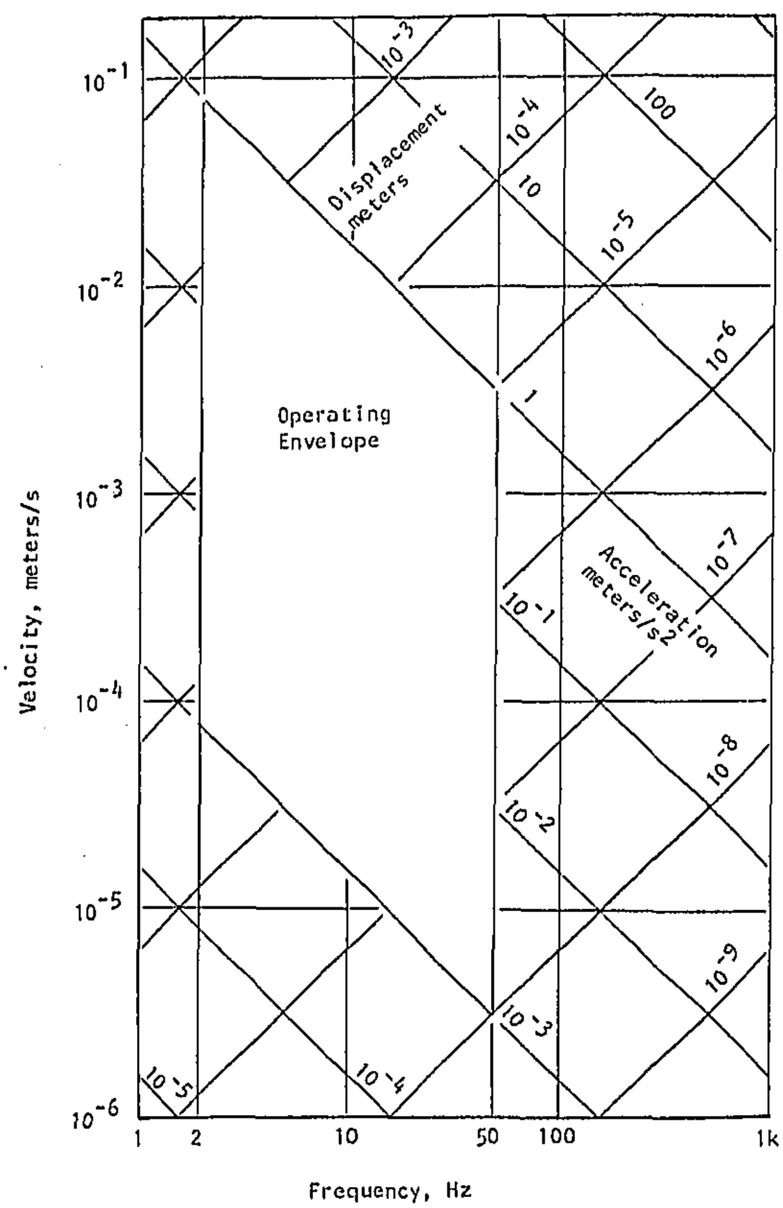

FIGURE 4-1 - INSTRUMENTATION OPERATING ENVELOPE

envelope required for instrumentation used to monitor traffic-induced vibration. All elements of an instrumentation system (transducers, amplifiers, meters and/or recorders) should exhibit linear frequency response* within this envelope. A 40 dB dynamic range on instrumentation is quite adequate to characterize the data. A good basic technical discussion of seismic instrumentation is presented in Chapter 9 of Reference 4.

The interpretation of Figure 4-1 is really quite simple. For a pure sine wave of frequency $f=\omega/2\pi$ Hz, the magnitudes of the vibration displacement, velocity, and acceleration are related as follows:

- Velocity ($\dot{X}$) and Acceleration ($\ddot{X}$) in terms of Displacement (X):

$$\underline{\dot{X} = \omega X = 2\pi f X} \qquad \ddot{X} = \omega^2 X = (2\pi f)^2 X \qquad (4\text{-}1a)$$

- Displacement (X) and Acceleration ($\ddot{X}$) in terms of velocity ($\dot{X}$):

$$X = \dot{X}/\omega = \dot{X}/(2\pi f) \qquad \ddot{X} = \omega\dot{X} = 2\pi f\dot{X} \qquad (4\text{-}2a)$$

- Displacement (X) and Velocity ($\dot{X}$) in terms of Acceleration ($\ddot{X}$):

$$X = \ddot{X}/\omega^2 = \ddot{X}/(2\pi f)^2 \qquad \underline{\dot{X} = \ddot{X}/\omega = \ddot{X}/(2\pi f)} \qquad (4\text{-}3a)$$

The underlined expressions in Equations (4-1) and (4-3) are the relationships used to relate the displacement and acceleration magnitudes to velocity in Figure 4-1. Hence, depending upon the type of transducer employed, equipment may be selected to measure traffic-induced vibration data if the operating characteristics comply to the envelope presented in Figure 4-1. The user must remember that the criteria presented in Figures 2-3 and 2-4 are based upon acceleration. By measuring either displacement or velocity instead of acceleration, the frequency content of the vibration data must be determined to use the acceleration criteria directly.

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* An exception is taken in the case of the frequency weighting characteristic for acceleration described in Reference 2.

Concerning the requirement for frequency analysis of traffic-induced vibration, the discrete frequency nature of the data simplifies the analysis requirements considerably. Figure 4-2 presents a typical acceleration time-history for the vertical component ground motion at a distance of 25 feet (7.62m) from the edge of the pavement. The record is for a heavy truck passing by at 35 miles per hour. The vertical axis is acceleration in "gravity units" or "g's". The horizontal axis is time in seconds. By counting positive or negative peaks per second (one positive and one negative peak per cycle) one obtains an estimate of 8 or 9 cycles (peaks) per second. Hence, an oscillograph record is quite adequate to estimate the general frequency characteristics of traffic-induced vibration provided that the 50 Hz upper frequency limit in Figure 4-1 is recognized. This limit is recommended so that high-frequency noise-induced vibration does not contaminate the data (See Figures 2-7 an 2-8) and preclude a simple form of frequency analysis such as described above. The 50 Hz upper frequency limit can be achieved by inserting a low-pass filter in the instrumentation.

Figure 4-2 introduces some terminology common to vibration engineers. The terminology is as follows:

- Peak Amplitude, X_{peak}: Either the maximum value or the minimum value of the oscillation in one cycle. For transient data, such as Figure 4-2, the peak amplitude will denote the maximum value during the entire time record.
- RMS Amplitude, X_{rms}: The root-mean-square value of the vibration amplitude. For discrete frequency data the rms amplitude is, ideally, $X_{rms} \approx X_{peak}/\sqrt{2}$.

The parameter, X, as used above may denote either a displacement, velocity, or an acceleration amplitude.

Vibration data may be expressed in decibel units just as the common practice for noise analysis. The vibration data is then called a vibration level in dB. The vibration level is defined as:

$$L = 10\log(X/X_o)^2 \quad \text{dB (re. } X_o) \qquad (4\text{-}2)$$

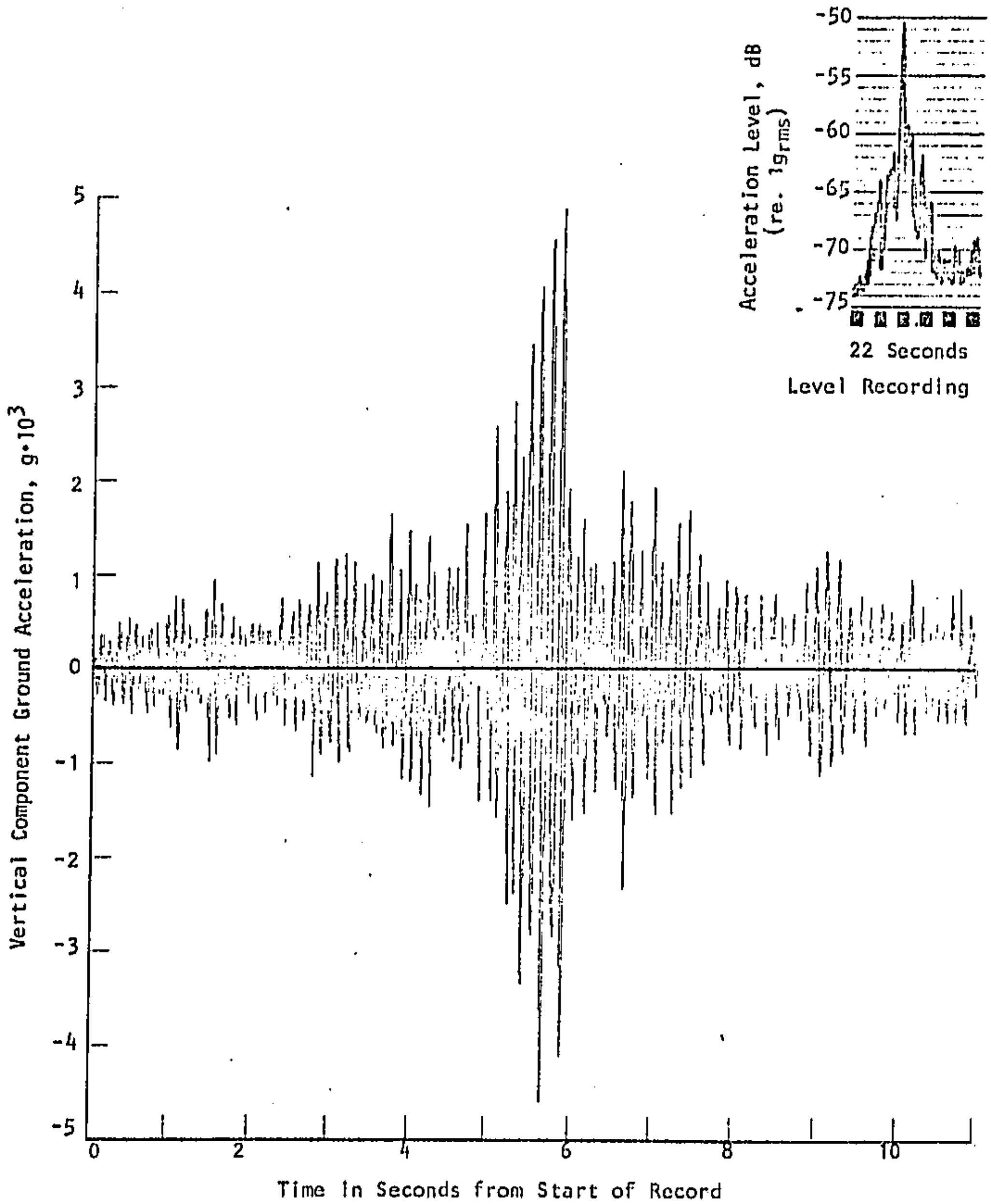


FIGURE 4-2 - TYPICAL TRAFFIC-INDUCED GROUND VIBRATION TIME HISTORY

where X denotes either a displacement, velocity, or acceleration
X_o denotes a reference value of displacement, velocity or acceleration.

The usage of the dB scale for vibration data is quite practical. However, the reference value must always be quoted – especially as to whether the value is a peak amplitude or an rms amplitude. The above should not appear confusing; especially if one considers that traffic noise monitoring is really the analysis of transient vibration of the air with noise levels referenced to an rms pressure of 2×10^{-5} N/m^2.

For vibration data characterized by a discrete frequency signal, the vibration level is called a spectrum level. The spectrum level is the level in a frequency bandwidth 1 Hz wide. The vertical scale in Figures 2-6 through 2-8 is in units of "g^2/Hz" or power per unit bandwidth. The spectrum level is simply ten times the logarithm of this number. The total power of the signal between any two frequency limits is the area under the spectra between the frequency limits. The low frequency content of traffic-induced vibration data is the important characteristic (See the Criteria of Figure 2-3). Hence, if instrumentation is sensitive to high frequency components, the high frequency data will dominate the total power and obscure the low frequency data.

For the measurement of traffic-induced vibration, a low-pass filter with an upper frequency limit of 50 Hz allows the use of the overall vibration level as a single number metric for criteria evaluation. This conclusion is based upon the general observation that traffic-induced vibration, as measured on the ground or in buildings, is almost a discrete frequency or "pure tone" signal.

For example, the inset in Figure 4-2 indicates a level recording of the data fo the time-history. The vibration level is expressed in dB (re. 1 g_{rms}) and represents the overall vibration level from 2 Hz to 1000 Hz. The maximum vibration level is -50.5 dB (re. 1 g_{rms}) or:

$$a_{max} = a_o 10^{L_{max}/20} = (1)10^{-50.5/20} = 2.985x10^{-3}\ g_{rms}$$

The maximum rms acceleration is converted to the maximum peak acceleration by multiplying by $\sqrt{2}$ to obtain:

$$a_{peak} = \sqrt{2}\ a_{max} = 4.222 \cdot 10^{-3}\ g_{peak}$$

and $$L_{peak} = 10\log(4.222 \cdot 10^{-3})^2 = 47.5 \text{ dB (re. 1 } g_{peak}).$$

From the time-history of Figure 4-2, the maximum peak acceleration is +0.005 and -0.0045 g. Hence, the assumption of a pure tone signal appears appropriate for the conversion of rms values to peak values.

If it is required to conduct frequency analyses of traffic-induced vibration data, the transient characteristics of the data must be recognized. The inset in Figure 4-2 indicates that the acceleration data within 20 dB of the peak persists for approximately 5 seconds. For traffic noise data (25), the duration of traffic noise level during noise emission testing is typically on the order of 2.5 seconds to the 6 dB down points. Hence, averaging time of the frequency analyzer must be considered. It is recommended that, if a frequency analysis of the data is required, the maximum band levels during the transient be reported. Due to the low frequency content of the data, 1/3 Octave Band analysis is the widest recommended filter bandwidth appropriate for the traffic-induced vibration problem. Such analysis techniques imply tape recorded field test data similar to that used for traffic noise testing. The low frequency content of the vibration data requires FM recording rather than direct recording common to noise analyses (27). Figure 4-3 illustrates a typical data recording system for monitoring traffic noise (direct record channel) and traffic vibration (FM record channel).

4.2 Multi-Channel Instrumentation Requirements

The requirement for multi-channel instrumentation to monitor traffic-induced vibration depends upon the purpose of the tests and the available instrumentation budget. To conduct propagation tests and building response tests, simultaneous measurement of vibration data at separated locations is required. Whether or not this is best achieved using several single-channel systems or a single multi-channel system is probably personal preference.

The total description of seismic vibration requires the measurement of three components of the ground motion: longitudinal, transverse, and vertical (4). For surface (Rayleigh) wave propagation the vertical component is the greatest in magnitude and attenuates most slowly with distance (4), (28). Additionally, the theory of wave propagation in simple elastic systems is mathematically complex and experimental verification is difficult. By comparing relative magnitudes, it appears that basing experimental results on only the vertical component of the ground motion could, at most, result in an error of approximately 3 to 4 dB for an idealized test. One must remember, however, that the vehicle dynamic forcing of the pavement and the pavement response is dominantly in the vertical direction. Further, the forcing and the response of the pavement is a random process and when viewed by a fixed instrument location adjacent to the pavement is a non-stationary random process. Hence, the simultaneous measurement of three-component ground vibration at a single location should not be a dominant consideration in determining multi-channel instrumentation requirements for traffic-induced vibration. It appears that the measurement of traffic-induced vibration requires only the vertical component of the ground motion.

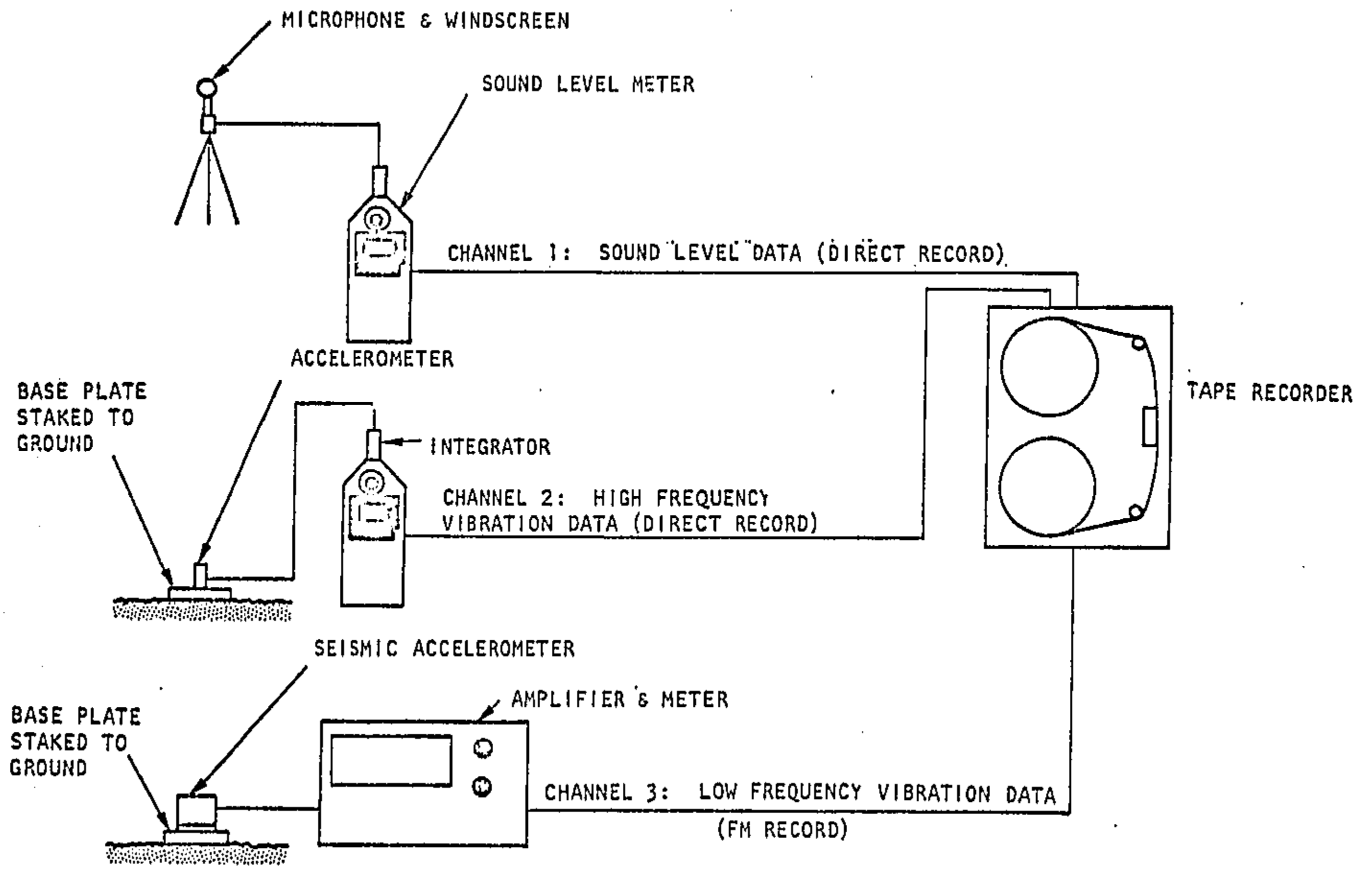


FIGURE 4-3 - TYPICAL DATA RECORDING SYSTEM

From a practical standpoint, each transducer should have an independent amplifier and indicating meter. The amplifier is required to set appropriate levels for each channel and the indicating meter is to ensure good signal quality during the test. The amplification should cover a 40dB range so that when coupled to the transducer sensitivity, peak levels fall in the operating envelope of Figure 4-1. A dynamic range of 40 dB for each channel appears to be very adequate for the traffic-induced vibration problem.

4.3 Site Data to be Recorded

The basic data to be measured to document vibration induced by highway traffic are:

- Maximum or peak vibration level
- Dominant frequency of vibration
- Duration of vibration above a level ΔdB below the peak (10 dB minimum).

These data reflect that even for dense high-speed traffic flows, each vehicle appears as a distinct single event vibration source. Figure 4-4 presents a typical road-side measurement for a local street and Figure 4-5 presents a typical road-side measurement for an interstate highway. The significant aspects of Figure 4-4 and 4-5 are:

- Each vehicle appears as a distinct source or peak
- Minimum levels are below the perception threshold level (See Figure 2-3)
- Maximum levels are comparable between sites (in the range of -55 to -40 dB (re. 1 g_{rms})).

The remainder of the data to be recorded at the site comprises documentation of the measurement locations and the documentation and/or measurement of vehicle and site parameters. The experimental program can be guided by the discussion of Section 2.3 and the theory of Section 3.

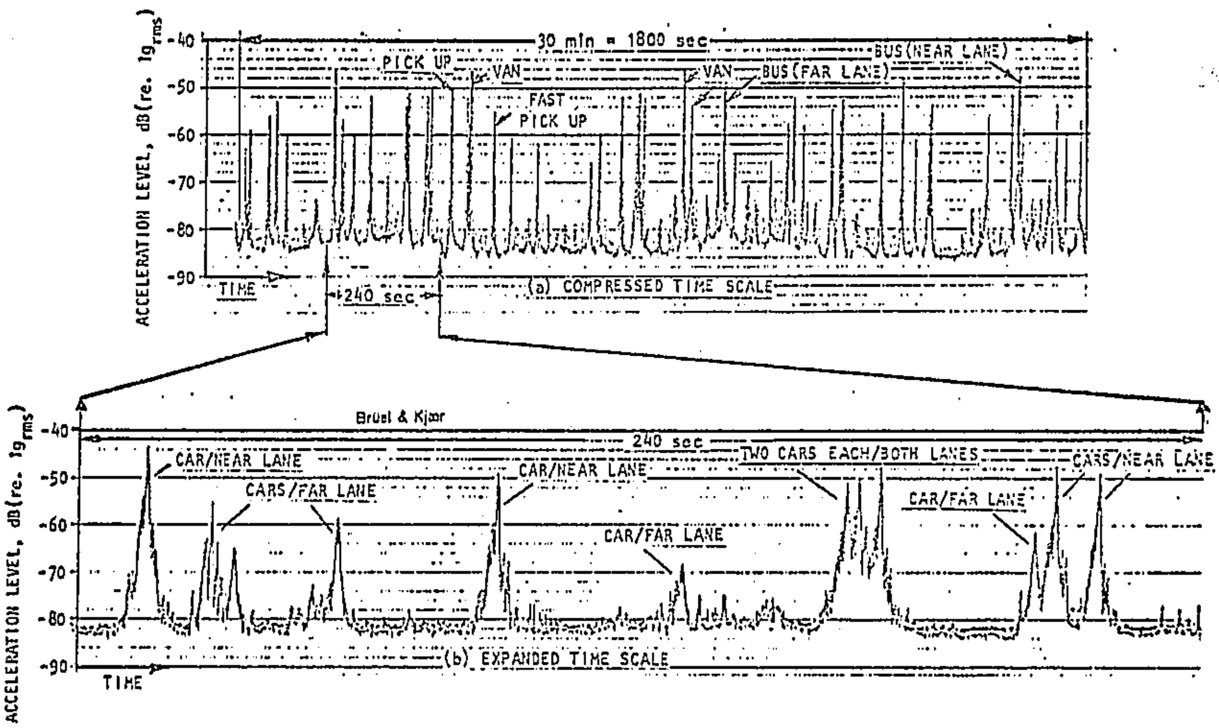
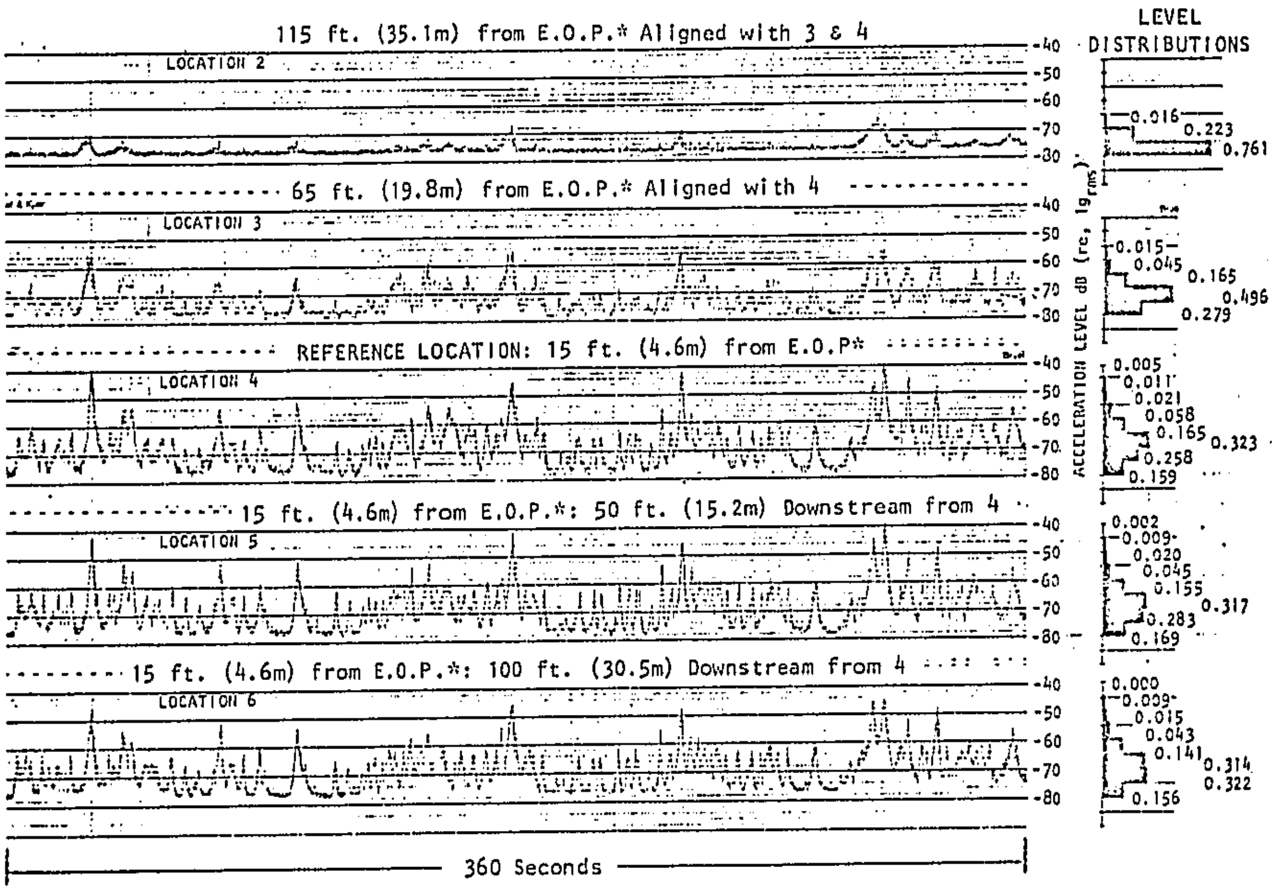


FIGURE 4-4 - TRAFFIC-INDUCED GROUND VIBRATION: RESIDENTIAL STREET



* E.O.P. denotes Edge Of Pavement

Figure 4-5. TRAFFIC-INDUCED GROUND VIBRATION: INTERSTATE HIGHWAY

As a minimum, the pavement surface roughness and the site soil classification should be estimated. In a purely technical sense, one would determine the surface roughness power spectra (See Figure 2-5 and References 6 and 14). The costs of such tests, however, would probably exceed the costs of the vibration tests. From a practical standpoint, photographs of the surface should be taken and a standard description of the pavement condition (29) should be reported. If possible, estimate the PSR (Present Serviceability Rating) of the pavement roughness. Document the location, depth or height, and width of all potholes and/or bumps on the pavement within 100 feet (30.5 m) on either side of the closest measurement point to the roadway. The number and widths of the traffic lanes should be reported as well as the posted speed limit.

It is desirable to also document pavement/subgrade data and site soil classifications if possible. The present "state-of-the-art" of traffic-induced vibration prediction does not warrant special soil testing specifically for the purposes of measuring vibration levels. However, if one can obtain such data from other sources, it should be reported.

As a minimum, the pavement type (flexible or rigid) and soil classification (See Table 3-2) should be reported. Other data to be reported, if available, are:

- Pavement Thickness, Material, and Density
- Subgrade Depth to Firm Base
- Density of Subgrade Material

4.4 Site Calibration Measurements

Site calibration measurements comprise the determination of vehicle vibration reference emission levels such as presented in Section 3.2 (See Equations (3-1) and (3-2)). For random surface roughness the basic form of the site "calibration" equation is

$$\begin{aligned} L_o = {} & A \cdot \log \text{ (Surface Roughness Parameter)} \\ & + B \cdot \log \text{ (Speed)} \\ & + C \cdot \log \text{ (Gross Vehicle Weight)} \\ & + \text{"site constant".} \end{aligned} \tag{4-3}$$

The above functional relationship is for a reference location, D_o, along the side of the roadway off the pavement. For traffic noise emission testing, the standardized reference distance is 50 feet (15.2 m). The reference distance for traffic-induced vibration is not standardized. As indicated in Section 3, the prediction results quoted are for a reference distance of 6.5 feet (2 mèters) from the pavement edge (approximately half a lane width). For all of the prediction equations in Section 3, the parameter D_o has been explicitly stated since the distance is not standardized.

Two basic considerations must be made to establish a reference distance for site calibration measurements. First, the reference distance should be far enough away from the edge of the pavement/subgrade structure so that the measurement location is in the "far field" of the vibration source (vehicle/pavement system). Little guidance can be given for the exact coupling characteristics of the pavement to the subgrade and the transmission of vibration to the soil system immediately adjacent to the pavement. It is evident, however, that significant vibration attenuation does occur immediately in the area adjacent to the pavement. Reference 30 indicates a consistent 20 dB attenuation of piling vibration from the piling to the adjacent soil. This attenuation may also be frequency dependent. It appears, however, that for the traffic-induced vibration problem, the selection of a reference distance on the order of one to one-half lane widths is sufficient if the site characteristics meet the next consideration.

The second consideration in selecting a reference distance for site calibration of vibration levels is that the transducer should not be attached to pavement, slabs, or other structures that may be adjacent to the roadway. The transducer should be in contact with the parent soil below the surface root or top soil. Figure 4-6 presents a sketch of a support plate for attaching a seismic accelerometer to the ground for measuring vertical component motion (1). The support plate is "attached" to the soil using four common "gutter" spikes.

As indicated in Equation (4-3), four parameters are required to establish the specific form of the site calibration equation. These four parameters are the coefficients A, B, C, and the "site constant". In the form of Equation (4-3), the surface roughness and the vehicle speed are presented as independent parameters. The available theory, however, indicates that the contribution of vehicle speed to the vibration level depends upon the pavement surface roughness (1). Also, the available theory indicates that gross vehicle weight is appropriate to distinguish between vehicle classes such as light vehicles and heavy vehicles but, for a specific vehicle, depends upon details of the vehicle weight and the dynamics of the suspension system. The "site constant" appears to be related to details of the pavement/subgrade structure and would include coupling losses between the pavement/subgrade system and the adjacent parent soil system. A refinement of the site calibration equation could be formulated. However, such a formulation would expand "site calibration" from a rather simple task to an extensive research project.

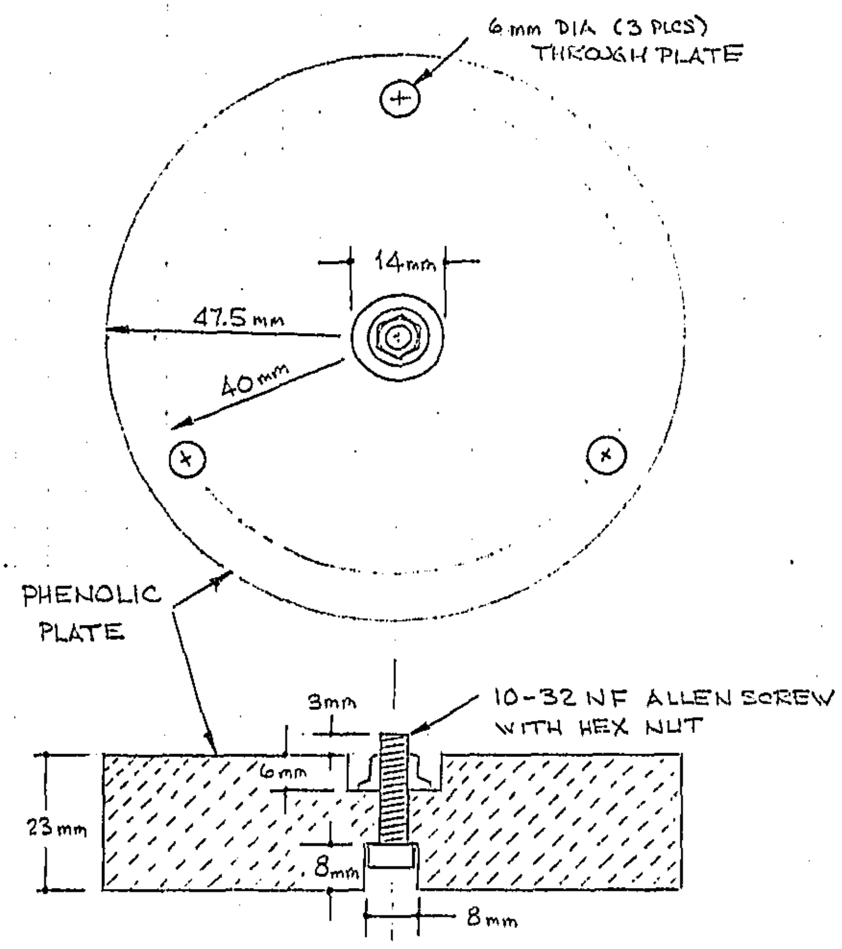


FIGURE 4-6 - ACCELEROMETER SUPPORT PLATE
FOR GROUND VIBRATION

To conduct a site calibration, the basic parameters are:

- surface roughness
- vehicle speed
- gross vehicle weight.

The most difficult parameter to estimate quantitatively is the "surface roughness". Figure 2-5 presents an approximate relationship between the surface roughness power spectral density and the Present Serviceability Rating. By estimating the rms surface roughness amplitude using the result of Figure 2-5, one obtains amplitudes on the order of 0.125 inch_{rms} (3.2 mm_{rms}) for PSR=1.0 to 0.020 inch_{rms} (0.5 mm_{rms}) for PSR=5.0. This is done by integrating the surface roughness spectral density formation over wavelengths from 1 foot (0.3 m) to 50 feet (15.2 m). These estimates are consistent with Tokita (2) who quotes rms surface roughness amplitudes on the order of 1 mm to 4 mm. Craigs (31) reports values of rms roughness as 0.168 inch (4.3 mm) for a "fair highway" and 0.098 inch (2.5 mm) for a "smooth highway". The accurate measurement of the rms surface roughness amplitude is perhaps as difficult as the measrument of the roughness spectral density. The quantitative description of surface roughness in a form other than a power spectrum on a measurement of the rms amplitude will be as difficult to obtain and perhaps not as accurate. The PSR relationship indicated by Equation (3-2) is such an attempt.

As an alternate, roadways may be classified as "smooth" and "rough" and the resulting experimental data grouped and analyzed accordingly. In this instance, the surface roughness parameter would be grouped with the "site constant". The result may be a greater "data scatter", but the present data base does not allow the estimation of the data scatter.

As indicated by Equation (4-3), the "speed" term applies to the vehicle passing along the roadway adjacent to the site. The dynamic pavement loading produced by a cruising vehicle is proportional to the vehicle speed raised to a power (i.e., V^n). The value of the exponent, n, depends upon the surface roughness.

The "Gross Vehicle Weight" term in Equation (4-3) is an approximation to theoretical estimates of the pavement loading. It appears, however, to be appropriate to consider only gross vehicle weight instead of a more complicated equation. The basic parameters which might be included in lieu of gross vehicle weight are: axle weight and total tire stiffness for the most heavily loaded axle. Ground vibration data do not exhibit any characteristics that indicate specific axles on a vehicle as a source. That is, the entire vehicle "appears" as a single source. However, the dynamic pavement loading is proportional to the total tire stiffness supporting an axle. In terms of the stiffness of a single tire (See Section 3.4.1), dual tire systems have twice the stiffness and dual-tandem tire systems have four times the stiffness of a single tire.

The "site constant" term in Equation (4-3) combines several effects that are difficult to estimate theoretically. The most important aspect of the "site constant" is the "coupling loss" between the pavement/subgrade system and the parent soil adjacent to the pavement. Other parameters that might be grouped with the "site constant" are the mass of the pavement/subgrade system and the pavement stiffness (See Section 3.4.1, pp. 50 to 55). Such detail, however, does not appear warranted at the present time since it is not possible to estimate "coupling losses" accurately. It will be good experimental practice, however, to document the pavement type as "flexible" or "rigid" during a site calibration test. The data would then be grouped to yield prediction equations for vibration emissions applicable to either flexible pavements or rigid pavements.

The above discussion is not intended to cloud the issue of site calibration testing but to present a discussion of the significance of each term in Equation (4-3). By documenting the site, the vehicle(s), and the speed(s) corresponding to the measured vibration data, a consistent data set will result. To obtain the specific values for the site calibration equation, a multi-variable regression analysis is required. Such procedures are identical to that used for developing noise emission prediction equations for traffic noise prediction models (25).

Site calibration tests should be conducted to achieve as wide a range of parameter variations of vehicle speed and weight as possible. To achieve this variation one should use light weight vehicles such as automobiles or station wagons, medium weight vehicles (medium trucks and transit buses), and heavy vehicles (heavy trucks with more than 3 axles). The speed range should correspond to limits appropriate for the vehicle cruise condition. This requirement is comparable to that used for standard vehicle noise emission testing (25).

It is not recommended that "bump" tests be utilized for the purpose of site calibration (See Section 3.4). Any surface roughness condition or vehicle operating mode that allows the tires to suddenly "launch" and impact the pavement will result in erratic data scatter. The data scatter may be reduced using the concept of an impact factor and the results of Section 3.4 of these Guidelines. However, very specific data concerning the vehicle suspension system, vehicle speed and "bump geometry" are required. If the site surface roughness is such that vehicles impact a natural bump, the bump geometry and location relative to the transducer should be noted in the site documentation.

4.5 Criteria Evaluation

Criteria Evaluation Measurements comprise tests to determine the severity of traffic-induced vibrations resulting from a complaint. Such tests should be conducted using the traffic source present at the site in order to quantify the nature and the severity of a complaint. Measurements should, preferably, be taken in the building at locations identified by the occupant. Guidance as to exact instrument location and direction of the measurements can only be stated in general terms. Basically, the direction for measurements should be vertical for footings and floors and horizontal for walls. It will be difficult to attach instrumentation rigidly to building interiors without damaging finished surfaces. This is perhaps the most difficult aspect of conducting measurements to evaluate a complaint.

If it is not possible to locate instrumentation in the building interior, measurement locations around the building must be selected. As a rule, however, locate the transducer as close as possible to the foundation. Coupling losses will occur between the foundation and the parent soil as described in Section 4.4. In this situation, the ground vibration measurements must be adjusted for "building amplification". The vibration magnification of a building can only be roughly estimated using either the results of Figure 3-8 or similar data that may be available. The degree of accuracy of this procedure is open to doubt unless the maximum levels of traffic-induced vibration are at least 20 dB below the criteria levels of Figure 2-3.

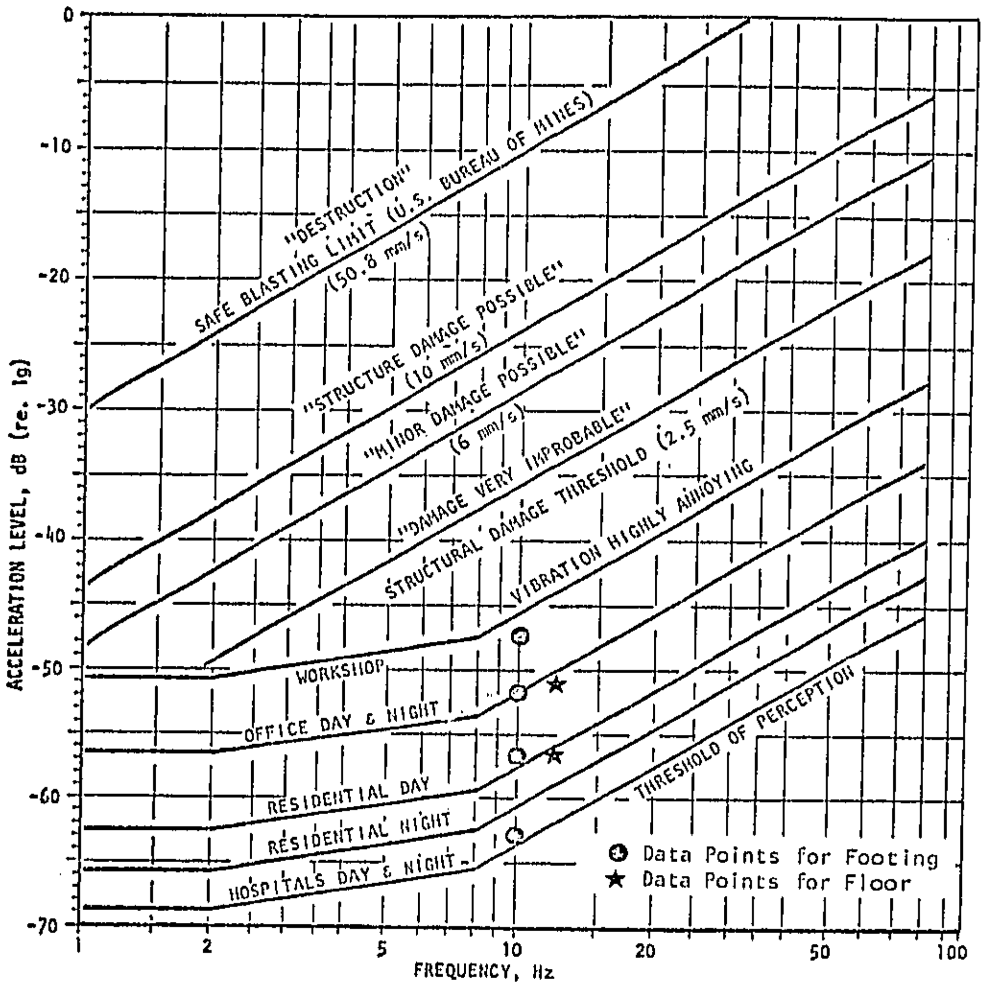
As an example of the procedures to be used to evaluate a complaint, reference is made to Figure 2-6 and the discussion on page 24. For a sample of six buses passing along the two lane street the following maximum rms acceleration levels (dB, re. $1g_{rms}$) were measured.

Bus	#1	#2	#3	#4	#5	#6	Mean	Standard Deviation
$(L_{rms})_{floor}$	-51	-51	-57	-51	-57	-57	-54.0	3.3
$(L_{rms})_{footing}$	-51	-51	-63	-47	-57	-57	-54.3	5.8
Amplification	0	0	+6	-4	0	0	+ 0.3	3.2

The frequency of the floor vibration was in the range of 10 Hz to 16 Hz. (See Figure 2-6.) The frequency of the footing vibration was in the range of 12 Hz. Figure 4-7 presents the data plotted on the criteria curves of Figure 2-3. Even considering the data scatter (mean level plus three standard deviations), the maximum vibration levels are approximately 12 dB below the indicated "structural damage threshold" and 20 dB below the "minor damage possible" curve. Based upon the criteria, however, it is indicated that the occupants would be highly annoyed. They were. Figure 4-8 indicates this data relative to the number of occurrences per day.

This example also indicates the extent to which the use of the "building amplification" data of Figure 3-8 is sufficient. For a one storey house, the estimated amplification for a probability of not exceeding of 0.9 is about 10 to 15 dB. For the mean value of -54.3 dB (re. 1 g_{rms}) for the footing in the above example, one would estimate resulting interior levels of -44.3 dB to -39.3 dB. The measured floor acceleration level was -54 dB and the standard deviation is 3.3 dB. For a peak level 3 standard deviations above the mean, one would estimate a "probable worst case" of -44.1 dB. Hence, the experimental values seem to be consistent with the approximation procedure.

In any event, to evaluate criteria it is required to obtain the building response spectra to the transient input. Figure 4-9 is an example of a 1/3 Octave Band spectrum for one of the bus pass-by measurements reported above. The spectra are each related to a probability of exceeding a level for the transient data. It is evident



(SEE Figure 2-3)

Figure 4-7. EXAMPLE OF CRITERIA EVALUATION OF VIBRATION LEVEL

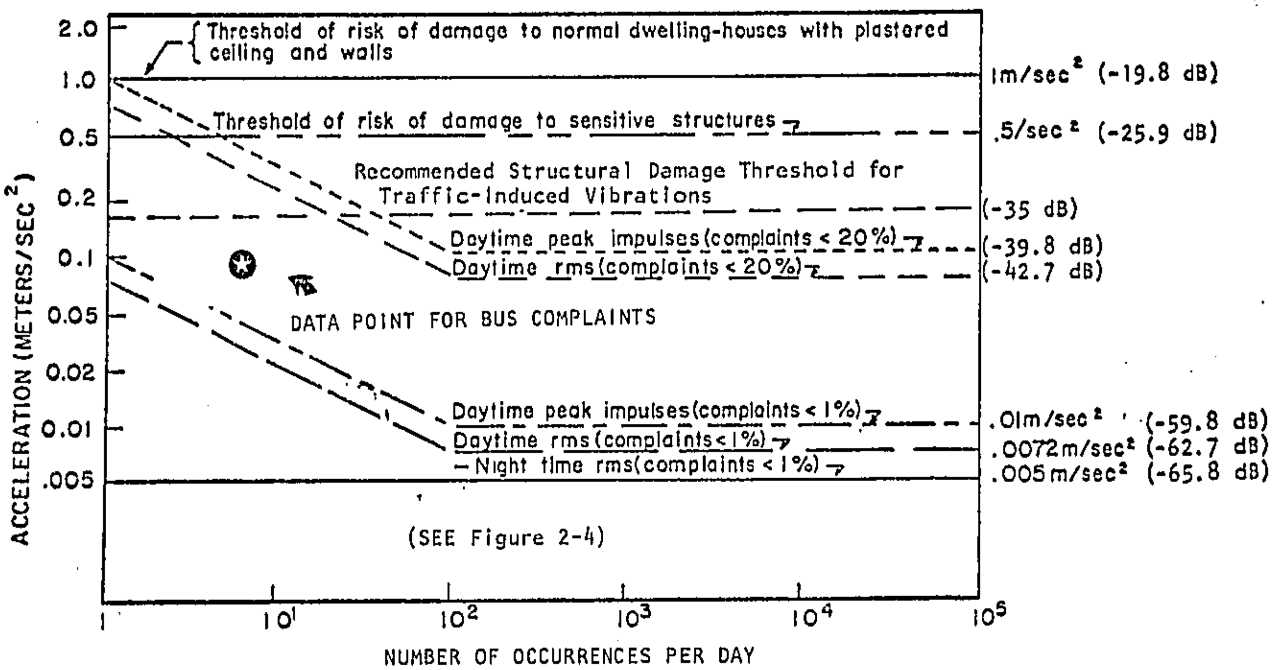


Figure 4-8. EXAMPLE OF CRITERIA EVALUATION OF NUMBER OF OCCURRENCES

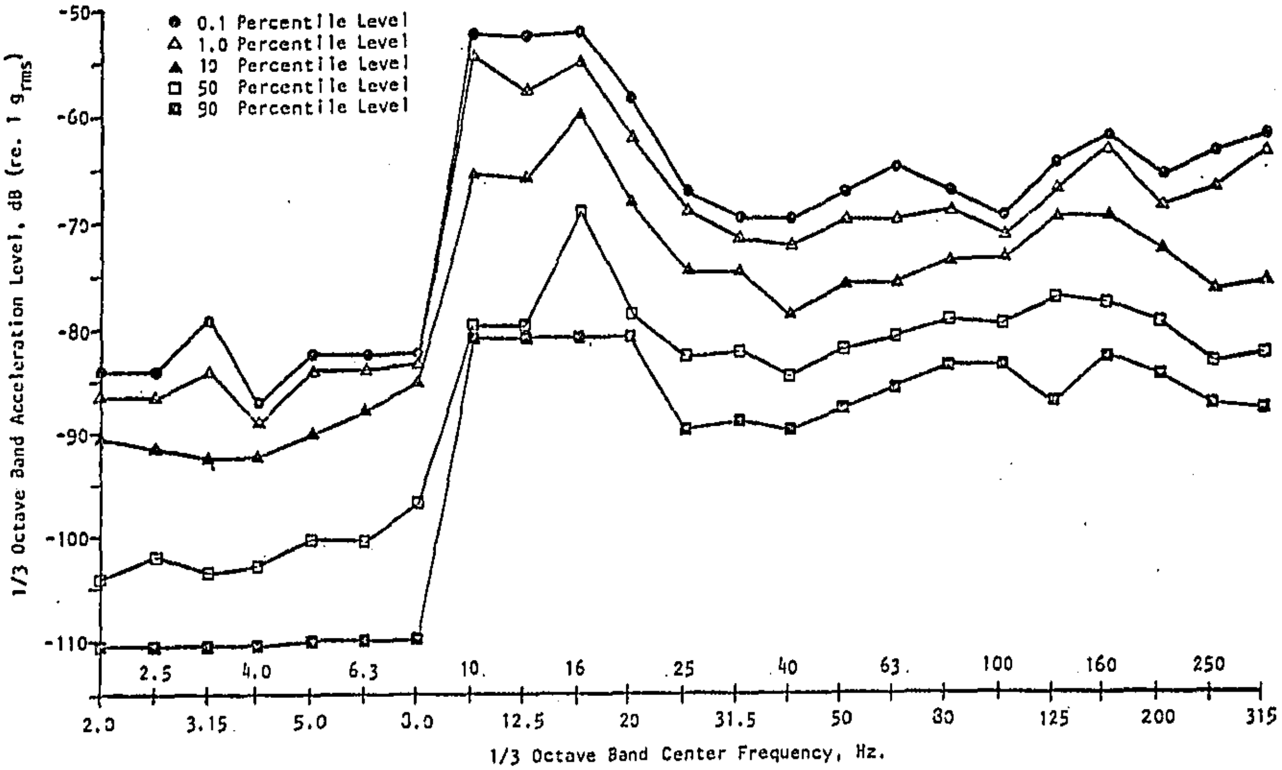


FIGURE 4-9 - THIRD OCTAVE TRAFFIC-INDUCED VIBRATION SPECTRUM:
BEDROOM FLOOR RESPONSE TO BUS PASS-BY

that to properly describe the vibration, peak levels in each 1/3 Octave Band are required. Hence, the resulting spectra are, properly speaking, "energy" spectra rather than "power" spectra (32, p. 106). To use the "peak" or 0.1 percentile level spectra of Figure 4-9 relative to the criteria of Figure 2-3, it is required to convert the band levels to spectrum levels (32). The conversion is:

$$L_{spectrum} = L_{band} - 10\log(\Delta f) \quad \text{dB} \qquad (4\text{-}4)$$

where Δf is the filter bandwidth.

For example, the conversion of the data of Figure 4-9 for the 1/3 Octave Band center frequencies at 10, 12.5, and 16 Hz is:

f_c, Hz	10	12.5	16	
L_{band}	-52.	-52.5	-52	dB (re. $1g_{rms}$)
Δf	2.30	2.90	3.70	(See Figure 4-10)
$L_{spectrum}$	-55.6	-57.1	-57.7	dB (re. $1g_{rms}$)

The spectrum level of -57 dB (re. $1g_{rms}$) corresponds to the "bus #3" event presented above.

4.6 Site Ambient Measurements

The documentation of site "ambient" vibration levels is an important aspect of the traffic-induced vibration problem. The reason for this is that maximum levels of traffic-induced vibration are generally about 20 to 40 dB above the ambient levels of ground vibration. For data measured in buildings, vibration generated by normal activities such as footsteps, closing doors, air-conditioning system vibration, etc., can easily exceed levels generated by highway traffic. This aspect of the problem does not alter the "intruding"

Centre Frequency Hz	Bandwidth Hz	Centre Frequency Hz	Bandwidth Hz
2	*0.46*		
2.5	*0.58*		
3.15	*0.73*	800	183
4	*0.92*	1000	230
5	*1.16*	1250	290
6.3	*1.45*	1600	370
8	*1.83*	2000	460
10	*2.30*	2500	580
12.5	*2.90*	3150	730
16	*3.70*	4000	920
20	*4.60*	5000	1160
25	5.8	6300	1450
31.5	7.3	8000	1830
40	9.2	10.000	2300
50	11.6	12.500	2900
63	14.5	16.000	3700
80	18.3	20.000	4600
100	23	*25.000*	*5800*
125	29	*31.500*	*7300*
160	37	*40.000*	*9200*
200	46	*50.000*	*11.600*
250	58	*63.000*	*14.500*
315	73	*80.000*	*18.300*
400	92	*100.000*	*23.000*
500	116	*125.000*	*29.000*
630	145	*160.000*	*37.000*

FIGURE 4-10 - STANDARD THIRD OCTAVE CENTRE FREQUENCIES AND BANDWIDTHS nature of traffic-induced vibration and the resulting "annoyance". People are generally sincere in their complaints. However, the documentation of ambient vibration levels is extremely important relative to assessment of building damage. Intruding levels of traffic-induced vibration may occur only a few times per day. The number of occurrences of footsteps, closing doors, or cycles of vibration from mechanical equipment that may occur per day is very large compared to the number of traffic intrusions. Building damage, if it is alleged, is the result of a long-term fatigue effect. That is, both level and "number of cycles" must be considered. Although it is probably impossible in the case of building construction to relate vibration level, from any source, to structural damage as a function of "number of cycles", it is highly probable that the more frequent high level "sources" are the cause.

Finally, during the measurement of traffic-induced vibration, the engineer or technician should always stop all pedestrian traffic and remain away from the transducers. Ground motion and building vibration resulting from footsteps can totally obscure the traffic-induced vibration data.

5.0 EXAMPLES OF TRAFFIC-INDUCED VIBRATION ANALYSES

This section illustrates the use of the procedures and methodology described in Sections 3.0 and 4.0. The purpose is to present examples typical of problems encountered in practice. Section 5.1 presents examples illustrating the prediction of the vibration reference emission level and a sample data reduction problem. Section 5.2 presents examples of estimating the propagation of vibration away from the highway source. Section 5.3 presents an example of vibration emissions from mixed traffic flows. Section 5.4 describes the "probability" of exceeding a vibration level and emphasizes the "single event" nature of traffic-induced vibration. Finally, Section 5.5 presents an example of the estimation procedures for vehicles striking a "pothole" or bump.

5.1 Estimation of Vibration Reference Emission Level

This example illustrates how to calculate the vibration reference emission level for vehicles on pavements with random surface roughness.

For a vehicle with a gross weight of 30,000 lb. (13,608 kg) travelling 35 mph (56.3 km/h) on a roadway with an estimated PSR index of 2.5, calculate the vibration (acceleration) reference emission level (referenced to 2m. from the edge of the pavement).

From Eqn(3-2a) page 32, one obtains the acceleration reference emission level using English units:

$$L_o = -4.155(2.5) + 17.2\log(35) + 10\log(30.0) - 87.7, \text{ dB}$$
$$L_o = -10.4 + 26.6 + 14.8 - 87.7, \text{ dB}$$
$$L_o = -56.7 \text{ dB (re. } 1g_{rms}).$$

From Eqn(3-2b) page 32, one obtains the <u>acceleration</u> reference emission level using metric units:

$$L_o = -4.155(2.5) + 17.2\log(56.3) + 10\log(13.608) - 87.8,\ \text{dB}$$
$$L_o = -10.4 + 30.1 + 11.3 - 87.8$$
$$L_o = -56.8\ \text{dB (re. } 1g_{rms}).$$

The difference is insignificant. This example problem is illustrated in Figure 5-1, indicating the use of the nomograph presented in Figure 3-1, page 34.

As an example of the evaluation of vehicle vibration emission levels during site calibration, the following test data is measured at 6.5 feet (2m) from the edge of the pavement (parking lane):

-40.5, -43.0, -44.5, -50.0, -56,0, -57.5, -58.0
-58.5, -58.5, -60.0, -61.0, -62.0, -62.0, -64.5

This data is the maximum vertical component ground motion rms acceleration level in dB (re. $1g_{rms}$). The approximate vehicle gross weights are estimated to be 100 thousand pounds (45.36 thousand kg). The pavement is smooth in front of the site with an estimated PSR=3.0. The average vehicle cruise speed is 33 miles per hour (53.1 km/h).

Using standard statistical techniques: The mean value of the data is -55.4 dB (re. $1g_{rms}$) and the standard deviation is 7.2 dB.

From either Equation (3-2a) or (3-2b), one estimates (using appropriate units) that the mean or expected reference emission level, $\overline{L_o}$, is -54.1 dB (re. $1g_{rms}$). In terms of an energy average of the population of test vehicles, the reference emission level is estimated using Equation (3-15) as:

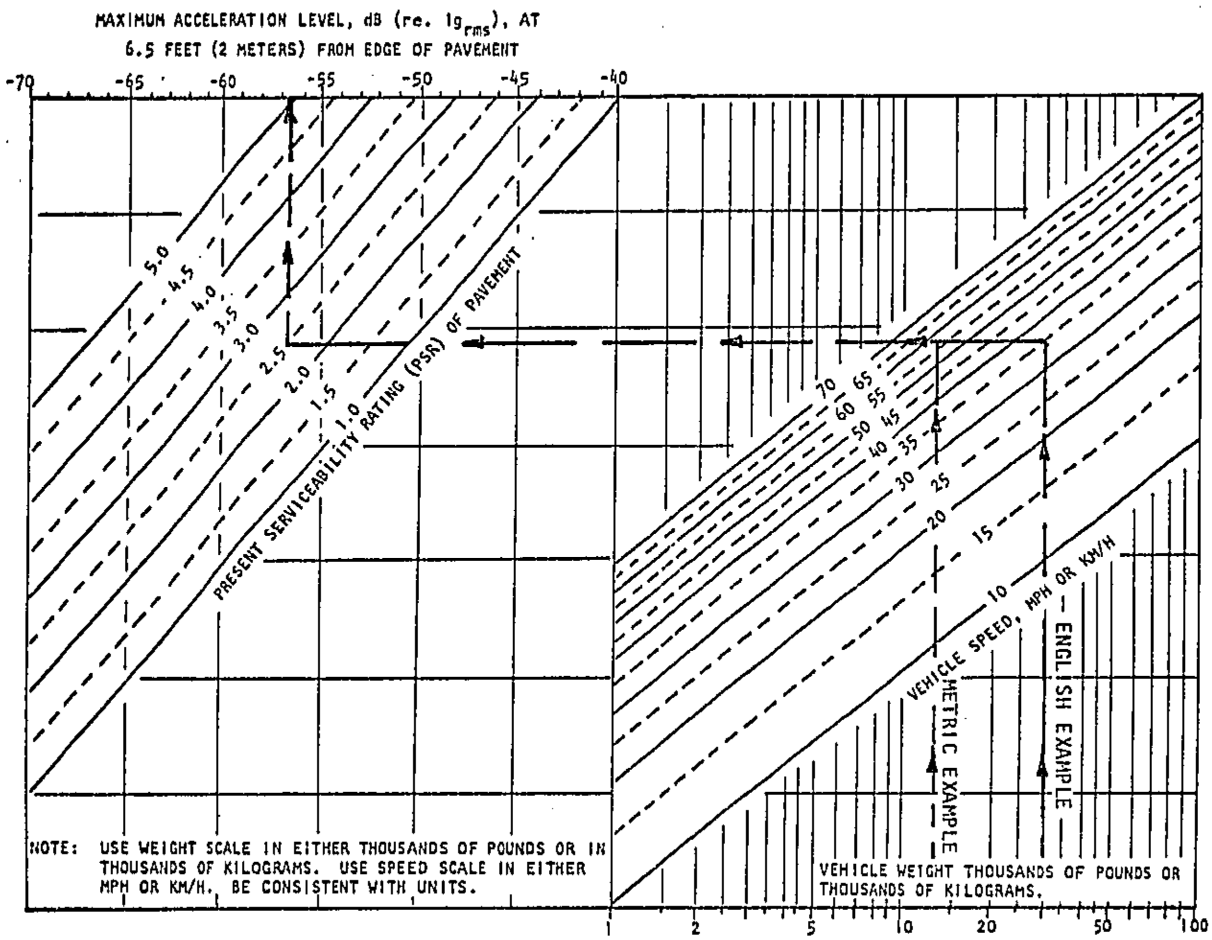


FIGURE 5-1 - EXAMPLE OF USE OF VIBRATION REFERENCE EMISSION LEVEL NOMOGRAPH
(Figure 3-1, page 34)

$$L_o = \bar{L}_o + 0.115\sigma_o^2 = 055.4 + 0.115(7.2)^2$$

$$L_o = -49.4 \text{ dB (re. } 1g_{rms}).$$

The data analysis may be continued (33), to estimate the probability density function for the data using Pearson's curve fitting technique (22, 33). The resulting Pearson's Type III curve for the probability of observing a vibration level (for the test conditions) is:

$$p(L) = 6.0716[1+(L+58.604)/12.965]^{4.0464}\ e^{-0.3121(L+58.604)}$$

$L > -71.5$ dB (re. $1g_{rms}$)

Mean = -55.4 dB   Standard Deviation = 7.2 dB

Mode = -58.6 dB   Skewness = 0.4444

For example, the probability of observing a maximum level of -49 dB is p(-49) = 2.86 percent. The most frequently observed level (the mode) is -58.6 dB.

To completely establish the validity of a "site emission" equation as described in Section 4.4, it is required to obtain data for other vehicle weight classes and operating speeds. Once these data are obtained, regression analysis is used to establish the "constants" for the site. Reference (25) presents a discussion of statistical analysis of traffic noise emission data. Identical techniques would be used for traffic vibration data.

5.2 Propagation of Traffic-Induced Vibration

This example problem illustrates the calculation of propagation of vibration away from the pavement. The basic assumption concerning the site is that the terrain between the source and the receiver is "essentially flat". The term "essentially flat" means that the local terrain does not rise and fall significantly between the highway source and the receiver. A significant variation in terrain is an elevation change on the order of 10 ft. (3m) between the edge of the pavement and the receiver. Such changes may result in attenuation losses in excess of those predicted using the basic propagation model. To exceed 1 dB, these elevation changes must be on the order of 20 ft. (6.1m) or more. Source-receiver distances of concern to the traffic-induced vibration problem are on the order of 100 ft. (30.5m). Hence, sites with slope variations of less than 2.5:1 are "essentially flat".

Basically two distance-attenuation "laws" are applicable to the traffic-induced vibration problem. First, there is "point" source attenuation (See Section 3.3.1). Point source attenuation applies to a vibration source located at a specific distance away from the receiver. Both geometric spreading and soil absorption are modelled in the results presented in Section 3.

Point source vibration attenuation (See Equation (3-3), page 37) is expressed as:

$$\Delta_{point}(D) = -10\log(D_o/D) - 20\log(e)\cdot\alpha\cdot(D-D_o) \text{ dB} \qquad (5\text{-}1)$$

where D is the source-receiver distance

D_o is a reference distance.

The parameter, α, is the soil absorption constant and is given in Table 3-2, page 40, for typical soils.

For traffic flow vibration analyses, the equivalent (energy-mean) vibration level is used. The distance propagation depends upon the roadway length just as is the case for traffic-noise analyses (10). As discussed in Section 3.3.1, page 37, all roadways are essentially "infinite" for the traffic-induced vibration problem contrary to the situation for traffic noise.

The equivalent (energy-mean) vibration level attenuates according to the "line source" rate (See Equation (3-4), page 41) as:

$$\Delta_{line}(D) = -5\log(\alpha D) - 20\log(e)\alpha(D-D_o) \tag{5-2}$$

where D is the source-receiver distance
D_o is the reference distance
α is given in Table 3-2, page 40.

Finally, for the analysis of vibration propagation away from a bump or pothole, a modified form of point source attenuation is presented in Section 3.4.3 (See Equation (3-24), page 60). The only difference between point source attenuation and attenuation of vibration from a bump in the pavement is that the reference distance is placed at the bump and that the absorption constant of Table 3-2 must be decreased to approximately one-third the value indicated in Table 3-2.

The propagation of the maximum pavement acceleration level away from the bump or pothole attenuates as:

$$\Delta_{bump}(D) = -10\log(D) - 20\log(e)\alpha D/3 \tag{5-3}$$

where D is the distance between the bump and the receiver
α is given by Table 3-2, page 40.

For example, the attenuation at a distance 100 feet (30.5 m) from a point source with an assumed absorption constant of 0.10 per foot (0.328 per meter) and a reference distance of 6.5 feet (2m) is:

$$\begin{aligned}\Delta_{point}(100) &= -10\log(6.5/100) - 20\log(e)\cdot(0.1)\cdot(100-6.5)\\ &= -11.87 - 8.686(0.1)\cdot(93.5)\\ &= -11.87 - 81.21\\ &= -93.1 \text{ dB}\end{aligned}$$

Figure 5-2 presents a plot of the point source attenuation with distance for various values of α typical of soils. The values of α are selected from Table 3-20 on page 40.

For the above example problem the line source distance attenuation is:

$$
\begin{aligned}
\Delta_{line}(100) &= 5\log(0.1(100)) - 20\log(e)\cdot(0.1)\cdot(100-6.5) \\
&= -5.00 - 8.686\cdot(0.1)\cdot(93.5) \\
&= -5.00 - 81.21 \\
&= -86.2 \text{ dB}
\end{aligned}
$$

Figure 5-3 presents a plot of the line source attenuation with distance for various values of α typical of soils.

For the above example problem, the "bump" source distance attenaution at 100 feet (30.5m) from the source is:

$$
\begin{aligned}
\Delta_{bump}(100) &= -10\log(100) - 20\log(e)\cdot(0.1)\cdot(100)/3 \\
&= -20.00 - 8.686\cdot(10/3) \\
&= -20.00 - 28.95 = -48.95 \\
&= -49.0 \text{ dB}
\end{aligned}
$$

The above results are rounded to the nearest tenth of a decibel. Generally, one may round to the nearest decibel unit after all calculations have been completed.

5.3 Evaluation of Vibration Emissions from Mixed Traffic Flows

This example presents an illustration of the prediction of traffic-induced vibration for mixed traffic flows. For highway noise prediction, generally the heavier vehicles produce the dominant noise exposure at the site although the heavy vehicles comprise a small

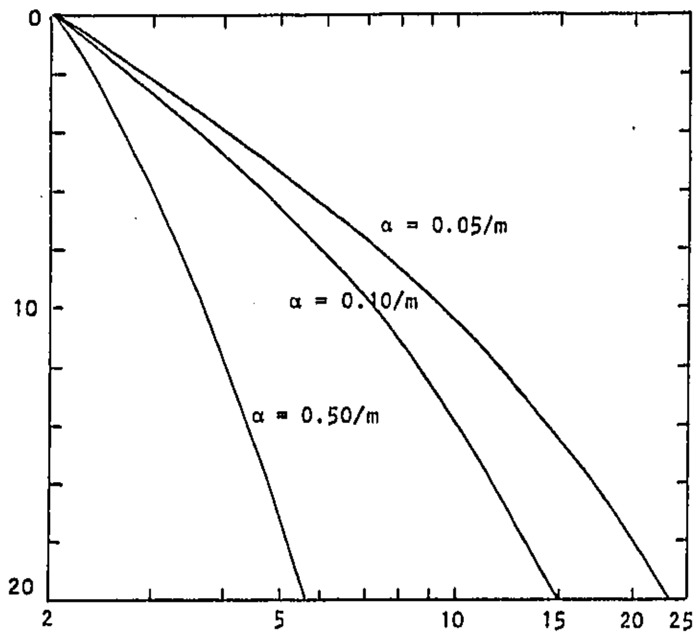


FIGURE 5-2 - POINT SOURCE DISTANCE ATTENUATION

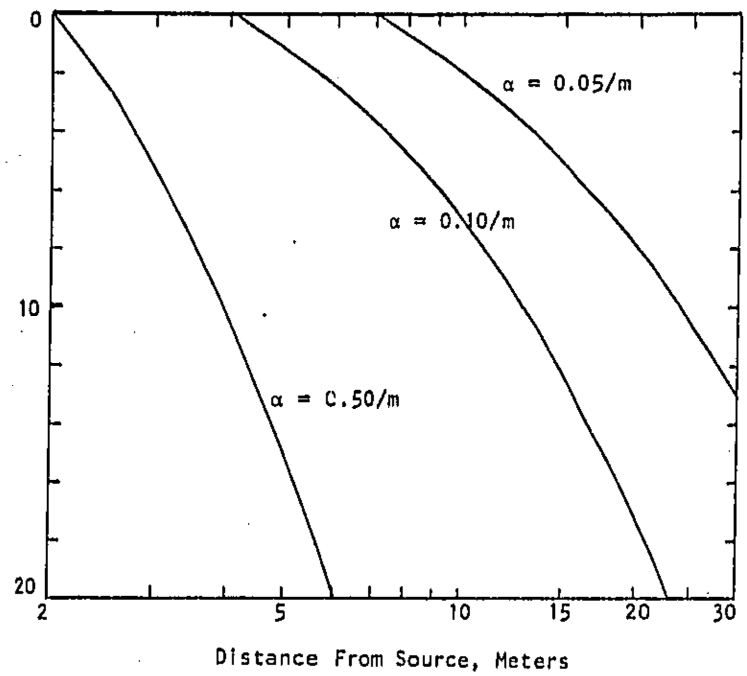


FIGURE 5-3 - LINE SOURCE DISTANCE ATTENUATION percentage of the traffic flow. For traffic-induced vibration, the analogous situation occurs to an even greater extent than for traffic noise. A few intrusions per day may be sufficient to result in complaints (See Section 4.5) for the traffic-induced vibration problem. For traffic-induced vibration, the changes in "vibration ambient" for heavy traffic flows is at least 40 dB below the peak levels induced by heavy vehicles. Hence, ground vibration induced by highway traffic does not result in an annoying level of vibration for urban areas where as traffic noise may be the dominant factor in the ambient urban sound level.

For mixed traffic-flow, the estimation of adverse impact due to highway traffic-induced vibrations is rather simple. One may make simple estimates based upon an approximation or one may make a complete analysis considering the total traffic flow. This example problem considers the latter approach to illustrate the use of the theory in Section 3. The simple approach is described in Section 5.4. The objective of the analysis is to estimate the probability of an intruding vibration level generated by mixed traffic flow.

Figure 5-4 presents a site comprising four traffic lanes and a narrow median. The roadway is structurally divided by the median so that the pavement/subgrade system for each direction of travel is considered as a source location. Table 5-1 presents a detailed estimate of the traffic count by <u>vehicle gross weight categories</u>. It is recognized that such detail may not be generally available, but the structural design requirements for the highway will provide an estimate of heavy vehicle traffic. Heavy vehicles are an important parameter. In Figure 5-4, the PSR values for each lane are indicated and the average cruise speed is assumed to be 35 mph (56.3 km/h).

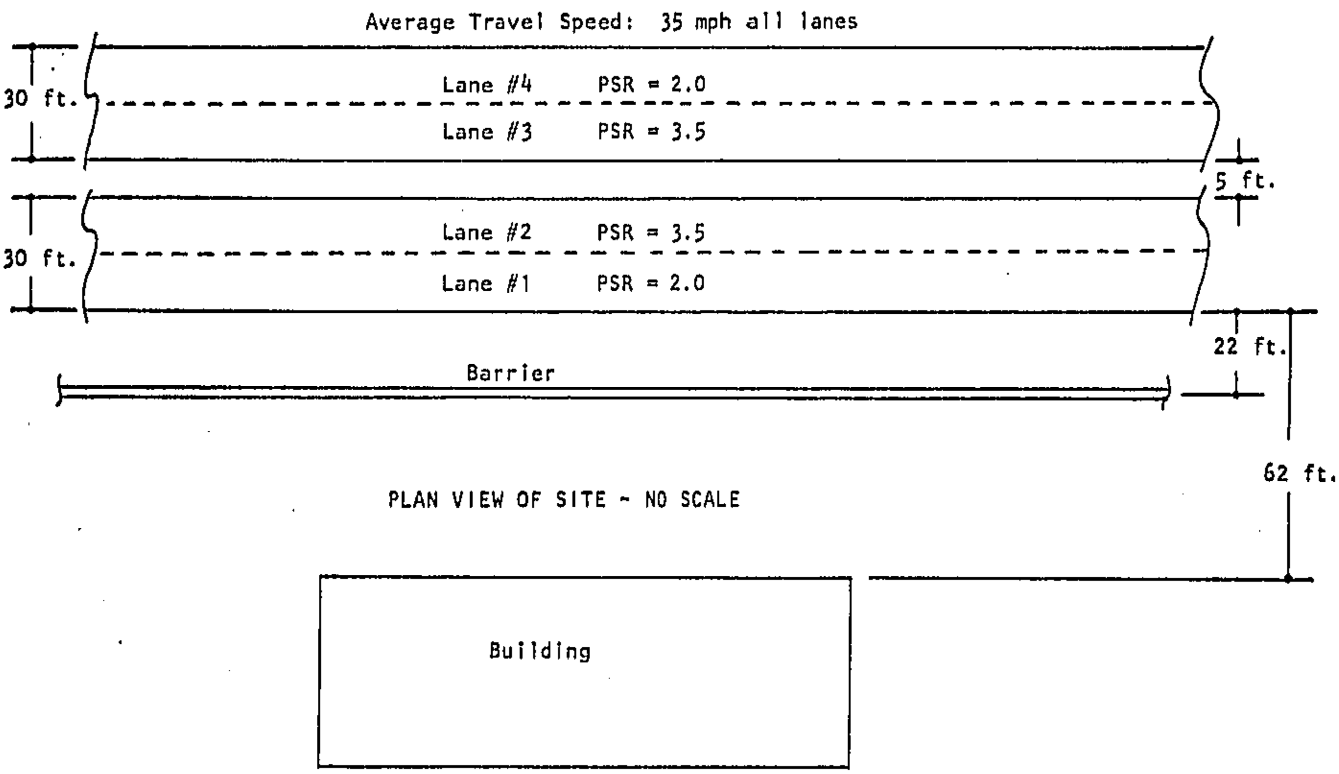


FIGURE 5-4 - SITE CONFIGURATION FOR EXAMPLE PROBLEM

TABLE 5-1

TRAFFIC COUNT FOR EXAMPLE PROBLEM

Vehicle Type	Gross Vehicle Weight, W_G lbf/1000	Lanes #1 & 2		Lanes #3 & 4	
		N_i	p_i	N_i	p_i
5 Axle Semi, Loaded	62.0	45	0.036	32	0.024
5 Axle Semi, Unloaded	30.7	18	0.014	13	0.010
3 Axle Single, Loaded	38.2	11	0.009	8	0.006
3 Axle Single, Unloaded	20.1	10	0.008	7	0.005
Buses	28.5	5	0.004	5	0.004
Automobiles	3.5	1162	0.929	1264	0.951
Totals		1251	1.000	1329	1.000

N_i denotes the hourly traffic count by gross weight category.

p_i denotes the fraction of lane traffic by gross weight category.

Vehicle Mix by Weight Estimated From Reference (8) Data.

Vehicle Weight for Loaded and Unloaded Conditions Estimated from Reference (7) Data.

The first step is to estimate the reference vibration emission levels for each vehicle type (gross weight category) for each value of pavement roughness. To do this, one uses Equation (3-2) to estimate the mean or expected levels, $\bar{L}_o$, and corrects these for variation due to the vehicle type using Equation (3-15). For the example problem, the results are presented in Table 5-2.

The reference levels given in Table 5-2 are for a location at a reference distance of 6.5 feet (2m) from the near edge of each pavement/subgrade system to the receiver. For the example problem, the receivers are the foundation of the barrier and the occupants of the building.

Since the problem comprises two pavement/subgrade systems, each will be considered as a single source. That is, traffic on lanes #1 and #2 represent a source and lanes #3 and #4 represent a source. Using the data from Tables 5-1 and 5-2, the rms acceleration level at the reference location for lane #1 is:

$$\begin{aligned}\sum_i p_i 10^{(\tilde{L}_o)_{Ei}/10} &= 0.036 \cdot 10^{-5.668} + 0.014 \cdot 10^{-5.974} \\ &+ 0.009 \cdot 10^{-6.061} + 0.008 \cdot 10^{-63.40} \\ &+ 0.004 \cdot 10^{-6.188} + 0.929 \cdot 10^{-7.329} \\ &= 1.4962 \cdot 10^{-7}\end{aligned}$$

Hence, the reference levèl for the traffic flow on lane #1 is $10\log(1.4962 \cdot 10^{-7}) = -68.25$ dB.

Continuing the calculations for each lane, the reference levels are:

Lane #1 = -68.25 dB
Lane #2 = -74.48 dB
Lane #3 = -75.55 dB
Lane #4 = -69.32 dB

TABLE 5-2

REFERENCE VIBRATION EMISSION LEVELS FOR EXAMPLE PROBLEM

Vehicle Type	W_G*	σ_o dB**	Lanes #1 & #4 PSR = 2.0 $(\overline{L}_o)_E$	Lanes #1 & #4 PSR = 2.0 $(\tilde{L}_o)_E$	Lanes #2 & #3 PSR = 3.5 $(\overline{L}_o)_E$	Lanes #2 & #3 PSR = 3.5 $(\tilde{L}_o)_E$
5 Axle Semi, Loaded	62.0	7.2	-56.68	-50.72	-62.92	-56.96
5 Axle Semi, Unloaded	30.7	7.2	-59.74	-53.78	-65.97	-60.01
3 Axle Single Unit, Loaded	38.2	6.0	-60.61	-56.47	-66.84	-62.70
3 Axle Single Unit, Unloaded	20.1	6.0	-63.40	-59.26	-69.63	-65.49
Buses	28.5	6.0	-61.88	-57.74	-68.11	-63.97
Automobile	3.5	4.0	-73.29	-71.45	-79.52	-77.68

* Gross Weight, Thousands of Pounds

** Estimated. These values are representative of field test data.

$(\overline{L}_o)_E = \overline{L}_o + 0.115\sigma_o^2$ $(\tilde{L}_o)_E = \overline{L}_o + 0.230\sigma_o^2$ $\overline{L}_o$ is From Equation (3-2), page 32

It should be noted that the differences in reference levels for each lane are a result of the differences in traffic flow mix (lanes #1 and #2 compared to #3 and #4) and to the differences in surface roughness (lanes #1 and #4 compared to lanes #2 and #3). For example, the difference in level between lanes #1 and #2 is 6.23 dB with the rougher lane (lane #1) being the higher level source. This level difference is simply the difference in surface roughness: $-4.155(2.0-3.5) = 6.23$ dB (See Equation (3-2a)).

For mixed traffic flow, the combination of vibration level from each lane is accomplished using Equation (3-8a). Since lanes #1 and #2 and lanes #3 and #4 are to be considered as two separate sources, Equation (3-8a) is applied to each source to estimate the resulting levels at each receiver.

Substituting the data for the problem into Equation (3-8a) one obtains the results:

- Lane #1 and #2

$$L_e = 10\log'\{\frac{1251}{2502}\,10^{-6.827} + \frac{1251}{2502}\,10^{-7.448}\}$$
$$+ 10\log\{\frac{(2502)(6.5)}{(35)(1)}\} - 5\log(\alpha D) - 8.686\alpha(D-6.5)$$
$$L_e = -59.76 - 5\log(\alpha D) - 8.686\alpha(D-6.5)$$

- Lane #3 and #4

$$L_e = -60.58 - 5\log(\alpha D) - 8.686\alpha(D-6.5)$$

As indicated in Figure 5-4, the soil classification is typically "silty clay". From Table 3-2, the absorption coefficient, α, is estimated to vary from 0.05 to 0.13 per foot. To obtain conservative results (i.e., higher vibration level predictions) it is assumed that α=0.05/ft.

Substituting α=0.05 into the previous result, one obtains:

- Lanes #1 and #2: $L_e = -50.43-5\log(D)-0.4334D$
- Lanes #3 and #4: $L_e = -51.25-5\log(D)-0.4334D$

The distances, D, to be used in these results are the distances from the near edge of each set of lanes. The distances are indicated in Figure 5-4. Substituting for D into the above results, one obtains:

	Barrier		Building	
	D	L_e	D	L_e
Lanes #1 and #2	22 ft.	-66.68	62 ft.	-86.26
Lanes #3 and #4	57 ft.	-84.73	97 ft.	-103.22

The total receiver equivalent acceleration levels in dB (re. $1g_{rms}$) are:

Barrier: $(L_e)_{total} = 10\log(10^{-6.668} + 10^{-8.473}) = -66.61$

Building: $(L_e)_{total} = 10\log(10^{-8.626} + 10^{-10.322}) = -86.17$

The example is still incomplete; the criteria of Section 2 requires an estimation of the maximum vibration levels resulting from the traffic flow. Hence, it is required to calculate the extreme percentile levels. Using the data in Tables 5-1 and 5-2, the percentile levels are estimated as follows:

First, the cumulants, κ_2, are estimated using Equation (3-14). For the data of the example problem, one obtains:

- Lanes #1 and #2: $\kappa_2 = 0.3207/\sqrt{D}$
- Lanes #3 and #4: $\kappa_2 = 0.3383/\sqrt{D}$

The combination of these results is obtained by simply summing the cumulants after substituting for the source receiver distance. The results are:

	Barrier		Building	
	D	κ_2	D	κ_2
Lanes #1 and #2	22 ft.	0.06972	62 ft.	0.04153
Lanes #3 and #4	57 ft.	0.1145	97 ft.	0.07588
	$\Sigma\kappa_2$	0.1145	$\Sigma\kappa_2$	0.07588

To estimate the percentile levels, one uses Equation (3-12) to obtain:

Barrier: $\sigma_L = 4.343\sqrt{\ln(1.1145)} = 1.430$
Building: $\sigma_L = 4.343\sqrt{\ln(1.07588)} = 1.175$

From Equation (3-13), the percentile acceleration levels are estimated in dB (re. $1g_{rms}$) as:

	Barrier	Building
L_e	-66.61	-86.
$L_{50} = L_e - 0.115\sigma_L^2$	-66.85	-86.33
$L_{10} = L_{50} + 1.28\sigma_L$	-65.01	-84.82
$L_{05} = L_{50} + 1.648\sigma_L$	-64.49	-84.39
$L_{01} = L_{50} + 2.33\sigma_L$	-63.51	-83.59
$L_{0.1} = L_{50} + 3.09\sigma_L$	-62.19	-82.70

To compute the problem, a brief discussion is required. First, assuming that the building amplification of the ground vibration was an extreme (See Figure 3-8, page 63), the estimate of $L_{0.1}$= -82.70+15 -67.7 dB is below the criteria of Figures 2-3 and 2-4. As illustrated in other examples in these guidelines, a 15 dB building amplification is apparently a rare occurrence. Next, it is apparent that the far lanes (#3 and #4) contribute little to the receiver vibration level.

To complete the discussion, the vibration prediction at the barrier footing was not included as a meaningless exercise. Although the estimated level of $L_{0.1}$= -62.43 dB at the barrier is approximately 30 dB below the "minor damage possible" curve, the prediction may be useful in the design of traffic noise barriers. For example, long-term low level traffic-induced vibration may result in irregular settling of foundations (4) or cracks in masonry joints and grout.

5.4 Probability of Exceeding a Peak Vibration Level

The example problem of Section 5.3 estimated the percentile acceleration levels at receiver locations. These level estimates are based upon an assumed Gaussian or Normal distribution of amplitude peaks from the individual vehicle types comprising the traffic flow (12). The procedure is known to yield high level estimates for traffic flows comprising a few occurrences of high levels and a mixture of occurrences of low levels (12). Whether or not it is totally accurate to estimate peak levels of traffic-induced vibration (the $L_{0.1}$ metric) using the theory of Section 3.3.5 of these guidelines will only be answered when tested in field use. It is believed, however, that the theory is adequate although cumbersome for hand calculations.

A simpler, and perhaps more accurate, method than that presented in Section 5.3 is to consider only the heaviest vehicles comprising the traffic flow and the roughest lane that is closest to the receiver. For the example problem of Section 5.3, this would correspond to the "loaded 5 Axle Semi" on Lane #1 (PSR = 2.0). Considering only the peak instantaneous level during a single vehicle pass-by, one obtains from Equation (3-9):

$$L_{peak} = L_{ol} + 10\log(6.5/D) - 8.686(0.05)(D-6.5) \text{ dB}$$

or

$$L_{peak} = L_{ol} - 10\log(D) - 0.4343D + 10.95 \text{ dB.}$$

The "peak" level is, of course, the maximum level at the closest pass-by location between the source and the receiver. The distribution of the peak level would be expected to follow the distribution of L_{ol}. Such a distribution is presented in Section 5.1 as the result of a "site calibration test". Hence, one should use actual field test data, if available, to establish the estimates of peak vibration levels. The assumption of a Gaussian distribution may be too restrictive.

For example, the mean or expected emission level for the "loaded 5 Axle Semi" considered above is $\overline{L}_o$ = -62.65 dB. Considering the estimated standard deviation of 7.2 dB, one would obtain, assuming a Gaussian level distribution for all "loaded 5 Axle Semi" vehicles, an extreme peak level estimate of:

$$L_{ol} = -62.65+3(7.2) = -41.05 \text{ dB.}$$

For the building location of Section 5-3, the source-receiver distance is D=62 feet and from the above results:

$$L_{peak} = -41.05-10\log(62)-0.4343(62)+10.95, \text{ dB}$$

$$L_{peak} = -41.05-17.92-26.93+10.95 = -74.95, \text{ dB}$$

This level is +7.12 dB above the -82.70 dB acceleration level estimated in Section 5.3. The estimated extreme peak level is, for the data of the example, below the perception threshold at the building footing and allows one to consider a building amplification of -64-(-74.95)=10.95 dB. (The -64 dB level is the perception threshold criteria curve of Figure 2-3 at 10 Hz.) From Figure 3-8, a nominal 10 to 11 dB building amplification would result in probabilities of <u>not</u> exceeding the perception threshold on the order of 0.70 to 0.90.

Whether or not the above tolerances are adequate may only be established based upon experience and engineering judgement. The purpose of the example is complete, however. That is, a simple estimate such as presented above may be sufficient to indicate the probability that traffic-induced vibration is not a problem.

5.5 Potholes

This example problem illustrates the use of the methodology presented in Section 3.4 for estimating impact loading and response of the pavement to the impact loading. The example, hopefully, illustrates the level of detail required for this type of analysis. (As an abatement procedure for traffic-induced vibration, it is good practice to fill any potholes or smooth abrupt surface discontinuities in the pavement.) Vehicle parameters will be assumed and pavement response will be estimated.

For the example problem, the following vehicle parameters are assumed:

- Suspension System Natural Frequency, f_n = 12 Hz (See Equation (3-17))
- Tire Stiffness, k_t = 4700 lbf/in (839.3 kgf/cm) (See Equation (3-18) and Figure 3-4)
- Static Tire Loading = 5200 lbf (2359 kgf)
- Vehicle Speed, V = 30 mph = 44 ft/s (13.41 m/s)

The vehicle parameters f_n and k_t would usually be measured for the specific vehicle. Reference 14 describes such tests.

The pothole geometry is taken as:

- Height <u>or</u> Depth, $\overline{h}$ = 1.0 inch (2.54 cm)
- Length (in direction of travel), ℓ = 3 feet (0.91 m)

With this data, the peak pavement loading is estimated using Equation (3-22) as follows:

$$\text{Characteristic Speed} = \overline{V} = 2(3)(12) = 72 \text{ ft/s} \quad (21.95 \text{ m/s})$$
$$\text{Speed Ratio, } v = V/\overline{V} = 44/72 = 13.41/(21.95) = 0.611$$

Selecting the appropriate form of Equation (3-22) based upon $v = 0.611$, one estimates the peak pavement loading as:

$$P_o = (4700)(1.0)\text{SIN}(2\pi(0.611)/1.611)/(1-0.611) \text{ lbf}$$
$$P_o = 4700 \cdot \text{SIN}(2.383)/(0.3890) \text{ lbf}$$
$$P_o = 4700(0.6879)/(0.3890) \text{ lbf}$$
$$P_o = 8311.3 \text{ lbf} \quad (3770 \text{ kgf})$$

The "impact factor" is the ratio of the peak load to the static load and is (for reference) estimated as:

Impact Factor = 8311.3/5200 = 3770/2359 = 1.598

or Impact Factor = 1.6

To estimate the pavement response to the impulse loading it is required to estimate the "speed ratio" for the pavement/subgrade system. The "speed ratio" is a measure of how well "tuned" the impulse loading is relative to the pavement/subgrade system. The peak pavement acceleration level, at the pothole, is estimated using Equation (3-23). For this estimate, it is required to determine the fundamental pavement/subgrade natural frequency, f_p. This would also normally be done by field testing at the site (See Reference 17). However, the theory of Section 3.4.1 will be used as an illustration.

For the pavement/subgrade system the following data are assumed:

• Pavement Slab Data (Rigid Concrete Slab)	Width, $b=24$ ft. (7.32 m) Thickness, $h_p=4.0$ inches (10.16 cm) Weight Density, $\gamma_p=150 lbf/ft^3$ (2403 kgf/m^3) Youngs Modulus, $E_p=3\cdot 10^6 psi$ ($2.11\cdot 10^5 kgf/cm^2$) Poisson's Ratio, $\nu_p=0.2$
• Subgrade Data* (AASHO A-1)	Depth, $H=10$ feet (3.05 m) Weight Density, $\gamma_f=120 lbf/ft^3$ (1922 kgf/m^3) Young's Modulus, $E_f=3\cdot 10^4 psi$ ($2.11\cdot 10^3 kgf/cm^2$) Poisson's Ratio, $\nu_f=0.31$

From Equation (3-20), the modulus of subgrade reaction, k_f, is estimated (See Figure 3-5) as:

$$k_f = (3\cdot 10^4)/(120(1-0.31^2)) = 276.6 lbf/in^3 \ (7.66 kgf/cm^3)$$

The slab bending rigidity, D_p, is estimated as (See page 55):

$$D_p = (3\cdot 10^6)(4)^3/(12(1-.2^2)) = 1.667\cdot 10^7 lbf\text{-}in. \ (1.920\cdot 10^7 kgf\text{-}cm)$$

and the radius of relative stiffness, l, is estimated as: (See Figure 3-6)

$$l = (D_p/k_f)^{\frac{1}{4}} = (1.667\cdot 10^7/276.6)^{\frac{1}{4}} = 15.67 \text{ inch } (0.398 \text{ m}).$$

The subgrade mass and stiffness parameters are calculated from Equation (3-20) as:

$$m_p = 150(4/12)/(32.2) = 1.553 \ lbf\text{-}s^2/ft^3$$

$$m_f = 120(10)/(3(32.2)) = 12.422 \ lbf\text{-}s^2/ft^3 = 0.00719 \ lbf\text{-}s^2/in^3$$

$$\mu = m_p/m_f = 0.125$$

$$\varepsilon = \sqrt{2(1-.34)/3}(10/24) = 0.276$$

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* See Table 3-3.

and the estimated fundamental frequency of the pavement/subgrade system is from Equation (3-19):

$$f_p = [((276.6)/0.00719))((1.276)/(1.125))]^{\frac{1}{2}}/2\pi \text{ Hz.}$$
$$f_p = 33.2 \text{ Hz.}$$

From Equation (3-21) the effective weight of the pavement/subgrade system is:

$$\overline{W} = 5.5(120)(24)(15.67/12)(10)(1.125)/3 \text{ lbf.}$$
$$\overline{W} = 77{,}566.5 \text{ lbf (35184 kgf).}$$

From Equation (3-23), the characteristic "speed" of the pavement/subgrade system is estimated, based upon the pothole length of 3 feet (0.91 m), as:

$$\tilde{V} = 2(3)(33.2) = 199.20 \text{ ft/s} \quad (60.72 \text{ m/s})$$

and the speed ratio, $v = V/\tilde{V}$, of the vehicle speed of 44 ft/s (13.41 m/s) to the characteristic speed, $\tilde{V}$, is:

$$v = (44)/(199.2) = (13.41)/(60.72) = 0.221.$$

Returning to the problem of the vehicle impacting the pothole, the parameters required to estimate the pavement response using Equation (3-20) are:

- Peak Impact Load, P_o = 8311.31bf (3770kgf)
- Effective Pavement/Subgrade Mass, $\overline{W}$ = 77,566.51bf (35,184 kgf)
- "Speed Ratio", $v = V/\tilde{V} = 0.221$.

From Equation (3-20), the peak impulse acceleration level at the pothole is:

$$L_{po} = 20\log(8311.3) - 20\log(77{,}566.5) + 20\log(|\mathrm{SIN}(\pi/2(.221))|) + 6.0 \quad \text{dB (re. } 1g_{peak})$$

$$L_{po} = 78.39 - 97.79 - 2.68 + 6.0 = -16.08 \text{ dB (re. } 1g_{peak})$$

From Equation (3-24) the approximate receiver peak impulse acceleration level is:

$$L_{pr} = -16.08 - 10\log(D) - 20\log(e)\alpha D \quad \text{dB (re. } 1g_{peak})$$

For the example problem of Section 5.3, assume that the pothole is in lane #1 approximately 80 feet from the building. For the assumed value of $\alpha = 0.05$/ft., the peak receiver acceleration level is (dividing α by 3 to consider the low frequency characteristics of the impulse):

$$L_{pr} = -16.08 - 10\log(80) - 8.686(0.05/3)(80)$$

$$L_{pr} = -46.69 \text{ dB (re. } 1g_{peak}).$$

It is difficult to estimate what the building response may be. However, the peak level is estimated to be some 28.3 dB above the Section 5.4 estimate and 36.0 dB above the Section 5.3 estimate for the building foundation. One might expect the pothole vibration to be perceptible and possibly damaging.

6.0 SUMMARY AND RECOMMENDATIONS

These engineering guidelines present a methodology for assessment of highway traffic as a source of environmental vibration. The guidelines describe the characterization of traffic-induced vibration, an estimation procedure, measurement and analysis procedures, and examples illustrating the use of the guidelines.

Relative to traffic noise, the problem of quantifying traffic-induced vibration is in its infancy. The prediction methodology in Section 3 appears to be adequate to address the problem. However, the present data base is insufficient to properly evaluate the theory for all situations that can arise. Only field measured data can be used to evaluate the procedures completely.

The criteria described in Section 2 and the measurement requirements of Section 4 illustrate the requirement for a "single number" frequency-weighted metric for environmental vibration. Such a metric would be analogous to the A-weighted sound level used in traffic noise analyses. Research in Japan (20) and recommendations of technical committees (2) are resulting in the development of a standardized metric. When standardization does occur, instrumentation will be available that will greatly simplify the analysis and interpretation of test data. Such standardization will not alter the methodology and conclusions of this report. The only change will be the use of "weighted" acceleration levels rather than spectrum levels. Due to the discrete frequency characteristics of traffic-induced vibration, the proposed frequency weighting would result in a shift in the vehicle emission levels presented in Section 3.2 This constant shift will be in the range of -3dB to -6dB due to the relative weighting in the frequency range of 10 HZ to 15 Hz. Standardized frequency weighting would result in constant values for both annoyance criteria and building damage criteria. Without standardized frequency weighting, criteria can only be presented as curves in the form of Figure 2-3.

The acceleration levels quoted in these guidelines are expressed in dB relative to 1 "g" or 9.807 m/s^2. This reference value was selected so that traffic-induced vibration levels would be negative numbers and, hence, distinctly different from traffic noise levels expressed in dB. For example, Reference (20) expresses frequency-weighted acceleration levels in dB (re. $10^{-5} m/s^2$). Using this convention, all acceleration levels expressed in these guidelines would be increased 120 dB. That is, a level of -50 dB (re. 1g) would be 70 dB (re. $10^{-5} m/s^2$). Standardization of a frequency-weighted vibration level may result in such a constant shift in "dB".

It is hoped that these guidelines assist the reader in evaluating engineering problems that might arise related to traffic-induced vibration.

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