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A study of the development of construction vibration criteria and the potentially damaging effects of pile-driving vibration on historical and modern buildings.

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A STUDY OF THE DEVELOPMENT OF CONSTRUCTION VIBRATION CRITERIA AND THE POTENTIALLY DAMAGING EFFECTS OF PILE-DRIVING VIBRATION ON HISTORICAL AND MODERN BUILDINGS

by

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ABSTRACT

This study was undertaken for two main reasons: First, to determine if the vibration caused by a diesel pile-driver could have a damaging effect on a fragile historical structure, and second, to examine existing standards governing the exposure of buildings to vibration, and any methods for predicting these exposure levels.

Part of this thesis includes a comprehensive survey of the current state of international vibration standards, while Chapter Three includes a table of the most popular vibration prediction equations which have been developed to date. It should be noted that Canada has no limiting vibration standards for construction vibrations, or for vibrations affecting historical structures.

Vibration measurements were recorded at three sites using a Brüel & Kjær Frequency Analyzer. Some vibration signals were also tape recorded for Fast Fourier Transform (FFT) analysis. Diesel pile-driving vibrations, which reached a maximum of 0.036 mm/s Peak Particle Velocity at a distance of 15 metres from the Cleary Auditorium, and 0.176 mm/s at a distance of 25 metres from the Baby House, were not considered large enough to cause damage to either building. However, vibratory compacting recorded at the Bellewood Estates subdivision reached a maximum Linear RMS Velocity of 14.1 mm/s at a distance of 44 metres, and was considered to be of sufficient amplitude to cause possible damage to several nearby houses.

The natural frequencies and damping of the Baby House were also determined through FFT analysis of the tape recorded vibration signal.

Spectral Response analysis was found to predict vibration values of about the same magnitude as the other vibration prediction techniques explored in this thesis. However, each technique was found to be quite conservative, and over-predicted the vibration response by several orders of magnitude. Nevertheless, the Spectral Response method did have the advantage of being able to predict the frequencies of highest and lowest amplitude response. Further research in the area of Response Spectra is recommended.

It is also recommended that vibration standards dealing specifically with the special case of construction vibration, and with the impact of vibration on historical buildings be established.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

c	propagation velocity (m/s)
c_1	damping coefficient (N/(m/s))
E	Young's Modulus (Pa)
f	frequency (cycles/second)
h	height of building (m)
Hz	hertz (cycles/second)
K	constant
k	stiffness (N/m)
L	length of building (m)
m	mass (kg)
N	number of storeys
p	undamped circular natural frequency (rad/s) $=2\pi f$
p_d	circular natural frequency of damped system (rad/s)
R	distance (m unless specified)
T	period (seconds)
u	absolute displacement of the ground (m)
W	charge weight of explosive per delay (kg unless specified)
W_o	rated energy of pile driver (Joules)
y	displacement of the mass (m)
$\ddot{y}$	absolute acceleration of the mass (m/s^2)
δ	displacement relative to the ground (m)
λ	wavelength (m)
ρ	density (g/cm^3)
τ	time integral (seconds)
ξ	damping ratio (fraction of critical damping)

I. INTRODUCTION

In recent years, concern about the environmental impact of many industrial activities has increased dramatically. This concern has extended to such areas as the environmental impact of noise and vibration; areas which had formerly been considered the unavoidable byproducts of technological progress. In 1987, vibration was designated as an environmental contaminant by Ontario's Environmental Protection Act [1]. This thesis examines one aspect of the environmental impact of vibration, namely the potentially damaging effects of construction vibration on modern and historical buildings.

The objective of this thesis was to study the effects of construction vibration, and specifically the effect of pile-driving, on buildings. An ancillary purpose was to study the historical development of standards or criteria limiting acceptable vibration levels. Currently in Canada, there is no standard that defines a "safe" level of vibration that would guard against possible damage to structures. This thesis does not deal with the human annoyance effects of vibration, but solely with those vibrations that may be large enough to raise concerns about possible structural or cosmetic damage to a building.

This subject was selected for a number of reasons. An initial reason was that time and money were being expended by several concerns, such as the Windsor Historic Sites Association [2], in an effort to protect buildings from the damaging effects of vibration, but that the criteria for what constituted damaging vibration remained unclear. Second was the opportunity to make vibration measurements using modern vibration equipment, an uncommon occurrence in university life. The final, and key justification for this project was the occurrence of a very interesting vibration event, namely the pile-driving work that was planned adjacent to the Hiram Walker Historical Museum. This presented a rare opportunity to make vibration measurements where the concern about damage to a fragile building was quite real.

Much of this thesis deals with the historical development of vibration standards and analysis techniques relevant to structural damage. Various analytical and empirical formulae for the prediction of safe vibration limits, and the prediction of the peak particle velocity of vibration, are examined in detail.

Vibratory field measurements were made at three locations in the City of Windsor over a period of several months in 1990. At two of these locations, diesel pile-driving operations were recorded during a commercial building renovation. One measurement location was within the building being renovated, and the second location was inside a historical building adjacent to the renovation site. A third set of field measurements was made in two residential buildings that were near a highway expansion project. In this latter case building damage had been reported by several homeowners in the neighbourhood, so measurements were made of the vibrations induced by a vibratory compactor. This was done to assist in evaluating the likelihood of building damage through vibration. Analysis of these measurements, and comparison of these data to the values predicted through the analytical and empirical techniques developed earlier, forms the remainder of the work carried out on this topic.

Most technical papers dealing with the vibration effects of pile-driving date back to the mid-1970's. As such, newer techniques like response spectra analysis, which has been successfully used to study blasting vibrations, have not yet been applied to the field of pile-driving vibrations. This presented an opportunity to do some original work in this area, in an attempt to further the understanding of vibration propagation through soil, rather than rock, as is the case in most blasting studies. Therefore, response spectra techniques have also been applied to the field measurement data, in an effort to apply this method of analysis to the field of pile-driving.

II. LITERATURE SURVEY

The sources consulted in the preparation of this thesis have been divided into two categories. The first section of this chapter discusses key research work in the development of construction vibration analysis techniques, while the second section deals with the development of vibration standards and damage criteria.

2.1 Development of Analysis Methods

Most theories dealing with the effect of pile-driving and other construction vibrations originated from the study of blasting vibrations. The U.S. Bureau of Mines [3] was the first to establish damage criteria for residential structures in the 1940's. Blasting vibration amplitudes were measured using variable capacitance displacement transducers, and the output was recorded on an oscillograph. The results of these tests were plotted as displacement amplitude versus frequency on log-log paper. The U.S. Bureau of Mines divided their data into three damage classifications:

-major damage (fall of plaster, serious cracking)
-minor damage (fine plaster cracks, opening of old cracks)
-no damage.

Under this classification system, the data indicated that major damage could occur if the vibration level of the ground exceeded a certain value of particle velocity, while minor damage could occur if a certain level of particle acceleration was exceeded.

In 1949, F. J. Crandell [4] published a paper in which he addressed his concerns in establishing safe blasting limits for contractors doing excavation work. He wanted to determine:

-how the transmitted energy varied with the amount of explosive detonated
-how the transmitted energy varied with the distance from the source
-the total amount of imposed energy, by ground vibration, that would damage a structure
-whether it was possible to predetermine a safe amount of explosive for a specific location.

He used a seismograph that he developed himself in a number of investigations, which resulted in his theory that if the Energy Ratio was kept below 3, no damage would result to buildings of sound construction. Energy Ratio was a term coined by Crandell to describe his empirical formula relating dynamite charge and distance, and will be discussed further in the next chapter. The Energy Ratio concept is still widely used in the construction industry, but it is generally limited to 1.0 to compensate for less sound structures.

The next major work in the area of blasting vibration damage criteria was that done by A. T. Edwards of Ontario Hydro, and T. D. Northwood of the National Research Council during the early 1960's [5, 6]. The goal of these researchers was to find a reasonably simple vibration measurement that would provide a dependable indication of damage risk. A secondary objective was to evaluate methods of monitoring blasting operations, in terms of determining the blasting charge for a given safe level of vibration. Their studies involved making measurements of displacement, velocity, and acceleration for increasing weights of charges, until the threshold points of minor and major damage were reached. Six structures on two different types of soil were used in the tests. Their conclusions indicated that there existed a well-defined threshold above which damage could occur. Peak particle velocity gave the best indication of this threshold, which occurred between 4 and 5 inches/second. Empirical damage threshold and particle velocity equations were also developed. Edwards and Northwood recommended that a safe vibration limit of 2 inches/second Peak Particle Velocity be established, and that a charge equation of $C^{2/3}/D=0.1$ be used for blasting operations, where C represents the explosive charge in pounds, and D is the distance to the structure in feet.

At this point, two key questions have developed in the study of damaging ground vibrations:

-What is a safe vibration velocity for construction vibrations, which ensures that damage will not occur?
-Is there any way that this velocity can be predicted, before the blasting or construction work is begun?

Most of the later research work follows one or both of these directions.

During the late 1960's, research in the area of construction vibration was beginning. In 1973, Dr.s P. B. Attewell and I. W. Farmer published a paper that has since become one of the key papers in the study of pile-driving vibrations [7]. In it, they discussed the theory of vibration wave fields generated by pile-driving, and the attenuation of these waves. They argued that for practical estimates of pile vibration amplitudes as a function of distance from the source, the influence of the geotechnical character of the ground could largely be ignored. In other words, while there is a progressive non-linear reduction in wave amplitude due to material damping, these losses are negligible compared to the geometrical losses. A geometrical loss is defined as the reduction in energy per unit area that is proportional to the increasing area covered by the expanding wave front, since the total energy of the wave front remains constant.

Attewell and Farmer also explained that the transmitted wave front close to the pile is composed principally of compressive body waves, and that the surface velocity amplitude decreases directly with distance, and inversely with the square root of the input energy at the source. This lead to the development of a Scaled Energy equation that Attewell and Farmer suggested could be used for ground surface velocity predictions.

Several researchers have built on the pioneering work done by Attewell and Farmer. The case studies reported by Heckman and Hagerty [8] in 1978 indicated that the prediction equation developed by Attewell and Farmer was conservative for all but a very few cases. However, Heckman and Hagerty did not agree with the theory presented by Attewell and Farmer suggesting that most of the energy transmitted to the soii at the pile-soil interface is in the form of body waves. Instead, Heckman and Hagerty cited the text "Vibrations of Soils and Foundations" [9] by Ritchart, Hall and Woods, which was published three years before the work of Attewell and Farmer. This text postulates that two thirds of the total input energy to an elastic half-space (the model for a ground surface) is transmitted by Rayleigh surface waves, and that "the Rayleigh wave is of primary concern for foundations on or near the surface of the earth".

Papers by D. J. Mallard and P. Barstow [10] and M. S. Langley [11] agreed with the body wave theory presented by Attewell and Farmer, while Holmberg, Lumberg and Rundqvist [12] followed the Rayleigh wave theory in their work. In 1978, T. G. Gutowski [13] suggested that Love waves were responsible for the dominant transverse vibration velocity component in his case study. Love waves are described as dispersive transverse surface waves that can occur in a layered medium. They can be likened to Rayleigh waves in that they are surface waves, but their mode of propagation is quite different.

In 1978, K. Medearis [14] published several papers detailing the need to develop more rational damage criteria for low-rise structures subjected to blasting vibrations. He argued that there was insufficient basis for a vibration standard of 2 inches/second Peak Particle Velocity, since it did not take into account the predominant frequencies of the ground motion excitation and the structure being excited. His was the first extensive research effort to correlate damage with the actual fundamental frequencies of residences. This was done through the application of Spectral Response theory. A Response Spectrum can be defined as the curve represented by the locus of the maximum response values of a single-degree-of-freedom (SDF) system when subjected to a transient ground motion forcing function, as determined for various values of the SDF system's natural frequency. The SDF system may be damped or undamped, and the maximum response values may be either displacement, relative velocity, or absolute acceleration. Medearis suggested that Pseudo Spectral Response Velocity (PSRV) was the best predictor of damage due to blasting vibrations. He also estimated that if the actual measured or calculated PSRV values were 1.5 inches/second or less, the damage probability could be estimated at no more than one percent.

C. H. Dowding [15] published a book in 1985 dealing extensively with Response Spectra calculations. His work forms the backbone of the Spectral Response theory presented in the next chapter.

Several researchers have reported the results of various construction vibration case studies in the literature. Some of these were also consulted in the preparation of this thesis [8, 10, 13, 16, 17, 18, 19].

2.2 Development of Standards and Damage Criteria

Over the past fifty years, there has been a great proliferation of recommended vibration limits published in the literature. Some are simple, while others involve complex calculations. The U. S. Bureau of Mines [3], and Edwards and Northwood [5] originally recommended safe levels of 50.8 mm/s (2 inches/second) for residential structures. Chae [20] classified buildings into four categories based on their age and condition, and recommended criteria that ranged from 12 mm/s to 100 mm/s (0.5 to 4.0 inches/second), the former for old residential structures in very poor condition, and the latter for structures of solid construction. Crockett [21] recommended criteria that take into account whether the building is ancient or modern, the type of building, specific construction details, the type of soil, the degree of distress for both ancient and modern buildings, the cumulative effect of pile-driving vibration, and an "importance factor". The importance factor ranged from a value of 1 for a light industrial building of no particular merit, to a value of 10 for a sensitive ancient building of high merit. Crockett's method is of interest because it is one of the more complex empirical techniques, and will be examined in greater detail in the next chapter.

Wiss [22], Rainer [23], and Gutowski [13] have compiled comprehensive lists highlighting the range of international vibration standards. These data have been condensed into Table 2.1. Information from other sources [24, 25] has also been included in this table.

One of the most recent major publications dealing with the vibration of buildings is ISO 4866 [26]. It was published in August 1990, and establishes the basic principles for carrying out vibration measurements, with regard to evaluating the vibration effects on buildings. The standard recommends that building-related factors such as type and condition of the building, natural frequency and damping, building base dimensions, and influence of the soil be considered when assessing possible vibration effects. Cumulative effects of vibration should also be considered where fatigue damage is a possibility. The classification system recommended is quite similar to that used by Crockett, except that no weighting factors are given; however, a relative degree of severity is suggested.

This standard also recommends that the criteria relating vibration to visible effects on buildings be approached in a probabilistic way, rather than attempting to establish an absolute lower limit of acceptable vibration. Under this method, minimal risk for a named effect would generally be considered to be a 95% probability of no effect.

Table 2.1 International Vibration Limits

Source	Code	Type of Criteria	Maximum Permissible Velocity	Comment
Bureau of Mines (1940's)		Blasting	50.8 mm/s	Peak Particle Velocity
Northwood		Blasting	50.8 mm/s	PPV
Langefors		Blasting	76.2 mm/s	PPV
Whiffen & Leonard (1971)		Traffic	33.0 mm/s 15.2 mm/s	risk no risk of damage
Jackson (1967)		Earthquake Nuclear event Blasting	0.51 mm/s	statistically significant sample may show minor damage
Bureau of Mines (1980)		Blasting	13-19 mm/s 51.0 mm/s	4-15 Hz >40 Hz
W. Germany	DIN 4150	Blasting (Historic Buildings)	4.0 mm/s	V_R
E. Germany	KDT 046/72	Blasting (Historic Buildings)	2.0 mm/s 6.0 mm/s 14.0 mm/s	<30 Hz V_z 60 Hz 100 Hz
Switz.	SN640-312	Blasting (Historic) M/C's and Traffic (Historic)	8.0 mm/s 8-12 mm/s 3.0 mm/s 3-5 mm/s	<60 Hz V_{max} 60-90 Hz <30 Hz 30-60 Hz
Czech.		Blasting (Historic)	5.0 mm/s	V_{max}
USSR		Blasting (Historic)	10.0 mm/s 30.0 mm/s	frequent V_{max} occasional
France	AFTES (Draft)	Blasting (Historic)	7.5 mm/s 2.5 mm/s	hard soil V_R soft soil >10 Hz
W. Germany	DIN 4150	Blasting (General)	8.0 mm/s 30.0 mm/s	houses, etc. well-braced structures V_R

Source	Code	Type of Criteria	Maximum Permissible Velocity	Comment
Vienna		Construct. (Cathedral)	0.02 mm/s^2	acceleration
Prague		Blasting (Subway)	10.0 mm/s	V_{max}
Montreal		Blasting (Subway)	80.0 mm/s	University Interchange V_{max}
Ontario MOE (1985)	NPC-212	Blasting (Quarry)	10.0 mm/s 12.5 mm/s	caution limit standard limit (monitor V)
U.S. EPA (1980)	Draft	General	1.0 m/s^2 0.5 m/s^2	peak acc. ancient structures

Notes:

V_{max} = maximum velocity component

V_z = vertical velocity component

$V_R = \mathrm{Sqrt}\,(V_x^2 + V_y^2 + V_z^2)$

III. THEORETICAL ANALYSIS

The contents of this chapter have been divided into two main sections. The theoretical development of ground vibrations and their effect on buildings will be addressed in the first section of this chapter. Wave theory and Spectral Response theory will be discussed, along with the derivation of key equations.

In the second section of this chapter, various empirical and theoretical methods for the prediction of peak vibration velocity, or the determination of safe vibration limits, will be examined. These methods will include the equations developed by Attewell and Farmer, Wiss, and others for the prediction of peak particle velocity. The method recommended by Crockett for the prediction of a "safe" vibration velocity will also be considered.

3.1 Theory

Construction vibrations can be grouped into three different categories:

-transient or impact vibration,
-steady-state or continuous vibration,
-pseudo steady-state vibrations.

Transient construction vibrations can be described as those that occur from blasting with explosives, impact pile-driving, and wrecking balls. Steady-state vibrations may be generated by vibratory pile-drivers, vibratory compactors, and compressors. Pseudo steady-state vibrations are termed such because they are of a random nature, or they are a series of impact vibrations that are at short enough intervals to approach a steady-state condition. Examples of these are jackhammers, pavement breakers, and bulldozers. This thesis will deal primarily with transient vibrations.

3.1.1 Wave Theory

There are three main types of ground waves: compressive waves (also known as Primary or P-waves), shear waves (also called Secondary or S-waves), and surface waves. The Rayleigh wave is the most important type of surface wave.

Compressive and shear waves are both classified as body waves, which means that they propagate in a spherical direction. Compressive waves can be characterized by a series of compressions and rarefactions in the ground, while shear waves are characterized by a vertical oscillation of the ground surface about its undisturbed position. Rayleigh surface waves propagate with a cylindrical front, where particles on the surface move in an elliptical pattern. They vibrate the soil to a certain depth below the ground surface, which is a function of the frequency of vibration. The differences between these wave types are illustrated in Figure 3.1 [39]. All vibration waves can be described by their frequency, propagation velocity, and wavelength.

In pile-driving, the pulse generated by the impact of the pile driving hammer at the pile cap sets up a body wave that travels along the pile at the sonic velocity of the steel (approximately 5000 m/s, or 3650 m/s for a concrete pile) to the pile/soil interface at the pile base. At the vertical pile/soil interface, virtually all of the wave energy is reflected at an incident angle of 90°, and only a negligible amount is refracted. The

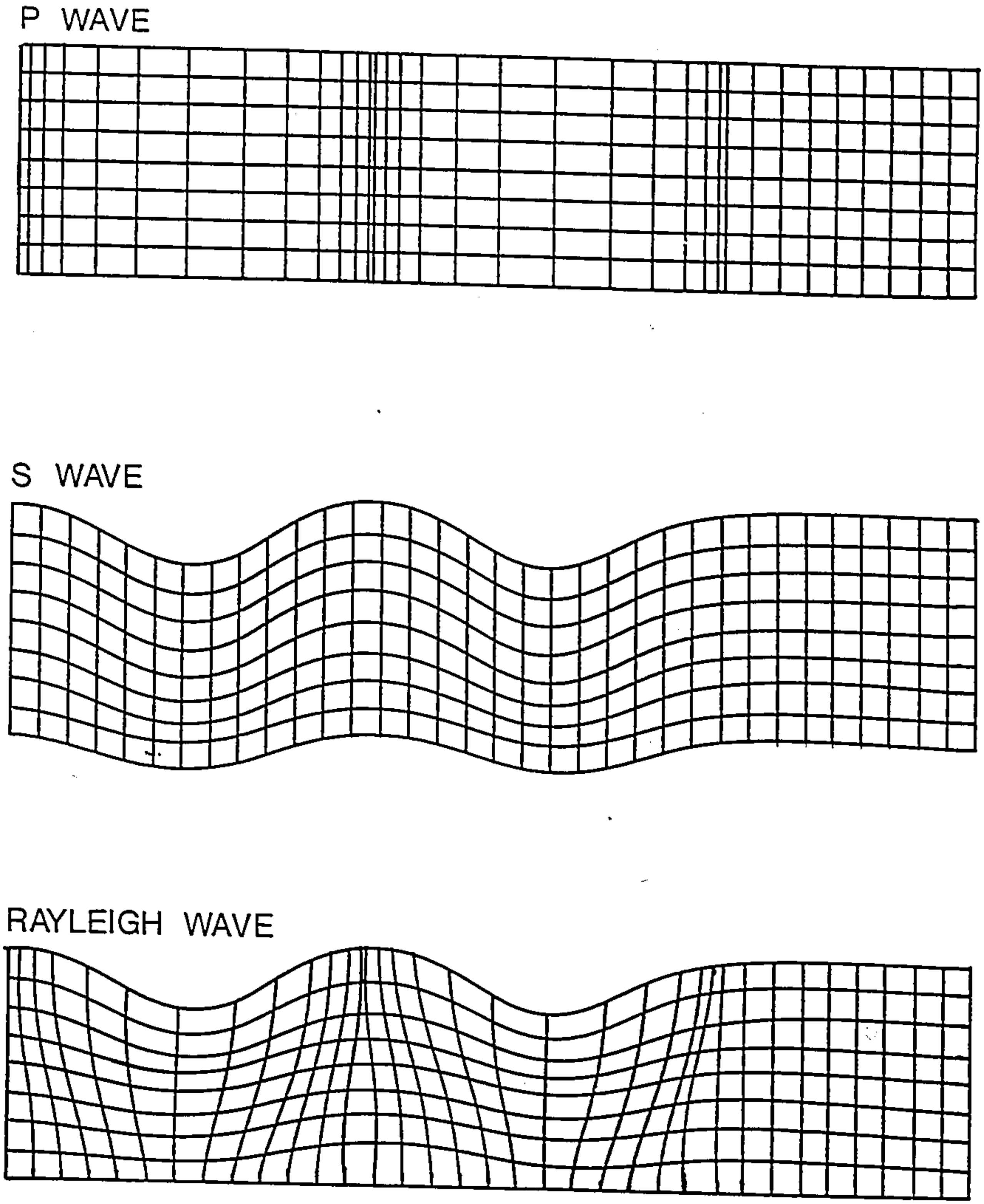

Figure 3.1: Illustration of Three Wave Types

major part of the wave energy is transmitted through the soil as a line source of body waves. Refraction and reflection of the P and S components of these waves at the surface will produce P- and S-wave transformations near the pile, resulting in ground movement. At a critical distance from the pile (approximately equal to the pile depth), S-wave arrivals at the surface can also create a headwave, as shown in Figure 3.2. True surface Rayleigh waves develop at a greater distance, depending on the source depth and wavelength.

Propagation velocities of different waves can be calculated from the theory of elasticity [27]. That is, the propagation velocity of a shear wave is one-half to two-thirds that of the compressive wave, while the propagation velocity of a Rayleigh wave is 90 to 95 percent that of the shear wave. But since soil reduces the propagation velocity for all wave types, Rayleigh waves are attenuated the most, since they travel only at the surface. Elastic theory, through the equations of motion for a spherically propagating wave from a point source in an infinite body, predicts that the peak particle velocities of body waves will decay at a rate proportional to $(1/R)^n$. In this equation n=2 near the disturbance, and n=1 at greater distances [15].

Transient pulses, such as those generated by blasting or pile-driving, normally last from 1 to 2 ms at locations close to the impact source. At larger distances the sinusoidal wavetrain can have a duration of 10 to 100 ms, due to a combination of direct transmission, reflection, and refraction of the input signal. For a sinusoidal approximation of a vibration wave, the wavelength can be determined by:

$$\lambda = cT \tag{3.1}$$

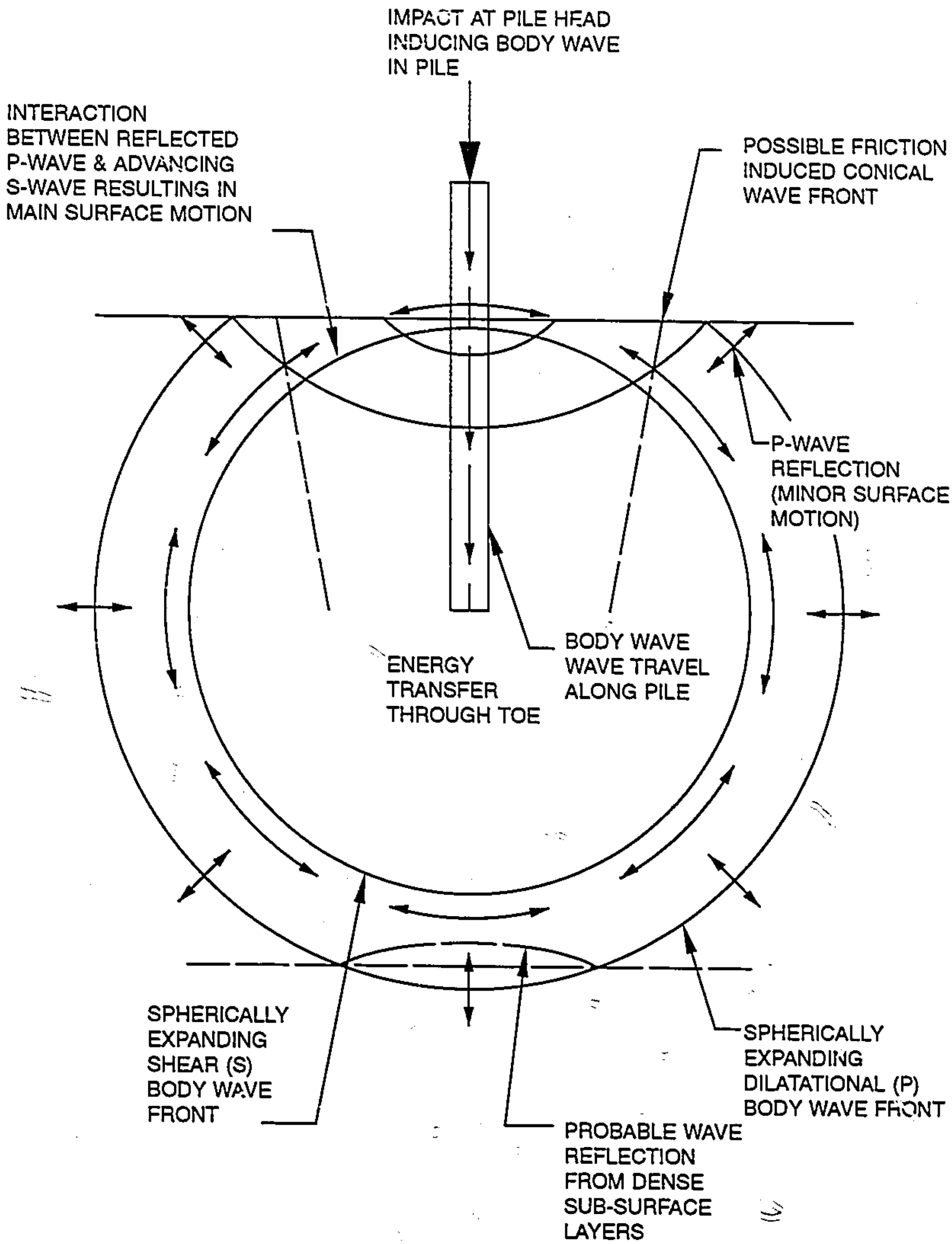

Figure 3.2: Wave Energy Transmitted Through the Soil

where c is the propagation velocity of the wave, and T is the period of vibration. Estimated propagation velocities for sand and clay are shown in Table 3.1. It should be noted that jointing greatly reduces the propagation velocity, therefore the values for unjointed material are rarely used.

Table 3.1 Estimated Propagation Velocities

Soil Type	Wave Velocity (m/s) Heavily Jointed	Non-jointed
Sand -Compression	500	2000
-Shear	250	850
-Rayleigh	244	---
Clay -Compression	400	1700
-Shear	200	800
-Rayleigh	183	---

3.1.2 Spectral Response Theory

It is now well-known that consideration of the frequency of vibration, type of structure, and building materials must be made in assessing the amplitude of vibration that a building will be able to sustain without damage. That is, structures will respond differently when excited by vibrations equal in all respects, but differing in principal frequency. Recognition of the importance of frequency has lead to the development of vibration monitoring approaches that include this parameter. The most simple approach is based on the single-degree-of-freedom model for above-ground structures, and the ratio of excitation wavelength to structure size for below-ground, or buried, structures. The single-degree-of-freedom model is generally employed through the response spectrum.

A plot of the maximum response of different single-degree-of-freedom systems to the same vibration is called a Response Spectrum. It shows the range, or spectrum of response of differing systems to the same excitation motion. The basic premise of Response Spectra calculations is that the response of any structure to vibration can be calculated if its natural frequency and damping are known or can be estimated.

For one, two and three storey structures, modelling as a single-degree-of-freedom system is valid when movement in only one direction is considered. The characteristics of a structure that govern its behaviour under vibratory loading are:

-masses of main components (floor mass)
-stiffness of main components (walls)
-amount of energy dissipated through damping (differential movement of joints and connections).

Here the differential movement is defined as the difference between the absolute displacement of the mass and that of the ground.

If a structure's undamped natural frequency (p) and its fraction of critical damping (zeta) are known, it is not necessary to know the mass (m), stiffness (k), and damping (c_1) to be able to model the structure accurately. The equation of motion for a single-degree-of-freedom system, when subjected to ground excitation, is:

$$m\ddot{y} + c_1\dot{\delta} + k\delta = 0 \qquad (3.2)$$

where $\ddot{y}$ is the absolute acceleration of the mass, and δ is the relative displacement between the ground and the mass (δ = y-u). By making the following substitutions:

$$p = \sqrt{k/m} \qquad \zeta = \frac{c_1}{2\sqrt{m/k}} \tag{3.3}$$

Equation 3.2 can be rewritten as:

$$\ddot{\delta} + 2\zeta p\dot{\delta} + p^2\delta = \ddot{u} \tag{3.4}$$

where zeta represents the damping ratio, p is the circular natural frequency, and $\ddot{u}(t)$ is the ground acceleration time history, which is integrated from time zero to time t. Zeta and p can be measured from a free vibration time history of the building response.

The damping ratio, zeta, can be found from the decay of free oscillations of the building using:

$$\zeta = \frac{1}{2\pi}\left(-\ln\frac{\dot{u}_{n+1}}{\dot{u}_n}\right) \tag{3.5}$$

where $\dot{u}_n$ and $\dot{u}_{n+1}$ are successive vibration amplitudes [28]. The undamped natural frequency, p, can be found from the damped natural period of the first mode of vibration, T, using:

$$T = \frac{2\pi}{p_d} = \frac{2\pi}{p\sqrt{1-\zeta^2}} \tag{3.6}$$

If the vibration response has not been measured, the natural frequency of the building superstructure can be estimated from:

$$p = 2\pi\sqrt{\frac{L}{0.5h}} \tag{3.7}$$

where L represents the width of the structure and h is the height of the structure [29].

Equation 3.7 can be simplified even further to:

$$p = \frac{2\pi}{0.1N} \tag{3.8}$$

where N is the number of stories. Damping is not as simple to predict, since it is a function of the building construction, and to a lesser extent the intensity of vibration. Detailed studies show that damping ranges between 2% and 10% of critical, and averages 5% for residential structures [14, 30]. The natural frequencies of superstructures range from 5 to 10 Hertz, and those of walls range from 12 to 20 Hz. Floors also have their own fundamental frequency of vibration, which tends to be lower than that of walls, especially if the floor spans are large.

The solution to Equation 3.4 for relative displacements at any time can be expressed in terms of the Duhamel's integral of the absolute ground acceleration time history as [31]:

$$\delta(t) = -\frac{1}{p\sqrt{1-\zeta^2}} \int_0^t \ddot{u}(\tau)\, e^{-\zeta p(t-\tau)} \sin\left[p_d(t-\tau)\right] d\tau \tag{3.9}$$

If a velocity time history is used as the input time history, the relationship between $\dot{u}$ and δ can be found by integrating Equation 3.9 by parts and combining terms to get [31]:

$$\delta(t) = \int_0^t \dot{u}(\tau)\, e^{-\zeta p(t-\tau)} \left[\cos p_d(t-\tau) - \frac{\zeta}{1-\zeta^2}\left[\sin p_d(t-\tau)\right]\right] d\tau \tag{3.10}$$

where the relative displacement and relative velocity are zero at t(0).

The resulting calculated time history will be one of relative displacements, rather than the measured absolute velocity. In the calculated relative displacement time history, there will be a maximum, δ_{max}, corresponding to the maximum velocity input, $\dot{u}_{max}$. If δ_{max} is multiplied by the structure's circular natural frequency p, the result will be:

$$PV = 2\pi f \cdot \delta_{max} = p \cdot \delta_{max} \tag{3.11}$$

where PV is called the Pseudovelocity [15], or Pseudo-spectral Response Velocity [14]. This pseudovelocity is a close approximation of the relative velocity, if the pulse associated with δ_{max} is approximately sinusoidal.

A pseudovelocity response spectrum of a single ground motion can be created by processing Equation 3.10, while holding zeta constant (for the subject structure) and varying the frequency f in the equation. The resulting pseudovelocities will form a solid line when plotted on four-axis tripartite paper, as shown in Figure 3.3.

The four-axis paper takes advantage of the sinusoidal approximation involved in calculating a pseudovelocity. The axis of maximum relative displacement is inclined to the left and is PV/(2πf). The Pseudoacceleration axis is inclined to the right, and is PV·(2πf). For small damping ratios, the pseudovelocity and pseudoacceleration, which are actually sinusoidal approximations, do closely approximate the absolute acceleration of the mass and the relative velocity of the system [31].

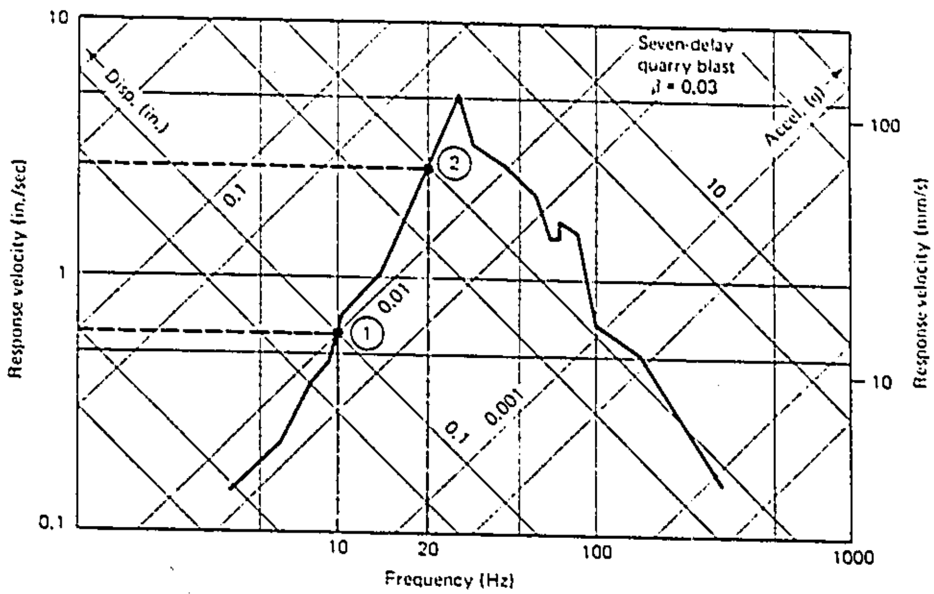

Figure 3.3: Example Response Spectrum - Blasting Vibrations

3.2 Theoretical Analysis

This section will discuss the various empirical and theoretical techniques that have been developed for the analysis of construction vibrations. Several empirical equations for the prediction of peak particle velocity, Crockett's empirical method for the prediction of a "safe" vibration level, and Spectral Response theory will be examined in detail. Example calculations will also be made using these methods. The results of these calculations will be compared to actual field measurement data in Chapter V.

3.2.1 Empirical Formulae

As mentioned previously, many researchers have developed empirical equations designed to predict the peak particle velocity of vibration, or a "safe" vibration velocity. Many of these equations are based on a curve-fit of the researcher's own field measurements, where vibration velocity was recorded simultaneously at several distances from the vibration source. Other investigators have used the data of previous researchers to supplement their own case work, and to verify the validity of their equations. Table 3.2 gives a synopsis of the major prediction equations to date.

Inspection of the equations in Table 3.2 reveals that many of them possess certain similarities. Most of the peak particle velocity prediction equations rely on some form of Square Root Scaling, where the predicted velocity is a function of the distance R divided by the square root of the input energy W. The equations are also similar in their omission of any reference to the frequency of vibration.

Two of these equations can be used to estimate the tensile stress in a structural member, if the velocity of vibration and Young's Modulus for the material are known. Equation 3.20 uses the propagation velocity of the structure's material, while Equation 3.18 relies on the mass density of the material and a dimensionless quantity K that is related to the sectional shape of the structural member.

Table 3.2 Empirical Formulae for Velocity Prediction

Source	Formula	Application	Comment	Eqn
Crandell (1949)	$(50^2/R)W^2K = ER$	blasting	ER= energy ratio= a^2/f^2	3.12
Edwards & Northwood (1958)	$V = KW^{2/3}/R$	blasting	K= ground constant	3.13
Attewell & Farmer (1973)	$V = 1.5(Wo)^{1/2}/R$	pile-driving	conservative equation	3.14
Wiss (1974)	$V = K(R/W^n)^b$	blasting	K= intercept where $R/W^n=1$	3.15
Crockett (1979)	$V = (10^4/L)^{0.25}$ $((Bl+0.6We)/IF)$	safe limit for piling	see Section 3.2.2	3.16
Mallard & Barstow (1979)	$V = 32.36(W^{1/2}/R)^{1.6}$	pile-driving	W is in kJ	3.17
Steffans (1979)	$\sigma_{max} = V_{max}K(E\cdot\rho)^{1/2}$	pile-driving	dyn. stress in struc. member	3.18
Wiss (1981)	$V = K(R/W^{1/2})^{-n}$	construction	R is ft W is ft-lb	3.19
Holmberg (1983)	$\sigma = V_{max}\cdot E/C$	blasting	tensile stress	3.20
Dowding (1985)	$V = 18.3(30.5/R)^{1.46}$ $(W/4.54)^{0.48}$ $(2.4/\rho)^{0.48}$	blasting	W is kg ρ is g/cm^3	3.21
MOE (1985)	Square Root Scaled Distance = $R/W^{1/2}$	blasting	use Appendix I to find V	3.22

To illustrate the range of velocities that these equations will predict, an example calculation has been done for each equation listed. The results are provided in Table 3.3. For these example calculations, a pile-driver input energy of 54200 Joules (40,000 ft·lbs), and a source to transducer location distance of 24.7 metres was used. These values are representative of the worst-case conditions encountered during the pile-driving operations in the vicinity of the Baby House. The measurement conditions will be described in greater detail in Chapter IV.

Table 3.3 Example Velocity Calculations

Eqn	Source	Predicted Velocity
3.13	Edwards & Northwood	K unknown
3.14	Attewell & Farmer	PPV = 14.13 mm/s
3.16	Crockett	Safe V = 1.08 mm/s
3.17	Mallard & Barstow	PPV = 4.61 mm/s
3.19	Wiss	PPV = 7.62 mm/s (0.3 ips) ref. [22] Fig. 5
3.21	Dowding	PPV = 1511 mm/s if ρ= 5.52 g/cm^3 [32]
3.22	MOE	PPV = 190 mm/s

Equations 3.21 and 3.22, which were developed for blasting vibrations, are obviously not well-suited for pile-driving predictions, as their erroneous results indicate.

3.2.2 Crockett's Method

The method developed by J. H. Crockett, a consulting engineer in Surrey, England, is of interest because of its complexity, and because it proposes to calculate a "safe" vibration velocity limit for a given structure, below which no damage will occur. The basic premise of this method is that all buildings disintegrate, but that vibration will hasten this disintegration by many times.

A key aspect of Crockett's method is the perceived importance of the building. In other words, a small but tolerable vibration damage in one group of buildings may be seen as unacceptable in another, therefore allowable vibration limits should be set accordingly. Crockett also postulates that vibration damage to a building is cumulative over its lifetime. This means that structural fatigue effects must also be considered.

Crockett's Equation is stated as:

$$V = \sqrt[4]{\frac{10^4}{L} \times \frac{(BASIC\ INTENSITY) + 0.6\,(WEIGHTING)}{IMPORTANCE\ FACTOR}} \qquad (3.16)$$

In this equation, the Basic Intensity is read from a Basic Intensity Pattern graph, where the predominant frequency of vibration (from field measurement) is matched with the basic building type to get a Particle Velocity start point. This value is then modified by the addition and subtraction of various weighting factors that are provided in eight tables relating to building construction details, such as the number of stories, type of construction material, presence of arches and condition of plaster. Soil compaction weighting suggestions are also provided. The total is then further modified by the Importance Factor, where buildings are rated on a scale from 1 to 10, depending on their architectural merit and historical importance. The tables and figures used for this method have been included as Appendix II.

Using the Hiram Walker Historical Museum (Baby House) as an example, the following assessment would be made:

Building Classification: -A- Ancient or Elderly House

Predominant Frequency of Vibration: 20 Hz (based on field measurements, see Chapter IV)

Approximate Number of Cycles: 90,000 (approximately 20 piles, driven 45 metres deep. Crockett's standard is 10 piles driven 10 metres deep = 10,000 cycles.)

Weighting Factors:

Table A:	a.	+1
	b.	+1
	25.	-2
Table J:	2.	0

Importance Factor:
Table K: 8

Thus:

$$V = \sqrt[4]{\frac{10^4}{90000}} \times \frac{15+0.6\,(1+1-2+0)}{8}$$

This equation results in an acceptable vibration velocity of 1.08 mm/s. This indicates that Crockett's method is much more conservative than the limits proposed by the Swiss Association of Standardization for historical buildings, which recommends a peak particle velocity limit of 3 mm/s for this situation.

Crockett's Method has two main weaknesses. The first weakness is that the technique is open to the interpretation of the user, especially in the use of the Weighting tables. Two different people performing an assessment of the same building may arrive at widely divergent acceptable vibration limits. The second weakness of this method is that no data have been provided, or suggested, to verify that the weighting values do indeed have meaning, and that the equation predicts realistic acceptable vibration values. Although Crockett states that this method is the culmination of his years of experience as a consulting engineer, the lack of substantiating data gives the impression that the method may solely be the product of his imagination.

3.2.3 Spectral Response Method

In Section 3.1.2, Equation 3.10:

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

was introduced as the solution to the second order differential equation of motion governing single-degree-of-freedom systems subjected to ground motion. The solution of this integral was the first step in calculating a Response Spectrum for a single ground motion. To briefly summarize from Section 3.1.2, the pseudovelocity response spectrum of a single ground motion can be created by processing Equation 3.10 while holding zeta constant and varying the frequency f in the equation (recall that $p = 2\pi f$). Maximum displacements δ_{max}, resulting from processing the equation with the maximum velocity $\dot{u}_{max}$, can be multiplied by the frequency to obtain the Pseudovelocity. Pseudovelocities are than plotted on tripartite paper to form a solid spectral response line.

The calculation of a Response Spectrum was undertaken using data relevant to the Baby House. This method had not been used previously for pile-driving vibrations, but has been employed successfully to predict building response to blasting vibrations. A rigorous computer solution of the integral using a tape recorded vibration time history, as mentioned by Dowding [15], was not attempted as it was clearly beyond the scope of this thesis, and would be suitable for study as a thesis topic on its own. Likewise, the approximation method presented by Dowding was not used, since examination of this method indicated that it was suitable only for the calculation of Response Spectra to blasting vibrations, and could not be applied to the case of pile-driving vibrations. The reference material used by Dowding in his approximation method uses blasting terms, such as density of rock and charge weight per delay, that cannot be applied to construction use with any accuracy. However, there is a good correlation between his approximations and the spectral response calculated from ground motions for the case of blasting vibrations.

Instead, a simple numerical quadrature method, Simpson's 1/3 Rule, was applied to approximate a solution to the above integral. Simpson's 1/3 Rule can be stated as [38]:

$$\int_a^b f(x)\,dx - \frac{h}{3}\left[f(x_0) + 4f(x_1) + f(x_2)\right] \qquad (3.23)$$

where: x_1 - the peak occurring particle velocity from free vibration measurements
x_0, x_2 - vibration velocities immediately before and after the occurrence of the peak
f(x) - equation 3.10.

Referring again to Section 3.1.2, the maximum velocity $\dot{u}_{max}$ will yield a maximum displacement solution, δ_{max}. This is the value required for calculation of the pseudovelocity for a given frequency. The damped natural frequency p_d and fraction of critical damping zeta, required for solution of Equation 3.10, were determined from the free vibration history of the Baby House response, and are described in greater detail in Section 4.3.2.

Calculations were performed using the spreadsheet package Quattro Pro, for the frequency range of 0.1 to 200 Hz. This is generally considered the range of concern for ground vibrations. The results of the calculations have been included as Appendix III.

Figure 3.4 shows the Response Spectrum predicted for the Baby House. It indicates that peak vibration response will occur at 20 Hz, with some elevated response between 8 and 100 Hz. A secondary peak occurs at 4 Hz. The maximum expected vibration velocity is indicated as approximately 0.35 inches/second, or 8.9 mm/s. Chapter V will include a comparison of the predicted Response Spectrum with the actual vibration response recorded during pile-driving at the Baby House.

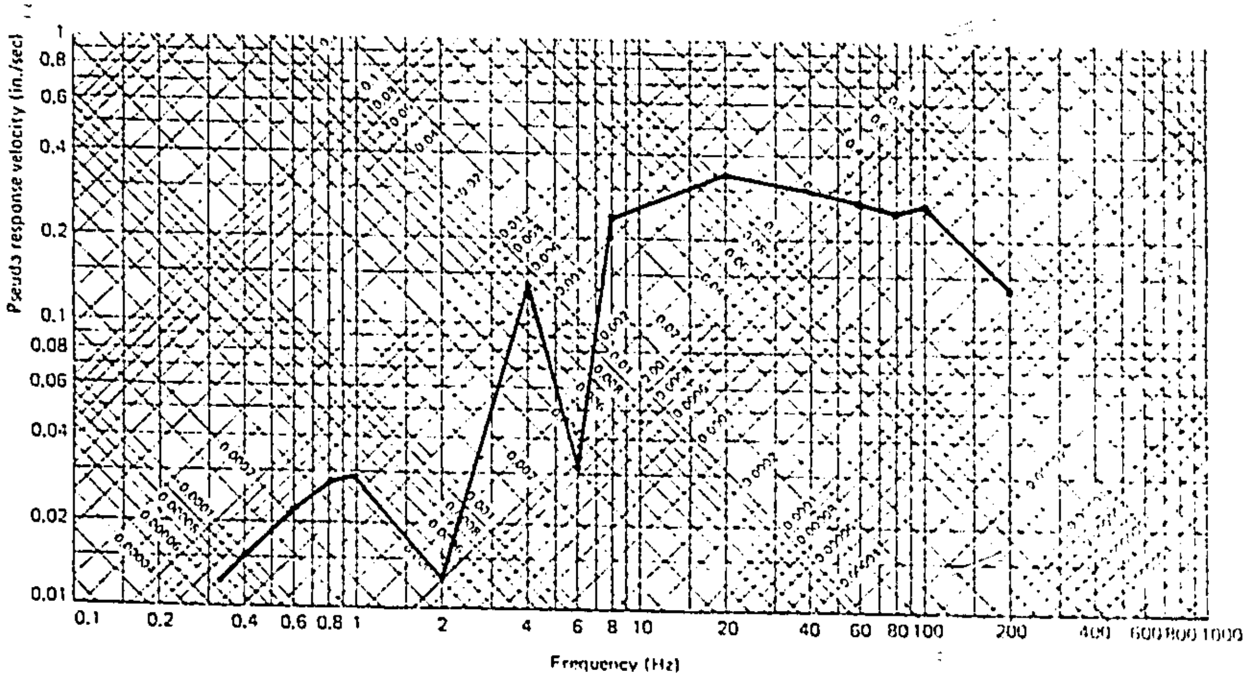

Figure 3.4: Baby House - Predicted Response Spectrum

IV. EXPERIMENTAL ANALYSIS

During the summer and early fall of 1990, vibration measurements were made at three locations in Windsor: The Cleary Auditorium on Riverside Drive West, the Hiram Walker Historical Museum (Baby House), located directly behind the Cleary Auditorium, and at two houses in the Bellewood Estates subdivision, on Huron Church Road. At the Cleary Auditorium and the Baby House, peak particle velocity measurements were made of vibrations induced by diesel pile-driving, while at the Bellewood Estates, the vibration created by a vibratory compactor was monitored.

4.1 Measurement Techniques

At all three locations, measurements were made by attaching one or two accelerometers to a point low on the main load-bearing external wall, on the interior of the building in question, as recommended by ISO 4866 [25]. Most measurements were made using one accelerometer mounted in the vertical direction only, since it was found that the vibration in the radial direction was generally 20 dB lower in magnitude than the vibration in the vertical direction. No simultaneous measurements were made in the transverse direction, as only two accelerometers were available, and transverse vibrations have historically been shown to be much smaller than vibrations in the vertical direction [37]. Since the objective of the measurements was to determine if the vibrations were large enough to cause damage to the building, extensive measurements in the radial or transverse direction were considered unnecessary.

Brüel & Kjær Type 4370 piezoelectric accelerometers were used as the vibration transducers for all measurements. The Type 4370 accelerometers have a lower frequency limit of 0.1 Hz, and a charge sensitivity of 10 pc/ms^{-2} [33]. The accelerometers were screwed into an aluminum mounting block, which was then glued to the foundation wall in the proper orientation using a hot glue gun. Hot glue was chosen as an appropriate mounting method because the mounts were intended to be temporary, and minimal damage to the mounting wall was desired. Since only the frequency range from 1 to 200 Hz is considered important for the study of ground vibrations, any small reduction in the 16 kHz resonance frequency of the accelerometer, caused by the mounting method, would not be significant [33].

The accelerometer was then attached to a Brüel & Kjær Type 2635 Charge Preamplifier. The transducer sensitivity of the preamplifier was set to 10 mV per unit output, and it was adjusted to integrate the acceleration signal to velocity (m/s), with a velocity low frequency limit of 1 Hz and an upper frequency limit of 1 kHz.

Finally, the charge preamplifier was connected to a Brüel & Kjær Type 2133 or 2143 Real-Time Frequency Analyzer via low-noise coaxial cable. The Frequency Analyzer provided a "real-time" graphical display of the vibration input by filtering the signal into 1/3-octave bandwidths. A 1/4 or 1/8 second exponential averaging time was used at the Cleary Auditorium and Baby House, since the vibration being measured was of a transient nature. At the Bellewood Estates, a one second averaging time, and the

Maximum Hold feature were used to record the continuous vibration. The data were stored as 15 x 30 arrays on 3 1/2 inch diskettes. Each array element would contain one complete frequency spectrum of the vibration being recorded.

At the Baby House, some vibrations were also recorded using a Brüel & Kjær Type 7005 Tape Recorder. Vibration was recorded simultaneously in the vertical and radial directions on Channels A and B of the recorder, with the same accelerometer and charge preamplifier arrangement as that used for the Frequency Analyzer.

In each case, a Brüel & Kjær Type 4294 Calibrator was used to provide a calibration signal at the start and end of each day's measurements. Figure 4.1 shows a schematic of the equipment set up.

4.2 Analysis Techniques

The tape recording made at the Baby House was analyzed using a Spectral Dynamics SD375 Dynamic Analyzer II. A Fast Fourier Transform, using the Hanning Window, was taken of each taped channel to determine the fundamental modes of vibration of the Baby House in the vertical and horizontal directions. The time-based signal was also analyzed to find the undamped natural frequency and damping of the structure using equations 3.5 and 3.6.

A software package called WT9309, developed by Brüel & Kjær, was used to examine the data obtained by the Real-Time Frequency Analyzer. Using the software, vibrational data stored on the 3 1/2" diskettes could be read into a personal computer, and one of two operations could be performed. Either a waterfall graphical display could be

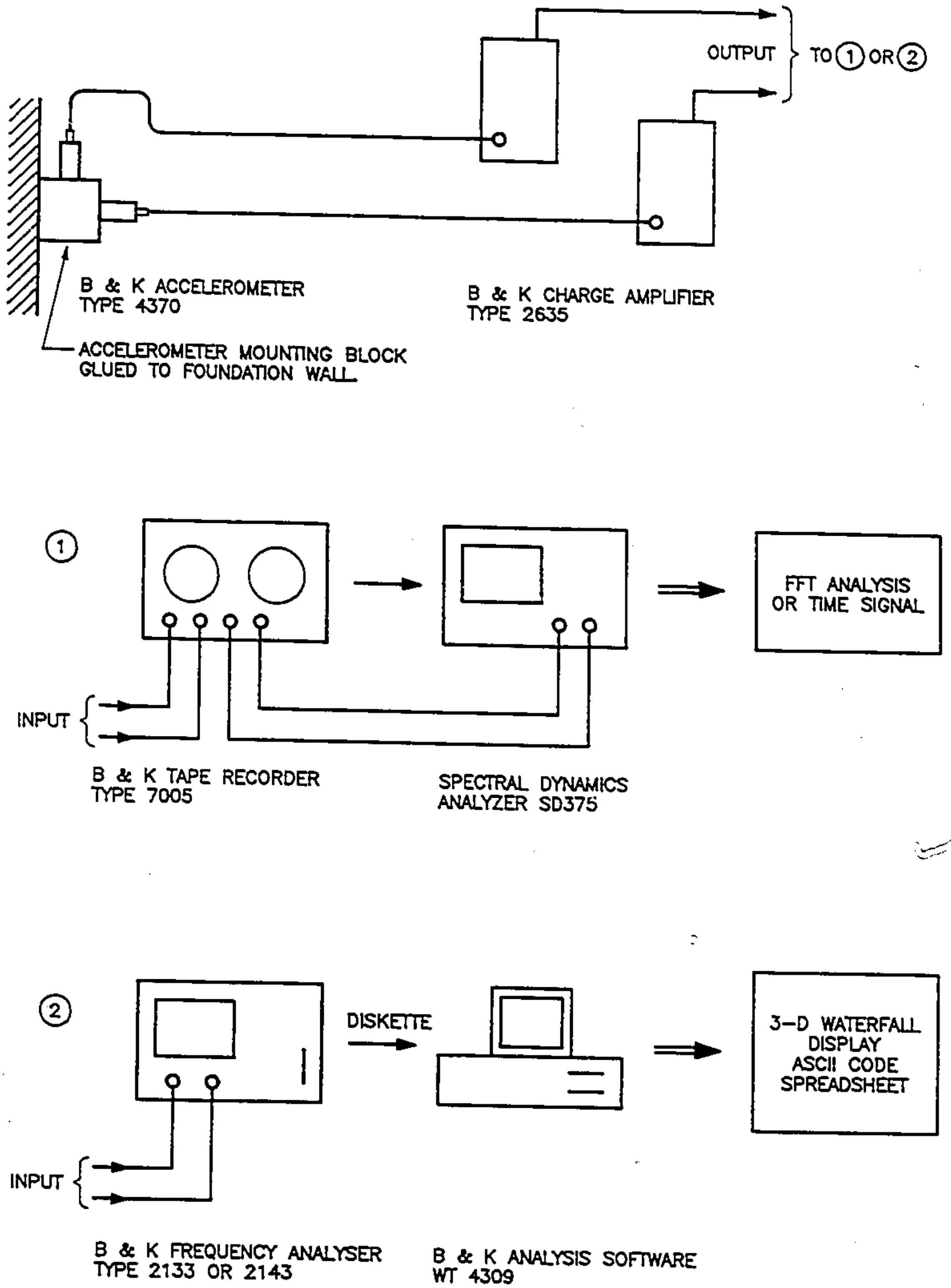

Figure 4.1: Schematic of Equipment Set Up

plotted of each data array (15 x 30 array size = 450 spectra, the maximum size of the graph), or the data could be converted into an ASCII file, suitable for reading into a spreadsheet software package.

It was found that the data convert program was awkward to use, since the resulting mass of numbers could not be easily related their meaning. The volume of data generated also proved to be too overwhelming for significant analysis when this method was used, although it was necessary to use this method to analyze several of the Baby House data files that could not be read by the graphical display program.

The graphical analysis technique was a more elegant method, since the frequencies of interest (those showing a higher amplitude vibration response) could be recognized immediately. Trends in the data were also easier to discover using this visual method of analysis. A typical waterfall display for the Baby House is shown as Figure 4.2. This graph shows an elevated response in the 6.3 to 20 Hz 1/3 octave bands, indicating that this low frequency range is the area of concern for ground vibrations affecting this building. This range contains the resonant frequencies of the building components, such as the walls and floors. This waterfall display also shows that there is no significant change in the vibration amplitude with increased depth of the pile, for this range of pile depths. Here increasing pile depth would be viewed as an increase in the incremental time, or positive z-axis direction. Finally, different waterfall graphs representing different depths of pile, or different pile locations could be compared to find the rate of vibration attenuation with depth, or with distance from the receiver.

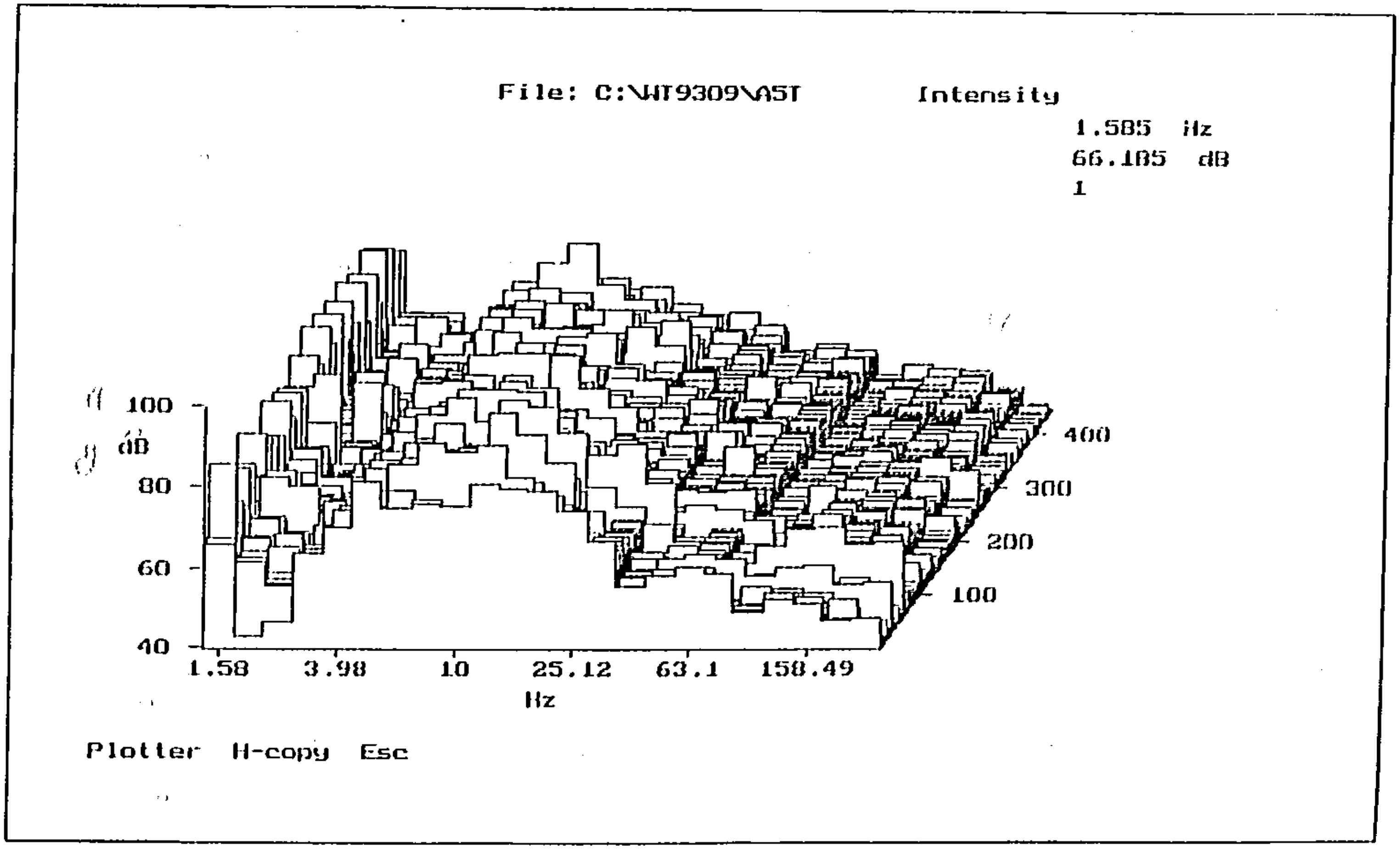

Figure 4.2: Typical Waterfall Display - Baby House June 22, 1990

An interesting phenomenon that became apparent while the actual vibration measurements were being made was that the highest amplitude of vibration generally occurred after the pile had been allowed to sit undisturbed for a period of time. This occurred when pile-driving was stopped in order to weld an additional section of pile onto the part that had already been driven. This procedure was necessary at the Baby House and Cleary Auditorium locations because the piles had to be driven to a depth of thirty-eight metres to reach bedrock (refusal), therefore three pile sections had to be welded together to create a pile of suitable length. This increase in vibration amplitude after a "rest" was also noted by Mallard and Barstow [10]; however they did not suggest any explanation for this occurrence.

4.3 Field Measurement Results

Each measurement location was unique in the types of measurements made, and the analyses that were possible. The most detailed analysis was possible at the Baby House, since vibration measurements were recorded using two types of equipment, and more information about the soil and building construction was available at this site. This section will discuss the different measurements made, and examine the results obtained at each location.

4.3.1 Cleary Auditorium

In May 1990, work began to construct a large addition on the north end of the Cleary Auditorium and Convention Centre. The Windsor Historical Sites Association had expressed concern that the construction work, and in particular the pile-driving, would have a detrimental effect on the Hiram Walker Historical Museum, which was situated immediately south of the Cleary Auditorium. In order to determine the magnitude of the vibration problem that would be created by the pile-driving, measurements of the first five piles driven were made at the Cleary's north basement wall. This was done primarily as a field test to give some indication as to whether the pile-driving on the south side of the Cleary (closest to the Baby House) might cause vibrations that would be large enough to damage that building.

The H-piles that were used at this site were manufactured from G40.21M Grade 300W steel in 15.2 metre lengths, which were to be welded together as the piling progressed. The piles were pre-augured to a depth of 15.2 metres, and were driven to refusal (bedrock) at 37.8 metres [34]. A Delmag Model 16-32 Diesel Pile-Driver with a rated energy of 54,000 Joules (40,000 ft.lbs) was used [35]. The piles were driven into cohesive silty clay, interspersed with non-cohesive silt and sand lenses up to eight inches thick. At a depth of 24.4 metres to bedrock, a "very hard" silty clay till with cobble and some boulders was present [36].

Analysis of the data diskettes recorded using the B & K Real-Time Frequency Analyzer revealed that the largest single amplitude response occurred when the third section of the first pile (closest to the vibration transducer) was being driven, at a pile tip depth of approximately 35 metres. The recorded value was 91.3 decibels in the 1/3 octave band centred about 25.1 Hz. Using 10^{-9} as the reference value for conversion from decibels to Peak Particle Velocity, and the equation:

$$dB = 20 \log \left(\frac{x}{ref}\right) \tag{4.1}$$

the Peak Particle Velocity obtained was 36.7 E-06 m/s, or 0.036 mm/s. This value is much smaller than even the most conservative criteria for vibration-induced building damage, as discussed in Chapter 3. Therefore, it would be impossible for the diesel pile-driving on the north side of the Cleary Auditorium to cause even cosmetic damage to the Cleary building.

Further examination of the vibration data from the Cleary Auditorium revealed other interesting occurrences. If the data were divided into three categories, corresponding to the driving of the first, second, and final section of pile, it can be seen that the frequencies where the increased vibration amplitudes occur do not remain the same during all three stages, but increase with depth of the pile toe. Figure 4.3 shows that during piling of the first 15.2 metres, the 1/3 octave bands centring on 5.01 Hz, 6.3 Hz, and 12.6 Hz showed the highest amplitude response. It is also worth noting that elevated 5.01 Hz and 12.6 Hz frequencies appeared together, as did elevated 6.31, 7.94, and 10 Hz frequencies. Since the elevated frequencies are generally indicative of the fundamental frequencies of the building superstructure or other building part, it bears speculation that the frequency pairs may be harmonics of each other, or simply the fundamental frequencies of joined building parts.

During the driving of the second stage of this pile (15.2 to 30.4 metres), the predominant frequencies were in the 1/3 octave bands centred about 6.31, 5.01 and 10 Hz,

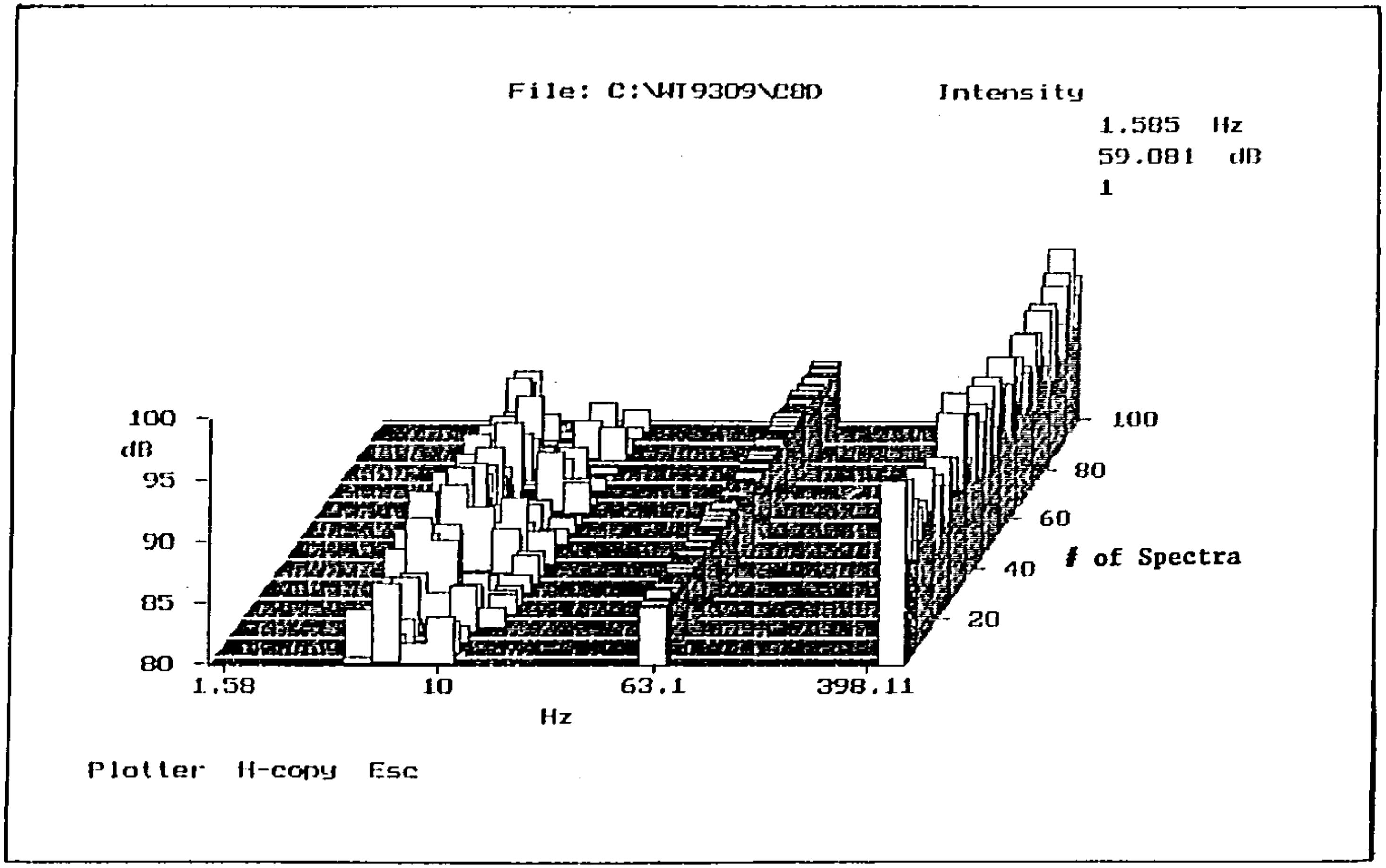

Figure 4.3: Waterfall Display of Vertical Vibration - Cleary Auditorium, Pile 1, Sect. 1

in order of decreasing vibration amplitude. However, data for the final piling stage shows that the highest amplitude response is now in the 1/3 octave band centred about 25.1 Hz, with 12.6 Hz and 10 Hz having the second and third highest vibration response amplitudes, as shown in Figure 4.4. The reason for this frequency shift is not clear, but it could be postulated that the shift may be due to the pile toe entering the "very hard" silty clay till at this depth. Or perhaps some other portion of the building, such as the basement floor instead of the superstructure, is exhibiting the largest vibration response when ground vibrations originate at this depth.

Based on the above observations, it may be concluded that the fundamental frequency of the Cleary Auditorium's superstructure is in the 5.0 to 6.5 Hz range, and that the other frequencies of interest are either harmonics of the superstructure, or are the fundamental modes of the building's north wall, or that of the basement floor. The vibrations were not of sufficient magnitude to cause any type of damage to the building.

4.3.2 Baby House

The Hiram Walker Historical Museum (Baby House) is located immediately south of the Cleary Auditorium, on Pitt Street West. It was built by Francois Baby in 1812, and served as the American Headquarters at the opening of the War of 1812. It has since been designated as a historical site under the Ontario Heritage Act.

A noteworthy construction feature of the Baby House is that it was built without a wooden or steel frame. In other words, only the brick and mortar of the walls hold the building together. This feature could make the building more susceptible to damage from vibration, since differential movement of the joints and connections in a building

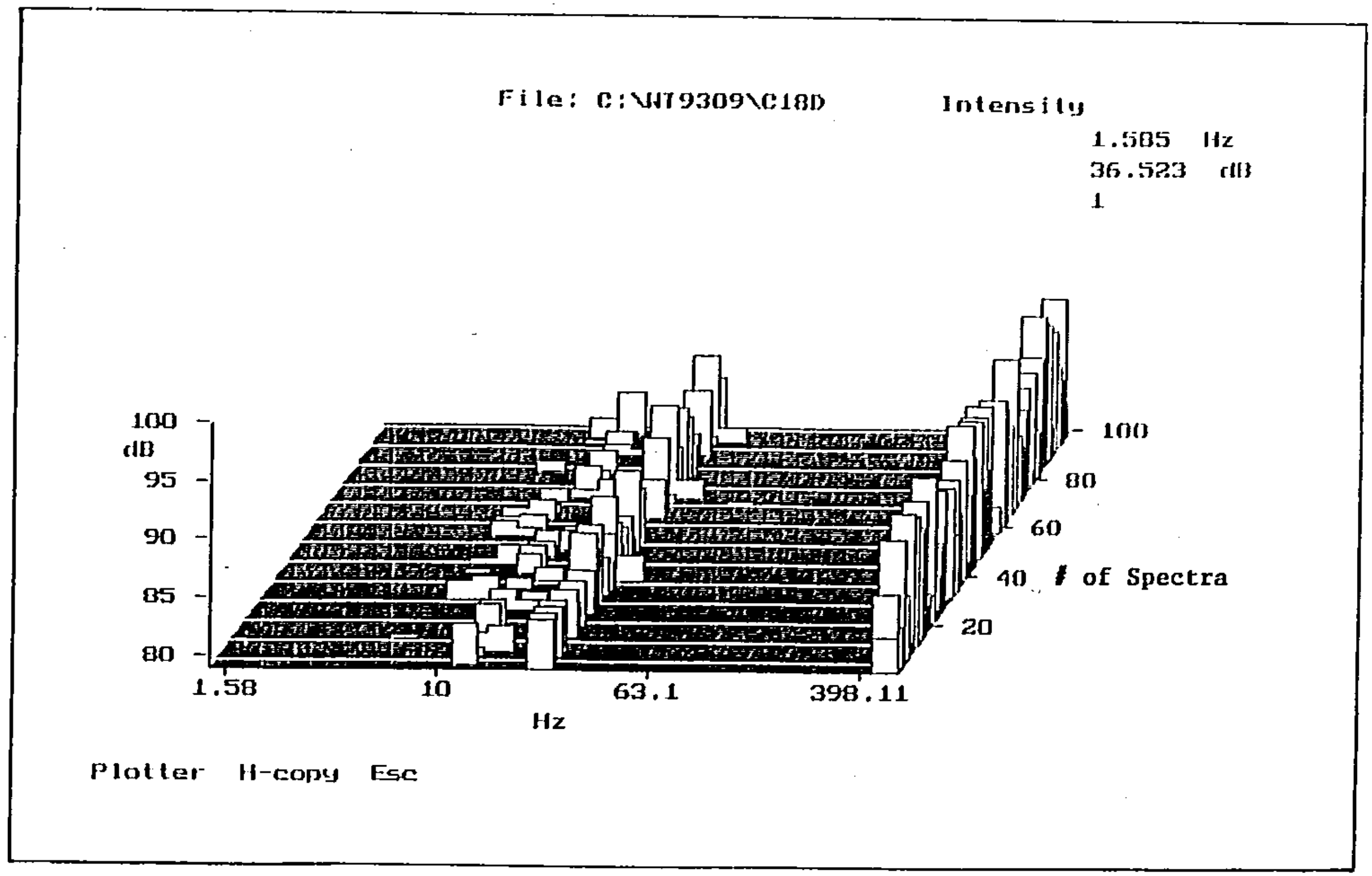

Figure 4.4: Waterfall Display of Vertical Vibration - Cleary Auditorium, Pile 1, Sect. 3

generally provides some vibration damping, while the exterior walls are responsible for much of the building's stiffness [15].

Due to the delicate condition of the Baby House, it was very important that the pile-driving at the south of the Cleary Auditorium did not damage the building or its contents in any way. An additional concern was possible damage to the newer underground archive structure, since it was actually closer to the pile-driving site. The underground archive is built of concrete, and is accessed through a tunnel from the basement of the Baby House. It is used to store artifacts, furniture and dishes not currently on display in the museum.

As mentioned in the Introduction, Canada does not have any guidelines governing suitable vibration limits for construction-induced vibration. It also does not have any guidelines relating to the impact of any type of vibration on historical sites. International vibration standards recommend limits generally in the order of 2 to 4 mm/s peak particle velocity, for vibration at frequencies under 30 Hz. The U.S. Environmental Protection Agency recommends a peak acceleration of 0.5 m/s^2 for ancient structures.

Pile-driving began at this site toward the end of June, 1990. The piles and diesel driver used were the same as those used on the north side of the Cleary Auditorium. The soil was also assumed to be similar to that found at the Cleary. The first five piles driven were monitored from a location in the basement of the Baby House, along the north foundation wall. The measurement site was in the furnace room, which was the only basement area where the exterior walls had not been covered by plaster or gypsum board. The first four piles driven were at a distance of 33.6 to 35.0 metres from the accelerometer location, while the fifth pile, at 24.7 metres, was the nearest pile to be driven. Figure 4.5 shows the relative layout of the building and piles.

FFT analysis of the tape recorded vibration signal revealed four main peaks in the vertical vibration direction, and one in the horizontal direction, as shown in Figure 4.6. In the vertical direction, peaks occurred at 13 Hz, 15.5 Hz, 22 Hz, and 28 Hz. The horizontal direction had only one main peak at 20 Hz. It would be trivial to suggest that the 20 Hz peak in the horizontal direction represents the natural (damped) mode of the walls of the Baby House in bending, but to attempt to quantify the peaks in the vertical direction would be more difficult. As discussed earlier, the Baby House was constructed without a supporting frame or superstructure, therefore this type of vibration cannot exist in this building. Thus the lower frequencies could represent the resonant frequency of the building's walls in flexure, while the higher frequencies, at 22 and 28 Hz, could describe the principal modes of the building's floors or windows.

Using Equation 3.7, and estimating the width of the Baby House at 18.3 metres (60 feet) and its height at three stories or 9 metres, then the natural frequency of the building superstructure would be predicted as:

$$p = 2\pi\sqrt{\frac{L}{0.5h}} = 2\pi\sqrt{\frac{18.3}{0.5(9)}} = 12.6\ rad/s = 2.0\ Hz$$

Equation 3.8 predicts:

$$p = \frac{2\pi}{0.1N} = \frac{2\pi}{0.1(3)} = 20.94\ rad/s = 3.3\ Hz$$

Neither of these equations would be valid predictors for the Baby House, as their

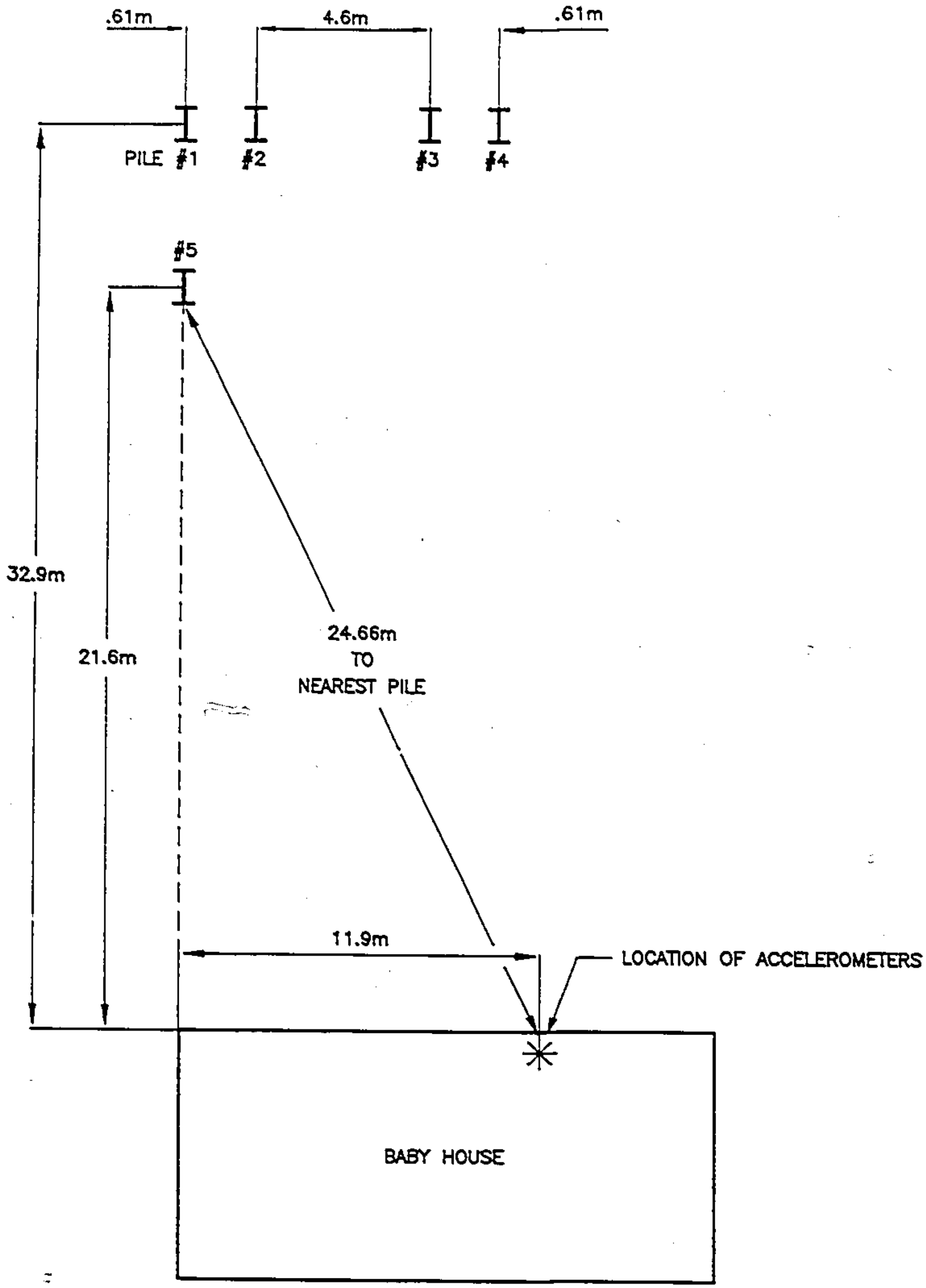

Figure 4.5: Sketch Showing Baby House and Layout of Piles

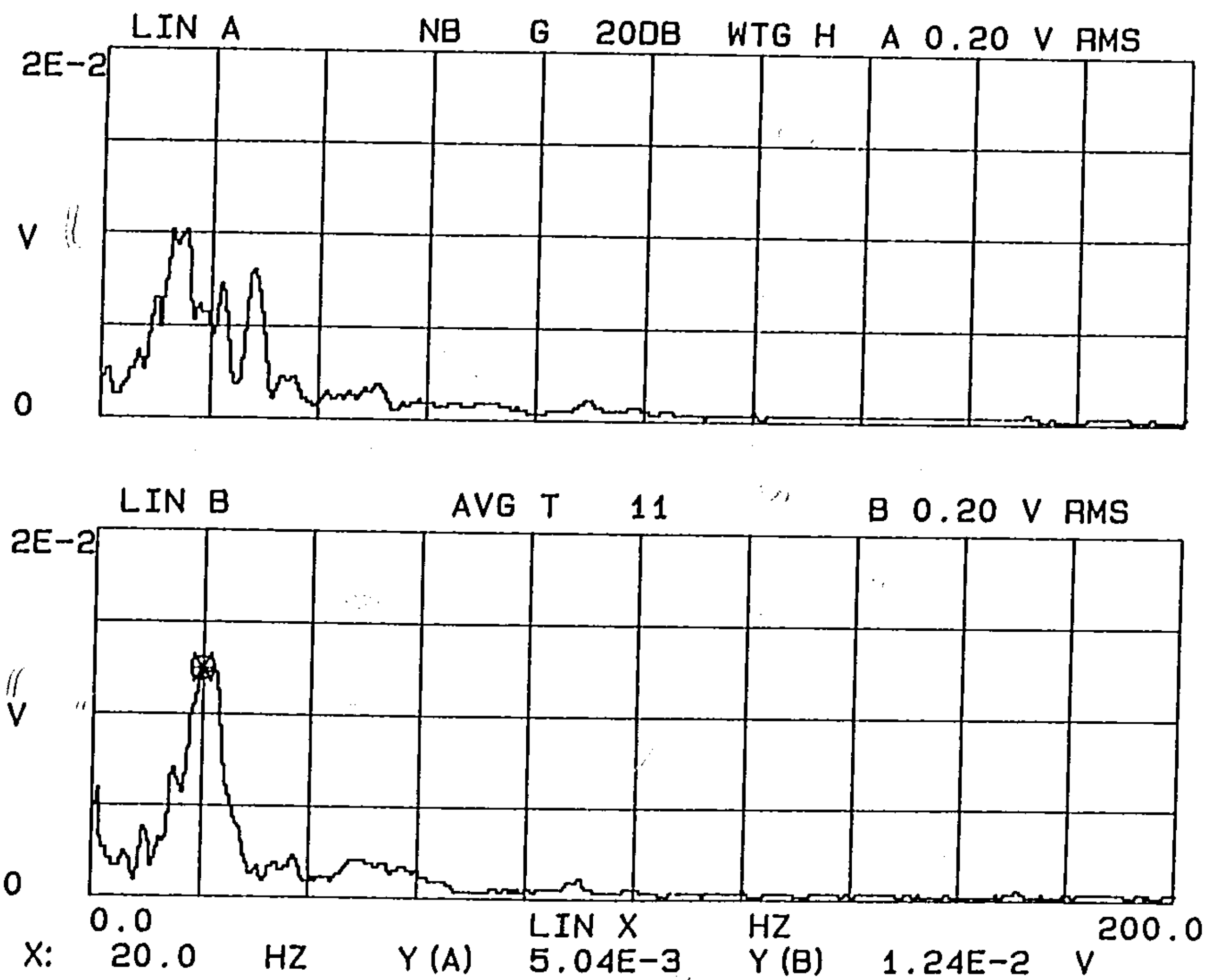

Figure 4.6: Baby House - FFT Spectrum in Vertical (A) and Horizontal (B) Directions

solutions indicate, since the equations are attempting to calculate the natural frequency of the superstructure of the building.

A time trace of the horizontal vibration signal, Figure 4.7, was used to determine the damped natural frequency, percent of critical damping and undamped natural frequency of the Baby House in the radial direction. The damped natural frequency was found from Equation 3.6, where the period T was read from the successive peaks on the time trace as being 0.05 seconds. Therefore:

$$p_d = \frac{2\pi}{T} = \frac{2\pi}{0.05} = 125.7\ rad/s = 20.0\ Hz$$

The damping ratio, zeta, was calculated using Equation 3.5, where $\dot{u}_n = 0.168$ V and $\dot{u}_{n+1} = 0.106$ V were the successive vibration amplitudes read from the time trace. Thus:

$$\zeta = \frac{1}{2\pi}\left(-\ln\frac{\dot{u}_{n+1}}{\dot{u}_n}\right) = \frac{1}{2\pi}\left(-\ln\frac{.106}{.168}\right) = 0.073$$

This calculation was repeated using a different segment of the time trace, and a damping ratio of 0.074 was obtained, which is reasonably consistent with the first result.

Finally, these two parameters were used with Equation 3.6 to find the undamped natural frequency in the horizontal direction:

$$T = \frac{2\pi}{p\sqrt{1-\zeta^2}} = \frac{2\pi}{p_d}$$

$$\therefore p_d = p\sqrt{1-\zeta^2}$$

$$p = \frac{p_d}{\sqrt{1-\zeta^2}} = \frac{125.7}{\sqrt{1-0.074^2}} = 126.0\ rad/s = 20.1\ Hz$$

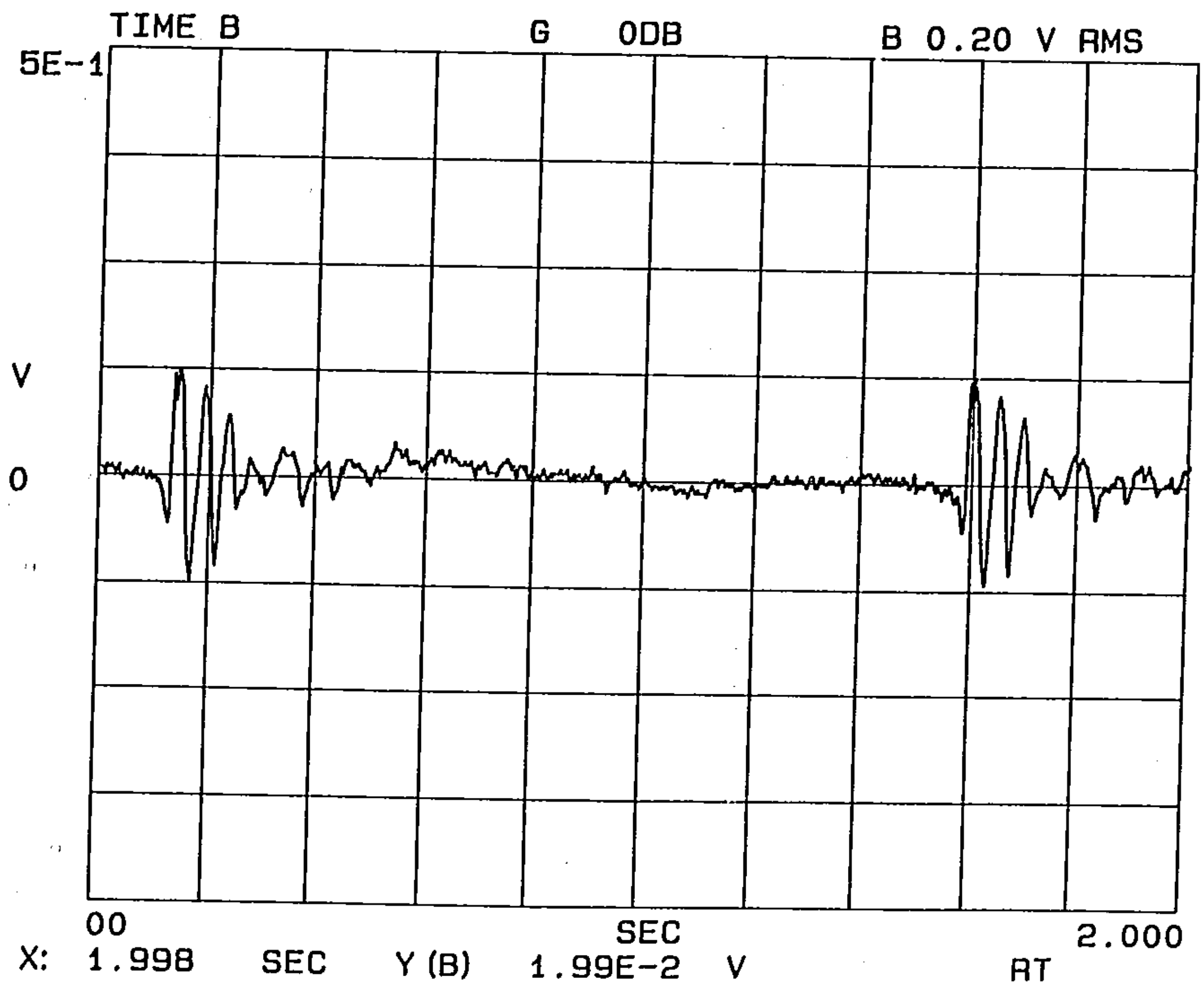

Figure 4.7: Baby House - Time Trace in Horizontal Direction

This sequence of calculations was not repeated for the vertical vibration data due to the author's inability to obtain a good graphical representation of the time signal in this direction. This may be due to the vertical vibration signal being composed of four resonant frequencies, rather than one, as was the case with the horizontal signal.

Data obtained using the B & K Frequency Analyzer were quite similar to that of the Cleary Auditorium. The results of the data analysis have been summarized in Table 4.1. This table lists the 1/3 octave frequency bands exhibiting the largest amplitude response to the pile-driving, for three different piles and varying pile depths. The corresponding Peak Particle Velocity, in decibels and millimetres/second, has also been indicated.

Table 4.1 Summary of Vibration Measurements - Baby House

Sect. (m)	Pile #1 (Hz)	PPV		Pile #3 (Hz)	PPV		Pile #5 (Hz)	PPV	
		dB	mm/s		dB	mm/s		dB	mm/s
1	12.6	93.4	0.046	8.0	98.6	0.085	16.0	92.0	0.040
0.0 to 15.2	10.0	88.9	0.028	16.0	98.6	0.085	12.6	91.8	0.039
	8.0	88.8	0.027	12.6	99.1	0.090	20.0	89.1	0.029
2	4.0	95.5	0.060	5.0	101.8	0.123	16.0	91.2	0.036
15.2 to 30.4	6.3	89.5	0.030	4.0	102.0	0.126	12.6	91.0	0.035
	8.0	87.4	0.023	8.0	101.5	0.119	20.0	90.8	0.035
3	8.0	96.6	0.068	25.1	92.2	0.041	12.6	104.9	0.176
30.4 to 45.7	50.1	96.8	0.069	31.6	91.8	0.039	16.0	104.8	0.175
	39.8	96.8	0.069	20.0	91.1	0.036	25.1	102.7	0.137

Examination of the data revealed that the pile driven closest to the Baby House (Pile #5) consistently caused elevated vibrations within the building that were at the principal frequencies of the Baby House in the vertical direction. This occurred at all pile depths. The other two piles listed in the table induced larger amplitude vibrations at the principal frequency of the building (approximately 13 Hz) for the first section of the pile driving, while the second section showed larger amplitudes at frequencies below the principal modes. Conversely, the third section of pile displayed larger amplitude vibrations at or above the third and fourth natural frequencies of the building, which are at 28 and 22 Hz, respectively.

The reason for this frequency deviation is not clear, but could be attributed to some unknown discontinuities existing in the transmission path between the first and third piles, and the vibration transducer location. These discontinuities could be caused by buried structural members from the original Cleary Auditorium construction, or it could be possible that the concrete work for the Baby House's underground archive structure extends into the transmission path. This shift in predominant frequencies with pile depth is similar to that exhibited by the Cleary Auditorium during earlier vibration measurements, and other possible causes have been discussed in Section 4.3.1.

Table 4.1 also does not indicate any clear relationship between vibration amplitude and depth of pile, or distance to the receiver. The literature has shown that if such a relationship can be determined, it is usually plagued by a significant degree of statistical uncertainty.

Again, as with the Cleary Auditorium, the vibration amplitudes appear to be too small to allow even cosmetic damage to the Baby House, with a maximum peak particle velocity of 0.176 mm/s being recorded. However, several small flakes from the exterior brickwork were discovered in the shrubbery surrounding the building during the driving of the first five piles. In the absence of a comprehensive pre-construction survey, it would be impossible to determine whether these flakes existed prior to the start of construction, but it would be reasonable to speculate that the piling may have accelerated the normal deterioration of the building, as suggested by Crockett [21].

To summarize, although the pile-driving vibration was not considered large enough to cause damage to the Baby House or to the underground archive structure and its contents, a small amount of flaking of the exterior brickwork was noted, which may or may not have been exacerbated by the construction activities.

4.3.3 Bellewood Estates

Vibration measurements were also made at two residential dwellings in the Bellewood Estates subdivision, just east of Huron Church Road. Several home owners living next to this highway had complained to the City of Windsor regarding the vibrations caused by a contractor who was involved in the widening of Huron Church Road. The home owners felt that the vibration caused by the vibratory compacting equipment used to compact the road base was of such a large magnitude that it was damaging their houses. Cases of cracked foundations and water leakage to the basements had been cited.

The purpose of this investigation was solely to determine if there was a reasonable possibility that the vibratory compactor could have been responsible for the building damage. Measurements were made at two houses that were adjacent to Huron Church Road. The homes were approximately 44 metres from the centre lane that was being compacted during these measurements. The alleged damage had occurred when the curb lane was being compacted several days earlier, at an approximate distance of 36.8 metres from the houses.

In both cases, an accelerometer was glued to the foundation (basement) wall at a location that was free from additional wallcovering. The equipment set up was the same as that for the Cleary Auditorium. The accelerometer was usually mounted in the radial direction, although some vertical vibration measurements were also recorded. A one second exponential averaging time was used, with the range of centre frequencies set from 4 to 400 Hz. The maximum linear RMS values were recorded, and are listed in Table 4.2. Data from the first location were limited since the vibratory compactor was stopped shortly after the measurements were started.

At both houses, the 20 Hz centre frequency was the predominant mode of vibration, although the spectral response was more linear than had been the case at either the Cleary Auditorium or the Baby House. The vibration was also of a continuous nature, while the pile-driving vibration had been transient.

Although there was very little data available from the first set of vibration measurements, their large amplitude would indicate that it may have been possible for the vibratory compactor to cause damage to the houses. Even though the vibration levels were considerably lower than the traditional limit of 50.8 mm/s (2 inches/s), they were close to the new U.S. Bureau of Mines limit of 13 - 19 mm/s set for blasting. Again, Canada does not have vibration standards that would apply to this situation.

Table 4.2 Vibration Measurements - Bellewood Estates

	Time	Linear Max. RMS Value (dB)	Velocity (mm/s)
House A 10.26.90	N/A	140.0 (vertical)	10.0
	N/A	143.0 (radial)	14.1
House B 11.7.90	14:10	110.4 (radial)	0.33
	14:14	119.6 "	0.96
	15:20	110.0 "	0.32
	15:25	110.0 "	0.32
	15:55	117.0 "	0.71
	16:10	118.2 "	0.81
	16:12	115.4 "	0.59

It should also be noted that complaints of damage came only from home owners whose houses were adjacent to the highway. Different builders had constructed these homes, which would tend to negate the possibility that the damage to these dwellings was due to the faulty construction practices of one builder.

V. COMPARISON OF RESULTS

In Chapter III, several empirical and theoretical techniques for the prediction of building response to ground vibration were examined. In Chapter IV, field measurements were described, and two different methods were used to analyze the field data. In this chapter, the empirical and theoretical predictions will be compared with the field measurement results.

5.1 Baby House

The Baby House was the primary structure studied for this thesis, therefore the bulk of this chapter will deal with a comparison of the results obtained for this building.

The first comparison made was between the actual vibration data and the vibration velocities predicted by the empirical equations noted in Table 3.3. Since the largest Peak Particle Velocity to be recorded at the Baby House was only 0.176 mm/s, all of the empirical velocity calculations listed in Table 3.3 were quite conservative. The pile-driving and construction equations (equations 3.14, 3.17, 3.19) predicted velocities in the range of 4.61 to 14.13 mm/s. The fact that all three equations are conservative may be the only noteworthy feature of these formulae. At least if the equations predict velocity values that are below any damage threshold criteria, one can be assured that no damage will occur as a result of the construction activity.

Next the vibration responses recorded in 1/3 octave frequency bands with the B & K Frequency Analyzer were compared to the Pseudovelocity values calculated using Dowding's Spectral Response method. A typical vibration response was plotted on tripartite paper, and is shown as Figure 5.1. The amplitudes of the field measurements were several orders of magnitude less than the predicted Response Spectra, therefore the scale of Figure 5.1 has been amplified to improve the legibility of the graph.

At first glance Figures 5.1 and 3.4 do not appear to resemble each other to any great extent, but closer examination shows that certain similarities do exist between the two graphs. It should be stressed that Figure 5.1 is a graphical depiction of a random sampling from all of the Baby House data available, and represents only a typical vibration response. Conversely, Figure 3.4 illustrates the maximum expected responses of a series of single-degree-of-freedom systems, having different natural frequencies, to a known vibration input. Nonetheless it is interesting to note that both graphs indicate a peak vibration response in the range of 15 to 20 Hz, with a distinct decline in vibration amplitude beyond 20 Hz. A secondary peak is predicted at 4 Hz, and occurs at 5 Hz. Finally, both graphs show very low vibration velocities at 2 Hz. These similarities should be sufficient to suggest that Spectral Response may be a viable indicator of building response to pile-driving vibration, and should warrant further research in this area.

It is also interesting to note that the maximum pseudovelocity of 8.6 mm/s predicted by the Spectral Response method is near the mean of those values predicted by equations 3.14, 3.17 and 3.19. Thus this method would seem to be at least as good as

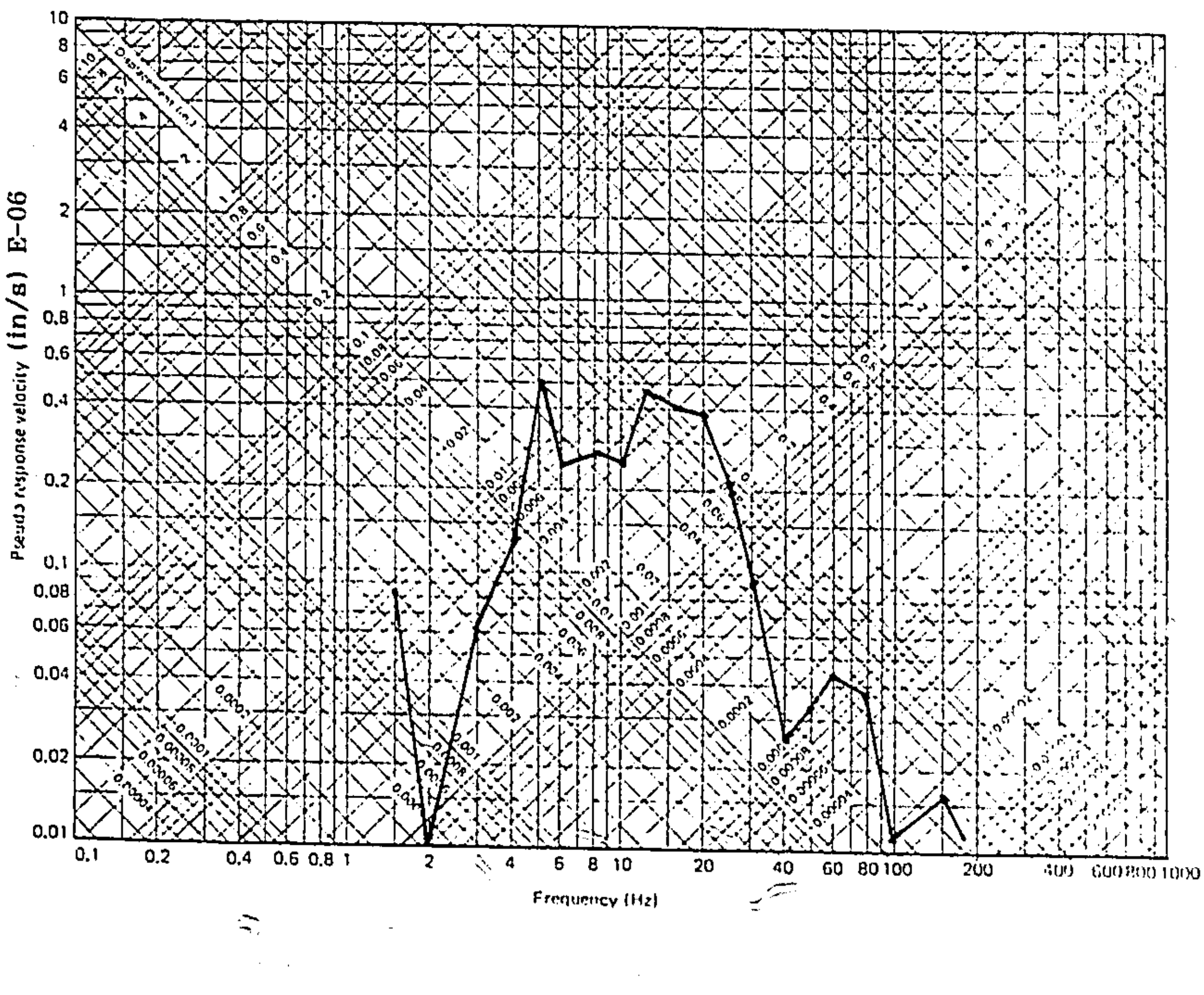

Figure 5.1: Baby House - Typical Vibration Response on Tripartite Paper

the other prediction methods, and has the advantage of being able to predict the frequencies of greatest amplitude response.

Finally, Crockett's Method predicted a safe vibration velocity of 1.08 mm/s, as calculated in Section 3.2.2. This value was much higher than the peak particle velocity of 0.176 mm/s recorded for the Baby House, and would therefore indicate that no damage would occur to the historical building. Since no damage was noted except for a small amount of flaking of the external brickwork, this comparison is inconclusive.

5.2 Cleary Auditorium

The distance between the nearest pile and the Cleary Auditorium was less than the distance between the nearest pile and the Baby House, therefore values predicted by the empirical equations described in Section 3.2.1 were somewhat larger for the Cleary than those predicted for the Baby House. A rated energy of 54000 Joules, and a source to receiver distance of 15 metres was used for these calculations, and the results are listed in Figure 5.1.

Table 5.1 Cleary Auditorium Velocity Calculations

Eqn	Source	Predicted Velocity
3.14	Attewell & Farmer	PPV = 23.3 mm/s
3.17	Mallard & Barstow	PPV = 10.4 mm/s
3.19	Wiss	PPV = 14.0 mm/s ref. [22] Fig. 5

The peak particle velocity of 0.036 mm/s obtained at the Cleary was significantly lower than that of the Baby House, although the mean vibration amplitudes were similar.

This means that the prediction equations in Figure 5.1 produce a larger error than they did for the Baby House. The low vibration response of the Cleary Auditorium may be due in part to the size and solid construction of the Cleary Auditorium compared to the Baby House, which would affect the transmission of the vibration waves throughout the building.

5.3 Bellewood Estates

Unfortunately it was not possible to compare the field data obtained at the Bellewood Estates with any empirical or theoretical prediction methods. This was due in part to the limited data that had been recorded at the homes, and also to the lack of information available regarding the vibratory compacting equipment used for the roadway construction. Since the vibration velocities recorded at the first site were much larger than those recorded at the second site, and the house to roadway distances were similar, it bears speculation that two vibratory compactors of different sizes were used for the project. A tall noise barrier blocked the line of sight between the roadway and the houses, thus preventing identification of the equipment being used while the vibration measurements were being made. Also, the construction company performing the work was not inclined to provide this information. Nonetheless, the literature has indicated that vibratory compactors can often be much more destructive than pile-drivers, and the data recorded at this site would seem to bear this out.

5.4 Field Measurements

After comparing the field location results with the empirical and theoretical predictions, it is also possible to compare the field measurements recorded at each site with each other. Sections 4.3.1 and 4.3.2 describe the similarities between the vibrations recorded at the Baby House and the Cleary Auditorium. These sections discuss the shift in the frequencies of greatest amplitude response with increasing pile depth. Section 4.2 examines the occurrence of peak vibration amplitudes after a "rest" in the piling operation. The magnitudes of vibration were similar at both sites, even though the construction of the buildings was quite different.

The most notable difference between the measurements recorded at the Baby House and Cleary Auditorium, and those recorded at Bellewood Estates is in the type of vibration recorded, due to the different construction equipment used. Pile-driving is characterized by a transient type of vibration response, where the peak vibration amplitudes generally occur at the fundamental frequencies of the building. Conversely, vibration caused by a vibratory compactor is of a steady nature, with a fairly uniform frequency distribution between 1 and 200 Hz. The maximum velocity of 14.1 mm/s recorded at Bellewood Estates was significantly larger than that recorded at either the Cleary Auditorium or Baby House.

VI. CONCLUSIONS AND RECOMMENDATIONS

Studies have shown that the sensitivity of humans to vibration is approximately one hundred times greater than that of buildings to the same vibration. In many cases, this means that what is perceived as a severe or intolerable vibration by a good percentage of the population may in fact be not nearly large enough to cause damage to a building. However, this does not imply that engineers do not need to be concerned about possible vibration damage. Vibration damage does occur on a regular basis, and even the smallest amount of damage can cause irreparable harm or accelerate the natural deterioration of an ancient building. The Baby House is a good example of this occurrence; even the low amplitude vibrations seen by the Baby House during the pile-driving work were sufficient to cause a possible increase in the flaking of the building's exterior brickwork, although no other damage was noted.

This does not mean that efforts to limit vibration should become so all-encompassing that normal construction work is impeded. Instead, the Canadian government should endeavour to create reasonable vibration limits for the construction industry based on the existing standards followed by several European countries. Contractors should be required to look at alternative construction methods, or employ vibration mitigation measures if there is a reasonable probability that their construction work will exceed the applicable vibration limits. Unfortunately, there have been few effective vibration mitigation techniques developed to date, therefore the suggestiion of vibration mitigation may be impractical for the majority of construction situations [40].

The empirical prediction equations used in this thesis do not seem to be good predictors of vibration response in general, although a particular equation may represent a specific set of measurements quite well. The failure of these equations to take into account the frequency of vibration is also an element that limits their usefulness.

It is recommended that further work be done in the application of Spectral Response techniques to the case of pile-driving vibrations. Initially, an attempt to develop a computer solution to equation 3.10 should be made, using the tape recorded vibration response of the Baby House as the input data. If this method is successful, it could then be applied to other types of construction vibration. Finally, it is also recommended that the Spectral Response method be tested for use in other vibration situations, such as road and railway vibrations.

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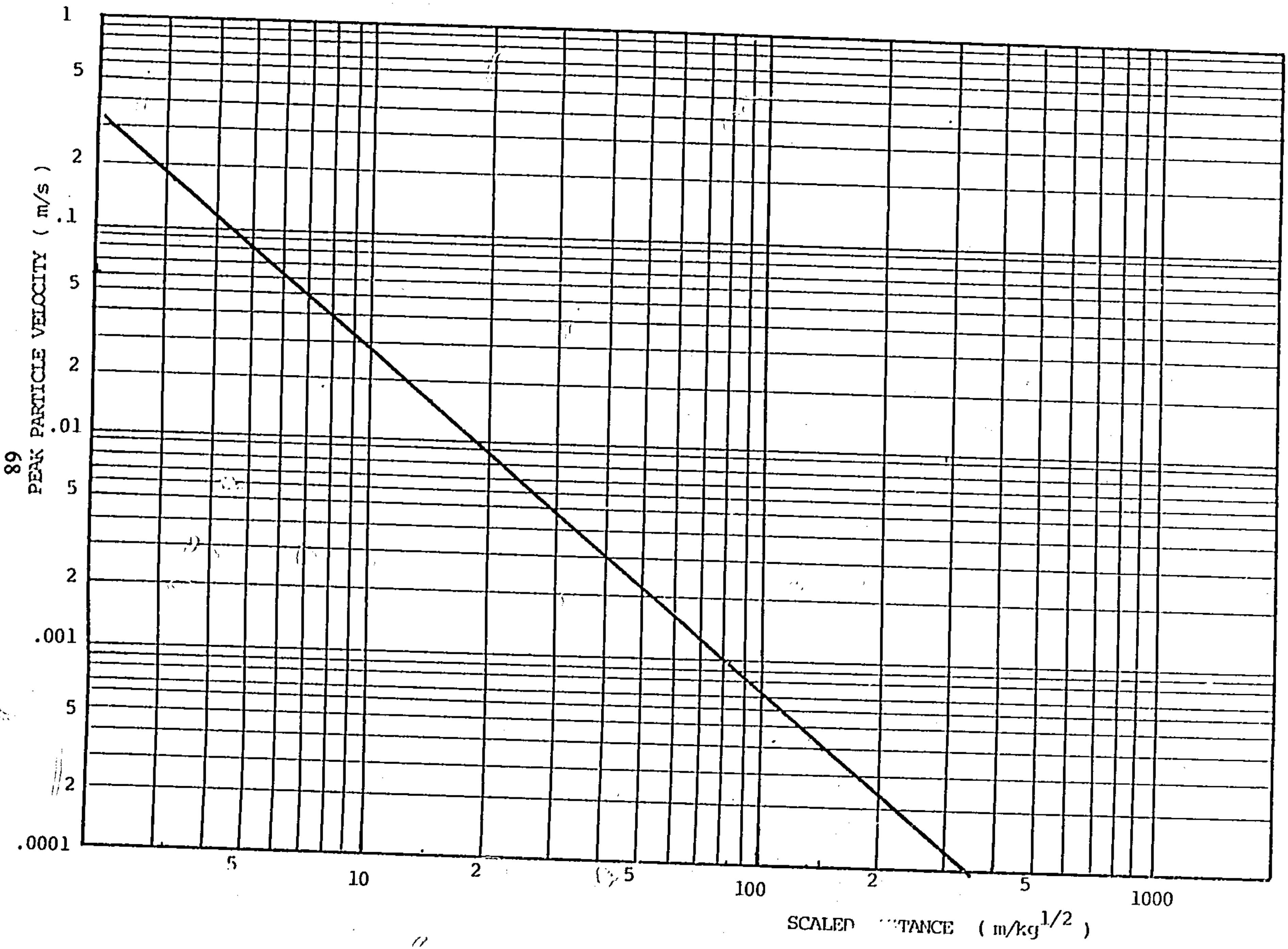
APPENDIX I

Ministry of the Environment
Blasting Vibration Prediction Graph

FIGURE 3. BLASTING VIBRATION PREDICTION



APPENDIX II

Crockett's Graph of Basic Intensity Patterns and Weighting Tables

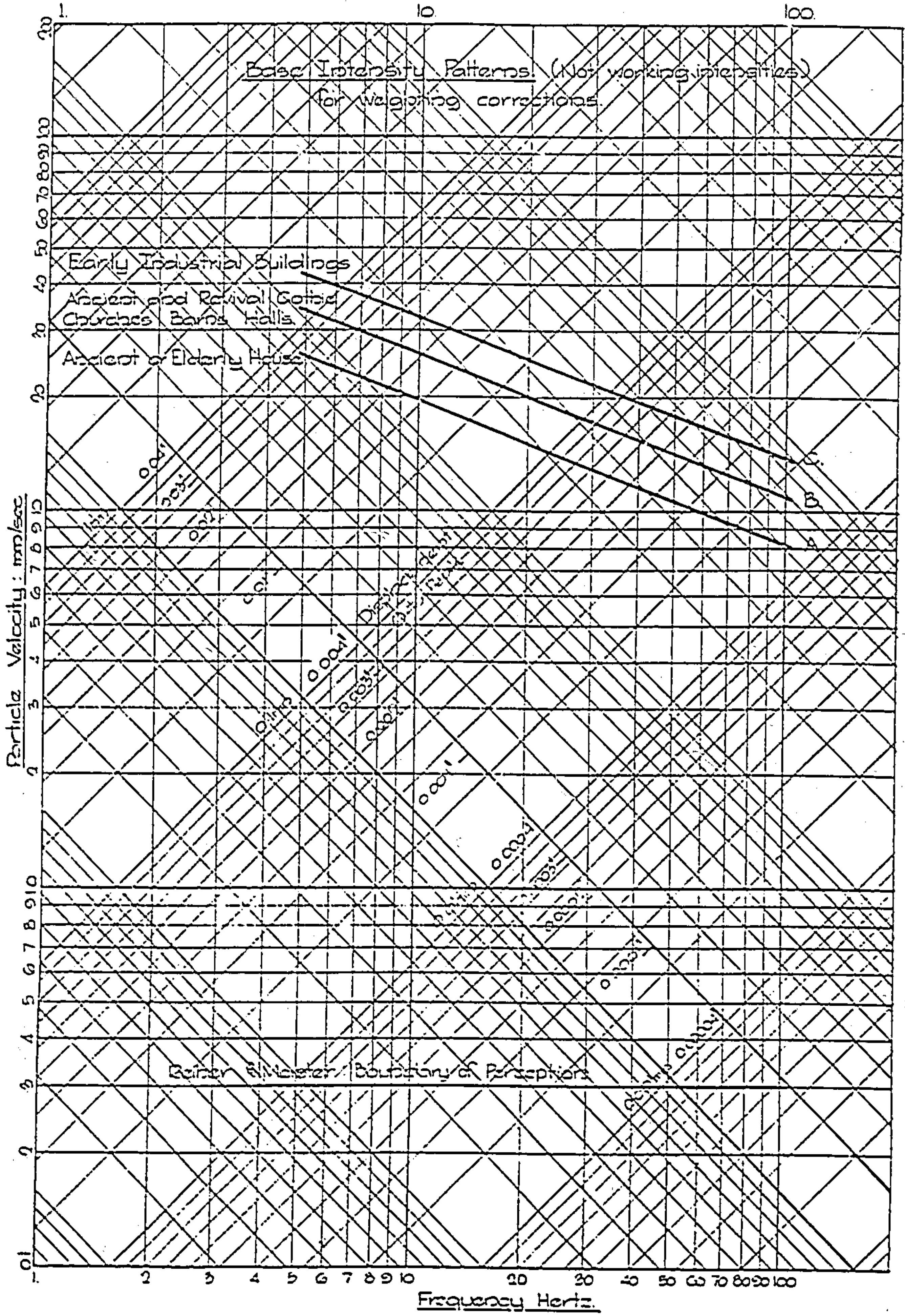


Fig. 1. Ancient buildings

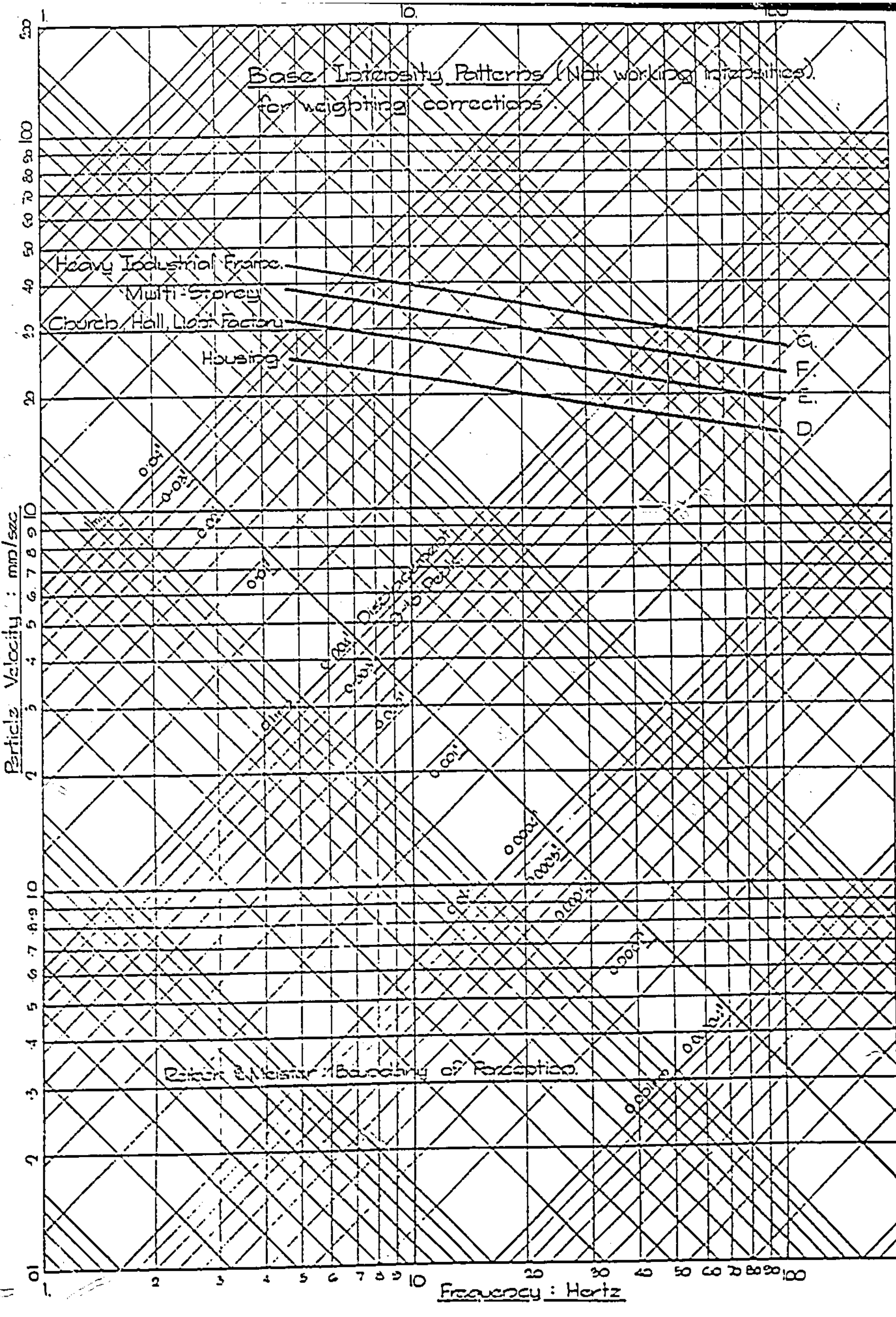


Fig. 2. Modern buildings

TABLE A. Detail Effects Weighting for Ancient or Elderly House.

		Weighting
		Positive
a.	Hard mortar in all jointing	+1
b.	Dis-similar materials on inner and outer wall faces, with hard mortar	+1
c.	Thicker walls than usual 1' 6" to 2' 3" (450 to 690 mm)	+1
d.	Very much thicker walls than usual	+2
e.	Very soft lime mortar, e.g. 1:6 of Lime : sand	+2
f.	Light springy timber roof on heavy walls	+1
g.	etc.	
		Negative
1.	Dis-similar materials on inner and outer wall faces, with soft mortar, e.g. flint and stonework	-2
2.	All walls, floors, and roof rather light weight	-2
3.	All walls of stone and brick rather light weight	-1
4.	Timber frame walls, floors, roof with lathe and plaster inside and outside.	-3
5.	Roof inadequately tied	-1
6.	Roof trusses very bad	up to -5
7.	Three storeys high	-1
8.	Four storeys high	-2
9.	Inadequate retaining walls	up to -5
10.	Rotting beam ends or broken beams	up to -5
11.	Hard thin horizontal mortar joints next to thick soft joints	-1
12.	Lack of bonding between inner and outer walling	-2
13.	Walls fairly badly cracked for any cause	-1
14.	Walls very badly cracked for any cause	-2
15.	Loose wall tiles	-1
16.	Loose roof tiles	up to -5
17.	Some loose plaster, walls and ceiling	-2
18.	Much loose plaster, walls and ceiling	-3
19.	Built in timber bressemers	-1
20.	Inverted pendulums, chimney stacks or gables	-1
21.	Stresses higher than usual	-1
22.	Stresses very high	-2
23.	Medium amount of settlement	-1
24.	Large amount of settlement	-2
25.	Resonant build up between ground waves and any main structural member	-2
26.	Bad maintenance for many years, up to no heating, water penetration, generally damp, rotting timbers	-4
27.	Arches inadequately side buttressed	-1
28.	Vaulting, up to 2 tons/ft. run (65 Kn/m) thrust	-1
29.	etc.	

TABLE B. Detail Effects Weighting for Ancient and Revival Gothic Churches, Barnes, Town Halls.

		Weighting
		Positive
a to f	as in Table A	
h.	Walls more than 4' 6" (1380 mm) thick	+3
j.	Walls founded on uncompactable sand, gravel, clay, rock	+1
k.	Small rather than large windows	+1
l.	Very heavy timber roofs well tied or buttressed	+1
m.	etc.	
		Negative
1 to 28	as in Table A.	
30.	Arches inadequately side buttressed up to 2 tons(20 Kn) thrust	-1
31.	Arches inadequately side buttressed up to 10 tons (100 Kn) thrust	-2
32.	Arches inadequately side buttressed up to 20 tons (200 Kn) thrust	-2
33.	Arches well side - buttressed above 20 tons (200 Kn)	-1
34.	Vaults up to 5 tons(50 Kn) side thrust per panel point	-2
35.	Vaults up to 15 tons (150 Kn) side thrust per panel point	-2
36.	Arcade arches on columns up to 10' (2.5 m) high	-1
37.	Arcade arches on columns up to 20' (5.0 m) high	-2
38.	Arches, arcade and single, badly distorted or "spread"	-1
39.	Arches very flat	-1
40.	Flying Buttresses in good condition	-1
41.	Flying Buttresses in cracked condition	-2
42.	Spreading arches distorting window tracery	-1
43.	Bulging tracery carrying valuable stained glass	up to -10
44.	Stresses up towards failure point in stones set in mortar	-1
45.	Inverted pendulum pinnacles	-2
46.	Inverted pendulum spires	-3
47.	Inverted pendulum parapets, especially pierred types	-1
48.	Vertically split bell towers	up to -3
49.	Alterations of widely different stonework	-1
50.	Altered foundations causing structural weakness	-1
51.	Circular and semi-circular walls with no adequate tying between inner and outer skins	-1
52.	Bad maintenance over centuries	-3
53.	etc.	

TABLE C. Detail Effects Weighting for Early Industrial Buildings.

	Weighting
	Positive
a to f as in Table A.	
h to l as in Table B.	
n. Very soft lime mortar in thick joints	+3
o. Cast iron or wrought iron beams built in	+2
p. Floors of tied brick jack arches	+3
q. Cast iron columns bolted or well fixed sideways	+1
r. etc.	
	Negative
1 to 28 as in Table A.	
31 to 52 as in Table B.	
54. Factories of three or more storeys with cast iron columns, and timber, iron or steel beams all untied and unbolted	-3
55. Ditto, but subjected to much horizontal machine forces causing structural damage	-1
56. Distorted and leaning walls	-2
57. Built in stresses from mining subsidence	up to -3
58. etc.	

TABLE D. Detail Effects Weighting for Modern Housing, including those to Codes of Practice.

	Weighting
	Positive
a to f as in Table A.	
s. Reinforced concrete suspended floors and roof, insitu or precast	+1
t. Mortar to C.of P., or 1:3:6	+1
u. Calculated Foundations	+1
v. Plaster board or similar on walls	+1
w. Plaster board or similar on ceiling	+1
x. Minimum cracking of structure and insitu plaster	+1
y. Expansion jointing in long or terrace housing	+1
z. Calculated roof structure rather than empirical	+1
aa. Calculated floors structure rather than empirical	+1
bb. Single storey	+1
cc. etc.	
	Negative
1 to 28 as in Table A.	
59. Greater elastic flexibility compared with older types	-1
60. Lighter external walling compared with older types	-1
61. Lighter internal walling compared with older types	-1
62. Small settlement cracks	-1
63. Extensive settlement cracks	-2
64. Contraction cracks from modern hard materials	-1
65. Extra storey added after original completion	up to -5
66. Other additions causing cracks	up to -5
67. etc.	

TABLE E. Detail Effects Weighting for Modern Church, Hall, Light Factory to Codes of Practice.

	Weighting
	Positive
a to f as in Table A.	"
h to l as in Table B.	"
s to bb as in Table D.	"
dd. Structural Steel roof structure	+1
ee. etc.	"
	Negative
1 to 28 as in Table A.	"
30 to 52 as in Table B.	"
55 to 57 as in Table C.	"
59 to 66 as in Table D.	"
67. etc.	"

TABLE F. Detail Effects Weighting for Multi-Storey Buildings to Codes of Practice.

	Weighting
	Positive
a to f as in Table A.	"
s to bb as in Table D.	"
dd as in Table E.	"
ff. Monolithic construction of reinforced concrete, structural steel or calculated brickwork	+1
gg. High grip stress	+2
hh. Low stress on concrete and steel	+2
jj. etc.	
	Negative
1 to 28 as in Table A.	"
59 to 66 as in Table D.	"
68. Badly non-tied cladding	-1
69. Special face cladding of thin marble, tiles	-1
70. Poorly tied or attached precast main structural units	-1
71. Poorly concreted suspended floor slabs or slabs cast on highly absorbent wood wool or similar	-2
72. Badly concreted suspended floor slabs or slabs cast on highly absorbent wood wool, or similar	-3
73. Weak grip strength	-4
74. Non-tied block or brick panels	-1
75. Non-tied parapets	-2
76. Non-tied parapets badly temperature cracked	-3
77. Tiled surfaces external	-3
78. Tiled surfaces internal	-1
79. Loose plaster on walls	-1
80. Soft plastered ceilings	-3

TABLE F. Continued

	Negative
81. Suspended ceilings	
82. etc.,	

TABLE G. Detail Effects Weighting for Modern Heavy Industrial and Open Frames.

	Weighting
	Positive
ff to hh as in Table F.	
kk. Designed specifically to withstand strong vibration	up to +10
11. etc.	
	Negative
68 to 81 as in Table F.	
83. Badly cracked or corroded R.C.	up to -5
84. Badly compacted R.C.	up to -5
85. Very highly stressed R.C. or S.S.	- -
86. Light sheet cladding	-
87. Unsuitable change of use	up to -5
88. Unsuitable alteration	up to -5
89. Bad maintenance	up to -5
90. etc.	

TABLE K
Importance Factor.

Factor.	Classification Suggestion.
1.	Light industrial buildings of no particular merit. House near end of life.
2.	Ordinary business premises of no particular merit. Quite old ordinary house.
3.	Ordinary private houses and the better industrial buildings and schools.
4.	Medium aged and modern private houses of better than ordinary architectural merit from cottage to mansion, with hospital and public buildings.
5.	Ancient listed houses.
6.	Ancient listed buildings of no particular architectural merit, including early industrial buildings.
7.	Listed modern buildings of high architectural merit.
8.	Listed ancient buildings of high architectural merit and historical importance, Special and "great" houses.
9.	Modern buildings of highest artistic merit.
10.	Sensitive ancient buildings of the highest archeological, historical, architectural, and artistic merit.

TABLE L
Number of Ground Cycles Ratio.

Total No. of Cycles		$\sqrt[4]{\frac{10^4}{L}}$
10,000	$= 10^4$	1.0
100,000	$= 10^5$	0.6
500,000	$= 5 \times 10^5$	0.4
1,000,000	$= 10^6$	0.3
1,500,000	$= 1.5 \times 10^6$	0.3
10,000,000	$= 10^7$	0.2
100,000,000	$= 10^8$	0.1

TABLE J.
Soil Compaction Weighting Suggestions.

Soil Type.		Weighting
1.	Rock	0
2.	Well filled cohesive clay	0
3.	Gravel, sand, and silt, well compacted, no grading "gaps"	0
4.	Ditto. poorly graded	up to - 5
5.	Ditto. badly graded	up to -10
6.	Peat and flowable silt	up to -15
7.	Important amount of "heave"	up to -15
8.	Waterlogged chalk dust, silt, etc. like toothpaste	up to -20
9.	Mining subsidence	up to -10

APPENDIX III

Spectral Response Calculations Using Simpson's 1/3 Rule

Frequency f (Hz)	Time tau 0.125	0.25	0.374	Velocity u(t) 0.014	0.126	0.038	Simpson's Rule (dmax)	PV (mm/s)	PV (in/s)
0.1	0.156687	0.078343	0.000627	0.013525	0.124235	0.037997	0.182803	0.114858	0.004522
0.2	0.313374	0.156687	0.001253	0.012732	0.121728	0.037993	0.179195	0.225183	0.008865
0.4	0.626748	0.313374	0.002507	0.010299	0.114585	0.037986	0.166859	0.424388	0.016708
0.6	0.940122	0.470061	0.00376	0.006984	0.104807	0.03798	0.154715	0.583264	0.022963
0.8	1.253495	0.626748	0.005014	0.003144	0.092689	0.037973	0.137278	0.690033	0.027167
1.0	1.566869	0.783435	0.006267	0.000832	0.078578	0.037966	0.117692	0.739479	0.029113
2.0	3.133739	1.566869	0.012535	0.011242	0.067492	0.03793	0.026378	0.331471	0.01305
4.0	6.267477	3.133739	0.02507	0.009027	0.101179	0.037854	0.150517	3.782916	0.148933
6.0	9.401216	4.700608	0.037605	0.007248	0.005301	0.037773	0.022073	0.832131	0.032761
8.0	12.53495	6.267477	0.05014	0.005819	0.081243	0.037686	0.122813	6.173237	0.24304
10.0	15.66869	7.834347	0.062675	0.004672	0.003683	0.037592	0.018997	1.193604	0.046992
20.0	31.33739	15.66869	0.12535	0.001556	0.042044	0.037041	0.068917	8.660365	0.340959
40.0	62.67477	31.33739	0.250699	0.000172	0.014008	0.035519	0.030571	7.683245	0.302489
60.0	94.01216	47.00608	0.376049	1.89E-05	0.00466	0.033469	0.017374	6.549705	0.257862
80.0	125.3495	62.67477	0.501398	2.06E-06	0.001548	0.030931	0.012373	6.219492	0.244861
100.0	156.6869	78.34347	0.626748	2.23E-07	0.000513	0.027954	0.010001	6.28405	0.247403
200.0	313.3739	156.6869	1.253495	2.98E-12	2.01E-06	0.008533	0.002847	3.577486	0.140846

Formulas:

Col. 2, 3, 4	2pi*f*sqrt(1-zeta^2)*(t-tau)
Col. 5, 6, 7	u(t)exp(-zeta*2pi*f*(t-tau))*(cos(formula1)-(zeta/(1-zeta^2))sin(formula1))
Col. 8	1/3(col5+4*col6+col7)
Col. 9	dmax*2pi*f
Col. 10	PV*0.03937

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