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State-of-the-Art Review: Prediction and Control of Groundborne Noise and Vibration from Rail Transit Trains

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16. Abstract

This report provides a comprehensive review of the state-of-the-art in the prediction and control of groundborne noise and vibration. Various types of impact criteria are reviewed for groundborne noise and vibration, building damage, and soil settlement. Vibration measurement and evaluation techniques are reviewed. Techniques which have been used by rail transit systems to control groundborne noise and vibration are discussed. These techniques include wheel and rail maintenance, track support system design, floating slabs, resilient wheels, tunnel wall thickness, trenches, and building isolation. Several procedures that have been used to predict groundborne noise and vibration are outlined. Finally, mathematical models are discussed which have been or could be used to analyze rail fasteners, resiliently supported ties, floating slabs, tracks, subway/soil interaction and radiation from the subway structure, vibration propagation, and attenuation in the soil, and building response to groundborne vibration.

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PREFACE

This document presents a comprehensive review of the state-of-the-art in the prediction and control of groundborne noise and vibration generated by rail rapid transit systems. The report was prepared by Wilson, Ihrig, & Associates, Inc. (WIA) under contract to the U.S. Department of Transportation. The report is part of the information dissemination activities of the Urban Rail Noise Abatement Program managed by the Transportation Systems Center, Cambridge, MA, under the sponsorship of the Office of Systems Engineering of the Urban Mass Transportation Administration, Office of Technical Assistance.

The report has been prepared with the assistance and cooperation of a number of people. In addition to the authors listed on the title page, invaluable contributions were made by other WIA staff including Harjodh S. Gill, Armin T. Wright and Thomas A. Mugglestone. Also participating in the preparation of the report were J. Richards and S.W. Nowicki of London Transport International and Michael Carroll, Chi-Yeun Wang and Anil Chopra of the University of California. Elizabeth Ivey of Smith College also provided valuable reviews and comments. Many useful suggestions have also been provided by the Noise and Vibration Liaison Board of the American Public Transit Association.

The technical effort on this report was coordinated by Michael Dinning of TSC. The authors express their gratitude for his assistance and encouragement during their preparation of the report.

METRIC CONVERSION FACTORS

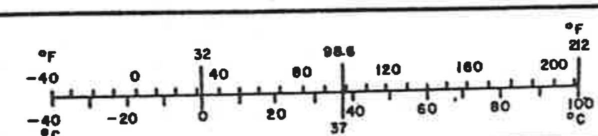
Approximate Conversions to Metric Measures

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. INTRODUCTION

Groundborne noise and vibration generated by rail transit trains is a problem of growing importance to the rail transit industry. Airborne noise has long been recognized as an important environmental problem, however, it is only in the past decade that groundborne noise and vibration has been widely recognized as a major environmental problem. The importance of groundborne vibration is reflected in the large quantity of ongoing research in the field. In the literature review performed for this study over 300 significant references were collected.

The first step in the review of the state-of-the-art in groundborne noise and vibration prediction and control was to read and evaluate the references. An annotated bibliography of the references was prepared and will be published as a separate document. This report uses the same reference numbers as the annotated bibliography. For the user's convenience the references of the annotated bibliography are listed at the end of this report.

This report provides a comprehensive review of the state-of-the-art in the prediction and control of groundborne noise and vibration. Chapter 2 presents an overview of transit train induced groundborne noise and vibration, and to some degree summarizes the material contained in the remaining chapters. The report focuses on summarizing earlier work instead of presenting new work. The only real exception to this is the criteria for groundborne vibration that are presented in Chapter 3. Suitable criteria for groundborne vibration from transit trains have not been available until recently, largely because of the lack of information regarding community response to groundborne vibration. As a result of the new transit lines in Toronto, Washington, and

Atlanta we have been able to obtain a significant amount of information regarding the reaction of building occupants to groundborne vibration. This information in conjunction with recently developed standards for human exposure to building vibration provides a good basis for transit induced groundborne vibration. This information is presented in Chapter 3.

Chapter 4 presents information techniques that have been used to measure groundborne noise and vibration. Covered are the techniques for measuring the passby noise and vibration using microphone and accelerometers along with narrow-band and correlation techniques that can be used for detailed study of the mechanisms of groundborne vibration radiation and propagation.

The techniques that have been successfully implemented for the control of groundborne noise and vibration are discussed in Chapter 5. The topics include wheel and rail maintenance, track design, resilient wheels, rail support systems, floating slabs, ballast mats, tunnel wall thickness, screening (trenches), and vibration isolation of buildings. All of these techniques have been used by rail transit systems at least in test installations. The techniques have all been shown to provide some reduction of groundborne noise and vibration, however, the effective frequency range varies widely. The conclusion is that the vibration control method that is implemented must be matched to the spectrum of the groundborne vibration to ensure that the reduction will be over the appropriate frequency range.

Accurate projections of groundborne noise and vibration are very important when constructing new transit lines. Inaccurate projections can lead to either expensive control measures that are unnecessary or community complaints and even legal action because of the annoyance caused by groundborne noise and vibration.

Chapter 6 outlines several of the procedures that have been used to predict groundborne noise and vibration. Most of the existing procedures are primarily based on empirical data. An effort has been made to outline each methodology, summarize the assumptions that have been made, and point out the shortcomings of the methods. One of the primary shortcomings of all of the prediction methods is the lack of analytical models for the mechanisms of groundborne noise and vibration generation, radiation, and propagation.

Analytical models that have been used and which could be used in the future are discussed in Chapter 7. Chapter 7 discusses models of rail fasteners, resiliently supported ties, floating slabs, tracks, subway/soil interation and radiation from the subway structure, vibration propagation and attenuation in soil, and building response to groundborne vibration.

2. OVERVIEW OF GROUNDBORNE NOISE AND VIBRATION

2.1 PROBLEM DEFINITION

Groundborne noise and vibration from rapid transit systems is caused by wheel/rail interaction and is influenced by such factors as wheel and rail roughness, discrete track supports and, perhaps, local variation of surface contact stiffness. Groundborne vibration is influenced by truck dynamic characteristics, rail support stiffness, transit structure design, soil characteristics, and building structure design. In general, groundborne vibration energy passes through the rail to the fastener and into the transit structure which radiates vibrational energy into the soil in the form of compression, shear and surface Rayleigh waves. Nearby building structures respond to incident groundborne vibration at the foundation, from where vibration propagates throughout the building. Vibrating building walls, floors and ceilings then radiate noise into interior spaces and may also excite sensitive instrumentation such as electron microscopes or precision milling machines. It is most common for transit generated groundborne vibration to create problems in the form of intrusion of building occupants rather than creating damage to building structures or interfering with sensitive instrumentation.

2.1.1 Effects on People

Groundborne noise in buildings is generally confined to the frequency range of about 20 Hz to perhaps 150 to 200 Hz. Groundborne noise intrusion at higher frequencies is virtually non-existent except under very isolated circumstances such as a building structure in direct contact with the subway structure. Usually the resulting building vibration is below the human threshold of perception, however, recent experience at WMATA, MARTA and NYCTA indicate that feelable vibration can occur. Feelable groundborne vibration is generally in the frequency range of 5 Hz to 40 Hz. If the building vibration at frequencies above 40 Hz is of a high enough amplitude to be perceptible by humans, the noise radiated by the vibrating building components will be more intrusive than the vibration.

Human response to groundborne noise is usually one of annoyance and a perceived reduction of property values and property damage. Although A-weighted sound levels from general community noise and interior "living" noise are usually higher than A-weighted sound levels produced by groundborne vibration, the objection to groundborne noise, when perceived, remains quite strong.

One particular reason for the strong objection to even very low, but perceptible, levels of groundborne noise may be the fact that the spectrum associated with groundborne noise is entirely different from that for the usual community and residential noise sources. Groundborne noise usually peaks between 16 Hz to 63 Hz, whereas most other residential noise contains its energy at higher frequencies. Thus, groundborne noise is perceived as a low frequency rumble that is easily heard even when the rumble noise is lower in level than the background noise.

In no case does groundborne noise present hearing damage problems.

In view of the frequency spectrum associated with groundborne noise and vibration, the A-weighted noise level must be used with some care. It is generally accepted that the A-weighted sound level can underestimate the annoyance potential of low frequency noises. An extension of the NC criteria curves to include octave band frequencies down to 31.5 Hz and perhaps even 16 Hz would seem more appropriate for characterizing groundborne noise than the A-weighted sound level. Suitable descriptors of vibration are equally difficult to determine, although the tendency today is towards using the vibration velocity level as the primary discriptor for human response. Weighted vibration velocity levels have been proposed, which de-emphasize vibration below 6 Hz to 10 Hz, and which correspond with the human sensitivity threshold. For groundborne vibration, due to the spectrum, these weighted vibration velocity levels are essentially the same as the overall vibration velocity level.

2.1.2 Effects on Buildings

In virtually all the literature surveyed, no direct evidence has been found indicating that transit-induced groundborne vibration has caused building damage. The maximum amplitudes of building vibration caused by transit systems are generally 1/10 to 1/100 that normally prescribed by building damage criteria. Indeed, settling of buildings after completion of a nearby subway system is most likely related to normal settlement, and perhaps to subsidence brought on by construction, and is not related to groundborne vibration.

2.1.3 Industrial/Commercial Interference

Only under very rare circumstances has groundborne vibration from a rapid transit system been considered potentially troublesome to industrial and/or commercial activity. Those activities most likely to be associated with vibration interference include electron microscopy, photo micrography, super-precision machining and vault intrusion detection. With ever advancing precision manufacturing techniques (eg. for micro-electronic chips and computer disc memories) groundborne vibration from all sources will be of increasing concern in the future.

Of great concern among commercial and industrial organizations is the sensitivity of their data processing equipment to groundborne vibration from rapid transit systems. In most cases, however, vibration interference with data processing equipment is minimal. The vibration produced by the data processing equipment and operators walking over flexible raised computer platform floors is usually in excess of that produced by transit systems, even if the building is in direct contact with the transit structure.

2.2 CRITERIA

A variety of criteria have been developed for transit related groundborne noise and vibration. These are discussed in detail in Chapter 3 of this report. Until recently most criteria have focused on the acceptable levels of groundborne noise with the maximum noise level dependent upon the type of building occupancy. The noise level limits have been specified in terms of A-weighted level and NC curves with the NC curves extended to the low frequency range. The criteria that have been used to specify acceptable levels of groundborne noise have generally been found satisfactory. When the criteria are not exceeded, community complaints are very rare.

In contrast, the criteria that have been used for specifying acceptable levels of building vibration have been found to be inadequate in many instances. Recent work by standard organizations (e.g. ISO and ANSI) and measurement results from buildings in which occupants have complained about groundborne vibration have allowed development of refined criteria for acceptable levels of building vibration. These criteria are given both in terms of overall weighted vibration levels and a family of curves analogous to the NC curves for 1/3 octave band spectra.

2.3 MEASUREMENT TECHNIQUES

As many measurement and analysis techniques have been used to evaluate groundborne noise and vibration from transit systems as there are performing organizations. The state-of-the-art includes measurement of 1/3 octave band vibration acceleration and/or velocity levels. Only rarely have more sophisticated techniques such as transfer function, impedance or correlation measurements been performed.

Measurement locations for subway structure vibration have not been standardized, although such standardization is being attempted by UITP. The proposed standard provides for measurements at both the invert and subway wall of normal vibration velocity or acceleration, and, unprecedented in the industry, further suggests additional identical measurements at 20 meters to either side of the first set. Measurement data are to be reported as 1/3 octave vibration velocity levels down to at least 10 Hz.

2.4 CONTROL TECHNOLOGY

Some of the methods that have been used to control groundborne noise and vibration are:

- Welded rail
- Soft primary springing on trucks
- Resilient wheels
- Wheel truing
- Rail grinding
- Resilient direct fixation rail fasteners
- Floating slabs
- Extra heavy tunnel structures
- Increasing tunnel depth
- Ballast mats for ballasted-and-tie track
- Trenches or underground barriers
- Reduction of train speed

As can be seen from this list, there are a large number of methods that can be used to control the levels of ground vibration. All of the methods listed above have been used with at least limited success. The most common methods include use of resilient instead of rigid, direct fixation fasteners, resiliently supported ties and the use of floating slabs. Also included in the list are modifications to the truck, particularly the truck suspension. There are no documented instances in which transit car trucks were modified for the specific purpose of reducing the groundborne vibration. However, there are several cases where one type of transit car cannot be used on a specific route because of ground vibration problems. Transferring the problem cars to other routes is in effect a modification in the truck dynamics for the purpose of reducing ground vibration.

Recent experience has indicated that design of the trucks may have a more significant effect on the levels of groundborne vibration than previously suspected. Some of the specific features that can be incorporated into a truck design to reduce groundborne vibration are:

- Reduce primary stiffness
- Minimize wheel-set and axle mass
- Effective load equalization between wheels
- Increase structural damping with the use elastomers and dynamic energy absorbers
- Eliminate metal-to-metal contact with rubber bushings or pads

One of the first steps in any program for controlling groundborne vibration is to minimize the wheel and rail roughness. This is accomplished through the use of welded rail in place of jointed rail and maintaining a continuing program of wheel truing and rail grinding. Wheel truing and rail grinding are now generally recognized as necessary for keeping the facilities in good condition and minimizing both airborne noise and groundborne noise and vibration. At this point most U.S. transit systems use resilient fasteners on concrete trackbed in new subway installations. The resulting rail support modulus of the fasteners is typically between 3000 lb/in^2 to 4000 lb/in^2. Unusually soft fasteners, specifically designed to reduce groundborne vibration, will reduce the rail support modulus to 1000 lb/in^2 to 2000 lb/in^2. If further vibration isolation is required either STEDEF resiliently support ties or floating slabs are used. At this point, the lightweight floating slabs of either the continuous type used in Washington, D.C. or the discontinuous type used in Toronto and Atlanta represent the most effective means of controlling rail transit groundborne vibration. Discontinuous floating slabs initially developed by the Toronto Transit Commission are now in use in several new transit systems. They provide substantial reduction of vibration usually at a significantly lower cost than the continuous, poured-in-place floating slabs.

Note that most of the lightweight floating slabs have been designed to have resonance frequencies of 14 Hz to 16 Hz such that they will provide significant vibration isolation at frequencies above 20 Hz. These designs have been found to work very well when the peak frequency of the groundborne vibration is above about 35 Hz. However, if the peak ground vibration amplitude is at lower frequencies, particularly in the 15 Hz to 20 Hz range, the design of the floating slab should be modified so that the resonance frequency is in the range of 10 Hz to 12 Hz. This can be accomplished by doubling the mass of the floating slab while leaving the support pads unmodified.

2.5 PREDICTION METHODS

The available methods for prediction of groundborne noise and vibration are at a relatively early state of development. Most are designed to estimate worst case or upper bound noise or vibration levels. Considering the spread in observed groundborne noise and vibration level data, typically 10 dB to 20 dB for any given octave band, these methods should not be termed "prediction" methods, but rather indicators of when or where groundborne noise and vibration may be a problem, for which appropriate controls should be implemented.

Only two comprehensive prediction methods actually exist. One, developed by Wilson (Ref. A-5) begins with representative spectra for groundborne vibration in the ground at a standard distance from earth, mixed-face and rock subways. The other primary prediction procedure, developed by Ungar and Bender (Ref. A-2, A-136), begins with worst case or upper bound subway wall vibration spectra. Corrections are then used in both major procedures to account for track configurations different from standard direct fixation, train speed, subway structure parameters, propagation distances, etc.

With respect to propagation, the two methods are substantially different. Wilson applies empirically determined attenuation functions of distance and frequency representative of typical soils. Ungar and Bender, on the other hand, employ loss factor and propagation velocity data for various soil types and account for reflection and transmission at soil layer interfaces. This method relies upon soil data obtained from preconstruction surveys or other sources - e.g. site investigations performed for construction of large commercial buildings. Assuming that sufficient soil parameter data is available and sufficient engineering time is devoted to the prediction process the method should predict attenuation with distance in soil with good accuracy. However, the attenuation law used by Ungar and Bender presupposes the dominance of compression waves over shear waves in the transmission process, thus giving a conservative estimate of attenuation as a function of distance. Shear waves may actually be more significant at distances close to the subway structure while compression waves, as well as Rayleigh surface waves, may dominate at large distances.

The natural approach to development of a suitable prediction method appears, as a result of the review of available literature, to be to combine Wilson's starting spectra for groundborne vibration in soil at a representative distance from various subways of different types and soil conditions, with

Ungar's and Bender's attenuation models. Starting with the vibration spectra in the ground rather than the subway wall vibration spectra avoids problems with possible non-radiating tunnel vibration modes and tunnel/soil coupling. Additionally, the starting spectra can be normalized to a given soil stiffness or, equivalently, to determine a source strength analogous to a force which is independent of soil stiffness, therefore removing the effect of soil stiffness on the starting spectra. The attenuation models can then be applied to include the effect of soil parameters on attenuation with distance. Additional development of the attenuation vs. distance models would include determination of energy partition between compression, shear and Rayleigh wave energy to which the wave types' respective loss factors may be applied. The remaining model development would essentially involve compilation of corrections for train speed, fastener stiffness, subway structure mass, truck design and so-forth.

2.6 MATHEMATICAL MODELS FOR PARAMETER EVALUATION

There are a number of mathematical models that have been used for describing floating slab performance, attenuation in soil, truck dynamics, etc. These models serve as useful tools which, together with measurement data, can be used to account for specific design parameters and to aid design and development of vibration control provisions.

Some of the limitations of existing models are:

1. No models have been developed for confident evaluation of the effect of tunnel wall thickness, stiffness, and mass or subway founding condition.

2. No models have been developed which allow estimating the partition of energy between compression, shear, and Rayleigh waves, necessary for accurate prediction of attenuation in soil as a function of distance.

3. There no are models presently implemented that allow investigation of the influence of the truck suspension parameters on the levels of groundborne vibration.

4. Factors such as layering of soil, proximity of the rock line, depth to the water table are too complex to be accounted for in existing models.

5. None of the existing models provide a reasonable explanation for groundborne vibration from transit operations exhibiting such a strong frequency dependence. The peak frequency appears to be dependent on the soil stiffness, however none of the models can use soil parameters to predict the peak frequency.

With respect to the remaining facets of groundborne noise and vibration - eg. floating slab/vehicle interaction, soil/building interaction, fastener isolation, etc., models are available which have either been applied specifically to rapid transit system vibration or which could be extended from such areas as dynamic truck stability analysis, soil structure interaction, or structural building dynamics.

2.7 EXTENT OF LITERATURE

The literature concerning groundborne vibration from rapid transit systems is very extensive. All of the various topics or facets of the problem are dealt with in some manner or another. The literature almost always concerns the results of measurement programs or design and implementation of vibration control provisions. As a result, a great wealth of knowledge is available today.

The difficulty in assimulating the existing knowledge concerning groundborne vibration is that little consistency exists in data presentation by different workers. In the United States and Canada, vibration data are usually presented in terms of 1/3 octave band rms acceleration re micro g. In Europe, however, it is more common for vibration velocity levels to be employed, and, to add to the confusion, two reference velocities are used, one being 5×10^{-8} m/s and the other 1×10^{-8} m/sec. Note that in the U.S. the reference velocity level most often used is 1×10^{-6} in/sec, or about 2.5×10^{-8} m/sec.

Some workers present groundborne noise data entirely in terms of A-weighted levels, and groundborne vibration levels in terms of overall acceleration or velocity. In almost all instances of published noise or vibration data, no data concerning soil parameters and layering are presented. This latter problem is perhaps one of the most serious limitations, as such parameters may seriously affect vibration propagation. Clearly the limitation is that these data are difficult to obtain, and once obtained are difficult to present in a simple summary.

Most of the major U.S. and Canadian transit systems are represented in the literature, including, but not limited to BART, WMATA, CTA, NYCTA, MARTA, SEPTA, and TTC.

Of these, the literature concerning TTC, NYCTA, and WMATA is most extensive. Curiously, much groundborne vibration data were collected at the BART Diablo test track, but virtually none after commencement of revenue operation. This is an indication of the lack of significant groundborne vibration problems at BART.

European transit systems covered in the literature include those located in Paris, Vienna, Cologne, Munich, London and Stockholm.

Additional systems represented are those being built in Melbourne (MURLA) and Hong Kong. A very great amount of work has been performed by the Japan National Railways regarding the Tokaido and Shinkansen Rail systems. The study of the groundborne vibration at these systems in Japan is all the more interesting because they involve some of the fastest trains in the world.

Of all the literature surveyed, that concerning the groundborne vibration and noise produced by the TTC/YSNE tunnels in Toronto, Canada, is by far the most extensive. Topics include numerous measurements of groundborne vibration and noise, multi-degree-of-freedom modeling of trucks, dynamic model and impedance measurements of trucks, transfer function measurements using drop impacts on the rail and invert, and soil surveys. Vibration control techniques tested include reduction of resilient fasteners and primary suspension stiffnesses, wheel and rail grinding, speed reductions, and resilient wheels. More recent work was performed to evaluate floating slabs, Chevron truck springs, screening and ballast mats. Thus, past and present work performed by the TTC and their consultants concerns much of the state-of-the-art in transit system noise and vibration control.

Within the United States, the literature concerning the WMATA system is perhaps most extensive, but is confined to floating slabs, fasteners and ground vibration for various founding conditions. Additional significant work concerning fasteners, floating slabs and trucks is reported by the NYCTA. Chicago's CTA, involved with car procurement, has conducted measurements of groundborne vibration for various truck designs with very significant results. Additional measurements are being conducted at CTA by CUTD for evaluation of various trackbed designs.

A substantial amount of literature has been found concerning soil parameter characterization using geophysical techniques. These techniques include (1) seismic refraction, (2) up-hole, down-hole, (3) cross-hole, and (4) continuous sine wave excitation. Indeed, the techniques are at a high state of development and may reveal such parameters as propagation velocities, damping and layer thicknesses. It is still necessary to integrate these techniques into the set of tools available for prediction and control of groundborne noise and vibration. Also, significant literature exists concerning wave propagation in soils, especially in the fields of geophysics and civil engineering.

3. GROUNDBORNE NOISE AND VIBRATION CRITERIA

Groundborne noise and vibration, transmitted from transit operations to adjacent buildings, can be a major source of community annoyance. The problem is generally groundborne noise radiated from room surfaces. The vibration is not often perceptible, although perceptible vibration has occasionally resulted as the principal grounds for complaint. Therefore, criteria for groundborne noise and vibration should address acceptable levels of both noise and vibration.

The purpose of this section is to review various types of groundborne noise and vibration criteria, vibration induced building damage criteria and soil settlement criteria.

3.1 VIBRATION CRITERIA

3.1.1 Background

Basically there are two kinds of human exposure to vibration transmitted through a structure: (a) vibration transmitted to the human body as a whole through the supporting surface, namely the feet of a standing person, the buttocks of a seated person or the supporting area of a reclining person; (b) vibration of the building and the resulting reactions of the occupants. This includes the gross structure vibration (rocking or shear deformation), floor vibration (primarily vertical motion), and wall vibrations (primarily horizontal motions producing secondary noises such as rattling). Noise is considered separately in Section 3.2. The reactions are typically fear of damage to the structure or its contents, startle, and interference with sleep, conversation or other activities.

In the past, many studies have been conducted by military and aerospace groups concerned with such things as human fatigue from vehicle vibration, and effect of the vibration on task performance (Ref. A-178 to A-181). Only very recently have researchers and environmental engineers considered the problem of perception of annoyance as well as general subjective sensation of the magnitude of a vibration stimulus from roadway traffic, construction, railroad and rapid transit operations.

When the original criteria for groundborne noise and vibration were developed, very limited information was available on human response to building vibration. Indeed, the threshold of perception as defined by various researchers, varied over a range of 10 dB to 20 dB. However, the recent amendments to ISO standard 2631/DAD1 (Ref. A-181), draft American National Standard on "Guide to the Evaluation of Human Exposure to Vibration in Buildings" (Ref. A-182), both concerning building vibrations, and the work of the Committee on Hearing, Bioacoustics and Biomechanics (CHABA) Working Group 69 (Ref. C-1), provide a good basis for developing criteria applicable to building vibration due to transit operations.

The primary purpose of this section is to summarize the principal existing and proposed vibration standards and limits that may be applicable to groundborne noise and vibration due to rapid transit systems. The secondary aim is to provide a brief historical review of the work relating to whole-body vibration. Whole-body vibration is generally considered to be that due to vibration of the principal supporting surface for the body. This definition is primarily intended to distinguish whole-body vibration from local vibration due to, for example, the vibration of hand-held tools.

The earliest work on the "whole-body" sensitivity to vertical vibration was first reported in 1931 by Reicher and Meister (Ref. A-178). They produced a set of equisensation curves, see Figure 3.1, similar in concept to the equal loudness curves for sound. Although this was developed over fifty years ago, its validity is still accepted for steady-state vibration. But for transient vibrations, for example, floor vibrations produced by people walking, there is recent evidence that amplitudes much greater than those given by the scale are necessary to produce a given sensation at a given frequency.

It was noted in the Reicher and Meister investigation that vertical vibration was most readily detected when people were standing, whereas horizontal vibration was more noticeable when they were lying down. This investigation revealed that the threshold of perception for vertical vibration, between the frequency range of 5 Hz to 70 Hz, is at 0.3 mm/sec peak velocity and a vibration is annoying if the velocity exceeds 2.5 mm/sec.

Another simple and widely used method for assessing the level of interference from vibration is to use the Dieckmann (Ref. A-179) K values, defined in Table 3-1. Graphical representation of K-values is also available, see Fig 3.1. Note that Dieckmann's data extends into a lower range of frequencies, starting at 0.5 Hz.

It has been verified by Dieckmann (Ref. A-179) and Miwa (Ref. A-180) that the human threshold to vertical vibration in the intermediate frequency range is best approximated by a constant velocity curve. However, as compared with Reiher and Meister, Dieckmann and Miwa usually give the lower bound of perceptible vibration in the intermediate range, see Figure 3.1. The threshold curves for vertical vibration given by Reiher and Meister, Dieckmann and Miwa have been shown to agree quite well (Ref. A-183).

TABLE 3-1 DIECKMANN K-VALUES (REF. A-179)

VERTICAL VIBRATION	HORIZONTAL VIBRATION
Up to 5 Hz, $K = Af^2$	Up to 2 Hz, $K = 2\ Af^2$
5-40 Hz, $K = 5\ Af$	2-25 Hz, $K = 4\ Af$
Above 40 Hz, $K = 200\ A$	Above 25 Hz, $K = 100\ A$

A = amplitude in mm, f = frequency in Hz.

The regions for vibration sensitivity are defined as follows:

$K = 0.1$, lower limit of perception,

$K = 1$, allowable in industry for any period of time,

$K = 10$, allowable for short durations only,

$K = 100$, upper limit of strain for the average man.

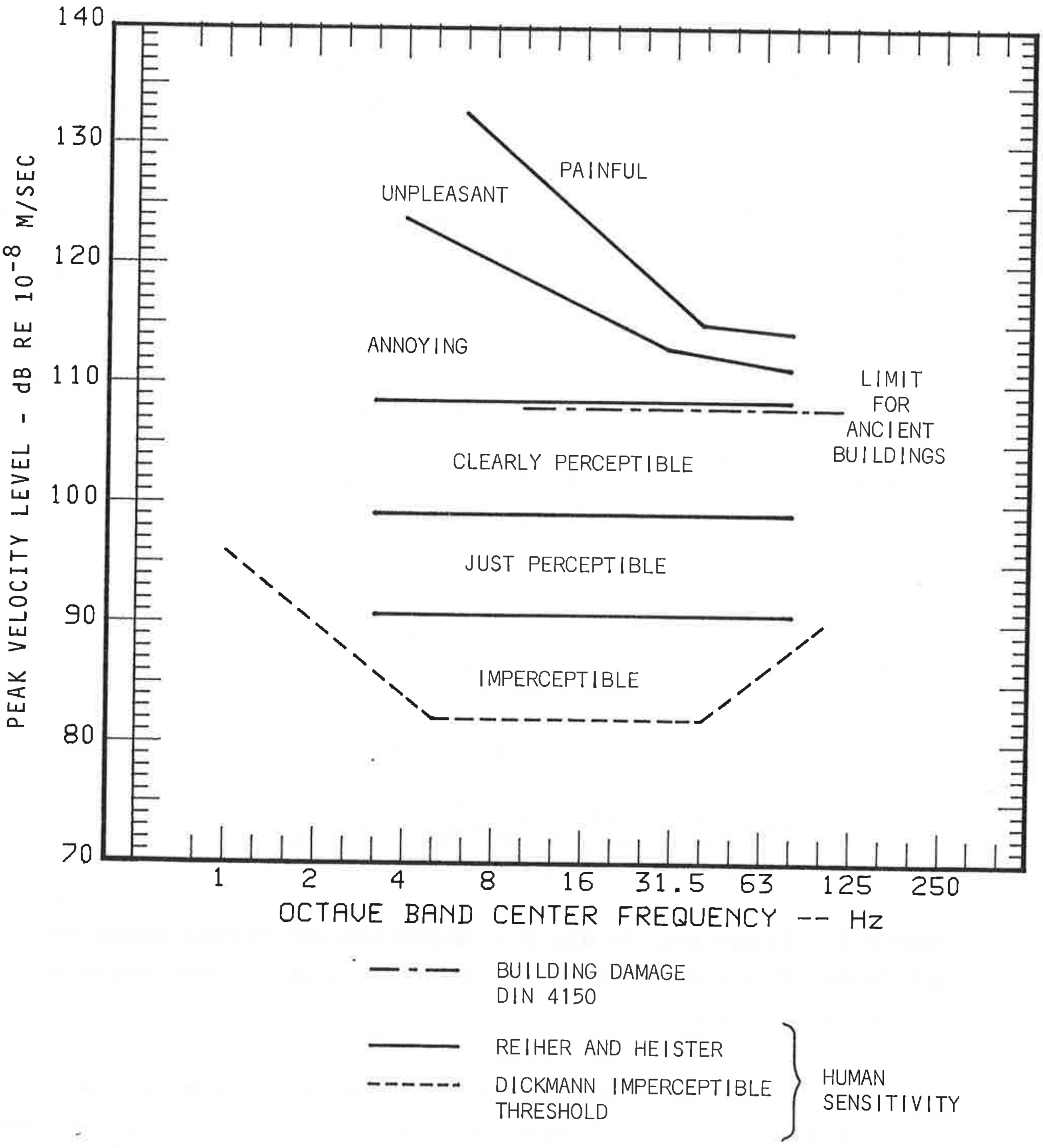


FIGURE 3.1 HUMAN SENSITIVITY: EFFECTS OF VERTICAL VIBRATION

One of the earliest national standards to incorporate Dieckmann's work was German Standard DIN 4025 (Ref. A-184) which was concerned with the effect of vibration on the ability to work. It does note that vibration slightly above perceptible is classified as: "Bearable, but moderately unpleasant if lasting for an hour." DIN 4025 also extends the frequency range downwards and gives criteria for horizontal as well as vertical vibration. A complete summary of the relevant data from DIN 4025 is shown in Table 3-2.

Steffens (Ref. A-185) records the details of the German Standards Institute draft revision of the older standard DIN 4150 (Ref. A-186). This standard gives charts for the calculation of K-values based on frequency and amplitude of vibration. The curves are applicable to vertical and horizontal vibration at frequencies between 0.5 Hz and 80 Hz, for people sitting or standing. For frequencies up to 5 Hz, intensity is roughly proportional to acceleration. For frequencies greater than 15 Hz intensity is roughly proportional to velocity, the velocity for a value of K=10 being about 15 mm/s and for K=1 about 1.5 mm/s. In this standard, K values are derived as:

$$K = \frac{0.005\ A\ f^2}{\sqrt{100 + f^2}} = \frac{0.8\ Vf}{\sqrt{100 + f^2}} = \frac{0.125\ a}{\sqrt{100 + f^2}}$$

where f = frequency in Hz; A = amplitude of displacement in micro-mm; V = maximum velocity in mm/s; and a = the maximum acceleration in mm/s^2.

The values thus obtained differ, but not usually widely so, from those given by the Dieckmann results (Table 3-1). The assessment of intensity according to the proposed standard is shown in Table 3-3.

TABLE 3-2 CLASSIFICATION OF K VALUES (D.I.N. 4025, REF. A-184)

K Value	Classification	Effect on Work
0.1	Threshold value. Vibration just perceptible	Not affected
0.1 - 0.3	Just perceptible. Easily bearable, scarcely unpleasant	Not affected
0.3 - 1	Easily noticeable. Bearable, but moderately unpleasant if lasting for an hour	Still not affected
1 - 3	Strongly noticeable. Still tolerable, but very unpleasant if lasting over an hour	Affected, but possible
3 - 10	Unpleasant. Can be tolerated for periods of up to 1 hour, but not for longer	Considerably affected, but still possible
10 - 30	Very unpleasant, cannot be tolerated for more than 10 minutes	Barely possible
30 - 100	Extremely unpleasant. Not tolerable for more than 1 minute	Impossible
Over 100	Intolerable	Impossible

TABLE 3-3 INTENSITY (K-UNITS) AND SUBJECTIVE EFFECTS (FROM REF. A-85)

K Value	Degree of Perception
below 0.1	not felt
0.1	threshold of perception
0.25	barely noticeable
0.63	noticeable
1.6	easily noticeable
4	strongly detectable
10	very strongly detectable

(K values of 25 and 63 are also given, but it is stated that it is not possible to distinguish between their effects on people)

Draft revision of DIN 4150 (Ref. A-185) summarized in Table 3-4, also gives acceptable levels of vibration for buildings such as houses, offices and hospitals. Sustained vibrations are defined as lasting for more than two hours continuously. Repeatedly occurring vibrations are sustained vibrations that occur only occasionally, or are shocks that recur at various intervals. Occasional shocks are transient vibrations lasting for a short time only (for example shocks from blasting that may occur only one to three times per day). Values given in brackets apply to cases where the frequency of vibration is below 15 Hz.

Another relevant German Standard (DIN 4150 Part 2, Ref. A-186) addresses the topic of acceptable guidelines for building vibrations. Unfortunately, details of this standard cannot be presented here since no translation is available.

Since 1974 the principal and widely quoted human vibration standard has been the International Standards Organization ISO 2631 (Ref. A-181), entitled "Guide for the Evaluation of Human Exposure to Whole-Body Vibrations," which is based upon the work of a large number of previous investigators. A similar document (Ref. A-187) was published in the United Kingdom at about the same time. The relevant standard issued by the American National Standards Institute, ANSI 53.18 - 1979 (Ref. A-188) is also essentially the same as ISO 2631. More recently, many other documents have appeared to amend or extend the guidance provided in ISO 2631. A major revision of ISO 2631 is currently being considered but this is not expected to be completed for several years.

ISO 2631 is framed mainly in terms of the effect of vibration on working ability and fatigue. It gives numerical limits for exposure to vibration transmitted from solid surfaces to the human

TABLE 3-4 ACCEPTABLE VIBRATION LEVELS (FROM REF. A-185)

Building Areas	Time	Permissible Intensities or K Value		
		Sustained Vibrations	Repeatedly vibrations occurring	Seldom occurring shocks
Health resorts, Hospitals Nursing homes (SO)	Day Night	Threshold of perception	Threshold of perception	2.5 Threshold of perception
Small building estates (WS) Purely residential areas (Wh) General residential areas (WA) Weekend living areas (SW) University areas (SO)	Day Night	Threshold of perception	0.2 (0.1) Threshold of perception	4 Threshold of perception
Village areas (MD) Mixed areas (MI) Central areas (MK)	Day Night	0.3 (0.15) Threshold of perception	0.63 (0.3) Threshold of perception	8 Threshold of perception
Business areas (GE) Industrial areas (GI) Port areas (SO)	Day Night	0.63 (0.3) 0.4	0.8 (0.4) 0.4	12 0.4

body over the frequency range 1 Hz to 80 Hz, applied to three axes of translational vibration. Exposure to vibration is divided into three categories:

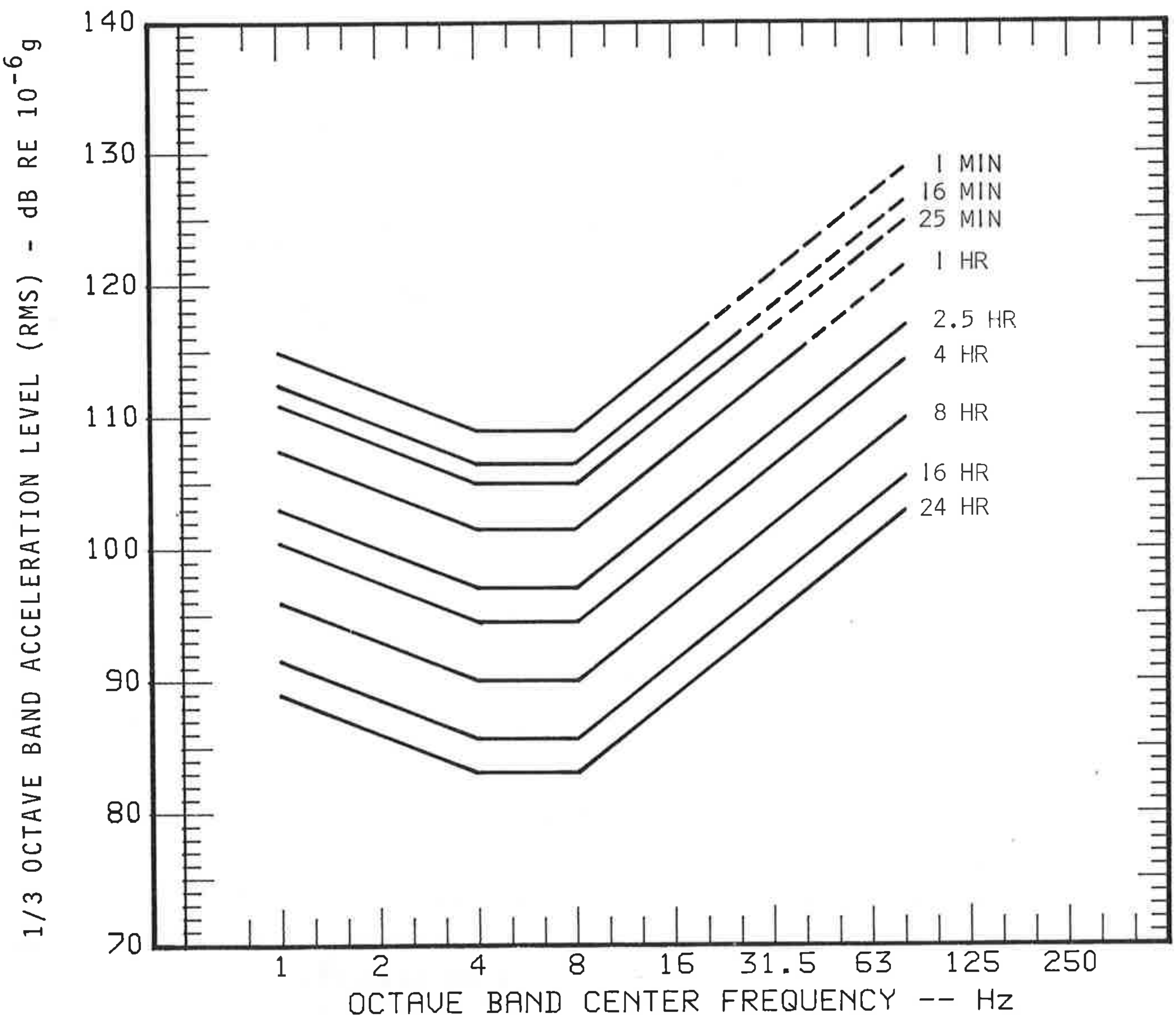
"exposure limits"	concerned with the preservation of health or safety
"fatigue-decreased proficiency boundary"	concerned with the preservation of working efficiency
"reduced comfort boundary"	concerned with the preservation of comfort

The limits corresponding to the above three criteria are given in a simple hierarchical relationship such that for any particular vibration frequency, axis and duration:

- exposure limits = 2 times "fatigue decreased proficiency" (FDP) limits (6 dB higher).

- reduced comfort boundary = 1/3 times "fatigue decreased proficiency" limits (10 dB lower).

It is indicated by ISO 2631 that the horizontal threshold of perception is higher than the vertical threshold for frequencies of 3.15 Hz and upwards, see Figure 3.2a and 3.2b.

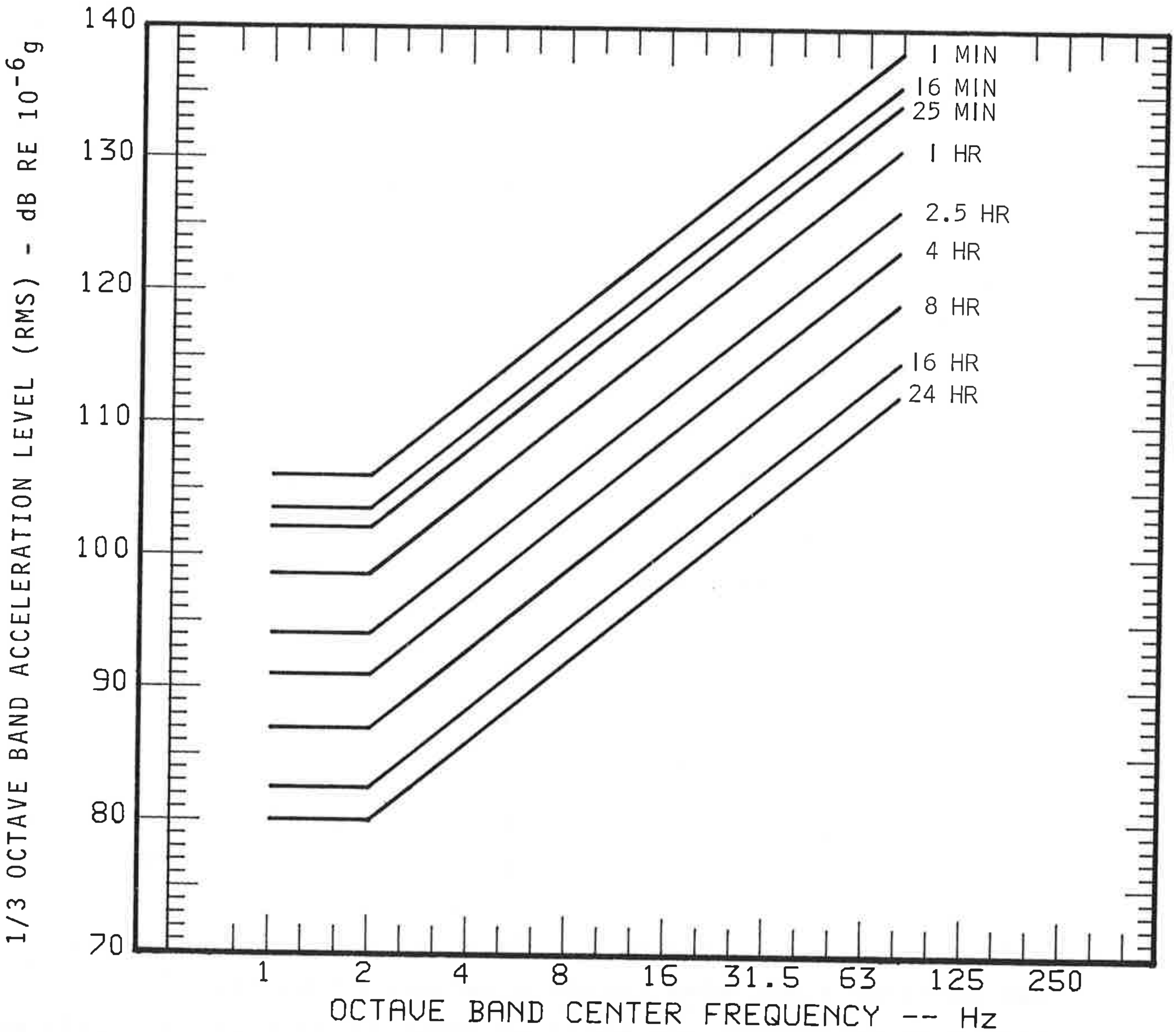


TO OBTAIN:

→ "EXPOSURE LIMITS":
ADD 6 dB

→ "REDUCED COMFORT BOUNDARY":
SUBTRACT 10 dB

FIGURE 3.2a LONGITUDINAL (a_z) ACCELERATION LIMITS, "FATIGUE-DECREASED PROFICIENCY BOUNDARY"



TO OBTAIN:

→ "EXPOSURE LIMITS":
ADD 6 dB

→ "REDUCED COMFORT BOUNDARY":
SUBTRACT 10 dB

FIGURE 3.2b TRANSVERSE (a_x, a_y) ACCELERATION LIMITS; "FATIGUE-DECREASED PROFICIENCY BOUNDARY"

A series of brief amendments to the principal standard have recently been proposed. Draft amendment ISO 2631/DAM1 (Ref. A-189) is intended to clarify and assist the application of the standard. This amendment emphasizes that the standard is concerned with the provision of only general guidance and states that factors not specified in the standard can have large effect. The crest factor limit of 3 given in the standard is raised to 6 and crest factors are refined less ambiguously. The frequency weighting method is advocated as the recommended procedure when assessing the effect of vibration on comfort and performance. It is proposed that when the vibration occurs in three axes the effect on comfort and performance should be determined by taking the square root of the sum of the squares of the weighted values in each axis.

Draft addendum ISO 2631/DAD1 (Ref. A-180) is a guide to the evaluation of human exposure to vibration and shock in buildings. More recently, draft American National Standard ANSI Standard S3.29-198X (Ref. A-182) provides guidelines that are essentially the same as ISO 2631/DAD1. The draft ANSI standard presents limits of vibration acceptability for various building types in the frequency range of 1 Hz to 80 Hz. In other words, this standard defines levels of vibration at which humans will perceive and possibly react when inside a variety of buildings. The frequency weightings are based on those in ANSI S3.18 - 1979 and ISO 2631, but the limits are at about the same level as the threshold of perception of vibration of the most sensitive humans. This threshold is approximately one-half of the value presented in ANSI S3.18-1979 as the mean threshold of perception. The standard notes that the threshold of perception for the most sensitive individuals is considered necessary in order to provide for extremely sensitive areas such as hospitals and hospital operating rooms. Essentially, the frequency-weighted characteristics contained in ANSI S3.18-1979 to describe human response to vibration have been maintained in this standard.

Since vibration measurements will generally be made on a part of the building (normally on the floor at the point of greatest vibration) the axis of vibration of the body will depend on the orientation of the body. For example, vertical building vibration will be z-axis for standing and seated persons but x-axis for persons lying on their backs. A combined standard has therefore been proposed in draft ANSI Standard S3.29-198X, which consists of a combination of the lowest levels of the limits for z, x, and y-axis vibration. This consists of the limits for x and y-axis vibration from 1 Hz to 2 Hz and the limits for z-axis vibration above 2 Hz. The combined curve is shown in Figures 3.3 and 3.4, based on acceleration and velocity respectively.

Figures 3.3 and 3.4, extracted from draft ANSI standard S3.29-198X show various curves corresponding to multiplying factors from 1 to 128. These correspond to the acceptable building vibration levels with corrections for various building types, time of day, and source characteristics. In addition, other corrections can be made based on duration and frequency of occurrence of the events. The levels corresponding to these multiplying factors are regarded as "good environmental standards." Draft Addendum ISO 2631/DAD1 states that vibration levels up to a factor of 2 greater are said to give rise to "moderate complaint." An increase above the basic levels by a factor of 4 will give rise to "major complaints" unless prior warning is given.

The vibration levels in draft Addendum ISO 2631/DAD1 and draft ANSI Standard S3.29-198X are based on a combination of the ISO frequency weighting and some data on vibration threshold (see for example McKay; Ref. A-192). As noted by Griffin (Ref. A-193),

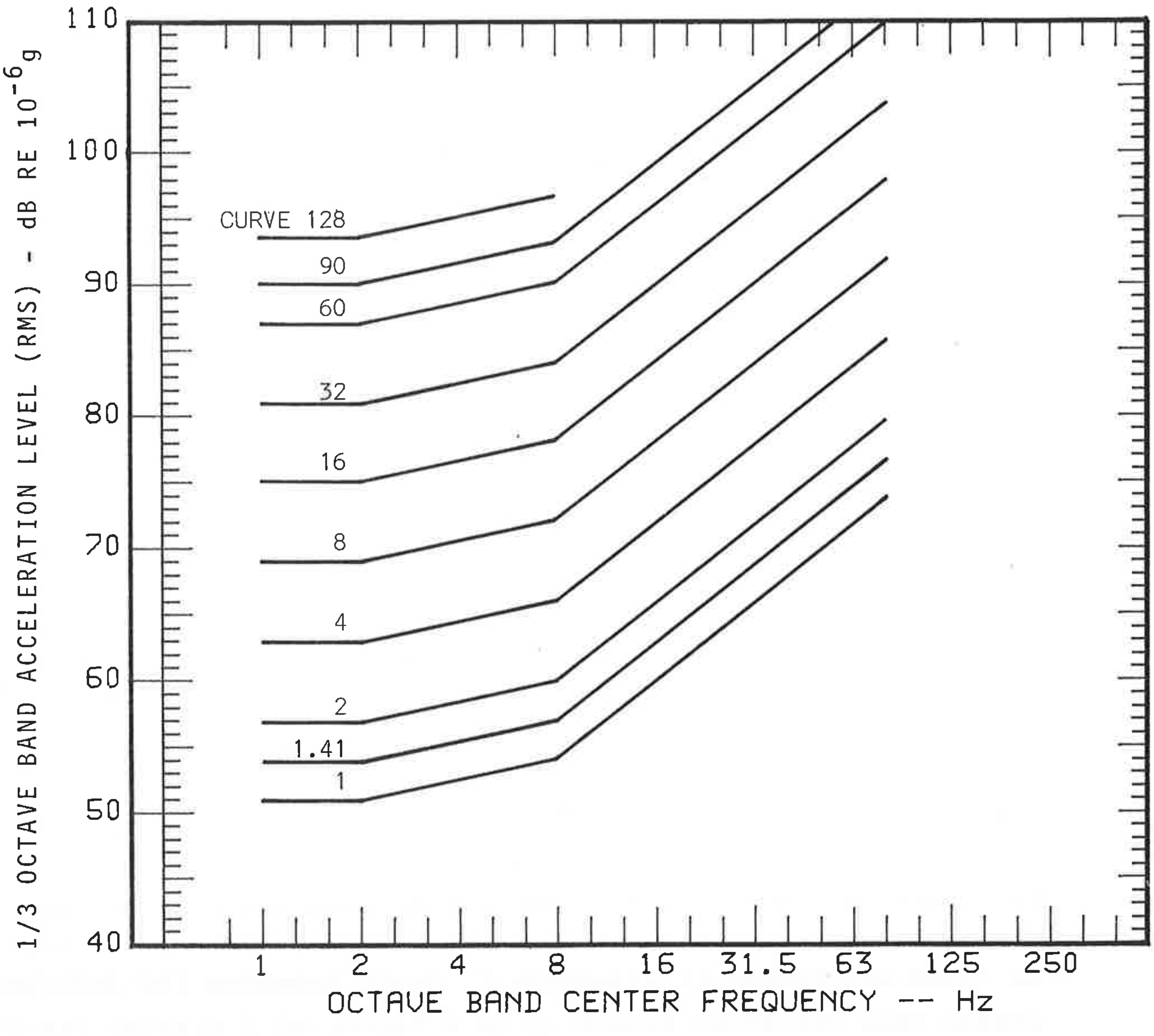


FIGURE 3.3 WORST CASE COMBINED RESPONSE CURVES FOR BUILDING VIBRATION EXPRESSED AS ACCELERATION LEVEL (ADDITIONAL CURVES CORRESPOND TO VARIOUS CORRECTION FACTORS, SEE TEXT)

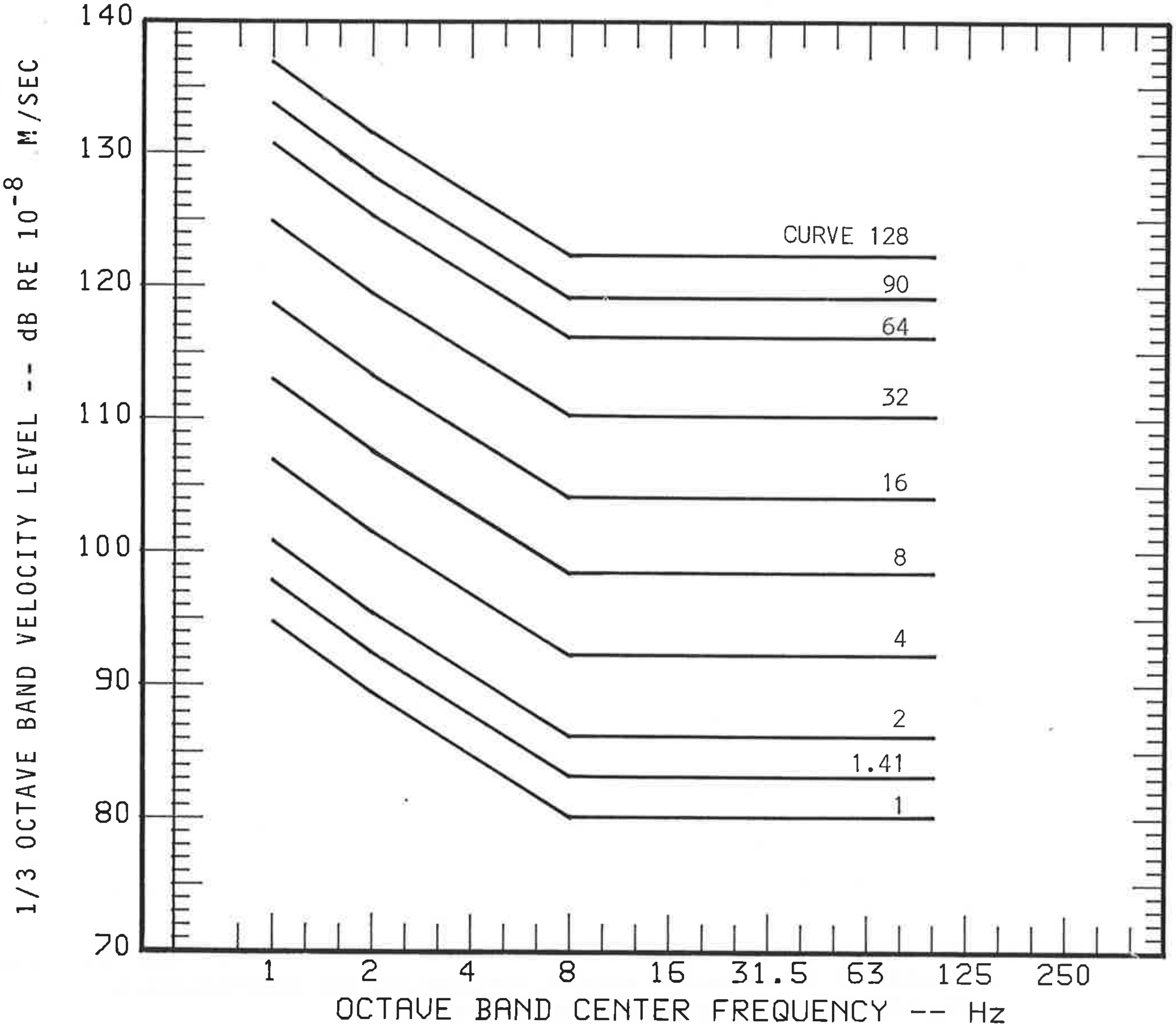


FIGURE 3.4 WORST CASE COMBINED RESPONSE CURVES FOR BUILDING VIBRATION EXPRESSED AS VELOCITY LEVEL (ADDITIONAL CURVES CORRESPOND TO VARIOUS CORRECTION FACTORS, SEE TEXT)

TABLE 3-5 WEIGHTING FACTORS FOR SUGGESTED SATISFACTORY MAGNITUDES OF BUILDING VIBRATION FROM ANSI S3.29-198X (REF. A-182)

Place	Time	Continuous or Intermittent vibration and repeated impulsive shock		Impulsive shock excitation with several occurrences per day (see Notes 4 and 5)	
Hospital operating room and critical working areas	Day	1		1	
	Night	1		1	Note 1
Residential (good environmental standard)	Day	2		90	
	Night	1.4		1.4	
Office	Day	4		128	
	Night	4	See Note 2	128	
Workshop	Day	8	See Notes 2 and 3	128	Notes 2 and 3
	Night	8		128	

Notes:

1. Magnitudes of impulsive shock in hospital operating rooms and critical work places pertain to periods of time when operations are in progress or critical work is being performed. At other times, levels as high as those for residences could be allowed provided there is due agreement and warning.

2. The levels for impulsive shock excitation in offices and workshop areas should not be increased without considering the possibility of significant disruption of working activity.

3. Vibration from certain processes, such as drop forges or crushers which produce high levels of vibration in working places may be in a separate category, from workshops as given in above Table. Vibration specified in ANSI S3.18-1979 will then apply.

TABLE 3-5 (CONT.)

Notes: 4. The trade-off between number of events per day and magnitudes are not well established. The following provisional relationship shall be used for cases of more than three events per day pending further research into human vibration tolerance. Weighting factors are multiplied by a number of event factor, F_n:

$$F_n = 2.13\ N^{-0.688}$$

where N is the number of events per day.

5. For discrete events with durations (T) exceeding one second, weighting factors can be adjusted by multiplying by a duration factor F_d:

$$F_d = T^{-1.22} \text{ for concrete floors}$$

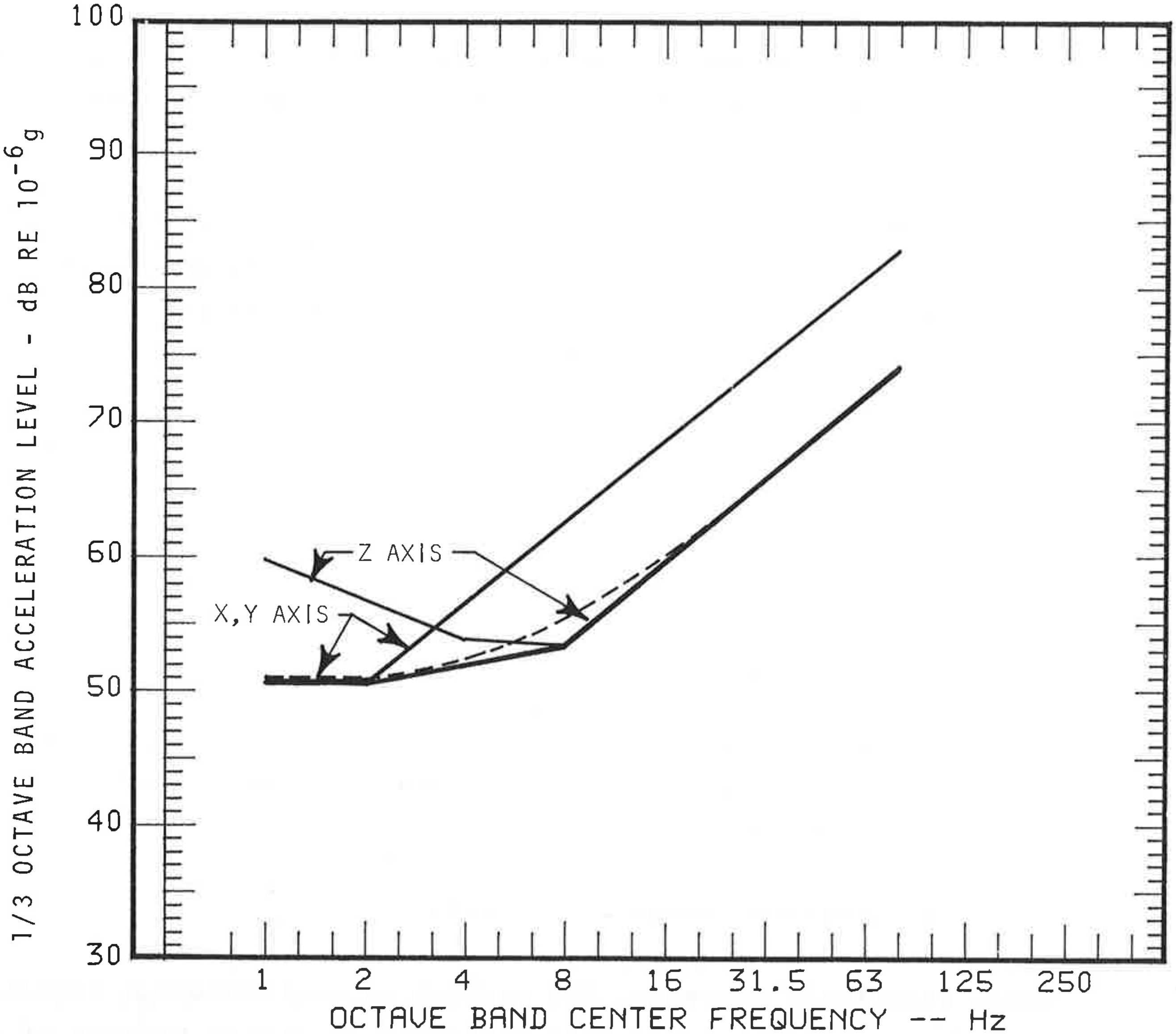
$$F_d = T^{-0.323} \text{ for wooden floors}$$

The event durations in seconds (T) can be estimated from the 10 percent (-20 dB) points of the motion-time histories.

this weighting curve may still require some refinement. Existing evidence suggests that the frequency weighting, and in consequence the guidance in ISO 2631/DAD1, may tend to over-emphasize sensitivity at low frequencies and relatively under-estimate sensitivity to high frequencies.

It is important to note that both draft Addendum ISO 2631/DAD1 and draft ANSI standard S3.29-198X make distinction between vibration generated from continuous, intermittent, and impulsive shock sources. Here the impulsive vibration is generally defined as short duration with rapid build-up to peak followed by damped sinusoidal decay involving one or more cycles. Intermittent vibrations are from repetitive impulsive sources which may be regular (pile drivers, forging presses) or irregular (traffic, intermittent machinery, elevators). The intention of these standards is to treat repeated intermittent vibrations in a manner similar to continuous vibration. It is reasonable to state that vibrations from rapid transit trains and railroads may also be regarded as irregular and thus treated in a manner similar to continuous vibrations.

It is interesting to note that frequency weighting characteristics proposed by the CHABA report (Ref. C-1) are almost identical to the combined curve shown in Figure 3.3. This comparison is presented in Figure 3.5 and shows that there are only minor variations. Figure 3.5 also distinguishes between the building vibration criteria for occupants in buildings for x, y and z-axes. Note that the CHABA weighting curve has been defined for the frequency range of 1 Hz to 80 Hz. Because groundborne vibration rarely has important components outside the range of 1 Hz to 80 Hz, the CHABA weighting appears to be adequate for transit applications.



DASHED LINE ILLUSTRATES FORM OF CHABA PROPOSED WEIGHTING FUNCTION [ATTENUATION (dB) = $20 \log \sqrt{1 + (f/5.6)^2}$]

FIGURE 3.5 WORST CASE BUILDING VIBRATION CRITERIA FOR OCCUPANTS IN BUILDINGS

CHABA also presents guidelines for the threshold of acceptable building vibration which are based on data reported in ISO 2631. These guidelines are reflected in the draft ANSI Standard S3.29-198X.

In summary, it is apparent from the above review that there is a wide variety of material and international standards on human vibration perception currently available. Review has shown that more recent standards, such as American National Standards Institute draft ANSI standard S3.29-198X (Ref. A-182), International Standards Organization draft addendum ISO 2631/DAD1 (Ref. A-190) provide realistic and convenient methods of assessing groundborne noise and vibration from transit trains. Note that both draft ANSI standard S3.29-198X and ISO 2931/DAD1 offer very similar guidelines and were developed for general non-specific vibrational sources.

The next section discusses the assessment of transit induced groundborne vibration using the criteria for human response to building vibration.

3.1.2 Recommended Vibration Criteria

Until relatively recently, the problem of environmental impact of groundborne vibration has been difficult to assess because of the paucity of information on human response to building vibration. However, as indicated in the discussion of the previous section, there are now generally accepted criteria for building vibration such as ISO 2631 (Ref. A-181), although even these standards include qualifications regarding the lack of information on human exposure to building vibration. In this section the results of acceptability criteria based on evaluation of community complaints with the standards of ISO 2631 (Ref. A-181) and the proposed ANSI standard (Ref. A-182) are compared.

As new U.S. transit systems have become operational, there have been a number of instances where community complaints regarding groundborne vibration have been evaluated. Normally the evaluation has included measurement of the structural vibration caused by the transit trains. These measurements provide a data base which can be used to make at least a preliminary evaluation of the suitability of various criteria for transit train induced vibration of residential structures. The remainder of this section presents an evaluation of 15 cases in which structural vibration from transit trains was measured. The acceptability of the vibration environment was divided into four categories (imperceptible, barely perceptible, definitely perceptible, and disagreeable) based on the subjective assessment of the building occupants and the person who took the measurements. This information was originally presented in Reference A-62.

One of the problems in evaluating vibration is the lack of a universally acceptable single number descriptor analogous to the A-weighted sound level used to evaluate sound. Since vibration is usually measured with accelerometers, acceleration is the most commonly reported single number description of vibration. Unfortunately, human response to acceleration is very nonlinear. At frequencies encountered in groundborne vibration from transit trains, human response is better correlated to vibration velocity level than to acceleration.

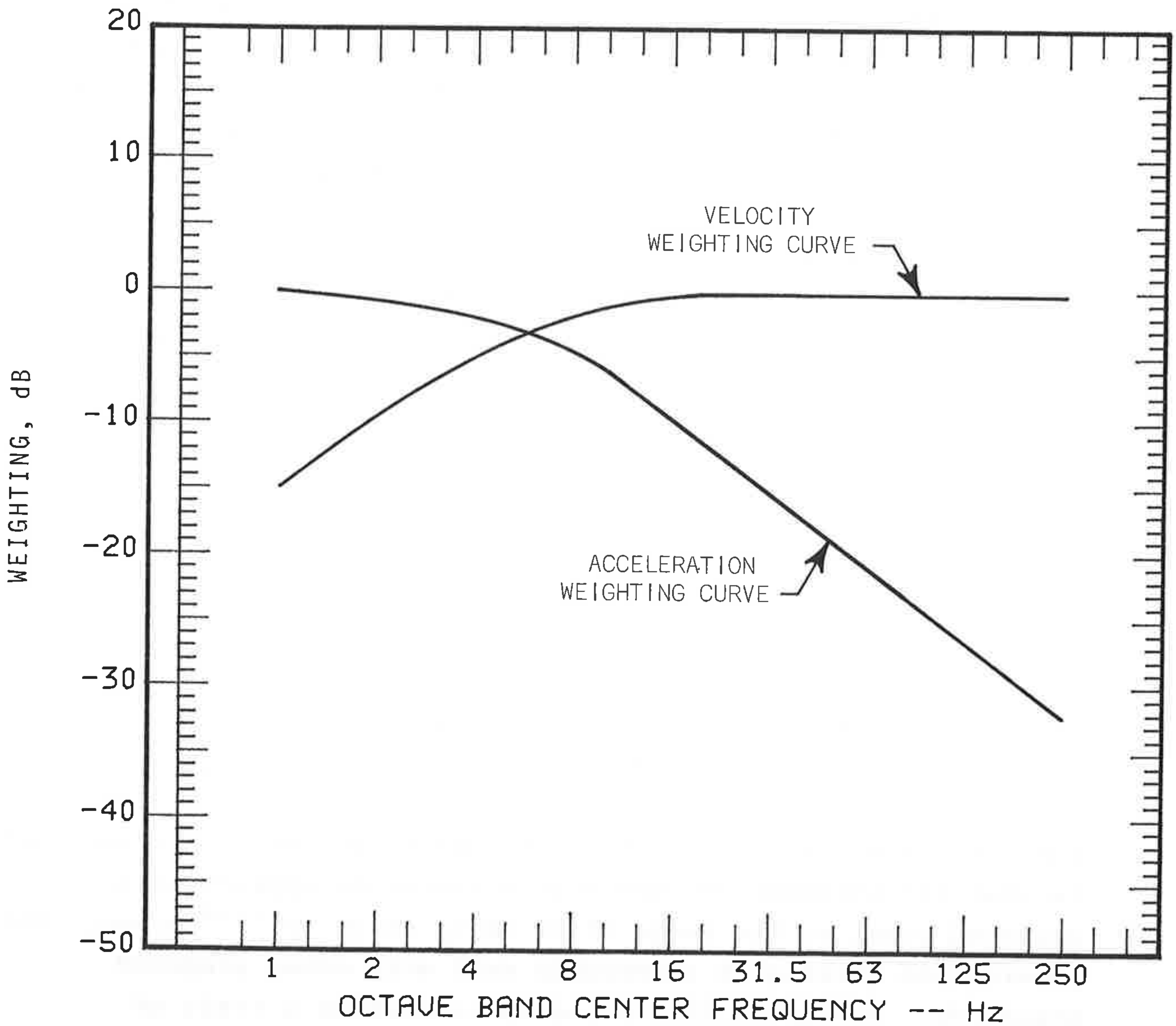
A first step in the development of a universal vibration descriptor is the weighting proposed by the CHABA working Group 69 report (Ref. C-1) and the proposed ANSI Standard (Ref. A-182). Basically the weighting curve provides a level proportional to velocity at high frequencies and proportional to acceleration at low frequencies. The curve presented in the CHABA report is essentially a smooth curve representation of that in the proposed ANSI standard.

For the evaluation in this report we have applied the CHABA smooth curve weighting, modified to be applied to vibration velocity level. The weighting curve is presented in Figure 3.5. For frequencies above 8 Hz, the level obtained with the weighting curve is approximately equal to the overall vibration velocity level. For signals that have significant vibrational energy at frequencies below 8 Hz, the weighted vibration level will be lower than the overall weighted velocity level.

In the CHABA report a weighting curve that can be applied to acceleration signals is defined. If the acceleration weighting curve is represented in terms of velocity level, the curve has the same shape as shown in Figure 3.5. However, it is offset by a constant that is dependent upon the acceleration and velocity reference levels that are used. For example, with reference levels of 10^{-6}g for acceleration and 10^{-6} in/s for velocity, the acceleration weighting curve expressed in terms of velocity level has a 21 dB offset. The result is that for typical transit induced ground vibration the weighted acceleration level will be 21 dB lower than the overall velocity level.

To avoid confusion regarding the weighted vibration level, in this report we use the weighting for velocity level that is shown in Figure 3.6. In the majority of cases, the weighted velocity level is within 1 dB of the unweighted overall velocity level.

For the 15 cases where vibration data were available and the response to the vibration could be realistically assessed, the weighted velocity level was determined using the weighting shown in Figure 3.6. In all cases floor vibration was measured. When possible the transducer was located in the center of the floor. If no center floor measurement was taken, and no reasonable estimate of center floor vibration could be made, then the data



ACCELERATION

$$\text{WEIGHTING (dB)} = -20 \log \sqrt{1 + (f/5.6)^2}$$

VELOCITY

$$\text{WEIGHTING (dB)} = -20 \log \sqrt{1 + (5.6/f)^2}$$

FIGURE 3.6 ACCELERATION AND VELOCITY WEIGHTING CHARACTERISTICS FOR BUILDING VIBRATION IN TERMS OF HUMAN RESPONSE were not used. The subjective response versus the weighted levels are shown graphically in Figure 3.7. Note that the subjective ratings for the vibration environments were not determined by a detailed, scientifically designed, sociological survey. Only the opinion of the person who took the data and the response of the occupants were available. Generally, vibration in the "distinctly perceptible" range was considered unacceptable by both the building occupants and the person who took the measurements. Many occupants were nervous that the vibration might cause damage to the building foundation, a fear that undoubtedly contributes to dissatisfaction, though we have yet to observe a situation in which vibration from transit trains is the cause of structural damage.

The data presented in Figure 3.7 clearly indicate that the weighted vibration level correlates well with the subjective human response to building vibration. Since none of the examples showed significant low-frequency vibration, the same correlation applies to the overall velocity level.

The most important observation relating to the data in Figure 3.6 is that the threshold of acceptable vibration appears to be a weighted level in the range of 79 dB to 84 dB re 10^{-8} m/sec. This observation correlates reasonably well with other proposed standards. The draft ANSI Standard recommends a limit of 1.4×10^{-4} m/sec (83 dB 10^{-8} m/sec) for building vibration or residential structures.

Figures 3.8 and 3.9 summarize the recommended criteria for transit induced groundborne vibration of residential structures. Figure 3.8 presents the levels in terms of vibration velocity level and Figure 3.9 presents the levels in terms of acceleration levels. Also shown are curves from the draft ANSI standard. As indicated

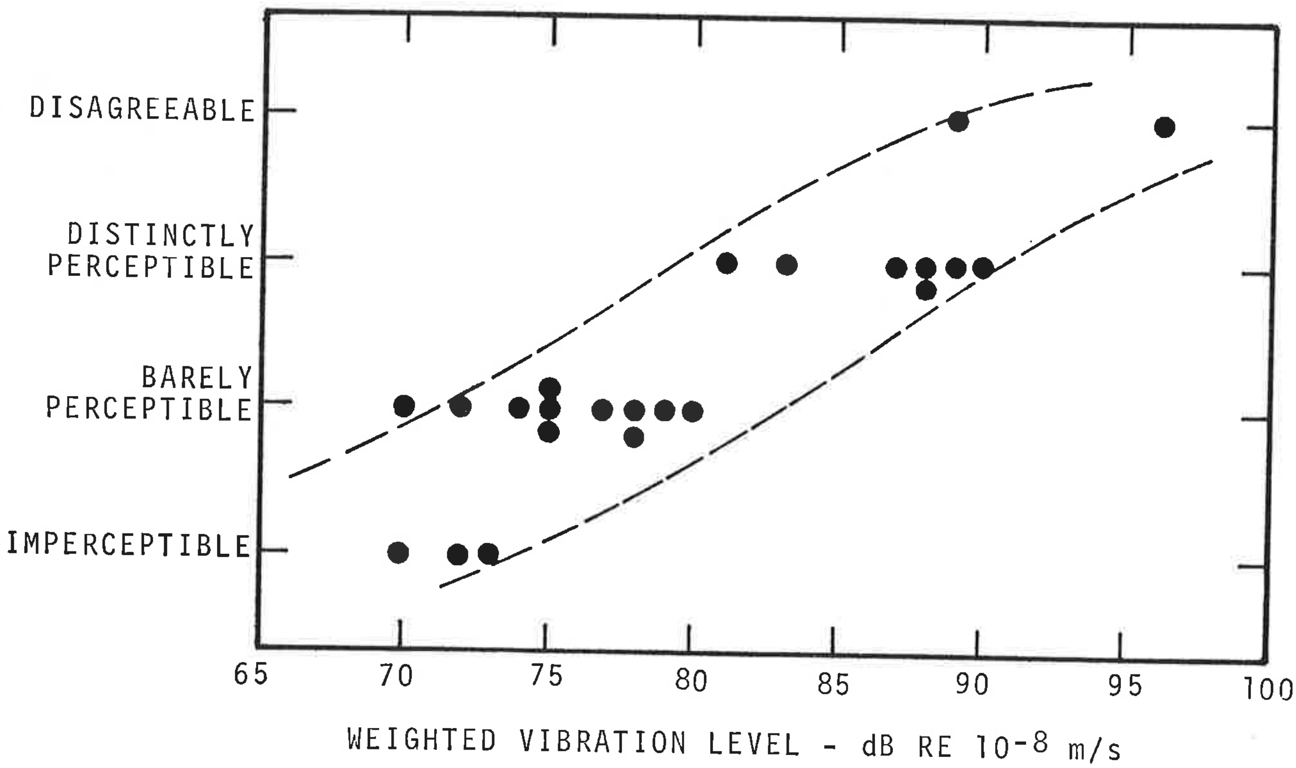


FIGURE 3.7 BUILDING OCCUPANT RESPONSE AS A FUNCTION OF WEIGHTED VIBRATION LEVEL OF THE FLOOR DURING TRANSIT TRAIN PASS-BYS

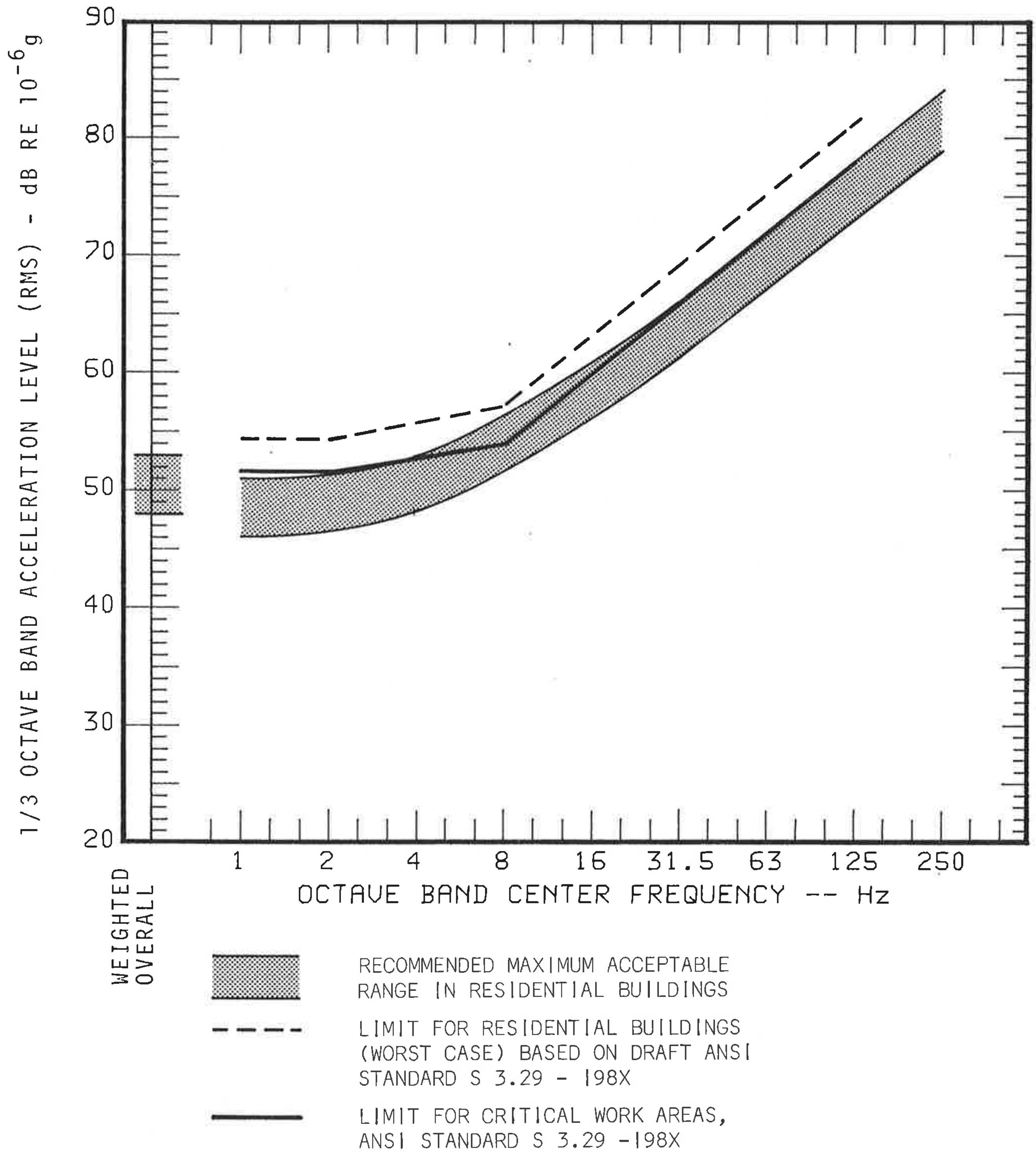


FIGURE 3.8 RANGE OF MAXIMUM ACCEPTABLE TRANSIT INDUCED BUILDING VIBRATION IN RESIDENTIAL AREAS (ACCELERATION LEVEL)

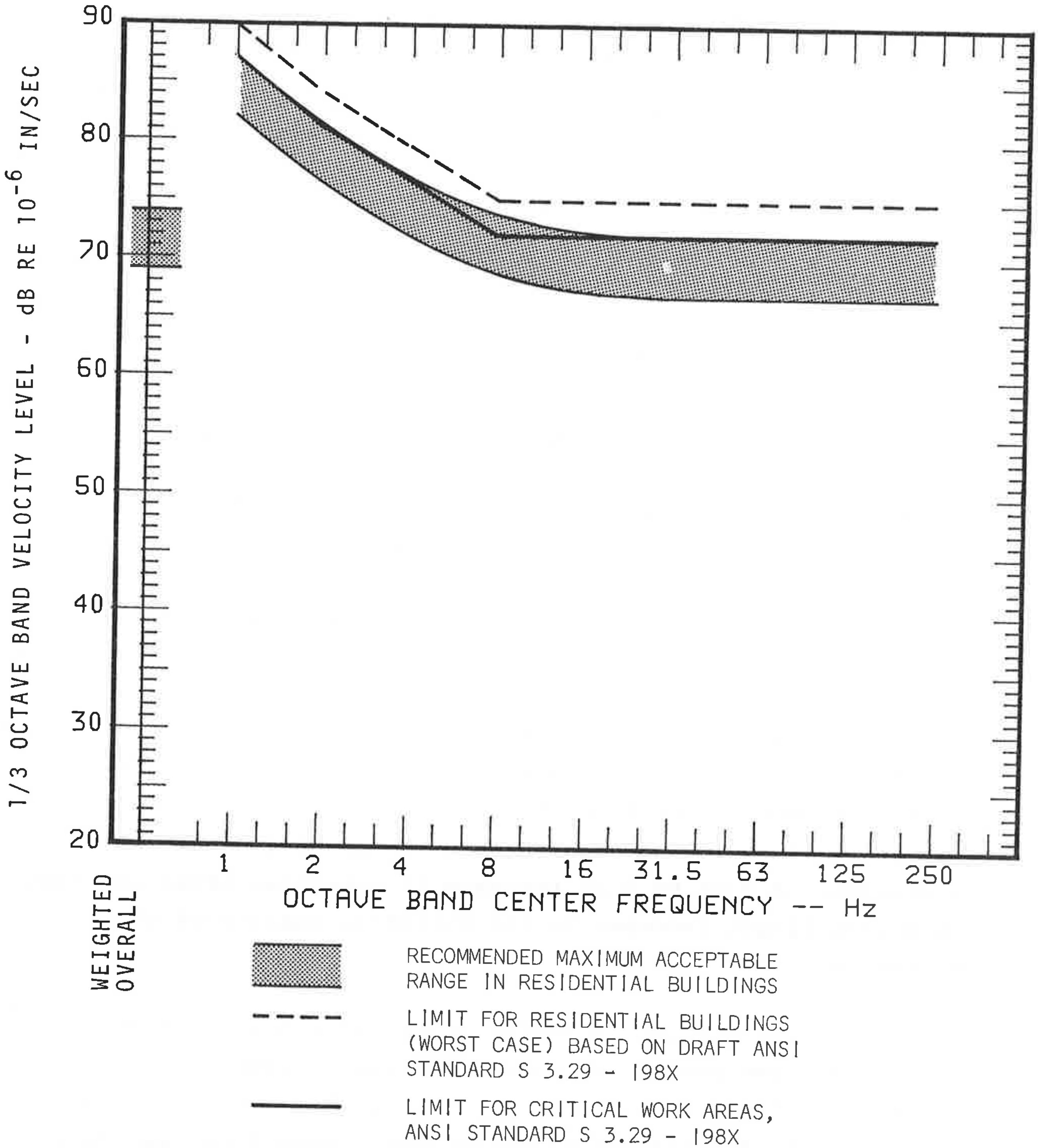


FIGURE 3.9 RANGE OF MAXIMUM ACCEPTABLE TRANSIT INDUCED BUILDING VIBRATION IN RESIDENTIAL AREAS (VELOCITY LEVEL)

in Figures 3.8 and 3.9, the recommended transit criteria are slightly lower than the criteria of the ANSI standard. Although the difference is only a few dB, because of the small difference between acceptable vibration, this difference is significant.

A recent evaluation of community complaints regarding groundborne vibration from WMATA Metro trains provided a unique opportunity to inspect closely the criteria for limits of groundborne vibration. A total of 22 train pass-bys were recorded spanning a two hour period. Two series of measurements were taken on the first floor because during the first series of measurements the resident indicated that the trains were not creating the usual vibration and noise. Both the resident and the personnel taking the measurements agreed that the noise and vibration levels during this series of measurements would generally be acceptable. During the second series of measurements the levels were noticeably higher, reaching the point of being generally unacceptable in a residential structure.

Subsequent analysis of the data indicated a small but consistent difference of only 2 to 3 dB in the vibration levels. The most important observation is that there is very little difference between acceptable and unacceptable levels of groundborne vibration. A 2 dB to 3 dB increase in vibration level can result in a significant increase in the intrusive quality of the vibration.

The general conclusion is that groundborne vibration from new transit systems should not exceed a weighted level of 77 dB to 82 dB re 10^{-8} m/sec (69 dB to 74 dB re 10^{-6} in/sec) and that the 1/3 octave band vibration velocity levels should not exceed the 5 dB range shown in Figure 3.9. The 5 dB range represents a transition region. Above this range complaints are likely; below the range complaints should be rare; and levels within the range are marginally acceptable.

3.2 NOISE CRITERIA

There is considerably less confusion regarding appropriate criteria for acceptable levels of groundborne noise. Although limits for groundborne noise have often been presented in terms of NC curves, it is now more common to specify the maximum acceptable levels in terms of A-weighted sound levels. The guidelines for allowable levels of groundborne noise that are presented in the APTA Guidelines (Ref. A-197) have been successfully applied at several transit systems. A summary of the guidelines is presented in Table 3-6. The general conclusion is that groundborne noise which meets the design goals of Table 3-6 will not be inaudible in all cases, however, the levels should be low enough that no significant intrusion or annoyance will occur (Ref. A-197).

3.3 VIBRATION INDUCED BUILDING DAMAGE CRITERIA

Vibration levels required to produce damage in buildings are generally much higher than those which would be tolerated by humans. The damage potential of buildings varies widely according to type of construction, age, size, the fatigue properties of the building material, and the possibility of structural resonance.

It is important to note different types of building damage. If a building is exposed to extremely high levels of ground vibration, then a building may suffer major damage, such as serious structural damage, glass breakage, and serious plaster cracking, possibly accompanied by falling plaster. A building may suffer

TABLE 3-6 CRITERIA AND DESIGN GOALS FOR MAXIMUM GROUNDBORNE NOISE FROM TRAIN OPERATIONS (REF. A-197)

A. RESIDENCES AND BUILDINGS WITH SLEEPING AREAS

		Maximum Single Event Ground-borne Noise Level		
	Community Area Category	Single Family Dwellings	Multi-Family Dwellings	Hotel/Motel Buildings
I	Low Density Residential	30 dBA	35 dBA	40 dBA
II	Average Residential	35	40	45
III	High Density Residential	35	40	45
IV	Commercial	40	45	50
V	Industrial/Highway	40	45	55

B. SPECIAL FUNCTION BUILDINGS

Type of Building or Room	Groundborne Passby Noise Design Goal
Concert Halls and TV Studios	25 dBA
Auditoriums and Music Rooms	30 dBA
Churches and Theaters	35 dBA
Hospital Sleeping Rooms	35-40 dBA
Courtrooms	35 dBA
Schools and Libraries	40 dBA
University Buildings	35-40 dBA
Offices	35-40 dBA
Commercial Buildings	45-55 dBA

minor damage due to lower levels of vibration. This is typically characterized by fine plaster cracking and the re-opening of old cracks, generally termed as architectural damage.

Review of literature has shown that many investigators have studied the damage risks due to vibration with a somewhat confusing spread of results. It was also noticed that previous investigators did not, in general, distinguish between horizontal or vertical motion when dealing with building damage criteria. A summary of the results of some of these studies is shown in Table 3-7. The wide variation in these results is probably due to the large variation among building structures and the different vibration sources considered, including earthquakes, traffic and blasting.

Note that the results are presented in terms of peak ground velocity because it provides the best correlation. This approach can be partially justified by modelling a complex structure as a single-degree-of-freedom system excited by a vibrating base (Ref. A-50). Under these circumstances, one will find that the strain across the building elements (springs) are proportional to the base velocity in the vicinity of the natural frequency of the system. (This model also shows that in some frequency ranges displacement or acceleration may be more appropriate criteria.)

International Standards Organization draft standard (Ref. A-205) and CHABA report (Ref. C-1) also address the subject of structural damage for building vibration. The CHABA report states that the proposed ISO standard recommends that 6 mm/s (5 mm/s to 30 mm/s for shock) be considered as the upper limit of the threshold of damage to structures. These velocities are considerably lower than the 2 in/sec (50.8 mm/sec) that has been commonly used in the United States. The CHABA report concludes that reducing the

TABLE 3-7 SUMMARY OF BUILDING DAMAGE CRITERIA

PEAK GROUND VELOCITY (mm/sec)	PEAK GROUND VELOCITY (in/sec)	COMMENT	REF.
193.04	7.6	MAJOR DAMAGE TO BUILDINGS (mean of data)	(A-198)
137.72	5.4	MINOR DAMAGE TO BUILDINGS (mean of data)	(A-198)
101.16	4.0	"ENGINEER STRUCTURES" SAFE FROM DAMAGE	(A-199)
50.8	2.0	SAFE FROM DAMAGE LIMIT (probability of damage <5%)	(A-198)
		NO STRUCTURAL DAMAGE	(A-203)
33.02	1.3	THRESHOLD OF RISK OF "ARCHITECTURAL" DAMAGE FOR HOUSES	(A-200)
25.4	1.0	NO DATA SHOWING DAMAGE TO STRUCTURES FOR VIBRATION <1 in. sec	(A-198)
15.24	0.6	NO RISK OF "ARCHITECTURAL" DAMAGE TO NORMAL BUILDINGS	(A-200)
10.16	0.4	THRESHOLD OF DAMAGE IN OLDER HOMES	(A-201)
5.08	0.2	STATISTICALLY SIGNIFICANT PERCENTAGE OF STRUCTURES MAY EXPERIENCE MINOR DAMAGE (including earthquake, nuclear event, and blast data for old and new structures)	(A-202)
		NO ARCHITECTURAL DAMAGE	(A-203)
3.81	0.5 to 0.15	UPPER LIMITS FOR RUINS AND ANCIENT MONUMENTS	(C1) & (A-200)

threshold from 50 mm/sec does not appear to be warranted on the basis of the results reported in the literature. However, the CHABA report recommends a peak velocity of 1 in/sec (25.4 mm/sec) as the threshold of damage to structures. This guideline is based on the results reported in Bureau of Mines Bulletin 656 (Ref. A-198) and is designed to cover all of the data points. Examination of the data reported in the CHABA report, adopted from Bureau of Mines Bulletin 656, reveals that only a small minority of data points are below 1 in/sec peak velocity. The reported data tend to suggest that it is realistic to maintain the criterion for the threshold of damage to structures at 2 in/sec (approximately 50 mm/sec) peak particle velocity. This threshold limit is designed to protect a significant proportion of buildings from structural damage. For vibration sensitive older buildings and special structures, this limit could be lowered by a factor of two, thus bringing it in line with CHABA guidelines.

Paolillo (Ref. A-204) has also reported that the New York City Transit Authority now uses 0.2 inches per second maximum (peak) particle velocity as the threshold of architectural damage for vibration produced by sources other than blasting. The Authority considers 2.0 inches per second to be an acceptable limit for blasting vibration. Paolillo also reports that the level of 0.2 inches per second has been reached in only one of the hundred complaints of subway produced vibration that the Authority has investigated. Therefore, New York City Transit Authority's position is that subway vibration, even in abnormal situations does not exceed the threshold of architectural damage. Note that these criteria are consistent with the general criteria reported earlier.

There is also a German Standard, DIN 4150 (Ref. A-207) which suggests velocities for sudden shocks, (see Table 3-8). DIN 4150

TABLE 3-8 MAXIMUM ALLOWABLE TRANSIENT VIBRATIONS, DRAFT D.I.N. 4150 (REF. A-185)

Class	Description	Max. Resultant Velocity (mm/s)
1	Ruins and buildings of great historical value	2
2	Buildings with existing defects, having visible cracks in brickwork	5
3	Undamaged buildings in technically good condition, apart from minor defects such as cracks in plaster	10
4	"Strong" buildings (for example, industrial buildings in reinforced concrete or steel)	10 - 40

suggests that the velocities should be reduced by one third for sustained vibrations.

Figure 3.10 provides a useful comparison to typical levels of ground vibration near transit system subways and various criteria for human and structure exposure to vibration. It is apparent from Figure 3.10 that any vibration near the limit for structural damage will be distinctly perceptible and may cause vigorous community complaints.

It must be stressed that in the case of old and historic structures, the situation is not very clear as regarding threshold of architectural damage. The level cited earlier as threshold of architectural damage (0.2 in/sec) is probably adequate for historic buildings as a single level, but it cannot account for long-term fatigue damage that may occur after many years of vibration (Ref. A-203). Such fatigue damage has been observed in very old structures (eg., European cathedrals erected in the Middle Ages) although no systematic impact criterion has yet been developed to analyze this problem (Ref. A-207).

It seems appropriate here to mention that the repetition of stresses can result in fatigue damage, and vibration would also cause additional stresses to be superimposed on an existing high concentration of stress, thus "triggering off" a failure.

In summary, it is apparent that the criterion of 50 mm/sec and 5 mm/sec peak velocity are becoming widely acceptable as the threshold of structural and architectural damage respectively. Structural damage is generally defined as glass breakage and serious plaster cracking, possibly accompanied by falling plaster. Architectural damage is characterized by fine plaster cracking and the re-opening of old cracks. These are reasonable criteria for

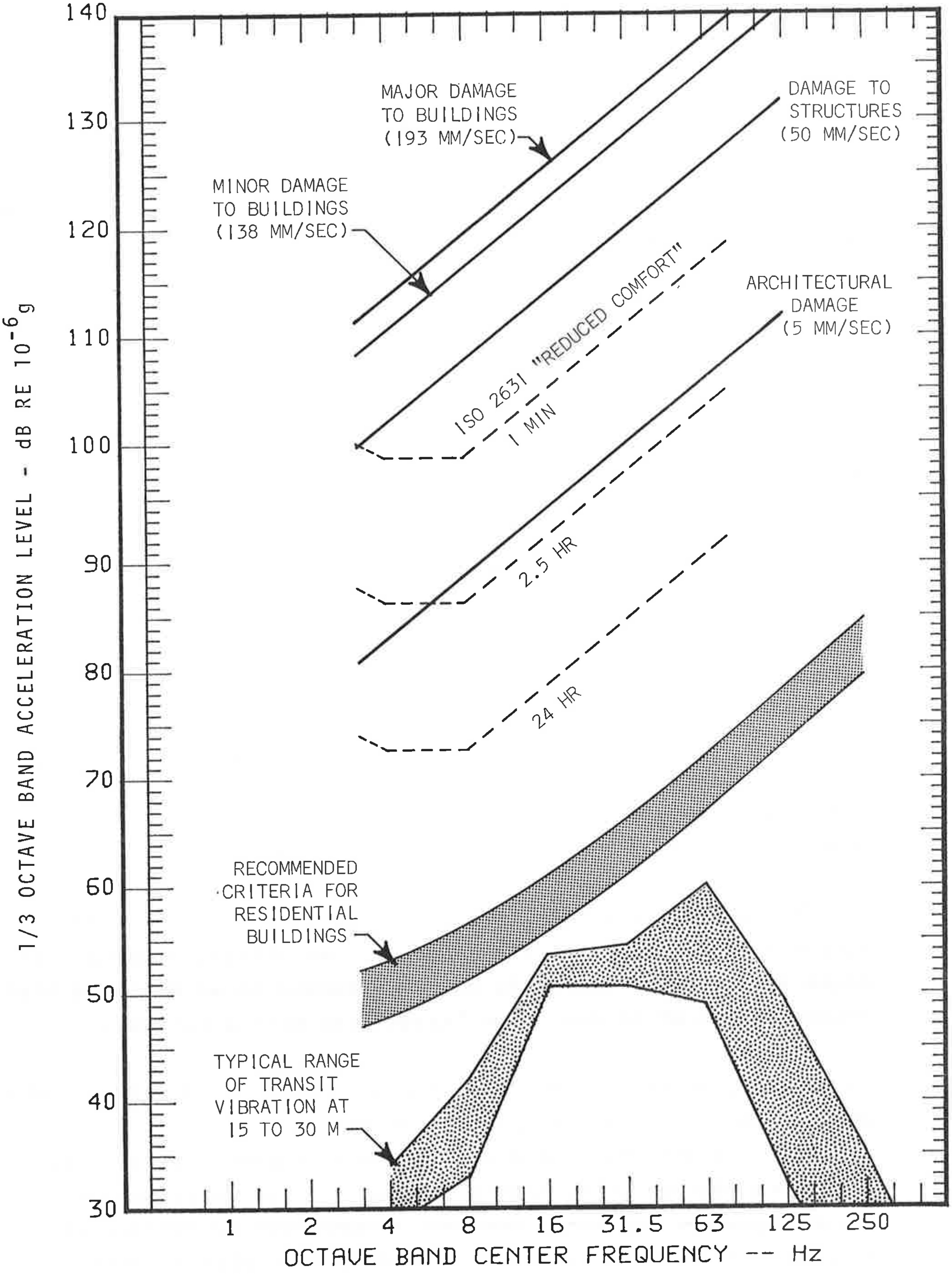


FIGURE 3.10 CRITERIA FOR STRUCTURAL DAMAGE COMPARED TO TYPICAL TRANSIT VIBRATION

preventing structural and architectural damage in virtually all circumstances. Additionally, it must be noted that it is extremely rate that vibration produced by transit trains will even approach the threshold of architectural damage.

3.4 SOIL SETTLEMENT CRITERIA

The literature search has revealed that soil settlement has not been as widely studied as has been the corresponding building damage problem. Gutowski et al. (Ref. A-203) have carried out an extensive literature review in order to determine vibration criteria for soil settlement. The conclusions from this study are that no dynamic settlement in granular soil is to be expected, if horizontal and vertical ground acceleration levels are kept below 0.5 g and 1.0 g respectively. The dynamic settlement refers to densification caused by some immediate, transient excitation such as construction and rapid transit vibration. Dynamic settlement tends to be more of a problem for granular soil, particularly sandy soils (Ref. A-203). This is because the high fraction of pore spaces (high void ratio) allows compression of the mineral skeleton to take place more easily than for highly compacted soils. Additional problems can develop because of the generally large permeability of granular soils.

A recent paper by Dorby et al., quoted in Reference A-204, indicates that very low acceleration (0.05 g to 0.2 g) may cause liquefaction of loose saturated cohesionless soils. Since the soil under existing buildings would not be loose, the high end of the range (0.2 g) would be a conservative criterion. However, it has been reported by Paolillo (Ref. A-204) that acceleration greater than 0.2 g have not been measured from subways, and the New York City Transit Authority's position is that vibration produced by subways will not cause settlement of buildings.

In summary, it can be concluded that horizontal and vertical acceleration levels of 0.5 g and 1.0 g respectively may cause some dynamic soil settlements in granular and sandy soils. However, the vibration produced by subways will not cause settlement of buildings.

4. MEASUREMENT TECHNIQUES

The development and evaluation of methods for prediction and control of groundborne noise and vibration depends strongly on reliable measurement data.

At this time, a considerable variety of measurement techniques and methods of data presentation exist. Techniques range from the use of seismographs for recording and presenting the time history of the vibration velocity data, to the use of 1/3 octave band or constant bandwidth spectral analyzers for a more detailed analysis of the frequency content of the vibration. In Europe, vibration data below 25 Hz is rarely presented, while in the United States the typical analysis bandwidth extends down to 6.3 Hz or lower. Secondly, European data is most often presented in terms of vibration velocity, while in the United States vibration acceleration is the most commonly presented quantity. In most cases, the bandwidth and types of analysis and presentation depend strongly on the type of instrumentation available to the researcher. Thus, the bandwidth of published groundborne vibration data varies considerably. The lower limit of analysis may be anywhere between 3.15 and 63 Hz.

The following discussion addresses the state-of-the-art in measurement and analysis of groundborne noise and vibration from transit systems, and emphasizes the problems or advantages of various measurement techniques, locations, and methods of presentation.

4.1 MEASUREMENT LOCATIONS

A number of locations have been used for measuring groundborne noise and vibration from rapid transit systems. Subway structure measurement locations include the wall, the ceiling, the invert, or a combination of all three. Measurements at the ground surface include over the structure and at a number of locations to the side of the structure. Inside buildings, measurements are often made on the foundation, floors, walls, and ceilings. Although the variety of measurement locations makes it difficult to compare measurement data, the variety has also provided a wealth of data useful for general study of groundborne vibration.

This section identifies those measurement locations which have been used and also those which are most likely to give useful data for comparison and evaluation purposes.

4.1.1 Subway Structures

The UITP has proposed a test code (Ref. A-99) that represents the first international effort towards standardized measurements for subway structure vibration. The proposed test code specifies a total of three measurement locations. They are as follows:

- On the track bed at the centerline of the track.
- On the tunnel wall nearest the running line, 1.20 m above the top-of-rail.
- On the rail web. This is considered an optional measurement location.

For complete tests, additional transducer arrays should be located 20 m from the primary array. Note that the draft code does not specify any measurements on the tunnel ceiling.

The measurement positions actually used when surveying subway vibration are strongly dependent upon the accessibility of measurement locations, the reasons for which the measurements are being made, and the availability of instrumentation. Examples are a measurement program in the WMATA Metro subway tunnels (Ref. A-98) and measurements at the Toronto Transit Commission (TTC) (Refs. A-40 and A-52). The measurement locations used for these and other studies of North American transit systems are illustrated in Figure 4.1. The WMATA study was limited to measurement locations on the safety walk, and on the opposite tunnel bench or the center bench between the trackways. At TTC measurement locations included vertical and lateral vibration on the invert, and vibration on the invert, and vibration normal to the surface of the side bench, wall, and ceiling.

Lang has presented measurement results for triaxial 1/3 octave band vibration velocity at the invert and wall of a circular steel tunnel for various train speeds (Ref. A-25). Additional data are presented by Lang for concrete box tunnel wall and invert octave band vibration velocity levels (Ref. A-45). The results of these measurements indicate substantially similar levels of vibration at both wall and invert locations. Interestingly, the wall vibration levels for the concrete box subway tend to be lower than those for the invert, while exactly the opposite relation exists for the circular steel tunnel. In both cases, the differences are about 9 dB.

Measurements reported by Heckle (Ref. A-49) for a concrete box subway structure with center gallery indicate substantial differences between wall and invert vibration on one side (Track I) and little difference on the other side (Track II) of the subway structure at frequencies above 125 Hz to 1000 Hz. Between 50 and 125 Hz, the 1/3 octave band wall vibration levels are typically 3 to 5 dB less than for the invert. No data are reported below 50 Hz. Additional data reported by Hauck et al. (Ref. A-28) show 1/3 octave band subway wall vibration levels to be about 10 dB lower

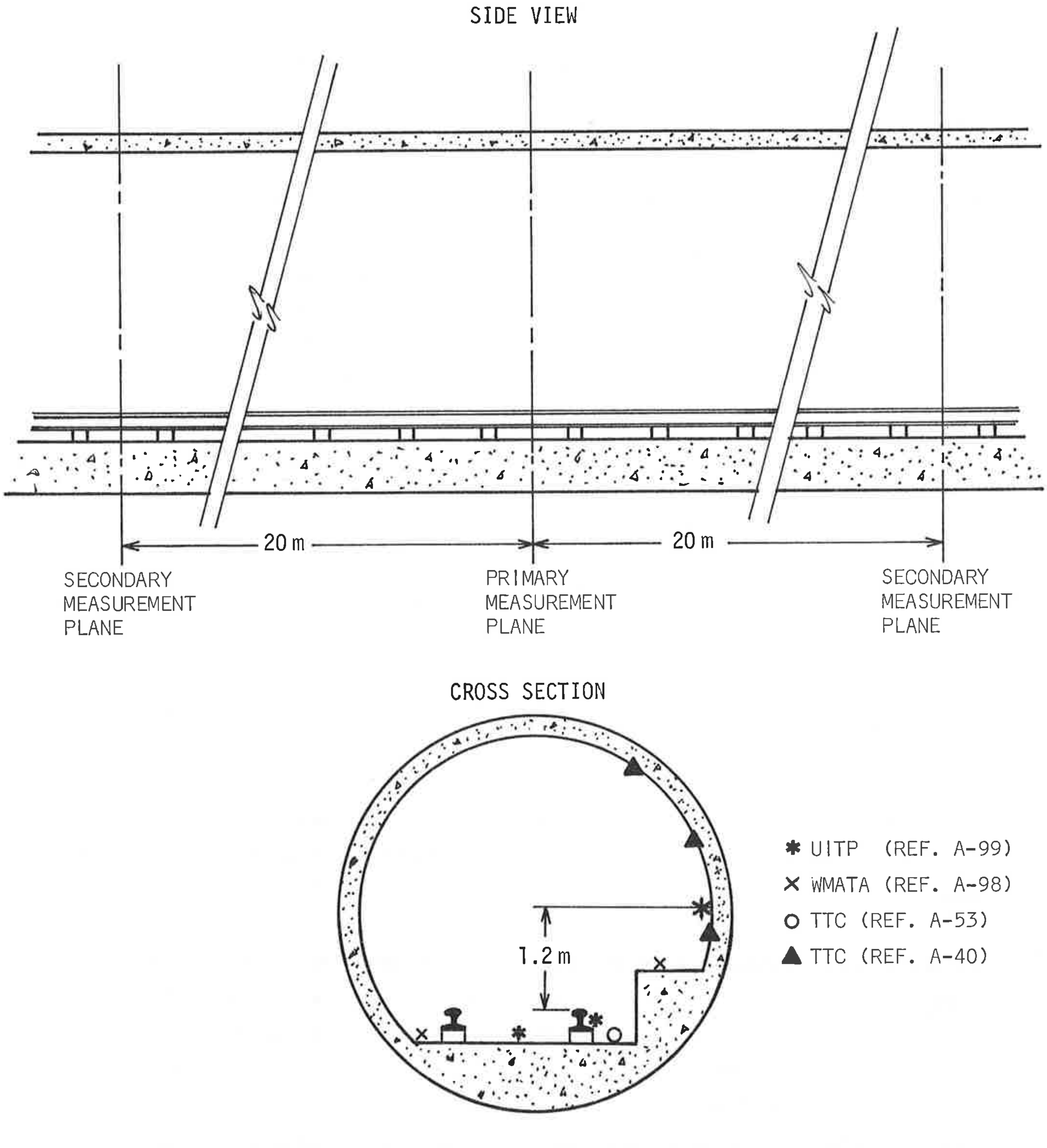


FIGURE 4.1 TYPICAL LOCATIONS FOR SUBWAY STRUCTURE VIBRATION MEASUREMENTS

than the invert vibration levels from 20 Hz to 1 kHz.

A number of subway invert and wall vibration measurements at the TTC system are reported by Bruce et al. (Ref. A-40). These results indicate that wall and ceiling vibration levels are less by about 5 to 10 dB than invert vibration levels at frequencies above 30 Hz. At lower frequencies, invert vibration levels are 10 to 15 dB higher than wall or ceiling vibration levels. Additional data are also given for a circular tunnel. Measurement points were at the tunnel rib near the ceiling, between the walkway and ceiling, and near the walkway, and at a grout pad on the invert. With the exception of the invert data, the tunnel wall vibration data were remarkably consistent at frequencies above 20 Hz. (No data were reported below 20 Hz.) At the invert, however, vibration levels were 15 to 20 dB higher than wall vibration levels at frequencies above 400 Hz. In the mid-frequency range of 20 Hz to 125 Hz, wall and invert vibration levels were very similar.

Of the various locations used to measure subway structure vibration, the subway invert appears to be particularly appropriate. The invert is the most massive and rigid portion of the subway structure, and necessarily participates in the transmission of vibration energy passing into either the soil or other parts of the structure. For floating slab evaluations it is not usually practical to measure on the invert, hence locations on the safety walk or side bench are often used instead. A significant disadvantage of measuring the subway wall is that the wall may or may not be the principal radiator of groundborne vibration.

4.1.2 Aerial/Elevated Structures

No standard measurement locations have been proposed for measuring vibration on aerial structures. The character of groundborne vibration and the geometry of aerial structures indicate that the most obvious measurement point is on the concrete foundation for the support column. Measurement of vertical vibration is the most important, followed by horizontal vibration.

4.1.3 Ballast-and-Tie Embankment

At present there are no standardized measurement locations for ballast-and-tie embankment vibration. Measurement of vertical and lateral ballast vibration, with a spike driven into the ballast midway between the rails, was performed on the TTC system to evaluate various track fasteners, concrete and wood ties, and 100 lb and 115 lb rail (Ref. A-52). Measurement of ballast-and-tie ground vibration is discussed by Hannelius (Ref. A-116) and by Verhas (Ref. A-111). No other literature specifically concerning ballast vibration with respect to groundborne noise and vibration was found. In view of the significant local deformation in the ballast, an additional measurement location at the base of the embankment would be desirable to more clearly represent the vibration source strength.

4.1.4 Ground Surface Vibration Measurement Locations

This section discusses the vibration measurement locations at the ground surface for all transit configurations, described in terms of the horizontal distance from the track centerline at the ground surface. Although the slant distance between the track centerline at the top-of-rail, or geometric structure center, and the measurement point is often used for prediction purposes, the horizontal distance is more readily determined from plan views, and is most closely associated with building locations along the right-of-way.

No standardized measurement locations exist for ground surface vibration. However, measurement locations are generally distributed so as to double the distance from the track centerline for each successive location. Although this progression seems at first glance to be reasonable, the presence of damping in the soil will tend to produce an attenuation rate in dB linearly proportional to distance from the source. This implies that measurement locations should be evenly spaced to accurately assess the effect of absorption in the soil. Note that in the vibration measurements at TTC (Ref. A-52) the measurement location furthest from the track was 240 m. Extending the measurement to 240 m in increments of 15 m would require extensive instrumentation; in such cases an increasing transducer spacing interval is appropriate. In general, at least one location should be at 15 m since this is one of the most often used measurement locations, and is a typical separation distance between buildings and the transit track.

4.1.5 Buildings

4.1.5.1 Vibration -- The selection of vibration measurement points in building structures is more difficult than in ground surface measurements. Different parts of building structures vibrate at different vibration levels. For example, the middle of a floor or wall will vibrate at a higher amplitude than the edge or foundation, and the amplitude will be strongly affected by the dimensions of the floor or wall.

Both wall and floor vibrations in residential structures were measured as part of a detailed series of measurements of groundborne noise and vibration in Toronto (Ref. A-52). Earlier measurements of cellar wall and floor vibrations near a TTC subway structure are reported by Bruce et al. (Ref. A-40). The measurement locations used were in the middle of the floor or wall panel, or at least 2 ft from the edge. Only vibration normal to the surface was measured. At the WMATA Metro system, foundation vibration was measured simultaneously with ground surface vibration (Ref. A-98) at a few isolated locations. In these cases, the transducer was mounted either directly on the foundation, or on a convenient basement window sill. Heckle (Ref. A-49) describes measurement locations and results for building structures near subway lines, including pile foundation vibration. Data are also presented by Lang (Ref. A-45). Hannelius (Ref. A-116) describes detailed vibration measurements on structures in Sweden using seismographic techniques. These measurements include vertical and horizontal vibration measurements at the foundation, and on upper floor exterior walls.

To completely characterize the vibration of a structure as complex as a building would require a very large number of measurement locations. Rarely is such detail required. For evaluation of occupants' complaints it is often only necesary to measure at one or two locations near the center of a floor span. For a more detailed evaluation of a specific building the measurements can usually be limited to:

- Foundation, at the point nearest the transit structure.
- Middle of the basement floor slab.
- Middle of the floor of the 1st floor room nearest the transit structure.
- Middle of the floor of the 2nd floor room nearest the transit structure.
- Additional locations as deemed appropriate.

In most cases, only the vibration normal to the measurement surface need be measured. A more complete test program would include vertical and transverse foundation vibration, and additional measurements on wall surfaces.

4.1.5.2 Noise -- Measurement of groundborne noise in buildings is usually made with a single microphone inside a room. The typical height of the microphone is 1.5 m above the floor, corresponding to the average height of the ear of a seated person. Locations close to walls should be avoided. Measurement of noise is usually performed in the same rooms used for vibration measurements with both measurements performed simultaneously.

A thorough measurement of groundborne noise in a room would include space averaging of the energy density of the noise field, together with a determination of the average absorption coeffieient for the room by standard reverberation time measurements. This measurement would be similar to that used for evaluating the transmission loss of architectural partitions. However, groundborne noise from transit systems is necessarily a transient event, which does not allow suitable time for operating a rotating microphone boom, such as is done for transmission loss measurements.

An alternative to the rotating boom technique is to use multiple microphones, recording each microphone signal individually for analysis and energy summation. Electronic mixing of the microphone signals should not be used. This is because groundborne noise from transit systems includes low-frequency noise components whose wavelength is comparable with room dimensions, thus producing coherent signals between each microphone, which can not simply be mixed to determine the mean sound energy in the room.

Another alternative is to measure groundborne noise with a single microphone at three or four locations evenly spaced on a diagonal that runs from one corner of the room through the geometric center to the opposite corner. In this method, noise from at least two train passbys should be measured at each location, after which the microphone is repositioned on the diagonal. Even if noise from only one train is measured at each point on the diagonal, this method is more desirable than using only a single measurement location in the room. Between train passbys, reverberation time measurements can be made for each measurement location, using a clapper board or starting gun. Through this procedure, the sound energy entering the room per unit wall area can be determined in accordance with the methods of sound transmission loss measurements (Ref. C-33). This method may correlate more closely with building vibration measurements and remove some of the scatter in groundborne noise data, thus improving the basis for prediction procedure development.

4.2 TRANSDUCER MOUNTING TECHNIQUES

In most cases, the mounting technique will determine the response characteristics of the measurement system, and thus the quality of vibration data. Following is a discussion of methods for mounting transducers on the soil surface. These methods are designed to maximize the ratio of base support area to total mass and retain dimensional compatibility with the minimum wavelengths associated with ground vibration. Mounting transducers on subway structures is not discussed. The structure surfaces are sufficiently rigid to guarantee adequate frequency response.

4.2.1 Types of Mountings

A variety of transducer mountings have been proposed and/or used for measurement of ground surface vibration and may be classified as:

1) Plate resting on or embedded in the soil surface.
2) Sidewalk or asphalt surface.
3) Spike.
4) Buried container with overall density matched to soil.

4.2.1.1 <u>Mounting Plate</u> -- A plate resting on the soil surface has the advantage of being easily modeled as a circular disc resting on an elastic half space. Thus, mathematical solutions are available to assess transducer response characteristics. The theoretical response of a massive disc resting on an elastic half-space has been studied by Arnold (Ref. C-32) and by Bycroft (Ref. C-33). The horizontal resonance frequencies for coupled sliding and rocking of a vibration pick-up resting on a soil surface is evaluated by Omata, et al. (Ref. B-1). Finally, the response of idealized rigid circular foundations resting on a half-space is discussed in a number of texts (Refs. B-8, B-4).

A specific mounting plate design for measuring ground surface vibration from a rapid transit system was recommended in Reference A-54. The plate consists essentially of a 1 to 2 cm thick aluminum disc with a diameter of approximately 15 cm. The plate is drilled and tapped to receive an accelerometer transducer. The plates may also be set in plaster on the soil surface, or at the bottom of a small pit. Verhas (Ref. A-111) used this technique and experimentally determined the resonance frequency to be 465 Hz, indicating that extremely good coupling with the soil surface was achieved.

The use of a lightweight aluminum plate placed in plaster maximizes the ratio of base contact area to assembly mass. The plaster thickness must, of course, be as thin as possible, but capable of providing intimate contact between plate and undisturbed soil.

Generally, the physical size of the transducer mounting plate must be less than 1/2 the expected minimum wavelength for the plate to follow the crests and troughs of Rayleigh waves. At 200 Hz, the typical Rayleigh wavelength is about 1 meter. Thus, the diameter of 15 cm is sufficiently small.

Excavating a 1 to 2 foot deep pit exposes relatively stiffer soil for mounting. However, one must consider that the amplitude of Rayleigh waves at 100 or 200 Hz varies significantly with depth, and a significantly lower vibration level would be measured for this type of wave at two feet below the soil surface at these frequencies. At lower frequencies, the effect of depth is less important.

Experiments with the use of a small plate embedded in plaster of Paris have shown that there can be problems if the mount is not carefully prepared. Generally it has been found to be easier and to provide more reliable information if the accelerometer is mounted on a sidewalk slab or an aluminum stake.

4.2.1.2 Sidewalk Slab -- Many transit system groundborne vibration measurements are made with accelerometers mounted directly on sidewalk or asphalt surfaces using wax. Accelerometers mounted on sidewalk slabs have a relatively high ratio of base contact area to slab mass, in spite of the concrete density, and maintain intimate contact with the soil. Experience with measurements on sidewalk slabs and on asphalt surfaces is that reliable data can be obtained up to a frequency of 200 to 400 Hz.

The main reasons for using sidewalks for mounting accelerometers are ease of use, the accessibility, and the consistency of the data. In urban areas they are often the only practical location for ground surface measurement. For fast, practical vibration measurements, a sidewalk surface is the best mount for vibration measurements.

4.2.1.3 Spike -- A convenient transducer mount is a spike, drilled and tapped for mounting, driven into the soil surface. The length and mass of the spike will determine its response to incident ground vibration. Since most practical spikes are on the order of 0.2 to 0.3 m long, the size of the spike will have little effect on its response to the slowest types of waves, e.g., Rayleigh waves, below about 200 Hz. Secondly, proper spike design will result in a high ratio of contact area to mass. Unfortunately, driving the spike into the soil necessarily disturbs the soil, sometimes resulting in incomplete or inconsistent contact with the soil. This possibility is particularly important when measuring horizontal vibration, as the spike acts as a cantilevered beam with a mass on the end.

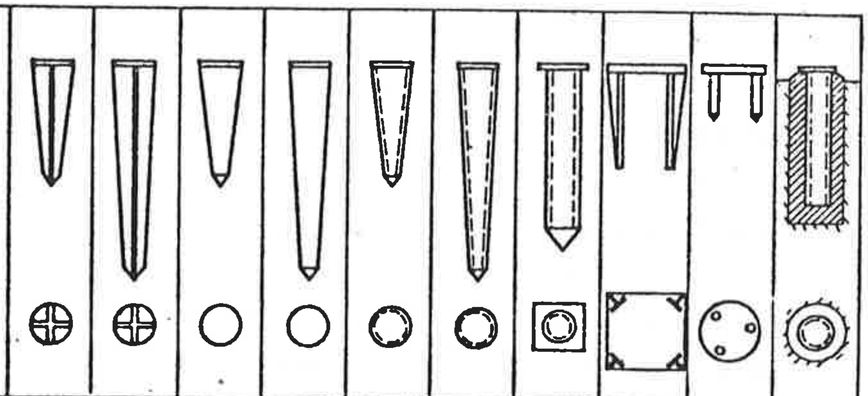
The best spike design maximizes the surface contact area relative to the mass. Several cross-sectional patterns provide good contact and are easily driven. An "X" pattern is one that has been successfully used. Nolle indicates (Ref. A-102) that the spike should have a slight taper of about 1:10 to 1:22, depending on soil type. For brittle, dry, cohesionless soils, "the taper used should be a maximum that still permits driving the peg without bounce or deep cracking of the soil." In wet, heavy clays, the taper has little effect. A distributed platform mount with short spikes about the periphery will also improve horizontal frequency response. The mounted resonance frequency for vertical response was found by Nolle to be between 200 and 500 Hz, depending on spike design and soil type.

4.2.1.4 Buried Container -- Enclosing the transducer in a sealed container with overall density similar to that of the soil, and burying the assumbly is a theoretically desirable technique. The size of the box must be significantly less than one wavelength, which is not difficult to achieve. Because of the density equivalence, the box will not significantly scatter low-frequency ground vibration waves and thus becomes an optimum transducer mounting. A problem with this approach may be that the soil is necessarily greatly disturbed, and the backfill must be watered to achieve compaction. Moisture may then interfere with transducer cabling, a problem that is particularly significant with piezoelectric accelerometers. Geophones, or velocity pick-ups, with their very low output electrical impedances are less prone to this problem. In any case, this technique is probably only practical when extreme accuracy or measurements below the soil surface are required.

4.2.2 Performance of Various Transducer Mounts

Measurements of the relative performance of several transducer mounting configurations are documented by Gutowski et al. (Ref. A-50) and Nolle (Ref. A-102). Gutowski compares stake mounts with the potentially ideal circular plate mount. Experimental data indicate that short stake mounts are comparable to circular plate mounts for vertical vibration up to about 100 Hz. However, above 100 Hz, incomplete coupling of the circular disk causes the disk to perform badly (resonate) above 100 Hz. The disk was not set in plaster, as described by Verhas (Ref. A-111). Therefore, in the absence of plaster as a coupler, short stakes with large contact surface are preferable.

Nolle reports measurements of the relative responses of mounting stakes of various cross-sectional designs. The results of Nolle's investigations are presented in Figure 4.2. Many of the spikes



Base Design No.		1	2	3	4	5	6	7	8	9	10
Heavy clay	H	+300	+360	+620	-660	+420	-780	-820	-620	+250	
	V	-580	-800	-660	-700	-800	-940	-650	-450	-520	
Sandy loam	H	-180	+ 80* -140	-260	+ 60* -180	-140	+110* -180	+ 75** -180	-160	+100	
	V	-180	-180* -320	-130	-190* -220	-140	-230* -300	-150** -230	-180	-140	
Sand (wet)	H						- 80		- 65		
Sand + loam mix (dry, hard)	H					+400 +170† +110††					
Mudstone	H										-1500
	V										>2000

* Top 125 mm above ground
** Top 100 mm above ground

† Bond damaged
†† Bond badly damaged

H and V designate response to horizontal and vertical forcing respectively. Construction details for bases are (dimensions are in mm):

1. 10 x 50 dia. top plate, 5 x 150 long tapered plate welded to form + section.
2. As per 1 above, 300 long.
3. 50 dia. x 150 long solid cone.
4. 50 dia. x 300 long solid cone.
5. As per 3 above, hollow.
6. As per 4 above, hollow.
7. 40 x 230 long, parallel hollow cylinder.
8. 10 x 115 square top plate, 5 x 40 x 150 T section legs.
9. 15 x 100 dia. top plate, 12 dia. x 65 legs on 80 P.C.D.
10. 20 dia. x 180 long capped tube in non-shrink grout.

FIGURE 4.2 LINEAR FREQUENCY RESPONSE LIMITS (+ OR - 3 dB POINTS) OF VARIOUS BASE DESIGNS IN DIFFERENT GROUND [FROM NOLLE, REF. A-102] gave reliable frequency response characteristics up to 200 Hz for most soils, but spikes driven into wet sand generally had very low horizontal resonance frequencies, on the order of 65 to 80 Hz. Not all spikes were tested with all soils, but the implication is clear that consideration of soil type should influence the selection of a spike design. Horizontal resonance frequencies are lower than vertical resonance frequencies for all spike designs. One can have reasonable confidence in the frequency response up to at least 150 Hz for sandy loam soils, typical of many top soils. In heavy clay, the bandwidth may easily extend to 500 Hz. Nolle suggests that the best results are obtained if the spike is driven the full length into the soil.

Recent measurements have been performed at the BART system to evaluate the differences in response between spikes, plaster of Paris pads, and sidewalk or asphalt surfaces. For the plaster of Paris pad, loose soil and vegetation were removed prior to pouring. A 4 cm^2 aluminum block was set in the plaster of Paris to support the accelerometer.

Summary results for the three mounting techniques are presented in Figure 4.3. Below 125 Hz the three techniques provided essentially the same result. Above 125 Hz the levels measured on the sidewalks are 2 to 7 dB lower than the results with the aluminum stakes. In contrast, above 125 to 200 Hz the plaster of Paris mounts exhibited distinct resonances. Using more care with the preparation of the plaster of Paris pad reduced the resonance effect.

The conclusions is that the added complexity of using the plaster of Paris pad in addition to the problems of obtaining adequate contact with the ground make it a less desirable method of attaching an accelerometer to the ground surface. Either an aluminum stake or a concrete or asphalt surface is preferable.

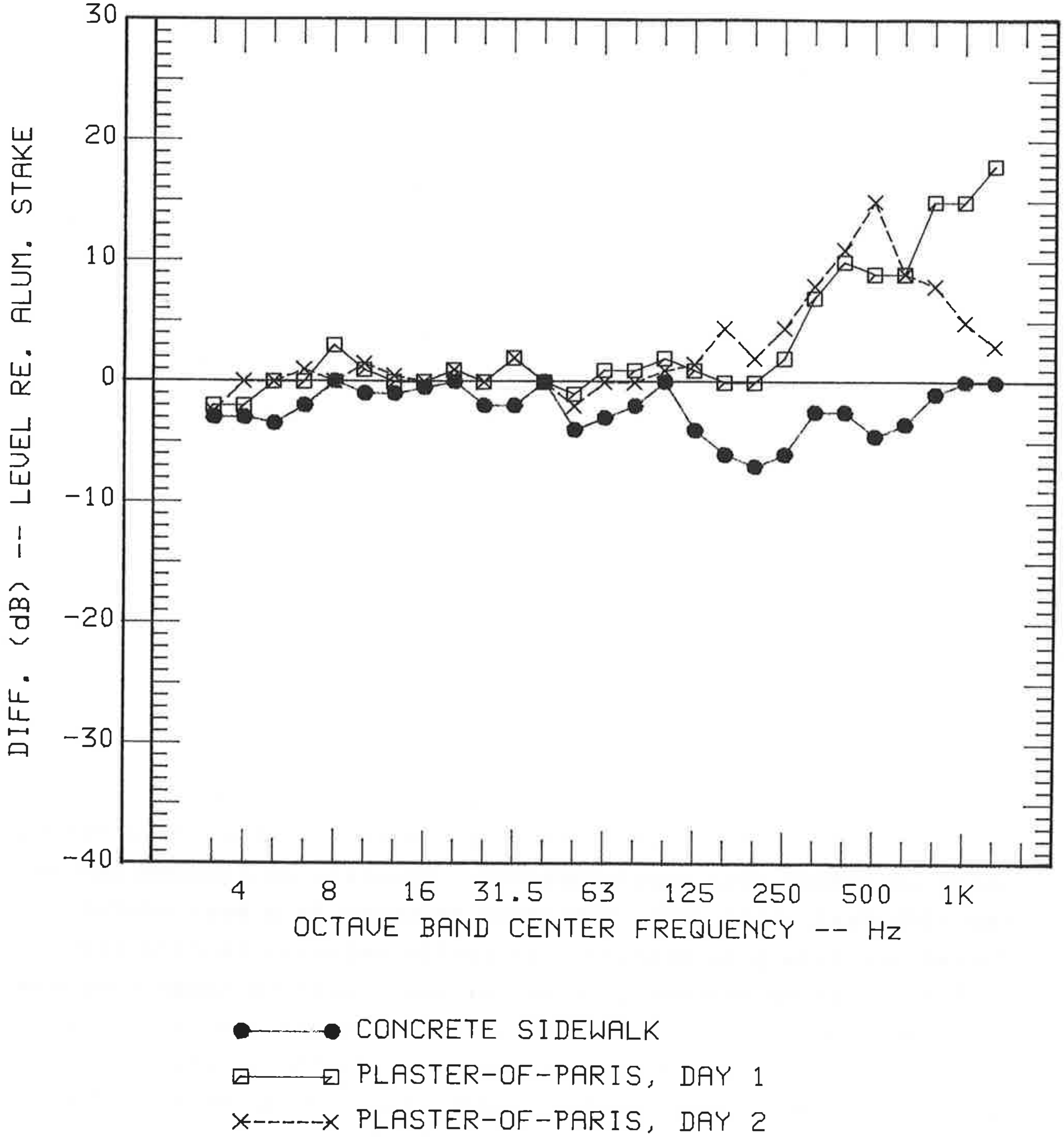


FIGURE 4.3 RELATIVE RESPONSES OF VARIOUS TRANSDUCER MOUNTINGS EVALUATED AT BART

4.3 DATA PRESENTATION

One of the difficulties associated with research in this field is the variability in methods for quantifying and presenting measurement results. Standardized procedures for data presentation are entirely lacking. The UITP is attempting to develop an international test code for subway structure vibration measurements (Ref. A-99). The draft code specifies that overall vibration data be presented in terms of vibration velocity levels relative to 10^{-8} m/sec. If accelerometers, rather than velocity pick-ups, are used for data acquisition, then the code allows the use of analog integrators to convert analog acceleration signals to analog velocity signals. The internationally standardized 1/3 octave filter networks are specified for presentation of spectral data. The tentative bandwidth for vibration measurements is specified from 10 Hz to 1000 Hz; however, extension of the lower limit to 3.15 Hz is under consideration. The main objection to extending the measurement bandwidth to 3.15 Hz is that the low frequency limits of most sound level meters, and other instruments used by acousticians, are in the neighborhood of 10 Hz.

In the U.S. most vibration data is presented in terms of acceleration level. This is largely a result of the measurements being performed with accelerometers. However, as recommended by the UITP draft test code, vibration velocity is a more useful format for data presentation. Vibration velocity is directly related to sound pressure, is better correlated to human response (over most of the frequency range) than acceleration, and is better correlated to structural damage. Another factor is that geotechnical engineers usually report vibration velocity. The conclusion is that reporting vibration velocity level has significant advantages over acceleration level.

4.4 NARROW BAND ANALYSIS - FFT

In the past ten years the use of equipment that utilizes the Fast Fourier Transform (FFT) algorithm to perform frequency analysis has become very common. The technique has the advantage of generating a narrow band frequency analysis almost instantaneously. It is very useful for identifying pure tone signals, periodic signals, highly resonant oscillation, and identifying vibration sources. It has not been used extensively for evaluation and presentation of groundborne noise and vibration. However, the FFT analyzer can certainly be a valuable tool in the analysis of noise and vibration problems.

4.5 CROSS-CORRELATION OF GROUND VIBRATION SIGNALS

Determining the velocity of propagation of ground vibration from transit systems can help to understand the nature of groundborne vibration, i.e.; whether the vibration consists of shear, Rayleigh, or compression waves. One method for such determination is cross-correlation between two analog vibration signals, obtained from separated transducers, one closer to the source, on a line between the source and the second transducer. The theoretical and practical application of cross-correlation is described in general texts (Ref. C-4). The application of cross-correlation to soil dynamics is discussed in detail by S. Roesler (Ref. B-3), who found the method useful for measuring both group and phase velocities, detection of anisotropy in soils, and evaulation of the effect of shear, compression, and Rayleigh waves. The method is particularly useful in the presence of high background vibration, and for measuring over long distances.

Cross-correlation techniques have seen only limited application to groundborne vibration from transit systems. Verhas used it to determine the velocity of ground vibration propagated from at-grade ballast-and-tie track (Ref. A-111). He found the velocity to be about 184 m/sec, at both 14 and 70 Hz. Agreement at these two widely separated frequencies indicates that the same basic wave type may be responsible for groundborne vibration from at-grade track over a wide frequency range. From this propagation velocity, Verhas deduced that Rayleigh and/or shear waves were the primary carriers of vibration energy from at-grade track structures. Cross-correlation of groundborne vibration from a subway structure provided Rucker (Ref. A-58) with a measurement of the shear and compression wave velocities for determination of the shear modulus and Poisson's ratios for the soil surrounding the subway structure. This information was then used in a finite element model of the subway structure and soil to determine the input power spectrum of the running trains.

Cross-correlation is useful for determining propagation delay times, which may be used in turn to infer propagation paths and identify vibration sources. Such a technique might thus be useful for identifying the most significant vibration-radiating portions of a subway structure. Roesler discusses the successful application of the technique to void detection between the Hamburg road tunnels and soil overburden (Ref. B-3).

4.6 IMPEDANCE/TRANSFER FUNCTION MEASUREMENTS

The development of two-channel FFT analyzers has made the measurement of impedance and transfer functions much more practical than in the past. Such measurements are being widely applied in the experimental analysis of complex structures and are fundamental to the various modal analysis systems that are now available.

Measuring the driving point impedance of the subway invert (or aerial structure roadbed) could yield information valuable for the design of direct fixation fasteners and floating slab vibration isolation systems. No such measurements have been performed to date for study of groundborne noise and vibration. Such measurements, from the standpoint of fastener design or floating slab design, would be significant with respect to aerial structures.

Transfer mobilities between source and receiver location are transfer functions (Ref. C-6) relating the response at the receiver location to the input force at the source as a function of frequency. Typically transfer mobilities are measured using impact excitation at the source location and a vibration transducer at the measurement location. Using a special hammer instrumented with a force transducer, transfer mobilities can be measured in an efficient, rapid manner. The technique can be applied to the study of:

- The mechanism of vibration radiation from a subway structure.

- The vibration transmissibility of various components in the transmission path between wheel, rail, and building wall.

- The effect of different installations and designs on groundborne noise and vibration.

- Parametric dependence of track and subway structures.

There are a number of advantages associated with using impact type testing to evaluate transit facilities. Some that particularly apply to groundborne noise and vibration are:

- Direct measurement of transfer mobilities is possible.

- Effects of different components in the vibration transmission path can be more easily determined.

- Direct comparisons of different conditions can be performed. The advantage is that a comparison can be achieved that is not dependent on the type of rolling stock, the condition of the rail running surface, the condition of the train wheels, or whether the track is jointed.

- Only short lengths of test track are required for an experimental assessment.

- Since there is no need to run test trains, large quantities of data can be collected and analyzed.

- The characteristics of vibration transmission can be studied.

- Most mathematical models of track forms are based on discrete degree of freedom systems with a single input. This method will provide information much more useful for inclusion into these studies.

- Effects of extraneous vibrations can be reduced.

Although the impact method of vibration testing is well proven for many types of structure, its application to railway vibration investigation requires careful consideration and development. The method of testing used must give results that relate closely to those obtained when service (or test) trains are running. A series of initial tests are planned by London Transport at Aldwych. At this site three different track forms are to be laid and the impulse method results will be compared with results from test trains for all three.

A particular area of uncertainty is the effect of the train dead load on the non-linear characteristics of many track support structures. A further set of tests is planned by London Transport to examine this relationship. Other areas requiring investigation are repeatability and sensitivity to positioning of transducers, although initial indications are that these latter two factors should not cause any problems.

While the value of vibration measurement during the passage of traffic over a track support structure is well appreciated, the cost of this exercise can be very great. Generally, at least one train's length of a system under investigation must be prepared at a location where sufficient straight track is available to allow a full range of train speeds. Where several track structures are to be compared, the requirement for them all to conform to this condition and also to be independent of other parameter variations (e.g., tunnel structure, local variation in terrain, different rolling stock) further compounds the practical problems and the cost of testing. It is anticipated that an impact method of testing will make installation of such elaborate test facilities unnecessary.

4.7 MECHANICAL VIBRATION INPUT POWER MEASUREMENT

One of the fundamental quantities that characterize vibration and sound is the vibration input power delivered to a dynamic system. No such measurements have been performed for groundborne vibration; however, Rucker inferred the input vibration power spectrum for trains running in subways by using measurement data and a finite element model of the structure and soil (Ref. A-58). Ottesen et al. (Ref. C-2) measured the vibration power delivered to wall structures with apparently good success.

Input vibration power spectra may be valuable for validating models of transit structure and soil interaction and might be used as an input to groundborne noise and vibration prediction procedures. Vibration power measurements may also provide direct comparison of the performance characteristics of various fastener designs, truck designs, and structure types.

The vibration power transmitted through the fastener into the trackbed must be measured with an instrumented fastener, and by velocity transducers mounted on the invert near the fastener. The transmitted vibration power spectral density will be the real part of the cross-power spectral density between the fastener force and invert velocity signals. Such measurements have not been performed. Indeed, suitable instrumenting of a fastener will be a difficult task for power measurements at audio frequencies.

4.8 GEOPHYSICAL METHODS

Geophysical methods, or seismic methods, are a group of measurement techniques used to determine the sub-surface characteristics of earth media. They are well known in the fields of civil engineering, mining, and geophysics and are well described in the literature (Ref. B-3, B-4, B-22, B-23). They generally involve artificial excitation of the soil and measurement of the resulting soil response at several points removed from the source.

The parameters which can be evaluated with these methods include:

- Shear and compression wave velocities.
- Soil layer thicknesses.
- Depth of overburden.
- Q-factor (damping) of soils.
- Depth of the water table.

Geophysical methods can be grouped under four basic headings:

- Down Hole/Up Hole
- Cross-Hole
- Seismic Refraction
- Continuous Sine Wave Excitation

The first two methods involve boring a hole or holes and placing geophones in the hole. In the down hole/up hole method, geophones are placed at various depths below the surface, and impulsive excitation at the surface, either shear or compression is used to determine propagation velocities as a function of depth. In the cross-hole method, the excitation can be applied at various depths within the source hole; similarly, receivers can be located at various depths within the receiver hole. Methods of excitation and placement of transducers can be difficult, and often require special equipment.

The seismic refraction method is the simplest to perform because it does not require test holes. The source may instead be a hammer or explosive charge. The arrival time of the resulting compression wave is then measured at several distances along a straight line from the source, with single or multiple transducers and a single- or multi-channel seismograph. The best results are obtained if the seismograph is of the signal-enhancement type which can average repeated impulses and display the result in a stacked fashion. Standard algebraic techniques are then applied to determine the compression wave speed within soil layers and the depth of the soil layers (Refs. B-4, B-22, B-23). The seismic refraction method is almost always used to determine compression wave velocities. Some workers have used the method for identification of shear wave velocities (Ref. B-23).

The final method is to use continuous sine wave excitation at a point on the soil surface and measure the far-field wavelength of vibration, also at the soil surface. As the frequency is lowered, the wavelength of the Rayleigh wave becomes longer, and at the same time the Rayleigh wave excitation extends to greater depth. If the propagation velocity of the Rayleigh wave is a function of the average soil parameters over the effective depth of the wave, the soil properties as a function of depth may be determined. One advantage of this method over the impulse methods described above is that highly selective filters may be used to improve the signal-to-noise ratio for the receiver data. The major disadvantage is that a shaker system, or other relatively massive exciter, must be used.

Geophysical techniques are now at a relatively advanced stage of development, and should be considered in the evaluation of soil properties for prediction of groundborne noise and vibration propagation. However, the techniques are not necessarily easily performed, and considerable expertise is required in both taking and interpreting the data.

5. VIBRATION CONTROL TECHNOLOGY

A variety of options are available to control groundborne vibration from rapid transit systems. Almost all of these options can be subdivided into the following classifications:

- Wheel/Rail Maintenance
- Transit Vehicle Design
- Rail Support Design
- Floating Slab Vibration Isolation
- Ballast Mats
- Subway Structure Design
- Location of Way
- Screening
- Building Isolation

This section will be devoted to discussion of each of the various vibration technologies with respect to the above classifications.

5.1 WHEEL/RAIL MAINTENANCE

A primary method for control of groundborne noise and vibration is the maintenance of the wheel and rail contact surfaces in smooth and uniform condition. Such maintenance is effective primarily on those systems with continuous welded rail. Rail joints can mask many benefits of rail grinding and wheel truing.

Measurements at the SEPTA system indicate that vibration with trued standard wheels is approximately 6 to 10 dB lower than with worn standard wheels above about 100 Hz. The vibration reduction obtained by rail grinding on the SEPTA system was less significant than for wheel truing, but was still about 4 to 8 dB above about

100 Hz on the subway structure and in a nearby building (Ref. A-9).

Measurements at BART indicate a 5 to 15 dB reduction of platform vibration in the 8 to 1000 Hz frequency range after rail grinding (Ref. A-173). However, the BART rails prior to grinding were newly placed and still had mill scale and other manufacturing irregularities. Additional measurement data are presented in Reference A-52. In all cases, the results are a clear indication of the importance of regular rail grinding and wheel truing for control of groundborne noise and vibration on modern systems which use continuous welded rail.

One of the current problems associated with assessing the quality of rail grinding and wheel truing concerns the lack of a reliable roughness measuring apparatus. Such a device was developed by Remington, Rudd, and Ver (Ref. A-77). Refinement and further validation of the measurement technique may provide a means for quantifying wheel and rail roughness in a manner which can allow correlation of groundborne noise and vibration data with rail roughness data.

5.2 TRANSIT VEHICLE DESIGN

Redesign or modification of the transit vehicle truck can be an effective control measure for groundborne vibration. Design considerations include reducing the dynamic load on the rail (by decreasing primary journal spring stiffness, providing flexible frames, or otherwise reducing the effective mass), and including damping devices to absorb vibration energy (for example, resilient wheels, axle mounted dynamic absorbers). This section discusses components in the truck design than can affect groundborne vibration. The effect of the wheels is discussed in the next section.

The primary journal stiffness has recently been identified by Paolillo (Ref. A-11) and Wilson (Ref. A-174) as a factor contributing to excessive groundborne vibration at 20 to 25 Hz on the NYCTA, WMATA, and MARTA systems. A primary stiffness resonance frequency in the range of 20 Hz is close to the design resonance frequencies of floating slabs and typical floor and/or wall resonance frequencies of building structures. Thus, redesign of the primary journal spring to lower the resonance frequency from 20 to 25 Hz down to about 10 Hz will substantially reduce groundborne vibration in the 16 to 30 Hz range.

Comparisons by Wolfe (Ref. A-7) and Keevil (Ref. A-175) of vibration produced by CTA vehicles (2400 Series vs. 2000 and 2200 Series) strongly demonstrate the effectiveness of reducing the dynamic load impedance for vehicles operating on continuous welded rail on ballast-and-tie track. Wolfe's measurement data are summarized in Figure 5.1. The vibration differences between the vehicles are less for aerial structure operation with jointed rail than for ballast-and-tie operation; this is probably a result of the greater flexibility (and reduced rail input impedance) of the aerial structure. The impacts at the rail joints could also contribute to the differences on the aerial structure being less dramatic.

The 2400 Series vehicles use Wegman trucks that are a very different design than the LFM-Rockwell and Budd Pioneer III trucks used on the 2000 and 2200 Series vehicles. The primary suspension resonance frequency of the Wegman truck is about 10 Hz, much lower than the 20 to 50 Hz primary resonance frequency of the Rockwell truck used on the NYCTA, WMATA, and MARTA systems. Note that the

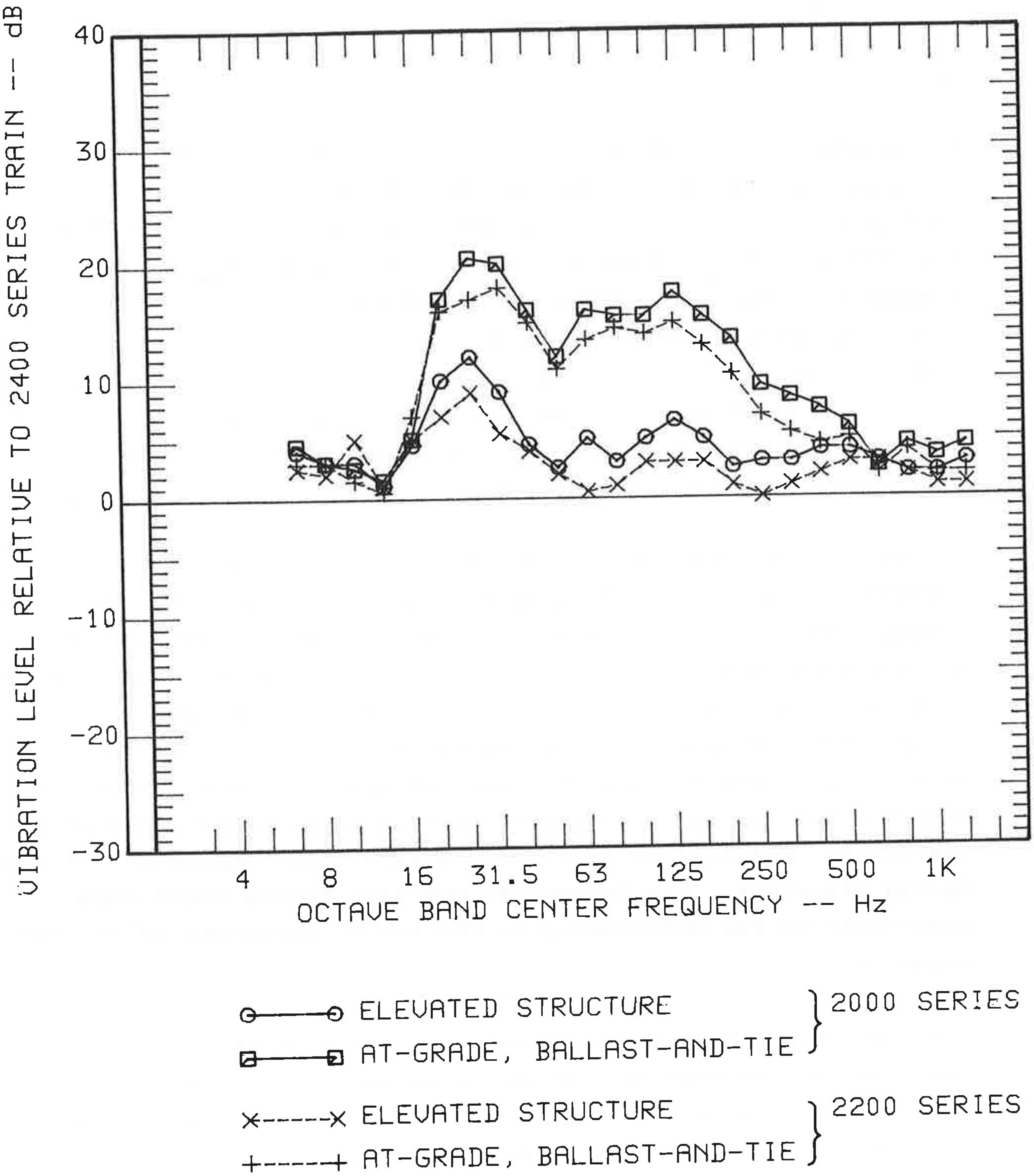


FIGURE 5.1 RELATIVE LEVELS OF GROUND SURFACE VIBRATION -- CTA 2000 AND 2200 SERIES CARS RELATIVE TO CTA 2400 SERIES CARS

trucks on the CTA 2000 and 2200 Series cars have the equivalent of very stiff primary springs.

The Wegman truck has a number of features that help reduce ground vibration. These features include (Ref. A-175):

- Low stiffness primary springs providing good isolation between the wheel/axle assembly and the truck frame.

- A flexible frame -- consisting of two side frames connected in the middle by a web structure -- to achieve wheel load equalization.

- Combination steel/rubber bolster springs to support the body.

- All points of contact supplied with rubber bushings or pads.

- Rubber and steel sandwich center bearing.

Modification of the primary stiffness of the Wegman truck is the most important factor contributing to reduction of groundborne vibration. However, damping provided by the resilient elements used to eliminate most metal-to-metal contacts may yield some additional vibration reduction by absorbing vibration energy.

In general, primary springs and flexible frames are probably the most effective way of reducing unsprung mass. However, aluminum centered wheels, hollow axle assemblies, and articulated axles with universal joints have been suggested to reduce the unsprung mass of the trucks (Ref. A-52).

The University of Toronto (Ref. A-52) also suggested an axle mounted vibration absorber (by redesign of the unsprung gearbox mass) to reduce resonant vibration of axles and wheels. Vibration reductions of approximately 10 dB were predicted by the University of Toronto from results of initial model studies for absorber masses of 12% to 25% of the wheel mass.

Rather than a vibration absorber, a dynamic absorber could be more effective. The dynamic absorber incorporates a viscous damping element which dissipates vibration energy before it can build up in axle and/or truck frame bending modes. The dynmaic absorber may also be more effective for random excitation and less sensitive to tuning, whereas the vibration absorber is probably best used for continuous sinusoidal vibration, such as produced by an engine or fan, where it simply causes a null in the response of the system at the excitation frequency. At present, no theoretical modeling of dynamic absorbers for transit vehicle trucks has been performed.

As indicated above, substantial reduction of groundborne vibration may be achieved with suitably designed vehicle trucks. However, proper design and parameter evaluation necessarily require a multi-degree-of-freedom model capable of representing translational or rotational modes of truck components, as well as axle bending modes if frequencies above 60 Hz are to be included in the analysis. The work performed by the University of Toronto is the most thorough in this regard (Ref. A-52). Also, the mathematical and computer models used for ride quality assessment are very similar to those which would be required for vibration analysis and may be adaptable to analysis of groundborne vibration.

5.3 WHEEL TYPE

Measurements on the SEPTA system (Ref. A-9) indicate significant reduction of groundborne vibration with the use of resilient wheels. Compared to solid steel wheels, the reduction amounted to 4 to 8 dB for aerial structure vibration and 8 to 12 dB for subway structure vibration over the frequency range of 31.5 Hz to 125 Hz. Vibration amplitudes (Ref. A-92) for axle box acceleration were found to be significantly lower for locomotives using SAB resilient wheels than locomotives using standard solid wheels by roughly 10 to 20 dB at frequencies above 30 Hz. Mixed results are reported by Wilson for operation on a BART aerial structure (Ref. A-20, A-172), where SAB resilient wheels produced lower levels of ground vibration at 16 and 31.5 Hz than BLH damped or ordinary steel wheels (see Figure 5.5).

5.4 RAIL SUPPORT

For the purposes of this discussion, the term "rail support" will refer to the track supporting hardware such as tie, tie plate, fastener, or other hardware necessary for direct support of the rail, and excludes various trackbeds such as the floating slab, double tie, or concrete trackbed. Floating slabs will be discussed in the following section, due to their unique and substantially more effective vibration isolation characteristics.

5.4.1 Types of Rail Fasteners and Performance

One of the first lines of defense against groundborne vibration from subways with concrete inverts is a resilient direct fixation system designed specifically to reduce groundborne vibration. Many different fastener designs have been developed, all of which perform in a predictable fashion. They can be grouped under the following general types:

1. Bonded and unbonded resilient fasteners with elastomer in compression.

2. Resilient fastener with elastomer in shear.

3. Longitudinally continuous resilient rail fixation.

The typical fastener currently used for new construction on concrete roadbeds usually employs an elastomer pad bonded between two steel plates. One plate is bolted to the concrete roadbed, and the rail is clipped or bolted to the other plate. Examples of such fasteners are those manufactured by Landis Sales Company, Lord Kinematics and Hixson which are currently being installed on the WMATA, BART, and MARTA systems. The standard Toronto Transit Commission fastener uses an unbonded elastomeric pad precompressed with hold down bolts and springs.

The selection and design of resilient rail fasteners is discussed by Wilson (Ref. A-39); he indicates that of the two types of fasteners using elastomer pads in compression, the bonded fastener has advantages over the unbonded design because of no precompression of the resilient element; vibration isolation in all three directions; and no surface abrasion. The unbonded fastener is more economical if the elastomer requires frequent replacement. This is not, however, viewed as significant, because the elastomer elements may be compounded to achieve very long life expectancies even in the presence of ozone and oil. The vibration isolation of the bonded fastener is complete in all three directions, where as with some unbonded designs, lateral shorting of the fastener top plate to the anchor bolt may occur. At special trackwork, the unbonded fastener may be most appropriate in view of the difficulty in acquiring unique and possibly non-standard bonded rail fasteners.

A new fastener design has been introduced by Clouth (Germany) which incorporates elastomer-in-shear to achieve relatively low vertical stiffness without unduly sacrificing the lateral stability of the rail. This fastener (Oberbau 1403c) has been dubbed the "Cologne Egg" and is reputed to have excellent vibration isolation performance (Ref. A-90). The Cologne Egg uses an elliptically shaped elastomer-in-shear ring (or collar) bonded between two conically cast elements, as detailed in Figure 5.2. The major axis of the ellipse is oriented transversely to the rail to achieve high lateral rail stability. The fastener comes in two versions -- one 90 mm high and the other 70 mm high for applications where height limitations exist.

The vertical stiffness of the Clouth fastener is about 30,000 lb/in. which will give a rail support modulus of about 1,000 lb/in^2 for a 30 in. fastener spacing. The static deflection under train load is 5 mm (0.2 in.) and 1.5 mm (0.06 in.) for the 90 mm and 70 mm high fasteners, respectively.

The vibration reduction reported by Braitsch (Ref. A-118) for the 90 mm high model relative to ballast and tie trackbed is 10 to 15 dB for frequencies between 25 and 40 Hz, and 25 dB for frequencies between 40 and 80 Hz, based on measurements performed at the tunnel wall at a number of systems. However, such a reduction for frequencies below 40 Hz is very difficult to achieve. Braitsch's conclusions are based on measurement data collected on different systems and may have been influenced by the variations between

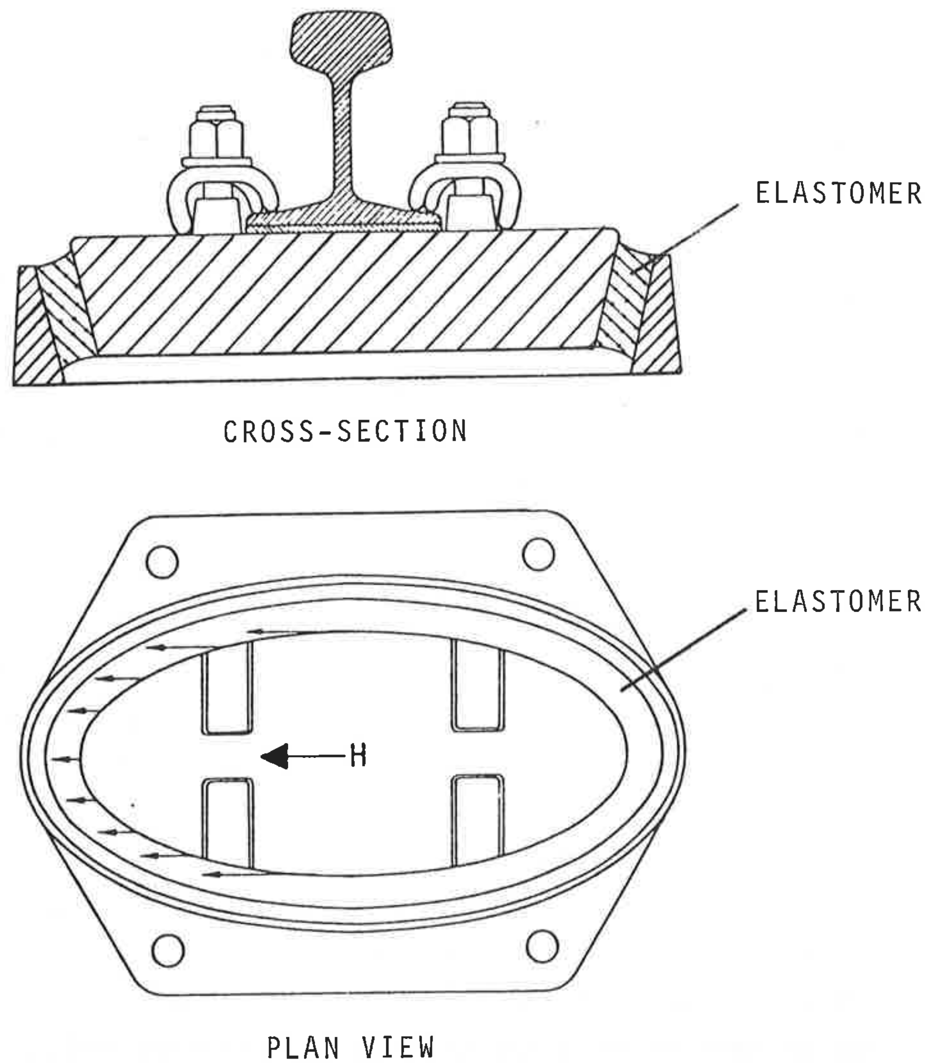


FIGURE 5.2 CLOUTH 1403/c ("COLOGNE EGG") RAIL FASTENER

subway founding conditions, subway structure parameters, or possibly rolling stock characteristics. A more meaningful comparison of the Clouth 1403c fastener performance with the performances of a Clouth 1403b fastener and a floating slab trackbed, all measured on the Cologne U-Bahn, is presented in Figure 5.3.

In Figure 5.3 the subway wall vibration velocity level shown for the Cologne Egg, fastener is similar to that shown for the Clouth 1403b fastener at 40 Hz and lower frequencies. At higher frequencies between 50 and 100 Hz, the Cologne Egg produced about 10 dB lower vibration levels than the 1403b design. The Cologne Egg produced higher vibration levels than the floating slab vibration isolation system, except between 50 and 80 Hz, where the levels were comparable. The dynamic stiffness of the Clouth 1403b fastener was not specified, but is undoubtedly higher than that of the Cologne Egg. The floating slab resonance frequency is probably in the neighborhood of 16 Hz. No data were presented for frequencies less than 31.5 Hz. The data presented in Figure 5.3 indicate that the Cologne Egg fastener provides significant vibration reduction at frequencies above about 40 Hz.

Another recent fastener design is the Metalastik resilient rail fastener illustrated in Figure 5.4. Compared to conventional fasteners using flat pads, this fastener enhances the moment reaction to rail overturning relative to vertical stiffness by moving the center of rotation closer to the rail head. The center of rotation can actually be located at or above the rail head. The vertical dynamic stiffness of this fastener can be varied by design of the resilient elements. The ratio of transverse to vertical static stiffness is also variable and depends on static load. For 3 mm (0.12 in.) vertical deflection, a ratio of 0.3 to 0.4 was achieved under test (Ref. A-176). No vibration isolation

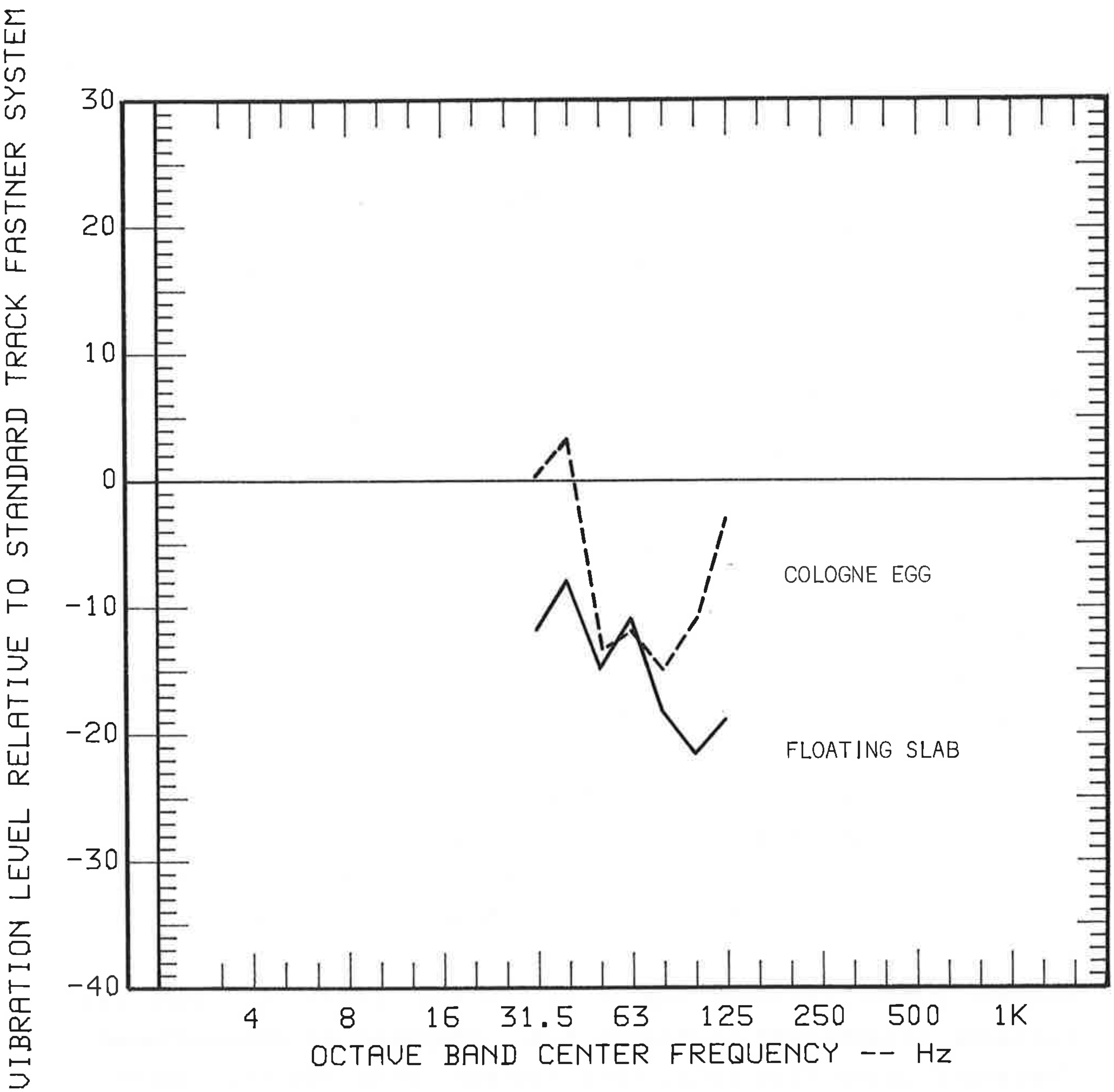


FIGURE 5.3 VIBRATION ISOLATION PERFORMANCE OF COLOGNE EGG (LEVELS RELATIVE TO STANDARD FASTENER) ADAPTED FROM REF. A-90

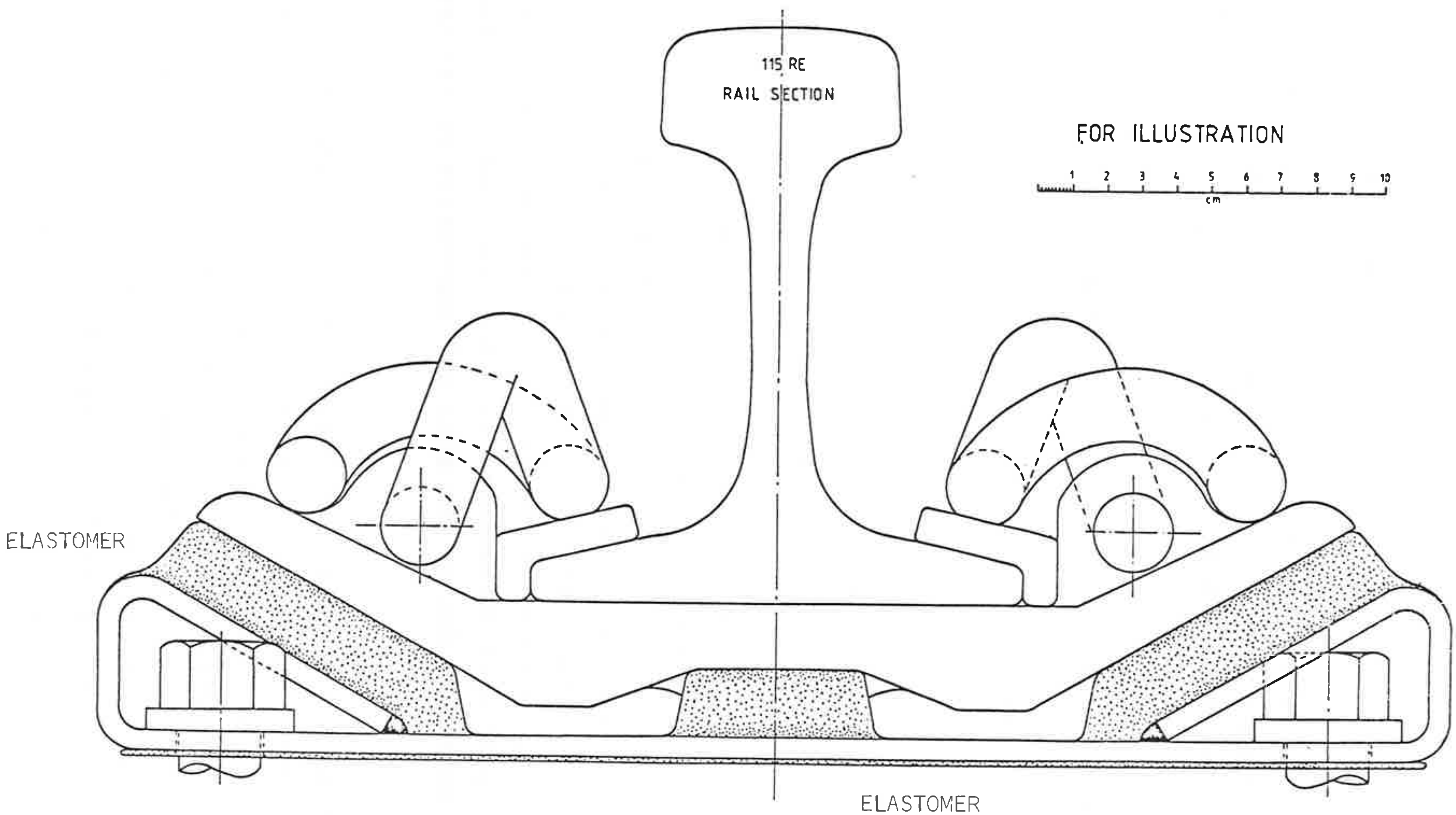


FIGURE 5.4 METALASTIC RESILIENT RAIL FASTENER

performance data has been acquired as of this writing.

The relative performance of various prototype rail fastener designs for the BART aerial structures were evaluated experimentally by Wilson (Ref. A-20, A-172). The fastener test included:

- Standard Toronto Fastener with 3/16" pad

- Pandrol Fastener with 1/8" pad and nylon clips

- General Tire Fastener

- B. F. Goodrich Fastener

Some of the results of the ground surface measurements at 30 ft (9.1 m) and 60 ft (18.3 m) from mid-span are presented in Figure 5.5. The tests showed significant differences between vibration levels with the standard Toronto and General Tire fastener, and little difference between the standard Toronto and Pandrol fastener. While, as shown in Figure 5.5, the various fasteners resulted in significant differences in vibration level, this could be partially accounted for by the physical separation of the test sections.

In passing, note that significant differences are observed in Figure 5.5 between vibration levels for car B and for cars A and C. Car A was equipped with BLH damped wheels, car B was equipped with SAB resilient wheels, and car C is equipped with standard wheels. The stiffness of the BLH damped wheel is comparable with that of the ordinary steel wheel. The data therefore indicates that a reduction of wheel stiffness can reduce groundborne

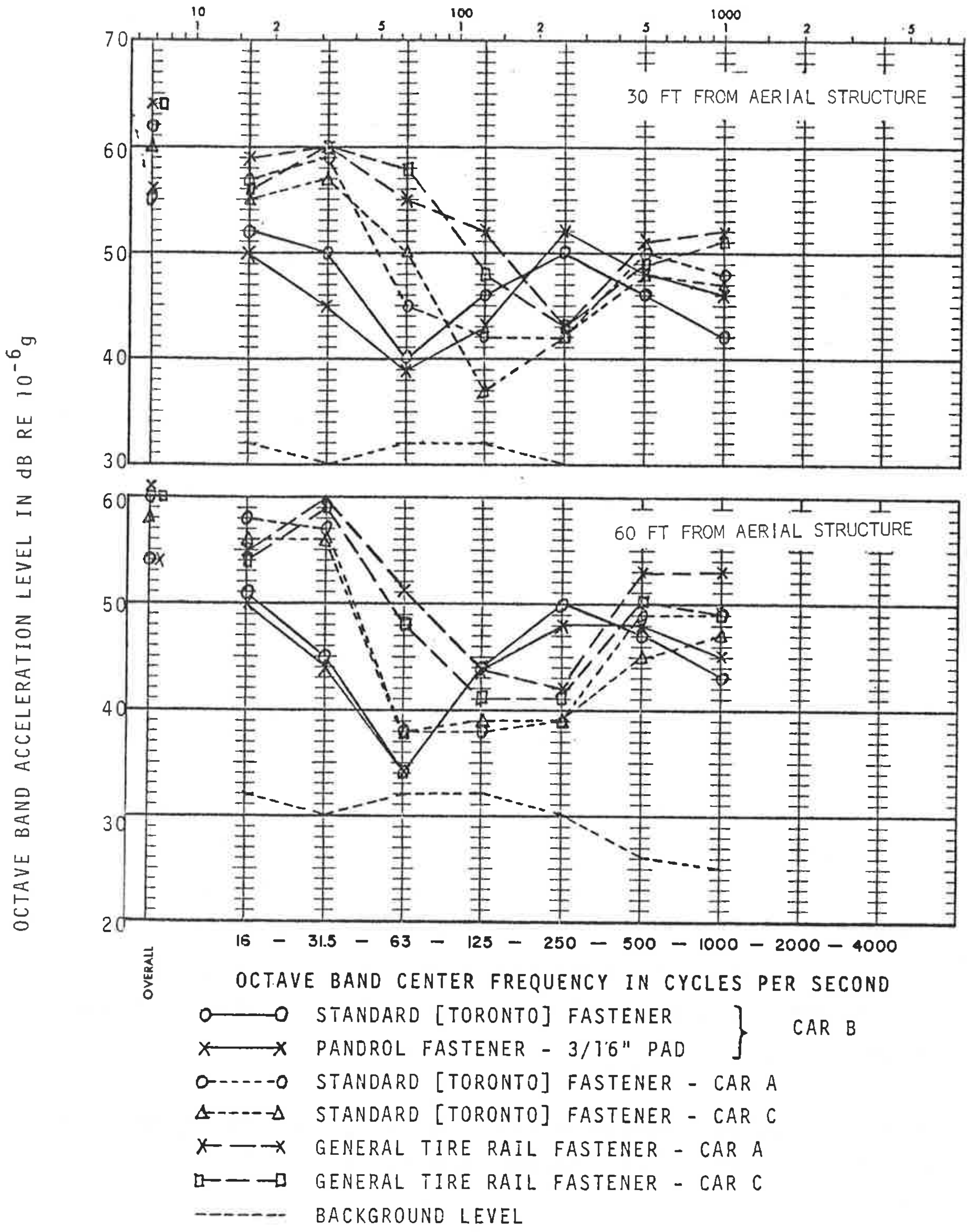


FIGURE 5.5 GROUND VIBRATION LEVELS NEAR BART AERIAL STRUCTURE - CARS PASSING BY AT 60 MPH

vibration at low frequencies more significantly than differences in fastener design.

A novel resilient fixation design has been studied in Japan by Satoh, et.al. (Ref. A-89) for control of noise and vibration at aerial structures. The design uses continuous rubber strips and clamps to hold the rail. Two variations of this design were installed for permanent service at the Akashi ballasted elevated structure and the ballastless Shimo Kanzakigawa Bridge in Japan. Octave band vibration reductions at stringer webs and upper flanges were roughly 5 to 10 dB at 63 Hz, 0 to 10 dB at 125 Hz, and 5 to 10 dB at 250 Hz. No data on the reduction of groundborne vibration were reported.

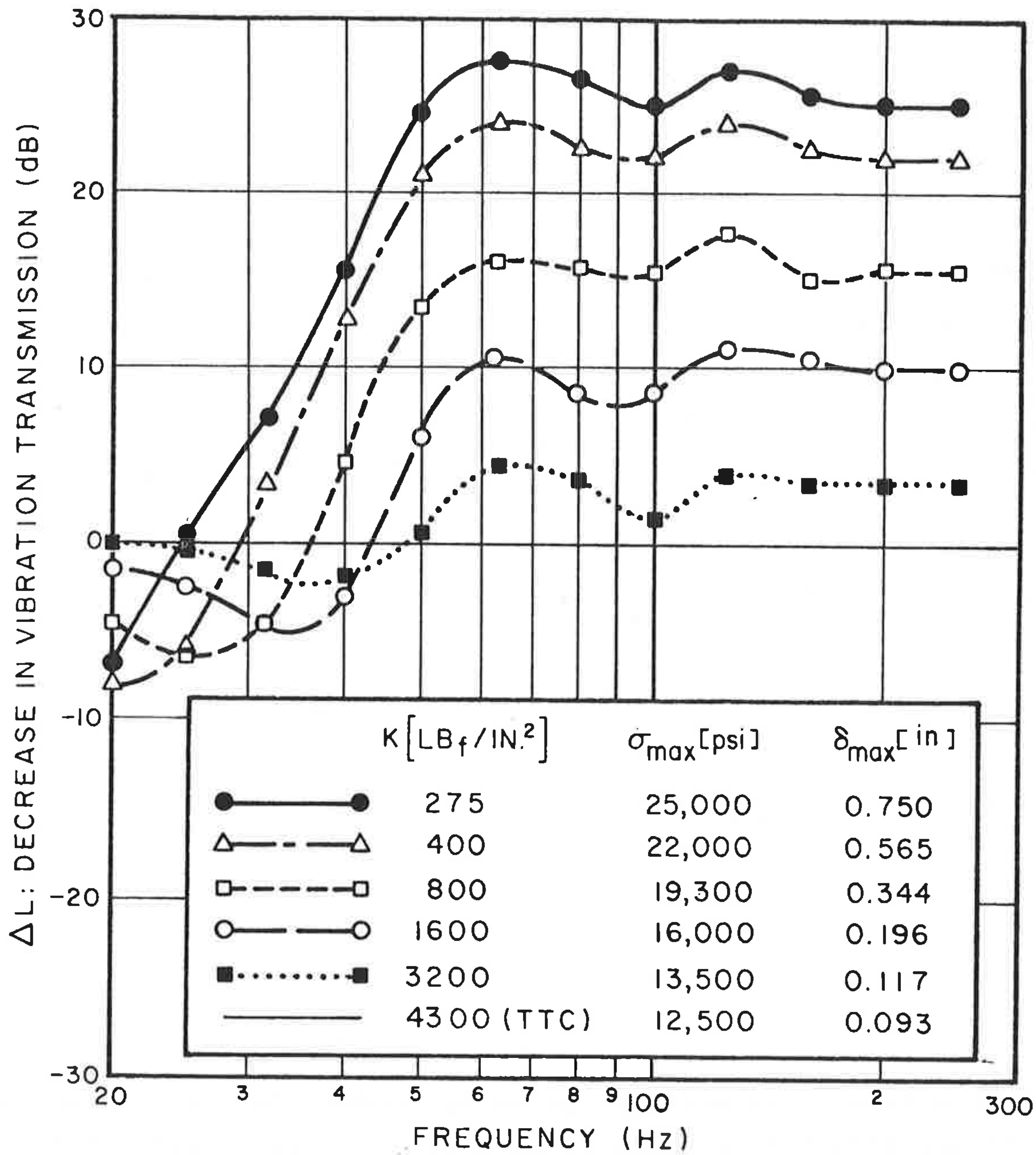
5.4.2 Rail Support Stiffness

Wilson (Ref. A-39) identifies an optimum rail support modulus of 3,000 lb/in. per lineal inch of rail. Although stiffer fasteners are probably required at special trackwork, the overall rail support modulus should be less than 6,000 lb/in^2. Lateral stability usually requires that the vertical rail support modulus for the elastomer-in-compression fasteners be a minimum of about 3,000 lb/in^2.

Based on a theoretical analysis (Ref. A-18), the magnitude of the integrated or dynamic force transmitted to the roadbed is equivalent to that which would be transmitted through a single-degree-of-freedom vibration isolation system. This indicates that the rail and fastener may be modeled accordingly as a first approximation. Using this model, and using impedance formulas for the wheel modeled as a mass and the rail as a beam on an elastic foundation, the theoretical vibration transmission for several values of rail support moduli were computed relative to the standard TTC rail support modulus of 4,300 lb/in^2. The results are presented in Figure 5.6. Also presented in Figure 5.6 are computed values of maximum rail stress and deflection under a 30,000 lb point load for standard AREA 115 rail.

Judging from Figure 5.6, the level of ground vibration for rail support modulus, K, of 275 to 4,300 lb/in^2 is roughly proportional to 20 log K at frequencies above 30 to 50 Hz. However, amplification at frequencies below 30 Hz is evident, indicating that too low a rail support modulus may not be beneficial. The result is that a "compromise" rail support modulus of 800 lb/in.^{2} is recommended by Bender, et.al. (Ref. A-18) as adequate for vibration control. This recommendation is based on noise and vibration considerations only and is much lower than the "usual" resilient fastener rail support modulus of 3,000 to 4,500 lb/in.^{2}, as used, for instance, by TTC. The introduction of the elastomer-in-shear resilient fastener, such as the "Cologne Egg," may allow a low rail support modulus without sacrificing rail stability. However, such fastener designs must necessarily be coordinated with requirements for rail stress, ride quality, stability etc.

Results from a later investigaton by Bender (Ref. A-81) indicate that the effect of rail fastener stiffness may be related to building vibration as in Table 5-1. Above 100 Hz the change in vibration level with a change in support modulus K is approximately proportional to 20 log K, while at frequencies between 10 to 30 Hz it is proportional to about 5 log K. For the frequencies between 30 and 100 Hz, no formula are given, because of the complicated nature of rail and wheel impedances. The analysis is limited to rail support moduli in the range of 3,000 lb/in.^{2} or higher.



DECREASE IN VIBRATION TRANSMISSION COMPARED WITH TTC FASTENERS FOR SEVERAL VALUES OF ISOLATOR STIFFNESS. ALSO SHOWN ARE THE MAXIMUM RAIL BENDING STRESS AND DEFLECTION FOR A 30,000 LB POINT LOAD.

FIGURE 5.6 THEORETICAL VIBRATION ISOLATION (BENDER, REF. A-18)

TABLE 5-1 EFFECT OF FASTENER STIFFNESS ON NOISE TRANSMITTED TO BUILDINGS (REF. A-81)

FREQUENCY RANGE	EFFECT
10 to 30 Hz	$\simeq 5 \log (K)$
> 100 Hz	$\simeq 20 \log (K)$
30 to 100 Hz	(not given)

K = fastener stiffness

The effect of doubling the pad thickness of the Toronto fastener was determined by measurements at the YSNE tunnels in Toronto (Ref. A-52). The question of variation in vibration propagation characteristics for different locations was removed by performing measurements before and after adding the extra pad. The results of these tests are summarized in Figure 5.7, which shows the vibration level at the invert and at the ground surface of the double pad installation relative to the original single pad installation. Also presented in Figure 5.7 for comparison are the vibration level differences estimated by Bender (Ref. A-18) for rail support moduli of 3,200 and 1,600 lb/in^2. A doubling of the Toronto fastener pad thickness should result in a rail support modulus in the neighborhood of 2,200 lb/in^2, and, indeed, the measured differences fall within the estimates of Bender, for values of K equal to 3,200 and 1,600 lb/in^2. Also the knee in the spectral difference is well predicted. Thus, the general model proposed by Bender, et al, is supported by these data, and the assumption of model validity for values of rail support moduli less than 1,600 lb/in^2 is perhaps justified.

Paolillo presents measurement data concerning the reduction of groundborne vibration with fastener stiffness (Ref. A-36) at NYCTA. Results of these measurements indicate that groundborne vibration levels are roughly proportional to 20 log K over almost the entire frequency range. The tests involved first insertion of 1 in. (25 mm) thick Butyl pads followed in their stead by 3/8 in. (0.95 cm) thick ribbed neoprene pads between the wood half-tie blocks embedded in concrete and the tie plate. The ribbed neoprene pad was shown to be consistently superior to the 1 in. (25 mm) thick Butyl pads over the entire spectrum. The reduction obtained with the ribbed neoprene pad compared to the original configuration was about 10 dB between 12.5 Hz to 40 Hz and 15 to 20 dB between 50 and 400 Hz.

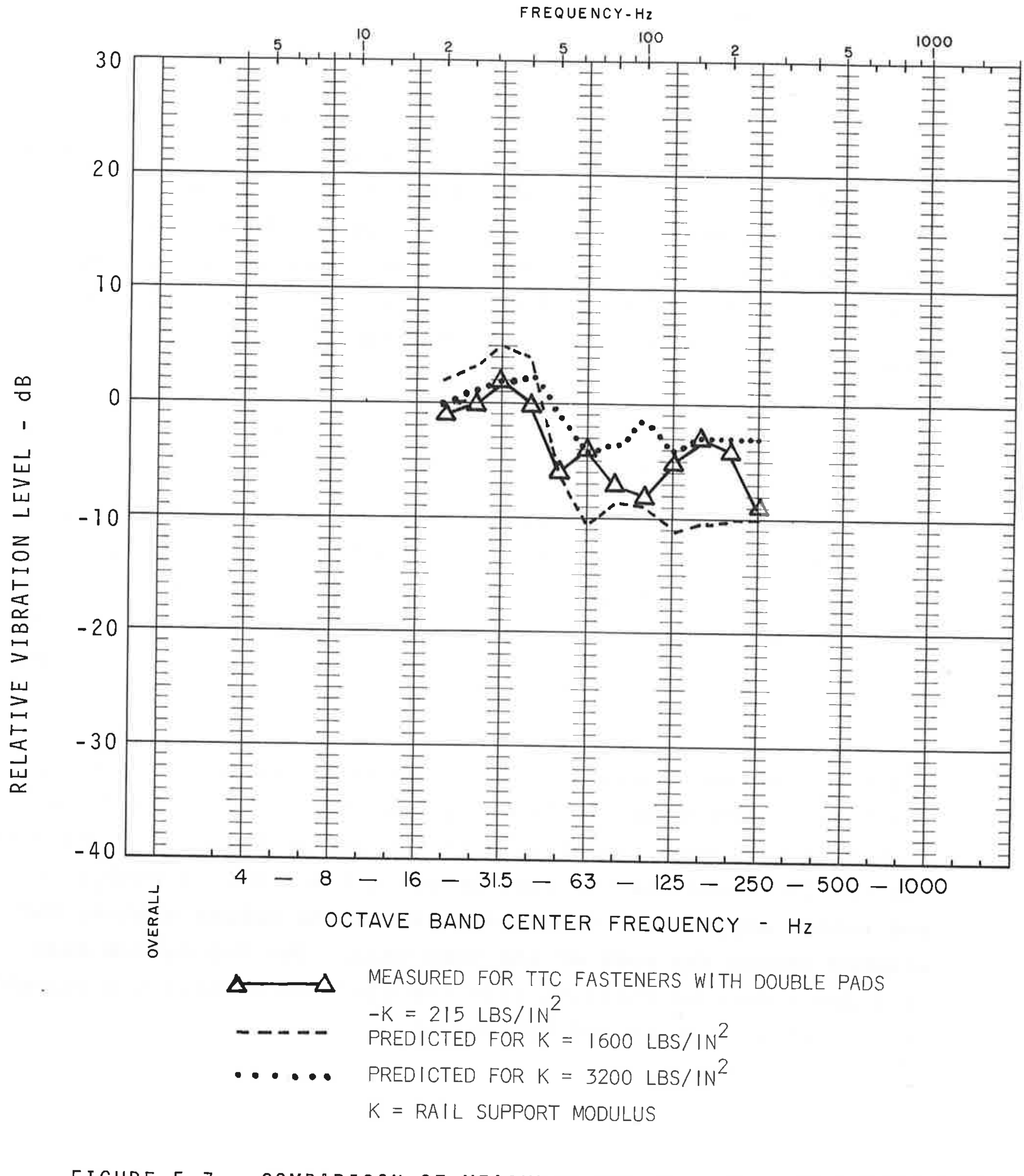


FIGURE 5.7 COMPARISON OF MEASURED AND PREDICTED VIBRATION LEVELS FOR RESILIENT DIRECT FIXATION FASTENERS RELATIVE TO THE STANDARD TTC FASTENER WITH RAIL SUPPORT MODULUS $K = 4300$ LBS/in^2

5.4.3 Resiliently Supported Ties

The resiliently supported concrete tie rail support system provides two stages of vibration isolation which can be particularly effective. The VSB/STEDEF two-block system is an example and is illustrated in Figures 5.8 and 5.9. The system consists of a two-block tie, with the tie blocks in pockets in a concrete invert with a neoprene rubber boot and expanded neoprene support pad between the tie and invert for vibration isolating. The rail is fixed to the tie block with an electrically insulating clip using a 3/16 in. (0.5 cm) rubber pad between the rail and tie block. The rubber pad is relatively stiff and provides little or no vibration isolation.

The STEDEF system has been tested extensively by the Paris Metro (RATP) and is being used extensively in place of the standard ballast and tie track used in their earlier double-track tunnel installations. The STEDEF system is also being used on the MURLA system in Melbourne, the BRRT system in Baltimore, and the MARTA system in Atlanta.

Another resiliently supported tie system is the Voest-Sempirit resiliently supported tie/floating slab system which has been installed in Vienna, Austria. The V-S system is a solid one-piece PVC tie (it could use pre-cast concrete ties) with a rubber boot and rubber support pad system similar to the STEDEF system, but wrapped around the ends of the ties only. The V-S system also includes a form of floating slab trackbed which makes the assembly unnecessarily complex and expensive.

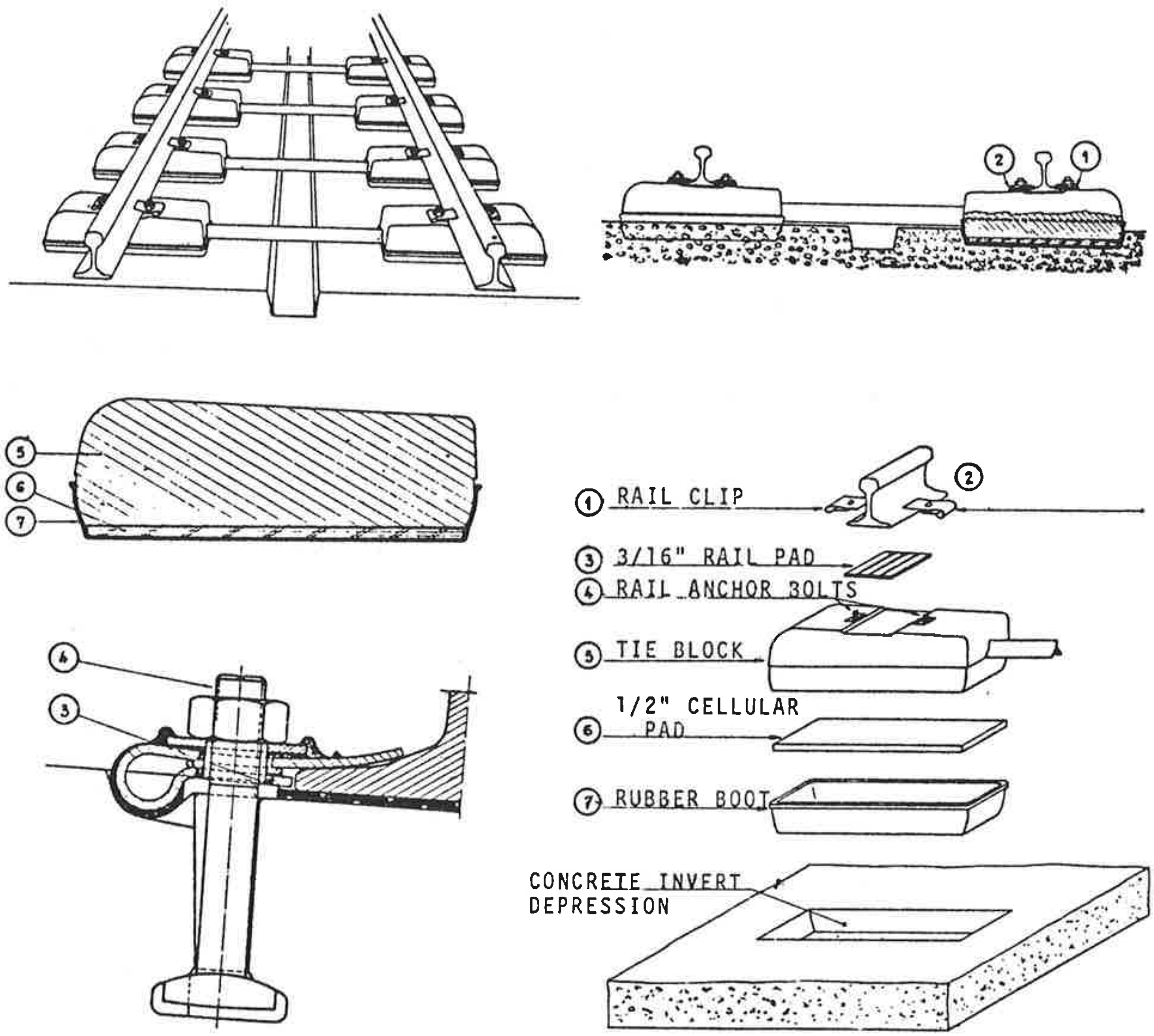


FIGURE 5.8 RS-STEDEF RESILIENTLY SUPPORTED RAIL TIE SYSTEM COMPONENTS

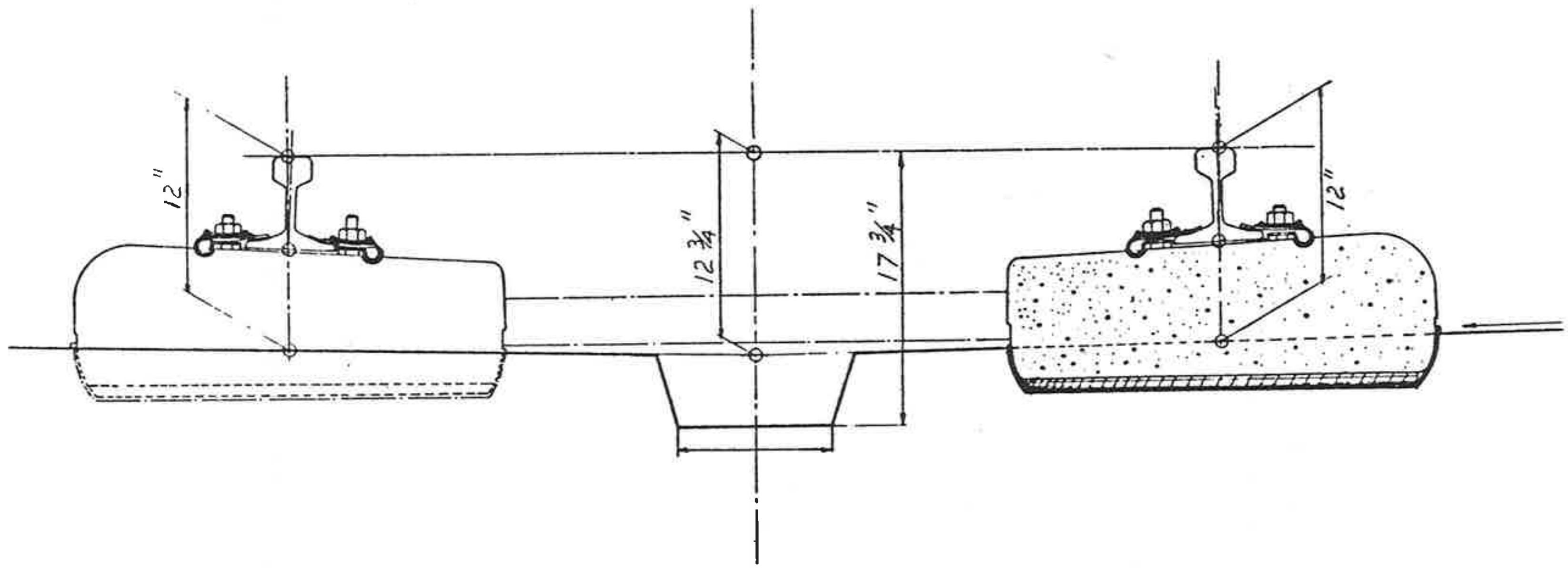


FIGURE 5.9 DRAWING OF COMPLETE RS-STEDEF RESILIENTLY SUPPORTED RAIL TIE SYSTEM SHOWING THE COMPLETE ASSEMBLY AND BASIC DIMENSIONS

The vibration isolation performance of resiliently supported ties has been evaluated experimentally at a number of systems in Europe (Ref. A-28, A-22, A-27). A variation including wood ties supported on resilient elements was evaluated by Lang (Ref. A-29, A-101). The measurements reported by Colombaud (Ref. A-28) are perhaps the most often quoted in the literature concerning the vibration isolation performance of the STEDEF resiliently supported tie, which gave the best overall performance in the Paris RER when compared to RAPT and SNCF (Type F) resilient fasteners and ballast and tie track. These data are summarized in Figure 5.10. A high degree of material damping in the STEDEF resilient element is claimed to be responsible for the relatively low 63 Hz octave band vibration level in comparison with other fastener designs.

The STEDEF system was also experimentally evaluated by Nolle in relation to standard ballast and tie track at the Jolimont Test Track (Ref. A-78). Very little reduction of vibration was achieved by the STEDEF system relative to ballast and tie track except for about 5 dB at frequencies above 250 Hz. An amplification of about 1 to 3 dB is evident in the frequency range of 50 to 80 Hz. Below 40 Hz, the STEDEF system produced 5 dB lower levels. These results are consistent, because the ballast and tie track provides a significant amount of vibration isolation at high frequencies relative to, for instance, direct fixation fasteners on concrete slab roadbeds.

5.5 FLOATING SLABS

5.5.1 Types of Floating Slabs

Floating slabs basically consist of a concrete trackway supported by rubber or loadbearing fiberglass pads. Floating slabs work on

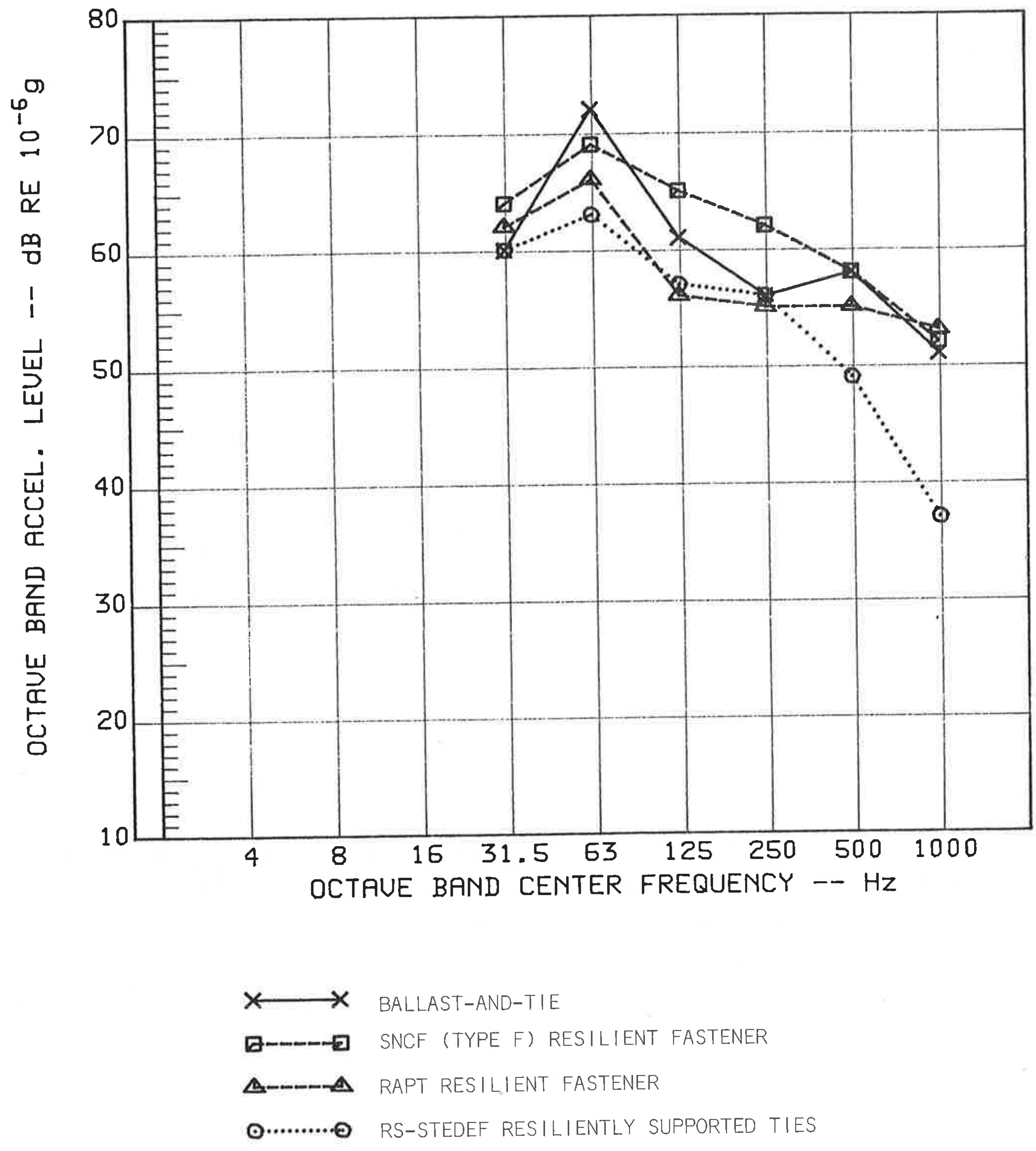


FIGURE 5.10 VIBRATION LEVEL MEASURED IN DRAINAGE TUNNEL FOR VARIOUS TYPES OF TRACK SUPPORT SYSTEMS (REF. A-28)

the same principal as inertia bases used to vibration isolate machinery. The typical inertia base consists of a concrete pad supported by springs with the machine to be isolated attached to the concrete pad. This system reduces the forces transmitted from the machine to the foundation at frequencies above the resonance frequency of the spring-mass system. Near the resonance frequency the transmitted force is amplified, and well below the resonance frequency the force is transmitted with no change in magnitude.

Several varieties of floating slab systems have been developed. Some of the earlier designs such as those used by London Transport under the Barbican and on the extension to Heathrow Airport, are very heavy and expensive to install (Ref. A-106). More recent designs which require less space and are much less expensive, include the insulated track slab design for the Lime Street Station of the Mersey Railway Extensions by British Railways (Ref. A-106), the floating slab trackbed designed for the Washington, D.C. Metropolitan Area Transit Authority Metro System (Ref. A13, A-33), and the discontinuous double tie systems developed for the Toronto Transit Commission (Ref. A-76).

The vibration isolation of floating slabs is provided by the mass of the slab acting as an inertia mass and the resilience of the support pads acting as soft support springs which, in combination, reduce the transmission of the vibration forces to the subway structure. The system is very effective in reducing groundborne vibration from transit train operations. Compared with resilient direct fixation fasteners on rigid invert, concrete floating slabs of approximately 0.3 m (1 ft) thickness have reduced groundborne vibration very significantly over the audible frequency range (Ref. A-162). The disadvantages of the continuous floating slab system include the cost of construction, the difficulty of forming and pouring the concrete slab, the non-replaceability of the resilient elements without special provision, and bending waves in the slab which generate higher in-tunnel noise levels at low frequencies. The continous floating slab is illustrated in Figure 5.11.

British Rail has developed a continuous floating slab system with adjustable and replaceable springs (Ref. A-132) for use at mainline railways. They prefer the use of continuous floating slabs over discontinuous slabs because of radiation damping provided by slab bending waves, which carry energy away from the point of excitation at frequencies above the resonance frequency of about 15 to 18 Hz. They believe that the radiation damping is substantially greater than that provided by the resilient supported elements. Thus, local build-up of vibration energy does not take place. This concept may be true for single point excitation, but in reality, groundborne vibration from trains necessarily involves a distribution of point sources along the train. Thus, whether the slab is discontinuous or continuous, the vibration energy is distributed. However, radiation damping due to slab bending may play a role with respect to controlling amplification at the design resonance. Again, the radiation damping at resonance is apparently low. Thus, the significance of radiation damping needs to be assessed.

Most of the floating slabs being installed in North American transit systems are pre-cast discontinuous floating slabs. This system, often referred to as the double tie system, was developed by TTC (Ref. A-76), and is now being recommended and installed on the MARTA, BRRT, and MURLA (Australia) transit systems (Refs. A-105, A-70).

The double tie vibration isolation system consists of pre-cast concrete double ties supported on pads similar to the continuous

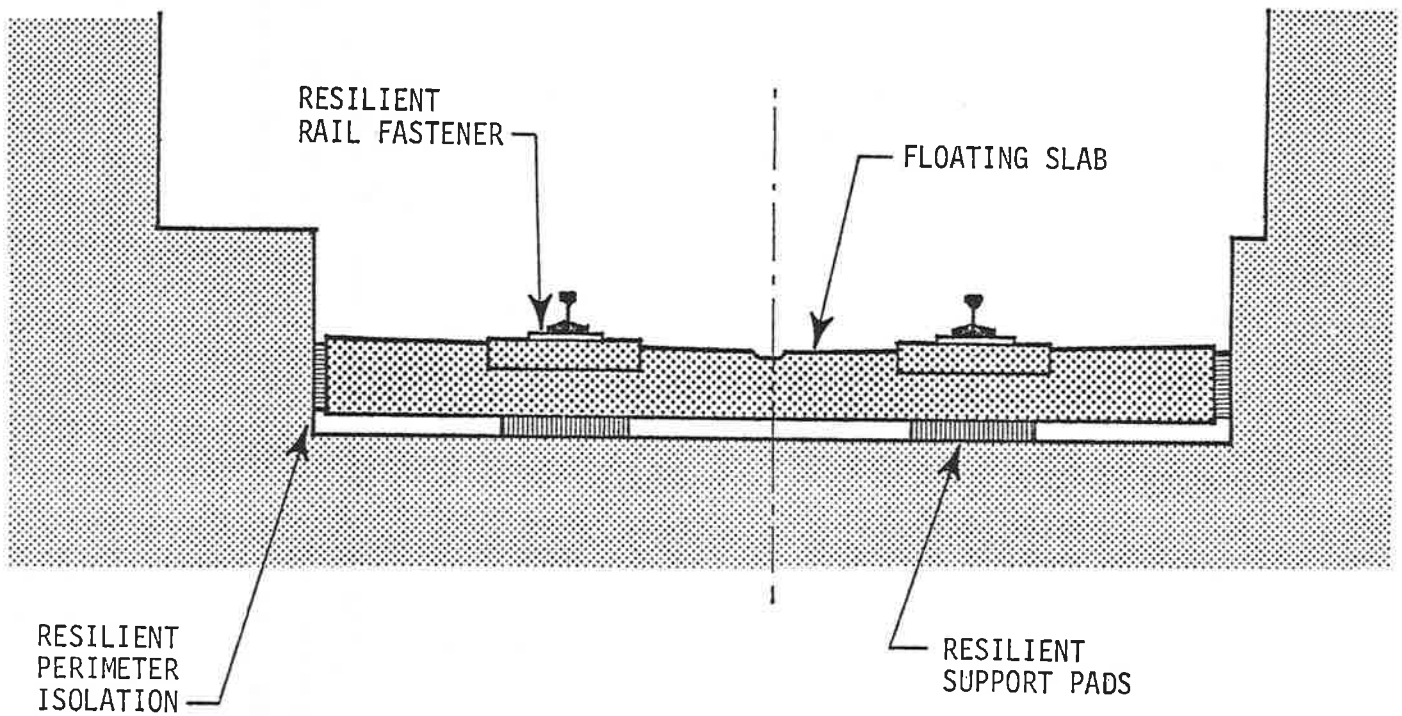


FIGURE 5.11 SECTION OF CONTINUOUS CAST-IN-PLACE FLOATING SLAB FOR CONCRETE BOX STRUCTURE

floating slab concept. Each double tie is isolated from neighboring double ties with additional resilient elements. The rail is fastened to the slabs with direct fixation fasteners using standard rail fixation hardware such as the Pandrol rail clip. The basic design of the double tie system is illustrated in Figure 5.12.

One of the major advantages of the double tie concept is that installation costs are significantly lower than for the continuous cast-in-place slab. The pre-cast concrete slabs are typically brought to their final resting places on fork-lifts and are easily positioned with hydraulic jacks and clamps (Refs. A-68, A-87). Alternatively, pneumatic bearings are sometimes used to ease final positioning of the slabs.

5.5.2 Floating Slab Performance

Estimates based on measurement data for vibration reduction performances of continuous and discontinuous floating slabs relative to direct fixation resilient fasteners are presented in Figure 5.13. These estimates are based on actual measurements of floating slab and non-floating slab sections of subway. The WMATA and TTC data are reported by Wilson (Ref. A-33, A-162), Nelson et al. (Refs. A-57, A-98), and Lawrence (Ref. 44), and in another TTC paper (Ref. A-14).

In a MURLA study (Ref. A-72) measurements of rock-borne vibration were made for trains running on ballast and tie. Then the ballast and tie track was removed and a discontinuous floating slab system was installed on a concrete invert poured directly on the base rock. The measured reduction for the MURLA discontinuous floating slab relative to ballast and tie track, was not as great as shown

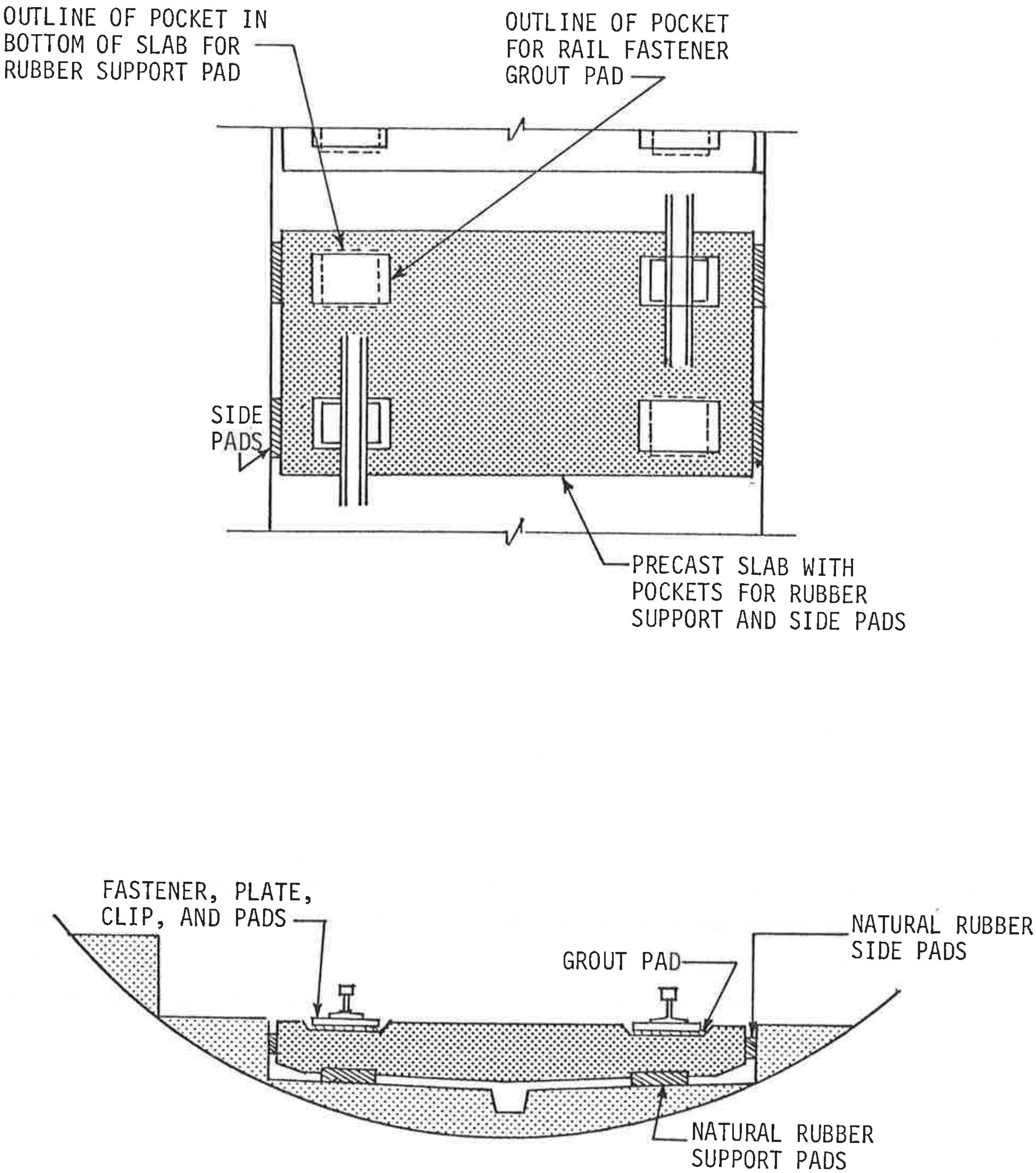


FIGURE 5.12 PLAN AND SECTION OF THE DISCONTINUOUS FLOATING SLAB

for the above other systems relative to direct fixation fasteners.

There is some limited data (Refs. A-57, A-98) which indicate that lightweight continuous floating slabs used in smaller cross-section circular tunnels may perform differently than the heavier continuous slabs used in the larger double-box structures. Figure 5.14 presents the measured vibration performances of floating slabs in concrete double box and circular tunnel subways. The data for the WMATA double-box structure is based on measurements at two different locations separated by several hundred feet, and is consistent with previous data reported by Wilson (Ref. A-33, A-162). The floating slab performance given for the WMATA circular earth tunnel is based on a limited set of measurements at two adjacent tunnel sections -- one with the floating slab and the other without. Thus, variations due to differences in subway founding and vibration propagation characteristics are to a large extent removed for the circular tunnel data.

A pronounced attenuation dip occurs for the circular earth performance curve at 20 Hz, followed by a peak at 25 Hz -- at which frequency virtually no reduction is obtained. Above 31.5 Hz, and below 16 Hz, both performance curves are essentially similar. Essentially the same qualitative performance was observed for both measurements on the subway benches as well as at points on the ground surface. Also, the measured ground surface and tunnel bench vibration levels are very similar to measurements at another circular concrete tunnel with floating slab at an entirely different location on the system, indicating that the dip at 20 Hz and peak at 25 Hz are not simply anomalies.

Additional data collected at MARTA also show a similar qualitative trend for the discontinuous double tie floating slab system used in the concrete double box structures, as illustrated in Figure 5.13. In addition to the resonance at 12.5 Hz, a second resonance appears at 31.5 Hz, followed by effective attenuation at higher frequencies (Ref. A-31).

The reason for this anomaly is not clear, but there are strong indications that it is related to coupling of the slab mass-spring system with the truck suspension. The primary suspension resonance of both the WMATA and MARTA Rockwell trucks is in the neighborhood of 20 to 25 Hz (Ref. A.-11, A-174), and the loaded floating slab resonance is about 16 Hz. Thus, the major resonance frequencies are relatively close, and coupling of the floating slab with the truck is likely.

At the MARTA system in Atlanta, experiments were also performed with adding mass to an existing discontinuous floating slab to see if the resonance frequency could be significantly lowered (Ref. A-50). This was done because of community reaction to groundborne vibration at frequencies in the neighborhood of 16 to 31.5 Hz. As discussed above, three factors combined to produce high levels of vibration. These factors are:

1. Primary suspension resonance in the neighborhood of 20 to 25 Hz for the Rockwell trucks.

2. Floating slab resonance frequency in the neighborhood of 16 Hz.

3. Floor and ceiling resonances at about 16 to 25 Hz within the residential wood-frame structures.

This type of vibration problem is similar to that described by

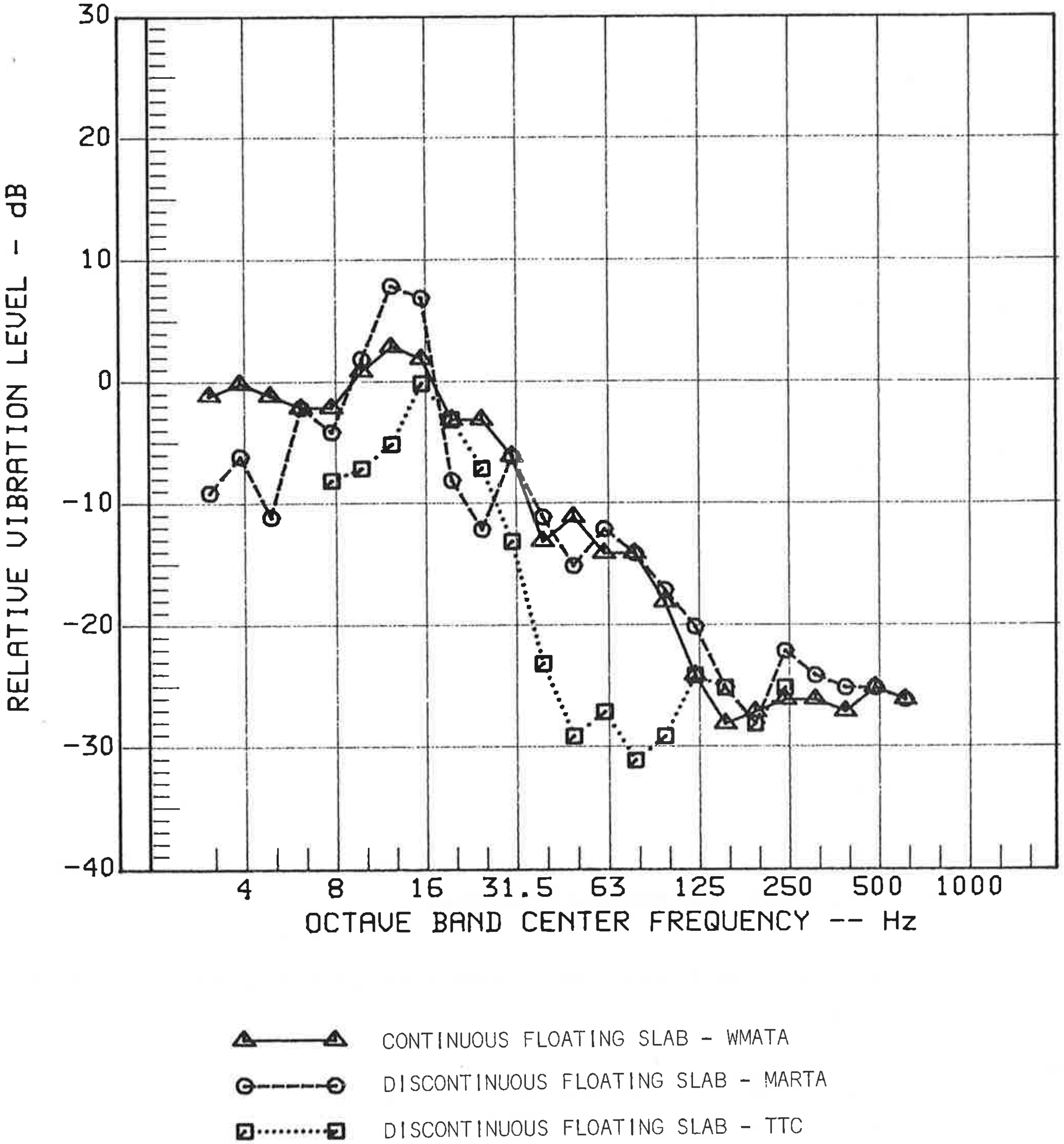


FIGURE 5.13 VIBRATION LEVELS WITH TWO TYPES OF FLOATING SLAB RELATIVE TO LEVELS WITH RIGID INVERT

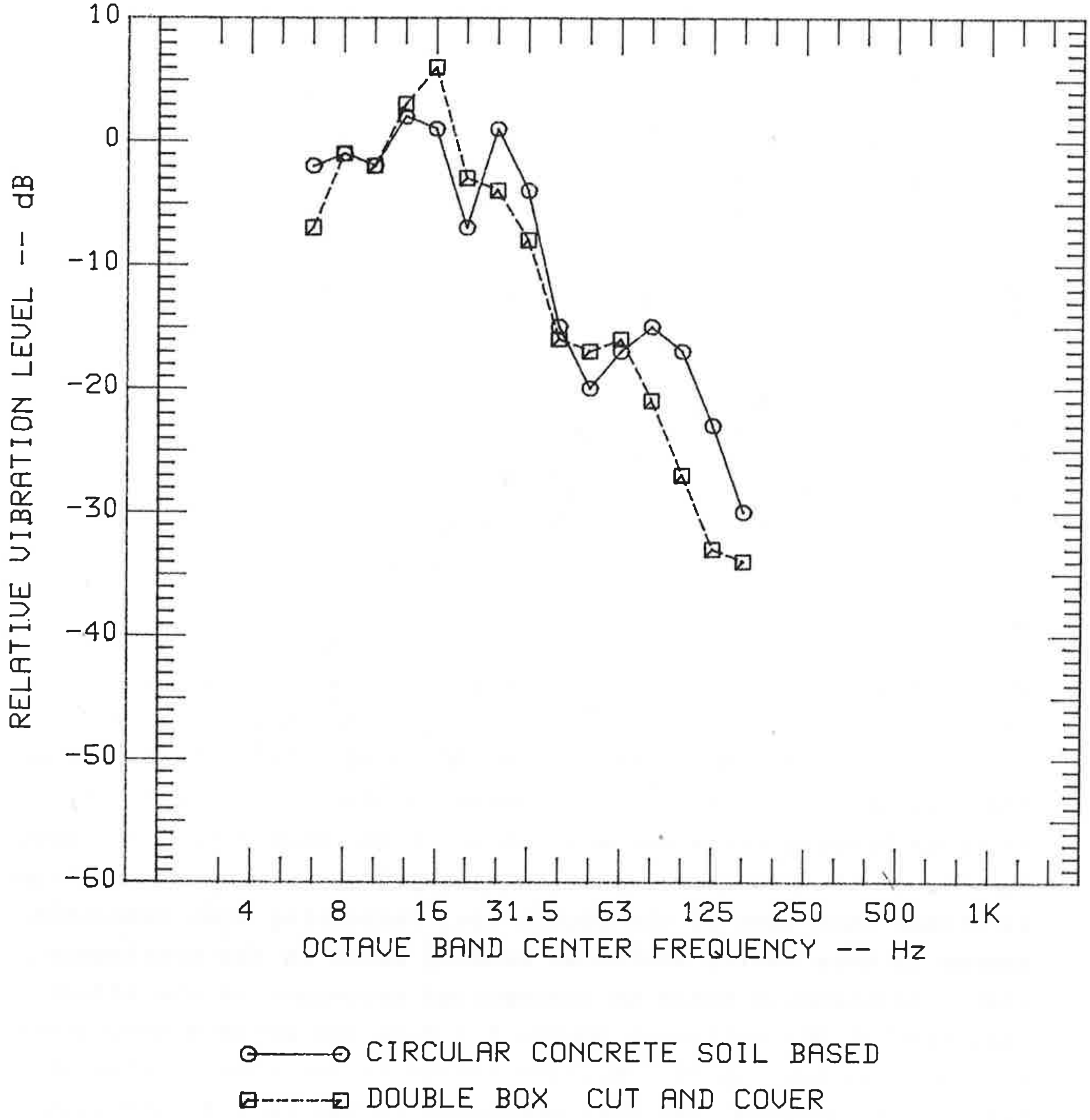


FIGURE 5.14 COMPARISON OF FLOATING SLAB VIBRATION REDUCTIONS FOR CUT-AND-COVER DOUBLE BOX AND CIRCULAR CONCRETE WMATA SUBWAYS FOUNDED IN SOIL

Paollilo (Ref. A-11) in regard to the NYCTA R-46 trucks.

The effect of doubling the mass of the MARTA floating slab is illustrated in Figure 5.15. Based on this data as well as on corresponding data recorded within the residential structures, the conclusion was reached that the loaded resonance frequency was reduced from 16 Hz to about 11.5 Hz. This is very similar to that which would be expected based on a single-degree-of-freedom model of the floating slab.

It is also of interest that the secondary peak at 31.5 Hz in the 1/3 octave spectrum for the unmodified floating slab was suppressed as a consequence of the mass doubling. This indicates that doubling the slab mass may help to uncouple vibration modes of the vehicle truck from the slab resonance, thereby achieving a more idealized isolation curve for the floating slab.

Grootenhuis indicates that the continuous floating slab design is preferable to the discontinuous double tie design as used at TTC because vibration energy may be concentrated in vibration bending modes of the double tie in the absence of damping (Ref. A-106). Although bending modes may well occur in the double tie, the modal density of the continuous floating slab of the same cross-section is higher than that of the double tie, indicating that vibration energy is more easily stored as bending waves in the continuous slab. Grootenhuis makes an unsupported statement to the effect that the TTC discontinuous double tie does not perform adequately due to local buildup of vibration energy in the slab, a statement that is at odds with the data presented in Figure 5.13 and 5.15. Since the first bending mode of a double tie slab is well above 150 Hz, if the vibration isolation is deficient because of slab bending, it is in a frequency region which is rarely important for groundborne vibration at U.S. transit systems. In any case, the

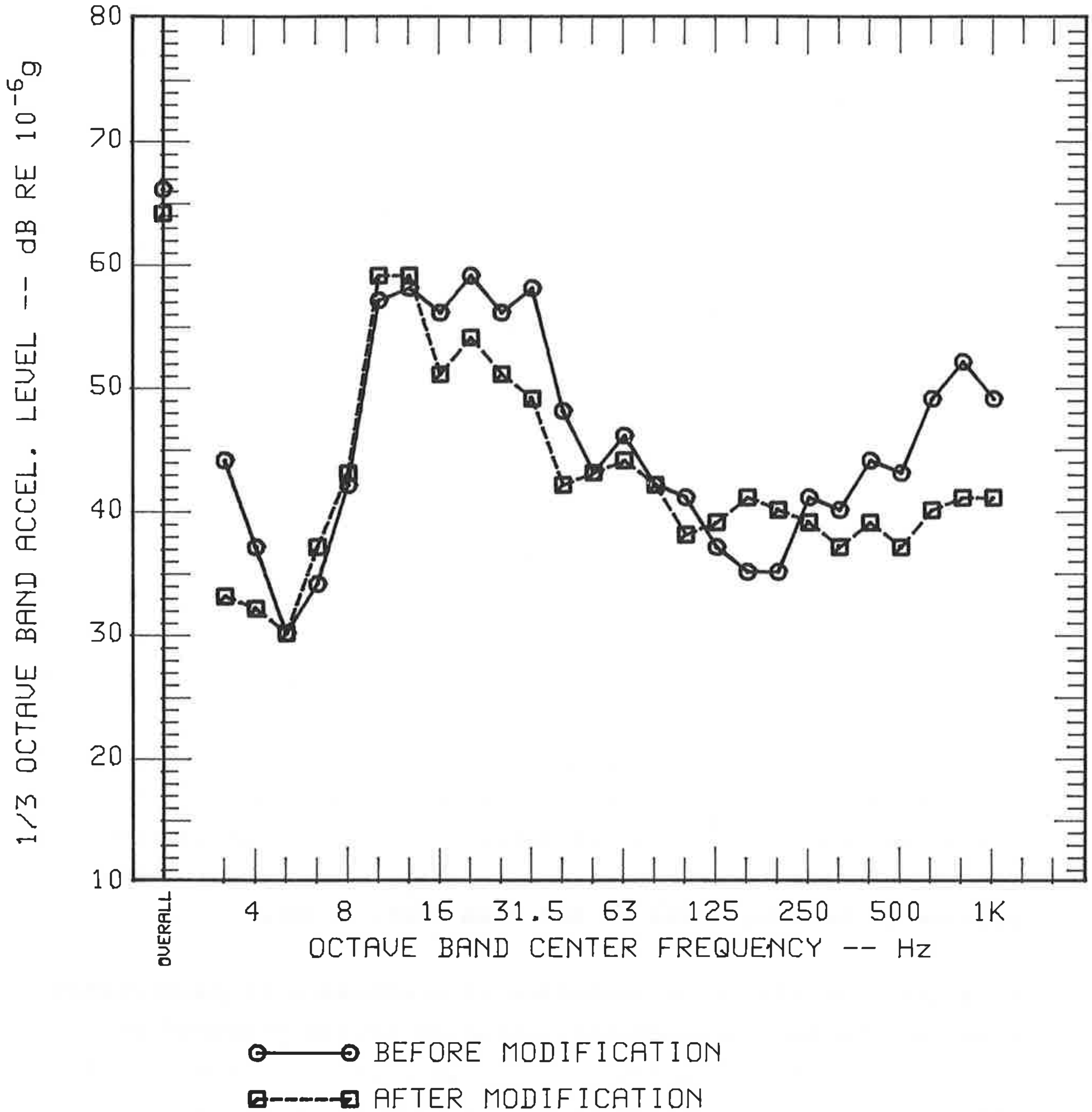


FIGURE 5.15 COMPARISON OF SUBWAY STRUCTURE VIBRATION LEVELS BEFORE AND AFTER ADDITION OF MASS TO DISCONTINUOUS FLOATING SLAB (AVERAGE OF SIDE AND CENTER BENCH DATA FOR TWO-CAR 50 MPH TRAINS)

peaks at 25 or 30 Hz indicated in the floating slab performance curves of Figures 5.14 and 5.15 are not caused by slab bending.

Grootenhuis' solution for controlling bending waves of the continuous floating slab is to use constrained layer damping in the concrete slab. However, this adds substantially to the cost of slab construction by requiring multiple pours and hence is possibly uneconomical, especially since the installation costs of the continuous slab without constrained layer damping are already quite high.

A relatively small increase in noise levels inside the subway may result with continuous floating slabs as a consequence of bending waves within the slab. At crossovers, if very wide single piece floating slabs are used, very significant rumbling noise may result, similar to the sound of thunder. The low-frequency noise radiation from the wide floating slabs is due to low-frequency bending modes, which also degrade vibration isolation performance at these frequencies. Thus, current practice for crossover slab design is to incorporate isolation joints to separate the slab into smaller units. On the TTC system, the crossover slabs use ballasted track with a longitudinal cut in the slab included to isolate the line tracks from one another, thus reducing the problem of bending waves in the slab (Ref. A-76).

Floating slab vibration isolation at crossovers is particularly important because groundborne vibration levels produced at crossover frogs and switches are roughly 10 dB higher over the entire 1/3 octave spectrum from 10 to 200 Hz relative to continuous welded rail. Thus, control of bending waves within the crossover floating slab is of particular importance. Indeed, the WMATA system incorporates a floating slab at all subway crossovers.

5.6 BALLAST MATS

A ballast mat is a resilient layer of material placed under the ballast. Ribbed rubber or neoprene sheets, fiber glass, mineral wool, and used automobile tires have all been used as ballast mats. They have received much attention and in-service evaluation in Europe and in Japan, but relatively little in the United States, possibly due to the emphasis on direct fixation trackwork which is more economical to maintain than ballasted track. A frequently encountered ballast mat is the ISOLIF mat consisting of two or three layers of ribbed rubber sheets interleaved so as to provide a very resilient sandwich construction.

The results of tests peformed in Europe to determine the vibration isolation performance of ballast mats are presented in Figure 5.16. All of the isolation curves shown in Figure 5.16 are for track with ballast mats relative to ballasted track without mats. The data are reported for the Munich S-Bahn (Ref. A-26), the Vienna U-Bahn (Ref. A-101), and the Paris Metro (Ref. A-155). The data are indicative of the range of vibration isolation which might be expected for a ballast mat installation.

The data given for the Munich S-Bahn are based on measurements on the invert, wall, and upper pedestrian gallery floor of a concrete double box subway structure, during actual train passbys. The Vienna U-Bahn data were determined with impulse excitation using a rail inspection vehicle. The Paris Metro data were determined from noise measurements inside rooms below the tracks at two train stations. Of these data, those given for the Munich S-Bahn and Paris Metro are most representative in that they are based on actual train passbys. From these data one might expect a vibration reduction between 5 and 15 dB at frequencies between 63 and 250 Hz. The data presented for the Vienna U-Bahn indicate a

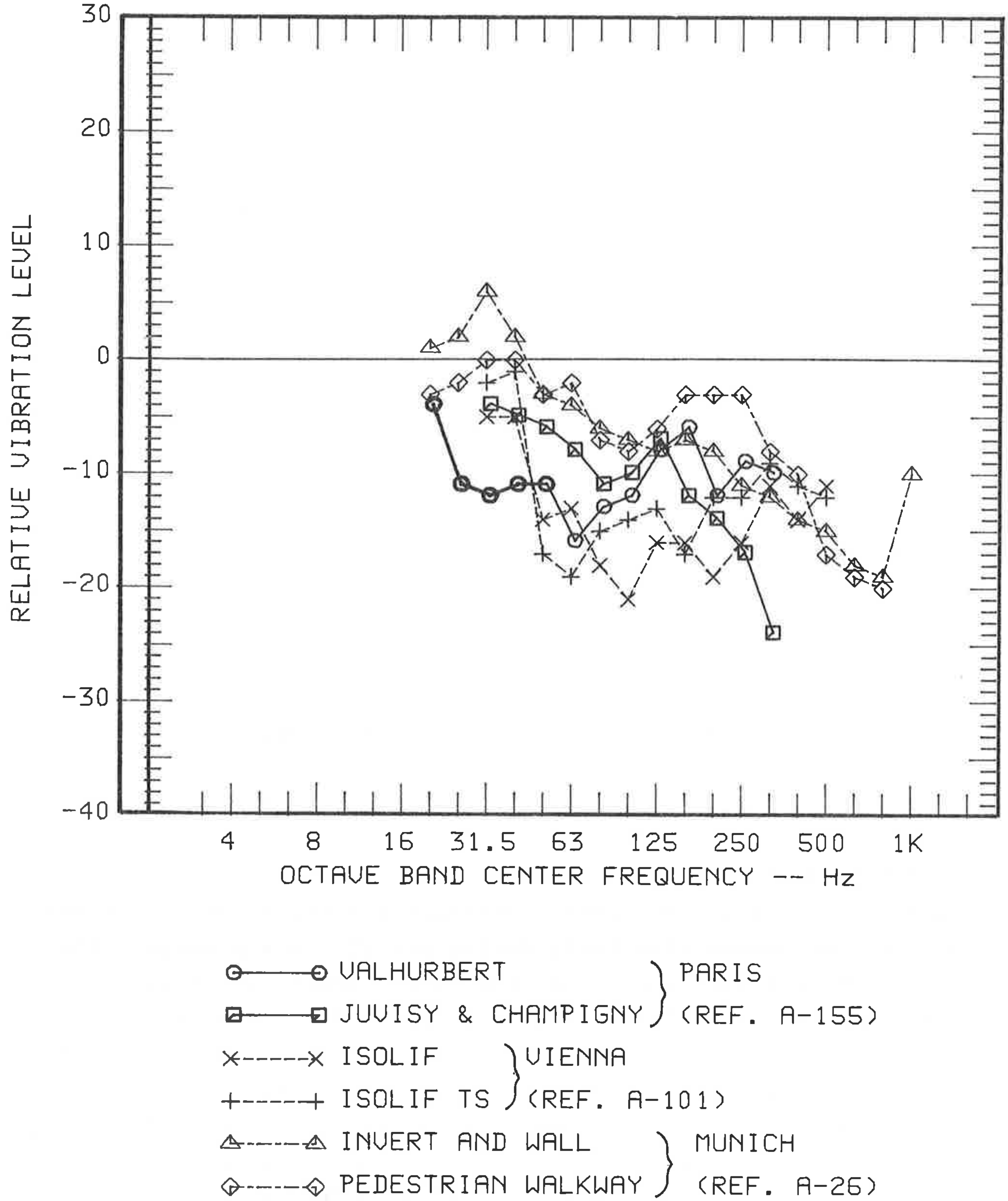


FIGURE 5.16 VIBRATION ISOLATION OF BALLAST MATS RELATIVE TO STANDARD BALLAST AND TIE TRACK

reduction of 13 to 20 dB over this same range for impact excitation. At 31.5 Hz and lower frequencies, little or no change in vibration level may be expected.

Octave band vibration isolation data for ballast mats used in a double box subway are given by Kazamaki (Ref. A-69). Subway floor vibration levels were roughly 20 dB lower with ballast mats compared to direct fixation track with train speeds of about 65 km/hr, over a frequency range of 63 Hz to 500 Hz. At 31.5 Hz, the ballast mat vibration reduction is only 10 dB. Subway floor vibration levels with the ballast and ballast mat track are between 10 and 20 dB lower than for the ballasted track with vibration damping cross-ties.

Vibration reductions for a ballast mat used on the Shinkansen aerial structure (viaduct) are given by Morii (Ref. A-53). Only a limited reduction of about 0 to 3 dB was obtained for girder vibration from 31.5 to 125 Hz, although above 150 Hz the reduction is between 10 and 20 dB. The reduced performance of a ballast mat used on an aerial structure relative to a subway structure is not surprising due to the lower impedance of the aerial structure trackbed against which the mat must act in order to provide adequate isolation.

Often the major motivation for use of ballast mats is to control ballast pulverization and increase ballast replacement intervals, as described by Tajima and Kiura (Ref. A-34). Their tests showed structure vibration magnitudes were reduced by about 20 to 30%, but, more importantly, manpower requirements for maintenance were reduced by about 50 percent. Similar reduction of maintenance costs would be expected for subway ballast and tie installations. For at-grade embankment, the ballast mat will reduce soil migration into the ballast and improve soil stability.

Other materials besides rubber or neoprene may be used for a ballast mat. Millbom (Ref. A-32) compares measurement data for ballast and tie track on three layers of 5 cm thick mineral wool sheets with ordinary ballast and tie track, "normal ballast in a concrete construction supported by rubber pads," and track directly fixed to a "sandwich plate" supported by rubber pads. The isolation of vertical vibration provided by the mineral wool installation was evidently similar or not quite as good as "normal ballast in a concrete construction." However, the transverse and longitudinal vibration isolation performance of the mineral wool was evidently superior to all of the other constructions. For these reasons, and because of lower cost and installation time, the ballast and tie track with 15 cm (three layer) of mineral wool ballast mat was selected for vibration control.

Based on the above data and discussion, the ballast mat is a prime candidate for vibration control and ballast maintenance when building new or renewing existing ballasted track. Their effectiveness has not been tested in the U.S. as of this writing. However, substantially similar performance with U.S. transit vehicles and ballast and concrete ties would probably result. No data or discussion concerning their performance on earthen embankments has been found. On earth embankments, their performance might be less than expected for a concrete slab base due to the lower input impedance of earthen subgrades relative to concete track slabs.

5.7 SUBWAY STRUCTURE DESIGN

The design of the subway structure can have a very significant effect on groundborne vibration and noise. A particular subway design may be influenced (but not necessarily determined) by vibration control requirements and for this reason the effect of the transit structure requires discussion.

The basic radiation characteristics of subway structures are not fully understood. Some researchers consider the tunnel wall to be the major source of vibration radiation, while others consider the invert to be the most significant. In addition, there is a possibility that the bending of the subway structure in a beam-like manner is a significant vibration mode. There is a lack of fundamental understanding regarding radiation of groundborne vibration by the tunnel.

The effect of subway structure mass is discussed by Wilson (Ref. A-3) who assumes a single-degree-of-freedom model for the subway supported by the soil. He states that at low frequencies, the subway vibration amplitude is controlled by soil stiffness, whereas at high frequencies, its amplitude is controlled by its mass. Thus at high frequencies, subway structure vibration levels would decrease by approximately 6 dB per doubling of subway mass while at low frequencies, no difference would exist (assuming similar structure dimensions, and therefore, similar soil reactions). Models of subway/soil interaction are discussed in Section 7.

An <u>empirical</u> relation between subway wall overall vibration velocity level and average subway wall thickness has been presented by Koch as (Ref. A-65):

$$A(\mathrm{dB}) = (69 \text{ to } 56) \log (d/40)$$

for values of average subway wall thickness, d, of 40 to 125 cm. The subway wall thickness is an average over the wall, ceiling, and floor panels and is stated to be a measure of overall subway structure mass. The relationship indicates a 17 dB reduction for a doubling of average wall thickness. The relation is based on measurements made by the Curt-Risch-Institute as published in a series of internal reports (Refs. A-23 and A-65). Note that this relation applies to subway wall vibration and that the groundborne vibration radiated may not be directly related to the wall vibration since the invert may be the main source of radiated energy.

Experience in the U.S. and Canada indicates that very significant differences exist between ground surface vibration levels for circular tunnels and for cut-and-cover double box subways. These differences are summarized in Figure 5.17, in which levels for light-weight circular tunnels are plotted relative to those for double box structures. These curves are based on data measured at 50 ft and/or 100 ft from the track centerline at the ground surface at the TTC system (Refs. A-53, A-15, A-14) and at 50 ft at the WMATA system (Ref. A-98).

At the time of the measurements reported in Reference A-15 no differentiation was made between precast concrete and steel tunnel liners, both of which are used in TTC structures. The different tunnel masses involved may influence the results. The steel or cast iron and precast concrete sections can be located randomly, depending on the water table. Hence there is a degree of caution needed when interpreting Figure 5.17. Both the steel and pre-cast TTC tunnels are significantly lighter than the WMATA poured-in-place tunnels.

The WMATA circular concrete tunnels are cast-in-place with a wall thickness of 8 in. to 12 in., as opposed to the TTC precast concrete tunnels with a thickness of about 6 in or less.

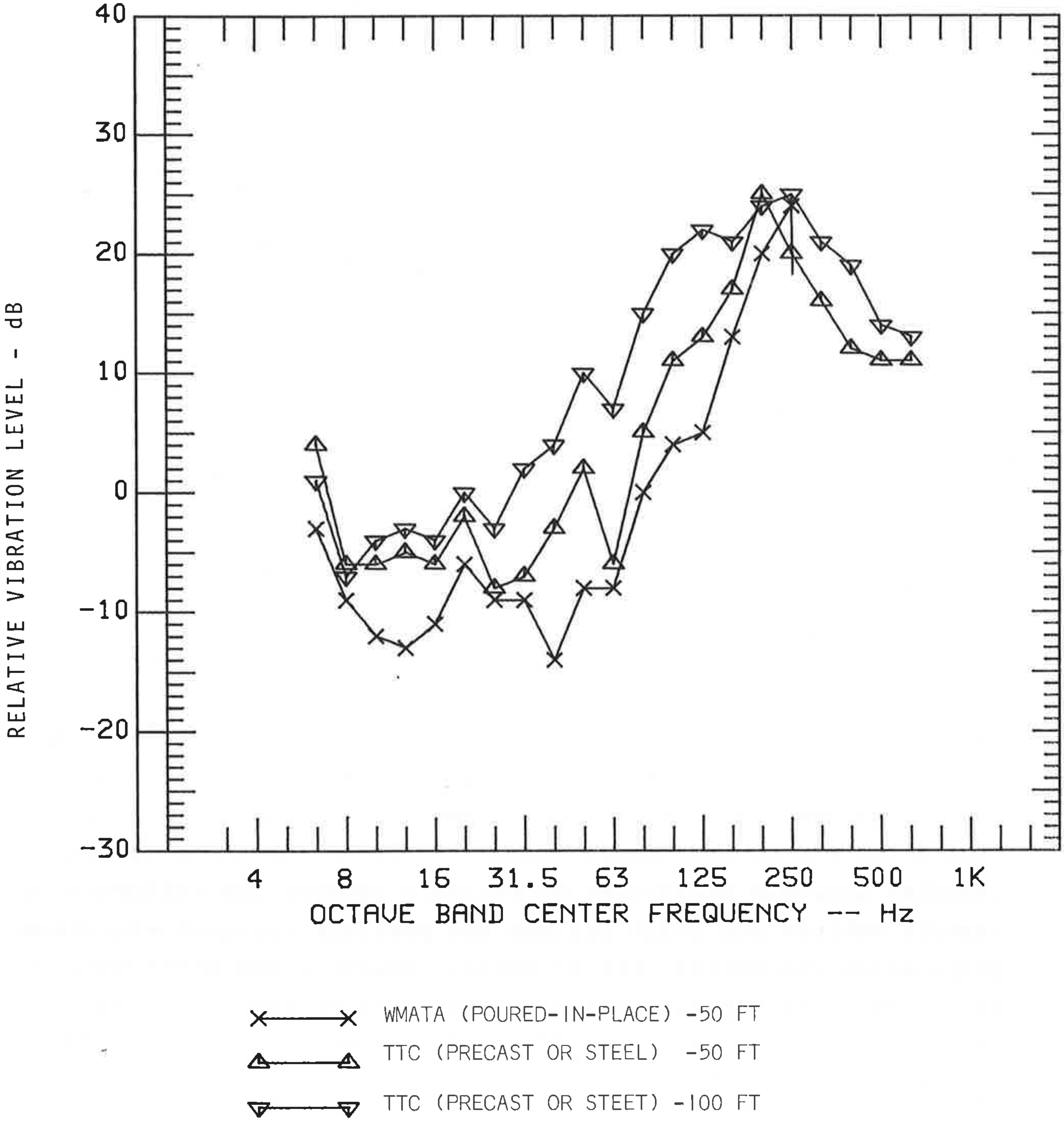


FIGURE 5.17 GROUND SURFACE VIBRATION FROM EARTH FOUNDED CIRCULAR TUNNELS RELATIVE TO CUT-AND-COVER DOUBLE BOX SUBWAYS ON THE TTC AND WMATA SYSTEMS

The same general trends are observed at the WMATA Metro system facilities as at the TTC system. Above 125 Hz the groundborne vibration levels from the circular tunnels are 5 to 20 dB higher than those from the concrete double box tunnels; while below about 31.5 Hz, the levels from the circular tunnels are lower, with a transition region between 31.5 Hz and 125 Hz. The data presented in Figure 5.17 indicate fundamental differences between groundborne vibration from lightweight circular tunnels and the larger, heavier concrete double box structures. This is not surprising in view of the large difference in overall size, subway mass per unit length, subway wall mass per unit area, corresponding bending stiffness, and differences in geometry.

Of recent concern is whether or not tunnels with lightweight precast liners produce lower levels of vibration than heavier cast-in-place circular concrete tunnels. Based on Wilson's model, and Koch's empirical formula discussed above, circular tunnels with precast concrete or steel liners should produce higher levels of vibration than cast-in-place concrete tunnels with thicker and more massive walls. The limited data presented in Figure 5.17 tend to support this conclusion, assuming that subtraction of levels for the cut-and-cover double box structures from levels for circular concrete structures effectively removes the influence of transit vehicle and track designs and possible regional vibration propagation characteristics of soils. However, the WMATA data presented in Figure 5.17 are based on measurements at a single circular tunnel and a single double box subway, so that the data are not at all conclusive.

Although the subway wall thickness, or the mass per unit area, is the most often quoted tunnel parameter, it is likely that the effective mass of the subway invert and/or the overall mass of the structure is a determining factor. Generally, the subway invert mass is much heavier and stiffer than the subway walls, and would increase in rough proportion with other subway parameters, e.g., wall thickness. The effective mass of the invert might include not only invert mass but a portion of the subway wall and an "added" mass contributed by the founding soil.

Thus, although the available information indicates that heavier tunnels tend to produce lower vibration levels at frequencies above 31.5 Hz, the difference may be due merely to the size and mass of the tunnel invert or overall mass of the structure. A tunnel with precast concrete or metal liner may produce groundborne vibration levels comparable with thicker-walled cast-in-place concrete subways provided that the overall masses of the inverts and/or total structures are comparable, although this may be difficult to achieve in practice.

Finally, the level differences given in Figure 5.17 should be placed in proper perspective because the 1/3 octave band groundborne vibration acceleration spectra generally peak in the neighborhood of 50 Hz, while at higher frequencies, the levels are less significant. In fact, significant community reaction has resulted due to groundborne vibration in the 15 to 31.5 Hz frequency range. Figure 5.17 indicates that the circular concrete tunnels may tend to produce lower levels of vibration in this frequency range than the larger cut-and-cover structures. This is an interesting result that is an indication of the complexity of the effect that tunnel wall thickness has on the levels of groundborne vibration. Clearly the influence of tunnel wall thickness is an area requiring more study.

5.8 SCREENING

Use of underground screens, or barriers, is a method for control of rapid transit groundborne vibration which has not received much attention in the United States. It is essentially analogous to controlling airborne noise with a sound barrier, and many of the same design considerations apply. With varying degrees of success, screening has been used for control of ground vibration from heavy machines.

Depending on the relative mass of the screen and the surrounding soil, screening techniques may be classified as either trenches or solid barriers. The first group includes open trenches and trenches filled with a lightweight waterproof filler such as styrofoam. The second group includes sheet piling and concrete walls poured into trenches. In both cases, the basic idea is to provide an impedance mismatch in the soil so as to interrupt surface or Rayleigh wave propagation. For body waves (e.g., compression and shear) the screens must extend to greater depth than for surface waves. Accordingly, before deciding whether or not screening may be effective, the wave types and wavelengths must be determined.

Examples of unsuccessful screening to isolate machine vibration are described by Barkan (Ref. B-8). Investigating the reasons for these failures, Barkan concludes that the screen depth should be at least one-third of the wavelength of the propagating wave, and that the depth should be measured from the bottom of the source of vibration. For most soils the velocity of Rayleigh wave propagation is on the order of 200 m/sec; for significant isolation at 20 Hz, the depth of the screen should be at least 3 to 4 m below the bottom of the source. Barkan further states that trenches should not be used if ground water will collect in the trench. In such cases, sheet piling might be substituted. If sand is found at the site, Barkan suggests forming a screen by chemical or cement stabilization of a narrow strip of soil.

Richart et al. (Ref. B-4) distinguish two types of isolation by (1) active isolation-trenches close to or completely surrounding the vibration source-and (2) passive isolation-trenches which are distant from the source but near the site where the vibration amplitude is to be reduced. They detail tests of these barrier types and specify two general criteria regarding the amplitude reduction and effective area. These criteria are summarized as:

1. Active Isolation: The trench is considered effective if the amplitude of ground vibration is reduced by 75% (12 dB) beyond the trench extending to a distance of about ten Rayleigh wavelengths from the trench. To achieve this, the trench bottom must be at least six-tenths of the Rayleigh wavelength.

2. Passive Isolation: The trench is considered effective if the amplitude of vertical ground surface vibration is reduced by 75% (12 dB) within a semi-circle of radius equal to one-half of the trench length extending from the center of the trench. For trenches located between two and seven Rayleigh wavelengths from the source, the trench depth must be 1.3 times the Rayleigh wavelength. The product of the trench height and length should be at least 2.5 times the square of the Rayleigh wavelength if the trench is two wavelengths from the source; it should be increased linearly with such distance to at least six times the square of the Rayleigh wavelength at seven wavelengths from the source.

Richart et al. further state that the trench dimensions must be determined from a knowledge of the frequency of vibration and propagation velocity, which together determine the wavelength. The propagation velocity should be measured in situ using standard geophysical methods or estimated from data on soil properties and confining pressure. Finally, soil layering, a high water table, or building foundation design may alter wave propagation characteristics and reduce the effectiveness.

Richart et al. (Ref. B-4) indicate that the width of the trench has little effect on screening effectiveness. Quite the contrary is reported for solid barriers by Haupt (Ref. B-36) who identifies a very strong dependence of barrier insertion loss, for solid barriers in sand, with the ratio of the cross-sectional area of the barrier and the square of the Rayleigh wavelength. This conclusion is based on both a two-dimensional finite element model and experimental modeling with a sand pit. The results of the modeling experiments which support this conclusion are presented in Figure 5.18. The barrier analyses described by Haupt were for barriers located in the far field of the vibration. Greater vibration reduction is achieved if the barrier is located in very close proximity to the vibration source.

Dolling (Ref. A-122) finds the theoretical vibration isolation of open trenches to be independent of the relative location of the trench between the source and receiver provided that the trench is in the far field and that the receiver is far from the trench. Dolling also indicates that the vibration reduction of open trenches is independent of the cross-section of the trench, consistent with Richart et al, and contrary to results reported by Haupt for solid barriers.

Little data exist concerning the use of trenches for control of groundborne noise and vibration from rapid transit systems. A trench filled with styrofoam has been constructed and tested by

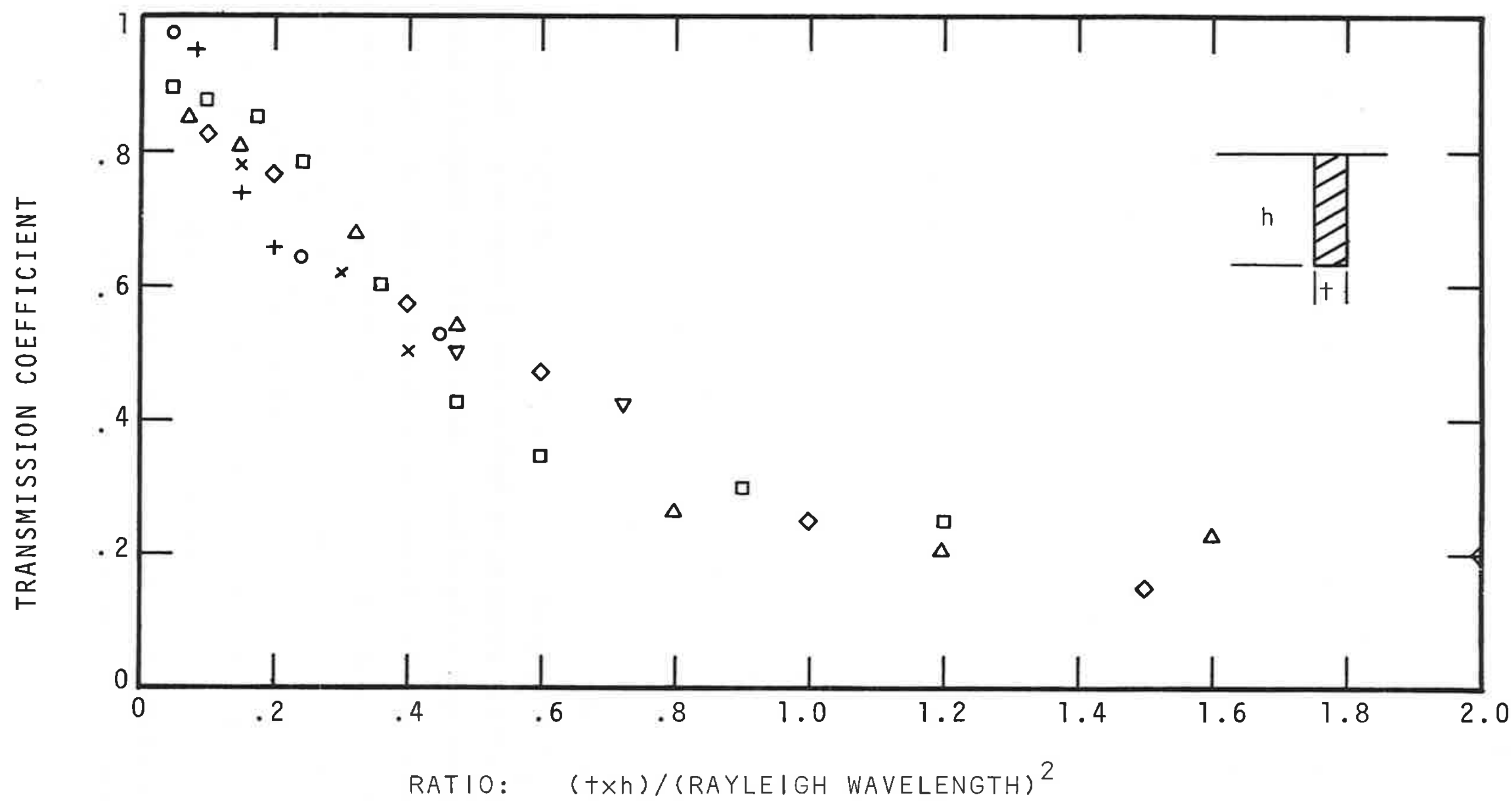


FIGURE 5.18 TRANSMISSION COEFFICIENT FOR SOLID BARRIERS (ADAPTED FROM REF. B-36)

the Toronto Transit Commission with evidently successful results which have been maintained over a period of at least one year (Ref. A-177).

Morii of the Japan National Railway presents measurement data for the reduction of groundborne vibration acceleration by two rows of sheet piling driven near a Shinkansen aerial structure (Ref. A-53). Evidently, the reduction is significant but less than 8 or 10 dB between 10 and 20 m from the aerial structure. Additional data are presented for two rows of continuous sheet piling driven at 3.5 m from an "existing line" (evidently at-grade ballast and tie). Vibration reduction of about 5 dB at 5 m and 10 dB at 7 m from the track are given for piles driven to a depth of 5 m. For piles driven to a depth of 3 m the reduction is much less at these distances. The data are rather sketchy and no spectra are presented, making interpretation difficult.

Part of the problem in assessing the published data on the effectiveness of screening on groundborne vibration is that most vibration reductions are given in overall acceleration and/or velocity magnitudes. The primary focus is usually to reduce the fundamental components of ground vibration due to heavy machine operation. In these cases, the offending frequency component may be at 10 Hz or less, generally below the frequency range of groundborne noise and vibration. An effort to compile additional measurement data, either from the literature or by testing, would be desirable to define the effect of screening in the frequency range of groundborne noise and vibration from transit systems. Also, although screening may be particularly effective for ballast and tie track or aerial structures, screening is in most cases totally impractical for subways. However, isolation of specific buildings at a large distance from the subway may be effective if a well developed Rayleigh wave can be expected. Thus, additional work could be directed towards identifying the distance from a subway structure beyond which a "passive" screen may be effective.

5.9 ISOLATION OF BUILDING STRUCTURES

Isolating building structures is achieved by placing vibration isolators between the foundation and structure. Such isolation can consist of large rubber pads placed beneath building columns of large multi-story buildings, or ribbed rubber or neoprene strips between the foundations and plates of wood-frame residential structures. Experience indicates that such isolation can be very effective, and, contrary to popular misconception, does not result in a "springy" building.

Technically speaking, vibration isolation of buildings might include the use of trenches or sheet piling as described above. However, this subsection is devoted to the use of springs to support the building structure and isolate it from transit induced vibration.

A more complete discussion of the general problem and literature concerning building vibration isolation, including human response and applicable criteria, is presented by Steffens (Ref A-85).

A general discussion of vibration isolation of buildings is given by Waller (Ref A-80) who indicates that one of the major uses for isolation is to reduce groundborne noise and vibration from railroads and rapid transit systems. According to Waller, the general design of building isolation systems is based on the assumption of simple rigid body motion of the structure mass, including effects of vertical, horizontal, and rocking modes. This approach is perhaps reasonable as a first approximation, but "local" resonances of walls, floors, and ceilings may coincide with spectral components of ground vibration and nullify the effect of the isolation. Allen et.al. (Ref. A-75) discusses the problems of vibration isolation of buildings constructed of lightly damped beam and plate elements. In such cases, the isolation may be compromised by resonant amplification of these building elements.

If the load impedance of the structure as "seen" by the spring is similar to or less than the spring impedance, little or no reduction of building vibration will be obtained. Northwood (Ref. B-29) indicates that buildings founded on soil have a fundamental resonance between 0.5 to 1 Hz; in a sense the building is already isolated. In fact, if significant reduction of building vibration is to be achieved, the total compliance of the isolators must be significantly greater than that of the soil. If appropriate attention is not paid to the soil compliance, little or no vibration isolation may be achieved. Conversely, buildings built on columns or foundations resting on stiff bedrock may benefit greatly from a building isolation system.

Waller (Ref. A-80) discusses vibration isolation applied directly to a multi-story apartment building constructed directly over the St. James Park Underground Station, London. In this case, laminated natural rubber constructions were used for the springs, the largest measuring 24 in. by 24 in. by 11.5 in. The vertical natural frequency was estimated to be about 7 Hz, with an initial deflection of 0.4 Hz. The horizontal natural frequency for these springs alone was estimated to be about 0.5 Hz, which was likely to produce an undesirable response to wind. The horizontal natural frequency was therefore increased to 2.5 Hz with use of three pairs of rubber springs sandwiched between vertical steel plates. A higher horizontal resonant frequency might have resulted in coupling between the vertical and horizontal vibration leading to reduced overall performance of the isolation system. Measurement data are presented for building vibration and ground vibration, but no clear estimate of insertion loss is given.

In a second example, Sowry (Ref. A-91) discusses the control of train induced vibration and noise. Two-hundred natural-rubber-and-steel laminated bearings were positioned atop the support columns of a multi-story hotel under which run heavy express trains. The bearings were surrounded by 10 cm thick asbestos coverings for fire protection, and horizontal resonances were controlled by fourteen stabilizing wind braces using additional bearings. Again, no data concerning the isolation effectiveness of the system is presented.

In almost all applications, natural rubber springs with steel reinforcement are selected over coil steel springs. The reason is that rubber is not subject to corrosion and provides a higher degree of damping than steel springs. Also, natural rubber is less subject to creep than other competing polymers.

One of the significant problems encountered with constructing buildings on isolation systems is that rubble is sometimes left between the foundation and structure with resultant shorting of the resilient element. In one unpublished case, the building contractor mistakenly poured concrete around the isolators not realizing their significance.

To summarize, buildings have been isolated from subway induced vibration by use of isolation springs. However, the effectiveness of these isolator systems is difficult to assess, and careful consideration must be given to local resonances of walls, floors, ceilings, beams, and columns, as well as to the compliance of the foundations and buildings relative to the spring. Improper design can easily lead to undesirable performance.

6. VIBRATION PREDICTION METHODS

A variety of prediction methods have been used by transit system designers for prediction of groundborne noise and vibration. Perhaps the main reason for this variety is that the mechanics of subway vibration radiation, wave type, and building response are very complex and difficult to model, and are not well understood. Also, accurate prediction of the magnitude of vibration and noise at specific sites requires detailed knowledge of soil conditions (including layering and water saturation) and the design of the affected buildings. Unfortunately, these data are rarely, if ever, readily available. In practice, prediction methods have relied upon in-house experience and generally accepted propagation laws, without detailed analytical modeling.

One of the major differences between prediction methods is the treatment or characterization of the source. Ungar and Bender (Ref. A-2) assume the subway wall and/or ceiling is the major source of vibration, while Tokita et al (Ref. A-4) consider an equivalent vibration source in the soil beneath the subway structure. In contrast to this approach, Wilson (Ref. A-3) starts with measured vibration at about 7.5 m from the subway structure, thus bypassing questions of source location and subway soil coupling.

Propagation of vibration and resulting attenuation is also modeled differently by different researchers. Reference A-2 employ detailed soil stiffness and damping data where available, including effects of layer thickness, while Wilson (Ref. A-3) suggests empirical attentuation curves as a funtion of frequency for a typical soil. Nolle (Ref. A-78) extends the empirical approach with the aid of regression analysis to determine a set of constants for an assumed propagation law as a function of 1/3 octave band frequency.

Other prediction methods are concerned primarily with A-weighted noise levels in buildings. Lang (Ref. A-29) has shown that the A-weighted noise level in cellars falls within about 10 dB of a best fit curve of level as a function of distance from the subway structure. However, current experience at major transit systems indicates that "feelable" low frequency vibration in the 10 to 31 Hz range can be as much of a problem as the A-weighted noise level (Ref. A-11, A-36, A-73, A-109).

The purpose of this section will be to identify the salient features of each prediction method and to identify where any consensus exists. In addition, areas where the prediction methods could be modified to improve the prediction accuracy are also discussed.

6.1 PRESENT METHODOLOGY

Each of the methods encountered in the literature are summarized below. Some of these methods, e.g., the method used by Wilson and by Ungar and Bender, are undoubtedly subject to revisions by their authors as new data and/or models become available. This should be kept in mind when reviewing the methods. Each method is referred to by the author of the source literature.

6.1.1 Wilson (Wilson, Ihrig & Associates)

Wilson's method was initially developed from measurements performed at BART and TTC (Toronto) and described in the report "Noise and Vibration Characteristics of High Speed Transit Vehicles" (Ref. A-3). The method was later revised for prediction of vibration from both rock- and earth-based subways at the WMATA Metro (Ref. A-5) and only minor changes have since been incorporated. The method has been used for general design review at MARTA, BRRT, and NFTA. It is important to recognize that this method was developed to provide a conservative prediction of the levels of groundborne noise and vibration. That is, it estimates the "highest expected" level of groundborne noise and vibration and not an "average" level.

Source: The starting point of Wilson's method is the expected range of vertical octave band vibration acceleration levels expected at about 10 m from the track centerline of double box subways during passage of 8-car WMATA Metro trains traveling at 110 to 120 km/hr. Three spectra are considered, each for a given subway founding condition:

1. Vibration at the ground surface for earth-based subways.

2. Vibration at the ground surface for rock-based (mixed-faced) subways.

3. Vibration in rock for rock-confined subways.

These spectra are presented in Figure 6.1. Standard resilient track fasteners giving a rail support modulus of about 4000 lbs/in^2 of rail are assumed.

Corrections: Corrections are used to account for train speed, subway mass, and special trackwork, and are added to the starting spectra presented in Figure 6.1. These corrections are presented in Table 6-1 and apply across the entire octave band spectrum.

Additional octave band corrections are used to account for vibration control provisions, such as the RS-STEDEF Ballastless Track system, and floating slab vibration isolation systems. These corrective curves are illustrated in Figure 6.2 and are to be added to the curves of Figure 6.1. These corrections represent revisions as of 1975.

Propagation: Attenuation as a function of distance is presented in Figure 6.3. The data were determined from measurements at TTC and BART and apply to "typical" soils. However, the soils at the TTC system may be relatively stiff compared to soils at other systems (Ref. 6.14) and may have relatively light damping properties. Note that the curves are ranges of vibration expected at 3 m, 15 m, and 30 m relative to 10 m from an earth-based or mixed-face subway track centerline.

For rock-tunnels, the dissapation loss is considered insignificant compared to spreading losses, and, in this case, train length becomes important. The resulting correction curve is illustrated in Figure 6.4 for 2- and 8-octave band vibration spectra for rock-tunnels. The correction applies to all frequencies.

Building Response: Estimated coupling losses for various building designs are presented in Figure 6.5 for three foundation categories; masonry buildings on spread footings, large masonry buildings on piling, and residential structures on spread footings. For cellar walls and floors or slab-on-grade floors, the coupling loss is assumeed to be negligible. The estimated amplification of vibration by floor slabs supported on columns or

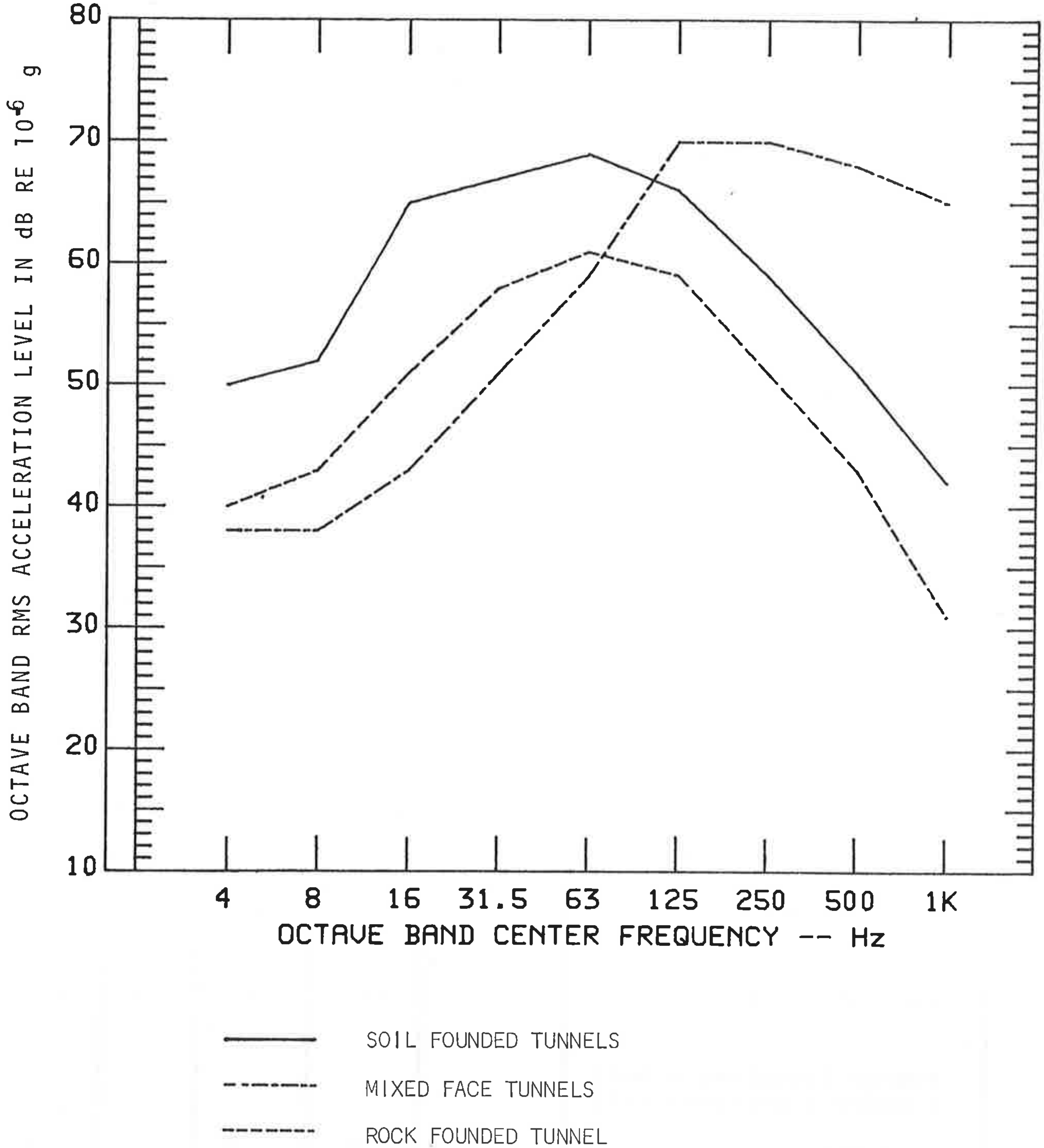


FIGURE 6.1 MAXIMUM EXPECTED VIBRATION ACCELERATION LEVELS 25 TO 30 FT FROM TRACK CENTERLINE AT THE TOP-OF-RAIL (THESE CURVES ARE FOR USE IN FUTURE PROJECTIONS OF GROUNDBORNE NOISE AND VIBRATION AT WMATA METRO 8-CAR TRAINS AT 70 TO 75 MPH)

TABLE 6-1 CORRECTION FACTORS TO BE APPLIED TO THE EXPECTED VIBRATION LEVELS INDICATED ON FIGURE 6.1

Factor Affecting Vibration Level	Relative Vibration Level (decibels)
All Tunnels	
Train Speed - 75 mph	0
60 mph	-2
50 mph	-3.5
45 mph	-4
40 mph	-5
Curves	+3 to +5 (with guardrail)

GROUNDBORNE VIBRATION LEVEL ADJUSTMENTS FOR SPECIAL TRACKWORK AND TYPE OF STRUCTURE

Factor Affecting Vibration Level	Relative Vibration Level - decibels Octave Band Center Frequency - Hz 16	31.5	63	125	250
Special Trackwork	+10	+10	+10	+10	+10
Subway Structure - soil founded structures only					
Double box	0	0	0	0	0
Single box or concrete tunnel	-3	-3	0	+5	+5
Cast iron or steel tunnel	-3	-3	+1	+6	+6
Triple box or crossover structure	0	0	-2	-2	-2
Station	0	-1	-3	-4	-4

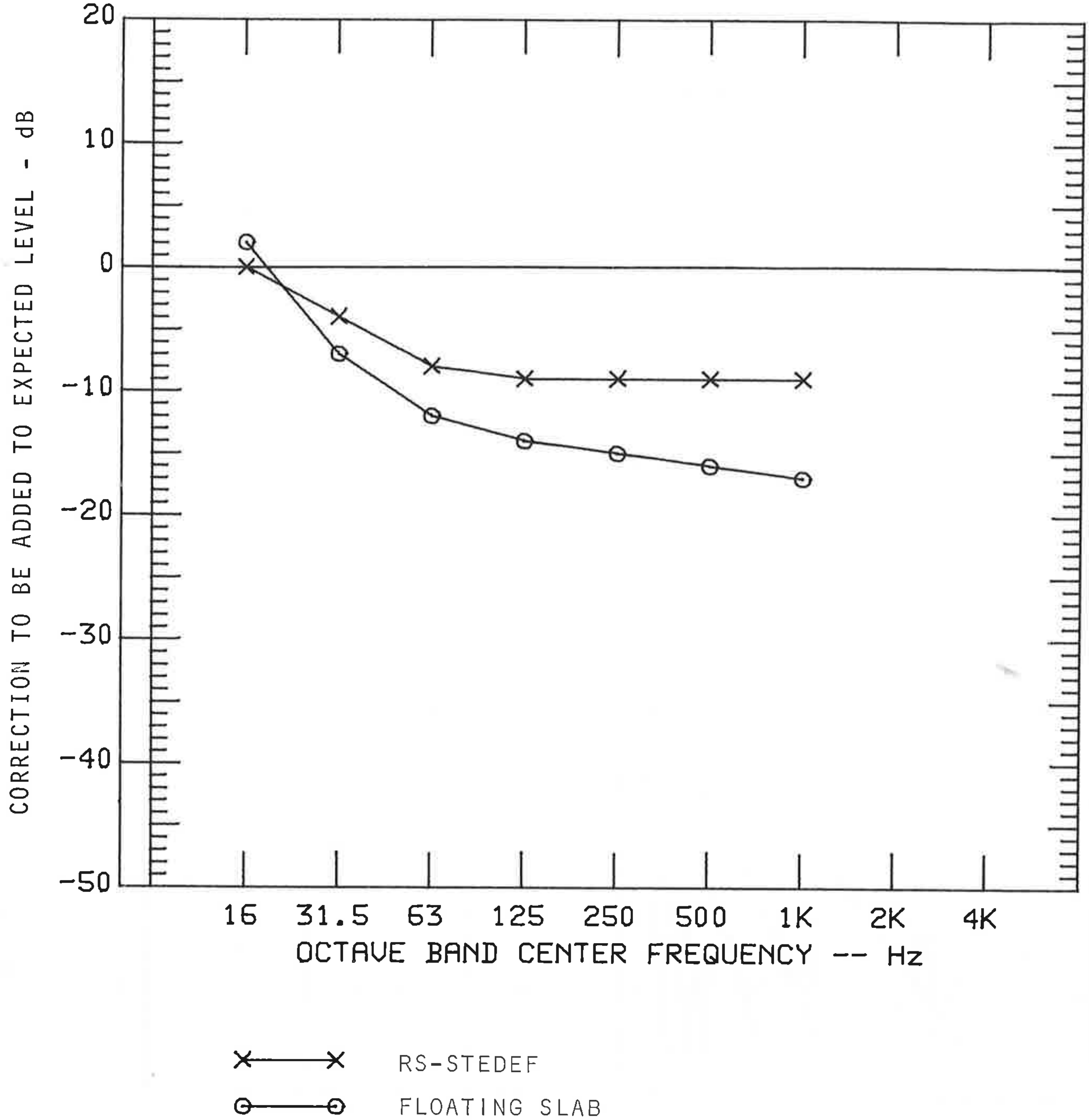


FIGURE 6.2 GROUNDBORNE VIBRATION FOR RS-STEDEF AND FLOATING SLAB VIBRATION ISOLATION SYSTEM RELATIVE TO WMATA RESILIENT DIRECT FIXATION FASTENERS

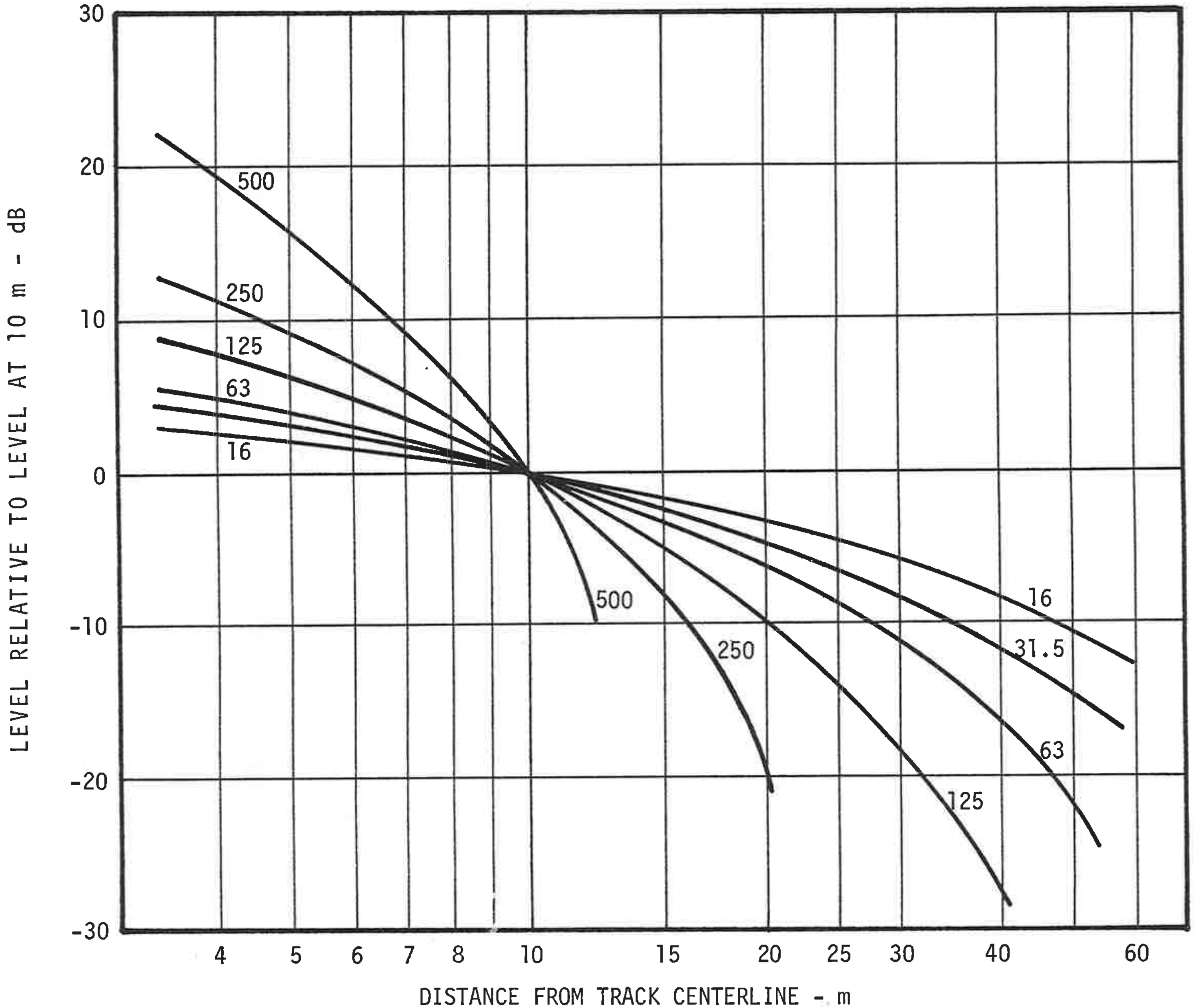


FIGURE 6.3 APPROXIMATE GROUND VIBRATION LEVELS RELATIVE TO THE LEVELS IN SOIL AT 10 m (33 FT) FROM A SOIL BASED OR MIXED FACE SUBWAY

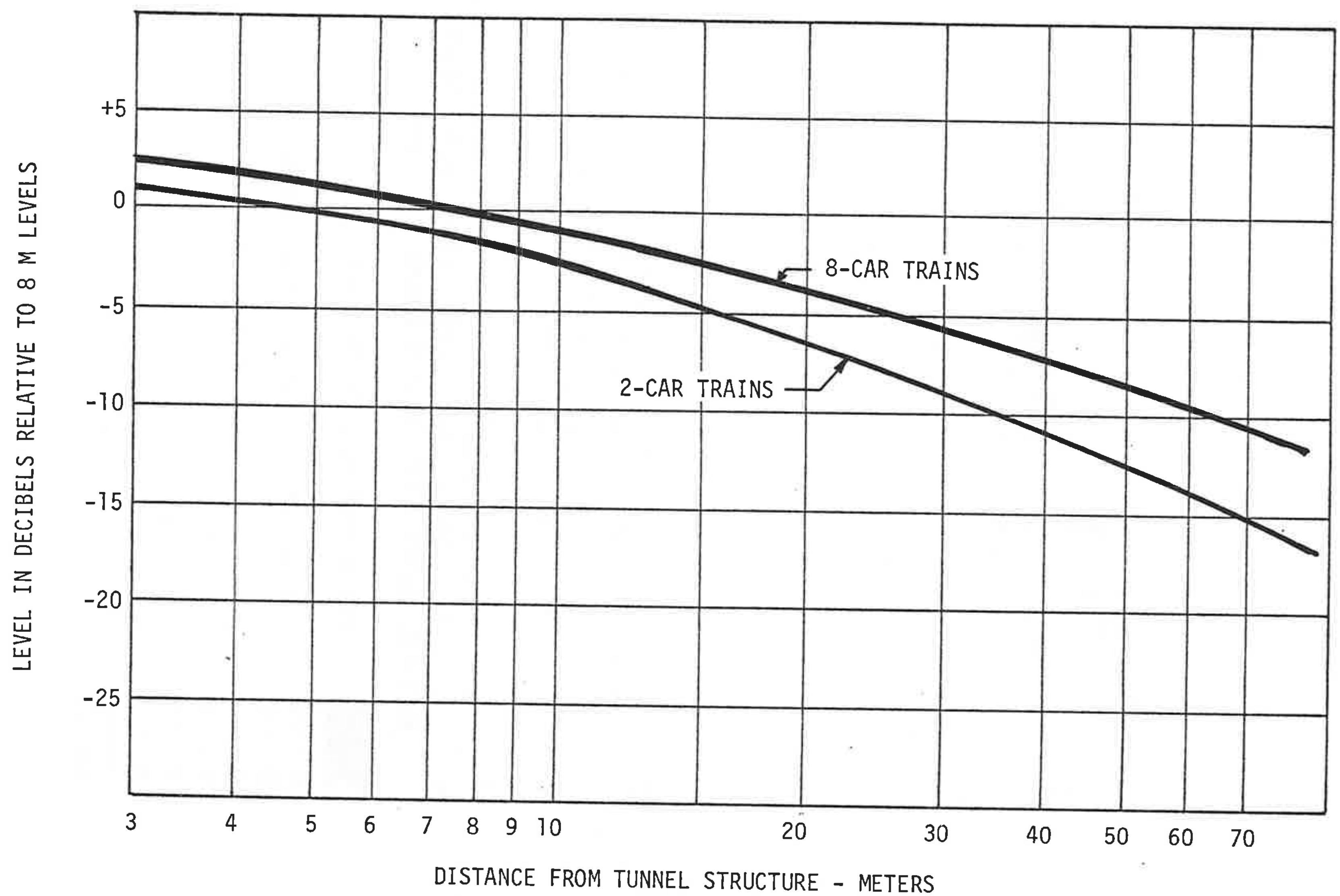


FIGURE 6.4 VIBRATION LEVELS IN ROCK AS A FUNCTION OF DISTANCE FROM A ROCK TUNNEL STRUCTURE

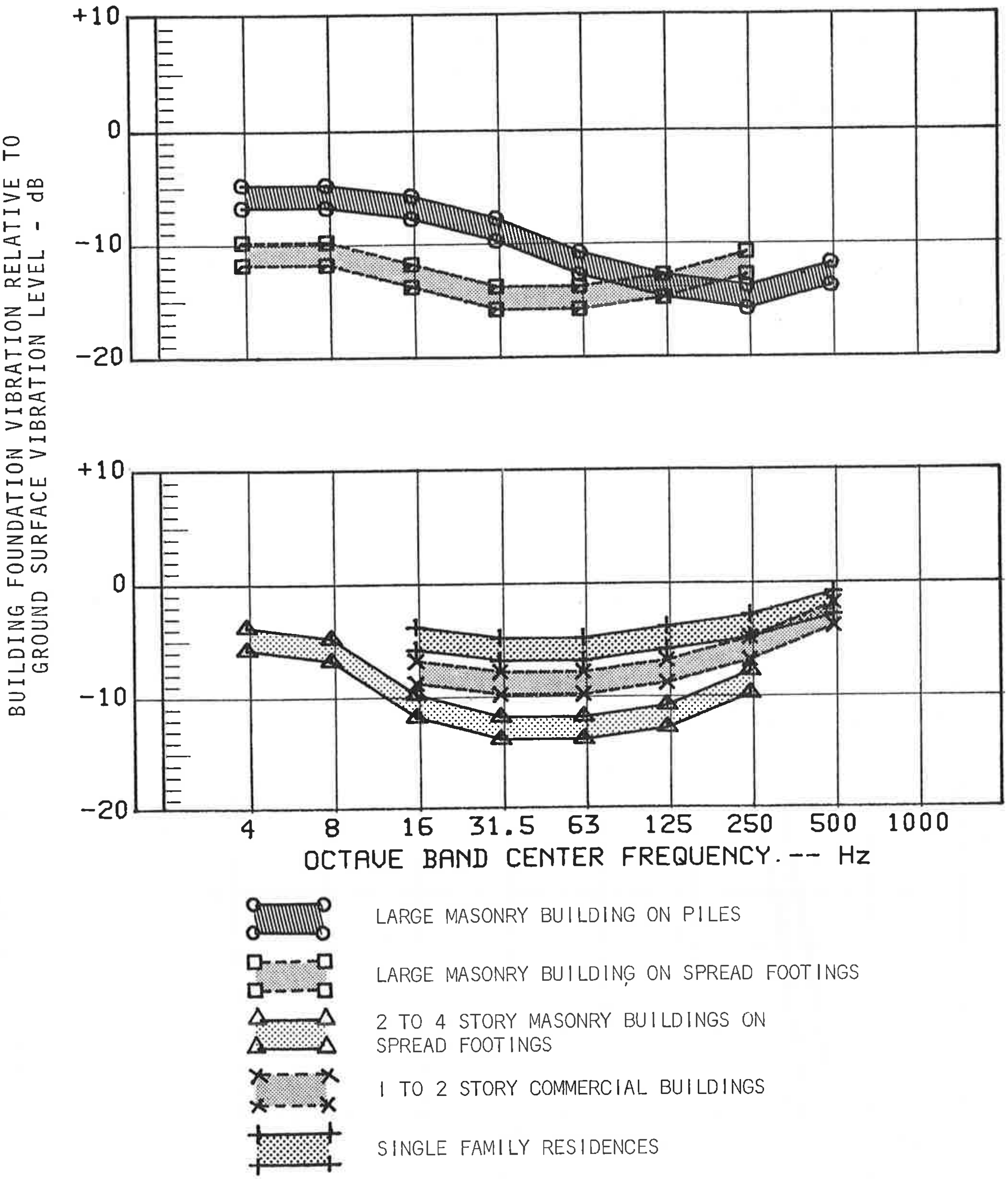


FIGURE 6.5 COUPLING LOSSES FOR BUILDING FOUNDATIONS

shear walls is presented in Figure 6.6. Note that the expected amplification due to resonances in the 10 to 40 Hz range is approximately 10 dB.

Also shown in Figure 6.6 is the floor-to-floor attenuation for large multi-story structures. Measurements reported by Ishii and Tachibana (Ref. A-148) have recently been incorporated for estimation of floor-to-floor attenuation as summarized in Table 6-2. Note that Table 6-2 indicates a reduction for the first few floors comparable with that given in Figure 6.6, e.g., about 3 dB per floor from 31 to 250 Hz. However, above the 8th floor the attenuation is reduced to 1 to 2 dB from 31 to 250 Hz.

Figure 6.7 illustrates the expected range of octave band sound pressure levels within rooms relative to the average octave band vibration acceleration of the floor. For rooms with little absorption, the upper part of the range is used, while for rooms with a large amount of absorption, the lower part of the range is used. A-weighted noise levels are then computed from the octave band sound pressure level spectra.

The foregoing discussion essentially summarizes the approach used by Wilson for prediction of groundborne noise. The method necessarily accounts for spectral effects and is useful for comparison with NC criteria. Alternatively, the A-weighted level may be computed and used for comarison with other criteria.

6.1.2 Ungar and Bender (Bolt, Beranek, and Newman)

The method of Ungar and Bender was determined from published literature and engineering reports for WMATA Metro and NYCTA (Ref. A-2, A-158). This method has been used by Copley for

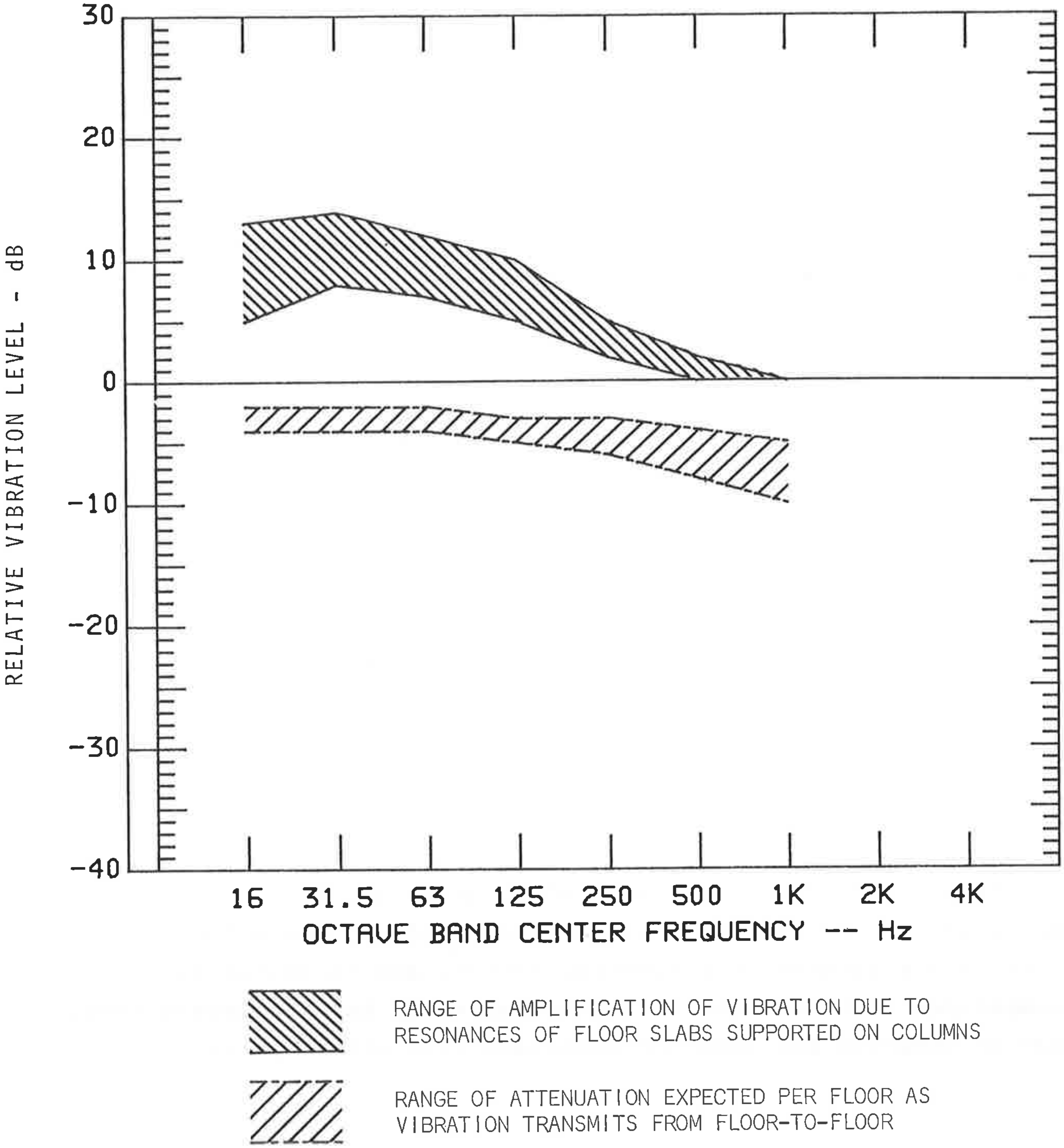


FIGURE 6.6 CORRECTIONS TO BE ADDED TO EXPECTED VIBRATION LEVEL DUE TO FLOOR VIBRATION AMPLIFICATION AND FLOOR-TO-FLOOR ATTENUATION

TABLE 6-2 POINT SOURCE BELOW BUILDING - ATTENUATION PER FLOOR WITH ACCELERATION IN dB

Frequency Hz	Floor Level Above Grade									
	1	2	3	4	5	6	7	8	9	10
	Floor-to-Floor Distance: 10 ft									
31	2	2	2	1	1	1	1	1	1	1
63	3	2	2	2	2	1	1	1	1	1
125	3	3	2	2	2	2	2	1	1	1
250	3	3	3	3	3	3	3	2	2	2
500	4	4	3	3	3	3	3	3	3	3
1K	5	5	4	4	4	4	4	3	3	3
	Floor-to-Floor Distance: 12 ft									
31	2	2	2	2	1	1	1	1	1	1
63	3	3	2	2	2	1	1	1	1	1
125	3	3	3	2	2	2	2	1	1	1
250	4	4	3	3	3	2	2	2	2	2
500	4	4	4	4	4	3	3	3	3	3
1K	5	5	5	4	4	4	4	4	4	4

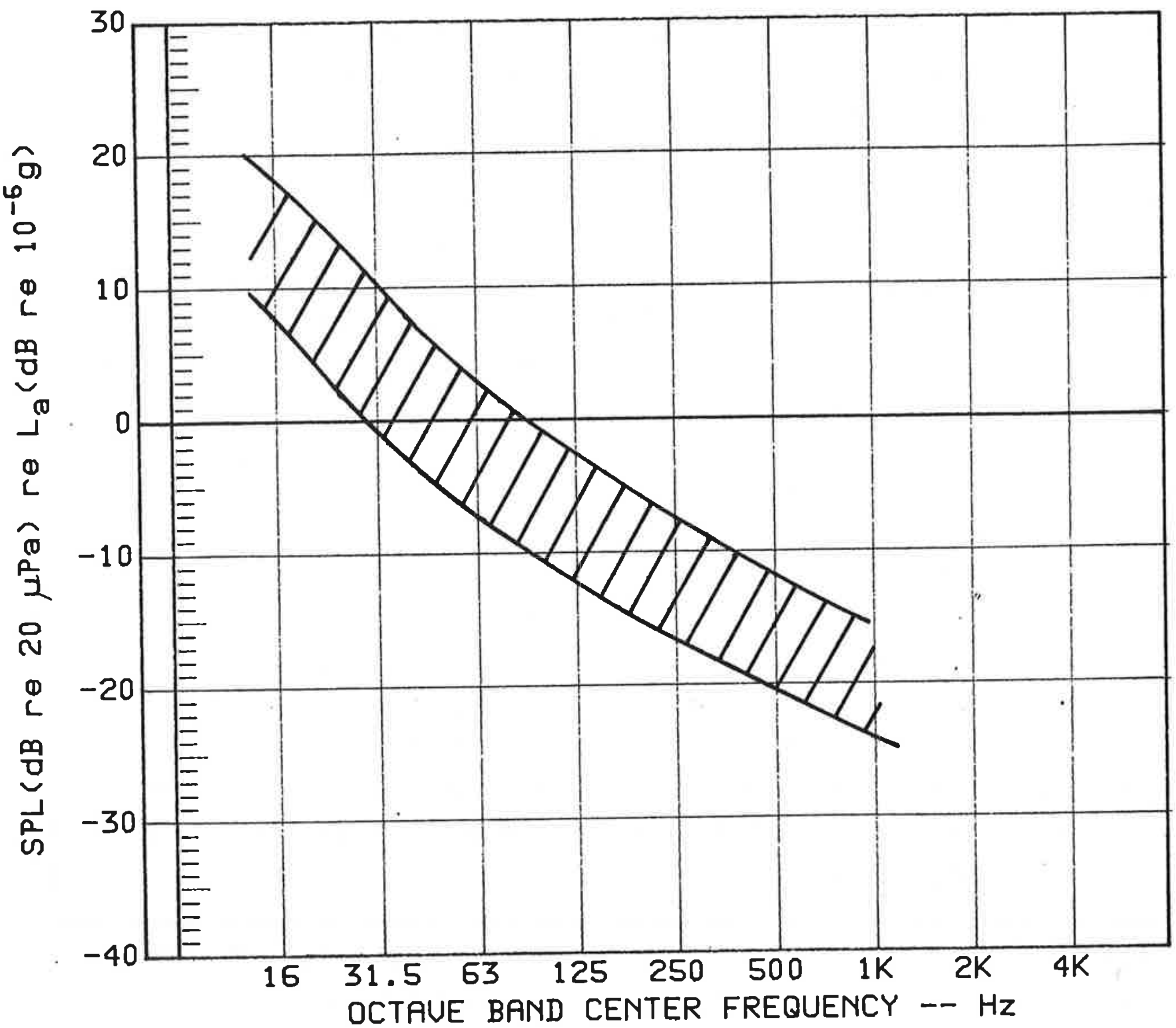


FIGURE 6.7 ROOM INTERIOR SOUND PRESSURE LEVEL RELATIVE TO AVERAGE FLOOR ACCELERATION LEVEL

estimation of vibration near MBTA with some modification (Ref. A-71). Tokita (Ref. A-4) also refers to the method, but assumes an entirely different type of source, e.g., an equivalent source below the subway structure.

The method is a conservative method based on an upper bound spectrum for subway trackbed and wall vibration as determined from experimental data from NYCTA, Paris Metro, and TTC. Corrections are then applied to estimate the subway wall vibration levels and effects of propagation. The salient points of the method are discussed below.

Source: The subway wall is assumed to be the primary source of groundborne vibration radiated horizontally away from subway structures. The starting point of the prediction method is an upper bound reference octave band acceleraton spectrum derived for the subway wall from subway trackbed vibration data collected at NYCTA, Paris, and TTC subways. The reference spectra for trackbed and subway wall vibration are presented in Figure 6.8 for subways with 35 mph trains operating on jointed rail and "stiff" fasteners giving a rail support modulus of about 20,000 lb/in^2 of rail.

Corrections: Reduction of rail support moduli below 20,000 lb/in^2 of rail gives a reduction of groundborne vibration levels in the audible frequency range of -20 Log K/20,000, where K is the rail support modulus. In the "feelable" low frequency range, the reduction is -5 Log K/20,000.

Elimination of major discontinuities, such as rail joints, is estimated to reduce groundborne vibration by 5 dB in the low frequency "feelable" portion of the spectrum and by 10 dB in the high frequency audible portion. These estimates are based on

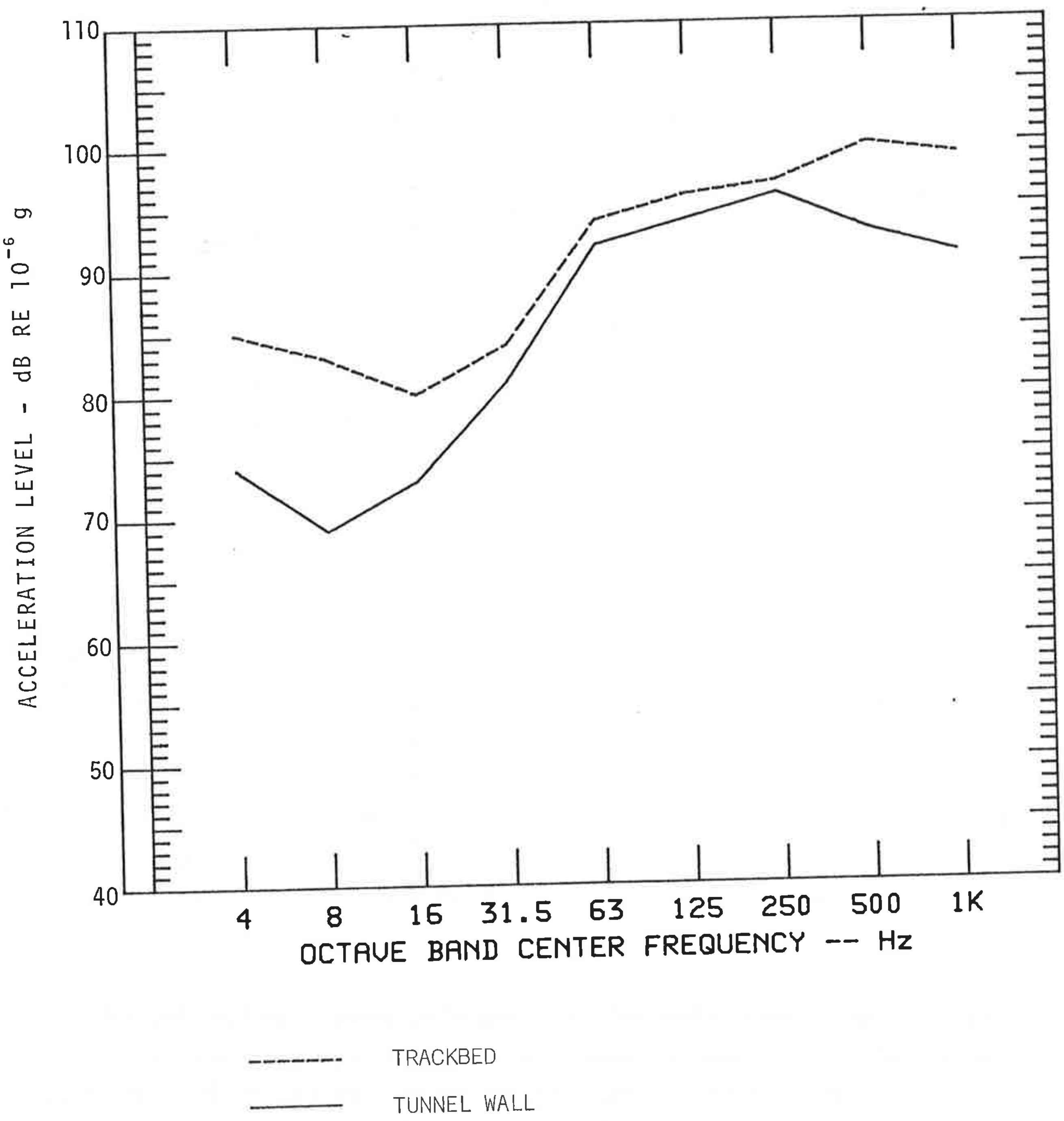


FIGURE 6.8 REFERENCE UPPER BOUND VIBRATION LEVELS [35 MPH TRAIN, TUNNEL IN ROCK, JOINTED TRACK ON STIFF FASTENERS] [AFTER UNGAR AND BENDER REF A-2]

observed reductions of wayside noise levels.

Unsprung vehicle weights are assumed to be similar for all vehicles and are, therefore, believed to have an insignificant effect on groundborne noise and vibration levels from one transit system to the next.

Above 40 mph doubling of vehicle speed is assumed to increase groundborne vibration by about 4 dB. An increase of 6 to 8 dB per doubling of train speed is expected at train speeds between 20 and 40 mph.

Propagation: The soil immediately surrounding the subway structure vibrates at the same amplitude as the subway wall. Ungar and Bender distinguish between near-field and far-field soil vibration and make the conservative assumption that all of the near-field vibration contributes to the far-field. The amplitude of vibration as a function of distance from the subway is then estimated as the sum of spreading loss and dissipation.

The subway structure is assumed to be a line source for estimation of spreading loss, giving a level reduction in dB of

$$A_s = 10 \text{ Log } (1 + x/r_o)$$

where, x is the distance from the structure wall to the point in question and r_o is the distance from the tunnel center to wall surface, or the effective radius of the structure.

The attenuation due to dissipation is modeled by

$$A_d = 27.3 \text{ fxn/c}$$

where, f is the frequency in Hz, n is a dimensionless loss factor, x is the distance from subway wall, i.e., the distance of propagation through the soil, and c is the velocity of propagation. The variables x, c, and f must have consistent units.

Both shear and dilational waves are assumed to have the same loss factor n with the same initial amplitude at the subway structure. Because the velocity of propagation for dilitational waves is generally much higher than shear waves, giving rise to longer wave lengths, the attenuation rate for shear waves will be much more than for dilitational waves. The result is that only the dilitational wave and its corresponding propagation velocity is considered significant for estimation of the dissipation loss. For simplicity, the various earth media are grouped into three classes with representative dilitational wave speeds, loss factors, and densities, as given in Table 6-3.

The loss caused by propagation across soil layer interfaces is modeled for a "first estimate" as

$$A = 20 \text{ Log } [\frac{1}{2}(1 + \frac{\rho_b C_b}{\rho_a C_a})]$$

where a, b refer to the two differing soils, ρ_a and ρ_b are the soil densities, and C_a and C_b are the dilitational wave propagation velocities within soils "a" and "b". The wave travels from soil a to soil b. Multiple layered soils are also treated with correspondingly more complex formulas.

For subways founded on or embedded in rock, the subway structure vibration is estimated to be less than that for soil by the factor

TABLE 6-3 WAVE PROPAGATION PROPERTIES OF TYPICAL SOILS

Soil Class	Longitudinal Wavespeed c(ft/sec)	Loss Factor η	Density ρ (g/cm^3)
1. Rock	11,500	0.01	2.65
2. Sand, silt, gravel, loess	2,000	0.1	1.6
3. Clay, clayey soil	4,900	0.5	1.7

$$a_r/a_s = (Z_{st} + Z_s)/(Z_{st} + Z_r)$$

where a_r and a_s are the acceleration of subway structures in rock and in soil, respectively, Z_r and Z_s are the rock and soil impedances, respectively, and Z_{st} is the subway structure impedance. The ordering of these impedances is assumed to be $Z_{st}, < Z_s, < Z_r$. Z_r is assumed to be approximately 10 times Z_s. Thus,

$$a_r/a_s \cong 0.1$$

That is, rock subway structure vibration is predicted to be about 20 dB less than for soil founded subways.

Building Response: The level of cellar wall vibration is assumed to be equal to that of the incident groundborne vibration. More complex relationships are used for estimating the response of buildings mounted on piles. For this latter case, experimental data were used to prepare an idealized curve of "coupling loss" for piles. Here, coupling loss is the difference between vibration level which would exist in the absence of the piles. This coupling loss is illustrated in Figure 6.9.

The effect of building load on the pile is represented by the relation

$$a_p/a_o = M_B/(M_B + M_p)$$

where a_p is the actual acceleraton of the head of the pile, a_o is the acceleration of the head of the free pile, M_B is the mobility of the attached building, and M_p is the mobility of the pile. As a first approximation, the mobility of the building corresponds to that part of the building mass supported by the pile with an elastic pad of stiffness K_B inserted between the pile and building mass. This model was used to derive the curves given in Figure 6.10 for the difference between building and free-pile acceleration levels for various values of building isolator static deflection. The formula used to generate these curves is presented in Reference A-158. Note that the curves presented in Figure 6.10 are to be added to the coupling loss given in Figure 6.9.

The floor-to-floor attenuation is 3 dB per floor, substantially the same attenuation rate used by Wilson.

The interior sound pressure level arising from groundborne vibration is represented by the formula

$$SPL = PWL - 10 \text{Log } S_t a + 16$$

where "S_t" is the total room area, "a" is the average absorption coefficient for the room, and PWL is the input sound power radiated into the room by the walls. Assuming that the radiation efficiency for the wall is unity, the resulting sound pressure level for an rms wall acceleration level is given as

$$SPL = L_a - 20 \text{Log} f - 10 \text{Log}(a) + 36$$

where L_a is the rms acceleration level re 10^{-6}g, determined over the entire room surface area, and f the frequency in Hz. For an average room, the absorption coefficient "a" is approximately 0.15.

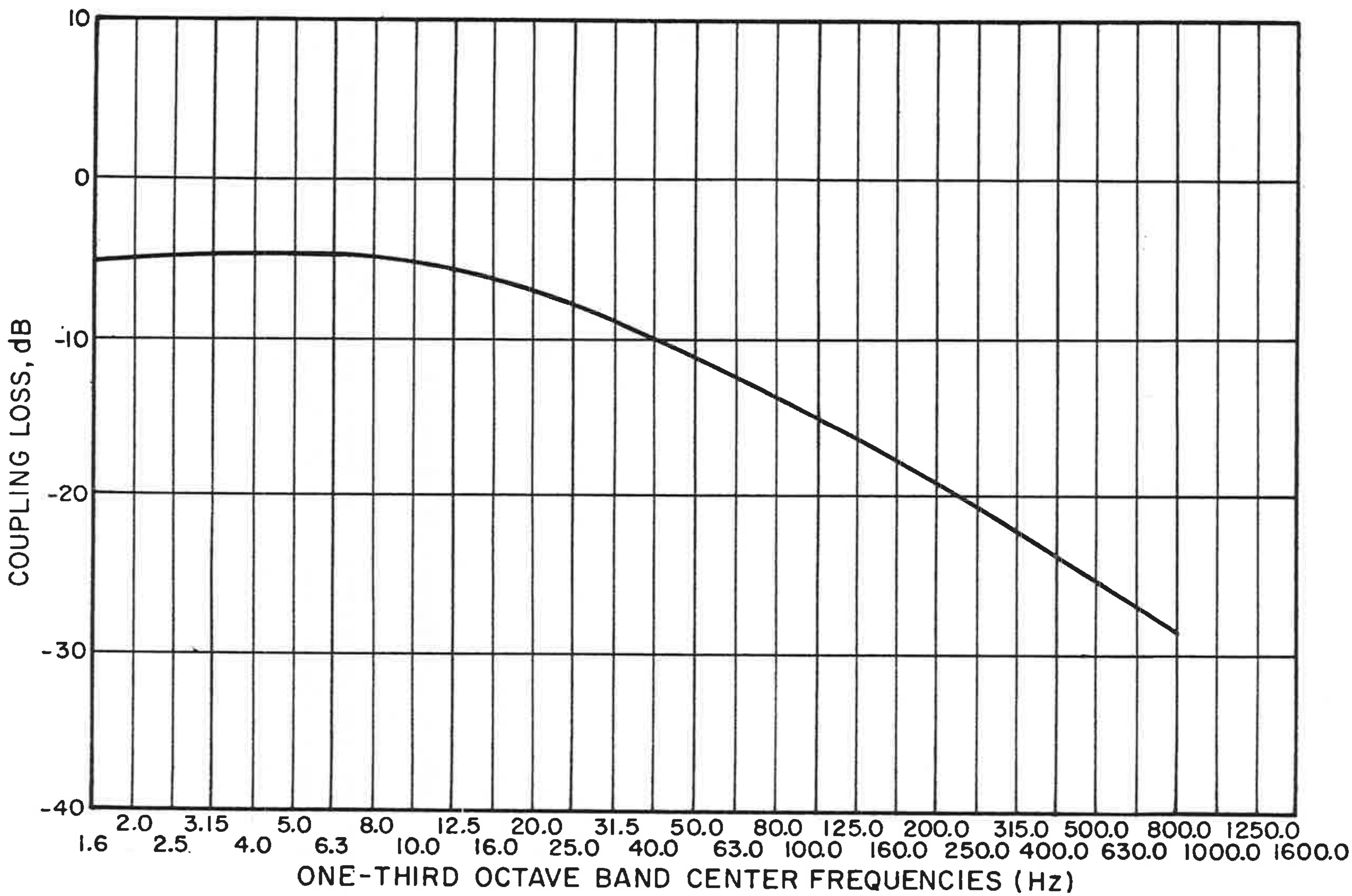


FIGURE 6.9 SOIL-TO-PILE "COUPLING LOSS" ATTENUATION [FROM BBN REPORT 1832, REF. A-18]

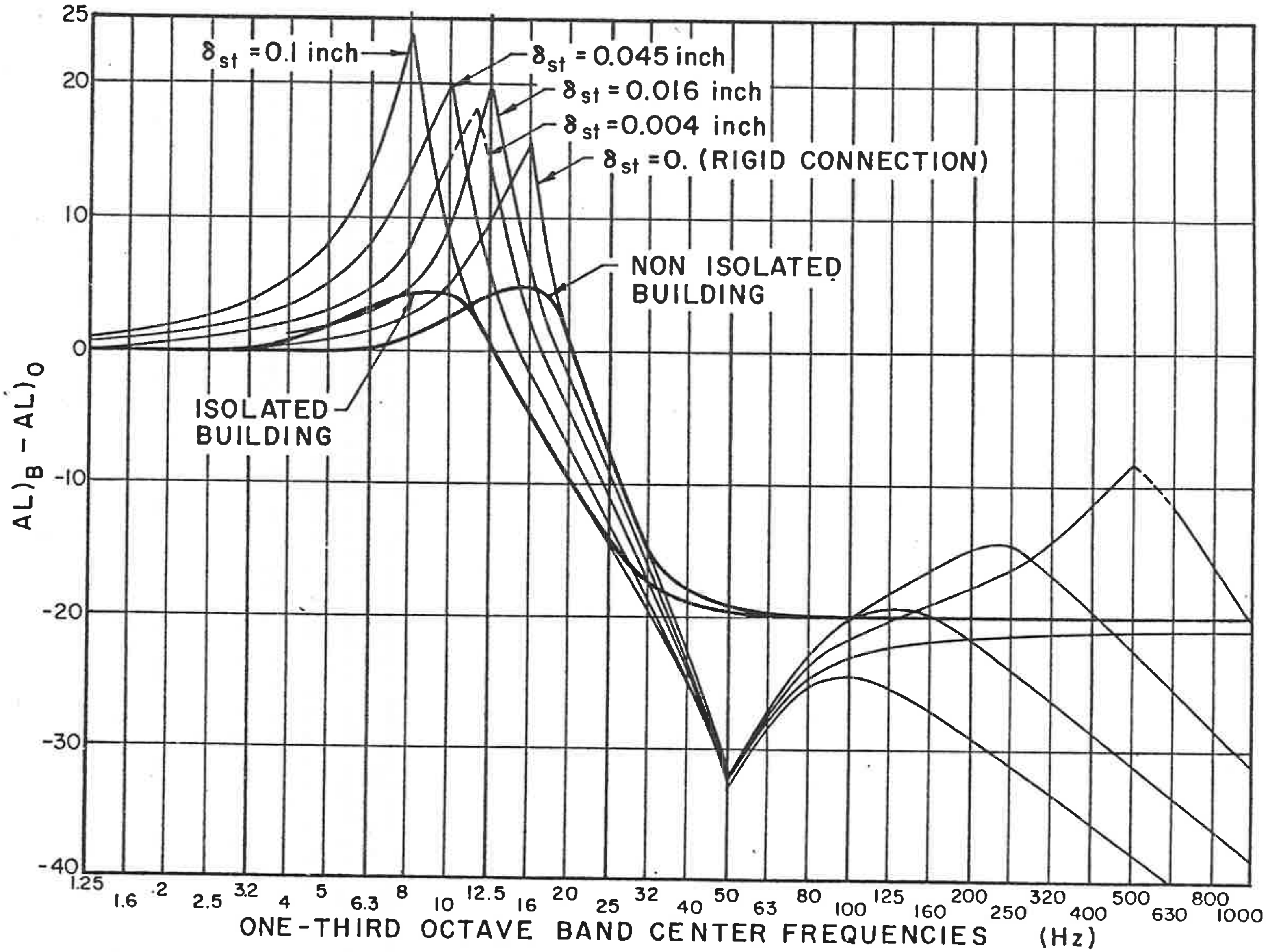


FIGURE 6.10 DIFFERENCE BETWEEN THE BUILDING ACCELERATION $AL)_B$ AND THE FREE ACCELERATION $AL)_0$ OF THE PILES WHICH SUPPORT THE BUILDING COLUMNS [δ_{st} IS THE STATIC DEFLECTION OF COLUMN/PILE ISOLATORS. (BBN REPORT 1832, REF. A-18)]

6.1.3 Tokita

A partial model of groundborne vibration radiation and propagation from subways has been presented by Tokita, et al (Ref. A-4). The features of the method are noteworthy, especially concerning the apparently good qualitative agreement claimed by the authors.

Source: Tokita employs an imaginery line source with level L_o located at a distance r_o beneath the subway structure. The concept is illustrated in Figure 6.11. An assumed propagation law was fitted to experimental data giving values of 82 dB re 10^{-5}m/sec^2 for L_o and 3 m for r_o. However, the attenuation law is not clearly presented.

The use of an imaginary line source located below the subway structure allows qualitative prediction of a local maximum of ground surface vibration level vs. distance from the structure. This trend was experimentally observed by Tokita at 15 to 30 m from the subway structure centerline at the ground surface. The distance of the local maximum increases with increasing subway depth.

The basic mechanism responsible for this maximum, as proposed by Tokita, is that the subway structure acts as a barrier around which vibration waves must diffract in order to reach the surface. The attenuation produced by the barrier effect is not clearly presented by Tokita.

Corrections: As shown in Figure 6.12 vibration levels are assumed to increase with train speed, rising by about 5 dB for a speed increase of 40 to 60 km/hr. Above 60 km/hr, effects of train speed are considered nil.

Of particular note is a projected increase of vibraton with

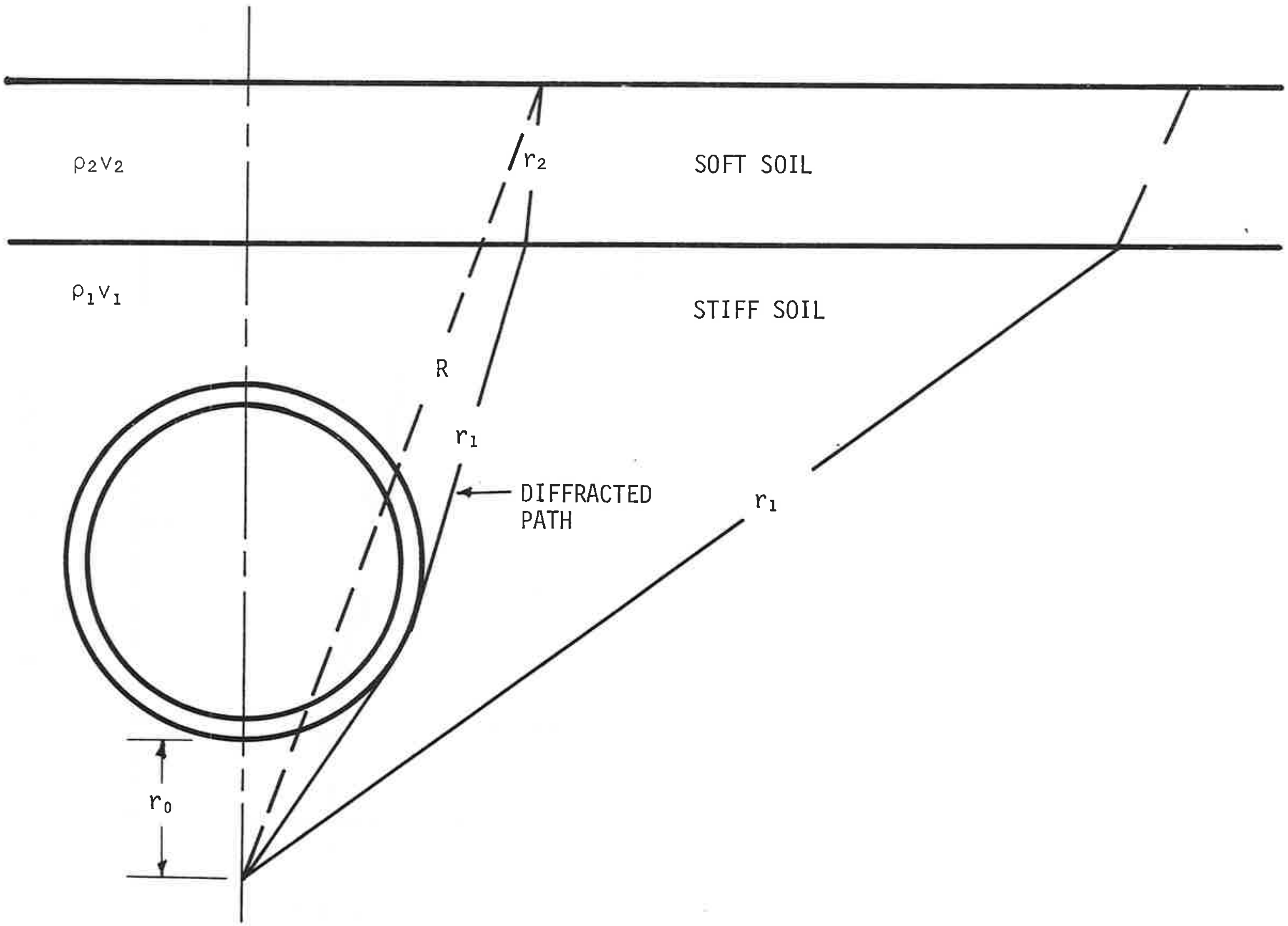
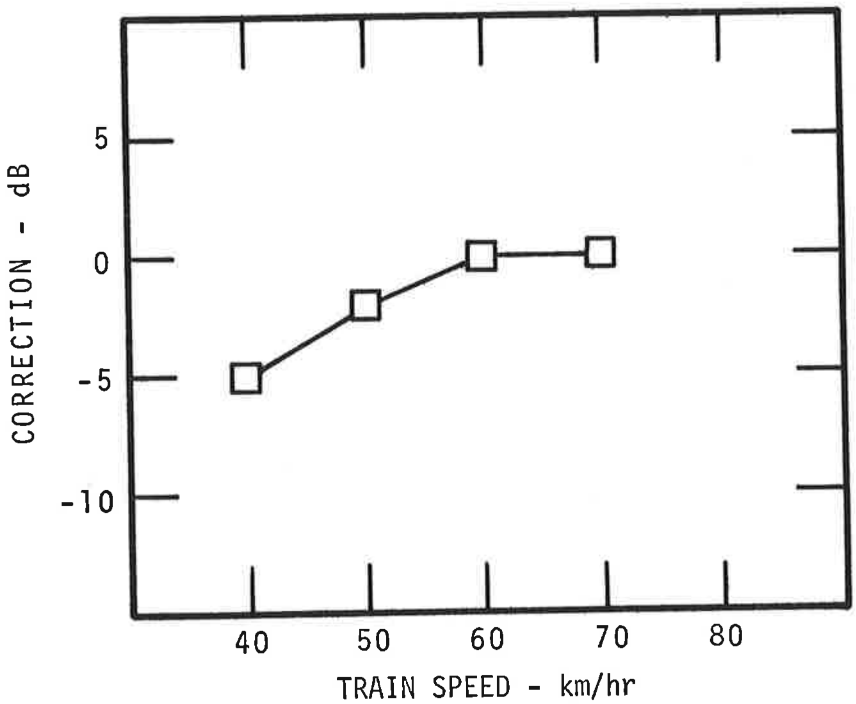


FIGURE 6.11 PREDICTION MODEL FOR GROUNDBORNE VIBRATION LEVEL VERSUS DISTANCE



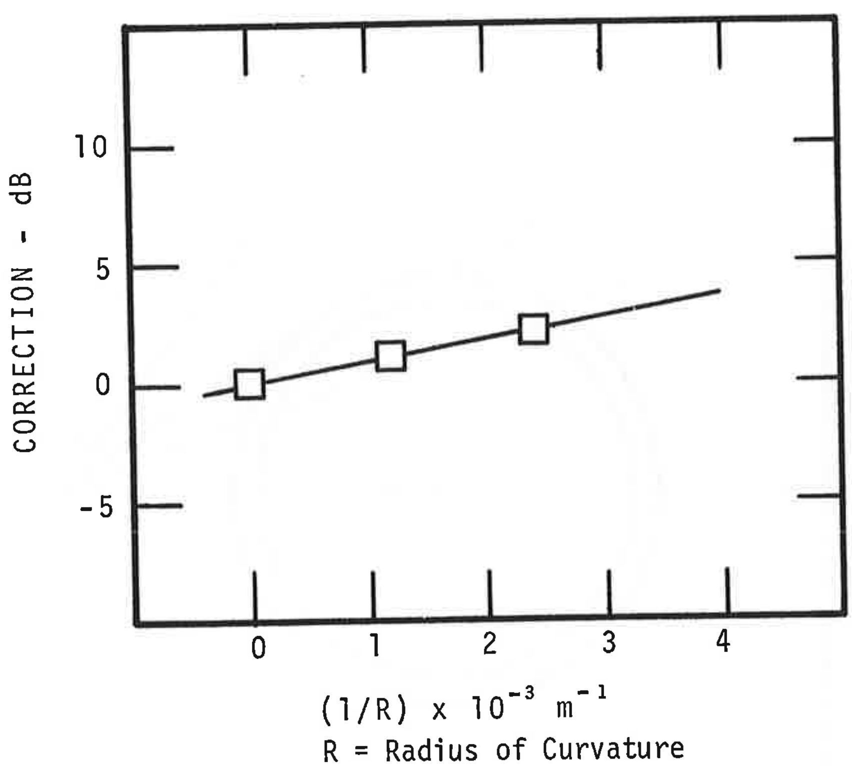


FIGURE 6.12 CORRECTIONS TO BE ADDED TO ESTIMATED LEVEL

decreasing track radius of curvature, unique in the literature. The relationship is illustrated in Figure 6.12.

No corrections were given for fastener stiffness, rail type, tunnel mass, etc. However, such corrections as discussed above with regard to Wilson's and Ungar's methods would apply.

Propagation: The attenuation of overall vibration with distance is considered as the sum of losses due to spreading and dispersion. The assumed formula for vibration propagation is:

$$L_v = L_o - A_1 \text{Log}(r/r_o) - A_2 r$$

where L_o is the source strength, A_1 is a factor accounting for spreading loss (A_1=10 for a line source), A_2 is a dissipative term, r_o is the depth of the line source below the subway and r is the distance from the line source location to the observation point.

Tokita classifies soil materials into three general categories: "soft," "rather hard" and "hard." Wave propagation velocities, densities and "damping factors" for these various soils are presented in Table 6-4. The velocities of propagation are those of shear waves, being in the range of 200 m/sec to 600 m/sec.

The use of the "damping factor" given in Table 6-4 is not clear. Assuming conventional relations betwen the attenuation coefficient and loss factor one obtains

$$A_2 = 0.35 \text{ to } 0.43$$

for all three classes of soil. The indication is that the attenuation with distance is relatively independent of the soil hardness.

TABLE 6-4 CLASSIFICATION OF SOIL MATERIAL (TOKITA, A-4)

Soil Class	N-Value	Density -g/cm³-	Velocity -m/s-	Damping Factor
I (soft)	<10	1.5	200	.05
II (rather hard)	10-40	1.6	400	.1
III (hard)	>40	1.8	600	.2

Soil interface transmission losses are accounted for by the relation

$$K_1 (dB) = 10 \text{ Log } [1- \frac{\rho_2 C_2 \cos\theta_1 - \rho_1 C_1 \cos\theta_2}{\rho_2 C_2 \cos\theta_1 + \rho_1 C_2 \cos\theta_2}]$$

where ρ and c are the densities and wave propagaton velocities for soils 1 and 2. The angles of incidence and refraction in soils 1 and 2, are θ_1 and θ_2 respectively. Here the wave is incident from soil 1 (See Fig. 6.11).

Bulding Response: Based on experimental data a simplified relation between residential structure interior A-weighted noise and ground surface vibration, is given as:

$$L_a \text{ (dBA)} = 0.88 \, L_v \text{ (dB re } 10^{-5} \text{ m/sec}^2\text{)} - 17$$

Differences in structural design, structural flexibility, etc., are noted to have a significant effect on interior noise levels. No discussion was presented concerning large masonry or steel buildings, or response of piles to ground vibration. The experimental data supporting the above formula fell within the range given in Figure 6.13.

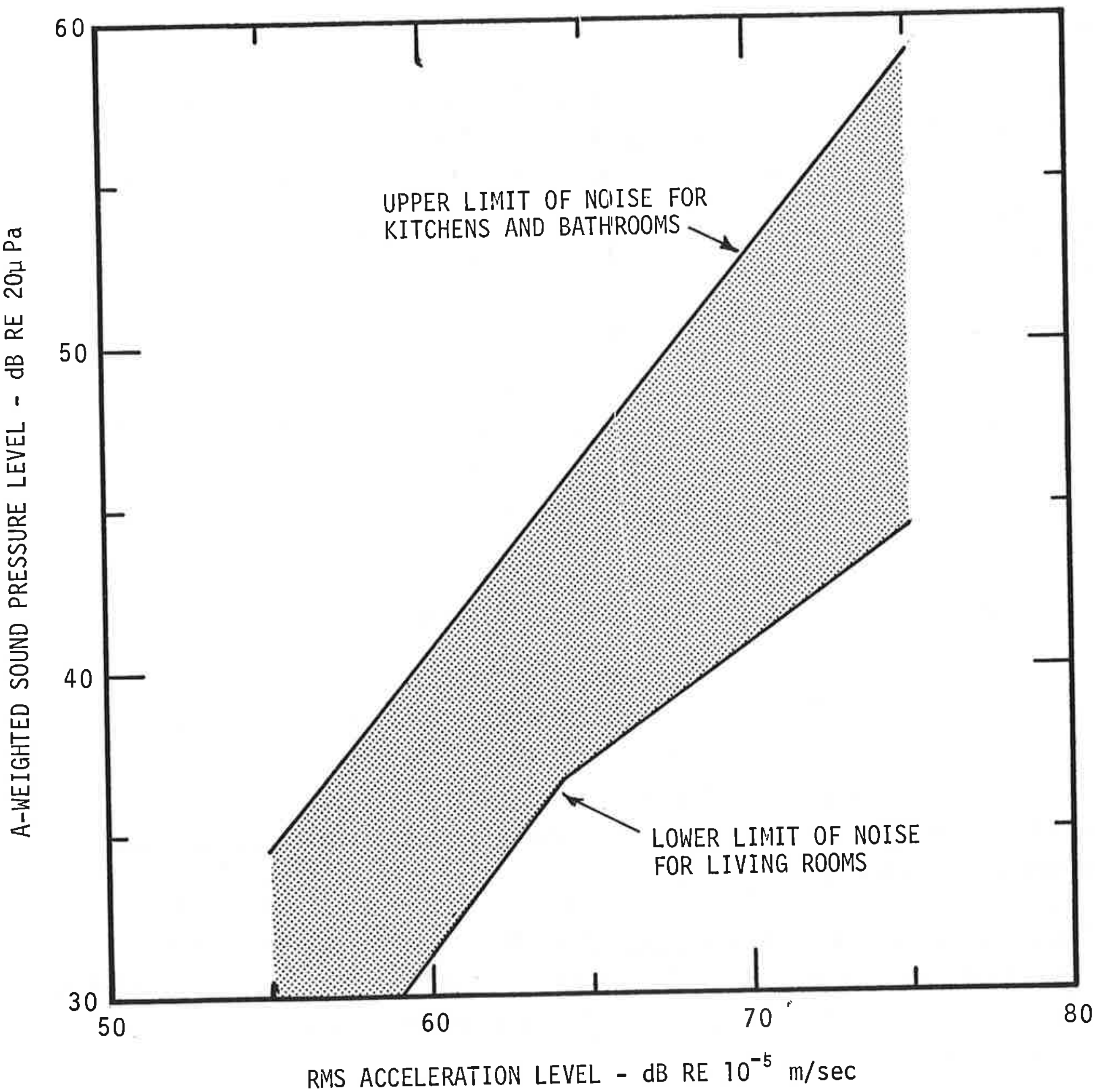


FIGURE 6.13 RELATION BETWEEN GROUND SURFACE VIBRATION AND INTERIOR NOISE IN RESIDENTIAL STRUCTURES

6.1.4 Lang

Lang has presented experimental data, Figure 6.14, which has been used to estimate A-weighted groundborne noise levels in cellars and ground floor rooms of residential structures next to and over subway structures during subway train passbys (Ref. A-29). The data fall within plus or minus 10 dB of the curve:

$$L_a = 59\text{-}20\text{Log}(d) \quad \text{(dBA)}$$

where d is the distance from the subway structure to the room in meters. The data are for a wide variety of train types, tunnel types, train speeds and conditions, track types and conditions, and tunnel and building construction.

These data have been quoted by Kurzweil (Ref. A-1) and by Manning, et al (Ref. A-54), as a simple estimate of A-weighted noise levels in buildings.

6.1.5 Kurzweil

Kurzweil has presented an amalgam of the methods used by Wilson, Ungar, and others for estimation of octave band groundborne vibration in buildings (Ref. A-46). Essentially, the groundborne floor vibration level in a building, L_a(room), due to train passage is modeled as:

$$L_a(\text{room}) = L_a(\text{tunnel wall}) - C_g - C_{gb} - C_b$$

$$\text{dB re } 10^{-6}\text{g (rms)}$$

where L_a(tunnel wall) = octave band acceleration level at the

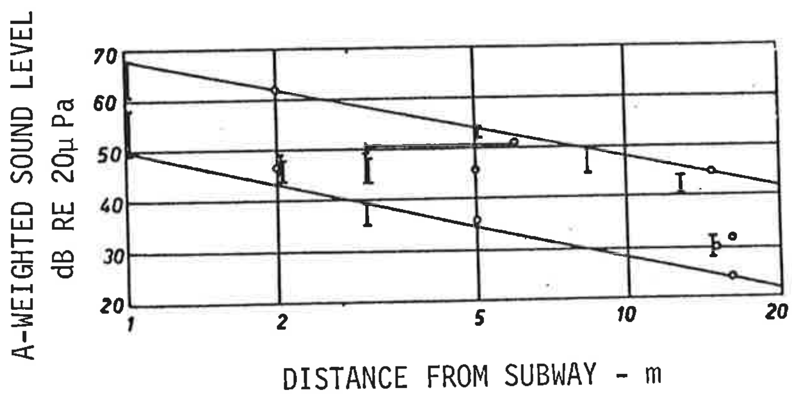


FIGURE 6.14 A-WEIGHTED GROUNDBORNE NOISE LEVELS IN BASEMENTS AND GROUND FLOOR ROOMS OF BUILDINGS OVER AND ADJACENT TO SUBWAYS

wall of a subway tunnel during a train passby in dB re $10^{-6}g$

C_g = the vibration attenuation due to propagation through the ground.

C_{gb} = Coupling loss between the ground and the building.

C_b = Vibration attenuation due to propagation in the building.

The losses associated with these loss terms are indicated as those determined by Wilson or by Ungar. Note that the starting point is the tunnel wall octave band vibration spectrum.

Kurzweil presents a summary of the literature concerning the effect of additional factors on the octave band subway wall vibration spectra. These factors include:

Train speed

Axle load

Suspension

Resilient wheels

Unsprung mass

Wheel and rail conditions

Special trackwork

Fastener resilience

Floating slabs

Ballast depth

Ballast mats

Tunnel construction

6.1.6 Nolle

Nolle has made some attempts at predicting groundborne vibration from the MURLA (Melborne, Australia) subway system (Ref. A-78). Basically, the approach used measurement data for groundborne vibration from open cut sections of existing track and correcting these levels by differences observed at TTC (Toronto) between vibration from open cut section and subway. Nolle applied regression techniques to 1/3 octave band vibration data for determination of vibration attenuation with distance due to both damping and geometric spreading. The amplitude dependence on distance from the track was modeled as

$$a(x) = a_o/(1+kx^n)$$

where x is the distance from the source, a_o an empirically determined source amplitude, and k and n empirically determined constants with respect to distance x. All three parameters a_o, b, and n vary with respect to frequency. The measurement data were of vibration in mudstone bedrock, not a typical soil. Minimum attenuation with distance was observed at 16 Hz and maximum attenuation at 400 Hz. The "error" associated with the fit was about 1 to 2 dB over the frequency range of 12.5 Hz to 400 Hz. This unique approach used by Nolle for measuring attenuation with distance in bedrock may be appropriate for soils in general, provided that sufficiently detailed measurement data are available.

6.2 DISCUSSION

Of the foregoing prediction procedures, the most highly developed are those used by Wilson and by Ungar, et al. The remaining methods are less well developed, or deal only with a particular facet of the prediction problem. There are several areas which are either neglected or are dealt with in a very approximate manner. The most important areas concern the effects of truck design and subway structure design. Additionally, there exist considerable differences regarding subway vibration radiation characteristics and the types of propagating waves.

The purpose of this section is to identify deficiencies in these methods and suggest possible extensions which might yield more reliable estimates of groundborne vibration levels.

6.2.1 Truck Design

None of the prediction procedures discussed above incorporate information on truck design. Because the truck and trackbed, driven by wheel/rail forces, can be viewed as the ultimate source of vibration energy, any extension of the above prediction procedures should consider the effect of truck design. Additionally, such extension should consider the truck and trackbed as a system. This does not imply that detailed multidegree of freedom modeling of truck and track dynamics be performed as part of the prediction procedure, but that general correction curves be developed for given truck and trackbed parameters.

6.2.2 Trackbed Design

The usual approach to incorporating the effect of trackbed design into the prediction of groundborne vibration is to include a set of octave band correction factors for each trackbed design relative to a standard direct-fixation with a rail support modulus of approximately 3,500 lbs/in^2 of rail. There is no particular reason to modify this approach, except insofar as the truck and trackbed should be considered as a system. This is especially important with respect to floating slabs and their possible interaction with the truck's primary suspension, as discussed above.

6.2.3 Subway Structure Design

The existing prediction procedures may not adequately account for the effect of subway structure design. Data presented above (see Section 5) indicate that substantial differences exist between circular tunnels and concrete double box structures. At frequencies above about 50 Hz, groundborne vibration from circular tunnels is about 10 to 15 dB higher than from double box structures, while the opposite effect occurs at low frequencies (below about 16 to 31.5 Hz). From the standpoint of prediction, the logical solution is to present different starting spectra for each basic structure design, e.g., double box or circular concrete tunnels.

This approach, however, does not directly deal with the question of the effect of subway wall thickness, or the difference between steel or precast tunnels and heavier cast-in-place circular concrete tunnels. One solution to this problem may be to statistically correlate measurement data with subway dimensional and mass parameters. This would be best performed with experimental data obtained for various subway structure designs on the same system, thereby removing variations due to differences in vehicles and perhaps regional soil characteristics. However, most new systems incorporate standardized subway structure designs with the result that all circular concrete tunnels have the same wall thickness on a given system.

In order to gain a better theoretical understanding of the effect of subway structure design, numerical and/or analytical modeling is suggested to augment experimental data. Sufficiently reliable quantitative information might thus be obtained which could be incorporated into a prediction procedure in the form of a table or set of curves giving level correction as a function of frequency and pertinent subway parameters.

6.2.4 Source Location

As can be observed above, some considerable differences of opinion exist concerning the "source" of vibration from subway structures. On the one hand the subway wall is assumed as the major source, while on the other hand an imaginary source is assumed located below the structure. The actual case, however, is probably far more complicated than these simple concepts would indicate. At relatively low frequencies, i.e., below 20 Hz or so, bending of the subway structure as a beam might be most significant. At frequencies above perhaps 100 Hz, the subway wall may be the primary source, although in this case the invert area is nevertheless a likely participant as well.

From the standpoint of prediction, knowledge of the actual source location may not be as important as knowledge of a reference ground vibration spectrum at 5 to 10 m from the structure. This is the approach taken by Wilson which essentially avoids in a practical way questions of source location, and soil structure interaction effects. The approach is analogous to using noise level data measured at 1 m from mechanical equipment to estimate noise levels within industrial spaces. In such cases, the vibration of a motor housing is never used as the starting point for noise prediction, due to lack of detailed knowledge regarding radiation efficiency, and so forth. Similarly, one can argue that using subway wall or invert vibration data is likewise less accurate than starting with the ground vibration spectra at some standard distance from the subway structure.

6.2.5 Wave Types

One major question concerning the above prediction procedures is the type of wave radiated by a subway structure. Ungar and Bender assume that compression waves radiated by the normal wall vibration are the most important. Tokita on the other hand, (judging from his assumed wave velocity data) predicts on the basis of shear waves. The type of wave is likely to be a combination of wave types, e.g., compression and shear.

Additionally, guided waves in soil layers may occur, and lightly damped coupled compression waves within saturated soils may be of great significance.

For vertical excitation of an homogeneous, isotropic, elastic, half space by a circular disc, Miller and Pursey have determined that the partition of energy between wave types is as follows (Ref. C-26):

Surface wave (Rayleigh)	67%
Shear wave	26%
Compression wave	7%

This type of information is quite relevant to groundborne vibration from aerial and at-grade track structures. The energy partition is not applicable to subways because of the subsurface location of the vibration source. However, the above partition does indicate that with respect to body waves, shear waves may be more important than compression waves.

Pekeris and Lifson have produced solutions to the problem of the motion of the surface of an homogeneous isotropic elastic half-space produced by a buried pulse (Ref. C-25) for the case of Poisson's ratio of 0.25. In particular, the results of this solution indicate that for a concentrated pulse applied vertically at depth H below the surface, a Rayleigh wave emerges at about a distance of 5 H from the epicenter of the pulse. At large distances, the asymptotic character of the solution approaches that of a pulse applied at the surface. Although the nature of loading of the soil by the tunnel is undoubtedly more complex than that of a buried pulse, the solution presented by Pekeris, et al., suggests that at sufficiently large distance from a subway structure a significant amount of vibration energy may be borne by a Rayleigh wave. This may also substantially explain the "shelving" of vibration level as a function of distance observed at TTC (Reference A-52), or the local maxima observed at about 15 m to 30 m from the subway structure by Tokita, et al. Note that Tokita attributes this effect as due to barrier attenuation caused by the subway structure.

The shelving or actual increase of vibration level as a function of distance from a subway structure might also be explained on the basis of energy partition and relative attenuation rates for different wave types. For example, if a great deal more shear wave energy is radiated than compression wave energy, but the compression wave energy is absorbed at a much lower rate, then relatively rapid attenuation of vibration with distance may be expected at locations close to the structure, while at large distances from the structure, the attenuation rate would be controlled by the surviving compression wave.

At the present time, no definitive data regarding energy partition has been obtained for subway structures, although measurements performed by Verhas indicate that shear and Rayleigh waves are the most significant for ballast-and-tie surface track. Shear and/or Rayleigh surface waves would also be expected from aerial structure foundations. If attenuation of vibration with distance in soil is to be confidently predicted using loss factors and wave propagation velocities for various soils, assuming that a reliable propagation formula is available, then it is necessary that the energy partition between various wave types be known.

6.2.6 Dissipation

Although there appears to be some discrepancy between methods for accounting for spreading and dissipation within soils, there is a general consensus that the attenuation rate increases with increasing frequency. This is primarily a result of dissipation being largely proportional to the number of wavelengths traversed. A formalism has been established for predicting such dissipation losses from loss factors which are assumed to be frequency independent. This formulation, as well as others, have been discussed in the literature regarding groundborne vibration by Gutowski and Dym (Ref. A-59). Although there may be motivation for gaining a further understanding of dissipation mechanisms, such study requires extensive research and detailed testing and analysis. For purposes of prediction of groundborne vibration from transit systems in a practical way, the methods outlined by Gutowski and Dym are probably appropriate for estimating dissipation losses. Attention should, therefore, focus upon the types of waves radiated from transit structures.

6.2.7 Building Coupling Loss

Building foundation coupling losses, i.e., the building foundation vibration levels relative to incident free-field groundborne vibration levels, is not well understood in the audio frequency range. Detailed modeling of the modal response of buildings to incident groundborne vibration of arbitrary frequency, velocity, and direction is at best impractical. Considering the wide variety of building designs, foundations, and the variability of soil parameters, the approximate curves and analysis presented by Wilson and also by Ungar and Bender are appropriate for a prediction method. Effort could be directed towards compilation of coupling losses for various structure designs, as is done in the field of architectural acoustics regarding transmission losses of various architectural partitions. Such effort might be summarized in a compendium which would serve as a valuable reference for prediction of groundborne vibration in buildings not only from transit system sources but from highway and industrial sources as well.

6.2.8 Vibration Attenuation in Buildings

The floor-to-floor attenuation of vibration in buildings is very difficult to predict analytically, but sufficient experimental data is available for prediction purposes. The assumptions of 3 dB attenuation per floor is reasonable and supported by measurement data reported by Tachibana for the first several floors above grade. At higher floor levels, the attenuation rate reported by Tochibana is closer to 1 to 2 dB per floor.

6.2.9 Noise Generation in Buildings

The mechanism of noise radiation by vibrating panels (i.e., walls, floors, and ceilings) into rooms is well understood. Provided that knowledge of the amplitude of wall vibration velocities are known, reasonably accurate estimates of interior noise may be made for given values of average absorption coefficient. Either of the methods proposed by Wilson or by Ungar and Bender are appropriate, the latter being more detailed due to the inclusion of the average absorption coefficient. Ungar and Bender also have developed formulas for prediction of interior noise levels at sub-modal frequencies for the room (Ref. A-136).

7. MATHEMATICAL MODELS FOR PARAMETER EVALUATION

One of the problems encountered in the development of a comprehensive prediction method is the lack of accurate analytical models that can be used to evaluate the generation and propagation of groundborne noise and vibration. There are many analytical techniques available in the literature which may have application to groundborne vibration; a number of those models are summarized in this chapter. The analytical models discussed are not generally appropriate for direct incorporation into a prediction method, however, the results of the evaluation might be applied in a prediction method in the form of tabulated correction factors. Covered in this chapter are mathematical models of:

- rail fasteners
- resiliently supported ties
- floating slabs
- truck dynamics
- subway/soil interaction and radiation from the subway structure
- vibration propagation and attenuation in soil
- building response to ground surface vibration

In several areas, e.g. subway/soil interaction, the extent of model development is very limited. Other models are very well developed and are only briefly summarized in this report. Instead of focusing on a complete development of the models, the discussion concentrates on the limitations and possible extensions to the existing models.

The general conclusion is that there are a number of techniques and mathematical models that could be, or have been successfully applied to groundborne noise and vibration. In some areas such as vibration propagation and attenuation in soil, much can be gleaned from the research in other fields. Both earthquake engineering and geology are concerned with the propagation of waves through earth media and earthquake engineering is intimately concerned with the response of structures to ground motion. Unfortunately, some of the techniques that have been very successfully applied in these fields are not practical for rail transit groundborne vibration.

Following is a summary of the conclusions and salient observations:

> Rail fasteners: The analytical models of rail fasteners that have been developed seem to represent reasonable approximations which should be adequate at frequencies below 200 Hz.
>
> Resiliently supported ties: Although multi-degree of freedom models of resiliently supported ties have been developed, the models are not very well documented in the literature.
>
> Floating Slabs: The simple single-degree-of-freedom model of a floating slab is adequate for most design purposes. More detailed analysis should be employed when the resonance frequencies of the floating slab and the truck primary suspension are close enough to interact or when the mass of the slab and the tunnel structure are approximately equal.
>
> Truck dynamics: The truck dynamics have a strong influence on groundborne vibration. Although there are many MDOF computer models of trucks, most are concerned with ride quality. The literature review did not reveal any easily used models that include the effect of truck dynamics on groundborne vibration. A 3 or 4 DOF model of a truck should be adequate for evaluation of groundborne vibration.

Subway/Soil interaction: Models of the interaction of the subway structure and the surrounding soil, and the radiation of vibration by the subway structure are inadequate. Simple models can be used to estimate the relative vibration levels of soil and rock founded subways, however, fundamental questions such as the relative importance of shear and compression waves and the effect of tunnel wall thickness can only be determined by measurements at this point. Some of the promising analytical approaches should be investigated.

Vibration propagation and attenuation in soils: This is another area where a considerable amount of research is needed. At present estimates of attenuation of vibration in soil must be based on empirical data. Several models have been preposed for dissipation of vibration in soil, however, they do not always show good agreement with test results. It appears that much can be learned from reference to the literature on dissipation of vibration in rock.

Building response to groundborne vibration: Although building response to ground motion is a central problem of earthquake engineering and has been the subject of considerable research, most of the techniques that have been developed do not have direct application to groundborne noise and vibration from transit trains. However, much can be learned about the interaction between ground motion and buildings from straight forward lumped parameter models of the buildings. Recent research shows promise for predicting the floor-to-floor attenuation of audio-frequency vibration and noise in large multi-story buildings.

7.1 RAIL FASTENERS

Bender, et al (Ref. A-18) have developed a model of direct fixation resilient fasteners that includes the wheel and rail roughnesses, the vehicle and rail impedances, and damping of the fastener elastomer. The analysis indicates that the integrated net force transmitted to the invert is equivalent to that which would be transmitted through a simple spring-mass vibration isolation system.

The rail impedance is determined by modeling the rail as an elastically supported Bernoulli-Euler beam. After determination of the rail impedance for the undamped case, the effect of damping in the fastener is approximated by introduction of a complex rail support modulus.

Figure 5.6 presents the predicted change in vibration isolation when the rail support modulus is reduced relative to the standard TTC fastener. The results from a test by the Toronto Transit Commission in which the groundborne vibration was measured before and after adding an extra pad to the fastener is presented in Figure 5.7 (Ref. A-52). The effect was to reduce the rail support modulus from 4300 lb/in to 2100 lb/in. As can be observed in Figure 5.7, the measured difference between single and double thickness pads is consistent with projection.

The model incorporates a complex rail support modulus whose imaginary loss term is linearly dependent on frequency. This approach represents a reasonable approximation which should be adequate under most circumstances where relatively light damping is concerned.

An exact solution to the problem of a Bernoulli-Euler beam supported on a visco-elastic foundation is provided by Mathews for the case of a stationary oscillating point load (Ref. C-8, C-9). In the event that fasteners with relatively high damping or very low stiffnesses are to be modeled, the solution by Mathews might be relatively easily incorporated into Bender's model, although the results should not change dramatically.

The assumption of continuous support should be reasonable over the frequency range of groundborne noise and vibration. The effect of discontinuous supports (e.g., fastener spacing) becomes important at frequencies for which the wavelength of bending waves in the rail is comparable with twice the fastener spacing. This condition will not be attained at frequencies below 200 to 800 Hz.

7.2 RESILIENTLY SUPPORTED TIES

Resiliently supported ties such as the STEDEF system have been shown to be an effective method of reducing groundborne vibration. The STEDEF system has been installed at a number of European systems and at the MARTA system in Atlanta.

Since resiliently supported ties are widely used, a model that could be used to optimize the vibration isolation would be valuable. Kazamaki (Ref. A-69) discusses a four-degree-of-freedom model used for evaluating two resiliently supported tie designs. The model includes the truck wheel set, rail, rail pad stiffness, tie, tie support stiffness, tunnel, and ground. The model was used to compare the performances of resiliently supported ties and ballasted tracks and showed good agreement with experimental data. Unfortunately, the model is not described in detail.

Another approach is the model developed by Manning, et al., (Ref A-19) for floating slab and rail interaction. The model could be simplified for use in analysis of the resiliently supported tie system by allowing the floating slab longitudinal bending stiffness to go to zero, ignoring transverse bending in the slab, adjusting the floating slab dimension, and adjusting the rail and slab support stiffnesses to correspond with resiliently supported ties. A simpler approach may be to directly model the system as a Bernouilli-Euler beam supported on a continuous system of springs and an inertial mass. In either case, damping in the tie support stiffness should be included as a significant parameter, judging from Colombaud's comment that such damping is responsible for the good performance of the R.S. STEDEF system at the RER Subway in Paris (Ref. A-28).

Finally, modeling of resiliently supported tie track as well as other trackbed designs should ideally include the truck, i.e., the multi-degree-of-freedom approach as indicated by Kazamaki (Ref. A-69). To the extent that the support stiffness of the STEDEF design is relatively high, approximating that of ballast, one might be able to ignore coupling effects between the truck primary suspension and the track support system. However, an axle bending mode at between 70 and 125 Hz could couple with the resiliently supported tie system, producing lower than anticipated vibration isolation.

7.3 FLOATING SLABS

Floating slab vibration isolation systems are presently the most effective method of reducing groundborne vibration. Two types are currently in use in the U.S. One is the continuous floating slab used at WMATA, and the other is the discontinuous type used at TTC and MARTA.

Wilson (Ref. A-13) based the design of the WMATA continuous floating slab on a single-degree-of-freedom isolator system. Only isolation of vertical vibration was considered in detail. Included in the model are the combined mass of the truck and floating slab per unit tunnel length, the isolator and perimeter board stiffness, and the entrained air stiffness. Note that the entrained air adds significantly to the dynamic stiffness of the floating slab. Decoupling of the floating slab with the tunnel stiffness was ensured by requiring the tunnel mass to be 3 to 10 times greater than the slab mass, while coupling between the truck and slab system was neglected in the analysis. Also included in the model is damping provided by the resilient slab supports. However, additional significant effective damping is assumed due to the translating nature of the load.

Wilson has extended the approach to the discontinuous floating slab (also referred to as the double tie system) used at TTC and MARTA. The major difference is that there is no entrained air to be included in the stiffness calculation. The truck masses are still assumed to be evenly distributed along the vehicle length so that the masses, stiffnesses, and damping are still computed per unit tunnel length.

The approach used by Wilson has been quite successful, especially for the WMATA double box floating slab and the TTC double tie system. At WMATA, although the truck suspensions are relatively stiff, giving primary resonances greater than 25 Hz, the large mass of the floating slab in double-box subways relative to the truck mass tends to decouple the truck and floating slab systems.

However, in the small tunnels, e.g., circular concrete or steel tunnels the floating slabs are of lower mass, with the result that the potential for coupling is increased at low frequencies. In such cases more complex multi-degree-of-freedom models would probably be desirable for designing floating slab vibration isolation systems.

A relatively detailed analysis of floating slab vibration and interaction with the rail was performed by Manning, et al. (Ref. A-19), drawing heavily upon a model developed by Bender, et al. (Ref. A-103). The rail fastener and floating slab support stiffnesses are both considered as continuous supports with damping. The model concentrates on coupling between bending waves of the rail and both transverse and longitudinal bending modes of the slab. One particularly interesting result of Manning's work is that for sufficient damping in the system, the force transmissibility predicted by the coupled mode model is similar to that predicted by the much simpler single-degree-of-freedom model, thus supporting the approach used by Wilson and the relatively good agreement of Wilson's model with experimental data.

Manning extends the coupled mode model for continuous floating slabs to the discontinuous double tie slab by setting the bending stiffness of the slab equal to zero. This approach could also be used to model resiliently supported tie systems.

Johnston (Ref. A-67) has treated the case of the discontinuous double tie floating slab during the course of his modeling of truck dynamics for the TTC. Essentially, the slab was represented as an inertial element in terms of mass per unit tunnel length, neglecting rail bending stiffness. Using this approach, Johnston obtained a rail driving point impedance which he then combined with a limited multi-degree-of-freedom model of the truck to assess ground loading and force transmissibility of the isolator system.

Of the above models, only the one employed by Johnston includes a detailed model of the truck, including an axle bending mode, to describe floating slab vibration isolation performance. The remainder of the methods either assume the wheel set mass or the mass of the truck as representative of vehicle loads. An extension of these models should include the effect of primary stiffness, the first axle bending mode, and possibly the bending of the truck frame. The model should be sufficiently general to allow analysis of resiliently supported tie systems such as the STEDEF, and should encompass the frequency range of at least 5 Hz to 100 Hz. Particular attention should be focused on damping in the resilient slab support elements and in the truck. Finally, consideration of coupled horizontal and rocking modes of both the truck and slab may be advisable, although the net force at the invert resulting from such modes will likely be less than that due to vertical translation.

The subject of dynamic modeling of the truck will be discussed below in greater detail. Emphasis must be placed, however, on viewing both the truck and the floating slab as a system.

Some discussion has occured regarding the relative effectiveness of continuous and discontinuous floating slabs. Most data from U.S. and Canadian installations of floating slabs indicate that the performance of the continuous and discontinuous floating slabs is very similar. However, Grootenhuis (Ref. A-106) states that local vibration build-up in transverse bending of the discontinuous double tie will compromise the isolation effectiveness of the slab, relative to the continuous design. He further states that the TTC double tie slab does not perform adequately on the basis of measurement data, but does not supply such data. Samavedam and Cross (Ref. A-159) state that longitudinal bending wave propagation in the continuous floating slab contributes additional radiation damping at frequencies above the design resonances of the slab and is therefore more desirable than the discontinuous double tie slab.

The model developed by Bender, et al., (Ref. A-103) and extended by Manning, et al., (Ref. A-19) should be sufficient to answer the question of whether or not a continuous floating slab is preferable to a discontinuous slab. For the present, however, the data presented by Lawrence (Ref. A-15) indicate that the vibration isolation of the double tie system is as good or better than that for the continuous slab design. Indeed, the first transverse bending mode of the double tie occurs at roughly 100 Hz, at which frequency virtually all measurement data collected to date indicate that the double tie system is quite as effective as the continuous design. Secondly, although continuous floating slabs may provide damping due to longitudinal bending wave radiation from a point, the fact that a train is a line source of vibration energy means that whatever vibration energy is radiated away from an excitation point is replaced by vibration energy produced at neighboring points in the absence of damping. The result is that dispersion of vibration energy along the slab is probably of only limited interest.

Finally the problem of a translating, oscillatory point load directed against an elastically supported Bernoulli-Euler beam has been studied by Mathews (Ref. C-8). This solution can be used to assess the effect of a moving load on the amplitude of the resonance peak in the force transmissibility curve of floating slabs, relative to that for a stationary load. Unfortunately, damping is not included in the closed form solution, although a numerical technique can be employed to include the effect of damping. Mathews has treated the case of a stationary oscillating point load directed against a visco-elastically supported beam (Ref. C-9).

7.4 TRUCK DYNAMICS

Of considerable importance in the generation of groundborne vibration is the impedance of the vehicle as "seen" by the rail. If this quantity were identically zero, the rail would not be excited into vibration by rail and wheel surface roughness. The nature of the vehicle impedance in conjunction with the rail driving point impedance and rail and wheel surface roughness, determines the magnitude of the rail vibration and thus ground loading.

The simplest approach to modeling the vehicle impedance is to consider only the wheel-set mass; this should be adequate for frequencies in the neighborhood of 60 Hz and higher. Included with the wheel-set mass should be the masses of axle mounted equipment such as gearbox and brake discs. At lower frequencies, e.g., in the range of 2 to 20 Hz, the entire mass of the truck may be the determining factor. At still lower frequencies, the vehicle body mass must be added. In addition to simple mass-like assumptions for the vehicle impedance, bending modes of the axle and the truck frame will influence the vehicle impedance.

Measurements reported by Wolfe (Ref. A-7) indicate that truck design can have a very substantial effect on groundborne vibration. Moreover, this same data indicates that the nature of trackbed structure influences the <u>effect</u> of such truck design on vibration. The implication is that the truck and trackbed must be considered as a system, for which a multi-degree-of-freedom model is desirable.

The most detailed analysis of truck dynamics with respect to trackbed configuration is described by Johnston (Ref. A-67) for the TTC (Toronto) system. Johnston concluded that the truck may be adequately approximated by considering only the following truck components:

- Single Wheel
- Axle (Rigid and first bending mode)
- Gearbox
- Frame

The first axle bending mode at 119 Hz was identified as significant, although higher modes were ignored on the basis of the resonance frequencies being above 450 Hz and higher. The case of resilient wheels was also considered.

Johnston includes the driving point impedance of the rail in the analysis. For the case of simple resilient fastening, the rail is treated as a beam on a continuous elastic foundation. For the double tie floating slab vibration isolation system, the rail impedance is modeled as that of a 2 degree of freedom oscillator, neglecting rail bending stiffness.

Numerous other multi-degree-of-freedom models have been developed for transit vehicles, resulting in publicly available software, (Ref. A-8), e.g. DYNALIST, HALF, FULL, and FLEX. All of these models were initially developed for ride quality and truck stability assessment, and include the vehicle body in the analysis. Of these models, DYNALIST is the most general, capable of analyzing up to 50 degrees-of-freedom, and could be directly extended to analysis of truck dynamics in the frequency range of groundborne vibration. However, it is a rather complex model and program to use. A simpler and perhaps more practical model is that provided by HALF, which is relatively unique in that it includes the track support modulus. HALF includes six degrees-of-freedom, which should be adequate for many truck vibration problems. However, the program has the disadvantage of being dedicated to a particular dynamic model, which, for instance, does not include the first axle bending mode nor the gearbox-axle vibration mode. HALF can be used for analysis of coupling between the primary suspension and a floating slab vibration isolation system and has the added advantage that optimization of floating slab design can be performed with consideration of ride quality and truck stability. HALF could be modified and extended to provide a more comprehensive model which includes the axle bending mode and the gearbox.

7.5 SUBWAY/SOIL INTERACTION AND VIBRATION RADIATION

One of the facets of groundborne vibration that is least understood is the interaction of the subway and surrounding earth and the radiation of vibration by the subway structure. At the present time, the effect of subway wall thickness can not be determined except by measurement. The relative importance of different parts of the subway structure as regards vibration radiation is not clear. Also, the partition of energy between shear and compression waves in soil is not known, but is necessary for a reliable estimate of attenuation with distance in soil.

One of the early models of the effect of subway founding conditions is provided by Bender, et al. (Ref A-158) for estimating the difference between subway wall vibration for earth based and rock based subway structures. The amplitude ratio between earth and rock based structures is given as

$$\frac{a_r}{a_s} = \frac{Z_{st} + Z_s}{Z_{st} + Z_r}$$

where a_r/a_s is the ratio of subway wall vibration between earth and rock based structure, Z_{st} is the subway structure impedance, Z_s is the soil impedance, and Z_r is the rock impedance.

Bender indicates that the subway structure impedance is less than either the earth or rock impedance, and that the soil impedance is about 1/10 the rock impedance. Therefore, subway wall vibration for rock based structures should be about 20 dB lower than for earth based structures.

Wilson (Ref. A-3) uses the same model to estimate the ratio between vibration in surrounding soil or rock for soil and rock based structures. Following Wilson's discussion, there are two general ranges where the impedance of the system or at least the ratios of impedances, can be estimated from easily determined properties of the system components. In the low-frequency region the mechanical impedance is stiffness controlled and the amplitude of vibration is proportional to the stiffness. In the high-frequency region the mechanical impedance is mass controlled and the amplitude of vibration is inversely proportional to mass times frequency squared, assuming constant amplitude forces in the frequency domain. In the transition area between low and high frequencies the amplitude of vibration is controlled largely by the damping and other coupling factors in the mechanical system.

At high frequencies where the subway structure impedance, Z_{st}, is large, the vibration levels for rock and earth supported subway structures are comparable. Since the ratios of low-frequency and high-frequency vibration levels are considerably different, it is necessary to determine the frequency range where the transition occurs. There are two basic sources of information on where this transition occurs. The literature on ground vibration created by blasting in rock and soil indicates that the frequency spectrum shifts upward by a factor of about 3 for rock and soil blasts. The peak acceleration amplitudes remain about the same for equal applied forces. For a typical single or double box subway in soil, the natural frequency of the subway structure on the earth spring is estimated to be in the range of about 17 to 32 Hz and, therefore, the transition from stiffness control to mass control starts at about 30 Hz for subway structures supported on soil. For a rock base subway, the transition will occur at a much higher frequency, on the order of 125 to 250 Hz. With the information from blasting and the calculated natural frequencies of subway structures, vibration amplitude ratio curves which bridge the transition section between the low-freqency, stiffness controlled region and the high-frequency, mass controlled region may be constructed.

Another method of estimating ratios of vibrations for rock and soil based subways is to use the ratio of the coefficients of subgrade reaction or the ratio of Young's moduli for rock and soil. This approach is only appropriate in the low-frequency region. These ratios indicate that at low frequencies the vibration of rock based subways will be 10 to 20 dB less than for soil based subways.

The lumped mass model described above may be compared to ground surface vibration data presented in Figure 7.1 for a variety of subway types and basic founding conditions at WMATA Metro (Ref. A-98). The data presented for soil based subway structures has been normalized to a slant distance of 50 ft from top-of-rail, assuming a 3 dB attenuation rate per distance doubling, and neglecting the effect of damping. The rock and mixed-face tunnel data were left unaltered due to the complexity of the transmission path. However, about 50 ft of soil covered the rock strata, and the propagation path for the rock subways was roughly divided equally between rock and earth.

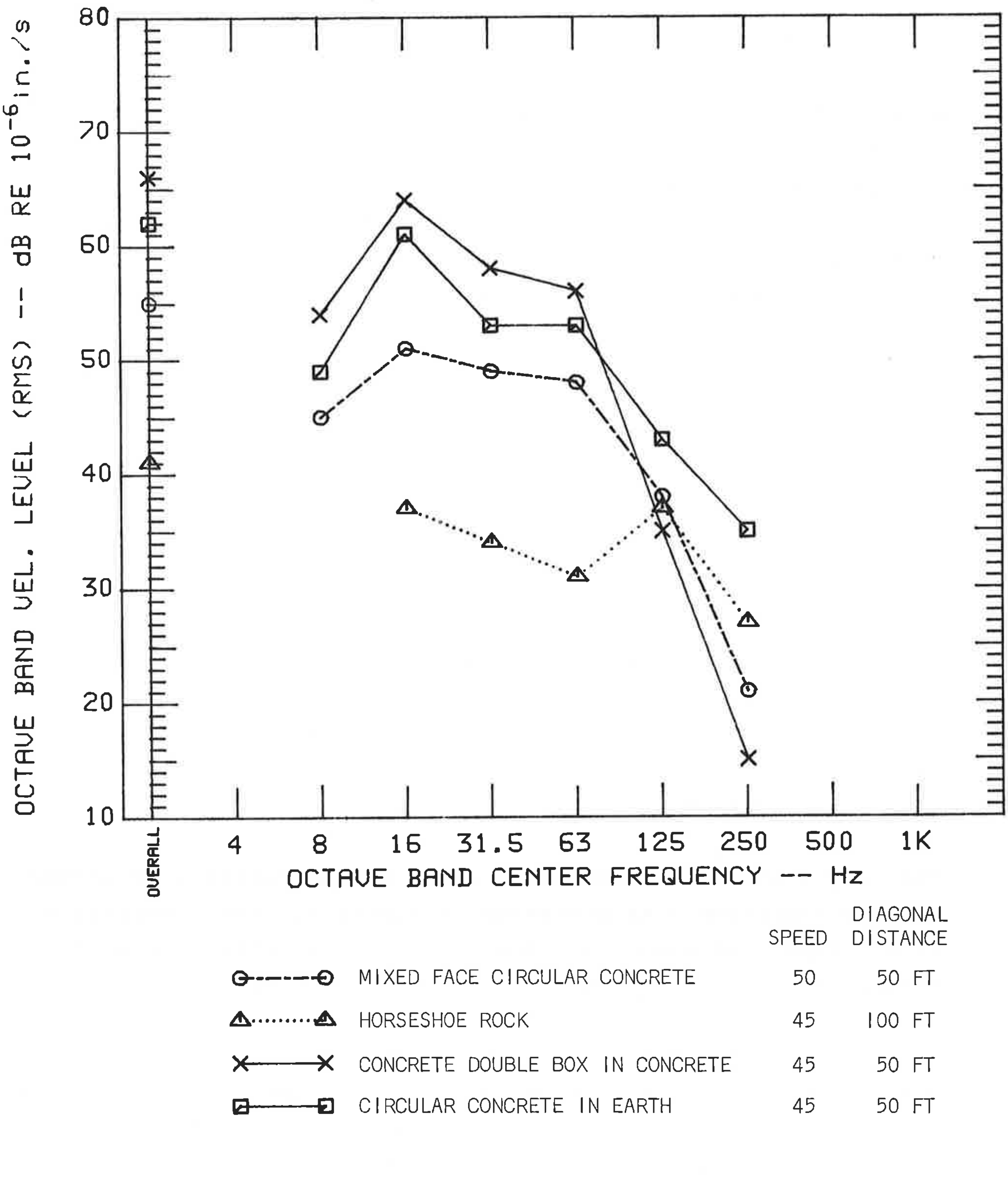


FIGURE 7.1 GROUND SURFACE VIBRATION MEASURED AT WMATA METRO, RIGID INVERT

The data presented in Fig. 7.1 indicate that the lumped mass analogy may provide a simple means of estimating the effect of structure mass and soil stiffness on groundborne vibration. The model is particularly suitable for use in regression techniques for an empirical evaluation of structure parametric effects.

Subway structures have also been modeled as a beam on an elastic foundation (Ref. A-52). The goal of this model was to identify the vertical resonance frequencies for the subway structure in soil to explain a peak at 50 Hz in the spectrum of groundborne vibration. Damping is indicated as a significant factor in the subway motion, but no quantitative values are given. The model ignores distortion of the subway cross-section and bending of the subway walls.

Greenfield (Ref. C-27) recently presented a solution to the problem of a harmonic point load acting against the surface of a cylindrical cavity within an infinite elastic homogeneous and isotropic medium. Radiation patterns for both shear and compression waves are discussed. For wavelengths in soil less than or similar to twice the cavity diameter, directivity effects are significant, and for wavelengths significantly less than the cavity diameter, vibration radiation is highly directional. Most of the energy is radiated into the half-space on the side of the cavity at which the force is directed. This shielding effect (combined with absorption in soil) may help to explain the rapid attenuation of groundborne vibration at frequencies above about 63 Hz to 125 Hz, as illustrated in Figure 7.1.

Note that for the tunnel structure to provide significant shielding, the wavelength must be smaller than the tunnel diameter. For a typical 5 m diameter tunnel to have significant shielding in the 30 to 120 Hz range, wave speeds would have to be in the 200 to 600 m/sec range which includes the range for shear waves and the lower range of pressure waves in soil.

No analytical model of a hollow circular cylinder in an elastic medium has been found which could be used for modeling circular tunnels in soil. Greenfield's solution discussed above might be extended to include a hollow circular cylinder. Such extension would presumably provide qualitative information regarding tunnel/soil coupling and wave energy partition as a function of tunnel mass, wall thickness, and soil stiffness. Questions concerning the effective length of a vibrating tunnel could be also considered. The model parameters could, conceivably, be adjusted to approximate the larger and heavier double box structures, in spite of the non-circular shape of double box structures.

Rucker (Ref. A-58) discusses a finite element model of a subway structure in soil. The model is evidently a plane strain model incorporating 290 elements and 355 nodal points, the greatest element size being 1/5 of the shortest shear wavelength at 100 Hz in soil. The model was applied to experimental data to determine the input power spectrum of the excitation process. The values of the soil shear modulus were determined for use in the numerical model by cross-correlation between two measurement locations of groundborne vibration from passing trains.

The approach taken by Rucker certainly represents the state-of-the-art in modeling/subway soil interaction. With a sufficient number of elements, such an approach can be used to model any subway shape. However, finite element models have the limitation of requiring a large amount of computer time to evaluate the sensitivity to parameter modifications. The question of plane strain vs. a more complete three-dimensional finite element model needs to be addressed. Three-dimensional finite element modeling would necessarily involve a great many more degrees-of-freedom and corresponding increase in processing time and cost. Plane strain solutions are quite valuable, however, and provide significant insight into the mechanisms of soil structure interaction.

Finally, although numerical or analytical models for an elastic, homogeneous, and isotropic medium may provide information regarding coupling of the subway structures with shear and compression waves, a more realistic approach would include effects of saturated soils. A tentative approach would be to use the theory developed by Biot (Ref. C-29) for elastic wave propagation in saturated porous media and consider the motion of the rigid sphere as an approximation. The primary goal would be to estimate the relative partition of energy between shear waves, the slow compression wave, and the faster, lightly damped, compression waves.

7.6 VIBRATION PROPAGATION AND ATTENUATION IN SOIL

Wave propagation in soils and rocks is the subject of much study and experimentation by researchers in the fields of foundation design (Ref. B-8, B-4), soil-structure interaction and earthquake response analysis (B-24), and geophysics.

Modeling soil, and to a lesser extent rock, can be difficult. Soil is not necessarily homogeneous, isotropic, nor solid, but rather a multi-layered porous medium at various degrees of saturation. For most purposes, limited by practicality, analytical modeling of soil or rock usually incorporates layered elastic models, each layer being homogeneous and isotropic or possibly porous and saturated. These models are usually restricted to linear theory for displacement amplitudes similar to those of groundborne vibration from rapid transit systems. These idealized concepts of soils are usually sufficient for interpretation of experimental data and qualitative analysis. All of the analytical wave propagation and attenuation models encountered in the literature concerning groundborne vibration from rapid transit systems are of the above type.

Following is a short introduction to basic wave types for the media usually considered in the literature, and a discussion of models for attenuation due to spreading and dissipation.

7.6.1 Basic Types of Waves

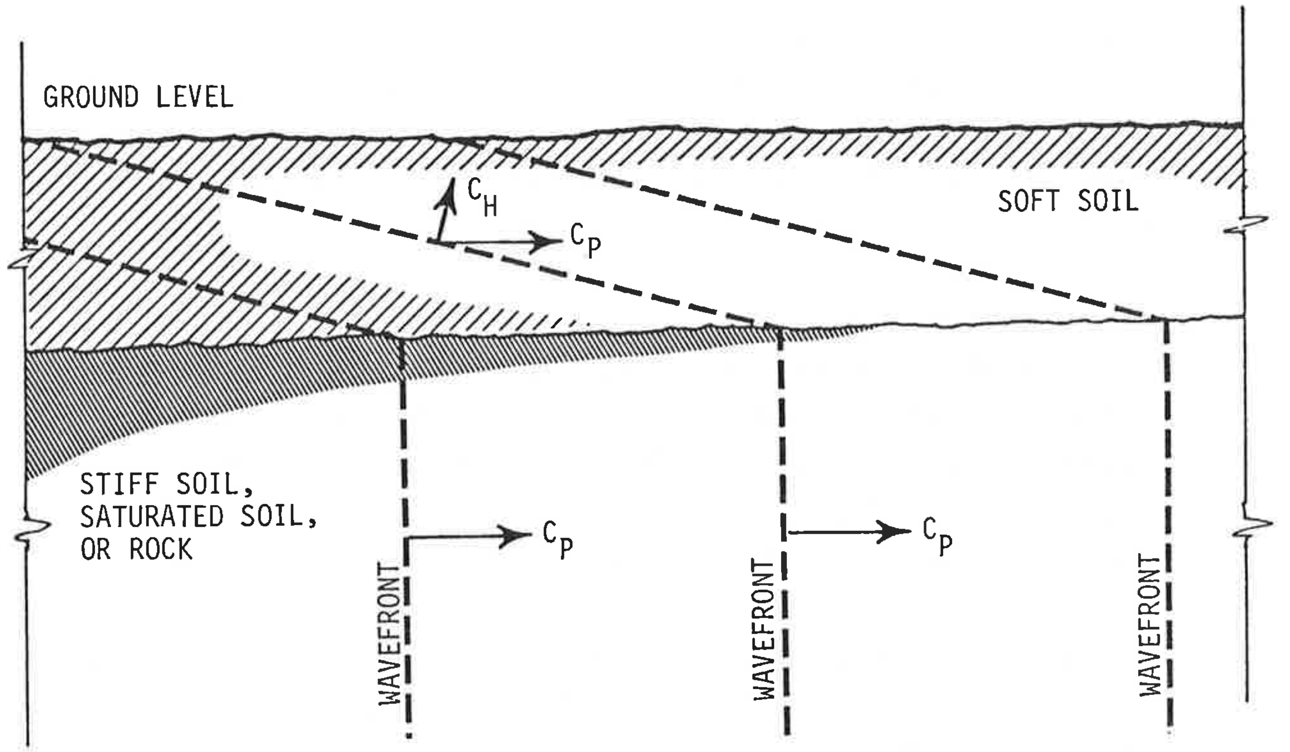
There are two basic types of body waves which may propagate in an infinite, homogeneous, isotropic, elastic medium (Ref. B-4). One is a shear wave or S-wave, otherwise identified as equivolumnal, rotational, or distortional wave. The second is a compression wave, or P-wave, which is also referred to as a dilatational wave. These two waves propagate independently and only interact at interfaces between media with differing elastic properties, or at a free surface. At such boundaries, wave energy is be exchanged between the two types of waves and can combine to form special types of waves such as Rayleigh, Love, and head waves. Of the waves discussed in the literature, with a few exceptions, only the body waves (i.e. shear and compression waves) and Rayleigh waves, have been associated with groundborne vibration.

The Raleigh wave can be an important component of groundborne vibration. Rayleigh wave's propagate at a speed slightly less than that of shear waves. Due to the nature of the Rayleigh wave, the motion associated with it is confined to within about one wavelength of the surface, and its spreading loss is much less than that of bodywaves. Consequently, at a significant distance from the source, Rayleigh waves can be the major cause of surface motion.

A second type of surface wave is the Love wave, involving a horizontal shear wave propagating within a soft layer at relatively slow speed. The layer can be, and usually is, the top-most layer of a multi-layered half-space. This layer acts as a wave guide by which energy is propagated over greater distance when compared with the case of an homogeneous medium.

Another type of wave that could be significant to propagation of groundborne vibration is the head wave, illustrated in Figure 7.2. Head waves develop in layered media when the compression wave speed is greater in the lower strata. The disturbance of the wave in the lower strata creates a disturbance in the upper medium, this disturbance is called the head wave.

For a porous saturated medium, the situation becomes even more complex since three types of body waves may occur. The first and slowest is the shear wave associated with the elastic solid. The second and third are coupled compression wave modes involving both the solid and fluid material. The first coupled compression wave, a low speed wave closely associated with the solid, occurs with significant damping and at a speed greater than that of the shear wave. The second coupled compression wave, faster than that normally associated with either the solid or the fluid, is relatively lightly damped. A coupled compression wave has been conjectured as responsible for the relatively long distance propagation of groundborne vibration from the Toronto subways (Ref. A-52).



C_H = PHASE VELOCITY OF HEADWAVE NORMAL TO WAVE FRONT IN SOFT UPPER SOIL LAYER

C_P = PHASE VELOCITY OF HEADWAVE IN SOFT SOIL AND IN STIFF SUBSTRATA IN HORIZONTAL DIRECTION

FIGURE 7.2 ILLUSTRATION OF HEADWAVE IN UPPER SOIL LAYER CAUSED BY FASTER WAVE IN LOWER STRATA

7.6.2 Attenuation with Distance

As waves propagate through soil, or in the ideal situation, an infinite medium, they experience attenuation due to geometrical spreading and dissipation. Accordingly, propagation through an elastic medium is modeled as

$$A = A_0 \, G \exp\left[-a(r-r_0)\right]$$

where "A" is the amplitude at distance "r" from the source, "A_0" the amplitude at distance "r_0" from the source. "G" is a function of "r" and "r_0" representing attenuation due to geometric spreading, and "a" is the attenuation coefficient.

7.6.2.1 Geometric Spreading -- The geometric spreading factor "G" depends on the wavefront geometry and is a result of conservation of energy. For a spherical wave, $G=(r_0/r)$ where "r" is the radius from the center of the sphere. For a line source such as a rapid transit line, $G=(r_0/r)^{1/2}$. For a plane wave, G=1. We may assume the attenuation due to geomtric spreading is only a function of the distance from the source and, for a particular region, the factor G may be approximated by the following function:

$$G = (r/r_o)^{-n}$$

where n is a constant characteristic of the particular region. Thus, the propagation model assumes the form

$$A = A_0 \, (r/r_0)^{-n} \exp[-a(r-r_0)]$$

The values of n for various wave types and sources are:

Wave Type	Source Type Point	Source Type Line
Body waves		
Shear	1	1/2
Compression	1	1/2
Surface Waves		
Rayleigh	1/2	0
Love	1/2	0

7.6.2.2 Dissipation -- The attenuation due to dissipation is less well understood than that due to spreading. The customary approach in geophysics is to describe the attenuation characteristics of rocks and soil in terms of a parameter Q, which is related to the ratio between the energy, E, dissipated in a cycle of vibration and the maximum stored elastic energy W by the following relation:

$$1/Q = - E/6.28W$$

In differential form, this becomes

$$1/Q = (-T/6.28W)dW/dt$$

where T is the period of the vibration. Integrating the above expression will result in:

$$W = W_0 \exp(-6.28t/QT) = W_0 \exp(-6.28ft/Q)$$

where f is the frequency of the vibration. The attenuation or absorption coefficient a is related to the Q-factor as

$$a = 3.14f/(Qc)$$

Since energy is proportional to the square of the amplitude of vibration, we have

$$A = A_0 G \exp [-3.14 ft/Q]$$

where G is the factor due to geometric spreading of the wave.

For a continuous traveling wave, the distance $(r-r_0)$ is related to the angular frequency f, the wavelength L, and the wave velocity c by the following relation

$$(r-r_0) = ct = fLt$$

Hence,

$$A= A_0 G \exp [-3.14(r-r_0)/(QL)]$$

or

$$A= A_0 G \exp [-3.14 f(r-r_0)/(Qc)]$$

In the engineering literature, the parameter called the loss factor is often used. The loss factor is related to the Q factor as:

$$N = 1/Q$$

Thus, the loss factor is related to the absorption coefficient as:

$$a = 3.14Nf/c$$

Gutowski, et al (Ref. A-59) review the current literature and simple models of spreading and damping effects of ground vibration, and point out that there exist a number of gaps in knowledge of attenuation laws. Specifically, Gutowski and Dym state that attenuation due to dissipation in dB between two points has been experimentally determined to be proportional to the logarithm of the number of wave lengths between two points for a particular set of data. These authors show that for a variety of surface soils the attenuation in decibels is related to distance traveled by

$$A = k \log (x/L)$$

where x is the distance traveled and L is the wavelength. Fitting this relation to a large number of data, they give the following results:

Soils	k
Clay	20
Saturated Clay	14
Dry Sand	12
Alluvial Fill	11
Wet and Dry Sand	10

These findings are furthermore indicated by Gutowsky and Dym to be in contradiction to the usual theoretical models proposed by Barkan (Ref. B-8) and others, namely that the amplitude attenuation due to dissipation be of the form:

$\exp(ax)$ (frequency independent)

$\exp(3.14Nf\ x/c)$ (proportional to the number of wavelengths)

where the variables a, N, f, x, and c are as defined above.

Representative soil absorption coefficients "a" for the former of the above equations are:

Soil Type	Absorption Coefficient (1/ft)	(1/m)
Water-saturated clay	0.012-0.037	0.040-0.120
Loess and loessial soil	0.030	0.100
Sand and silt	0.012	0.040

For the latter equation, representative values of the loss factor "N" are:

Soil Type	N(1/wavelength)
Clay	0.50
Loess	0.30
Sand	0.10

The above values for "a" and "N" are from Barkan (Ref. B-8).

The loss factor is generally applied equally to shear waves and compression waves, resulting in a lower attenuation rate with distance for compression waves than for shear waves. Part of the problem in using these models is the extent to which the loss factors for shear and compression waves may be assumed equal. Work in the area of geophysics regarding propagation and dissipation in rocks may shed some light in this regard, as discussed below.

Rocks and soils, when subjected to small amplitude oscillatory stresses do not depart seriously from being perfectly elastic. But some of the strain energy always is converted into heat and is lost. The mechanisms by which the conversion takes place are collectively termed "internal frictions." There are numerous mechanisms of internal friction and we have no certain knowledge which of them is most important in rocks, soils, or the earth as a whole.

There are very few studies which compare the loss factor for compression, shear, and Rayleigh waves in earth media. According to some recent experimental works (Ref. B-25, B-26), the loss factor in the shear mode is about equal to that in the compressional mode if the rock is dry. The loss factor for both modes increases several folds when water enters the media; the rate of increase for the compressional mode is greater than that for the shear mode. However, upon saturation the loss factor for the compressional mode decreases greatly, almost to the value for the dry case and becomes much smaller than that for the shear mode.

For most crystalline rocks, the energy loss factor is about 1 to a few percent. For some cap rock, it increases to over 10 percent. For Navajo sandstone, it goes to 30 percent, and for Pierre shale in Colorado it can be as large as 60 percent (B-27).

Some peculiar differences exist between the energy loss factor for the compressional wave and that for the shear wave. Some values are listed below for ease of comparison (Ref. B-27):

Rock Type	Loss Factor (%)		
	Compression	Shear	
conglomerate, Jelm	.41	.95	
limestone, Pa.	.56	.19	(dry)
limestone, Solenhofen	.91	.51	(dry)
sandstone, Berea	.32	1.2	
shale, Pierre	2.9	9.5	(in situ)

As stated at the beginning of this section, the relative magnitude of the shear and compression loss factors appears to vary with the environmental conditions.

There are a few scattered values for different soils in texts on foundation design (Ref. B-8, B-4). Detailed information is given by Ungar and Bender (Ref. A-2) which yields the generalized results as follows:

Material	Loss Factor N - %
Rock	1
Sand, silt, gravel, loess	10
Clay, clayey soil	50

A recent study of San Francisco Bay mud by W. Silva (private communication) yielded compressional loss factors ranging from 5 to 50 percent.

The loss factor has been measured for rocks in the laboratory by a number of techniques. A remarkable fact is its insensitivity to frequency, over a range from a few Hz to 10^6 Hz. Examples are listed below:

Brea Sandstone		Wingate Sandstone	
Frequency (Hz)	N (%)	Frequency (Hz)	N (%)
10,000	1.8	8100	2.0
335	1.5	170	1.9
77	1.3	67	1.8

The differences in N are all within experimental error ($\pm$20%). This particular characteristic makes rocks stand out as different materials from the "standard linear solid" (which shows a characteristic frequency at which N peaks out) or metals (which usually show a number of distinct peaks). The implication is that rocks contain a broad variety of relaxation mechanisms given such a wide range of relaxation times that frequency dependence is smeared out.

Whatever the sources for this particular property of rocks are, the frequency-independence in N makes it possible for us to estimate N for a particular region (or rock specimen) from the recorded vibration signals of a traveling wave. We shall show the basic principle:

Assume there are two recording stations at which the vibrations due to the same source are recorded. Fourier analyzing the records gives A_1 and A_2 for the two records. These are related to the "N" of the region traversed by the waves by the following equations:

$$A_1(f) = A_o(t)\ G_1 \exp\ [-3.14\ Nf\ (r_1 - r_0)/c]$$

$$A_2(f) = A_o(t)\ G_2 \exp\ [-3.14\ Nf\ (r_2 - r_0)/c]$$

where "N" is assumed to be frequency independent. Dividing the two relations, gives

$$A_2(f)/A_1(f) = G_2/G_1 \exp\ [-3.14\ Nf\ (r_2 - r_1)/c]$$

Plotting Log $[A_2(f)/A_1(f)]$ against f, the slope of the resulting curve yields N, where the slope is given by:

$$-3.14\ N\ (r_2 - r_1)/c$$

At shallow levels of the earth's crust N is quite low, usually less than 1 percent and may be dominated by "friction" at grain boundaries.

If this independence of vibrational frequency in the energy loss factor, generally observed for many different types of rocks, is also the case for soils, then the absorption coefficient for the propagation of groundborne noise and vibration must be a linear function of frequency, as assumed by Gutowski and Dym (A-59).

If we have a number of stations at different distances "r" from a vibration source, and if the amplitudes of the traveling waves at these stations are registered, a least-squares procedure may be used to fit the experimental data to the above equation to evaluate the attenuation due to damping and the attenuation due to geometrical spreading. This approach is perhaps the most practical for characterizing groundborne vibration propagation for general types of soils.

7.6.2.3 Effect of Saturation on Attenuation -- Water is important in the surface layers of the earth, and this introduces a variety of added complications. For example, rocks subjected to a high vacuum which removes water from the internal grain surfaces give much higher Q, in the range of 1000 to 2000 or alternatively, much lower N in the range of .01 percent to .05 percent.

In a very general sense, fluctuation of the water table would change the wave path and the velocity of the waves since saturated rocks and soil transmit acoustic waves at a very different speed as compared with dry or partially saturated rocks and soil. Given the degree of saturation in rock or soil, existing theories allow computation of velocities for various types of waves in the medium. It has been demonstrated that the computed results are not inconsistent with observation if we allow a number of essential assumptions and simplifications in the computation. Several publications have discussed this topic. A well-known work is that by O'Connell and Budiansky (Ref. B-28).

Insight into the effects of pore fluid in rocks on attenuation and the corresponding mechanism is gained by comparing laboratory results for attenuation of shear waves and compressional waves. Shear attenuation is minimal in dry rock, is greater in partially saturated rock, and is maximal in fully saturated rock. For compressional loss, while also at a minimum in dry rock and greater in partially saturated rock, the compressional loss is greatly reduced in fully saturated rock. In partially saturated rocks, however, the loss in compressional energy is about twice as large as shear energy loss, and both increase with degree of saturation. This continues until approximately 95 percent saturation, above which the compressional loss decreases to less than one third of the shear loss. This minimum in loss in compressional energy is also predicted by a mechanism involving flow between cracks (Ref. B-28). Since the water table and the degree of saturation of soil changes with time, the attenuation characteristics of soil layers could be extremely complicated.

7.6.2.4 Characteristic Frequency for Soil -- Although the empirical approach by Gutowski and Dym (Ref. A-59) provides a tentative method for predicting the level of groundborne noise and vibration, it will not predict the characteristic frequency which is so conspicuously present in the observed ground vibration spectra at many locations near transit systems. In fact, assuming that, like rocks, soils also have loss factors which are independent of frequency, then, as stated in Section 7.6.2.1, the absorption coefficient must be linearly dependent on frequency. Then, for constant amplitude A(0), the amplitude of vibration will then be related to frequency by

$$A(x) = A(0)e^{-kf}$$

where $k = Nx/c$ and is independent of frequency. Thus, the amplitude decreases exponentially with increasing frequency.

Or, if we assume, as Barkan claims (Ref. B-8), that the absorption coefficient is independent of frequency, the amplitude will then be independent of frequency.

An idealized standard linear solid will have a peaked loss factor at a characteristic frequency. Groundborne noise and vibration, on the other hand, show a peaked amplitude in a narrow range of frequency.

In short, nothing we know today about the damping characteristics of earth media or a simple idealized medium will predict the observation of the characteristic vibration at the ground surface. More likely, the "characteristic frequency" at which the vibration amplitude peaks reflects the thickness of the top soil, the depth of groundwater table, subway radiation characteristics, mechanical resonances of the vehicle and track support system, and high-frequency roll off due to dissipation.

7.6.2.5 Effect of Soil Layering and Surface -- Vibration amplification as well as attenuation may be expected at layer interfaces. Secondly, energy may be exchanged between various wave types at layer interfaces. At the soil surface, a Rayleigh wave may develop, as discussed above with respect to subway coupling. In general, a type of resonance, subject to dissipation, may develop within soil layers, leading possibly to a very complex transfer function between lower soil strata and the surface.

Of the literature concerning groundborne vibration from transit systems, the approach taken by Ungar and Bender (Ref. A-2) is most representative of attempts to compute the effect of layering. With respect to foundation design and general ground vibration work, the standard texts by Barkan (Ref. B-8); by Richart, Hall and Woods (Ref. B-4); and Ewing, Jardetsky, and Press (Ref. B-35) provide analytical models of wave propagation in layered media. Finally, a great deal of research on the theory of propagation in layered media is provided by researchers in the field of geophysics. Specifically, Harkrider (Ref. C-28) has developed a general matrix formulation for the problem of a point vibration source within a multi-layered three-dimensional elastic half-space, using displacement potentials.

Gregory (Ref. C-34) has developed a solution to the problem of a harmonic, uniform normal pressure acting on the wall of a cylindrical cavity in a two-dimensional, homogeneous, isotropic, elastic half-space.

Although Gregory's analytical model is not exactly the most appropriate model, it is still representative enough of the actual problem to give useful information on the effect of the free surface. Further, an examination of the vibration data obtained by Rucker (Ref A-58) indicates that the soil vibration, in the case of double box construction, is dependent to a large degree on what appears to be compression waves emmanating from the bottom of the tunnel. Alternatively, it may be possible to approximate the two-dimensional line load with a symmetric normal pressure which in the limit becomes a concentrated load.

A solution to the more complicated problem of conical waves radiated by a submerged cylinder (e.g., gas pipe) into an elastic half-space has been developed by Jette and Parker (Ref. C-36, C-37). The treatment assumes azimuthal symmetry for the source, i.e., the source strengths are symmetric about the cylinder axis. Image sources above the surface are employed to give zero normal and tangential stress at the surface. The solutions reduce to the two-dimensional case, i.e., that of Gregory's model, as the phase velocity of the conical waves in the direction of the cylinder axis approaches infinity. In the second paper by Jette, et al (Ref. C-37), a hollow circular cylinder is included in the analysis to represent a gas pipe and theory is compared with measurement data for surface displacements as a function of frequency. The distance at which Rayleigh waves form as a function of source depth and trace velocity along the cylinder axis is discussed.

The solutions of Gregory and of Jette and Parker should provide meaningful information regarding the effect of the soil surface on groundborne vibration produced by a buried source. Additionally, the approach of Jette and Parker provides a first order approach to determine the effect of tunnel wall mass and stiffness on soil/tunnel coupling. In this case, "first order" refers to the case of an axi-symmetric source pressure distribution.

7.7 BUILDING RESPONSE TO GROUNDBORNE VIBRATION

7.7.1 Earthquake Response

The problem of analyzing the response of structures to specified ground motion is a central problem in earthquake engineering and has been the subject of considerable research over the last four decades. The results of this research on the earthquake response problem may seem to be directly applicable when the excitation is associated with groundborne noise and vibration.

Ground motions during the larger earthquakes have the appearance of broad-frequency-band random processes with significant amplitudes up to 25 Hz. This frequency range includes the lower frequencies of natural vibration of many buildings. (The fundamental frequency may be around 10 Hz for a one-story building and around 0.5 Hz for a 30-story building.) The contributions of all the vibration modes with natural frequencies less than 25 Hz should be considered. However, generally only the first few of these modes need be included in the analysis because the response of buildings to horizontal ground motion is primarily contained in the lower modes of vibration. The response of many buildings to vertical ground motion is relatively small and is usually not considered with respect to earthquake motion.

In the simplest mathematical model employed for computing the lower horizontal vibration modes of buildings, the mass of the building is considered to be concentrated at the floor levels, the floor systems are assumed to be rigid so that the lateral displacements, relative to the ground displacement, are due entirely to the deformations in columns, thus leading to a single degree-of-freedom per floor. This "shear building" idealization is adequate for low-rise moment-resisting frame buildings. For medium and high rise buildings of this type and buildings with other structural systems, the effect of joint rotations and axial deformations in columns may be significant; consequently, refined mathematical models and analysis procedures have been developed to form the lateral stiffness matrix of buildings including these additional degrees of freedom.

The mode superposition method (Ref. B-24) is most effective in linear analysis of response of the above mentioned idealized systems to earthquake ground motion. Transformation to modal coordinates leads to an uncoupled set of differential equations, one for each normal mode of vibration, identical in form to the equation for a single DOF system. Uncoupling of the equations is, of course, a very attractive feature of the modal method. A more significant aspect of the method is that, in general, only a few modal equations need to be solved because, as mentioned earlier, response to earthquake ground motion is primarily contained in the lower modes of vibration. Even for a building with many stories, say 20 or more, three to five modes will usually suffice to produce satisfactory results.

During the past decade, recordings of motions of several buildings during actual earthquakes have been obtained. Typically, accelerations at three locations -- base, mid-height and top -- in a building have been recorded. Considering the base acceleration as the excitation, the response of the building has been analyzed by the mode-superposition procedure mentioned above and compared with recorded accelerations. The analyses are capable of producing satisfactory agreement with recorded motions provided the stiffness, mass, and damping properties used in the analyses are representative of those effective during the earthquake (Ref. B-30). Thus, accurate values and detailed description of these properties are essential if the analytical results are to agree with recorded responses.

The motion at the base of a building due to nearby rail transit operations is three-dimensional, including horizontal and vertical components with significant motions in the frequency range of 10 to 200 Hz. Response of the building to these high-frequency base motions would be primarily in the form of local distortion of wall panels and other building components. Noise in the building arises from these local distortions of structural elements. Unlike earthquake response, the vibration and noise in a building due to the vertical component of base motion is important; hence, the response to both components of base motion should be considered. Thus, mathematical models of buildings should be capable of (1) accurately predicting the vibration modes of structures with natural frequencies up to 200 Hz, and (2) satisfactorily representing the response to vertical ground motion.

In principle, these requirements could be satisfied by refining the mathematical models that have been employed for earthquake response analysis, resulting in detailed finite element idealizations of the structure with several hundred degrees of freedom. Accurate values and detailed description of the stiffness, mass, and damping properties would be required to accurately predict the response, especially at high frequencies. Considerable computational effort would be required in analysis of such a system. The total computational effort required in analysis of the many buildings in the vicinity of rail transit lines would be prohibitive. Therefore, straightforward extension of standard mathematical modeling and response analysis procedures does not appear to be a practical approach to the prediction of vibration and noise in buildings arising from rail transit operations.

For buildings supported on soft soil, the effects of soil-structure interaction must be considered in the analysis of earthquake response. Consider the aforementioned planar "shear building" idealization of a multi-story building, supported through a rigid circular foundation mat at the surface of a half space composed of a homogeneous, viscoelastic soil or rock. The foundation impedances relating forces and displacements for the rigid plate on a half space depend on the excitation frequency. The governing equations for the structure-foundation system are written most conveniently in the Fourier transformed frequency domain. The steady state response to harmonic ground motion at a particular excitation frequency is determined by solving the frequency domain equations. Efficient solution of these equations is possible by extensions of modal analysis concepts in which only the lower few modes that have significant contribution to the response are included (Ref. B-31).

A reasonable approximation to the maximum building response may be obtained by assuming that soil-structure interaction influences only the response component contributed by the fundamental mode of vibration (Ref. B-32). This component may be evaluated by a simple, practical procedure, utilizing published data and charts. The contributions of the higher modes to the response may be determined by the above mentioned procedures disregarding the effects of interaction.

In addition to the mass, stiffness, and damping properties of the structure, the properties of the foundation medium are also required in the analysis. These include the shear modulus of elasticity, the mass density, Poisson's ratio and the specific energy loss factor. Within the range of values that are of interest in practical applications, the response of the structure is generally insensitive to variations in the Poisson's ratio, but it can be influenced significantly by the other soil parameters.

By appropriate modifications and extensions, the analysis procedure outlined here for structures supported on the surface of a homogeneous half space has also been applied to systems with embedded foundations and layered foundation media. It can also be generalized to relax the assumption of a rigid foundation mat. Foundation flexibility may be a significant factor in the response of structures of very large plan dimensions. A problem requiring additional research is the response of structures supported on isolated spread footings. Probably the most pressing current need, however, is for studies of the dynamics of structures supported on pile foundations.

The influence of soil-structure interaction on building response is related to the shear and moment imposed by the building on the foundation. Because the most significant base shear and moment are associated with response components contributed by the lower modes of vibration, soil structure interaction has the most influence on these response components. This is the basis for the assumption in the above-mentioned simple analysis procedure that soil-structure interaction influences only the earthquake response component contributed by the fundamental mode of vibration.

Noise in buildings arising from rail transit operations is associated with vibration in the higher modes with natural frequencies in the range of 10 to 200 Hz. The base shear and moment associated with vibration in these modes is expected to be very small, to the point of being almost negligible. Consequently, soil-structure interaction is expected to have little influence on the contributions of these higher modes to the noise and vibration.

In analyzing the response of structures to earthquakes, it is normally assumed that all points of the foundation are excited simultaneously and experience identical motion. However, different frequency components of the seismic waves may have different velocities and each is associated with a separate wavelength. The resulting spatial variations in the ground motion have the influence of reducing the effective translational ground motion for structures with stiff mat foundation but adding torsional excitation. The significance of these effects depends on the excitation frequency (or wavelength) of the associated seismic wave; these effects increase for high frequency (or short wavelength) excitations (Ref. B-33).

Groundborne vibration and noise arising from rail transit operations is associated with motions in the 10 to 200 Hz frequency range. The wavelengths of motions at these frequencies would be shorter than the plan dimensions of many structures. Thus the resulting spatial variations in the ground motion are expected to significantly affect the vibration and noise in buildings.

Thus, the conclusions of the above discussions are:

(1) Given the free-field ground motion at the site of a building due to nearby rail transit operations, predictions of resulting vibration and noise in the building may be based on analyses neglecting soil-structure interaction.

(2) Straightforward extension of mathematical modeling and response analysis procedures that have been employed in earthquake engineering does not appear to be a practical approach to the prediction of vibration and

noise in buildings induced by rail transit operations. Such an approach would require accurate and detailed description of the stiffness, mass, and damping properties of the buildings. Detailed dynamic analyses of all buildings in the vicinity of rail transit systems would require prohibitive computational effort.

(3) Spatial variations in the ground motion arising from traveling wave effects should be considered in the analysis of vibration and noise in buildings induced by nearby rail transit operations.

7.7.2 Lumped Parameter Models

Richart, et al, (Ref. B-4) and Woods (Ref. B-3) summarize the literature concerning interaction of foundations with an elastic half space for vertical, torsional, rocking, and horizontal motion. In particular, lumped parameter models are presented for such interaction in response to excitation forces. These models can be re-cast to model the response of massive foundations to free surface ground vibration which would otherwise exist in the absence of the foundation, as discussed by Holzlohner (Ref. B-3).

The resulting equation for vertical motion is:

$$M\ddot{z} + C_z\dot{z} + K_z z - L_z = C_z\dot{z}_f + K_z z_f$$

where z is the vertical displacement and z_f is the vertical displacement of the soil in the absence of the foundation.

For coupled horizontal and rocking motion, the equations of motion are:

$$M\ddot{x} + C_x\dot{X} + K_xX - C_xh_o\dot{\theta} - K_xh_o\theta - L_x = [C_x\dot{X}_f + K_xX_f]$$

and

$$I\ddot{\theta} + [C_\theta + h_o^2C_x]\dot{\theta} + [K_\theta + h_o^2K_x]\theta - h_oC_x\dot{x} - h_oK_xx - L_\theta$$

$$= [C_\theta\dot{\theta}_f + K_\theta\theta_f - h_oC_x\dot{X}_f - h_oK_xx_f]$$

x = Horizontal Displacement

θ = Angular displacement

h_o = height of center of mass.

The stiffness and damping coefficients (K_Z, K_X, K_θ , C_Z, C_X, and C_θ) are defined for circular footings in Table 7-1. The loads representing the building are L_z, L_x, and L_o for the vertical, horizontal, and angular displacements. The translational and angular displacement variables are identified in Figure 7.3. Similar expression for torsional vibration about the vertical (z) axis can also be determined, but such vibration is not considered here. Equivalent radii are given in Table 7-2 for rectangular footings.

The coupling loss as a function of frequency can be determined from the above equation for vertical vibration. For example, setting $z_F = z_oe^{iwt}$, where z_o is a complex coefficient, and representing the building load by the impedance, Z_z:

$$L_z = Z_zz_o$$

TABLE 7-1 VALUES OF EFFECTIVE STIFFNESSES AND DAMPING COEFFICIENTS FOR CIRCULAR FOUNDATIONS (Ref. B-3)

Mode	Vertical	Damping
Vertical	$K_z = \frac{4Gr_o}{1-\nu}$	$C_z = \frac{3.4r_o^2}{1-\nu} \sqrt{\rho G}$
Horizontal	$K_x = \frac{32(1-\nu)Gr_o}{7-8\nu}$	$C_x = \frac{18.4(1-\nu)r_o^2}{7-8\nu} \sqrt{\rho G}$
Rocking*	$K_\theta = \frac{8Gr_o^3}{3(1-\nu)}$	$C_\theta = \frac{0.8r_o^4 \sqrt{\rho G}}{(1-\nu)(1+B_\theta)}$

$$*B_\theta = \frac{3(1-\nu)}{8} \frac{I_\theta}{\rho r_o^5}$$

G = Soil Shear Stiffness

ν = Soil Poisson's Ratio

ρ = Soil Density

I_θ = Mass moment of inertia about y-axis through center of mass

m = Mass of Foundation

TABLE 7-2 EQUIVALENT RADII FOR RECTANGULAR FOOTINGS OF LENGTH, L, AND WIDTH, W*(Ref. B-3)

Mode	Radius r_o
Vertical	$(\frac{LW}{\pi})^{\frac{1}{2}}$
Horizontal	$(\frac{LW}{\pi})^{\frac{1}{2}}$
Rocking	$(\frac{LW^3}{3\pi})^{\frac{1}{4}}$

*$\frac{L}{W} \leq 2.$

$$x = x_b + h_o \quad \theta$$

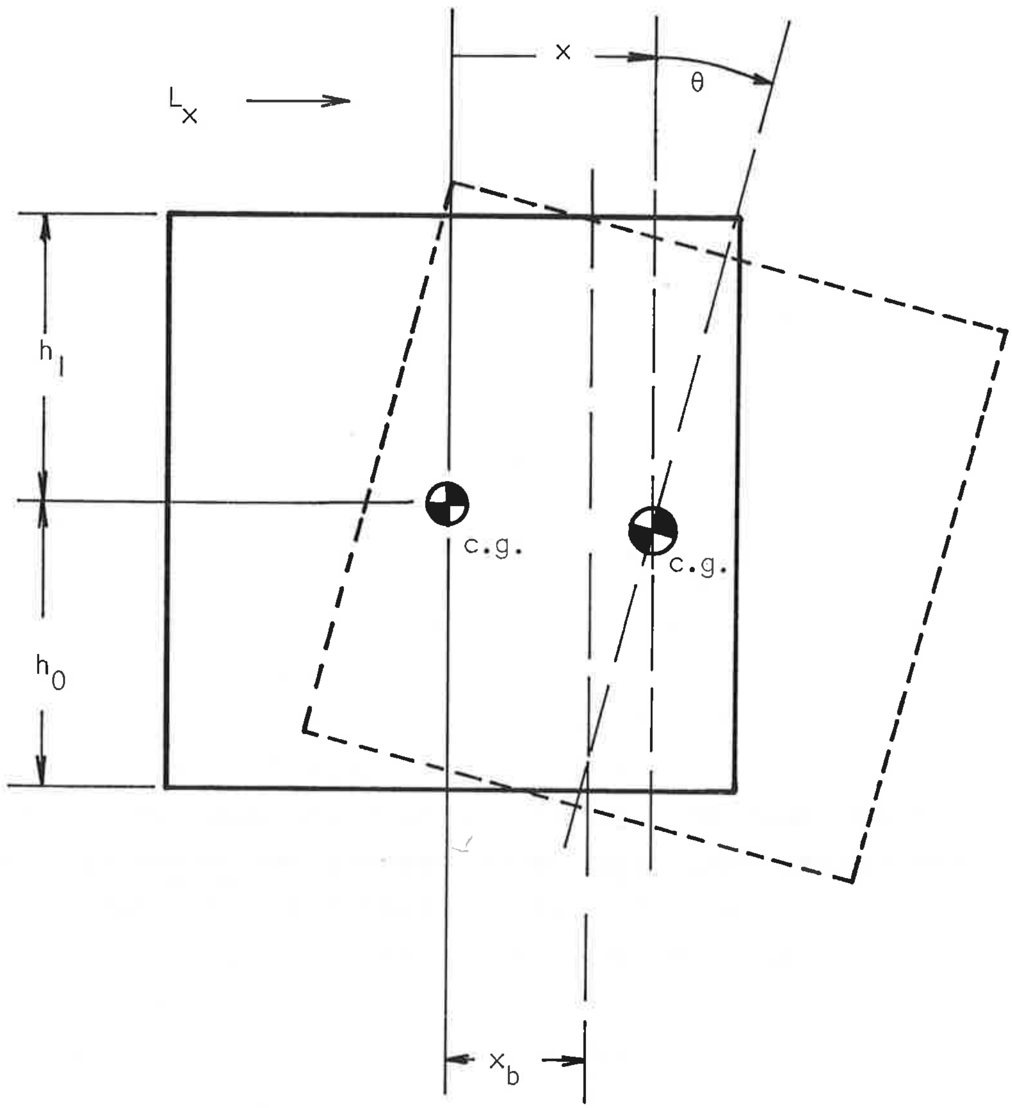


FIGURE 7.3 VARIABLES USED IN LUMPED PARAMETER MODEL OF FOUNDATION RESTING ON HALFSPACE

The coupling loss for pure vertical vibration becomes

$$L_c = 10log \frac{w^2C_z + K_z{}^2}{(K_z + wIM\{Z_z\} - w^2M)^2 + w^2 (C_z - Re\{Z_z\})^2}$$

The real part of Z_z is negative by requirement for stability. Thus, the coupling loss is determined by soil stiffness, radiation damping, and the building load.

Analogous formulas may be derived for horizontal and rocking modes. Together, they can be used to estimate the response to a variety of wave types. For example, the vertical, horizontal, and angular displacement of a point on a free soil surface can be determined for a Rayleigh wave. Using this information for input, the response of the foundation can be determined, provided that a meaningful representation of building load can be obtained.

As discussed above, the form of the building load impedance is quite complex due to many modes of vibration for walls, ceilings, floors, and so forth. For building columns resting on the foundations, a significant part of the column mass must be included for vertical motion. Indeed, the usual assumption is to include that portion of the building mass which is supported by the foundation. For large masonry buildings this added mass would be substantial. For single family residential structures, the building mass would be much less. In any case, caution must be exercised when making assumptions regarding building load in that it can be easily over-estimated. The problem of defining building load can be simplified for a structure supported on springs placed on the foundation, provided that the spring impedance is much less than the building impedance.

At audio frequencies a great many modes of vibration may be expected within a building. The result is that the building impedance may be lower than expected from a simple mass representation. When the modal density becomes large, one might expect that the building impedance is largely dissipative. Finally, one might conservatively assume that the foundation mass controls foundation motion, resulting for example, in an asymptotic form for the coupling loss for vertical vibration of

$$L_C = 10 \log C_z^2/M^2w^2$$

which gives an eventual 6 dB increase of coupling loss per frequency doubling. This assumption would give a worst case or conservative estimate of foundation response. Analogous asymptotic relations can be determined for horizontal and rocking modes.

Evident in the above equation for vertical, horizontal, and rocking motion is that the damping coefficients play a significant role in coupling of the foundation to groundborne vibration. The damping coefficients represent radiation damping, sometimes called geometrical damping.

The presence of the radiation damping in the coupling of incident or free surface vibration with the foundation reflects the radiation reaction due to scattered wave energy in the soil. Thus, at sufficiently high frequencies the radiation reaction term will dominate the stiffness term and, neglecting effects of building impedance, at most a 6 dB increase per frequency doubling in coupling loss may be expected above resonance, i.e., at audio frequencies. This latter result is perhaps the most important conclusion of this discussion. Also, because the radiation reaction is directly proportional to shear stiffness, one may expect less coupling loss for stiff soils than for soft soils.

The agreement of the lumped parameter models with the more exact continuum models is good at least up to about 50 to 100 Hz for foundations with equivalent base area radii of 0.5 to 1 meter. At higher frequencies, the agreement is not discussed in the literature. However, as the shear wave length in soil approaches the dimension of the foundation, the geometric damping factor will likely increase, thus increasing coupling. At very high frequencies, i.e., at 500 to 1000 Hz or higher, the wavelength of compression waves in soil begin to approach the dimension of a foundation footing of perhaps 1 meter width in which case the bulk acoustic impedance of the soil will enter the damping term significantly. Because the acoustic or dilatational impedance is generally higher than the shear wave impedance, significantly stronger coupling of the foundation with soil would be expected relative to that predicted by the above lumped parameter model.

For large foundations, significant phase differences can be expected for free-field amplitudes at the soil foundation interface. Holzlohner, in discussing the state-of-the-art in soil-structure interaction analysis, indicates that an averaging procedure may be used to calculate the free-field excitation from the free-field displacements, evidently with good success (Ref. B-3).

All foundations do not rest on the soil surface. Most are embedded in the soil, although the condition or degree of contact between the sides of the foundation and the soil will significantly affect the response. Kausel reviews recent literature regarding embedded foundation models, and Woods discusses the effects of embedment in some detail (Ref. B-3). Essentially, the equation of motion may still be represented in terms of lumped parameters, but the expression for stiffness and damping become quite complex. A number of general conclusions are given by Woods for embedded foundations. Namely, 1) the amplitude of vibration decreases with depth; 2) the resonance frequency increases with depth; 3) the damping increases with depth.

The problem of modeling a pile foundation in soil is considerably more difficult than modeling foundations resting on soil surfaces. A pile normally is supported on bedrock or at least on stiffer soil layers. In the former case, vertical motion will be largely controlled by the bedrock, and high frequency resonances can be expected when the longitudinal wavelength in the pile is equal to one-fourth of the pile length (Ref. B-4). In the latter the surrounding soil as well as supporting strata will have a combined effect on vertical motion. In both cases, the lateral or transverse response of the pile will be strongly influenced by the surrounding soil. Again, Richart, et al, provide a detailed discussion concerning the response of piles to lateral forces.

Navak reviews recent extensive literature regarding the effect of piles on dynamic response of footings and structures (Ref. B-3), presenting two models of piles subject to horizontal soil motion, one more rigorous than the other. In the more rigorous model, modal resistance factors are used to represent soil reaction against various transverse bending modes of the pile. The resistance factors are determined from solutions to the equation of motion for a linear visco-elastic medium with hysteretic damping, using cylindrical coordinates. The resulting formulae are very complicated but evidently computing costs are low. Thus, extensive parametric studies may be conducted.

In the simpler approach, Novak derives the soil reaction under the simplifying assumption of only horizontally propagating waves. Thus, soil resistance per unit pile length for horizontal vibration are represented by complex stiffness constants which are complicated functions of frequency but not of pile depth. The soil reaction to the pile at a particular depth is then given as proportional to the pile displacement at that depth. In this way, a closed form solution for impedance functions can be obtained for all lateral vibration modes using a Bernoulli-Euler beam model for the pile. Evidently, agreement of the simple model with more exact finite element models is good.

Novak finally indicates that the impedance of the pile foundation can be represented in a manner analogous to that discussed above with respect to foundations resting on a half space. Although the stiffness and damping coefficients are frequency dependent, they can often be taken as approximately frequency independent over a particular range of interest. Of course, a suitable representation of the building impedance must be obtained. Bender, et al, (Ref. A-158) employ this approach for modelling the effect of resilience placed between the pile cap and building columns in large buildings, as discussed in Chapter 6. The approach is based on empirical data for vertical pile vibration.

7.7.3 Audio Freqency Building Vibration

A model has been recently proposed by Lubliner (Ref. A-93) for predicting the floor-to-floor attenuation of audio-frequency vibration and noise in large multi-story buildings. Transverse bending and shear waves in building walls and story columns are assumed to be the significant modes of vibration transmission.

The transmission and attenuation of vibration from story to story depends strongly on the degree of coincidence between bending mode frequencies for neighboring story columns. Based on extensive numerical computation using a multi-degree-of-freedom model for generalized bending and shear modal vibration, an approximate relation among the percentage variation, U, of resonance frequencies from floor to floor, the floor-to-floor attenuation, a, in decibels, and the number of floors, n, is given as:

$$a = C_1 \, (U/n)^{1/2}$$

where C_1 is a constant equal to about 17.5. Thus, for a 2% frequency variation and 10 story building, the attenuation from floor-to-floor would be about 8 dB. For 1% frequency variation and a 30 story building, the attenuation would be about 3 dB.

Detuning of story columns can be accomplished by varying the area moment of inertia for steel columns, or by modifying the amount of re-enforcement of concrete columns. No supporting experimental data is presented for the model.

Detailed modeling of building structures may best be accomplished using methods such as statistical energy analysis (SEA) (Ref. C-14). The method has been applied to a variety of problems where high modal densities (number of vibration modes per unit bandwidth) may be expected for various building elements. The method, however, pre-supposes a detailed knowledge of coupling factor for vibration energy transmission from one building element to the next. A similar, if not greater, limitation exists for direct multi-degree-of-freedom models, however. The advantage of the SEA method lies in the ability to model complex dynamical systems where the number of vibration modes would normally make generalized multi-degree-of-freedom modeling prohibitively expensive.

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APPENDIX B

REPORT OF NEW TECHNOLOGY

This report represents the first time an effort has been made to review the state-of-the-art of the prediction and control of groundborne noise and vibration. The information in the report is most important in presenting recent techniques used by transit systems in controlling groundborne noise and vibration. Utilization of these techniques will continue to enhance efforts to reduce and control groundborne noise and vibration in the near future.

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