253

National Cooperative Highway Research Program

PB98-122567

NCHRP Synthesis 253

Dynamic Effects of Pile Installations on Adjacent Structures

A Synthesis of Highway Practice

Transportation Research Board
National Research Council

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

PB98-122567

2. REPORT DATE
1997

3. REPORT TYPE AND DATES COVERED
Final Report

4. TITLE AND SUBTITLE
NCHRP Synthesis of Highway Practice 253: Dynamic Effects of Pile Installations on Adjacent Structures

5. FUNDING NUMBERS
HR 20-5

6. AUTHOR(S) Transportation Research Board

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
Transportation Research Board/National Academy of Sciences
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

8. PERFORMING ORGANIZATION REPORT NUMBER
Project 20-5 Topic 25-16

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
American Association of State Highway and Transportation Officials
44 North Capitol Street, N.W. Suite 225
Washington, D.C. 20045

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES Sponsored in cooperation with the Federal Highway Administration

12a. DISTRIBUTION/AVAILABILITY STATEMENT Available for $24.00 from: Transportation Research Board 2101 Constitution Avenue, N.W., Washington, D.C. 20418

12b. DISTRIBUTION CODE:
unlimited

13. ABSTRACT (Maximum 200 words)

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This report of the Transportation Research Board presents information on the theory of ground motion and vibrations due to pile installations, vibration mitigation measures, and the consequences of groundborne vibrations. Additional detailed information on instrumentation is used for vibration measurement and the management of vibrations is also included. The appendices include a bibliography, a primer on pile support mechanisms related to pile driving vibrations, several examples of existing agency vibration specifications and contract wording, and an example vibration specification from the literature.

14. SUBJECT TERMS Soils, Geology, and Foundations; and Maintenance

15. NUMBER OF PAGES
86

16. PRICE CODE $24.00

17. SECURITY CLASSIFICATION
Unclassified

18. SECURITY CLASSIFICATION OF THIS PAGE Unclassified

19. SECURITY CLASSIFICATION OF ABSTRACT Unclassified

20. LIMITATION OF ABSTRACT
Unlimited

National Cooperative Highway Research Program

Synthesis of Highway Practice 253

Dynamic Effects of Pile Installations on Adjacent Structures

RICHARD D. WOODS, Ph.D., P.E.
University of Michigan

Topic Panel

DONALD J. ARCARI, *New York State Department of Transportation*
CARLOS BRACERAS, *Utah Department of Transportation*
MICHELLE M. CRIBBS, *Federal Highway Administration*
CARL D. EALY, *Federal Highway Administration*
ALFRED J. HENDRON, *University of Illinois at Urbana*
G.P. JAYAPRAKASH, *Transportation Research Board*
PAUL D. PASSE, *Florida Department of Transportation*
JOHN J. RESETAR, *Vibra-Tech*
CLIFFORD J. ROBLEE, *California Department of Transportation*

Transportation Research Board
National Research Council

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

NATIONAL ACADEMY PRESS
Washington, D.C. 1997

Subject Areas
Soils, Geology, and Foundations;
and Maintenance

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communication and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NCHRP SYNTHESIS 253

Project 20-5 FY 1993 (Topic 25-16)
ISSN 0547-5570
ISBN 0-309-06109-1
Library of Congress Catalog Card No. 97-62280

Price $24.00

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

and can be ordered through the Internet at:

http://www.nas.edu/trb/index.html

Printed in the United States of America

PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of interest to state DOT geotechnical, bridge, and structural engineers, engineering geologists, vibration monitoring consultants, contractors involved with pile driving, and researchers. It describes the current state of the practice for preventing and assessing damage to structures adjacent to pile driving installations. This was accomplished by conducting a literature search and review and an extensive survey of U.S. and Canadian transportation agencies and practitioners, as well as limited international information collection.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

This report of the Transportation Research Board presents information on the theory of ground motion and vibrations due to pile installations, vibration mitigation measures, and the consequences of groundborne vibrations. Additional detailed information on instrumentation used for vibration measurement and the management of vibrations is also included. The appendices include a bibliography, a primer on pile support mechanisms related to pile driving vibrations, several examples of existing agency vibration specifications and contract wording, and an example vibration specification from the literature.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the research in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

CONTENTS

ACKNOWLEDGMENTS

Richard D. Woods, Ph.D., P.E., Professor and Chairman, Department of Civil and Environmental Engineering, University of Michigan, was responsible for collection of the data and preparation of the report.

Valuable assistance in the preparation of this synthesis was provided by the Topic Panel, consisting of Donald J. Arcari, Associate Soils Engineer, New York State Department of Transportation, Geotechnical Engineering Bureau; Carlos Braceras, Chief Geotechnical Engineer, Utah Department of Transportation, Geotechnical Division; Michelle M. Cribbs, Geotechnical Engineer, Federal Highway Administration; Carl D. Ealy, Highway Research Engineer, Federal Highway Administration; Alfred J. Hendron, Ph.D., Department of Civil Engineering, University of Illinois at Urbana; G.P. Jayaprakash, Engineer of Soils, Geology, and Foundations, Transportation Research Board; Paul D. Passe, State Geotechnical Engineering, Florida Department of Transportation; John J. Resetar, Area Manager/Vibration Consultant, Vibra-Tech; and Clifford J. Roblee, Associate M&R Engineer, California Department of Transportation, Office of Research.

This study was managed by Stephen F. Maher, P.E., Senior Program Officer, who worked with the consultant, the Topic Panel, and the Project 20-5 Committee in the development and review of the report. Assistance in Topic Panel selection and project scope development was provided by Sally D. Liff, Senior Program Officer. Linda S. Mason was responsible for editing and production.

Crawford F. Jencks, Manager, National Cooperative Highway Research Program, assisted the NCHRP 20-5 staff and the Topic Panel.

Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance are appreciated.

DYNAMIC EFFECTS OF PILE INSTALLATIONS ON ADJACENT STRUCTURES

SUMMARY

It is known that pile driving creates vibrations in the ground and that, occasionally, those vibrations can damage structures or disturb people or other activities in the vicinity of the pile driving. It is also known that human perception of vibration is not an accurate gauge of the damage potential of the vibration. Although careful planning and execution of pile driving can avoid actual physical damage from vibrations, it is necessary to accommodate the most sensitive neighbor at a construction project if damage claims are to be avoided.

Pile driving vibrations (and all construction vibrations, for that matter) present a two-pronged hazard: first, potential for *real damage* due to the construction activity, and second, potential for *litigation* based on human perception. There are ways of mitigating both aspects of the problem of vibrations from pile driving, and these are presented in this synthesis.

The transmission of vibrations through the ground is well enough understood that the principal parameters that influence the potential for vibration damage can be identified and, in most cases, controlled. Furthermore, knowledge of the pile driving operation is sufficient that the principal parameters can be identified and usually controlled through selection of the driving details (hammers, piles, or both).

The first hazard from pile driving vibrations, *real damage*, can be mitigated in most cases with sufficient prior planning and precautions. Real damage usually takes the form of structural damage, including cracking and breaking of structural elements or ground settlement. At some sites structural damage from pile driving can be minimized by selecting the appropriate pile installation technique, that is, impact driving (appropriate hammer section for pile type using impedance approach); switching from impact driving to vibratory driving; or changing pile type to auger cast, thereby avoiding driving altogether. Experience has shown, however, that direct damage to structures is not likely to occur at a distance from the driven pile of *(a)* more than 15 m for piles 15 m long or less, or *(b)* one pile length for piles longer than 15 m.

In some situations, it is necessary to perform detailed site characterization to understand fully the conditions and prevent damage due to settlement. This is particularly true of a site where liquefaction or shakedown settlement of loose sands may occur. Vibration-generated settlement may be a problem at much greater distances from the pile driving than is direct damage, as described previously. In the extreme, distances as great as 400 m (0.25 mi) may need to be surveyed to identify settlement damage hazards. Furthermore, the cumulative effects of shakedown settlement may not become evident until many piles (in some cases 200 or more) have been driven at a site. Prevention of this type of damage requires a site investigation with complete characterization of the granular soils in terms of grain size distribution, grain shape, and plasticity of the fines.

The second hazard, *litigation*, usually can be charged to human perception and can be avoided only by careful predriving surveys and education. In the past the distance to which damage surveys were performed were determined by applying an assumed attenuation rate to ground motion at the pile driving location and computing the distance required to attenuate the ground motion to a level of 50 mm/sec. To minimize complaints due to vibrations, damage surveys should extend further than this approach would predict. Again, to avoid claims, distances of as much as 400 m (0.25 mi) may need to be surveyed and people

within this distance warned. Education in terms of public hearings and written materials can avoid some claims.

Vibration amplitudes as small as 24×10^{-6} mm may be damaging to very sensitive functions (electron microscopes, for example) near a pile driving operation. Sites must be screened to determine the existence of sensitive functions including research labs and hospitals. On the other extreme, many structures can sustain large vibrations, up to 200 mm/sec, without damage. In most cases it is necessary to monitor the actual magnitude of vibration to avoid damage. This is done using readily available instruments—geophones or seismographs. Data from these instruments can be compared with vibrations standards for humans and structures as established by agencies such as state departments of transportation (DOTs), municipalities, other governmental agencies, or standards agencies such as the American National Standards Institute (ANSI) and the International Standards Organization (ISO).

The results of a survey of state DOTs, pile driving contractors, and engineering consultants on their experiences with pile driving vibrations provided insight into the extent of pile driving vibration problems and the variety of ways in which the problem is addressed. About half of the respondents to the survey indicated experience with pile driving vibration problems, relating to either actual damage or litigation arising from pile driving. However, the survey indicated little uniformity in the way DOTs respond to this challenge. The principal means of mitigating vibration problems, as reported by the state DOTs, pile driving contractors, and engineering consultants were *(a)* changing pile driving equipment, *(b)* switching to drilled shaft piles, *(c)* jetting or partial jetting of piles into place, *(d)* switching to vibratory driving, and *(e)* scheduling pile driving to specially selected hours for the specific site to minimize the disturbance to the neighborhood.

The consensus for the best way to manage pile driving vibrations is to implement a well-conceived pile driving specification that includes *(a)* vibration limits for structures and human activities, *(b)* a means of assessing predriving conditions, *(c)* methods of vibration monitoring, *(d)* informational activities, *(e)* ground elevation surveys for settlement observation, and *(f)* a standard procedure for achieving records.

As for the problem of pile driving vibration, the following conclusions can be drawn:

- Pile driving vibrations can be a problem with all kinds of pile drivers and all kinds of driven piles. The main controlling factor is the amount of energy that is coupled into the ground. Detailed knowledge of the soil profile at and near the pile, as well as the proximity of adjacent surface and buried structures is essential to avoid damage.
- There are few cases of direct damage to structures at distances from the pile of greater than the pile length; however, settlement damage to surface and buried structures may occur up to 400 m (0.25 mi) from the pile driving. To avoid settlement damage, the ground conditions must be well known in advance so that planning of reduced vibration driving is possible.
- For impact driving, the selection of pile driver should be based on the *impedance* of the pile. For vibratory pile driving, variable-frequency drivers are necessary and variable-force vibrators are highly desirable.
- Although some pile driving specifications have prohibited driving piles in the vicinity of curing concrete, there is no documented evidence that justifies this provision.
- Predriving surveys must be performed to minimize claims and litigation, and public hearings should be held in which more than a few neighbors are involved. All pile driving should be performed under a vibration specification. An example specification containing many of the appropriate provisions is included in Appendix E. This specification is presented as an aid to agencies that are developing their own specifications. Individual agencies may need to modify the specification to match the needs of their monitoring programs.

CHAPTER ONE

INTRODUCTION

The process of driving piles into the ground for any purpose usually causes the ground surrounding the pile to shake. This shaking is more or less intense depending on the way the pile is inserted into the ground, the physical properties of the pile (material, weight, length, size, etc.), and the soil type (classification, void ratio, cementing, water content, etc.). These vibrations may cause damage to surrounding structures or settlement of the soil, depending on the intensity of ground shaking or vibration. It is important to understand the conditions under which the vibrations will cause damage and those under which vibrations are benign (nondamaging).

Piles are used to support nearly all major structures in some geographic areas of the United States and a large fraction of highway bridges throughout the country. Sheetpiles and soldier piles are also widely used to retain earth or water (or both) in highway and bridge construction. Piles are used to some degree in most urban areas either as permanent support of structures or as temporary elements during construction. In urban settings, neighboring properties are particularly vulnerable to ground and structure shaking due to pile driving because of the proximity of structures. Even in remote locations, vibrations from pile driving may cause damage to new and preexisting highway facilities, so pile driving vibrations must be understood.

It is of great importance to know the likelihood of damage from pile installation (or in some cases pile removal) procedures to minimize costs associated with damage due to vibration. This synthesis reports the current state of knowledge and practice in judging the potential for damage and determining the practice of agencies that must assess and prevent damage due to pile driving vibrations.

To initiate the synthesis study, a literature search was conducted to determine the state of documented experience and research associated with this problem. Results of the search are included throughout this synthesis. The most readily accessible literature is in journals, proceedings, and some books, but a mass of data exists in transportation agency files, individual consultants' files, corporate files, contractors' files, and other places not easily accessible to outsiders. Unfortunately, this latter form of data exists for many engineering problems but is seldom brought into the light. The questionnaire associated with this study and described in the following was partly an attempt to gain access to the contents of some of those files. Materials from the conventional literature sources that have been cited in this synthesis are included in the reference list, those that have not been specifically cited are included in a bibliography following the references.

To determine, in part, the state of practice, a questionnaire was developed to ask agencies about their experience with respect to damage from pile driving and any provisions that they use to reduce or prevent damage from pile driving. That questionnaire was sent to all state departments of transportation (DOTs), and similar agencies for the provinces of Canada, and to some city and county transportation agencies. Additionally, questionnaires were sent to a select group of piling contractors, vibration consultants, and vibration monitoring firms. Details of this questionnaire and the specific results thereof are presented in Appendices B through D; a summary of the results forms the contents of chapter 7.

To help some readers better understand the potential for damage caused by vibration from pile driving operations and appreciate some of the precautionary measures used or suggested, several aspects of the pile driving process are described in chapters 2 through 4, and a short tutorial on piles in general and wave propagation in the earth is presented in Appendix A. The tutorial provides only the basics, and readers are referred to the references for details. Many readers can ignore Appendix A and read the body of the report straight through.

CURRENT PRACTICE

Many entities, including DOTs, contractors, and other owners, are addressing the problems associated with vibrations from pile driving with some common or standard practices. The most common practice is to include vibration criteria in the pile driving specifications. Doing this gives the contractor guidelines under which all pile driving operations must be performed. The criteria may relate to structural damage or interference with human or operational activities (such as hospital operating rooms or research labs).

To ensure contractor conformance, it is common to specify predriving surveys of nearby structures and monitoring of vibrations during driving. Both precautionary activities are performed by the contractor, the DOT or other owner entity, or by a third party (who is not part of the contractor's firm, or the DOT, or employed by owners of potentially vulnerable structures).

In situations in which DOTs or other owners know of problems specific to a particular site or region, the agency may suggest or specify pile driving methods or equipment to use (predrilling, for example) and specify an offset distance from certain potentially vulnerable structures. The agency also may specify certain hours during which pile driving may be performed—for example, avoiding nighttime operations or pile driving during other critical periods of the day.

The choice of vibration control and mitigation measures depends on a knowledge of wave propagation in the earth and on the mechanics of the pile driving operation. This knowledge will be developed in the following chapters and appendices.

CHAPTER TWO

MECHANICS OF GROUND MOTION

The energy applied to piles for the purpose of driving them into the ground is used up partly in losses due to the mechanics of coupling of energy from the driver into the pile, partly the process of pile penetration, and partly by transmission (or radiation) of energy away from the pile through stress waves in the ground. The stress waves travel outward from the pile and may or may not cause damage depending on their intensity. The complete description of stress waves created by driving piles can be quite complicated and can be best understood by examining the various components separately. The basic components are included in this chapter; more details are included in Appendix A.

Stress waves in the ground, sometimes called seismic waves or sound waves, result from energy introduced in the ground in the form of time-varying stresses. For example, when the pile hammer hits the pile, energy travels down the pile, losing some energy due to friction (shear) along the shaft and some due to compression by penetration at the tip. All of this takes place in a very short time, during which the energy of the blow on the pile causes a stress wave to propagate in the surrounding ground. These traveling stresses are superimposed on the static stresses already in the ground from geostatic conditions and from the static loads of buildings or bridges or other structures located nearby.

VIBRATORY MOTION

The simplest form of vibratory motion is represented by sinusoidal or harmonic motion. This motion can be expressed mathematically for vertical vibration as:

$$z = A_m \sin \omega t \tag{1}$$

where

z = vertical displacement
A_m = displacement amplitude
ω = circular frequency (rad/sec)
t = time.

This equation is plotted in Figure 1. In Equation 1 and on Figure 1, A_m represents the displacement amplitude from the mean position and is often called single amplitude, single peak, or peak amplitude. Sometimes double amplitude or peak-to-peak amplitude is used, and that is represented on Figure 1 by $2A_m$. Circular frequency, ω, is the rate of oscillation in terms of radians per second, and it can be related to frequency in cycles per second (Hertz) by

$$f = \omega/2\pi \tag{2}$$

Also in Figure 1, the time between crests or troughs on this wave is the period of the wave, T, and the inverse of the period is the frequency, f.

TRAVELING WAVES

The nature of traveling waves can be deduced from the theory of elasticity as demonstrated by Richart et al. *(1)* and others. The mathematical development of these waves will not be presented here; however, the character of these waves and their consequences will be described. The easiest ground conditions to model consist of uniform soil or rock deposits that do not have layering. (Please note that "ground" will be used to denote both soil and rock and combinations thereof.) In this ground the speed or velocity of stress waves depends on the unit weight and moduli (Young's modulus and shear modulus) of the ground. For most rocks and many clays, the moduli are not greatly influenced by geostatic stress (weight of

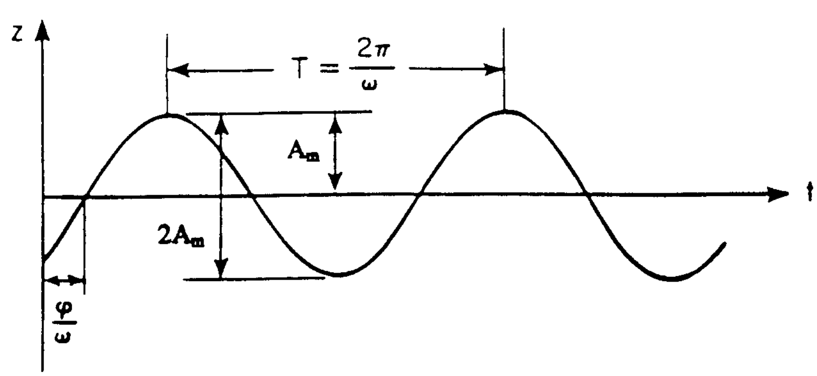


FIGURE 1 Quantitites describing harmonic motion *(1)*.

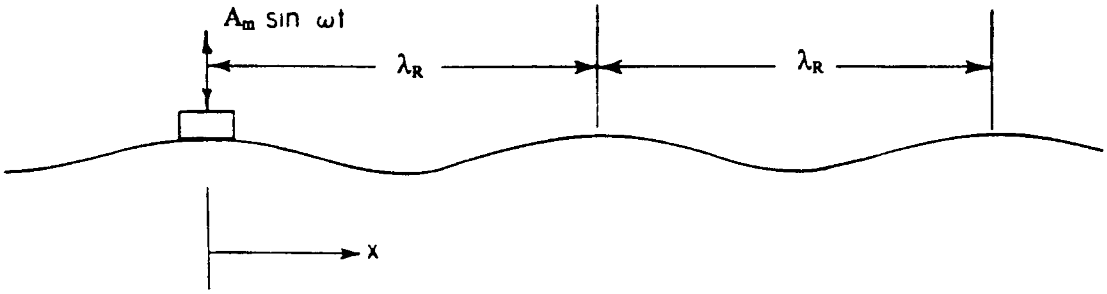


FIGURE 2 Deformed shape of ground surface (*1*).

material above) and can be considered as nearly uniform with depth. This is the simplest ground condition and results in the fewest potential stress waves.

In uniform ground (technically described as homogeneous, isotropic, and linearly elastic ground), two basic types of waves are generated: body waves and surface waves. The body waves are also of two types as long as the ground is not saturated: primary and secondary. These waves are often referred to as compression waves and shear waves, respectively. The primary wave is not precisely a compression wave, but for purposes of a general understanding of wave propagation in the ground around driven piles, this description is adequate. In this ideal uniform ground, there is only one kind of surface wave, Rayleigh wave.

The deformed surface of the ground associated with any of the seismic waves can also be idealized as a sinusoidal wave or harmonic wave, as shown in Figure 2 for the Rayleigh wave. The peak-to-peak amplitude is shown here as well as the wavelength, λ_R, (distance between successive crests or troughs of the wave). The velocity or speed of a traveling wave (V_R for example) can be deduced from frequency, f, and wavelength, λ_R, as

$$V_R = \lambda_R/T = \lambda_R/(\omega/2\pi) = \lambda_R(2\pi f)/(2\pi) = \lambda_R f \qquad (3)$$

Three important features of all stress waves help distinguish them from one another: wave velocity (speed), wave direction, and particle motion. Wave velocities are controlled by the elastic properties of the ground and can be expressed mathematically for primary wave, V_P, and secondary wave, V_S, as

$$V_P = \sqrt{(\lambda + 2\mu)/\rho} \qquad (4)$$

$$V_S = \sqrt{(\mu/\rho)} \qquad (5)$$

V_R = complex function dependent on E, λ, μ, and ν

(equation not shown)

where

λ, μ = Lamé's constants (μ = also known as shear modulus),
E = Young's modulus,
ν = Poisson's ratio, and
ρ = mass density of ground (ρ = unit weight/g).

All of the elastic wave velocities can be related through the previous constants, and a graphical relationship between V_P, V_S, V_R, and ν is shown in Figure 3, in which the ratios of V_P/V_S and V_R/V_S are plotted against ν.

The important deformation characteristics of the two basic body waves and the Rayleigh wave are shown in Figure 4. In the primary wave (P-wave), the motion of a minute particle of material (could be envisioned as a single grain of sand) is to-and-fro in the direction of wave travel. In the secondary wave (S-wave), particle motion is in the plane perpendicular to the direction of wave travel; and in the Rayleigh wave, it is a complex combination of vertical and horizontal motion depending on the depth below the ground surface and Poisson's ratio.

TIME DERIVATIVES OF DISPLACEMENT

Equation 1 can be used as a mathematical representation of the time dependency of particle displacement amplitude associated with a seismic wave. This equation can be differentiated with respect to time to obtain

$$\dot{z} = A_m\omega \cos \omega t \qquad (6)$$

where $\dot{z}$ is the particle velocity error.

Equation 6 can be differentiated with respect to time to obtain

$$\ddot{z} = A_m\, \omega^2 \sin \omega t \qquad (7)$$

where $\ddot{z}$ is the particle acceleration error.

Because the amplitude (magnitude) of a sinusoidal or harmonic motion does not depend on the sine or cosine, simple relationships can be used to relate displacement, velocity (z), and acceleration (z) amplitudes using only A_m and ω, that is,

$$z = A_m$$
$$\dot{z} = \omega A_m$$
$$\ddot{z} = \omega^2 A_m \qquad (8)$$

Calculations using these relationships can be used for all combinations. For example,

$$A_m = \dot{z}/\omega = z$$

or

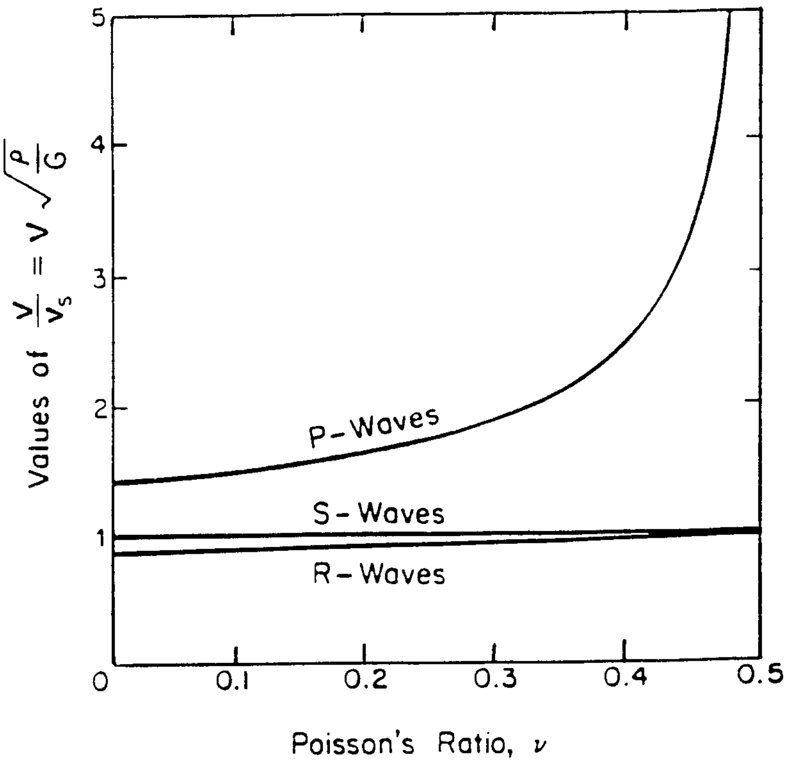


FIGURE 3 Relation between Poisson's ratio, ν, and velocities of propagation of compression (P), shear (S), and Rayleigh (R) waves in a semi-infinite elastic medium *(2)*.

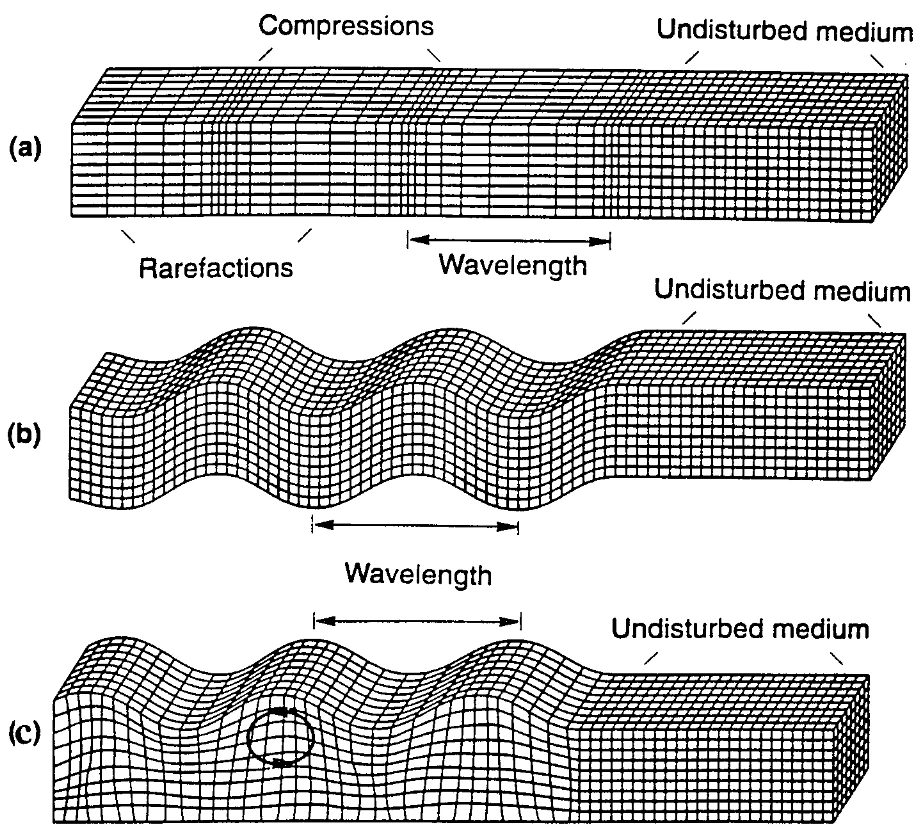


FIGURE 4 Displacement characteristics of body waves—*(a)* primary wave *(b)* secondary wave—and *(c)* surface wave (Rayleigh wave) *(3)*.

$$A_m = \ddot{z}/\omega^2 = z$$

The significance of the difference between wave velocity (V_P, V_S, V_R) and particle velocity ($\dot{z}$) cannot be overemphasized. Wave velocity refers to the speed at which a seismic wave travels through the ground. Particle velocity refers to the speed at which an individual grain oscillates about an "at-rest" position.

A simple way of visualizing the difference between wave velocity and particle velocity is to consider a fishing bobber floating on the water. If a water wave comes by, it is traveling horizontally at wave velocity, but the bobber will float up and down at *particle* velocity and does not travel along with the wave at *wave* velocity.

Particle velocity is often used to characterize wave motion because the simplest instruments (velocity transducers or geophones) can be used to measure particle velocity. More complex, therefore more expensive and sensitive, instruments are necessary to measure displacement or acceleration.

ATTENUATION (OR DECAY) OF SEISMIC WAVES

Seismic waves in real situations are often created by finite sources such as a pile being driven, or a footing or a pile supporting vibrating machines. As the waves travel outward from the source, they encounter ever larger volumes of ground, resulting in a reduction of energy per unit volume in the ground. This phenomenon is known as *geometric* or *radiation damping;* it can be visualized by recalling the results of dropping a pebble in a still pool. The waves travel in circular ripples outward from the point where the pebble entered the water. As the waves get farther from the source, the height or amplitude of the waves decreases.

Each of the seismic waves described previously decays geometrically with distance according to separate mathematical rules, but all three can be described through an equation of the form

$$A_2 = A_1(r_1/r_2)^n \quad (9)$$

where

r_1 = distance from source to point of known amplitude,
r_2 = distance from source to point of unknown amplitude,
A_1 = amplitude of motion at distance r_1 from source,
A_2 = amplitude of motion at distance r_2 from source, and
n = power depending on type of wave:
n = 1/2 for Rayleigh waves,
n = 1 for body waves, and
n = 2 for body waves at the surface.

The ground itself has some damping capacity known as *material* or *hysteretic damping*; it can be combined with the previous geometric damping as suggested by Bornitz *(4)* and shown by Woods and Jedele *(5)*:

$$A_2 = A_1(r_1/r_2)^n \exp[-\alpha(r_2 - r_1)] \quad (10)$$

where exp is the base of natural logarithm and α is a coefficient of attenuation in units of 1/distance.

The value of α in Equation 10 depends on the character of the ground: softer materials generally have greater α-values, whereas harder materials have smaller α-values. Woods and Jedele *(5)* presented a proposed classification for coefficients of attenuation, α, for earth materials (Table 1). In Table 1 the coefficient of attenuation is frequency dependent—being linearly dependent on frequency. If α is known for one frequency, it can be computed for any other frequency by

$$\alpha_2 = \alpha_1 (f_2/f_1) \quad (11)$$

where α_1 is a known value at frequency f_1, and α_2 is an unknown value at frequency f_2.

Another attempt to model attenuation (pseudo-attenuation) was presented by Wiss *(6)*. Wiss obtained the best fit of field data in an equation of the form

$$v = kD^{-n} \quad (12)$$

where

v = peak particle velocity of seismic wave,
k = value of velocity at one unit of distance,
D = distance from vibration source, and
n = slope or attenuation rate.

TABLE 1
PROPOSED CLASSIFICATION OF EARTH MATERIALS BY ATTENUATION COEFFICIENT *(5)*

Class	Attenuation Coefficient α (1/m) 5 Hz	Description of Material
I	0.01 to 0.033	*Weak or Soft Soils*—lossy soils, dry or partially saturated peat and muck, mud, loose beach sand, and dune sand, recently plowed ground, soft spongy forest or jungle floor, organic soils, topsoil. (shovel penetrates easily) (N < 5)
II	0.0033 to 0.01	*Competent Soils*—most sands, sandy clays, silty clays, gravel, silts, weathered rock. (can dig with shovel) (5 < N < 15)
III	0.00033 to 0.0033	*Hard Soils*—dense compacted sand, dry consolidated clay, consolidated glacial till, some exposed rock. (cannot dig with shovel, need pick to break up) (15 < N < 50)
IV	< 0.00033	*Hard, Competent Rock*—bedrock, freshly exposed hard rock. (difficult to break with hammer) (N > 50)

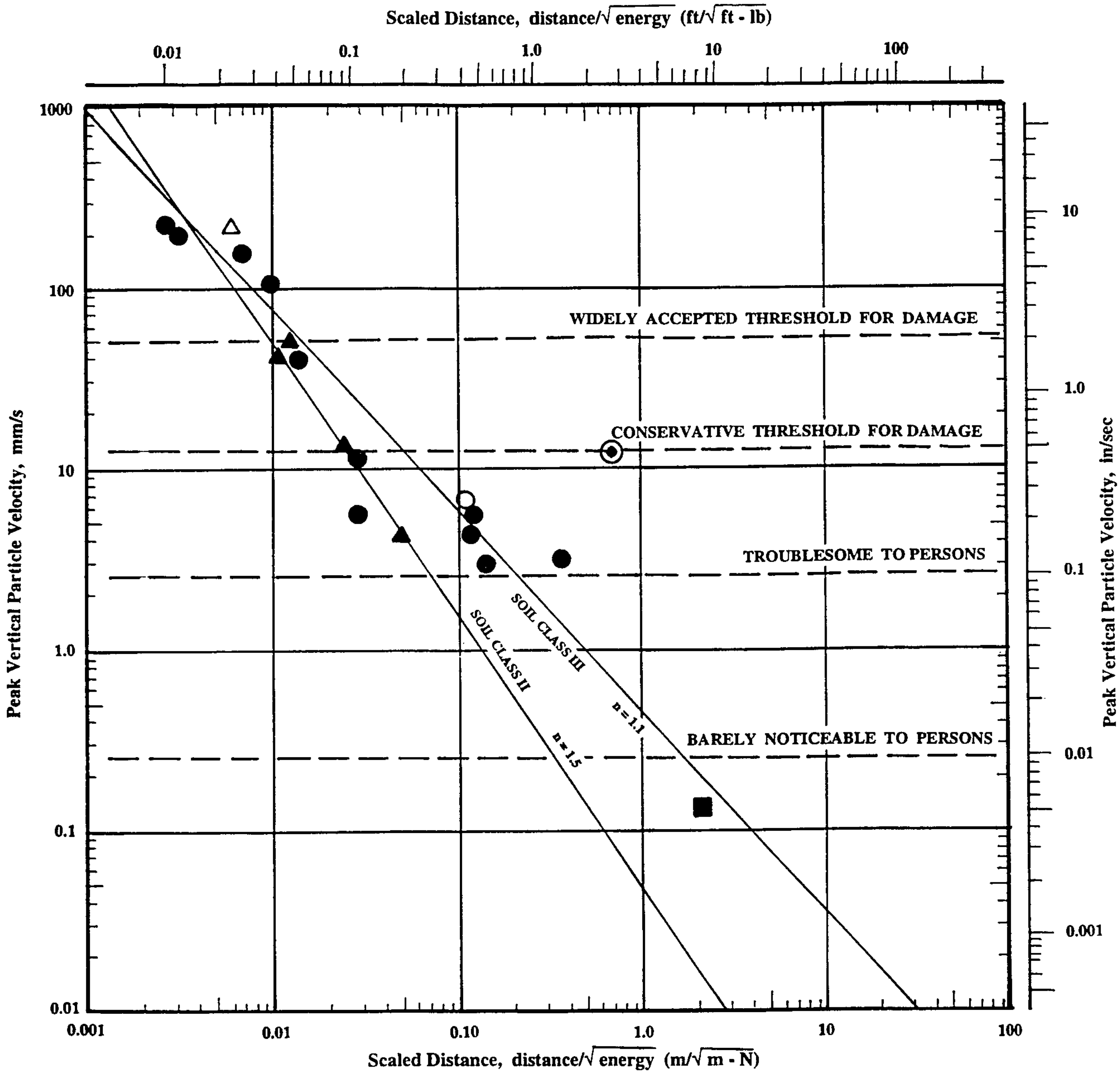


FIGURE 5 Peak vertical particle velocity versus scaled distance *(5)*.

The n rate is not classical attenuation as used in the Bornitz equation (Equation 10), but it may be considered as a pseudo-attenuation coefficient.

Wiss *(6)* also suggested an expression to include source energy in an attenuation equation, a so-called *scaled-distance* equation as follows:

$$v = k\,[D/\sqrt{E_n}]^{-n} \qquad (13)$$

where E_n is the energy of source.

Woods and Jedele *(5)* gathered data from field construction projects at which the vibratory energy was known or could be estimated and developed a scaled distance chart that correlates with ground types, (Table 1) and vibration criteria, (Figure 5). The ground type is defined by the n-term in Equation 13 and is represented by the sloping lines with n = 1.5, Class II soils from Table 2-1, and n = 1.1 representing the Class III soils. Classes I and IV ground were not included in the data base because measurements were not made in these ground types.

For a source of known energy content, the peak vertical particle velocity can be predicted at any distance by entering Figure 5 from the bottom with "scaled distance" (distance/√energy) projecting upward to the line representing the soil class through

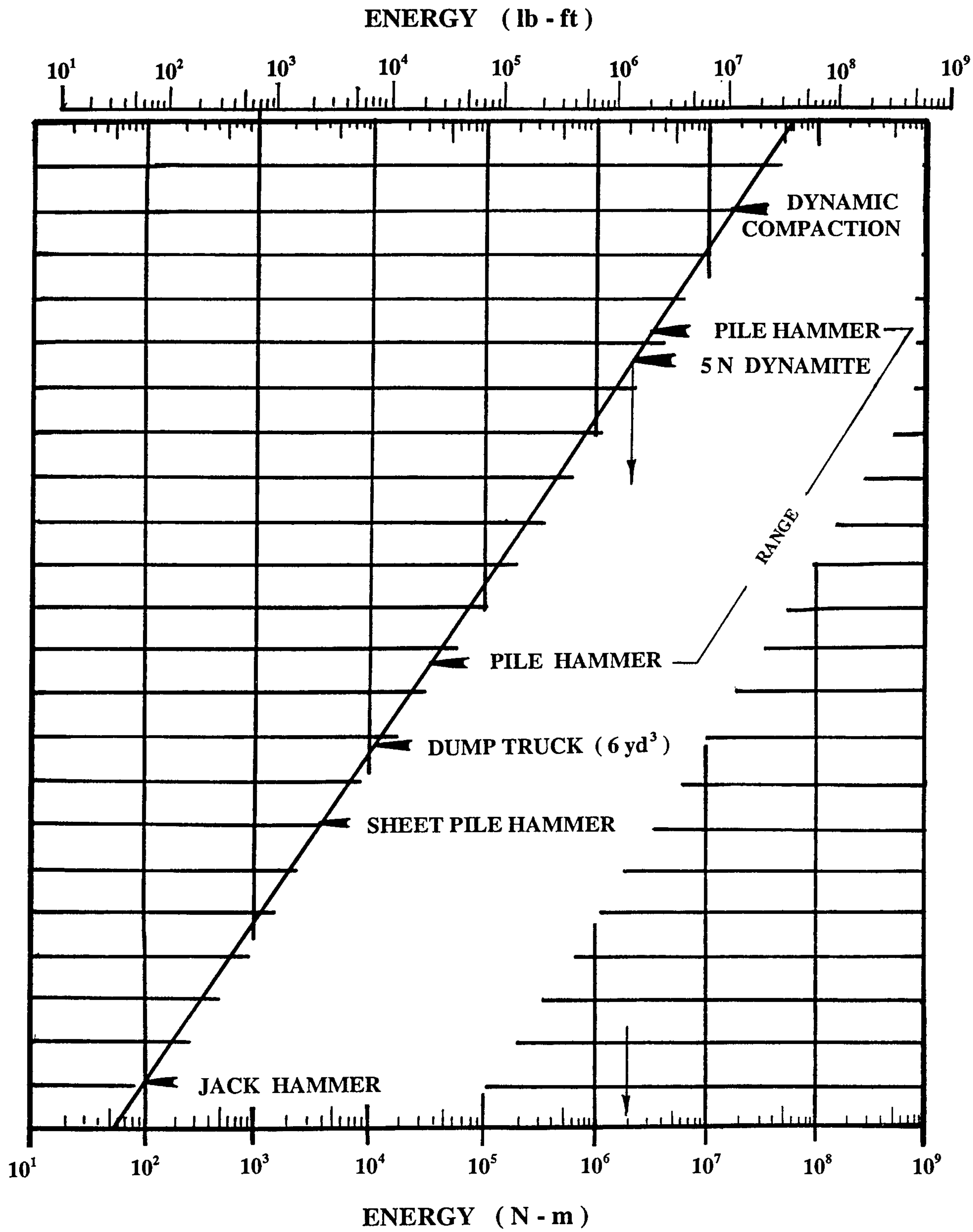


FIGURE 6 Relative energy of sources.

which the energy is being transmitted (distinguished by n), and then translating horizontally to the peak vertical particle velocity value. As is demonstrated in Appendix A, this representation for attenuation is appropriate over a wide range of energy sources from a jackhammer to a large dynamic-compaction, drop hammer (Figure 6).

Although it is demonstrated in Appendix A that the Bornitz equation (Equation 10) is the best representation of attenuation in earth materials, the pseudoattenuation approach presented in Figure 5 is satisfactory for analyzing pile driving vibration. The main reason is the simplicity and accuracy of the pseudoattenuation approach in the short distance range. The pseudoattenuation approach does not represent material damping at large distances very well, but for pile driving vibrations, it is the near field that is of concern; therefore, this approach is satisfactory.

Figure 5 also shows damage levels related to peak vertical particle velocity as might be expected from pile driving activities. These levels of damage will be discussed in chapter 3.

Heckman and Hagerty *(7)* also developed an equation relating pile driving energy to the distance from source to a target structure. This equation, based on work by Wiss *(6)*, is much like that proposed by Woods and Jedele *(5)* but includes a K-factor related to the pile impedance as shown in Figure 7:

$$z \cong K\sqrt{E_n} \div D \tag{14}$$

where

z = peak particle velocity (mm/sec),
K = factor dependent on pile impedance,
E_n = energy of blow, and
D = distance from source.

In Figure 7, impedance I is computed as

$$I = \rho V_p A_p \tag{15}$$

where

I = impedance,
V_p = P-wave velocity in pile, and
A_p = cross-sectional area of pile.

This relationship and Figure 7 will be cited later in analyzing pile driving by impact. The values of K in Equation 14 were developed from eight pile-type versus hammer energy combinations.

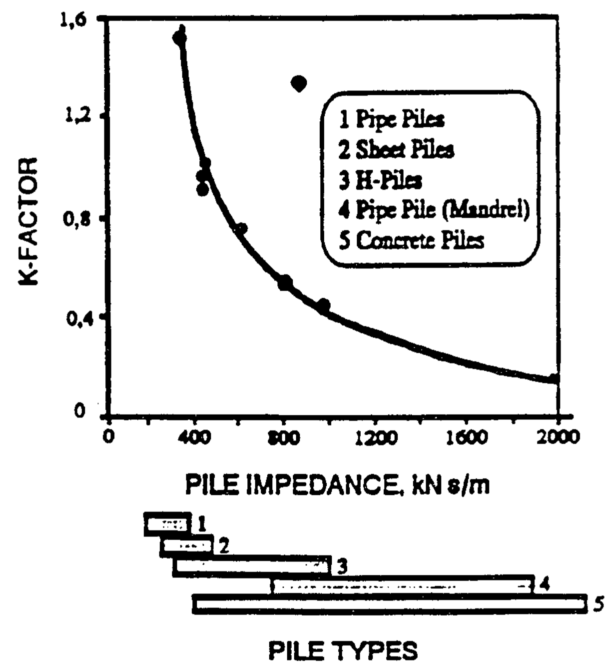


FIGURE 7 Influence of pile impedance on transmission of vibration energy from pile to soil *(7)*.

CHAPTER THREE

VIBRATIONS DUE TO PILE INSTALLATION

Energy that is produced by pile driving and coupled into the ground travels through the ground from the pile to structures within and on top of the ground by seismic or stress waves as described in chapter 2. The amplitude of this energy depends on many factors, some of which have been discussed already (such as the soil type or classification). The energy propagating away from the driven pile is also dependent on the pile driver and the pile itself. In this chapter the mechanisms of vibration generation at the pile will be described for impact- and vibratory-driven piles.

IMPACT-DRIVEN PILES

Just as the support of piles comes about through two mechanisms—skin friction and end bearing—seismic waves are generated by piles through the same two mechanisms. Shear waves are generated along the surface or skin of the pile by relative motion between the pile and the surrounding soil or by elastic deformation of the soil in contact with the pile as the pile is driven. Shear stresses enter the soil first near the upper contact point between soil and pile and, as the impact stress in the pile travels down the pile, the shear waves propagate out from the pile on a conical wave front (Figure 8). The cone angle is quite shallow because the velocity of the driving impulse traveling down the pile at the compression wave velocity in the pile is usually 10 or more times the shear wave velocity in the soil; so, for practical purposes, the wave front emanating from the pile can be assumed to be cylindrical.

The wave front from skin friction encounters cylinders of ever-increasing size, so the energy density along the front decreases as the square root of the distance from the pile. The surface of the cone or cylinder is known as the wave front because it is the leading edge of increase in stress caused by the interaction between the skin of the pile and the soil. The direction of wave travel is perpendicular to the wave front—in other words, radially away from the pile—for a cylindrical wave front. Particle motion in this wave front is parallel to the pile as suggested by the arrows on the wave front in Figure 8. A line perpendicular to the wave front is known as a ray. Often it is easier to trace a wave through complex strata by following a ray rather than a wave front. Both rays and wave fronts are shown in Figure 8.

At the tip of the pile, each impact causes a volume displacement, which results in both primary waves and shear waves traveling outward from the cavity (here idealized as a spherical cavity) (Figure 9). Both P-waves and S-waves travel outward from the tip of the pile on spherical wave fronts, decaying as the first power of distance. The P-wave travels faster than the S-wave, so its wave front precedes the shear wave at any given point in the ground. When the P-wave and S-wave

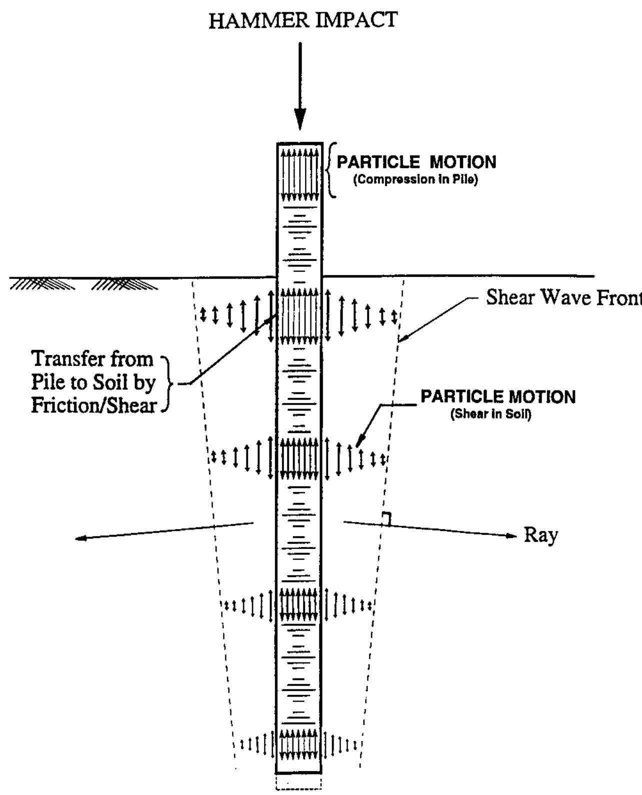


FIGURE 8 Ground waves from pile-soil shear.

encounter the surface of the ground, part of their energy is converted to surface waves (Rayleigh waves, R-waves) and part is reflected back into the ground as reflected P- and S-waves. The newly created R-wave then travels along the surface with the characteristic of Rayleigh waves, so some distant surface locations will "see" three waves: P-wave, S-wave, and R-wave. The amplitude of energy associated with each wave will depend on many factors, including the depth of the pile into the ground, hardness of the ground, uniformity of the ground, and energy delivered to the pile. It is these waves that transmit energy to the ground surrounding a pile that are potentially damaging to neighboring structures and annoying to people.

The basic mechanisms of stress wave generation presented in Figures 8 and 9 are shown for a uniform soil profile. If the ground is layered or otherwise nonuniform, the wave propagation becomes more complex. Each time a P-wave or S-wave encounters a boundary between ground of different properties, two reflected waves and two refracted waves will be generated for both of the original P- and S-waves (i.e. eight resulting waves from two incident waves). The direction and amplitude of each wave can be calculated from the Zoeppritz or Knott equations presented by Richart et al. *(1)*. Figure 10 shows an example of those complexities.

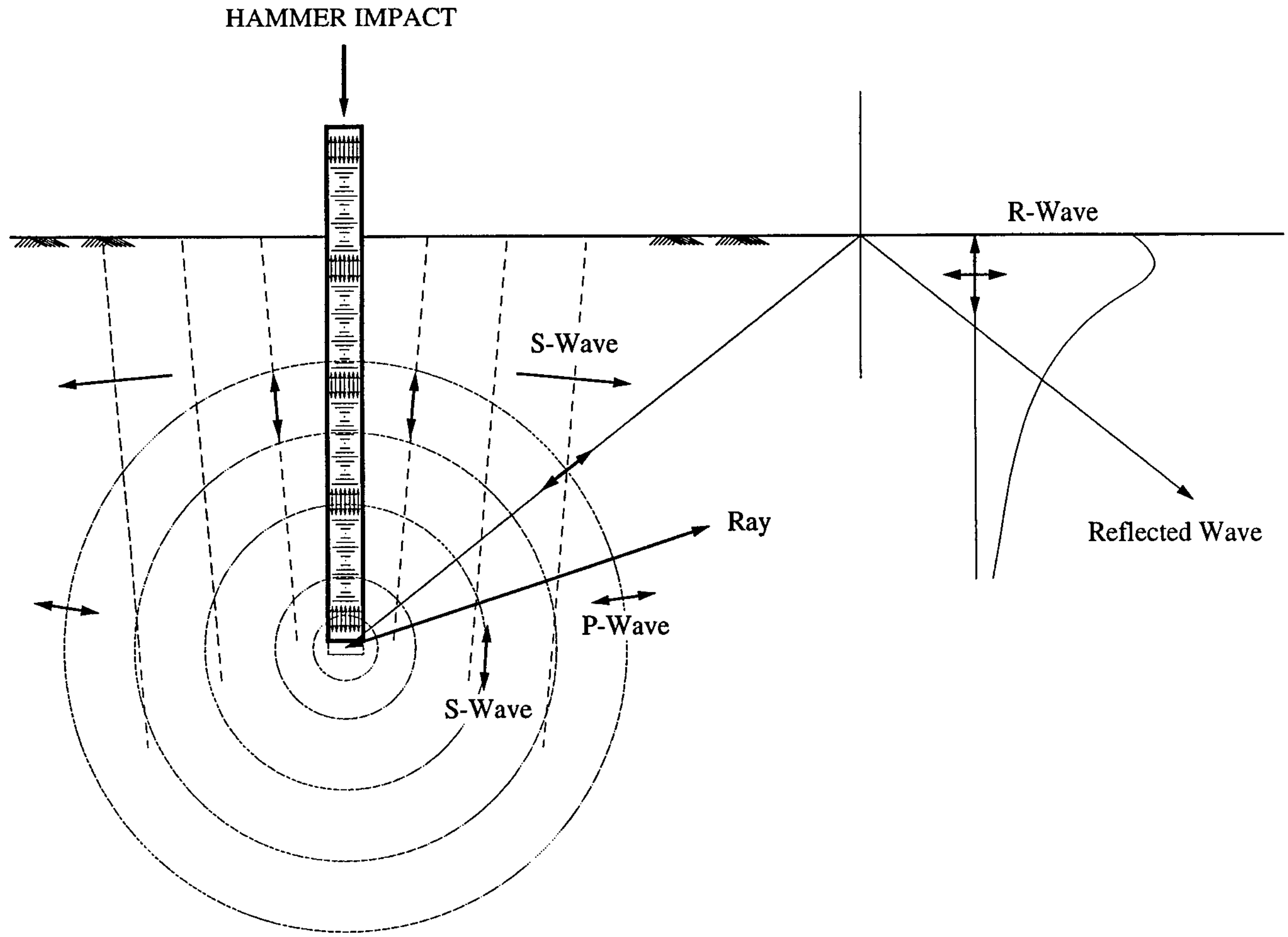


FIGURE 9 Composite of waves emanating from impact-driven hammer.

In most instances a point in the body of the ground—for example, Observation Point 1 in Figure 11—will experience two principal waves passing by from a pile blow, one from the P-wave and one from the S-wave. A wave trace is a representation of the particle motion due to the passage of a stress wave and is represented as an amplitude-versus-time plot (Figure 12). At a more distant surface point—for example, Observation Point 2 in Figure 11—the Rayleigh wave will have already formed and the wave form passing a surface point would have three arrivals, as shown in Figure 13.

These basic principles of elastic wave propagation near the surface of the ground provide some insight into the expectations of relative amplitude of vibrations from pile driving. For example, when a pile is driven starting at the ground surface, the first vibrations are propagated as Rayleigh waves. Rayleigh waves decay more slowly along the ground surface than other seismic waves and consequently, transmit damaging energy to greater distances than body waves. As the pile is driven deeper, the mechanisms of skin friction and end bearing dominate wave generation and body waves are produced, which decay more rapidly with distance. Therefore, for targets at the surface, vibrations may be more intense when the pile tip is at or near the surface than when the pile is deeper in the ground. This may suggest the use of preboring to minimize the transmission of wave energy near the ground surface. However, in the case in which the target is buried, such as pipeline or tunnel, the Rayleigh wave generated near the surface will not cause damage, but body waves generated by the pile tip when it is deeper may be damaging.

The ground motion generated by impact-driven piles is, surprisingly, less dependent on the soil (which is a secondary factor) into which the pile is being driven than on the pile material and dimensions. Energy is coupled into the pile by the impulse created by the ram of the driver at the top of the pile. The capability of the pile to transmit the force from this impact depends on the impedance *(I)* of the pile expressed as

$$I = \rho V_p A_p \tag{15}$$

where

ρ = mass density of pile material (unit weight/g),
V_p = compression wave velocity in pile, and
A_p = cross-sectional area of pile.

The force created by the impulse can be expressed as stress times area:

$$P = \sigma A_p$$

and stress can be expressed as

$$\sigma = \rho V_p \dot{z}$$

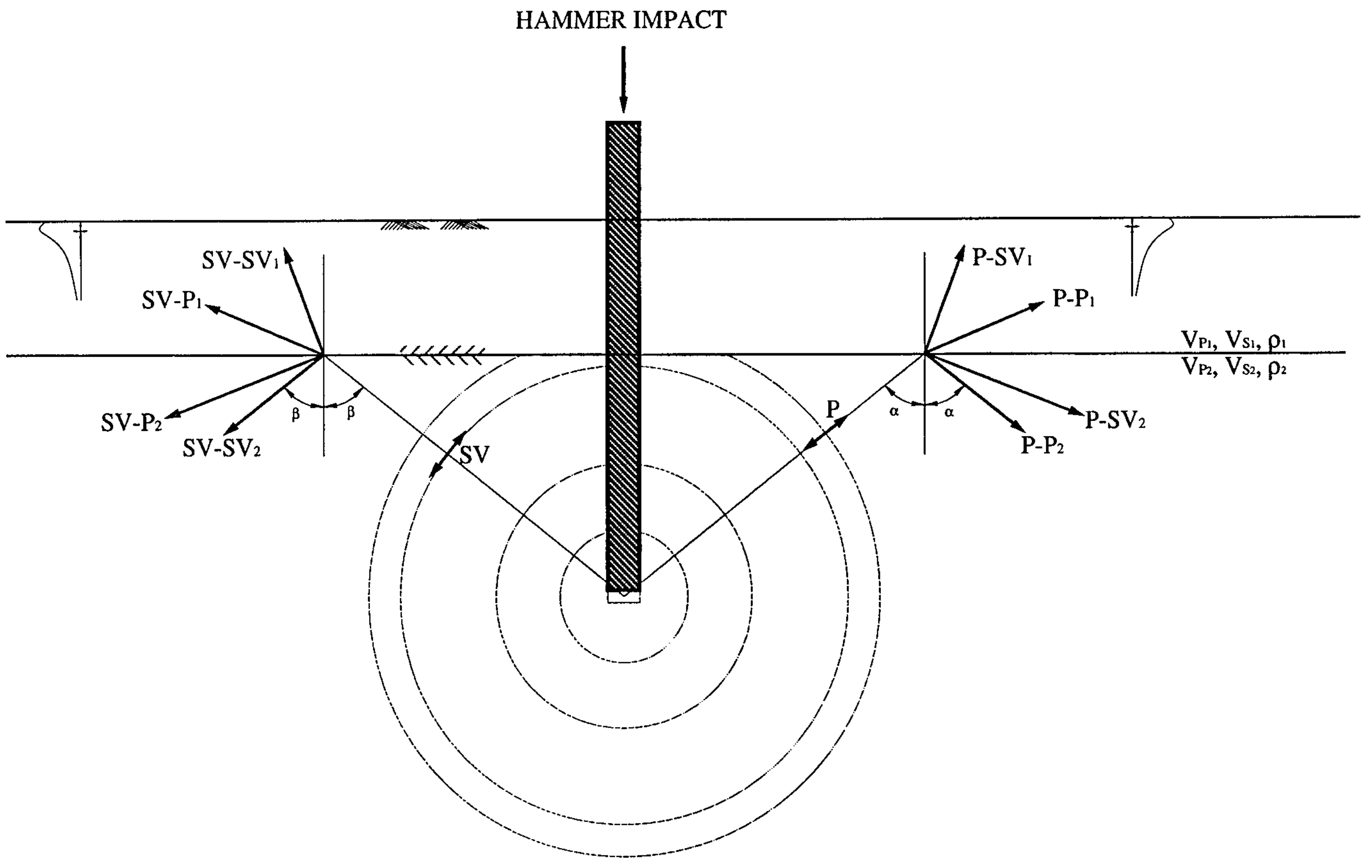


FIGURE 10 Partition of waves from impact-driven pile at soil layer boundary.

thus

$$P = \rho V_p A_p \dot{z} = I \dot{z} \tag{16}$$

where $\dot{z}$ is the particle velocity at the top of the pile.

If there were no cushioning at the top of the pile, $\dot{z}$ would be the velocity of the ram. For a free-falling ram, $\dot{z}$ can be calculated as

$$\dot{z} = \sqrt{2gh} \tag{17}$$

where g is the acceleration of gravity and h is the drop height of free-falling weight.

To drive the pile, the force at the top must overcome the resistance of the pile to penetration, but impedance limits the amount of force that can be transmitted along the pile, Equation 16. However, impedance is independent of the impulse applied at the top of the pile and independent of the soil into which the pile is being driven. Impedance is only a function of the pile material and dimensions. Peck et al. *(8)* give impedances for several types and sizes of piles, which are shown in Table 2. In this table it can be seen that piles with approximately the same exterior size but greatly differing material properties, particularly Young's modulus, have substantially different impedances.

The energy transmitted from the pile to the ground depends principally on hammer and pile properties. Heckman and Hagerty *(7)* showed the significance of pile impedance on the vibration energy coupled to the surrounding soil during driving. In evaluating ground-borne vibrations, they used the semi-empirical equation presented earlier as Equation 14:

$$\dot{z} \cong (K\sqrt{E_n}) \div D \tag{14}$$

The K factor was dependent on the impedance of the pile as shown in Figure 7. It can be inferred from Figure 7 that pile impedance must be considered in the evaluation of the transmission of vibrations away from impact-driven piles. At the same distance from a pile being driven at a common site, the range of possible pile impedances is about 5 while the range of K factor is about 10, as shown in Figure 7. The significant observation is that pile impedance influences the amount of vibration transmitted into the ground from a driven pile.

The engineer responsible for inspection of impact-driven piles should consider the following:

- Hammer energy (ram fall height or rated energy),
- Pile material (properties), and
- Pile dimensions (cross-section area and length).

Pile driving analysis programs account for these factors and are used routinely to prevent pile damage caused by use of hammers with excess energy.

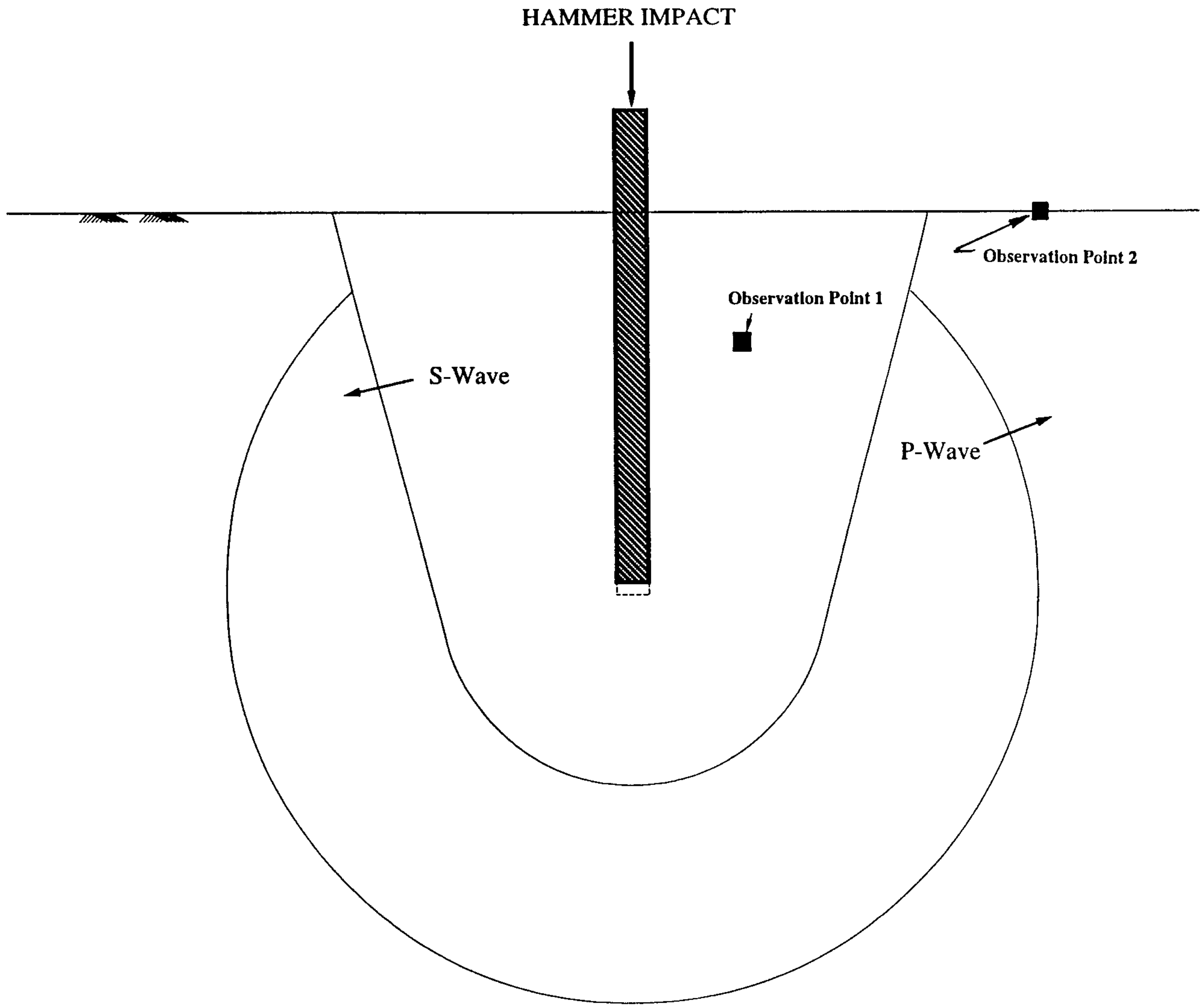


FIGURE 11 Observation points in wave field from impact-driven pile.

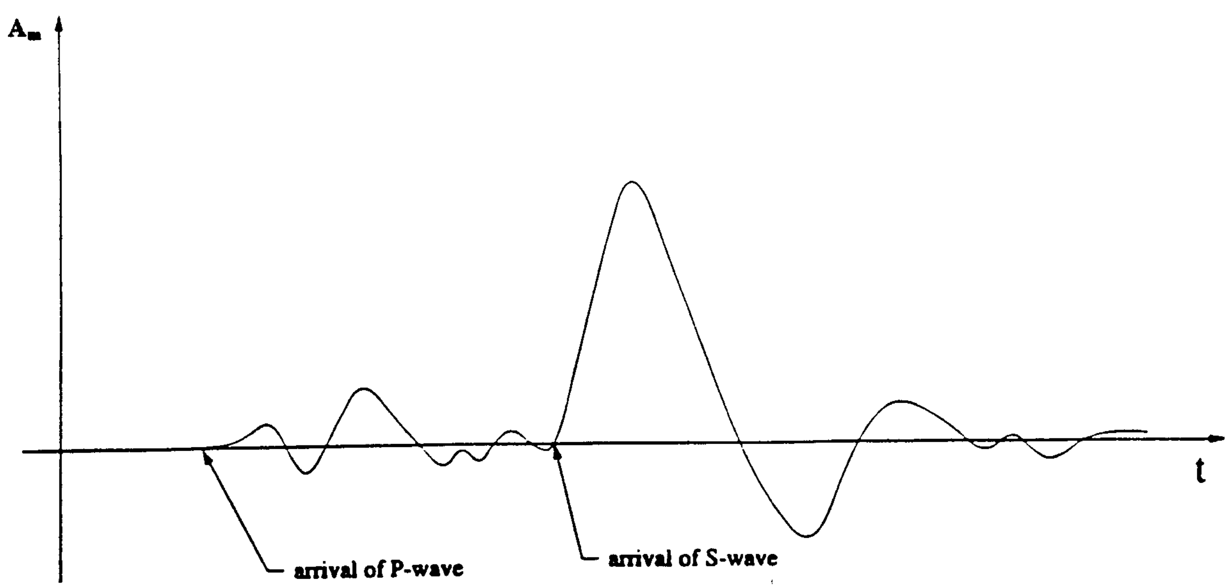


FIGURE 12 Wave form passing Observation Point 1 in wave field of impact-driven pile.

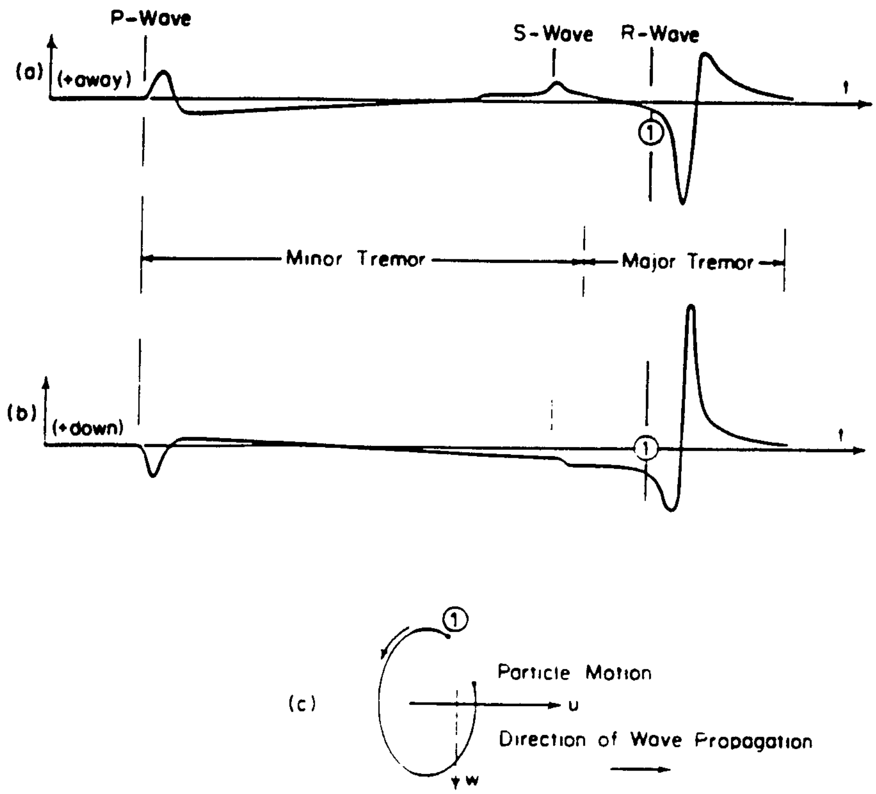


FIGURE 13 Wave system from surface point at Observation Point 2 in ideal ground *(1)*.

TABLE 2

STRESS-TRANSMISSION CHARACTERISTICS OF TYPICAL PILES *(8)*

	γp (lb/ft^3)	$\rho = \gamma p/g$ (lb sec^2/ft^4)	c (ft/sec)	ρc (lb sec/ft^3)	A (in.^{2})	ρcA (lb sec/in.)	Ratio[a]
Wood							
10-in. diameter kiln dry	40	1.24	13,600	16,900	78.5	768	1
10-in. diameter treated southern pine	60	1.86	10,600	19,750	78.5	898	1.2
Concrete	150	4.66	11,100	51,800			
10-in. diameter					78.5	2360	3.1
20-in. diameter					314.2	9410	12.3
Steel	490	15.2	16,900	257,000			
HP 10 × 57					16.76	2500	3.3
HP 12 × 53					15.58	2430	3.2
HP 14 × 117					34.44	5370	7.0
10-3/4 × 0.188 pipe					6.24	928	1.2
10-3/4 × 0.279 pipe					9.18	1440	1.9
10-3/4 × 0.365 pipe					11.91	1770	2.3
10-3/4 × 0.188 pipe[b]					53.30	7930	10.3
Steel/concrete							
10-3/4 × 0.279 pipe filled with concrete	185	5.76	12,100	69,800	87.9	3550	4.6

[a]Ratio of ρcA to that for 10-in. wood pile.
[b]With steel driving mandrel weighing 160 lb/ft.

VIBRATORY-DRIVEN PILES

It is helpful to understand some basic theory of vibratory pile driving to best apply this pile driving technique. Most piles are driven with counterrotating mass vibrators, whose basic components are shown in Figure 14 [from Richart et al. *(1)*]. These vibrators are nowadays usually driven by a hydraulic motor; which means that they have the potential to operate at a wide range of frequencies.

The static moment, M, of a rotating mass vibrator is the product of the mass (m) of the rotating elements times the radius of eccentricity (r_e),

$$M = m \times r_e \tag{18}$$

where

M = static moment of rotating mass vibrator,
m = mass of rotating elements of rotating mass vibrator, and
r_e = radius of eccentricity of rotating elements.

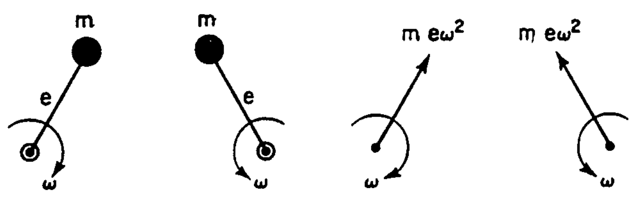


FIGURE 14 Forces produced by two counterrotating masses *(1)*.

The peak centrifugal force, Q (Q = rotating force vector), is a product of the static moment and the circular frequency squared, so

$$Q = M \times \omega^2$$
$$Q = m \times r_e \times \omega^2 \tag{19}$$

Thus, it is clear that centrifugal force is function of frequency, but static moment is not.

In vibratory pile driving another factor in evaluating performance is the displacement amplitude (double amplitude) generated by the rotating mass vibrator. Displacement for a free-hanging driver-pile system is determined from

$$A_m = 2M/m_t \tag{20}$$

where m_t is the total oscillating mass (mass of vibrator + clamp + pile).

Vibratory pile driving efficiency is generally proportional to the amplitude of displacement, A_m, so from Equation 20, the dynamic mass, m_t, should be minimized. Note again that the dynamic displacement is not a function of frequency.

When a pile is driven by a steady-state vibrator, the energy per cycle of vibration may be much less than the energy per blow from the driven-pile situation. In most instances this means the damage potential is less than for impulse-driven piles. There are exceptions, however, that may control the situation. It has been observed in practice that the ease or difficulty of vibratory pile driving depends on the characteristics of the soil as well as the characteristics of the pile and the vibratory pile driver. This is different than for impact-driven piles, for which the soil is not as important, and is true because two of the three potential resonances that occur during vibratory pile driving involve soil properties and layer thicknesses. For optimum efficiency, the pile and the soil should not be vibrating "in phase," that is, the pile and the driver must not move together with the surrounding soil, or no penetration occurs.

The following are the three distinct frequencies that affect vibratory pile driving:

1. Driver-pile resonant frequency, which results in maximum particle velocity at the pile top;
2. Soil-pile-driver system resonant frequency, which results in maximum displacement of the soil surrounding the pile; and
3. Soil stratum resonant frequency, which is a function of stratum thickness and properties.

The first is the optimum frequency for driving because the relative motion between the soil and the pile is maximum. The integration of this relative motion produces the pile penetration. The pile is most efficiently driven by vibrations when the combination of mass of driver and pile and the frequency of vibration combine to produce a driver-pile resonance. As long as the frequency of this resonance does not coincide with a resonant frequency of any nearby target or the stratum frequency of the site, large ground vibrations do not occur and damage is unlikely to happen.

Maximum ground vibration amplitude adjacent to the pile during vibratory driving will be encountered when the vibratory pile driver is operating at the resonant frequency of the soil-pile-driver system and this frequency is dependent on properties of the soil stratum that the pile is penetrating. The amplitude of motion under these conditions will also depend on the force generated by the vibratory driver, the mass of the system, and the stiffness of the soil. However, at this frequency the pile and soil are "in phase or are moving together" and all penetration stops. Also, the ground motion is maximum, exacerbating transmission of vibrations to the neighborhood.

The third resonance involves the soil stratum that the pile is currently penetrating and may change throughout the driving process. At this resonant frequency, the stratum resonates generating large ground motion that very efficiently transmits vibrations throughout the neighborhood. Soil layers or strata will selectively transmit and amplify specific frequencies depending on V_p, V_S, and strata thicknesses H. The resonant frequency of a soil layer can be estimated from Richart et al. *(1)* as

$$f = V/4\,H \tag{21}$$

where

f = frequency;
V = seismic velocity of layer, P-wave, or S-wave depending on motion observed; and
H = layer thickness.

A potential hazard with vibratory drivers exists in the form of matching the frequency of a soil layer. Vibrators often operate in the range of 20 to 30 Hz. For soils with shear wave velocities of 120 to 600 m/sec, this frequency range may spell a hazard for layers between about 1 and 5 m thick, not uncommon in nature. This phenomenon points out the advantage of providing a vibratory hammer with completely variable frequency, and hammers like this are available. Not only might one optimize the driving by adjusting the frequency, but one might minimize damage due to accidental resonance of the ground itself. A similar situation may arise for impact-driven piles, but the opportunity for resonance is much less because the impact is not a single frequency, and only a few cycles of any given frequency occur, so resonance does not develop.

Ideally, it is possible to compute all resonant frequencies, but for real field conditions, it is not realistic to do so. If one were to use a completely frequency-variable vibrator, all frequencies could be determined in situ by starting the vibrator at zero frequency and increasing frequency until all resonances have been identified by observing the amplitude of pile motion and ground motion near (within 1 m) the pile. Usually the stratum frequency will be lower than the system frequency, which is lower than the driver-pile frequency. The stratum frequency would be encountered first in the run-up of frequency from zero, and, if the force amplitude associated with the vibrator is great, large ground motion may be generated and transmitted away from the pile. The effect is similar for the soil-pile-driver system frequency. It would be better if the force amplitude of the vibrator could be varied so that stratum and soil-pile-driver resonances could be approached and passed with low power or force. Then, when the pile-driver system frequency is found, the force level could be increased to optimize driving. This kind of vibrator now exists .

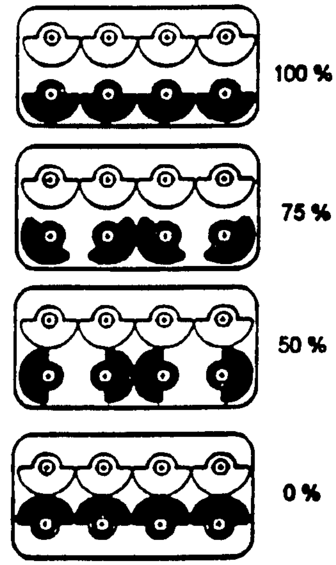


FIGURE 15 Operating principle of vibrator with dual rows of eccentric masses, allowing variation of static moment (displacement amplitude) *(9)*.

Not only can state-of-the-art vibrators vary the frequency of operation, but they can vary the force amplitude by changing the static moment. A vibratory driver that can do this is shown schematically in Figure 15 *(9)*. In this figure there can be seen two sets of eccentric masses arranged in two rows. By changing the orientation of the eccentric masses of one row, compared to the other row, the static moment can be changed, thereby changing the force at all frequencies. With this type of vibrator, the first two resonances associated with vibratory driving can be passed through with low static moment and then, when pile-driver resonance is reached, the static moment can be increased and the force level increased to drive the pile efficiently. For some hammers the optimum static moment could be monitored and adjusted on the basis of the amperage consumed by the electric motors driving the hydraulic pumps

CHAPTER FOUR

VIBRATION MITIGATION MEASURES

It is often suggested that vibrations can be interrupted using wave barriers or that alternative pile installation techniques will eliminate vibrations. These concepts are attractive, but the execution of an effective barrier system is difficult and expensive, and alternative installation methods are not always possible. Some of the common vibration mitigation techniques will be discussed and their potential effectiveness evaluated.

WAVE BARRIERS

The principle of operation of a barrier is to reflect wave energy back toward the source or absorb energy while preventing energy from propagating beyond the barrier toward a target building or other vulnerable structure or location. Wave barriers usually take the form of a trench or thin wall made of sheet piles or similar structural members. Woods *(10)* has described the basic parameters that dictate the effectiveness of this type of wave barrier. This work was experimental and based on model structures and barriers, however, the extension to prototypes has been verified with full-scale tests and by numerical methods. The general applicability of wavelength scaling in wave propagation problems has also been established.

As long as a trench-type barrier has physical separation between the two walls in the direction of wave travel, the barrier has a chance of being effective. The other dimensions for an effective wave barrier—depth (D_t) and length (L_t)—must be proportioned relative to the waves to be screened (Figure 16). The basic parameter of the wave to be screened is its wavelength (λ_R). The wavelength depends on the wave velocity and the frequency of the wave, as shown in Equation 3. Common frequencies for ground vibrations emanating from pile driving range from about 4 to 30 Hz. Then, with common wave velocities in soil of 61 to 610 m/sec, wavelengths range from 3 to 152 m.

Both experiments and numerical models show that an effective wave barrier (one that reduces vibration by about 88 percent) must be at least two-thirds of a wavelength deep to screen a seismic surface wave. The length of the barrier usually must be at least equal in depth to one wavelength of the incoming waves to screen even a small area. For example, the trench barrier in Figure 17 is 1.19 wavelengths deep and 1.79 wavelengths long, and it causes a reduction in amplitude of about 88 percent only in two small areas behind the trench (shaded zones within 0.125 contour lines). In most instances this leads to barriers that must be very deep and long. And, for the barrier to be most effective, it must be empty (void except for air). It is difficult to create and preserve deep, empty barriers.

Haupt *(11)* has studied solid concrete barriers and has found that with appropriate sizes they may be effective, but in no case as effective as an empty barrier of the same size. Hayakawa et al. *(12)* are currently using precast hollow concrete panels as permanent wave barriers along railroads in Japan. Woods et al. *(13)* have also studied rows of thin-wall, steel-lined holes as barriers and have found that they can be effective if they are spaced closely and are large relative to the wavelength of the offending wave. In one application, 450-mm-diameter corrugated pile shells were installed at a horizontal spacing of 1.5 diameters center to center and with a depth of 12 m; these were effective, but the installation costs, while tolerable for this permanent installation, would have been prohibitive for temporary applications such as pile driving. The specific configuration of the barrier depends on several factors discussed by Woods *(10)* and by Haupt *(11)*.

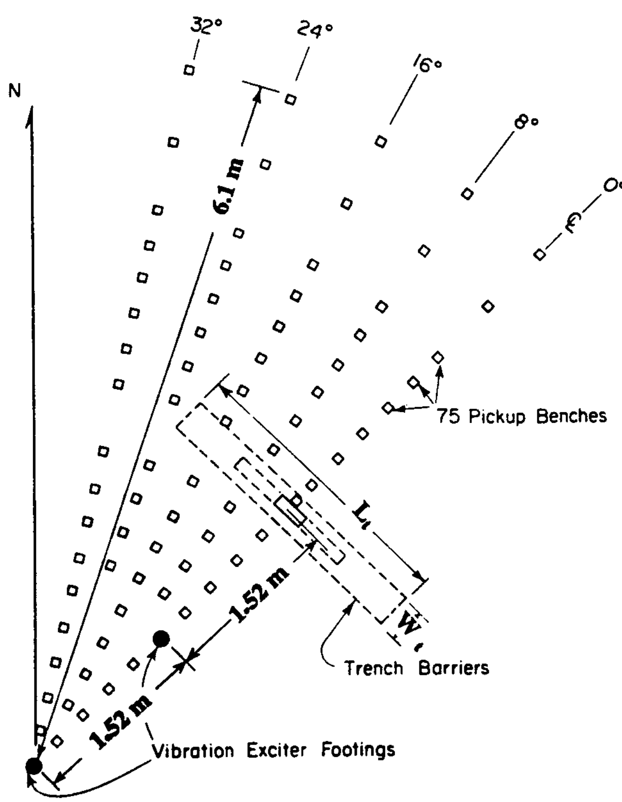


FIGURE 16 Plan view of field-site layout for passive isolation with a trench *(10)*.

Massarsch *(14)* described gas cushions that can be inserted into isolation trenches to preserve the openness of the trench for permanent wave barriers. Although these gas cushion barriers would be economical for permanent installations in some situations, it is unlikely that they would be cost-effective for temporary pile driving because the 1990 cost of these cushions was about \$420/m^2 of trench wall. It can be concluded that

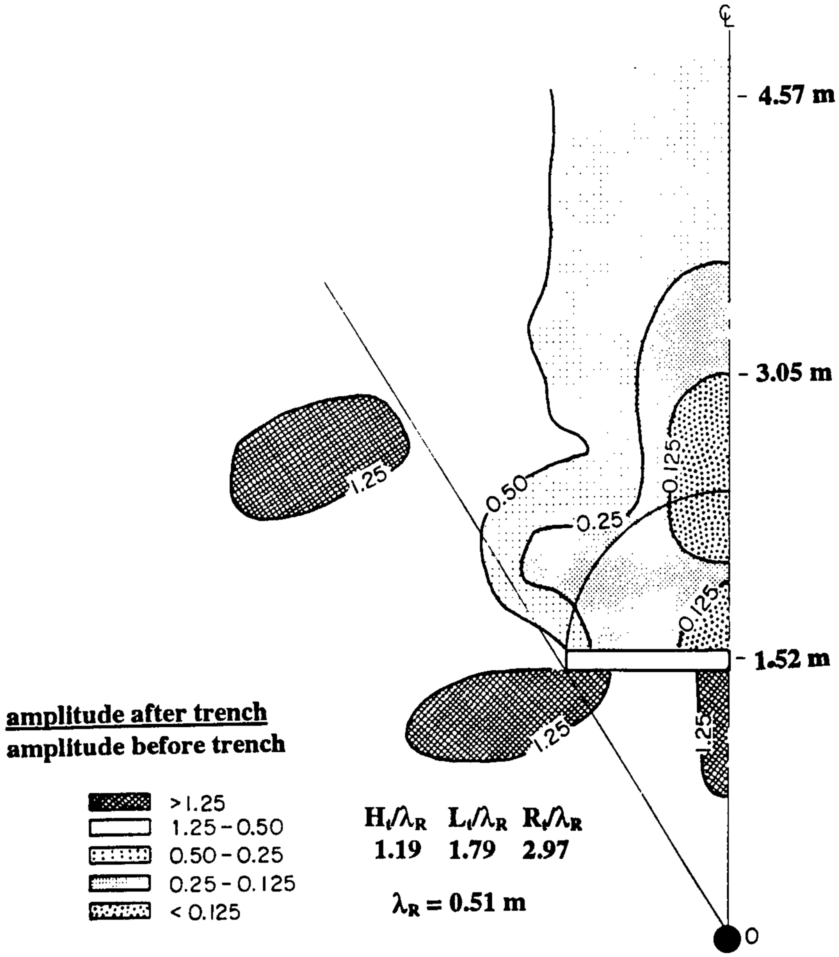


FIGURE 17 Amplitude-ratio contour diagram *(10)*.

wave barriers are not, at this time, cost effective for temporary application as in pile driving vibrations mitigation.

OTHER OPPORTUNITIES

Selection of the pile installation method, on the other hand, may provide the solution to some difficult pile driving vibration problems. Among the methods that might be chosen in this vein are

- Jetting,
- Predrilling,
- Cast-in-place (CIP) or auger cast piles,
- Nondisplacement piles like H-piles,
- Type of driver (vibratory in place of impact), and
- Pile cushioning.

Some or all of these may not be appropriate in specific situations, but where they are appropriate, these techniques will often reduce vibration amplitudes to acceptable levels. Some DOTs and other agencies allow contractors to adjust driving methods to reduce vibrations.

Jetting of piles often allows the contractor to sink a pile to a depth near to that at which pile bearing is expected. The jetting may bypass shallow, harder layers that could generate high levels of vibrations at or near the surface. Predrilling will be effective in those cases in which the soil will remain open long enough to insert the pile into the prebored hole. Usually these piles will have to be seated at their ultimate depths at which depth vibrations will be generated, but as with the jetting, the troublesome soil layer may be avoided. CIP piles avoid the requirement of driving, thus minimizing vibrations; however, in some cases pile capacity may be difficult to achieve with CIP piles.

Nondisplacement piles (H-piles) may minimize the vibration problem because load carrying capacity is expected in end bearing and large friction transfer along the shaft is not expected; furthermore, end bearing is affected without large volume displacement at the tip of the pile. However, environmental conditions such as saltwater may contraindicate steel H-piles.

As pointed out in chapter 3, the vibration potential of impact pile drivers is different than that of vibratory pile drivers. By switching from impact to vibratory hammers, some vibration problems can be solved, but layer resonances can defeat this process. Caution should be exercised and the situation analyzed carefully before the switch is recommended. A good knowledge of the soil profile is necessary to avoid potential layer resonances when using vibrators with fixed or only slightly adjustable frequencies. The engineer authorizing a change should be assured that the behavior of the vibratory hammer has been analyzed carefully.

The impulse (force-time record) of impact-type hammers on cushioned piles is substantially different depending on the condition of the cushioning. Keeping fresh, absorbing cushions in the system can reduce vibration by as much as a factor of 2. Allowing the cushions to be used well beyond their effective life may create unnecessary vibration problems.

CHAPTER FIVE

CONSEQUENCES OF GROUNDBORNE VIBRATIONS

Vibrations transmitted through the ground range from those that are unnoticed by humans and do not damage structures to large fault displacements that result from earthquakes and destroy anything in their paths. The vibrations generated and propagated by pile driving operations are in a small window at the low end of that range. Clearly the vibrations undetected by humans will seldom cause difficulties in pile driving applications. Some extreme exceptions may exist for cases in which sensitive instruments or manufacturing operations are located near the pile driving, or in which delicate medical operations are taking place. On the lower end, vibration displacements as low as 10×24^{-6} mm may be "damaging" to the operation of very sensitive instruments. The upper end of the range of vibrations associated with pile driving occurs on the piles themselves as they are being driven and may be up to about 100-mm/sec particle velocity.

For the potential development of problems from pile driving vibrations, three elements must be present: sensitive targets or receivers of vibration, media through which the vibrations are transmitted, and a source of vibration. The last two elements have already been discussed, so only targets remain for examination.

TARGETS OF VIBRATIONS—HUMANS

A target can be any person or object that may be sensitive to vibrations. Sensitivity takes on many forms ranging from the lower threshold of human perception to physical damage. Human perception is the most difficult component to deal with, and the entire vibration measurement and mitigation discipline can be held hostage to the most sensitive person who can hire an attorney. Objective measures of human vibration perception and tolerance have been attempted and are applicable for the "average" individual, but it is the least tolerant individual who may control the situation. Even then, whether or not the vibrations are in the interest, economic or otherwise, of the individual will dictate whether vibrations will be detrimental in the eyes of the receiver.

Figure 18 shows vibration sensitivities of people determined in early studies on human perception of vibrations undertaken by Reiher and Meister *(15)*. Other vibration limits for machines and structures are also given in this figure. Even though the human perception data are from an early source, the general form of these relationships has been confirmed as useful through the years up to the present. For more recent studies see, for example, those by Wiss and Parmelee *(16)* and Siskind et al. *(17)*.

In Figure 18 the vertical axis is single peak particle velocity and the horizontal axis is frequency. The 45-degree lines upward to the right and upward to the left are lines of displacement and acceleration amplitudes, respectively. Actually, Figure 18 is the classical tripartite diagram in which the three relevant derivatives of motion (displacement, velocity, and acceleration) are shown relative to frequency.

Another factor to consider for human exposure to vibration is the duration and time of exposure to vibration. Figure 19 shows the influence of duration of exposure time on human perception of vibration. The level of barely perceptible motion decreases from about 2.5 mm/sec for 1 sec of exposure to about 0.5 mm/sec at 100 sec exposure. Notice, in Figure 19, that shorter transient pulses are less perceptible and thus less potentially annoying than are long transient and continuous motions.

ISO *(18)* has established standards for human tolerance based on duration, magnitude, and orientation of motion. ISO considers the posture (reclining, standing, sitting at a desk, etc.) of the individual sensing the vibration and the length of time to which the individual is exposed to a given level of vibration to judge comfort and function. Using these standards, the hours during which pile driving is allowed might be limited so that residents in the vicinity of pile driving could have the opportunity for a good night's sleep.

ANSI *(19)* also establishes consensus standards on human tolerance to vibration, and the ANSI and ISO standards are usually identical. The ANSI 1993 basic standard for motion in all directions for human comfort is plotted on Figure 18.

Different activities and expectations for a quiet environment also influence human perception levels. ANSI presented Table 3 in standard S3.79–1983, which applies weighting factors to vibrations for hospitals, residential locations, offices, and workshops. These weighting factors recognize that human tolerance to a few motion exposures (three or fewer per day) is much greater than to continuous vibration. For example, from Table 3, exposure to fewer than three impulsive shocks generating 128 times the standard vibration level from the ANSI line on Figure 18 could be tolerated in an office setting, but for continuous or repeated impulsive shocks, only 4 times the standard from Figure 18 could be tolerated in the office setting.

Much is left to be learned about human response to vibration, so evolution in standards is to be expected. Nevertheless, human perception and tolerance are more often the trigger for complaints and litigation than actual physical damage to structures.

DIRECT DAMAGE DUE TO VIBRATIONS—STRUCTURES

Structurally damaging vibrations may also be of a wide range of amplitudes and frequencies. Evidence of structural damage often starts with the development of cracks in a

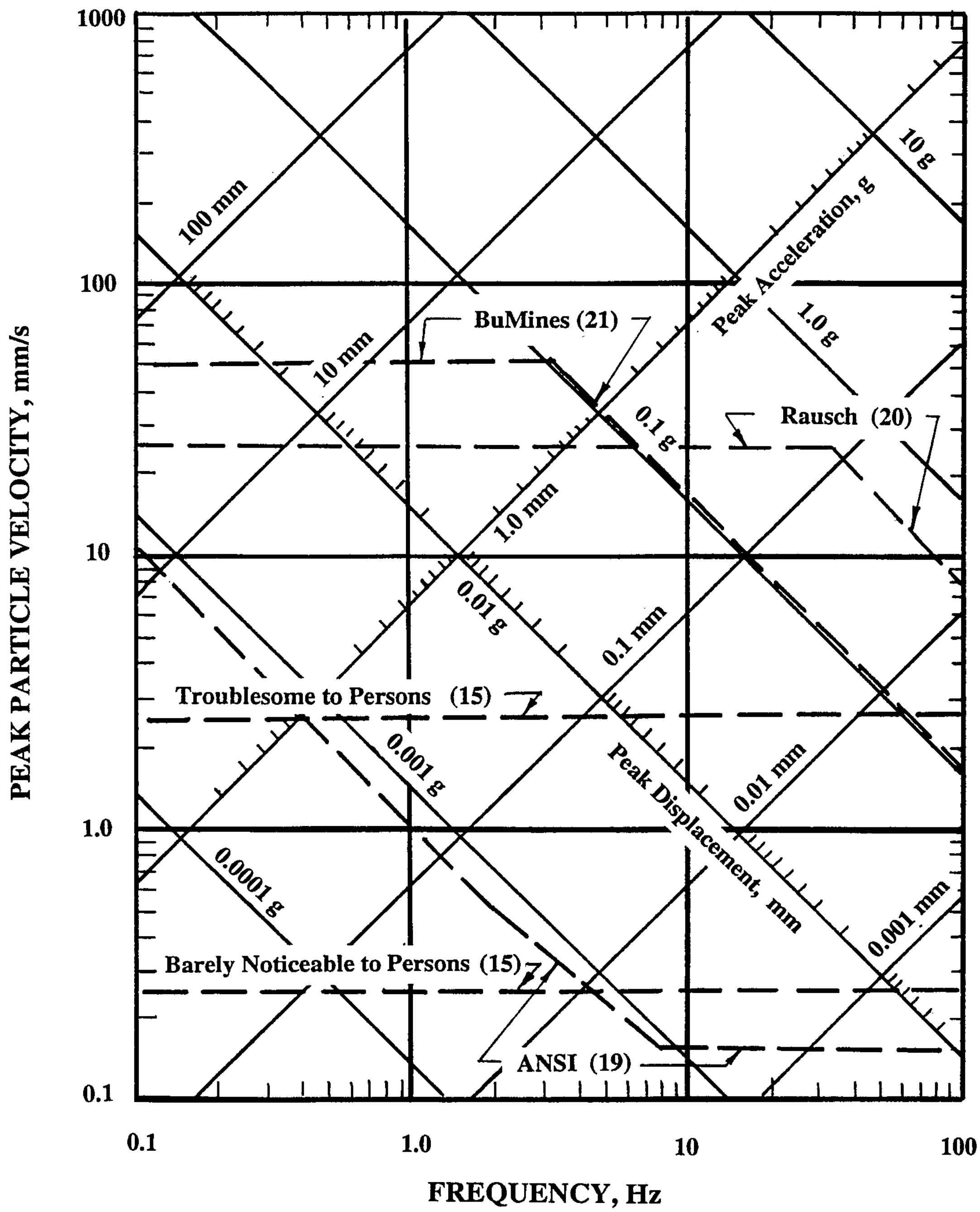


FIGURE 18 Vibration limits for people, machines, and structures *(1)*.

structure. Other evidence may be broken or cracked windows, building distortion due to settlement, or water leaking into a basement or out of a sewer or other conduit. Figure 18 shows historically early vibration limits for structures and machines combined with human perception limits. Rausch *(20)* determined the limit for machines and machine foundations as 25 mm/sec up to 32 Hz, then 0.5 *g* to 100 Hz. The limits for structural damage were set at 50 mm/sec peak particle velocity up to 3 Hz and at 0.1 *g* acceleration to 100 Hz by the Bureau of Mines, as reported by Siskind et al. *(17)*.

More recent studies have shown that the frequency of vibration must be included to a greater extent in criteria for vibration damage as shown in Figure 20. This figure contains information from the U.S. Office of Surface Mining (OSM) and the German Standards Office (DIN). From Figure 20 it can be seen that low-frequency vibrations require lower tolerances than high-frequency vibrations. There are also provisions for the type of structure under consideration. In Germany and Italy, vibration amplitudes are limited to 25 mm/sec to protect historical structures and other antiquities.

Probabilistic methods are also being applied to damage to structures in work by Siskind et al. *(17)*. Table 4 describes levels of damage in the probabilistic study, and Figure 21 presents a plot of measured vibration displacements at various

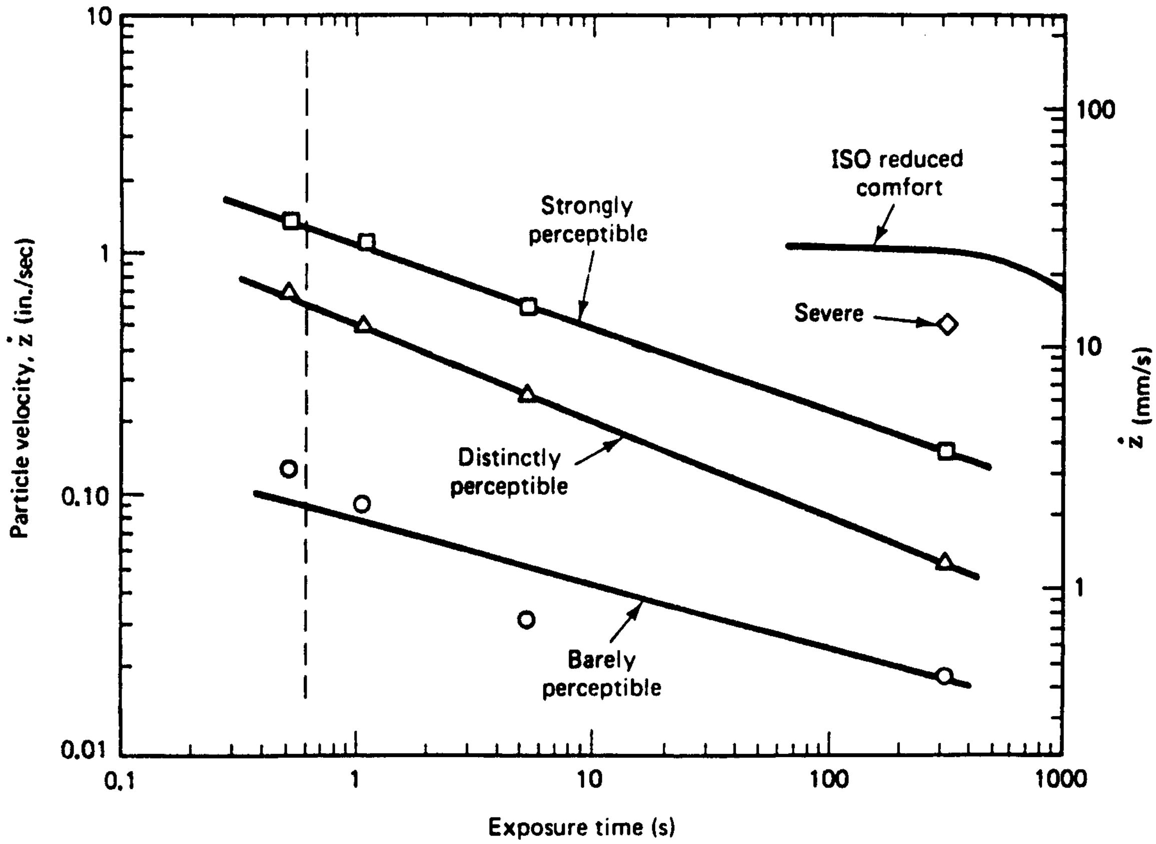


FIGURE 19 Human response to transient pulses of varying duration *(16, 17)*.

TABLE 3

WEIGHTING FACTORS FOR SATISFACTORY MAGNITUDES OF BUILDING VIBRATION WITH RESPECT TO HUMAN RESPONSE *(19)*

		Site Multiplying Factors	
Place	Time	Continuous and Intermittent Vibrations and Repeated Impulsive Shock	Impulsive Shock Excitation with Few Occurrences per Day (three or less)
Hospital operating room and critical working areas	All	$0.7\text{–}1^{(1)}$	$1^{(1)}$
Residential (good environmental standard)	0700-2200	1.4–4	90
	2200–2400	1–1.4	1.4
	0000–0700	1–1.4	1.4
Office	All	$4^{(2,3)}$	$128^{(2,3)}$
Workshop	All	$8^{(2,3)}$	$128^{(2,3)}$

NOTES:

(1) Magnitudes of impulsive vibration in hospital operating rooms and critical working places pertain to periods of time when surgical operations are in progress or critical work is being performed. At other times, vibration magnitudes as high as recommended for residences may be allowed provided there is prior agreement and warning.

(2) Impulsive vibration magnitudes in offices and workshop areas should not be increased without considering the possibility of significant disruption of working activity.

(3) Magnitudes of impulsive vibration in hospital operating rooms and critical working places pertain to vibration in workshops from certain industrial processes (such as drop forging or crushing) may be in a separate category from the vibration environment of workshops given in Table 3 of *(19)*. Vibration values specified in ANSI S3.18-1079 should then apply.

frequencies and the classification of damage. Data from Figure 21 were used to produce Figure 22, which shows damage probability for particle velocities and three classifications of damage. Siskind's data were collected from blasting-induced vibrations, but the damage potential of those vibrations is the same as that of pile-driving vibrations except for the potentially large acoustic component from blasting.

Vibration limits to prevent damage and human discomfort are not fully defined, although much research has been expended without a complete resolution of the issue. One of the

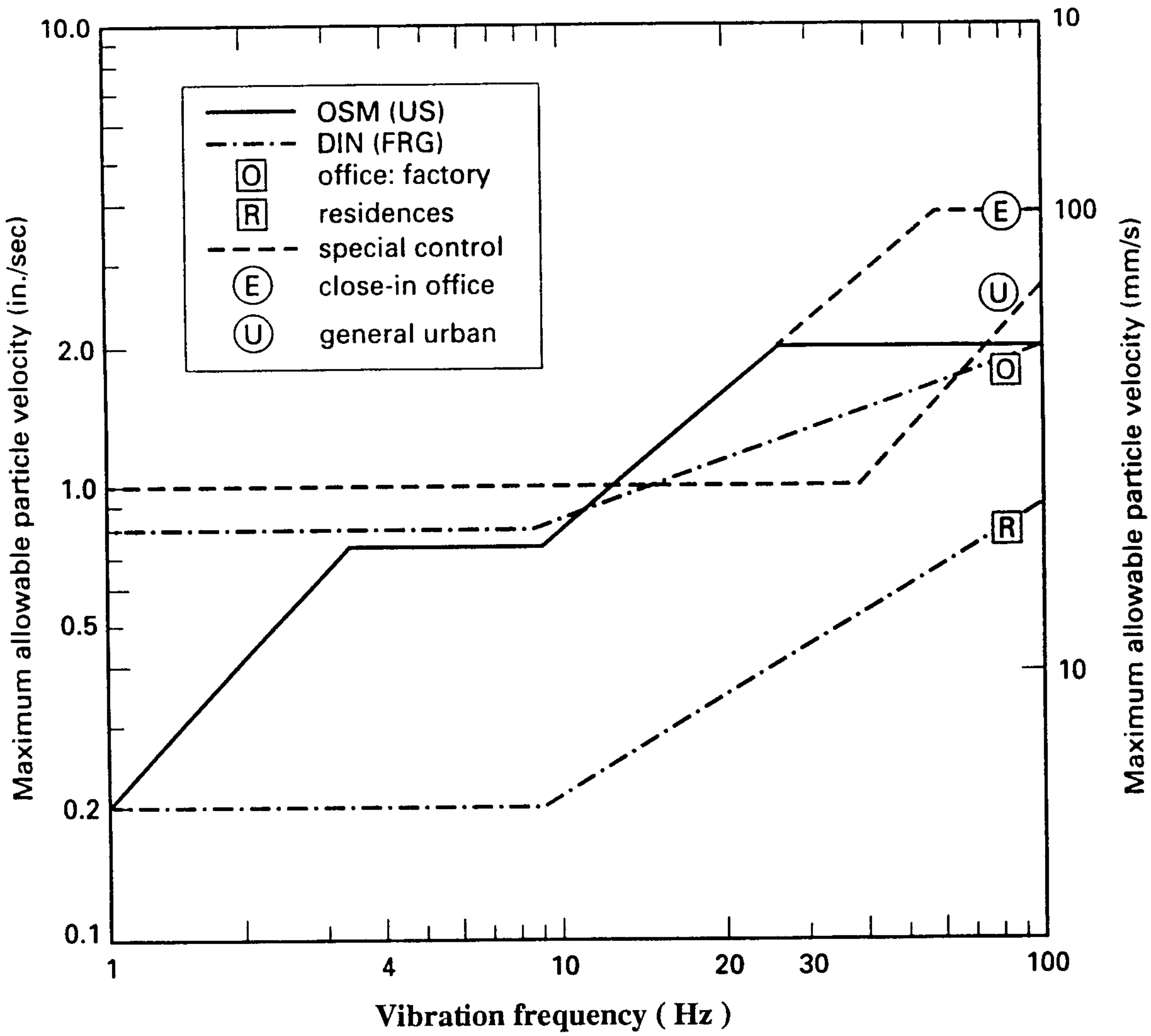


FIGURE 20 Comparison of frequency-based allowable particle velocity controls to show increasing particle velocity with increasing dominant frequency *(21)*.

TABLE 4

COMPARISON OF DAMAGE CLASSIFICATION IN PROBABILISTIC STUDY *(17)*

Description	Study	Uniform Classification
Loosening of paint Small plaster cracks at joints between construction elements	Threshold Dvorak (1962) Edwards and Northwood (1960) Northwood et al. (1963)	Threshold
Lengthening of old cracks	Minor Thoenen and Windes (1942)	
Loosening and falling of plaster Cracks in masonry around openings near partitions Hairline to 3-mm (0-1/8-in.) cracks Fall of loose mortar	Minor Dvorak (1962) Edwards and Northwood (1960) Northwood et al. (1963) Jensen and Rietman (1978) Langefors et al. (1958) Major Thoenen and Windes (1942)	Minor
Cracks of several millimeters in walls Rupture of opening vaults Structural weakening Fall of masonry (e.g., chimneys) Load support ability affected	Major Dvorak (1962) Edwards and Northwood (1960) Northwood et al. (1963) Langefors et al. (1958)	Major

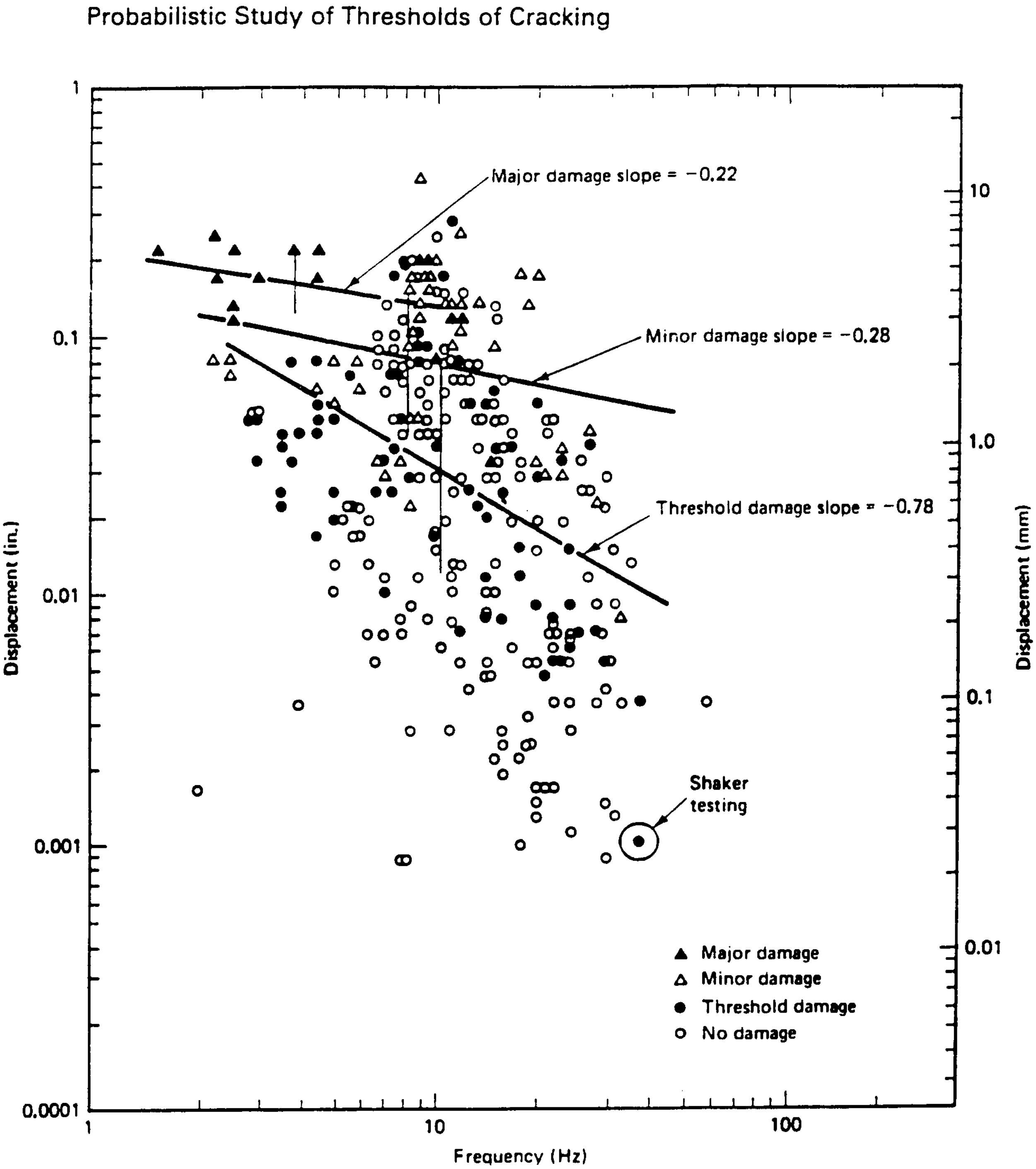


FIGURE 21 Displacement damage data; mean and variance analysis: low frequency *(17)*.

most complete treatments of this issue is included by Dowding *(21)*.

DAMAGE DUE TO VIBRATION-INDUCED SETTLEMENT

It is well known that loose granular materials will change volume when shaken. The familiar salt shaker and sugar container are examples of this phenomenon. Liquefaction of sands during earthquakes is an extreme result of this phenomenon. The key condition for this type of damage is a relatively clean granular material, initially in a loose condition and saturated with water. Looseness may be determined by relative density (D_r), where relative density is essentially a measure of the density of sand in comparison with its maximum density. Varying ways of measuring relative density are in use (even though there is an ASTM standard), but the concept is straightforward. Sands with relative densities less than about 75 percent can be easily shaken down by vibration. Hydraulic fills usually fall in this category, as do many natural fluvial deposits. However, quantifying the amount of settlement resulting from any

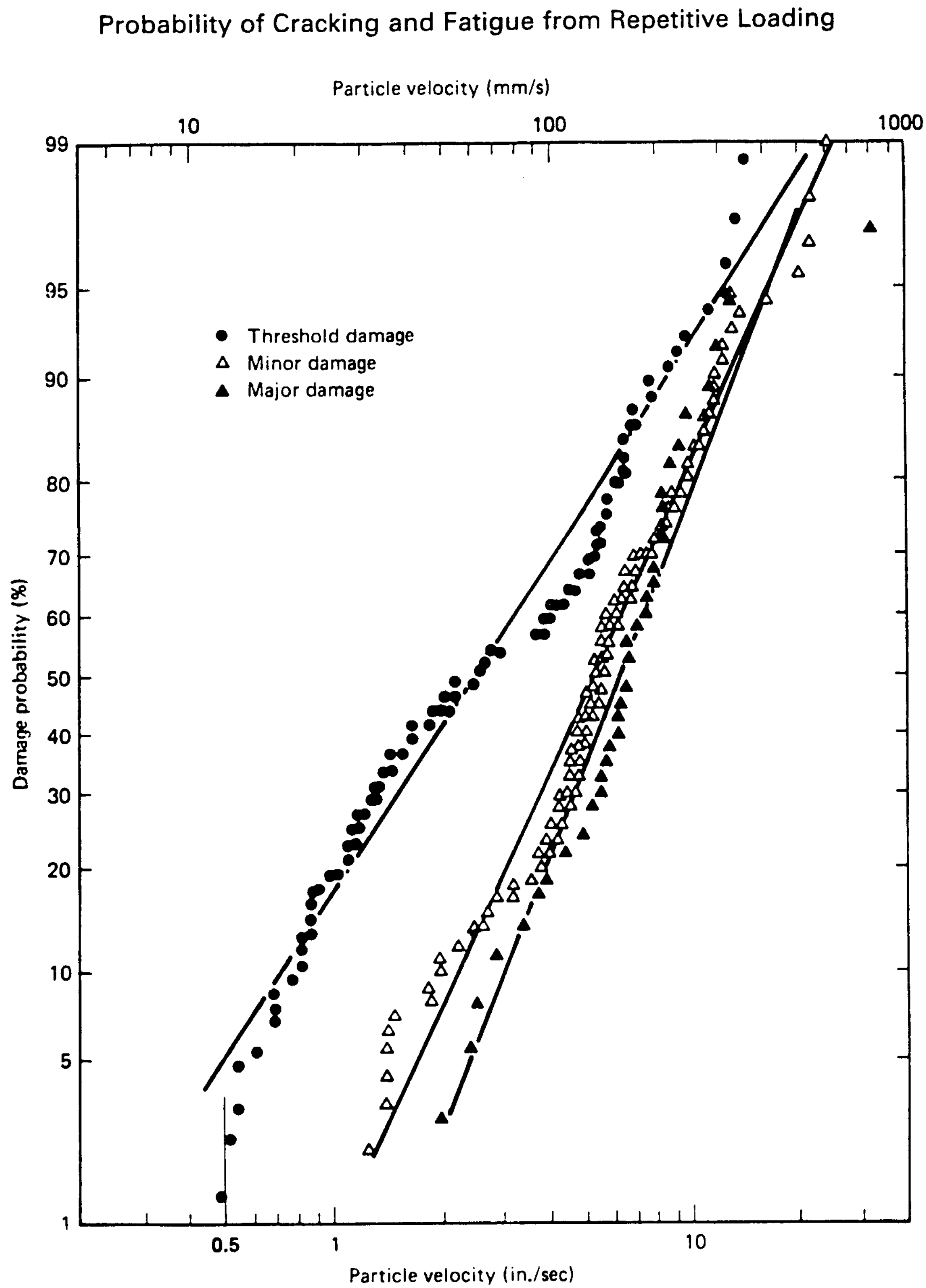


FIGURE 22 Damage data: probability analysis *(17)*.

specified vibration is not so easy to do. Some of the attempts to complete that relationship will be discussed.

Most of the available research relating cyclic strains to settlement have been developed for large shearing strains and low numbers of cycles as is applicable to earthquake situations. In contrast, vibrations from pile driving that cause settlement are likely to contain many cycles of low-amplitude shearing strains. Seed and Silver *(22)* performed research on the settlement of dry sand during large-amplitude shearing to predict settlement from a few cycles of strong motion vibration from earthquakes. Their results have been extrapolated on log-log plots to predict settlement for many cycles of low strain vibrations.

For example, this approach has been used to estimate settlement for the emergency diesel generator station for the Midland Nuclear Power Plant, Consumer's Power Company, in Jackson, Michigan *(23)*, and for traffic vibration-induced settlement of the Blackwater River Bridge, Florida *(24)*. In both of these instances, vertical strain was computed on the basis of relative density, magnitude of shearing strain, and number of cycles of shearing strain.

Borden and Shao *(25)* estimate volumetric strain from shear strain and number of cycles from laboratory cyclic shear tests. Their relation for this is

$$\Delta\hat{e}_{vol} = a\,(\gamma - \gamma_c)\,(\log N_{cyc})^b \qquad (22)$$

where

$\Delta\hat{e}_{vol}$ = dynamic volumetric strain,
γ = shearing strain amplitude,
γ_c = threshold shear strain amplitude at current confining pressure,
N_{cyc} = number of cycles of shearing strain (torsional strain in lab), and
a,b = functions of soil type and confining pressure determined from laboratory resonant column tests.

The authors confirmed this relationship with driven-pile tests in the field; however, to use this approach, laboratory testing must be performed to determine the a and b constants.

There have been many reported cases of settlement of sands caused by pile driving vibrations. A few references are reviewed here to show the nature of information available and the conditions under which excessive settlements were generated.

1. Dalmatov, et al. *(26)* describe two cases of settlement caused by pile driving, one from sheet pile driving and the other from structural pile driving. In the case of the steel sheet pile driving, 20 sheet piles, 8 m deep were driven within 7 m of the settlement point, which settled less than 6 mm. The soil was a silty sand with void ratio of 0.71.

In the case of the structural pile driving, settlements were measured on a 750-mm (inside diameter or id) water main at a depth of 3.25 m below ground and offset from the driven pile by 23.2 m. Maximum vibrations on the pipe occurred when the tip of the pile was at the elevation of the pipe. The soil at this site was random fill (building waste and organic and natural glacial soil) and no settlement was measured even though accelerations as large as 200 mm/sec^2 were measured on the pipeline. Hammer energy was about 34 000 N-m driving a hollow, 600-mm-diameter, reinforced concrete pile.

2. Youd *(27)* describes the volume change of sands undergoing cyclic shearing in the laboratory. He developed curves relating void ratio, number of cycles of shearing, and shear strain amplitude (Figure 23). Youd's work was primarily for earthquake phenomena, but the low end of his strain amplitudes fall in the range of strains measured in the ground next to piles supporting a bridge and suffering traffic-induced vibrations *(24)*. Youd's work, for the specific sand he used, can be related to vibration limits, shear wave velocity in the soil, and shearing strain amplitude. From Figure 24, the strain causing a specific vibration criterion to be exceeded can be determined when the shear wave velocity of the soil is known. Enter Figure 24 from the bottom or top (SI units) with shear wave velocity, project upward or downward (SI units) to the vibration criterion chosen, then translate horizontally to the shearing strain amplitude generated. This amplitude can be

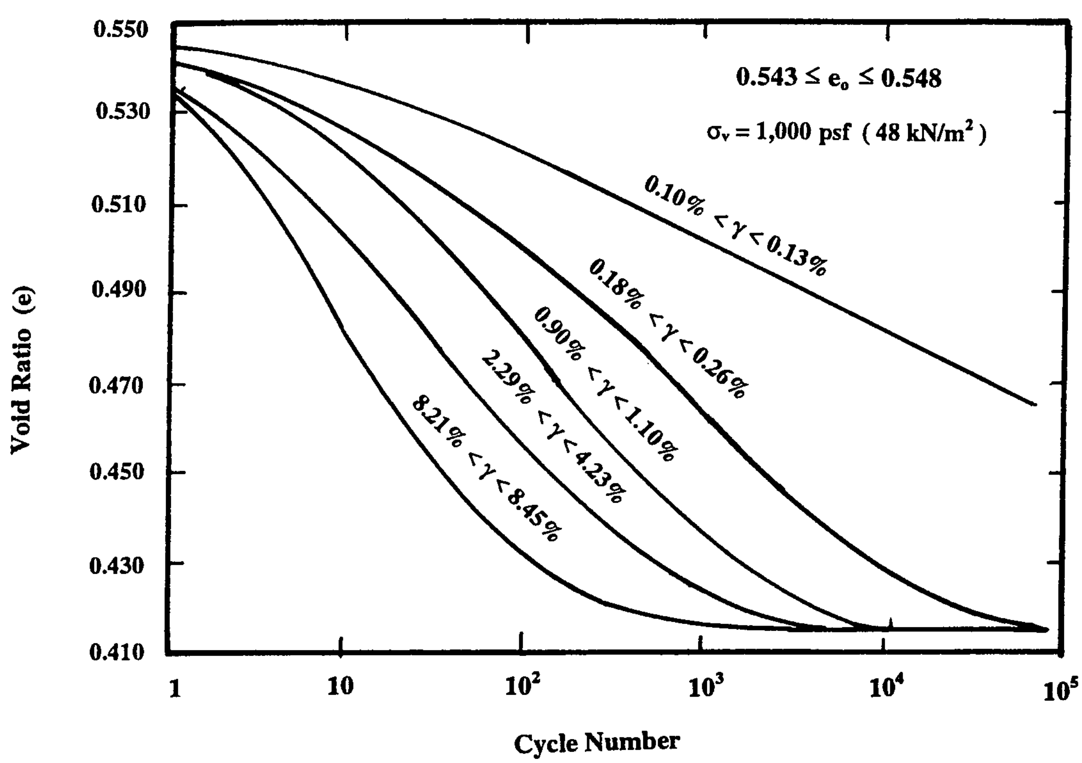


FIGURE 23 Effect of shear-strain amplitude on compaction *(27)*.

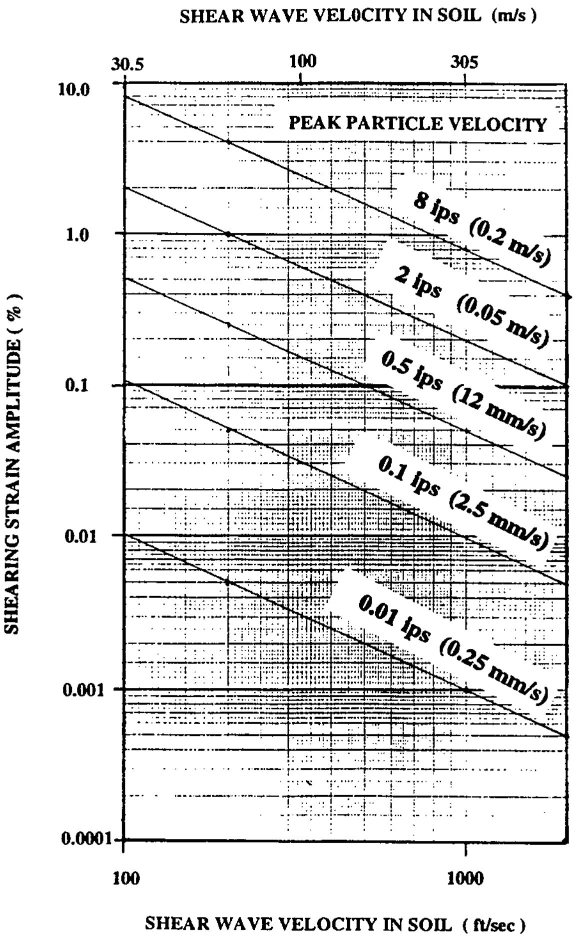


FIGURE 24 Shear wave velocity and particle velocity versus shearing strain.

used to enter Figure 23 with number of cycles of strain to get the void ratio after accumulating those cycles of vibrations. Using the initial void ratio and the void ratio just determined, the settlement can be calculated. This approach also requires the laboratory generation of curves like in such as those in Figure 23 for each specific sand.

3. Clough and Chameau *(28)* describe measurement of settlement due to driving sheet piling in San Francisco. Piles 11 to 15 m long were driven in the soil profile shown in Figure 25 near Borings E1 and E2. Vibration attenuation from the pile driving for both vertical and horizontal vibrations were measured; the attenuation for the vertical component is shown in Figure 26. The rapid decay shown here fits well with Equation 10 and indicates that vibrations die out quickly at the ground surface. The settlement associated with pile driving at these two locations, shown in Figure 27, indicates that settlement reduces to zero at a horizontal distance of about one pile length from the pile.

4. Lacy and Gould *(29)* provide a review of 9 cases of pile driving settlement from their corporate experience; and 10 cases from the literature. Tables 5 and 6 summarize these cases. In some of the reviewed literature cases, serious settlement did not occur, but in all of the in-house cases, settlement was a problem. Five of the cases dealt with bearing pile installation and four with sheet piling. The conclusions from this study are pertinent and are summarized as follows:

—Settlement from pile driving in loose to medium compact uniform sand can cause settlement with peak particle velocities much lower than the damaging vibration levels, usually taken as 50 mm/sec. Peak particle velocities as low as 2.5 mm/sec as measured on the ground have caused significant settlements at vulnerable sites.

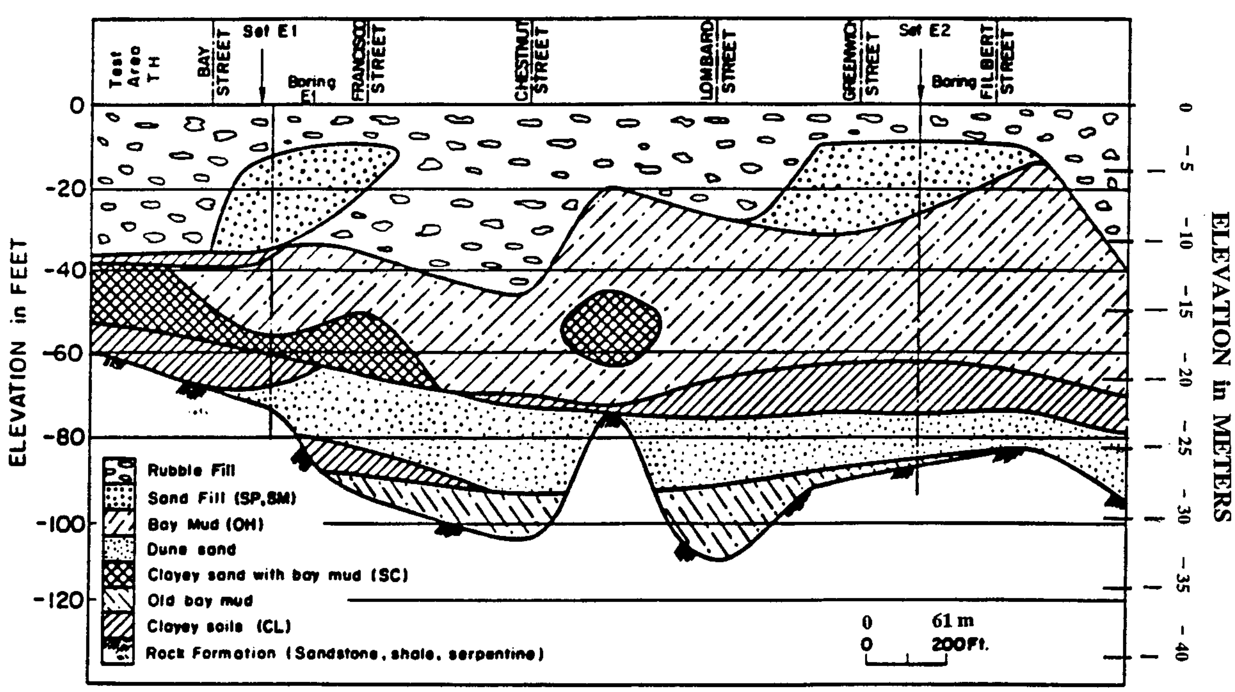


FIGURE 25 Idealized soil profile: Embarcadero area *(28)*.

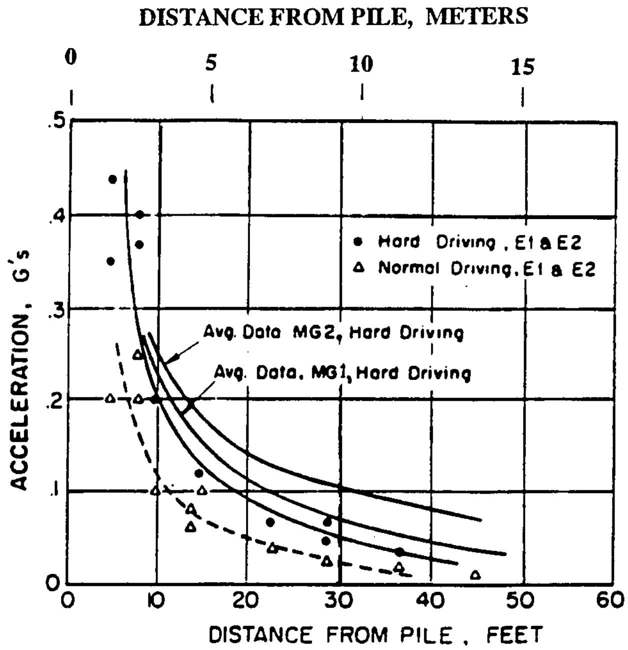


FIGURE 26 Peak vertical accelerations recorded during sheet pile driving: Areas E1 and E2 *(28)*.

—Job characteristics influencing the amount of settlement include distance from source to position of settlement or vulnerable structure, location of water table, location of adjacent foundations, and unbalanced hydrostatic heads at a site.

—Potentially damaging particle velocities are much lower than values associated with modest seismic events. Pile driving operations superpose many small effects for many cycles to produce much greater settlements than earthquakes that have peak accelerations between 0.05 and 0.1 *g*.

—Prediction of magnitude of settlement is not yet amenable to calculation.

—In some cases it may be counterproductive to drive dewatering sheeting extra deep for hydraulic purposes when extraction of those piles may cause settlement of sewers or other underground facilities.

—Potentially dangerous conditions arise when driving piles through cohesionless materials where that material is relied on for passive resistance for stability.

—Much more research is needed to understand vibration settlement.

5. Leathers *(30)* describes a site at which serious settlement occurred from driving load-bearing piles. Precast concrete piles, 355 mm^2, were driven to depths of 29 to 39 m in the profile shown in Figure 28. The most serious settlement occurred at the southwest corner of Building K. After driving about 180 piles on this site, the settlement contours shown in Figure 29 were measured. The piles were driven with an ICE 640 diesel hammer with rated energy of 54,000 N-m. The average volume change of the soil at the site within the depth of pile driving due to displacement of soil was about 1.3 percent. Average densification in the east quarter of the site amounted to about 1.7 percent, which, when added to the volume change from displacement, would predict a volume change of about 3 percent. The quantification of cause and effect was not attempted.

6. Picornell and del Monte *(31)* describe settlement from driving steel H-piles at a steel mill that caused settlement of pier foundations up to 254 mm. The predominant soil was loose to medium dense sand. The site was underlain with karstic materials with boulders over ledge rock and buried

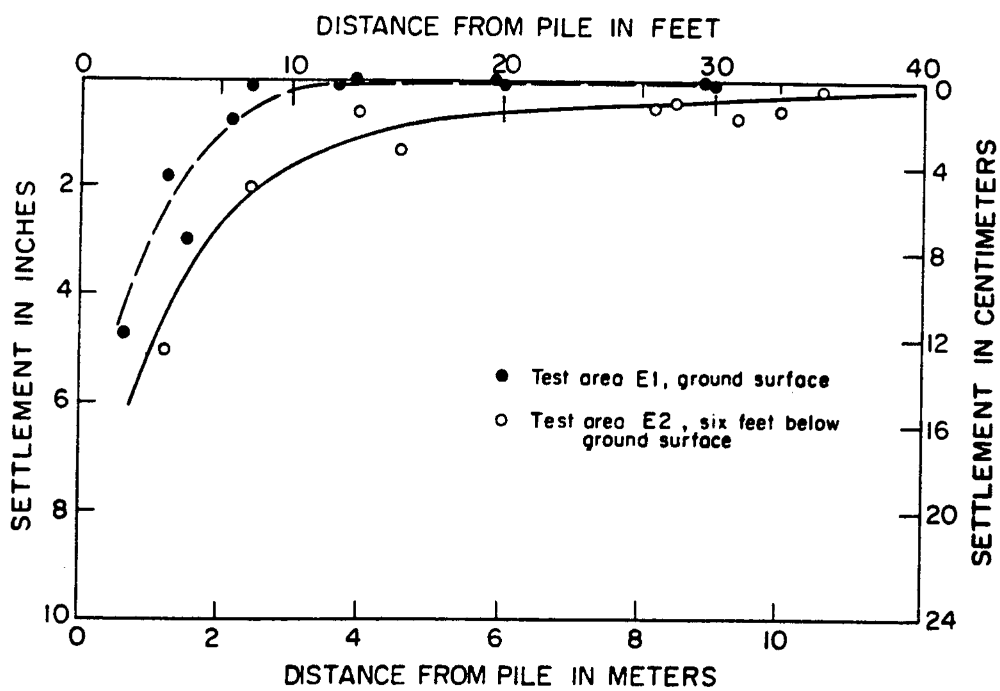


FIGURE 27 Settlements caused by sheet pile driving Areas E1 and E2 *(28)*.

TABLE 5

CASE HISTORIES FROM CORPORATE EXPERIENCE *(29)*

MRCE Case Desig-nation	Location	Pile Type	Source of Vibration: Hammer	Source of Vibration: Input Energy (ft/kips)	Source of Vibration: Distance Pile to Measure-ment (ft)	Source of Vibration: Peak Particle Velocity (in./sec)	Properties of Stratum Chiefly Involved: N (blows/ft)	Properties of Stratum Chiefly Involved: D_{60} (mm)	Properties of Stratum Chiefly Involved: D_{10} (mm)	Properties of Stratum Chiefly Involved: $U^{(1)}$	Properties of Stratum Chiefly Involved: $D_r10^{(2)}$	Remarks
A	Foley Square NYC	14HP73	Impact "Subsonic" Bodine-"Sonic"	26 – –	20 20 20	0.19 0.14 0.19	22–40 29	0.02 0.32	0.005 0.10	4 3	42–49 53–57	Buildings settled 3 in.
B	Lower Manhattan NYC	18 in. open-end pipe	Vulcan 010	32	–	–	20–40	0.35	0.10	3	40–60	1.5 ft settlement of street
C	West Brooklyn NYC	14HP73	Vulcan 08	26	5–30	0.1	8–25	0.12	0.03	4	30–50 40–60[3]	Structure settled 3 in. as 40 piles were driven
D	South Brooklyn NYC	10.75 closed-end pipe	Vulcan 08	26	10–80	0.9–0.1	21–35	0.26	0.13	2	40	Structure settled 3 in. as 220 piles were driven
E	Lower Conn. River	12HP53	MKT 10B3	13–20	3.5 c-c	–	20	0.42	0.10	4	40	Ground between piles settled 2.75 ft
F	West Brooklyn NYC	Hoesch 134	ICE 812	4.0	3	–	27	0.12	0.03	4	48[3] 40–60	Building settled 2.4 in.
G	N. Syracuse NY	PZ-27	ICE 416	2.2	10–25	–	1	Sandy silt/silt/coarse to fine sand			25	Ramp settled 3 in. as sheeting removed
H	Syracuse NY	PZ-27	ICE 812	-	4 ft from sewer	–	7	Fine sandy silt/fine to coarse sand			30	Sewer settled 6 in. as sheeting removed
I	S. Queens NYC	Hoesch 134	ICE 812	4.0	4 ft from sewer	–	25	0.40	0.10	4	45	Sewer settled 3 in. as sheeting removed

NOTES:

(1) U = Uniformity coefficient = D_{60}/D_{10}.

(2) Relative density based on Bazarna.

(3) Relative density from actual measurement.

TABLE 6

CASE HISTORIES FROM LITERATURE REVIEW *(29)*

Case Designation (Reference)	Driven Pile of Sheeting	Source of Vibration: Hammer	Source of Vibration: Input Energy (ft/kips)	Source of Vibration: Distance Pile to Measurement (ft)	Source of Vibration: Peak Particle Velocity (in./sec)	Soil Data	Remarks
P *(5)*	PZ (Belgium)	ICE 812	4	5 to 150	2 to 0.03	Loose fill with boulders soft clay and dense sand	18 Hz. D_r = 35 to 65 percent. Plots of vertical strain versus ground acceleration for various values of D_r. Settlement up to 6 in.
U *(7)*	PZ (Russian)	Drop	6.5	3 to 21	–	Medium sand	Measured peak particle accelerations. Settlement 0.2 in. Soil vibrating at 18 to 56 Hz.
V *(7)*	24 in. Hollow concrete	Drop	20	–	–	Fill over sandy silt	Soil vibrating at 24 Hz. No measured settlement of water main.
W *(12)*	Bearing piles	Percussive bored pile	–	10	–	–	Pile installation method avoided settlement from use of impact hammer.
X *(12)*	Bearing piles	Impact hammer	–	Very close	–	–	Piles at varying distances were driven with different restrictions.
Y *(13)*	12 in. pipe and 14 in. shell	Vulcan OR	30	3.5 to 28 ft	–	Sand fill; organic silt loose to medium dense sand (N = 25); limestone, compact sand.	Previously driven 60 to 80 ft piles settled up to 7 in. Telltales showed vibration caused downdrag loading the pile top to 35 tons. Required all piles be driven within 30 ft before placing pile cap.
R *(9)*	14 in. pipe	Link–belt 520 diesel	30	–	0.1 to 0.4	Rubble fill; 10 ft to 30 ft silty clay; 30 ft + loose silty sand; stiff silty clay	Piles driven with mandrel. Peak particle velocity did not increase when driving resistance in lower stiff clay increased.
S *(9)*	H–piles for trench	Diesel	–	–	–	Asphalt and fill/compact silty loam	Measured peak particle velocity was higher for 12 in. H–pile than for 14 in. H–pile.
Q *(9)*	Steel sheeting	Vibratory	–	110 12	0.09 0.2	Loose to medium dense sand	1/4 in. settlement 35 ft from trench. 1/8 in. settlement 90 ft from trench.
T *(9)*	12 in. H–pile for trench	MKT 9B3	9	30	0.02	Loose to medium dense sand	Reported previous large settlement and extensive damage to buildings at another site with similar soils and same hammer with steel sheeting. No settlement with H–pile.

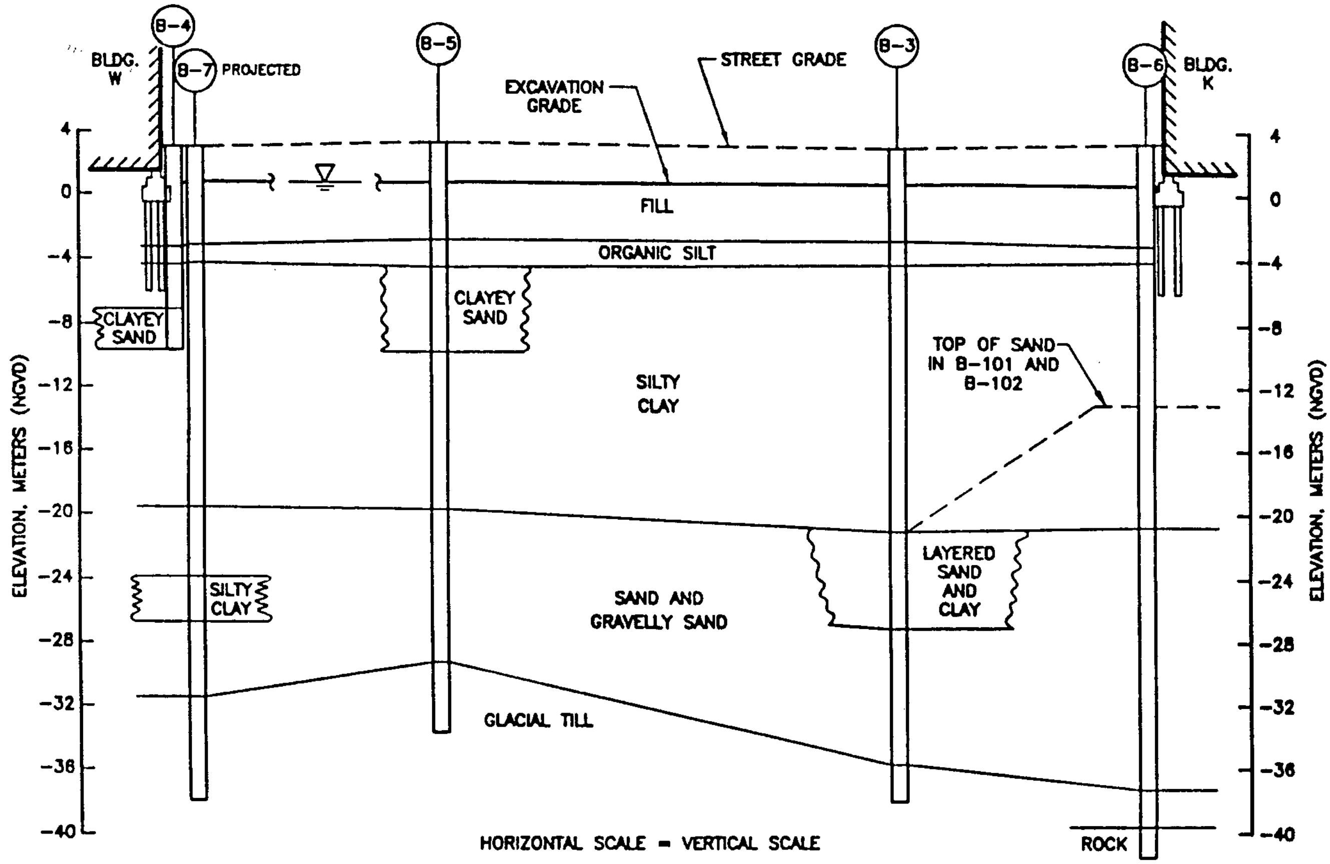


FIGURE 28 Subsurface profile A-A *(30)*.

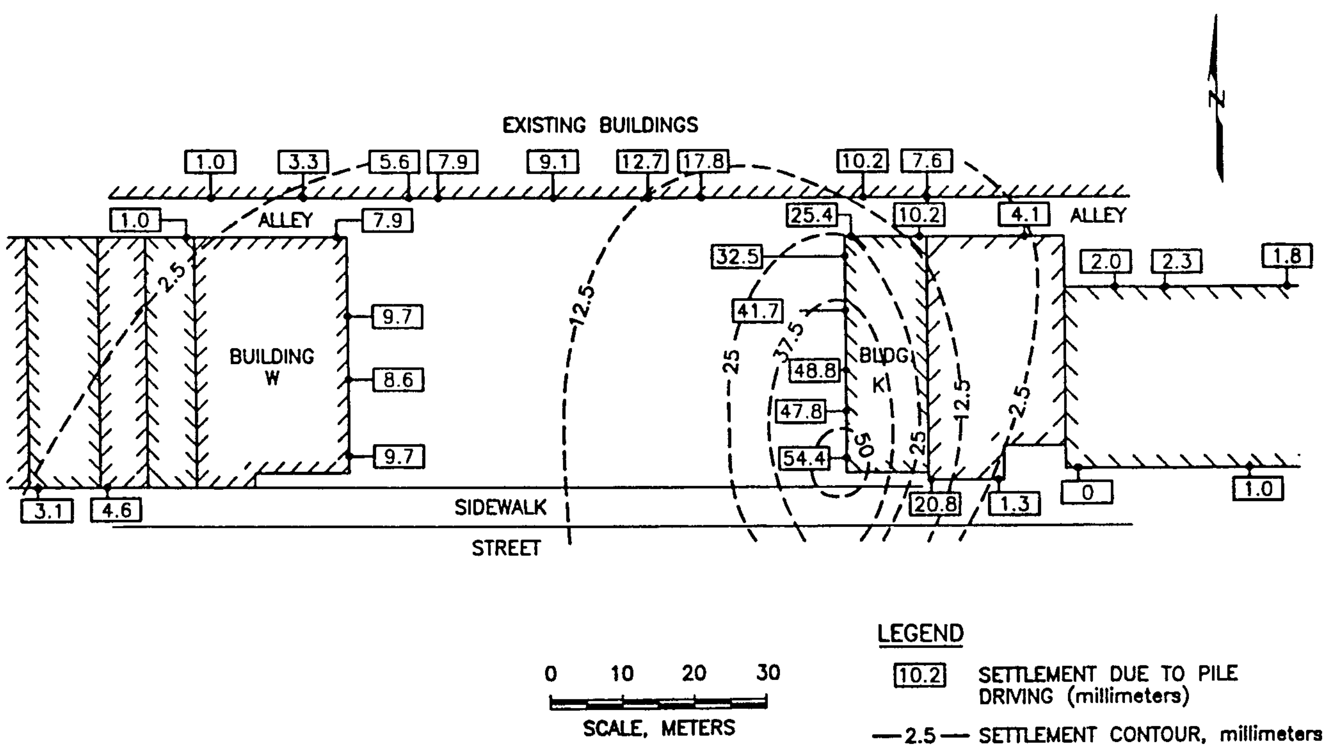


FIGURE 29 Settlements due to pile driving *(30)*.

sand deposits. Usual methods of settlement prediction would not have identified this problem.

Settlement of loose sand during pile driving is clearly a problem, and simple methods of estimating the magnitude of settlement are still out of reach. The prudent approach for inspecting engineers is to always proceed with caution when this condition is known to exist. On the other hand, the settlement of cohesive soils due to vibrations from pile driving is unlikely and would occur only under unusual conditions.

POTENTIAL HAZARD TO CURING CONCRETE

Another consequence of groundborne vibrations from pile driving might be the disturbance of nearby concrete in pile shells recently placed or other fresh concrete in the vicinity. In the version of the American Concrete Institute's (ACI) *Manual of Concrete Practice* currently being considered, the effects of vibrations on curing concrete are discussed. The following is taken from the version of the ACI 231–97, Chapter 5 under consideration. It is not yet issued as a final section of the manual.

5.5 Effect of Shock and Vibrations
5.5.1 Background

Shock refers to significant, external shaking or impact disturbance of recently placed, low maturity concrete prior to its reaching about 70% of its 28 day strength. The chief concern is not to exceed the threshold of force input which will critically disrupt the concrete matrix during the formative bond development stage. Typically this will occur within seven days. Since the hydration process is irreversible, the physically broken developed bond will essentially not be restored (except for autogenous healing) and the concrete's normal strength attainments will be weakened by the degree of ineffective paste bonding. However, if the concrete matrix stays intact, i.e. no relative shear, rigid confinement, etc., bonding will occur. This discussion does not refer to any type of vibrating or re-vibrating of plastic concrete to achieve consolidation or further improvement of its properties.

5.5.2 Historical Criteria Development

The most common peak velocity vibration limit has been 2 in/sec (50 mm/s). This limit originated from blasting operations where criteria were established to protect masonry and plastered structures to avoid disturbing the public and consequential legal struggles, Nicholls, Johnson and Durvall (1971), Crandell (1949), Edwards and Northwood (1960), and Northwood, Crawford and Edwards (1963).

Atkins and Dixon in 1977 compiled an extensive history and list of references on the development of vibration criteria relative to construction activities and concrete work. Although some reports of peak particle velocities in excess of 2 in/sec (50 mm/s) were noted, insufficient evidence was available to confidently establish higher general usage criteria. The resulting recommended vibration limits, as low as 0.2 in/sec (5 mm/s), can be very conservative for engineered reinforced concrete structures.

5.5.3 Reported Information on High Vibration Inputs

Some case histories have been cited to support good results when allowing vibration limits well above the "conventional peak particle velocity standard" of 2 in/sec (50 mm/s). The difficulty is much of the data contain insufficient details to support confident extrapolations. Schrader, Coplen and Lindholm (1967), Ferahian and Hurst, Neuman (1947), and Bastian (1970). However, all of the reports confirm that young concrete can sustain much higher vibration inputs than those imposed for masonry and plaster.

The two independent test programs by Howes (1979) and Hulshizer and Desai (1984) covering blasts effects on maturing concrete, provide significant data for evaluating vibration magnitude and duration influence on the structural properties of concrete. The test program reported by Howes (1979) addresses simulated blast vibrations induced every 60 minutes into standard 6 x 12 cylinders for seven days. The second test program reported by Hulshizer and Desai (1984) utilized actual field blasts and a laboratory shaker table to study effects that high peak particle velocity shocks have on compressive strength, shear capacity and bond values for induced shocks up to 24 hours after casting. Failure thresholds were never reached in these two test programs.

5.5.4 Vibration Acceptance Levels

Combing the results reported by Howes (1979) and Hulshizer and Desai (1984), there appears to be ample information available to permit the establishment of "high" peak particle velocity limits on early age concrete. Where no environment, public structures, human tolerance or other safety considerations are involved, considerable latitude is available in establishing vibration limits in the order of 5 in/sec (125 mm/s), or higher, for concurrent placement work without approaching a failure threshold.

Constant vibration situations should easily tolerate peak particle velocities in the range of 1 to 2 in/sec (25–50 mm/s) and as high as 4 to 5 in/sec (100–125 mm/s) with proper pretesting and evaluation. Blast, shock type vibrations should easily tolerate 5 in/sec (125 mm/s), and most likely 10 in/sec (250 mm/s) will not produce adverse effects during the first 24 hours of concrete placement. Extending the peak particle velocity above 5 in/sec (125 mm/s) in the 2 to 4 day range should be evaluated due to the lack of reported information regarding concrete/high vibration performance during this formative period of concrete strength development.

Since no threshold of failure has been reported in the test programs cited, with peak particle velocities up to 16.0 in/sec (400 m/s), establishing high level criteria in the 2 to 7 day range in the 5 to 10 in/sec (125–250 mm/s) range should easily be demonstrable with limited compressive cylinder testing.

References cited in this section from the ACI are included in the synthesis bibliography.

CHAPTER SIX

INSTRUMENTATION FOR VIBRATION MEASUREMENT

The basic transducers and other instruments for vibration measurements are described in detail in many places *(1, 20, 21)*. Vibration measurements usually take the form of determining magnitude or amplitude of motion (displacement, velocity, or acceleration) as a function of time. Therefore, the transducers selected must be designed to measure one of these derivatives of motion. Occasionally, the parameter chosen to measure is strain, in which case strain gauges must be applied to a target surface before the vibration-causing event occurs.

VELOCITY TRANSDUCERS (GEOPHONES)

A transducer is any device or instrument that converts a physical phenomenon into an electrical signal. Common examples are strain gauges, thermistors, electrocardiographs, and seismographs. Most measurements associated with pile driving and other construction vibration are made with portable seismographs of one design or another. These are usually based on the particle velocity component of motion, which is measured with velocity transducers.

The choice of transducer depends on the frequency and amplitude of motion to be measured. In most cases, the amplitude of ground or structural motion generated by pile driving will be in the relatively low frequency range (0 to 100 Hz) and in the moderate to low amplitude range (0 to 13 mm). For this range of motion, the velocity transducer is very convenient. The principle of operation is that of an electrical coil moving in a magnetic field (see Figure 30, for example). The permanent magnet produces a magnetic field through which the coil moves when the transducer case is shaken. A voltage is produced by this mechanism that is proportional to the relative velocity between the coil and the magnet. Either the coil or the magnet may be the moving element.

A geophone can be designed so that its own natural frequency is below 1 Hz and the electrical output is large enough that no amplifying accessory equipment is required to magnify the signal before recording. It is necessary to have different velocity transducers to measure vertical and horizontal vibrations because of the internal mounting of a mass on springs, but this is no disadvantage because both types are easily obtained.

Velocity transducers in which the magnet is the moving element are sensitive to external magnetic fields, whereas those with moving coils are insensitive to external fields. The manufacturer can tell the purchaser whether or not the magnet moves. Most low-frequency velocity transducers (less than 2 Hz natural frequency) use a moving magnet; higher-frequency (greater than 4 Hz) units have a moving coil and a fixed magnet.

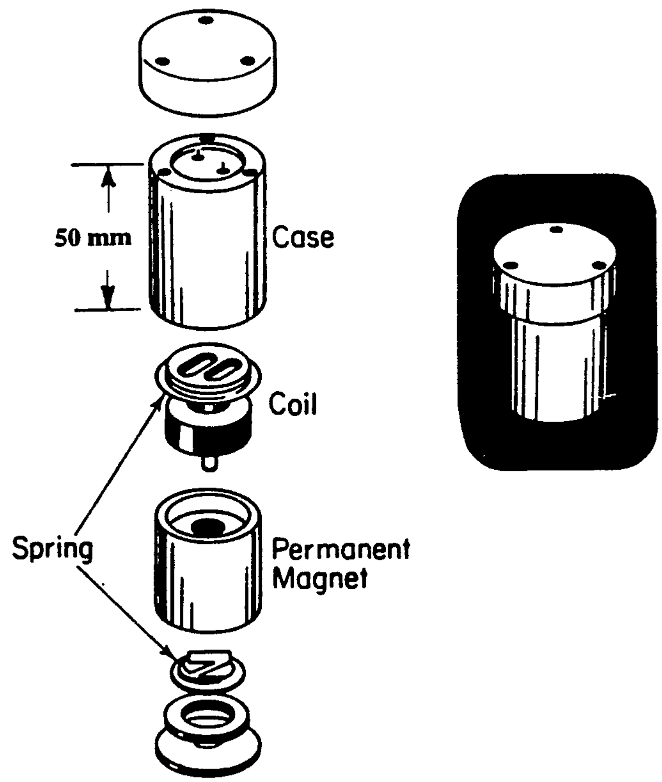


FIGURE 30 Schematic diagram of components of a velocity transducer *(1)*.

Velocity transducers produce a voltage that is proportional to the particle velocity associated with the surface on which they are mounted. Mounting must be done with care so that faithful representations of motion are made. Sometimes the mass of the transducer is sufficient to couple the transducer to the surface on which it is resting. In this case a three-legged support is recommended. Sometimes the transducer can be glued or bolted to the surface or buried underground. When measuring at the surface, where acoustic noise can reach the transducer, or when the mounting surface is uneven, a sandbag draped over the transducer provides good coupling to the surface.

Voltage in a velocity transducer is not adversely affected by cable length, so these transducers can be located at considerable distance from the recorder with little or no consequences as long as the cable is shielded from electrical noise and is waterproof. This is not true for accelerometers, as will be described later.

For many field vibration measurements it is convenient to use a portable seismograph. Most DOTs and independent vibration monitoring agencies use portable seismographs. These instruments are self-contained with at least three velocity transducers to measure three components of motion (vertical and two horizontal directions) and usually a microphone to measure sound level. They are battery powered, have their own internal record storage, and have a printer to produce a record of a vibration event at the site. The common portable seismograph is about the size of a briefcase in plan and twice as thick. Two examples of portable seismographs are shown in Figure 31. Figure 32 is an example of the printout of a portable seismograph. Table 7 lists some characteristics of portable seismographs for which the engineer should look for when this type of instrument is used in the field.

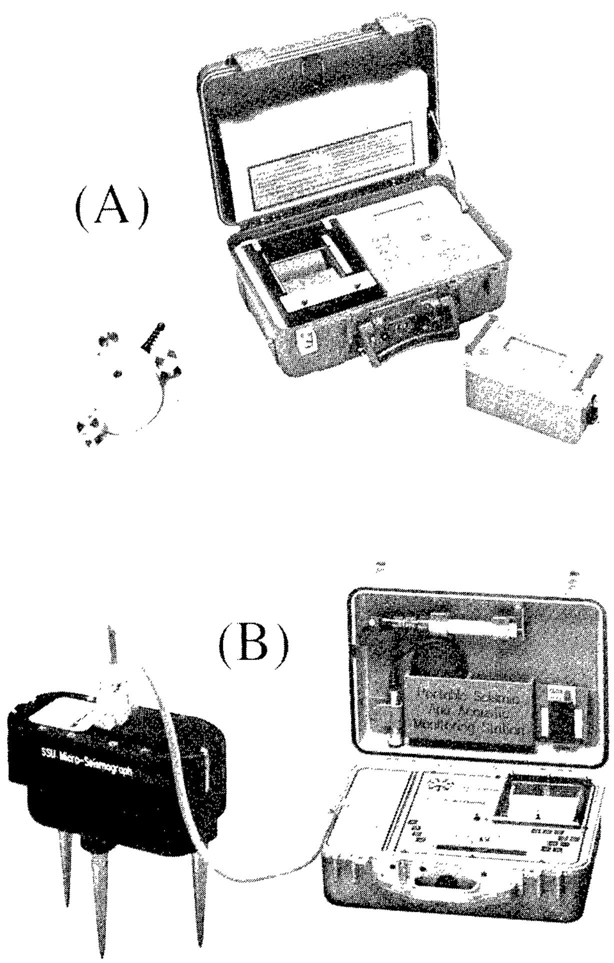


FIGURE 31 Examples of portable seismographs.

Acceleration Transducers (Accelerometers)

For ground motions greater than 250 mm/sec and frequencies higher than about 500 Hz, it may be convenient or necessary to use accelerometers (i.e., acceleration transducers). Accelerometers are produced with several transduction principles, but the most common type uses the piezoelectric properties of certain natural and artificial crystals. The basic principle is that if one of these piezo crystals is squeezed or sheared, it will cause current to flow in a conductor attached to opposite sides of the crystal. The amount of current is proportional to the pressure or shear force.

In an accelerometer, the crystal is squeezed by a seismic mass, which produces a force proportional to its acceleration (from $F = ma$). The lower portion of Figure 33 shows both the compression and shear type accelerometers. With either of these arrangements, the current from the crystal is also proportional to the acceleration of the base of the transducer. Other typical accelerometers are shown in the upper part of Figure 33.

The most commonly used accelerometers are about 19 mm in diameter and 25 mm high; however, those shown in Figure 33 range from as small as 6 mm in diameter and 6 mm high to 75 mm in diameter and 64 mm high.

One drawback of the accelerometer is that it requires accessory (signal conditioning) instrumentation in addition to recording instrumentation. Another drawback is that the type of cabling necessary for piezoelectric accelerometers is quite vulnerable in field applications and the calibration of the transducer may also be dependent on the cable length. Figure 34 gives a schematic of the accelerometer setup with signal conditioner and cables as well as a typical accelerometer with dimensions shown. Because of the need for signal conditioning and the low output for accelerometers, they are usually not the transducer of choice for ground vibration measurements from pile driving. However, recent advances in accelerometer design are making them more robust, and they may soon be more likely choices for many applications.

Most ground and structure vibration measurements are made with velocity transducers either as independent, single-component units or as triaxial packages such as portable seismographs. Occasionally vibrations with large accelerations are generated by pile driving, or measurements of vibration on the pile itself, are necessary and accelerometers are needed. Table 8 provides a checklist of characteristics of both velocity transducers and accelerometers on a comparative basis to aid in selecting vibration transducers for any use.

Vibration Records

Vibration records are obtained in many ways, but most represent a display of voltage versus time. Voltage is related to motion through a calibration factor for each transducer. Older forms of recording include many kinds of strip chart recorders. The most recent form of this kind of recorder is represented by the fiber optic oscillograph. This instrument produces a voltage trace on a light-sensitive paper that develops under ultraviolet light (sunlight or fluorescent light). Up to 18 channels of recording are available with paper speeds up to about 5000 mm/sec. Other older forms of recorders included FM and digital tape recorders, and light beam oscillographs that wrote on film.

Today, most recording is done in digital format by converting the analog voltage signal to a digital form and then storing

Event Summary Report

Date/Time	March 30, 1994 13:56:36	**Serial Number**	1928 V5.21 BMII-477
Trigger Source	GEO (Long) 1.778 mm/s	**Battery Level**	6.5 Volts
Record Time	2 sec	**Calibration Date**	January 26, 1994
Sampling Rate	Normal	**Code**	C9284TOX.EC0

User Notes

Location-Best Rock Quarry
User-Bob Best

Post Event Notes

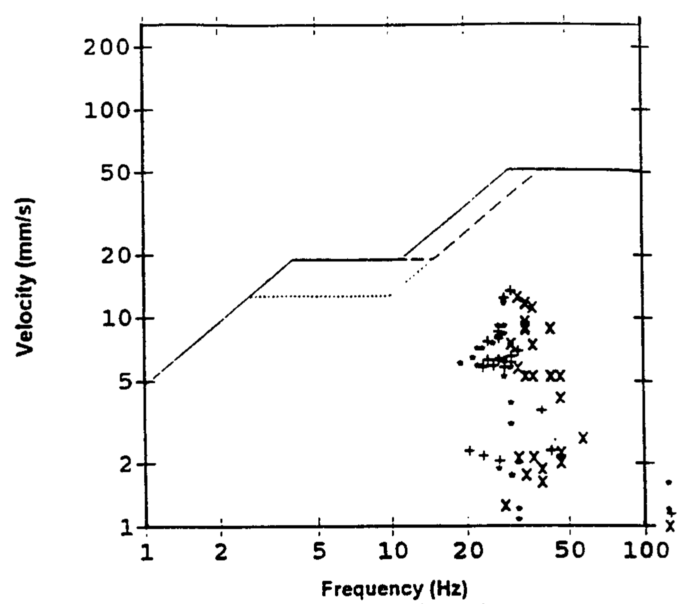
Production blast # 327, Seismograph setup at McPhail res. 258 m N. of blast

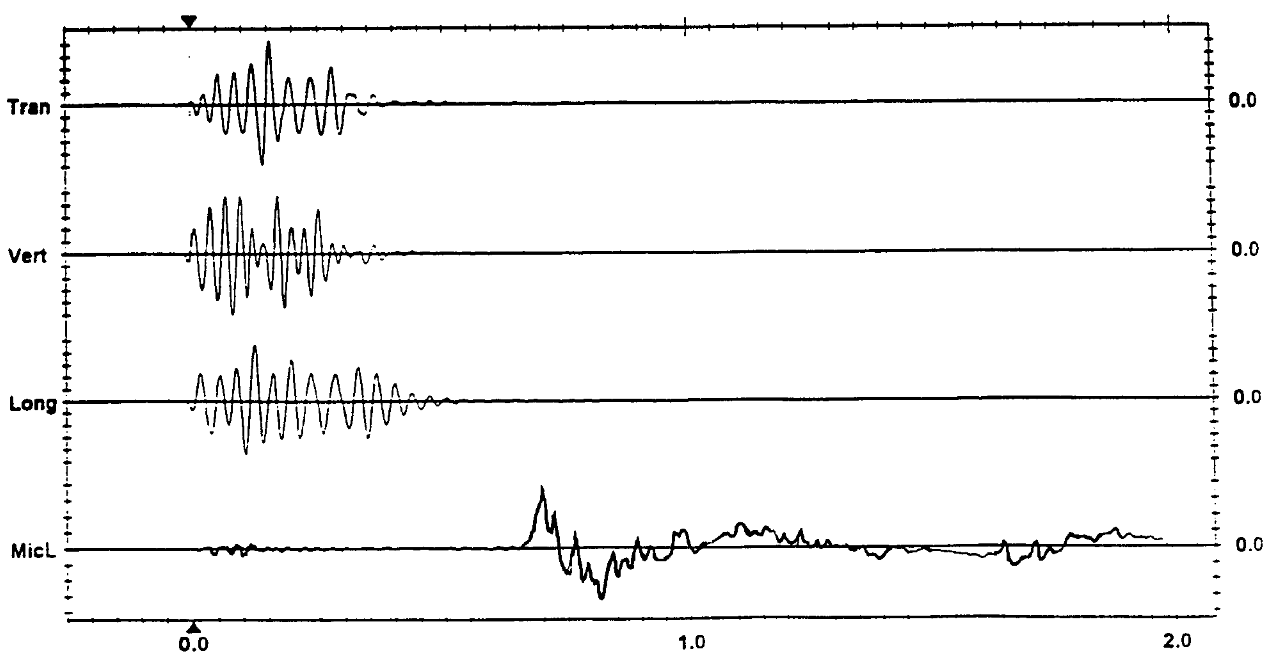
Microphone	Linear Weighting
PSP	80.50 pa(L) at 696 ms
ZC Freq	7 Hz
Channel Test	Pass (Freq = 20 Amp = 549)

	Tran	Vert	Long	
PPV	13.335	12.700	11.684	mm/s
ZC Freq	30	32	28	Hz
Time (Rel. to Trig)	155	81	123	ms
Peak Acceleration	0.32	0.29	0.23	g
Peak Displacement	0.0691	0.0619	0.0692	mm
SensorCheck ™	Pass	Pass	Pass	

Peak Vector Sum 15.621 mm/s at 156 ms

USBM RI8507 And OSMRE Analysis





Amplitude Scale: GEO: 2.500 mm/s/div MIC: 20.00 pa(L)/div
Time Scale: 50 ms/div Trigger = ►·········◄

FIGURE 32 Example of printout of portable seismograph.

TABLE 7

PORTABLE SEISMOGRAPH CHARACTERISTICS

Function	Desirable Characteristics	
Resolution	0.25 mm/s	0.01 in/sec
Range	250 mm/s	10 in/sec
	(in 5 auto ranging scales)	
Triggering levels	0.50 mm/s to 40 mm/s	0.02 to 1.5 in/sec
	(in steps of 0.25 mm/s)	(in steps of 0.001 in/sec)
Frequency response	2–250 Hz	
Sample rate	333, 500, 1000 per second	
Continuous recording time	5, 10, 15 seconds	
Pretrigger	0.05, 1.00, 1.5 seconds	
Calibration	Internal by dynamic response of geophone	
Accuracy	3%	
Internal storage	Writes on 3.5 inch, 720K disks up to 250 events	
Print time	Writes in 45 sec after event	
Modem	For remote monitoring	
Battery life	Up to 12 days	

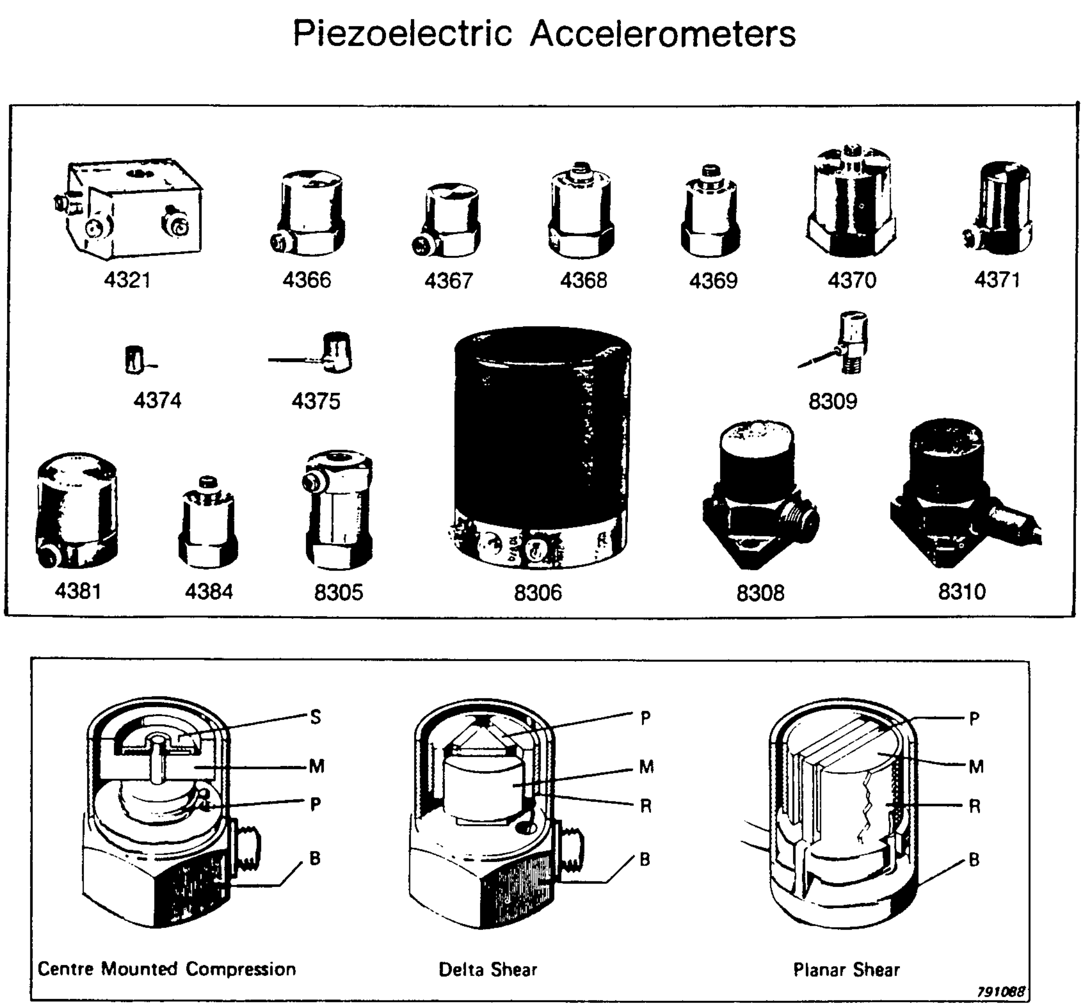


FIGURE 33 Schematic of B&K accelerometer configurations:M = seismic mass, P = piezoelectric element, B = base, R = clamping ring, and S = spring *(32)*.

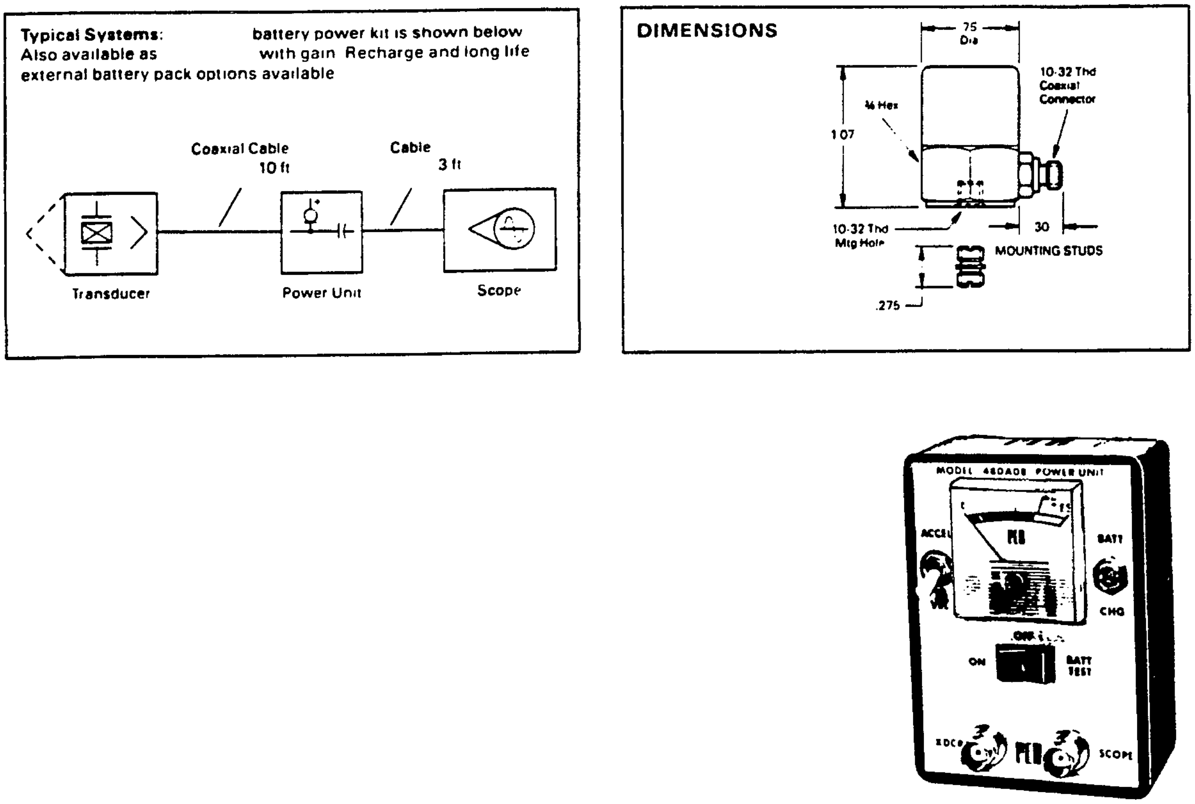


FIGURE 34 Schematic of accelerometer with single-channel power unit at right *(33)*.

TABLE 8

TRANSDUCER CHARACTERISTICS

Function	Velocity	Acceleration
Frequency range (Hz)	0.5–500	0.025–5000
Sensitivity	0.01–0.4 V/(mm/s)	0.1–10 V/*g*
Temperature range	-29 to 60° C	-50 to 120°C
Maximum range	2.3–6.0 mm	+/-10 to +/-1000*g*
Mass	56–1150 grams 0.14 to 2.3 Kg	
Cable	2 conductor w/shield	Coaxial (low capacitance)
Signal conditioners	none	Charge amplifier or constant voltage or current (power source)

the data on a floppy disk drive or in hard disk space. Computer speed and storage capability at this time are sufficient for monitoring vibrations from pile driving operations.

Vibration records from digital storage can be produced in several ways, including ink writing on plain paper at a slower rate than the recording rate, or printing on an electrostatic paper either at slow speed or directly from the transducers. Portable seismographs usually print on an electric-sensitive paper at a slower speed than real time after collecting and storing the data in digital format.

The range of means of producing permanent vibration records is very broad, so it is not appropriate to attempt to describe all methods in this synthesis, but the preceding discussion provides a starting point for interested engineers.

CHAPTER SEVEN

RESULTS OF SURVEY

To determine how several segments of the highway construction industry perceive the status of pile driving vibrations, survey questionnaires were sent to 187 entities in this industry. Most of these entities were government agencies, but others included consultants and piling contractors. Eighty-one responses (43 percent) were obtained as follows:

- 39 state DOTs,
- 7 Canadian provincial transportation agencies,
- 1 county government,
- 1 city government,
- 20 consultants, and
- 13 contractors.

A copy of the questionnaire is included as Appendix B. Most of the responses indicate that the entities have not encountered damage to structures from pile driving vibrations or have no planned response to vibration situations. Although this is good information for the evaluation of the pile driving vibration problem, it does not provide a wealth of information for those who find that these construction activities do cause problems.

The questionnaire was designed to solicit answers from three user entities: DOTs (or similar agencies for provinces, cities, and counties), pile driving contractors, and vibration consultants. Each of these entities views the pile driving vibration problem from a different perspective as was reflected in separate sections of the questionnaire.

Appendix C contains an analysis of the responses to the survey questionnaire. Tables of responses are presented for each question in the three sections of the questionnaire, A,B, and C, and the following is a summary of the salient results.

Claims for vibration damage have been leveled about equally at contractors and private and governmental owners. Consultants are not often the target of litigation, but they are called for expert testimony in litigation.

Predriving surveys of nearby buildings and homes are often prescribed for load-bearing piles and sheet piles but seldom for soldier piles or other pile applications. These predriving surveys encompass many forms and are called for by about half of the responding states. About half of the contractors perform them on their own initiative even if not required. These surveys range from a simple walk-through to detailed surveys that include videotaping, still photographs, inspection notes, strain gauges and crack gauges, survey markers, tilt monitors, inclinometers, and seismometer installations. Often these inspections are required by a contractor's insurer.

Requirements for reporting and preserving vibration records are highly variable. Some states require the report to be submitted to a state agency; others leave it in the hands of the consultant who prepared the report. Similarly, the source for retrieving records is not uniform. Sometimes state agencies, sometimes the contractor, and sometimes the consultant retains the records. Often, however, the contractor or consultant will preserve records even if not required to do so by the contract or specifications.

There is no unanimity in how to perform vibration monitoring. Some states call out specifications for the measuring equipment, transducers, and the locations where vibrations are to be measured. Others leave it to the contractor or a vibration consultant to choose the locations and equipment.

Most agencies agree that the critical measure of motion, either on the ground or on a structure, is peak particle velocity. This is also the most often cited derivative of motion limited by specifications. Whether vibrations must be measured and recorded in a manner in which frequency data can be extracted is not uniform, but from one-half to two-thirds of the agencies collecting vibration data can interpret frequency as well as amplitude of motion from those data.

Ground conditions that are most susceptible to vibration damage, as reported by respondents involved, include loose, clean, saturated sand at the top of the list, followed by shallow bedrock, cemented sand, gravel and cobbles, and high water table. Included often, although not a ground condition, was the condition of densely populated construction sites.

Special precautions or actions to mitigate against pile driving vibration problems include cast-in-place piles or drilled shafts, prebored piles, jetting, hammer/pile selection, change in driving method (impact or vibratory), minimum or nondisplacement piles such as H-piles, education of neighbors (public relations), and performing work during "off-hours." About half the contractors have in-place procedures to deal with vibration problems in the ways just cited, but there is no uniformity in how they go about implementing the precautions, or in controlling their own operations. Much depends on the sophistication represented by employees of the contractor.

Consultants, both vibration specialists and others, are engaged in pile driving vibration problems through all possible avenues. They are hired by concerned owners of nearby property, by owners of the new construction (private or government), by contractors doing the installation, and by agencies responsible for enforcing vibration statutes and specifications.

Vibration criteria applied to the pile driving vibration environment come from many sources. Many can be traced to blasting control criteria that have been developed over the past half century by mining and insurance interests. Some of the states and some consultants presented copies of specifications for vibration criteria, contract footnotes dealing with vibrations, or standard specifications as promulgated by government agencies or voluntary standards organizations. The criteria provided by respondents to the questionnaire are collected in Appendix D.

None of the criteria reported in the questionnaire were as comprehensive as those suggested by Dowding *(21)* which have been abstracted and presented as an example specification in Appendix E. Dowding's recommendations have been developed over more than 25 years of experience with blasting and other vibration problems.

CHAPTER EIGHT

MANAGING VIBRATION PROBLEMS

While it is physically impossible to eliminate vibrations from the pile driving operation, it is possible to manage operations such that vibrations do not cause damage, do not delay the completion of the job, and do not lead to claims and litigation. The first element of vibration management is a specification that gives all entities involved the guidelines under which they are expected to perform. Those specifications should include limiting vibration levels and the steps required to plan a pile driving operation within the specifications and without damage claims.

One of the most difficult aspects of managing vibrations is controlling public reactions to pile driving because of the perceived notion that a "noisy" operation must be doing some damage. Noise travels in all directions from a construction site, but weather conditions can enhance or diminish noise in certain directions from day to day. Some of this reaction can be alleviated through public relations initiatives and public education.

As pointed out in chapter 7, many DOTs have vibration specifications for blasting or for vibrations from pile driving. As the responses to the questionnaire show, these specifications take many different forms, but an ideal specification would take the best provisions from all of these and combine them. That approach has been already adopted by Dowding *(21)* in the form of a recommended specification for controlling vibration damage from blasting and pile driving operations. Dowding's specification, customized for pile driving vibrations, is presented in Appendix E for information purposes. This information may be useful to agencies that are developing their own specifications.

Some of the basic elements of a specification designed to mitigate damage due to pile driving operations consist of

- Vibration limits for damage to structures (including sensitive items housed in structures) and for interference to certain activities,
- Means for assessing predriving status of neighboring structures and facilities,
- Methods of monitoring ground and structure motion during pile driving,
- Locations of monitoring stations,
- Methods for informing residents and occupants of nearby structures about the commencement of operations,
- Surveying level net in region of pile driving, and
- Requirements for recording and preserving vibration records (recordkeeping).

One of the elements of vibration damage mitigation that is often neglected or performed incompletely is the predriving survey. One reason for this is that it is not always clear how far from the pile driving site this survey must extend. In ideal situations, with uniform soil conditions around the site and no "hard" layers to be driven through, the usual recommended distances may be adequate. However, because of unusual ground conditions, wave guide effects may take over and vibrations will transmit greater distances in some preferred direction than in all other directions. These conditions can be identified only by extensive geotechnical and geological exploration. Common radii of influence range from 60 to 150 m, but in some cases, a 400-m (0.25-mi) radius should be surveyed. Which range to use depends on the site.

Predriving surveys consist of performing inspection, photographing existing damage, videotaping existing damage, and affixing crack gauges to existing cracks in critical situations. The inspection needs to be performed by an experienced engineer who can identify potential causes of damage and relate them to observed damage. Detailed notes with sketches and photos should be made and archived for future review when necessary. In some instances, surveys of elevations and horizontal positions will be necessary to avoid damage claims. If this is required, third-party surveyors should be employed.

Public hearings are often part of any construction project, and this is an ideal place to introduce to the public the basics of pile driving vibrations and their control. An informed public is less likely to become startled upon hearing pile driving and "feeling" vibrations associated with pile driving. Much misinformation is passed around when public education is neglected. If public hearings are not conducted, meetings between the engineer, contractor, and nearby owners should be arranged to inform all concerned of the operations to take place and the consequences of those operations.

An engineer facing the potential of groundborne vibrations from pile driving should survey the site and surrounding area to observe the following conditions:

- Distance to nearest structure,
- Function of nearest structure,
- Condition of nearest structure,
- Type of piles being driven,
- Ground (soil) conditions at site and in vicinity including water levels, and
- Unusual structures, functions, or facilities nearby.

Each of these conditions should be examined in detail as described in the following:

- Consider potential direct vibration damage with peak particle velocities greater than 50 mm/sec within 15 m or a distance equal to the length of the pile in all situations. If there are no structures or facilities belonging to some entity other than the owner for whom piles are being driven, there is no danger.
- Is the nearest structure housing sensitive functions, such as hospital operating rooms, research labs with electron

microscopes, or sensitive manufacturing operations with lasers or microchips?

- Is the nearby structure well maintained and in good overall condition? Is there obvious existing damage that should be catalogued?
- Are the piles being driven as displacement piles, thereby causing maximum energy transfer to surrounding ground?
- Does the soil profile show shallow hard or dense soils that must be penetrated, or are there loose sands in the profile and if so how far laterally do they extend? (Additional borings may be necessary to check this condition.) Does the soil show any cementation?
- Do any nearby buildings house art collections or antiques, or other unusual activities that might be affected by vibrations?

After recording the answers to these questions, the engineer/inspector should determine whether a predriving inspection has been performed on all structures within the danger radius and, if so, examine a copy, and, if not, find out why not. He or she should also determine the monitoring plan if it is required and ascertain the qualifications of the agency or entity making measurements. Finally, the engineer/inspector should determine and record the place and means of storing all records relative to pile driving on that site.

Most, if not all, of these questions are contained in the example specification. Use of the items proposed in this specification may help to minimize actual and perceived damage during pile driving, but, unfortunately, it cannot eliminate all exposure to damage claims and litigation.

CHAPTER NINE

CONCLUSIONS

Vibration problems have occurred during driving of most pile types using drivers of all kinds. More than half—28 of 48—of the transportation agencies that responded to a survey questionnaire for the study reported awareness of vibration problems associated with pile driving. The level of severity of these problems ranged over a wide band, from human perception to excessive settlements that destroyed structures. Other parts of industry (mining, for example) have long recognized vibration as a source of damage, and there is little difference in damage potential of vibrations based on the source (i.e., blasting or pile driving). The main issues are proximity and amount of energy released by the source of vibrations. Pile driving operations, which transmit large amounts of energy away from the pile through the ground, may possibly damage nearby structures.

In few reported cases has there been direct damage to a structure when the pile driving was done at a distance of at least one pile length from the target. The main exception to this is associated with settlement of soils densified by vibration; such settlement can take place at greater distances than one pile length. The best way to avoid settlement damage by pile driving is to have the foreknowledge of the existence of loose, clean sands in the zone of influence (maybe up to 400 m) of pile driving. With this knowledge, the pile driving contractor or owner can plan and select driving equipment to avoid or minimize settlement of loose sand.

For impact pile driving, it is important to select a pile driver appropriate for the pile being driven. The choice of driver is generally independent of the soil type being driven into, but it is very dependent on the impedance of the pile. Using piles with low impedance increases the proportion of energy emanating from the driver, which is then translated into propagating seismic waves. Piles with high impedance can efficiently propagate energy along their length and expend energy in penetrating the ground, not shaking the neighborhood.

For vibratory pile driving, the driver should have sufficient frequency and force variability to pass through two resonances, one due to soil layers and the other due to the soil-pile-driver system. Under these circumstances the vibratory driver can be operated at the driver-pile resonance without causing significant vibrations. With variable vibrator power, the first two resonances can be passed through at low power while increasing power at the driver-pile resonance to achieve maximum penetration efficiency. When fully variable vibratory hammers are not available, more care in planning is necessary to prevent damaging vibrations caused by coincidence of frequencies.

To avoid or minimize claims and litigation originating from pile driving vibrations, it may be advisable to perform predriving surveys within an appropriate region (up to 400 m; see chapter 8) that might be influenced by vibrations. This distance is much larger than often proposed by energy/distance scaling from blasting experience. Local conditions should guide in determining that distance. Educating the property owners within the region about pile driving operations, and about the difference between perception and reality of damage due to vibrations, is a most effective deterrent to claims. Finally, selecting pile installation equipment that is compatible with the location is critical. In some locations driving of piles will not be feasible. Cast-in-place or auger cast piles may be necessary. For those situations in which driving is necessary and there is potential for vibration damage, the principles identified in chapter 8 for selecting the driver and monitoring plan may be employed.

It is a practical precaution for agencies specifying and contracting for pile driving services to develop and implement vibration specifications or contract provisions. The survey questionnaire for the study shows that many agencies have provisions addressing pile driving vibration damage, but differ widely in content and detail. Appendix E contains a recently developed example that includes many of the essential components. It might be used in developing agency-specific specifications or contract provisions, but should not be considered as all-encompassing for any one agency's needs.

It appears that damage to newly placed concrete from pile driving vibrations may not be the risk it was once thought to be. However, several specifications were found that included limiting provisions regarding pile driving near newly placed concrete. Further research into the effects of pile driving vibrations on the setting and curing of concrete may be warranted.

Vibrations due to pile driving are real and must be dealt with, but with proper knowledge it is possible to minimize vibrations, thereby minimizing potential for damage and claims in most circumstances.

LIST OF SYMBOLS

Symbol	Dimension	Explanation
a	ND	constant from cyclic shear test *(25)*
b	ND	constant from cyclic shear test *(25)*
e_r	(L)	radius of eccentricity of rotating mass vibrator
e	ND	void ratio
exp	ND	base of natural logarithm
f	(1/T)	frequency (Hz) or (cycles/sec)
g	(L/T^2)	acceleration of gravity
h	(L)	drop heights of free-falling ram
k	(L/T)	value of particle velocity at one unit of distance
m	(M)	mass of rotating elements in rotating mass vibration
m_t	(M)	mass of rotating mass vibrator + mass of clamp + mass of pile
n	ND	slope of attenuation rate
t	(T)	time
v	(L/T)	peak particle velocity
v_1	(L)	distance from source to point of known amplitude
v_2	(L)	distance from source to point of unknown amplitude
$\dot{z}$	(L)	vertical particle displacement
$\ddot{z}$	(L/T)	vertical particle velocity
z	(L/T^2)	vertical particle acceleration
A_p	(L^2)	cross-section area of pile
A_m	(L)	amplitude of displacement
A_1	(L)	amplitude at distance from source v_1
A_2	(L)	amplitude at distance from source v_2
D	(L)	distance from vibration source
D_r	ND	relative density
E_n	(LF)	energy of source
E	(F/L^2)	Young's modulus
H	(L)	soil stratum thickness
H_t	(L)	depth of barrier trench
I	(ML^3S)	impedance
K	ND	factor dependent on pile impedance
L_t	(L)	length of barrier trench
M	(LF)	static moment of rotating mass vibration
N	ND	blows per 0.3 m in standard penetration test
N_{cyc}	ND	number of cycles of shearing strain
P	(F)	force in pile from driver impact
Q	(F)	rotating force vector
R_t	(L)	distance of trench barrier from source
T	(T)	period of vibration
V_P	(L/T)	velocity of primary wave
V_R	(L/T)	Rayleigh wave velocity
V_S	(L/T)	velocity of shear wave
V_p	(L/T)	compression wave velocity in pile
W_t	(L)	width of barrier trench
α	(1/L)	coefficient of attenuation
$\Delta\hat{e}_{vol}$	ND	dynamic volumetric strain
γ	ND	shearing strain amplitude
$\gamma_{\underline{c}}$	ND	threshold shearing strain amplitude at given confirming pressure
ω	(1/T)	circular frequency (rad/sec)
λ	(F/L^2)	Lamé constant
$\gamma_{\underline{R}}$	(L)	Rayleigh wavelength
μ	(F/L^2)	Lamé constant, also shear modulus
υ	ND	Poisson's ratio
ρ	(M/L^3)	mass density
ϕ	(rad)	phase angle
π	ND	3.14159

ND = dimensionless

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APPENDIX A

PILE SUPPORT MECHANISMS RELATED TO PILE DRIVING VIBRATIONS

PILE BASICS

Piles are structural elements that transfer loads from shallow, weaker soils to deeper, stronger soils. The methods of dynamic installation of piles are covered in chapter 3 of this synthesis, but the basic mechanisms by which piles develop support and the types of members used for piles are presented here. Knowledge of these mechanisms and structural elements is important to understanding the propagation of vibrations from pile driving. A broad coverage of mechanisms of pile support and behavior is presented in *NCHRP Synthesis of Highway Practice 42: Design of Pile Foundations (34)* and the two-volume, FHWA publication *Design and Construction of Driven Pile Foundations (35,36)*, but a brief description is presented here.

Mechanisms of Support

Knowing the principles of pile support is essential for understanding pile types, pile selection, pile installation methods, and generation of vibrations during pile installation (and, in some cases, pile removal). There are two basic mechanisms for generating support from piles: *a)* skin friction or side shear, and *b)* tip or end bearing. These forms of support are illustrated in Figure A-1. These components of support are generated by relative motion between the structure of the pile and the ground supporting the pile. It is generally true that the relative movement (or strain) required to generate maximum skin friction is different from the end movement of the pile required to generate maximum end bearing. Therefore, both components of support do not become maximum at the same time. Skin friction usually develops with about 6 mm of relative motion regardless of the pile diameter, whereas end bearing develops only after a tip movement of about 10 percent of the pile diameter.

The common way to calculate allowable static pile support is to sum both skin friction and end bearing components and apply a factor of safety. Each of the two mechanisms of vertical pile support also has a role in the generation of vibrations when the piles are driven into the ground; this is covered in detail in chapter 3. A major difference between static support provided by the two components and driving resistance during installation of a pile by driving is that maximum skin friction strains are exceeded during driving before developing pile capacity and end bearing stresses are caused to exceed ultimate bearing capacity until pile capacity is achieved. During pile penetration, while both skin friction and end bearing are exceeded, energy is transmitted away from the pile by both mechanisms. After the pile is driven nearly to its capacity, one or the other of the support mechanisms becomes the dominant energy source. If the pile is end bearing, the relative motion between the skin of the pile and the surrounding soil will transmit relatively less energy than the tip of the pile, which is nearly fixed in position. If the pile is basically a floating pile, skin friction is the dominant support mechanism, and most energy will be transmitted by skin friction while the end bearing contributes relatively less energy. These differences in energy transmission may change the potential for damage to nearby structures from the pile driving operation.

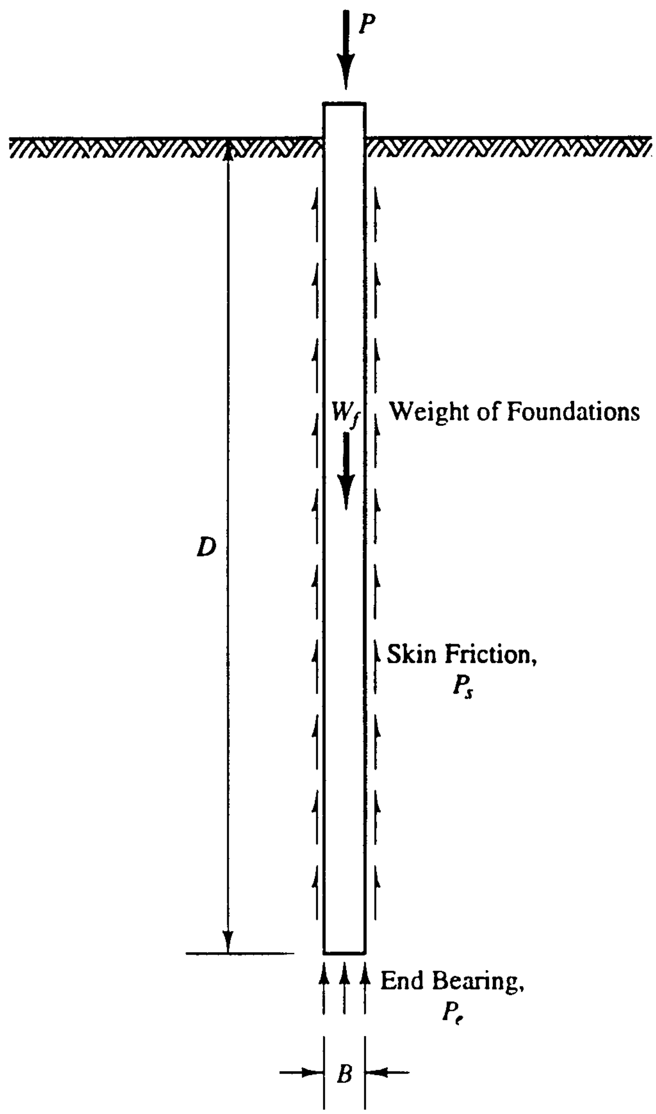

FIGURE A-1 Transfer of axial loads from a deep foundation into ground by skin friction and end bearing *(37)*.

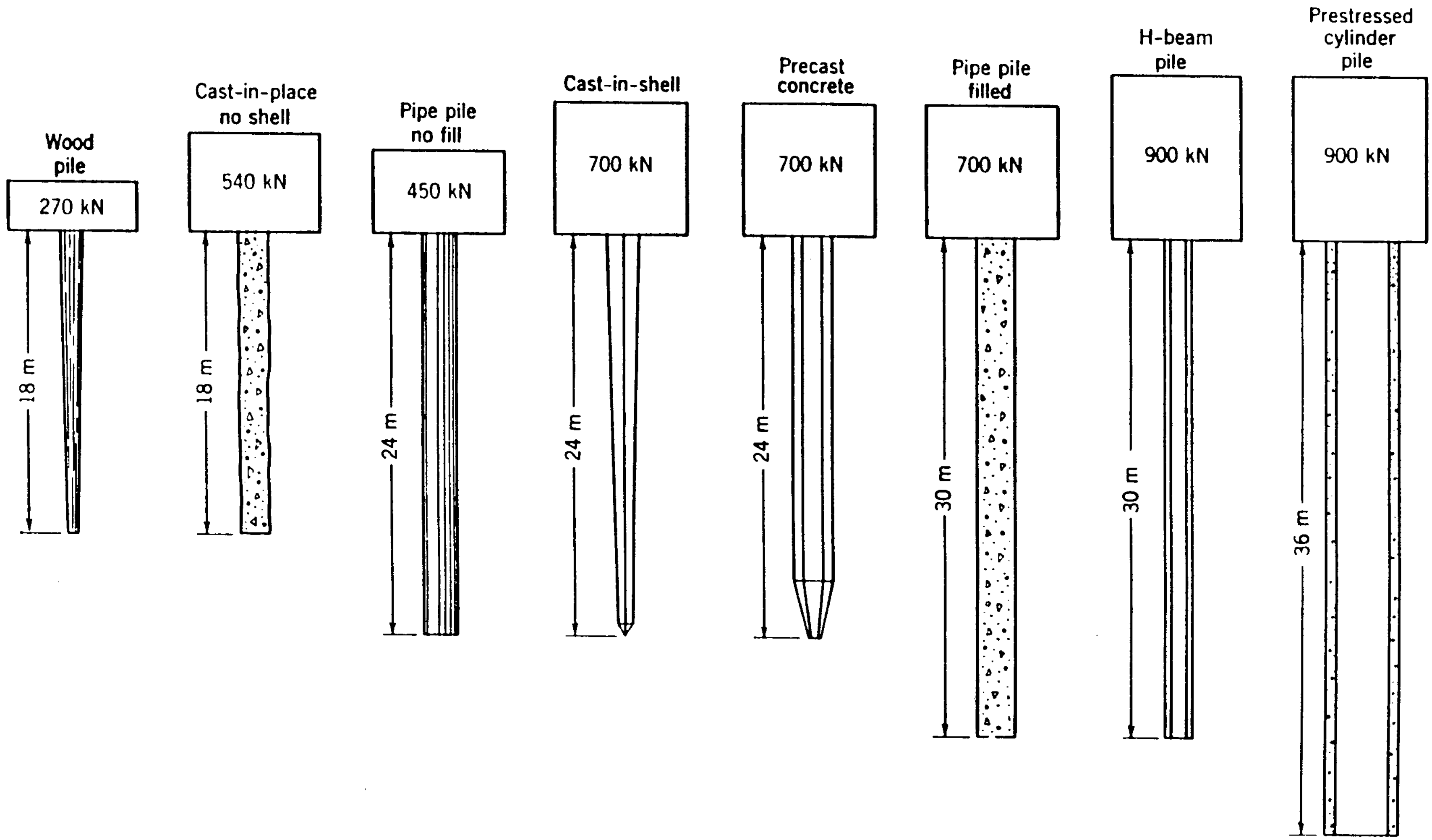

FIGURE A-2 Usual maximum length and maximum load for various piles (design values); greater lengths are common *(38)*.

Pile Structures

Chapter 3 introduced the concept of the pile property called *impedance*; it was shown that common pile structures have greatly differing impedances, so it is necessary to describe some pile types and relate impedance to pile types. Figure A-2 shows many pile types with some common dimensions. The impedance of a pile, as defined by Equation 15, requires knowledge of the compression wave velocity of the material of the pile (a function of Young's modulus and mass density) and cross-sectional area of the pile. Pile materials are commonly wood, concrete, and steel; however, the impedance of piles made of these materials will depend on how their cross sections are configured to form the pile. Timber piles generally will have the lowest impedance because the modulus of elasticity of wood is lower than that of either concrete or steel, but the cross-sectional area and shape affect the impedance to a greater extent.

The significance of pile impedance comes from the fact that the energy coupled from driven piles to the surrounding soil is dependent more on the impedance of the pile than on the type of soil into which the pile is driven (chapter 3). Pile impedences range from about 122 to 1750 (kN-sec)/m, a range of more than 14 times. For large driven concrete piles, the range may go to 20 or more times that of a timber pile. To drive a pile, the force at the top of the pile must overcome the resistance to penetration of the pile, but the impedance limits the amount of force that the pile can transmit from the top to the bottom. Therefore, the pile driver must be selected to match the impedance characteristics of the pile. Tables A-1 and A-2, from Prakash and Sharma *(39)*, list many kinds of impact hammers and vibratory hammers with their rated energies or power. Basic operating principles of the types of hammer described in Tables A-1 and A-2 are shown in Figure A-3.

VIBRATION TRANSMISSION IN GROUND

Some of the basic phenomena of wave decay or attenuation in the ground were presented in chapter 2; however, it is useful to present some additional background material for the development of Figure 5 in chapter 2 and the justification for use of the pseudo-attenuation approach for predicting pile driving vibration decay.

The basic Bornitz equation for wave attenuation was presented as Equation 10 and is repeated here for convenience:

$$A_2 = A_1(r_1/r_2)^n \exp[-\alpha(r_2 - r_1)] \quad (10)$$

where

r_1 = distance from source to point of known amplitude;
r_2 = distance from source to point of unknown amplitude;
A_1 = amplitude of motion at distance r_1 from source;
A_2 = amplitude of motion at distance r_2 from source;
n = power depending on type of wave:
= 1/2 for Rayleigh waves,
= 1 for body waves, and
= 2 for body waves at the surface;
exp = base of natural logarithm; and
α = coefficient of attenuation in units of 1/distance.

The term

$$A_2 = A_1(r_1/r_2)^n$$

represents the geometric or radiation damping behavior of the ground, and the term

$$\exp[\alpha(r_2 - r_1)]$$

represents the material damping behavior of the ground.

TABLE A-1
IMPACT PILE DRIVER HAMMER DATA[a]

Rated Energy (kip-ft)	Make of Hammer	Model Number	Type[b]	Blows per Minute (max/min)	Stroke at Rated Energy (in.)	Weight of Striking Part (kips)	Total Weight (kips)
1800.00	Vulcan	6300	S-A	38	72	300.0	838.00
300.00	Delmag	D100-13	Dies.	45/34	NA	44.894	70.435
225.00	Delmag	D80-23	Dies.	45/36	NA	37.275	58.704
200.00	Raymond	RU-200	—	40/30	40	60.0	—
180.00	Vulcan	060	S-A	62	36	60.0	121.00
165.00	Delmag	D62-22	Dies.	50/36	—	27.077	42.834
150.00	Vulcan	530	S-A	42	60	30	141.82
149.60	Mitsubishi	MH80B	Dies.	60/42	—	17.6	43.9
130.00	MKT	S-40	S-A	55	39	40.0	96.0
127.00	MKT	DE-150	Dies.	50/40	129	15.0	29.5
120.00	Vulcan	040	S-A	60	36	40.0	87.5
113.5	Vulcan	400c	Diff.	100	16.5	40.0	83.0
107.177	Delmag	D46-32	Dies.	53/37	NA	19.58	30.825
97.5	MKT	S-30	S-A	60	39	30.0	86.0
83.88	Delmag	D36-32	Dies.	53/36	NA	17.375	26.415
79.6	Kobe	K42	Dies.	52	98	9.2	22.0
70	ICE	1072	Dies.	68/64	72	10.0	25.5
68.898	Delmag	D30-32	Dies.	52/36	NA	13.472	20.704
60.0	Vulcan	020	S-A	60	36	20.0	39.0
60.0	MKT	S20	S-A	60	36	20.0	38.6

TABLE A-1 (Continued)

Rated Energy (kip-ft)	Make of Hammer	Model Number	Type[b]	Blows per Minute (max/min)	Stroke at Rated Energy (in.)	Weight of Striking Part (kips)	Total Weight (kips)
58.248	Delmag	D25-32	Dies.	52/37	NA	12.370	18.50
50.2	Vulcan	200C	Diff.	98	15.5	20.0	67.815
48.75	Raymond	150C	Diff.	115/105	18	15.0	32.5
48.7	Vulcan	016	S-A	60	36	16.2	30.2.
48.7	Raymond	0000	S-A	46	39	15.2	23.0
44.5	Kobe	K22	Dies.	52	98	4.8	10.6
44.0	MKT	MS-500	S-A	50/40	48	15.5	—
42.0	Vulcan	014	S-A	60	36	14.0	27.5
40.6	Raymond	000	S-A	50	39	12.5	21.0
39.8	Delmag	D-22	Dies.	52	NA	4.8	10.0
39.366	Delmag	D16-32	Dies.	52/36	NA	7.166	11.079
37.5	MKT	S14	S-A	60	32	14.0	31.6
36.0	Vulcan	140C	Diff.	103	15.5	14.0	27.9
33.0	Vulcan	33D	Dies.	50/40	120	7.94	—
32.5	MKT	S10	S-A	55	39	10.0	22.2
32.5	Vulcan	010	S-A	50	39	10.0	18.7
32.5	Raymond	00	S-A	50	39	10.0	18.5
32.0	MKT	DE-40	Dies.	48	96	4.0	11.2
30.2	Vulcan	OR	S-A	50	39	9.3	16.7
30	ICE	520	Dies.	84/80	71	5.07	17.04
28.1	Mitsubishi	MH15	Dies.	60/42	—	3.31	8.4
28.0	MKT	DE-33B	Dies.	50/40	126	3.3	7.75
26.3	Link-Belt	520	Dies.	82	43.2	5.0	12.5
26.0	MKT	C-8	D-A	81	20	8.0	18.7
26.0	Vulcan	08	S-A	50	39	8.0	16.7
26.0	MKT	S8	S-A	55	39	8.0	18.1
25	Vulcan	505	S-A	46	60	5.0	29.5
24.4	Vulcan	80C	Diff.	111	16.2	8.0	17.8
24.4	Vulcan	8M	Diff.	111	NA	8.0	18.4
24.3	Vulcan	0	S-A	50	39	7.5	16.2
24.0	MKT	C-826	D-A	90	18	8.0	17.7
22.6	Delmag	D-12	Dies.	51	NA	2.7	5.4
22.4	MKT	DE-30	Dies.	48	96	2.8	9.0
19.8	Union	K 13	D-A	110	24	3.0	14.5
19.8	MKT	11 B 3	D-A	95	19	5.0	14.5
19.5	Vulcan	06	S-A	60	36	6.5	11.2
19.2	Vulcan	65C	Diff.	117	15.5	6.5	14.8
18.2	Link-Belt	440	Dies.	88	36.9	4.0	10.3
18.0	Delmag	D8-22	Dies.	52/38	NA	4.0	6.147
17.0	MKT	DE-20B	Dies.	50/40	126	2.0	6.4
16.2	MKT	S5	S-A	60	39	5.0	12.3
16.0	MKT	DE-20	Dies.	48	96	2.0	6.3
16.0	MKT	C5	Comp.	110	18	5.0	11.8
15.1	Vulcan	50C	Diff.	120	15.5	5.0	11.7
15.1	Vulcan	5M	Diff.	120	15.5	5.0	12.9
15.0	Vulcan	1	S-A	60	36	5.0	10.1
15.0	Link-Belt	312	Dies.	100	30.9	3.8	10.3
13.1	MKT	10B3	D-A	105	19	3.0	10.6
12.7	Union	1	D-A	125	21	1.6	10.0
9.0	Delmag	D5	Dies.	51	NA	1.1	2.4
9.0	MKT	C-3	D-A	130	16	3.0	8.5
9.0	MKT	S3	S-A	65	36	3.0	8.8
8.75	MKT	9B3	D-A	145	17	1.6	7.0
8.8	MKT	DE-10	Dies.	48	96	11.0	3.5
8.7	MKT	9B3	D-A	145	17	1.6	7.0
8.2	Union	1.5A	D-A	135	18	1.5	9.2
8.1	Link-Belt	180	Dies.	92	37.6	1.7	4.5
8.1	ICE	180	Dies.	95/90	57.0	1.725	5.208
7.2	Vulcan	2	S-A	70	29.7	3.0	7.1
7.2	Vulcan	30C	Diff.	133	12.5	3.0	7.0
7.2	Vulcan	3M	Diff.	133	NA	3.0	8.4

TABLE A-1 (Continued)

Rated Energy (kip-ft)	Make of Hammer	Model Number	Type[b]	Blows per Minute (max/min)	Stroke at Rated Energy (in.)	Weight of Striking Part (kips)	Total Weight (kips)
6.5	Link-Belt	105	Dies.	94	35.2	1.4	3.8
0.4	Vulcan	DGH100A	Diff.	303	6	0.1	0.8
0.4	MKT	3	D-A	400	5.7	0.06	0.7
0.3	Union	7A	D-A	400	6	0.08	0.5

[a]Table revised and updated from the original table by Vesic *(34)* based on Manufacturer's catalogue data from Pileco, Inc. of Houston, TX. Vulcan Iron Works Inc., Chattanooga, TN. International Construction Equipment (ICE), Matthews, N.C., MKT Geotechnical Systems, Dover, N.J., and Raymond International Builder Inc.
[b]S-A, Single Acting; D-A, Double Acting; Diff., Differential; Dies., Diesel; and Comp., Compound.

TABLE A-2
VIBRATORY PILE DRIVER DATA *(39)* [a]

Make	Model	Total Weight (kips)	Maximum HP	Frequency (cps)	Force (kips)
Bodine (USA)	B	22	1000	0–150	63/100 to 175/100
Foster (France)	2–17	6.2	34	18–21	—
	2–35	9.1	70	14–19	62/19
	2–50	11.2	100	11–17	101/17
ICE (USA)	1412	31.7	550	6.67–20	500
	416	13.1	200	6.67–25	200
	116	4.2	94	6.67–26.67	100
Menck (Germany)	MVB22-30	4.8	50	—	48
	MVB65-30	2.0	7.5	—	14
	MVB44-30	8.6	100	—	97
MKT (USA)	V-36	18.8	550	26.67	386
	V-30	15.0	510	26.67	320
	V-20	12.5	315	28.34	214
	V-17	12.0	260	26.67	160
	V-5B	6.8	99	26.67	80
Muller	MS-26	9.6	72	—	—
(Germany)	MS-26D	16.1	145	—	—
(Russia)	BT-5	2.9	37	42	48/42
	VPP-2	4.9	54	25	49/25
	100	4.0	37	13	44/13
	VP	11.0	80	6.7	35/7
	VP-4	25.9	208		198
Tunkers	HVB 260.02	22.0	1072.8	23.34	573.04
(Germany/USA)	HVB 130.02	12.6	547.1	23.34	286.52
	HVB 60.02	7.05	288.3	29.17	132.24
	HVB 30	2.1	111.7	30	65.20
	MVB 10	2.0	42.9	35	23.60
Uraga (Japan)	VHD-1	8.4	40	16–20	43/20
	VHD-2	11.9	80	16–20	86/20
	VHD-3	15.4	120	16–20	129/20
Vulcan (USA)	Vulcan 1150	6.5	125	1600	85.6

[a] Original table by Vesic *(34)* revised and updated based on Manufacturer's Catalogue data from Pileco, Inc. of Houston, TX; Vulcan Iron Works Inc. of Chattanooga, TN; International Construction Equipment, Matthews, NC; and MKT Geotechnical systems, Dover, N.J.

Woods *(10)* has shown that for most ground vibration problems, the Rayleigh wave is the most damaging to nearby structures. This wave transmits about two-thirds of the total energy applied at the ground surface. Even if the energy is inserted at a depth in the ground, the Rayleigh wave develops quickly at the surface, as shown in Figure A-4. For a soil in which the V_p is 600 m/sec, the V_R is 250 m/sec, and a pile is being driven at a depth of 6 m, the Rayleigh wave would develop within 3 m of the pile. The significance of the Rayleigh wave generation is that these waves contain a large fraction of the total energy and travel long distances with minimal decay or attenuation. Note that in Equation 10, n for the Rayleigh wave is 1/2, while n is 2 for the body waves along the surface.

A graphical representation of the attenuation of Rayleigh waves is shown in Figure A-5. The open circles represent real field data points for amplitude decay of the vertical component of the Rayleigh wave. Three methods of mathematically modeling these data are also presented. Curve 1 represents geometric damping only, Curve 2 represents combined geometric and material damping, and Curve 3 represents a pseudoattenuation model. The pseudoattenuation model is that of Wiss *(6)*, Equation 12 of chapter 2.

The geometric damping-only curve (Curve 1) does not fit the data very well, whereas the combined attenuation curve (Curve 2) fits the data very well in this figure and in hundreds of other measurements made by the author. The pseudoattenuation line

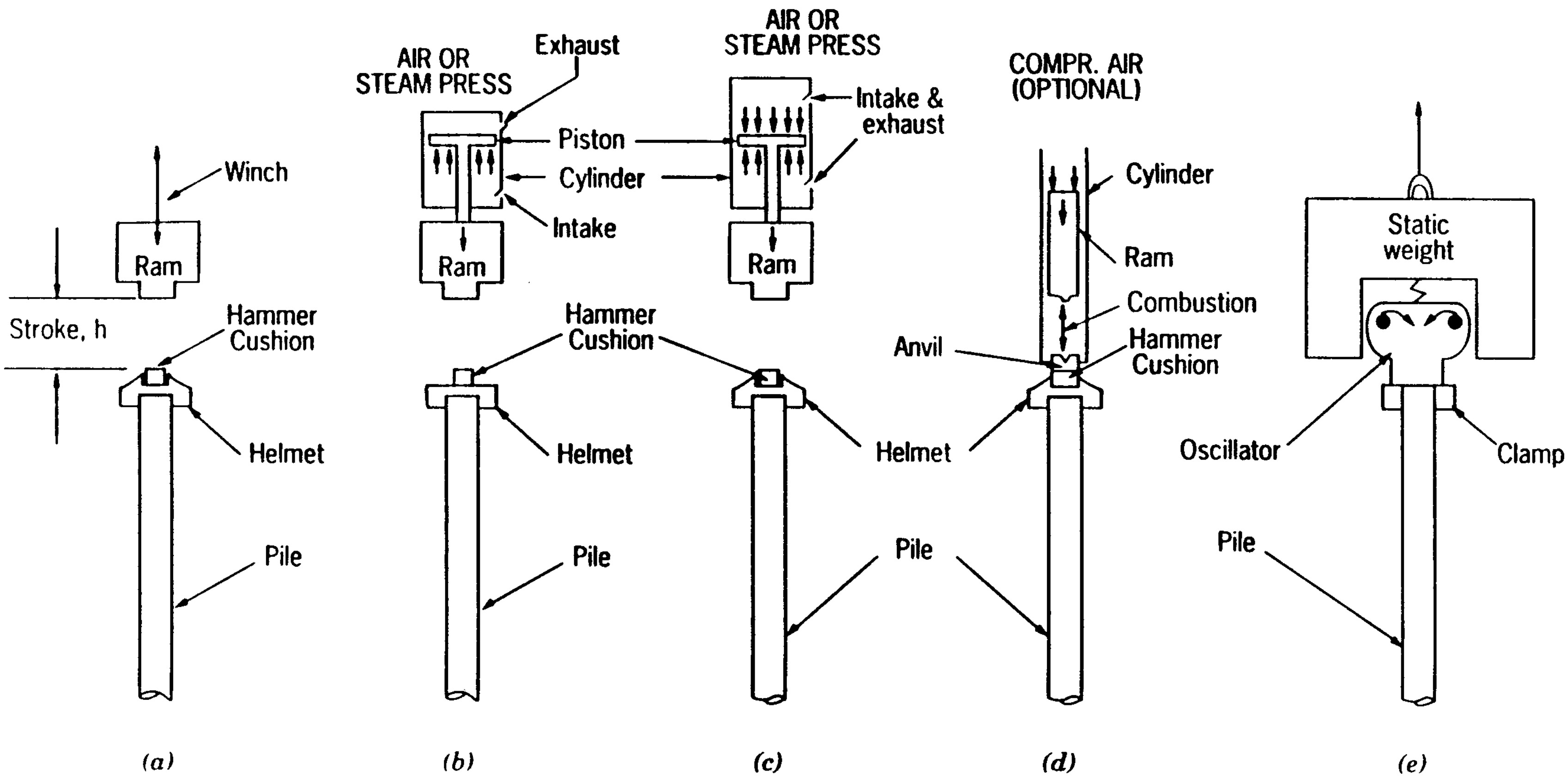

FIGURE A-3 Principles of operation of piledriving hammers: *(a)* drop hammer, *(b)* single-acting hammer, *(c)* differential and double-acting hammer, *(d)* diesel hammer, and *(e)* vibratory driver *(34)*.

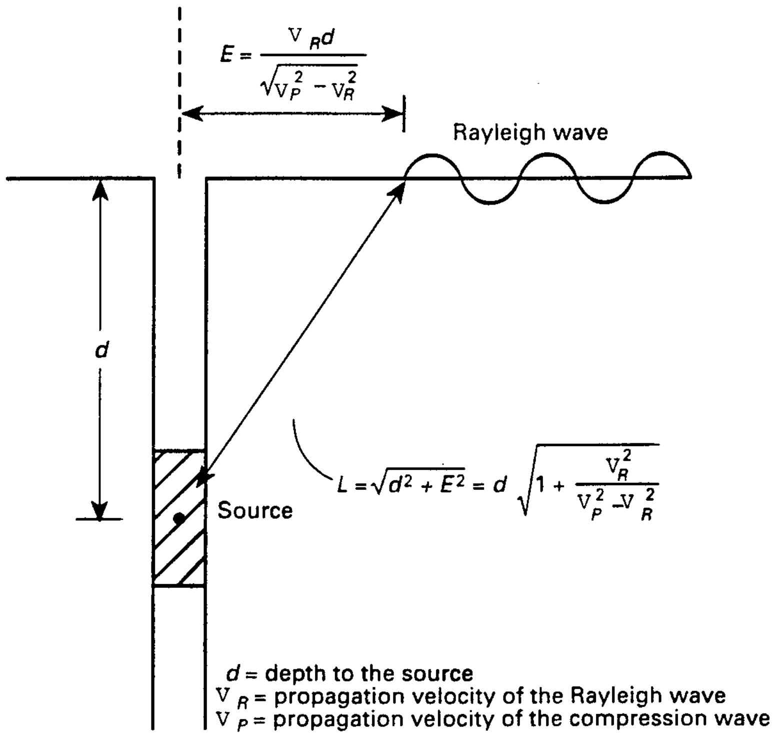

FIGURE A-4 Distance between source and reflective origin of Rayleigh waves at surface *(40)*.

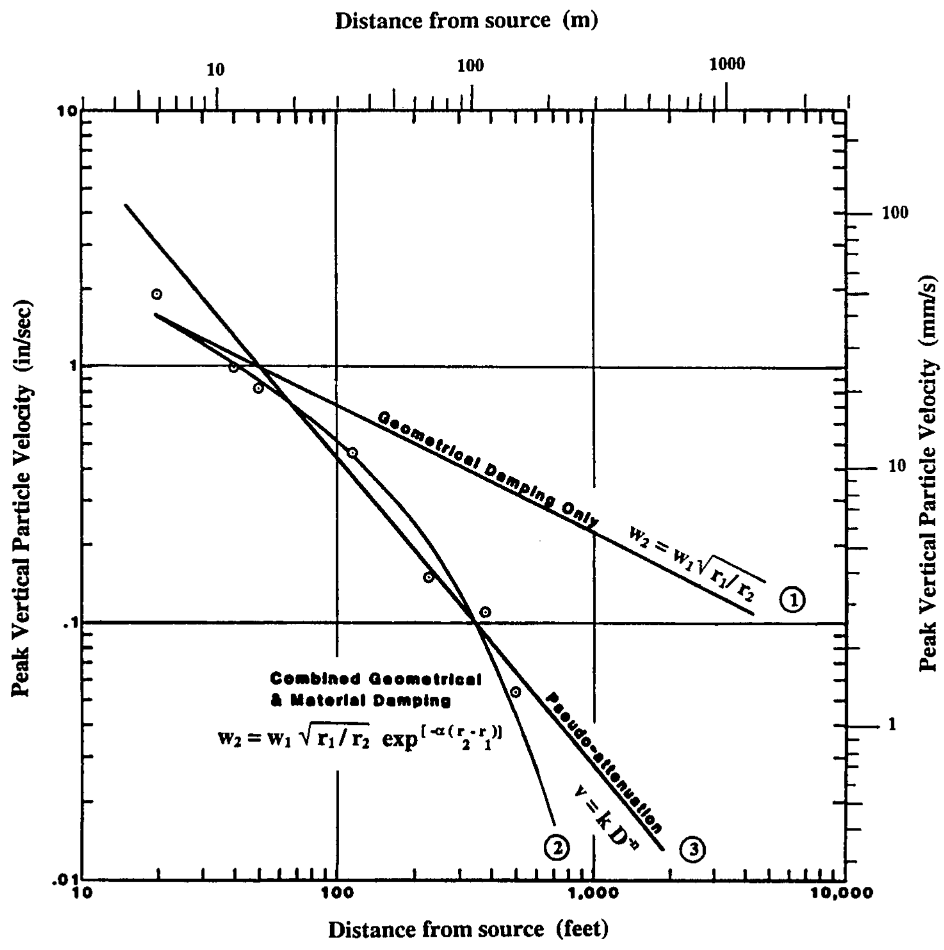

FIGURE A-5 Various forms for vibration attenuation *(5)*.

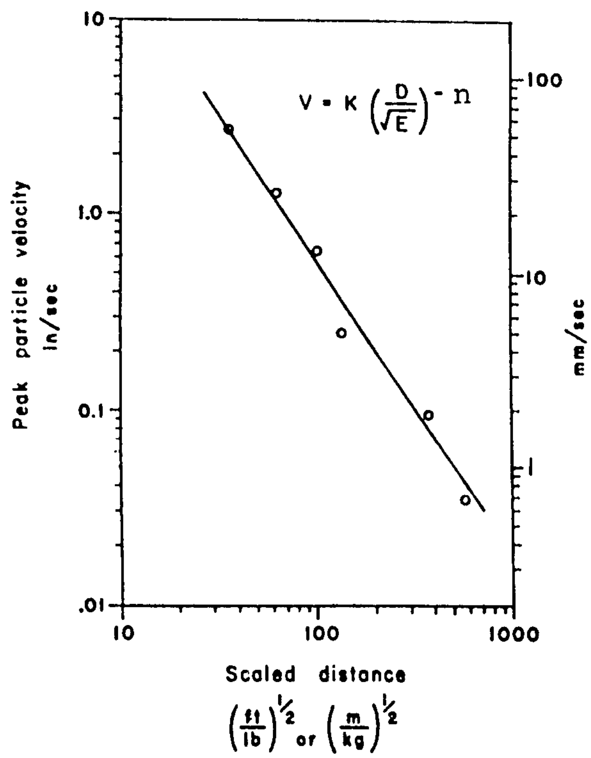

FIGURE A-6 Typical form: peak particle velocity versus scaled distance *(5)*.

(Curve 3) fits data well over a portion of the range but does not represent the data over the full distance range. However, by adjusting the slope and shifting the pseudoattenuation curve parallel to itself to the left in Figure A-5, it can very well represent the attenuation in the range of 3 to 30 m. Then, as will be seen next, this simple form of model can be used for those situations, such as pile driving, in which near-source vibration attenuation is of prime concern.

While Equation 10 and Figure A-5 are useful if the attenuation is known in advance, Wiss *(6)* suggested a model of attenuation based on the "scaled distance" in which the energy of the source can be incorporated (Equation 13). This equation is particularly useful for pile driving vibrations and is repeated here for convenience:

$$v = k\,[D/\sqrt{E_n}\,]^{-n} \qquad (13)$$

where

v = peak particle velocity,
k = value of velocity at one unit of distance,
D = distance from vibration source,
n = slope or attenuation rate, and
E_n = energy of source.

The n-rate is not classical attenuation as used in the Bornitz equation (Equation 10), but it may be considered a pseudo-attenuation coefficient. This relationship is shown in Figure A-6.

TABLE A-3
VIBRATION-ATTENUATION DATA FROM EACH SITE *(5)*

Site	Soil Type	Energy Source/ Description	Energy or Power	Frequency Range (Hz)	Attenuation Factor α (1/ft)	α * Adjusted to f = 5 Hz	Pseudo-attenuation Factor - n
M1	Sand	Drop weight/15 tons–80 ft drop	2,400,000 ft-lb	5-10.5	0.002-0.0025	0.0022	1.445
M2	Clay	Drop weight/6 tons–30 ft drop	360,000 ft-lb	8.5-17	0.0041	0.0016	1.195
M3	Sand	Vibroflot	100 Hp 30 Hp	31 31	0.0101	0.0016	1.65 1.65
M4	About 20 ft soil over bedrock	Diesel pile hammer 700 lb. hairpin	18,200 ft-lb 2,500 ft-lb	18–33 40–44	0.0319 0.0356	0.0063 0.0043	1.52 0.698
M5	Clay	Drop weight 13 ft below G.S./1.5 ton–1 and 2 ft drop	6,000 ft-lb 3,000 ft-lb	12–33 30–48	0.0102	0.0023	1.476
M6	Sand	Pump-generator	–	11–60	0.000707	0.0001	0.778
01	Clay	Drop weight/8 ton–30 ft drop	480,000 ft-lb	9–12	0.0049	0.0023	1.412
NC1	Sand	Drop weight/300 lb–5 ft drop	1,500 ft-lb	20–40	0.0103	0.0017	1.15
NC2	Sand	Dump truck driving over 3 in. high plank	7,000 ft-lb (loaded) 2,500-ft-lb (unloaded)	10–40	0.002	0.0004	0.666
Ref 42	–	1 lb of dynamite	980,000–2,050,000** ft-lb	–	–	–	1.483

*See Equation 11.
**Estimated on basis of information on page 70 of Blaster's Handbook *(42)*.

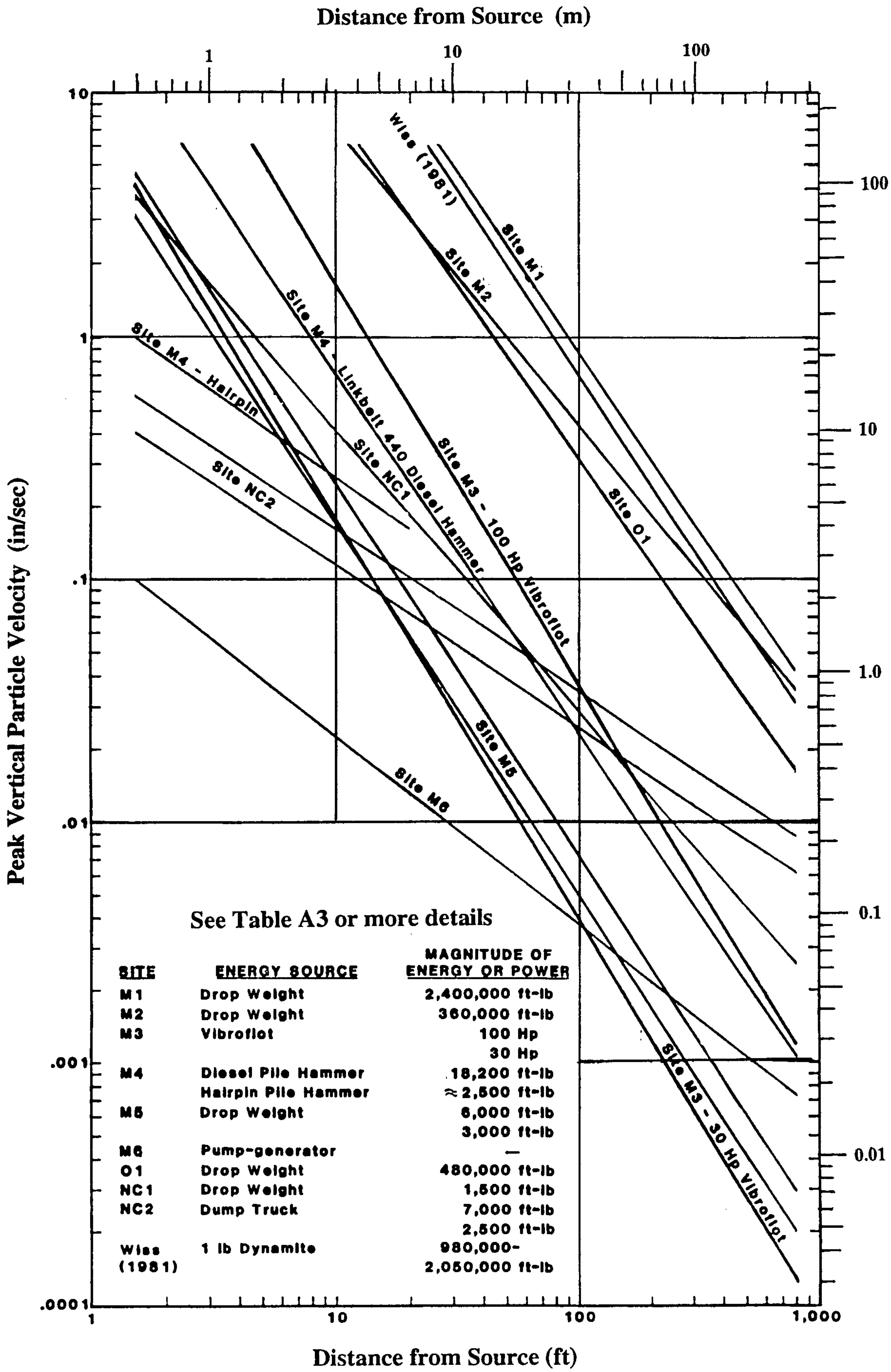

FIGURE A-7 Magnitude of construction-related vibrations *(5)*.

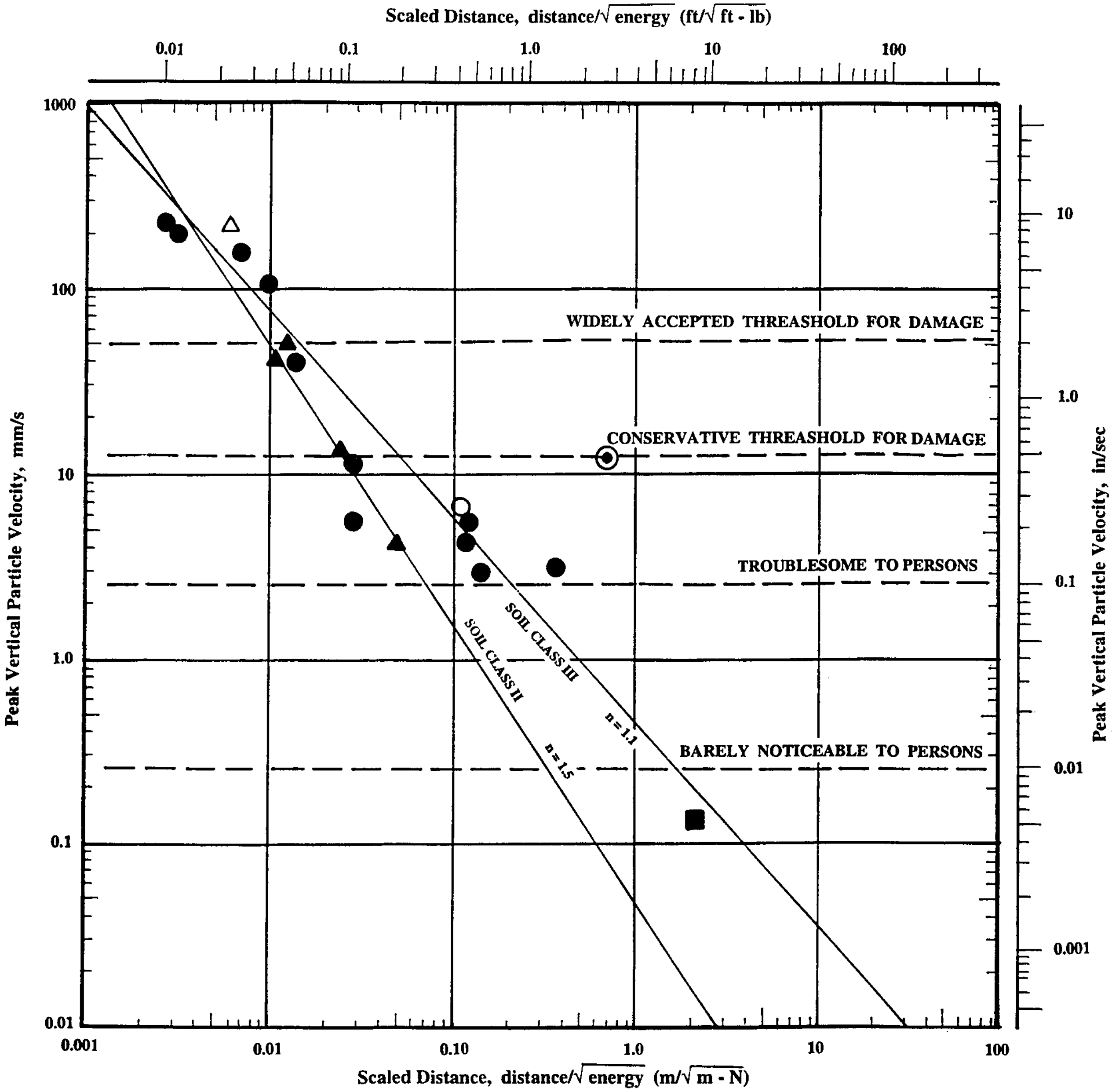

FIGURE A-8 Peak velocity versus scaled distance *(5)*.

TABLE A-4

PILE DRIVING VIBRATIONS *(41)*

Symbol	Energy Source	Soil Type	Reference	Number
●	Compressed air or steam hammer	sand	Lacy and Gould	*29*
⊙	Vibratory hammer	sand	Lacy and Gould	*29*
O	Diesel		Wiss	*6*
■	Jackhammer		Wiss	*6*
Δ	Dynamic compaction		Wiss	*6*
▲	Dynamic compaction	clayey sand, silty sand	Mayne	*41*

Woods and Jedele *(5)* gathered data from field construction projects for Table A-3 where the vibratory energy was known or could be computed and developed energy-attenuation curves (Figure A-7). Wherever possible the ground type or class as described in Table 1 of the synthesis was determined so that energy-attenuation relationships could be plotted as in Figure A-8. The ground type is included in the n-term with n = 1.5 representing the Class II soils from Table 1 (chapter 2) and n = 1.1 representing the Class III soils. Classes I and IV ground were not included in the data base mainly because construction vibrations were not commonly reported for either very poor or very good ground conditions.

To bring this work into line with pile driving vibrations, new data from several sources as shown in Table A-4 were plotted on Figure A-8. These data include pile drivers of a wide range in energy, dynamic-compaction operations, and vibrations from a single jackhammer. In most instances it was not possible to determine the soil class involved, but the data points fall generally in the range for the two attenuation lines, n = 1.5 and n = 1.1 in Figure A-8. Figure A-7 is the basis for Figure 5 in chapter 2. With Figure A-8 or Figure 5 it is possible to predict the vertical vibration amplitude on the ground at any distance from the point of pile driving as long as the hammer energy and soil type are known or can be estimated.

APPENDIX B

QUESTIONNAIRE

The questionnaire presented on the following pages was sent to 187 entities, most of whom were government agencies, but others included piling contractors and consultants. Eighty-one responses (43 percent) were obtained as follows:

- 39 state departments of transportation (DOTs),
- 7 Canadian provincial transportation agencies,
- 1 county government,
- 1 city government,
- 20 consultants, and
- 13 contractors.

Most of the responses indicated that the entities had not experienced pile driving vibration problems. While this response is good for the overall evaluation of the problem, it does not help in providing guidelines for those who find that this construction activity causes problems.

The complete questionnaire follows in this appendix, while the responses from the three basic entities—DOTs, contractors, and consultants—follow in Appendix C.

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Project 20-5, Topic 25-16

Dynamic Effects of Pile Installations on Adjacent Structures

QUESTIONNAIRE

Name of respondent: ____________________
State DOT or Other Affiliation: ____________________
Title: ____________________
Phone No: ____________________

A) Questions for State DOTs

If you are not a representative of a state DOT, skip section A) and go directly to section B)

Circle Y (Yes) or N (No) or write a short response on the lines provided.

1. Has your DOT had experience with vibration damage caused by pile driving operations?

   Y N

2. Have any of these incidents resulted in claims against the:

State	Y	N
Contractor	Y	N
Consulting engineer	Y	N
Other agency? ________	Y	N

3. Does your state have a standard specification dealing with vibrations due to pile driving operations?

   Y N (Please supply a copy of your "standard" specification if possible)

4. Does your state require a pre-driving survey of nearby structures? Y N

   If yes, does this survey include:

videotaping	Y	N	
installation of crack gages	Y	N	
photographing existing defects	Y	N	
inspection notes	Y	N	
other?	Y	N	if yes, please describe

____
____
____

NCHRP Project 20-5, Topic 25-16
Agency: ____________________

5. Who collects this record?

state	Y	N
contractor	Y	N
other?	Y	N

____
____

6. Does your specification cover:

load carrying piles	Y	N
sheet piles	Y	N
other piles?	Y	N

7. Does state require continuous monitoring of ground motion while pile driving is on-going?

   Y N

   or intermittent monitoring?

   Y N

8. Does state specify location(s) where ground motion measurements are to be made?

   Y N

   If yes, how is location prescribed?

   ____
   ____

9. May contractor do monitoring with their own personnel?

   Y N

10. Must an independent third party be employed to do monitoring?

    Y N

11. Must a state inspector observe all monitoring?

    Y N

12. Must a record of vibrations due to pile driving be submitted to the state?

    Y N

13. What are the state's vibrations reporting requirements?

____
____
____

14. Does your specification limit:

Peak particle displacement	Y	N
Peak to Peak particle displacement	Y	N
Peak particle velocity	Y	N
Peak to Peak particle velocity	Y	N
Peak particle acceleration	Y	N
Peak to Peak particle acceleration?	Y	N

15. Does your specification have any frequency reporting requirements?

Y N

16. Are the records of vibrations exceeding specifications available to the public?

Y N

17. How are these records obtained?

____
____
____

18. Does your state permit the use of vibration attenuation measures like wave barriers (e.g. trenches, broken rock zones, sheet-pile walls, etc)?

Y N

19. Are there specific geologic profiles or other site conditions which prevail in your state that exacerbate pile driving problems?

Y N if yes, please describe:

____
____
____

20. If excessive vibrations occur due to pile driving, can the contractor on his/her own initiative switch to an alternative driving approach, e.g. change rated energy of hammer, change from driving to vibratory installation, change from compressed air hammer to diesel hammer, etc.).

Y N if yes, please explain: **(next page)**

NCHRP Project 20-5, Topic 25-16
Agency: ____________________

____
____
____

21. Has experience in your state led to development of any unique criteria or ways to deal with pile driving vibration problems?

Y N if yes, please explain:

____
____
____

B) Questions for Pile Driving Contractors.

<u>If you do not represent a pile driving contractor or state DOT, go to section C)</u>

Circle Y (Yes) or N (No) or write a short response on the lines provided

1. Has your firm been involved in situations where pile driving vibrations have been an issue either because of real damage or perceived damage?

Y N

Driving load bearing piles	Y	N
Driving sheet-piles	Y	N
Driving soldier piles?	Y	N

2. Has your firm experienced problems associated with generation of vibrations during pile driving (e.g. delays, expenses, law suits)?

Y N

3. Do you have a standard operating procedure with regard to vibrations created by pile driving?

Y N

4. If you are required to monitor vibrations, do you:

rent equipment and do it yourselves	Y	N
hire a vibration monitoring firm (like VME or similar)	Y	N
hire a vibration consultant?	Y	N

5. Does your firm routinely perform a pre-driving survey of surrounding structures before starting pile driving?

Y   N   If yes, answer 6., otherwise go to 7.

6. Please describe the type of survey which you perform?

____
____
____

7. What options have you used or considered to mitigate alleged vibration damage:

Change energy of hammer	Y	N
Change hammer type	Y	N
Use vibration barrier?	Y	N

8. What, if any, approaches to pile driving have you adopted in situations where vibrations are a problem?

____
____
____

9. Which have been effective?

____
____
____

10. Does your firm maintain records of vibration measurements:

always	Y	N
only when required by specifications	Y	N
leave in possession of vibration firm or consultant	Y	N

C) Questions for Vibrations Measuring Firm and/or Vibration Consultant.

Circle Y (Yes) or N (No) or write a short response on the lines provided

1. Respondent is representative of:

vibrations measurement firm	Y	N
vibrations consultant?	Y	N

NCHRP Project 20-5, Topic 25-16
Agency: ____________

2. Have you ever measured vibrations caused by pile driving:

load bearing piles	Y	N
sheet piles	Y	N
other? ________	Y	N

3. In instances where you have measured vibrations from pile driving, were you engaged by: (include all applicable)

owner of new construction	Y	N
contractor	Y	N
victim of alleged vibration damage	Y	N
other? ________	Y	N

4. In those cases where you have measured vibrations from pile driving operations, specifications for vibration level were:

spelled out in the contract documents	Y	N
non-existent	Y	N
left up to you to establish	Y	N
other? ________	Y	N

5. Do you obtain vibrations from which frequency data can be extracted:

always	Y	N
only when specified	Y	N
never?	Y	N

6. Do you maintain duplicates of all vibration measurements:

always	Y	N
only when required by specifications	Y	N
only when required by you client	Y	N
never?	Y	N

7. From your experience in measuring vibrations from pile driving operations, have you found any universally applicable principles which can be used to minimize the hazard of vibrations from pile driving or minimize the exposure to imagined damage from such operations? Please describe:

____
____
____

APPENDIX C

QUESTIONNAIRE RESPONSES

Part A: Transportation Agencies (United States and Canada)

Questions A1, A2, and A3 of the questionnaire provided the separation between agencies that had experienced the pile driving vibration problem and those that did not. In the following table, agencies that had both problems and specifications, agencies that had vibration problems and no specifications, and agencies that used other related but not pile driving controls are reported separately.

Group A responses are from those agencies (9) that have pile driving vibration problems *and* that have a standard specification for control of pile driving.

Group A′ agencies (7) have had claims due to pile driving vibrations but do not have a reporting or controlling specification.

Group B agencies (12) have had claims due to pile driving vibrations, did not have a specification for control of pile driving vibrations, but have *required* vibration monitoring and reporting in special situations.

TABLE C-1

TRANSPORTATION AGENCY RESPONSES TO QUESTIONNAIRE—PART A

	Group		
	A	A′	B
1. Has your DOT had experience with vibration damage from pile driving operations?	9	7	12
2. Have any of these resulted in claims against:			
state	6	6	12
contractor	6	7	9
consulting engineer?	1	1	2
3. Does your state have a standard specification dealing with vibrations due to pile driving operations?	9	–	–
4. Does your state require a pre-driving survey of nearby structures?	6		3
Does it include			
video-taping	5		10
installation of crack gauges	2		6
photographing existing defects	7		11
Inspection notes	7		11
Others: tilt gauges and settlement monitoring?	–		4
5. Who collects this record?			
state	5		9
contractor	7		9
consultant	4		5
6. Does your specification cover:			
load carrying piles	8		9
sheet piles	5		9
other piles?	–		5
7. Does your state require monitoring of ground motion while pile driving is on-going?			
continuous	5		8
intermittent	0		5
no monitoring	4		–
8. Does your state specify location(s) where ground motion measurements are to be made?	4		9
(others leave it to contractor or consultant to decide)			
9. May contractor do monitoring with their own personnel?	8		7
10. Must an independent, third party be employed to do the monitoring?	2		3
11. Must a state inspector observe all monitoring?	4		5
12. Must a record of vibrations due to pile driving be submitted to the state?	5		8

TABLE C-1 (Continued)

	Group		
	A	A′	B
13. What are the state's vibrations reporting requirements?	–		–
	[reporting requirements not clear, confusing overall, varies with job, only when velocity over 12 mm/sec, or according to AASHTO (R-8-81 (1990)]		
14. Does your specification limit:			
Peak particle displacement	2		1
Peak-peak particle displacement	–		1
Single peak particle velocity	7		7
Peak-peak particle velocity	–		2
Peak-peak particle acceleration?	–		1
15. Does your specification have any frequency reporting requirements?	4		3
16. Are the records of vibrations exceeding specifications available to the public?	4		7
17. How are these records obtained? (Varies with all states, no uniformity; examples: from regional offices, contact project engineer, on request to headquarters.)	–		–
18. Does your state permit the use of vibration attenuation measures like wave barriers (trenches, broken rock zone, sheet-pile walls, etc.)?	6		7
	(While some say barriers can be used, barriers are seldom used.)		
19. Are there specific geologic profiles or other site conditions prevalent in your state that exacerbate pile driving problems?	6		–
	(All report loose sands as a problem profile, also, dense sand, cobbles, boulders, gravel in deep alluvial deposits at bridge sites, and shallow friction piles in clayey silt and silts, land shallow rock.)		
20. If excessive vibrations occur, can contract or switch driving method?	7		–
21. Has experience in your state led to development of any unique criteria or ways to deal with pile driving vibrations problems?	–		–
	(jetting, pre-boring, non-displacement piles, drilled shafts, hammer energy control)		

Note: Numbers represent agencies responding "yes."

PART B: PILE DRIVING CONTRACTORS

TABLE C-2

CONTRACTOR RESPONSES TO QUESTIONNAIRE—PART B

	Yes Responses
1. Has your firm been involved in pile driving where vibrations have been an issue either because of real or perceived damage?	
while driving bearing piles	11
while driving sheet-piles	8
while driving soldier piles	7
2. Has your firm experienced problems associated with generation of vibrations during pile driving including: litigation, delays, other problems?	6
3. Do you have a standard operating procedure with regard to vibrations created by pile driving?	8

TABLE C-2 (Continued)

	Number of Yes
4. If you are required to monitor vibrations, do you:	
rent equipment and do it yourselves	5
hire a vibration monitoring firm	10
hire a consultant?	6
5. Does your firm routinely perform a pre-driving survey of surrounding structures before pile driving?	5
6. Describe types of surveys:	
walk-through with engineer	>1
physical assessment by consultant	>1
inspection by insurer or own engineer	>1
video taping	>1
7. What options have you considered to mitigate alleged vibration damage?	
change energy of hammer	13
change hammer type	13
used vibration barriers	0
8. What, if any approaches to pile driving have you adopted in situations where vibrations are a problem and which have been effective?	0
pre-auger part way, auger cast piles	
vibratory driving to bearing layer, then impact	
deep mixed soil piles, work "off hours"	
replace sheet piles with soldier piles and lagging	
9. Does your firm maintain records of vibration measurements?	
always	3
only when specifications require	10
leave in possession of vibrations consultant	3

Note: 13 contractors responded.

PART C: VIBRATION MEASUREMENT FIRMS AND VIBRATION CONSULTANTS

TABLE C-3

VIBRATION FIRM RESPONSES TO QUESTIONNAIRE—PART C

	Yes Responses
1. Respondent is representative of:	
vibrations measurements firm	13
vibrations consultant	14
2. Have you ever measured vibrations caused by pile driving for:	
load bearing piles?	17
sheet piles?	17
other (Franki, mini-piles, H-piles, soldier piles)	9
3. In instances where you have measured vibrations from pile driving, were you engaged by:	
owner of new construction	15
contractor	15
victim of alleged vibration damage	7
other (geotech firms, const. managers, building auth. concerned parties, research and development)	18
4. In those cases where you have measured vibrations from pile driving operations, specifications for vibration level were:	
spelled out in contract documents	13
non-existent	10
left up to you to establish	18
federal or state criteria	4

TABLE C-3 (Continued)

	Yes Responses
5. Do you obtain vibration records from which frequency can be extracted?	
always	11
only when specified	7
never	1
6. Do you maintain duplicates of all vibration measurements?	
always	9
only when required by specifications	4
only when required by client	3
never	1

7. From your experience in measuring vibrations from pile driving operations, have you found any universally applicable principles that can be used to minimize the hazard of vibrations from pile driving or minimize the exposure to imagined damage from such operations. Please describe. NOTE: SI units not used in the following as these are responses presented in the questionnaire:

- Vibrations not significant at distance greater than pile length.
- Vibrations at distance from pile greater than 11 m not damaging to green concrete.
- Human perception of vibrations is greatest problem.
- To help reduce vibrations, determine the ground response spectrum, compute pile impedance, determine where energy is likely to be initiated.
  Good PR lessens claims, particularly when home inspections are thorough.
- Multiple seismographs to get good attenuation data, show that vibrations are felt where damage is not happening.
- Do not use or specify driven piles!!
- Usually vibration amplitudes are less than 0.5 mm/sec at a distance of 23 or more meters from the driving.
- PR important, settlement most serious problem, pre-driving survey a must.
- Use pneumatic or hydraulic hammer in urban areas.
- Use H-piles with low stroke hammer.
- Pre-driving education, measure out to 2.5 mm/sec, limit vibration strains to environmental strains.
- Pre-auger holes.
- No direct damage under 100 mm/sec, repetitive vibrations may cause cosmetic damage at lower vibration levels.
- Primary problem is densification of loose sands at 0.25–2.5 mm/sec at 10–100 Hz.
- Measure building settlement and horizontal displacement.
- Total number of piles driven on a site is important (case cited where serious settlement did not occur until after 200 piles had already been driven.)

In above responses, 6 cited Public Relations (PR) as critical, 5 cited pre-driving surveys, 2 cited pre-boring of holes, and 3 cited loose sands as cause of settlements. There were also 3 citations that gave distances at which vibrations were no longer a problem: distance equal to pile length, 50 ft and 75 ft.

APPENDIX D

Vibration Criteria

Vibration criteria were included with the responses to the questionnaire from seven states, two provinces of Canada, and a German DIN. These criteria are presented here as representative of kinds of criteria that are in use now and for comparison with the "Example Vibration Criteria" presented in Appendix E.

FLORIDA

A455-3 General Requirements

A455-3.1 Protection of Existing Structures—When the plans require pile driving operations in close proximity to existing structures, the Contractor shall take all reasonable precautions to prevent damage to such structures. The requirements described herein apply to all types of structures (on or off the right of way) that may be adversely affected by foundation construction operations (including phase construction) due to vibrations, ground loss, ground heave, or dewatering. Utilities shall be protected as described in 7-11.6.

The Contractor shall monitor adjacent structures for settlement in an approved manner, recording elevations to 0.001 foot, during driving when the pile driving operations are required within a distance, in feet, equal to 0.5 times the square root of the hammer energy, in foot-pounds or the distance shown in the plans. Required measurements shall be taken before the initiation of driving and then daily on days when driving occurs or as indicated in the plans and weekly for two weeks after driving has stopped.

In addition, when pile driving operations occur within a distance, in feet, equal to 0.25 times the square root of the hammer energy, in foot pounds, or the distance shown in the plans, the Contractor shall engage the services of a qualified Professional Engineer registered in the State of Florida to conduct a survey of all (except as noted herein) structures, or portions thereof, within this distance before pile driving begins and again after all pile driving is completed. The Department will make the necessary arrangements for entry by the Contractor's engineer in the survey. The condition of the structures shall be adequately documented with descriptions and pictures. All existing cracks shall be thoroughly documented. Two reports shall be prepared documenting the condition of the structure; one report before driving begins and a second report after driving is complete. Both reports shall become the property of the Department. Pre-driving and post-driving surveys of the condition of bridges owned by the Department will not be required except when shown in the plans or Special Provisions.

When shown in the Contract Documents, the Contractor shall also engage the services of a qualified Professional Engineer registered in the State of Florida to monitor and record vibration levels during the driving operations. Vibration monitoring equipment shall be capable of detecting velocities of 0.1 inch/second or less.

When shown in the Contract Documents or when authorized by the Engineer, the Contractor shall install the piling to the depth required to minimize the effects of vibration or ground heave on adjacent structures by approved methods other than driving (preformed holes, predrilling, jetting, etc.) In the even that preformed pile holes are authorized to meet this requirement, payment for this work shall be as described in A455-3.

Piles shall not be driven within 200 feet of concrete less than two days old unless authorized by the Engineer.

When the plans require excavations for construction of footings or caps supported by piling, the Contractor shall be responsible for evaluating the need for, design of, and providing any necessary features to protect adjacent structures. Sheeting and shoring shall be constructed according to plans provided by the Contractor except when the sheeting and shoring are detailed in the plans. Sheeting and shoring installed to protect existing structures shall be designed by a Professional Engineer, employed by the Contractor, registered in the State of Florida and who shall sign and seal the plans and specification requirements. Plans and specifications for sheeting and shoring provided by the Contractor shall be sent to the Engineer for his record before construction begins.

Existing structures within a distance of three times the depth of excavation for the footing shall be monitored for movement. The number and location of monitoring points shall be as approved by the Engineer. Elevations shall be taken before the driving of any sheeting, daily during the driving of any sheeting and during excavation, read and recorded to 0.001 foot. The Contractor shall notify the Engineer of any movements detected and immediately take any remedial measures required to prevent damaging the existing structure.

When shown in the plans or directed by the Engineer, the Contractor shall install a piezometer near the right of way line and near any structures that may be affected by lowering the ground water when dewatering is required. The piezometer shall be monitored and the ground water elevation level recorded daily. The Contractor shall notify the Engineer of any ground water lowering near the structure of one foot or more.

At any time the Contractor detects settlement or heave of 0.005 foot, levels of vibration reaching 0.5 in./sec, the level otherwise shown in the Contract Documents, or damage to the structure, he shall stop driving immediately and notify the Engineer for instructions.

LOUISIANA

Transportation Related Earthborne Vibrations

- Use AASHTO DESIGNATION: R 8-81 (1990) (following 4 pages)

Standard Recommended Practice for Evaluation
of

Transportation Related Earthborne Vibrations

AASHTO DESIGNATION: R 8-81 (1990)

1. SCOPE

1.1 This recommended practice is to provide guidance for the assessment of potential or alleged structural damage due to earthborne vibrations related to transportation facility construction, maintenance or operations.

2. INTRODUCTION

2.1 In the field of vibration measurement, literature indicates that earth particle velocity adjacent to the building is the statistic that should be considered. Much of the data concerning structural damage that have been collected through the years, have been related to blasting, an activity that produces vibration pulses that are infrequent, and may be of considerable magnitude. Structural damage due to blasting has been documented to a greater extent than for other sources of vibration.

2.2 Vibrations due to transportation related activities are far more restricted in the dimensions of the area affected, but may produce many more cycles of application than blasting. There is concern on the part of many authors for fatigue and "triggering" effects of transportation related vibration, but documentation of such damage is scarce. There is considerable controversy over the limiting amount of vibration that should be permitted. Extreme sensitivity of people to low level vibrations is a complicating factor in all situations where people and their properties are unwillingly subjected to vibration.

2.3 Practically any building contains numerous fine cracks that are only evident through very close examination. Experience has shown that when local residents are subjected to construction vibrations, many of them assume that vibrations that can be felt may also damage the building. A close examination of the house then reveals cracks, and the person is convinced that the cracks are a result of the vibration. Such cases then lead to complaints, litigation and sometimes to court orders to halt the construction operations. Consequences of such action to the transportation authority are obvious. Therefore, it seems reasonable to take action early to prevent such situations wherever possible. Measurements should be made at the start of pile driving or pavement breaking operations that are within a few hundred feet of dwellings, to determine the effects of the operations under the conditions that exist. Judgments then must be made concerning the risk of damage, and sometimes concerning the risk of concerted action to halt a project. Therefore, it may be advisable to alter the construction operations at localized sites, even though damage probabilities appear to be small, in order to keep the project on schedule. Predrilling or jetting of piles, saw and/or lift out operations for pavement removal, may get the job done at some additional cost, while preventing adverse public relations and possible legal action.

3. DEFINITIONS

3.1 *Threshold Damage*—Opening of old cracks and formation of fine new cracks in plaster; dislodging of loose materials such as plaster or bricks.

3.2 *Architectural or Minor Damage*—Non-structural damage that does not affect the strength or function of the building; for example; cracked plaster or wallboard, cracked or broken windows, hairline cracks in masonry walls.

3.3 *Major Damage*—Structural damage resulting in serious weakening of the building, for example, major settlements or shifting of foundations, distortion or weakening of the superstructure, large cracks in foundation or bearing walls, walls out of plumb.

3.4 *Limiting Velocity*—Maximum vibration level not to be exceeded in order to prevent damage.

4. EQUIPMENT

4.1 Transducers: Velocity sensing transducers should be used, with a minimum of 2 channels and 3 mutually perpendicular axes per channel. Frequency response should cover the range from less than 5 Hz to more than 100 Hz. Sensitivity should range from below 0.001 in./sec to more than 1.0 in./sec (less than 0.02 mm/sec to more than 25 mm/sec).

4.2 Associated electronic gear (power supplies, amplifiers, logic circuitry): Electronics should be compatible with the transducers, and should contain capability for field calibration. Suggested but not required, are the capabilities for vectorially summing the three components of velocity $\hat{V} = \sqrt{V_x^2 + V_y^2 + V_z^2}$, and also for holding and digitally displaying the maximum resultant signal that has been measured. (See appendix, reference No. 19.)

4.3 Tape recorder or chart recorder.

5. PROCEDURE

5.1 On-site inspections should be made to determine the nature of the problem, vibration source, transmission path, affected structures, and any other subjective or objective factors. Also, sketches or photographic records may be made, if required for files and reports. In specialized cases where poor support conditions are suspected under the structure, deep soil sampling may be required, along with reference soil penetration data.

TABLE 1 Maximum Vibration Levels for Preventing Damage: Intermittent Excitation on Transportation Construction or Maintenance

Type of Situation	Limiting Velocity in./sec (mm/sec.)
Historical sites, or other critical locations	0.1 (2.54 mm/sec.)
Residential buildings, plastered walls	0.2 to 0.3 (5.08 - 7.62 mm/sec.)
Residential buildings in good repair with gypsum board walls	0.4 to 0.5 (10.16 - 12.70 mm/sec.)
Engineered structures, without plaster	1 to 1.5 (25.4 - 38.10 mm/sec.)

Note that public relations for minimizing complaints and legal action would require maximum values in the vicinity of .05 in./sec. (1.27 mm/sec.).

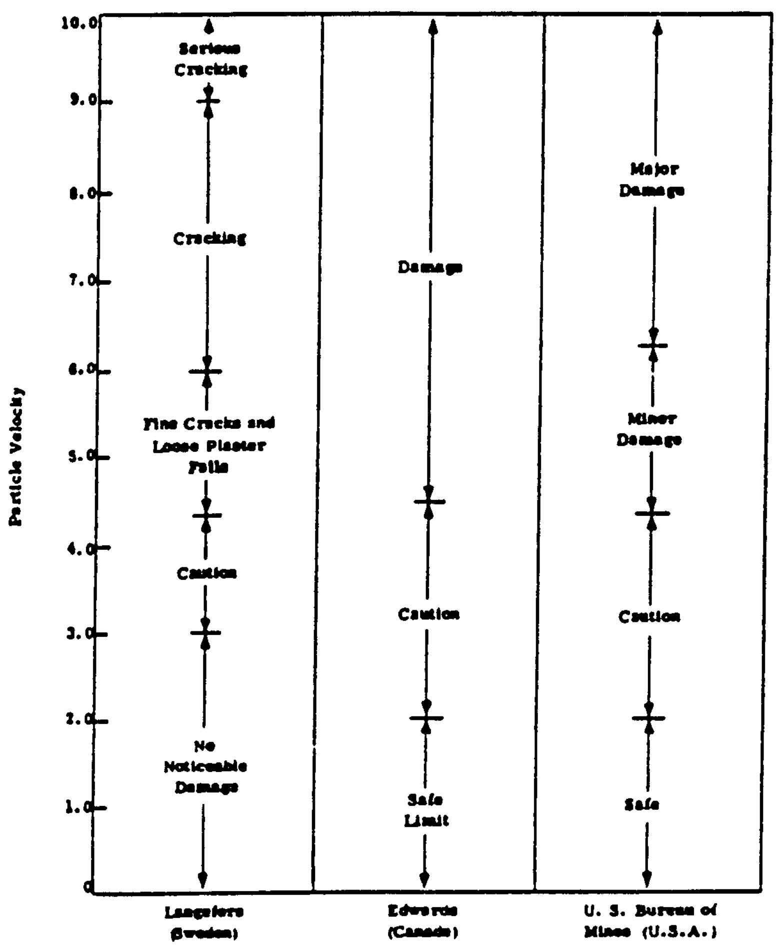

Figure 1 From Reference (5)

5.2 The velocity-sensing transducers are field calibrated, and set up with the primary axis directed radially toward the source of vibration. One channel should be set up on the ground approximately 3 ft from the building, the other channel at the property line, curb line, or other similar point closer to the source (low-level vibrations may be measured with transducers firmly seated on the soil, but more severe vibrations require additional coupling to the ground; see instructions from transducers manufacturer).

5.3 Data Recording: Recordings are made of the earth vibrations caused by the objectionable source. Variations in mode of operation of the subject source should be checked if possible (for example, height of drop of pavement breaker, different distances of operation from source to building, pile driver with pile penetration just beginning, deep, and at refusal. Watch especially for difficult driving with the tip of the pile near the surface of the ground).

5.4 Analysis: Measured earth velocity values are examined for magnitude, (sometimes frequency as well), and compared with established damage or annoyance criteria. (See appendix for further discussion of limiting velocity values.)

5.5 Reporting: The collected data are reviewed and formally reported, including the type of problem involved, damage alleged, type of measurements made, along with tabulated data and photographs, if required. Measured values are compared to reference damage criteria (Table 1), and conclusions recorded.

APPENDIX

A.1 GENERAL INFORMATION

A.1.1 Since vibration measurement and analysis procedures are not yet firmly established in the transportation field, and there is a wide variation in the recommendations for maximum allowable values, this appendix is included to provide limited information and to stimulate further discussions and experimentation. A list of selected references is included for those interested in further review of the literature.

A.1.2 When heavy transportation vehicles were first operated many years ago, some people were concerned that building vibrations caused by the vehicles would cause long term structural problems in the cities where numerous buildings and vehicles were in constant proximity. Now, after many hundreds of millions of cycles of low-level vibration, there does not appear to have been a significant problem in general.

A.1.3 Most experimentation concerning vibration damage to buildings has been in the field of blasting, where areas affected can be fairly large, and the number of cycles applied is quite low. In contrast, transportation construction activities provide vibrations of moderate magnitude, greater numbers of cycles of application, limited duration, and rela-

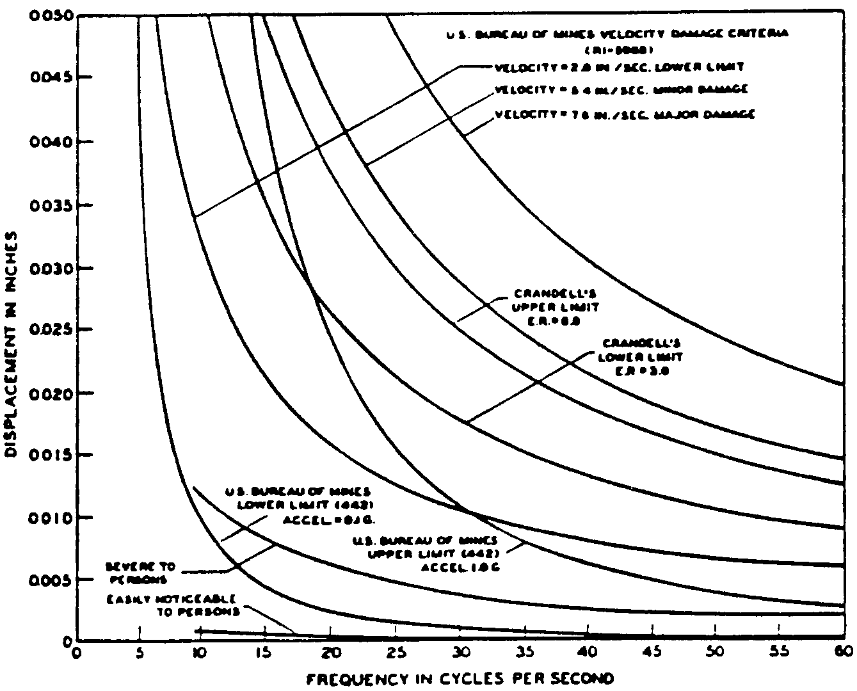

Figure 2 From Reference (15)

tively small area of effect. Traffic operations cause still lower level vibrations, but affect extensive areas continuously over very long periods of time.

A.1.4 In general, the literature shows only blasting, pile driving and pavement breaking to have documented examples of potential for building damage, and the latter two, only a relatively close range.

A.1.5 Early work by the U.S. Bureau of Mines, Edwards and Northwood in Canada, Langefors and others in Sweden (Figure A1) led to a caution limit extending approximately from 2 to 4.5 in./sec of earth particle velocity, and a recommended safe limit of about 2 in./sec. for blasting vibrations near residential structures. However, of the total number of damage points listed, about 3 percent (all minor damage points from BuMines experiments), fell between 1 and 2 in./sec. This has led several later authors to recommend lower maximum values. Jackson's analysis (6) suggested that the threshold for damage should be placed at about 0.2 in./sec to include fatigue effects from repeated cycles of application, and the effects of age and the elements on the buildings.

A.1.6 In contrast, Wiss and Nicholls (14) report on tests in Minnesota, involving experimental blasting near a substantial 30-year-old, 1½-story house with gypsum wallboard interior and stucco over wood siding exterior, where measured earth velocities exceeded 20 in./sec. and damage was very minor. The Bureau of Mines data combined with those of Langefors and Edwards and Northwood, showed documented minor damage at an average of 5.4 in./sec., and major damage at an average of 7.6 in./sec. Many other experiments have subjected selected buildings to vibrations of these magnitudes with little or no damage.

A.1.7 From these few examples, the wide range of limiting values is evident, and an investigation of the literature will expand the examples manyfold.

A.1.8 A complicating factor in the case of transportation related vibrations, is the extreme sensitivity of people to vibration. Figure A2 is taken from a report from the Louisiana Department of Highways (15) concerning a project done in cooperation with the Federal Highway Administration through the HP&R program. Notice that vibrations "severe to persons" fall far below the 2 in./sec. line, and "easily noticeable to persons" is near the very bottom of the graph. Figure A3 gives an indication of the information used by the Illinois State Department of Transportation.

A.2 LIMITING CRITERIA

A.2.1 Based on the information that is available, it appears that for normal housing in relatively good repair, limiting velocities in the range of 0.5 to 1 in./sec. would provide exceedingly low probabilities of damage. Engineering buildings can stand at least twice as much. However, any activities that create vibrations in excess of about 0.05 in./sec. (which is somewhat above the threshold of perception) can be expected to produce complaints from the occupants of the buildings even though no physical damage may result.

A.2.2 The British Road Research Laboratory report (12) discusses recommended upper limits of vibration to be incorporated in the proposed revision of the German DIN 4150—"Protection Against Vibration in Building Construction," that would set values of approximately 0.08 in./sec. (2 mm/sec.) for ancient buildings or relics, 0.2 in./sec. (5 mm/sec.) for intermittent vibrations of normal residential housing, 0.4 in./sec. (10 mm/sec.) for residential buildings in good condition; with maximum values of 0.8 to 1.6 in./sec. (20 to 40 mm/sec.), for industrial buildings without plaster. This is coupled with a requirement that the peak particle velocity be derived from the three component velocities V_x, V_y, and V_z by taking $\sqrt{V_x^2 + V_y^2 + V_z^2}$. These recommendations incorporate variation for the type of construction involved, and seem to provide a reasonable basis for future developments in this country.

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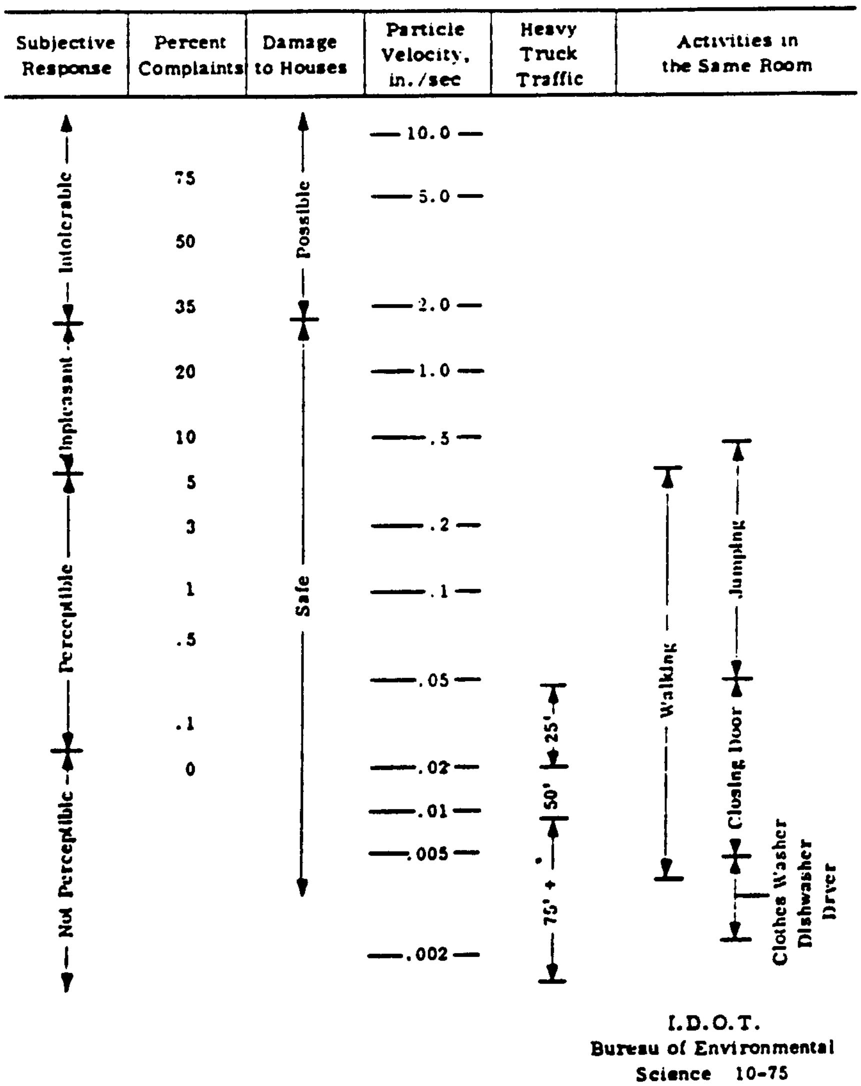

Figure 3 From Reference (17)

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- Plus : Specification Requirements for Engineering Seismograph
- Proposed Revision to AASHTO R8-81 Letter from Gordon Shaw OZA

MARYLAND

Attachments to contracts:

CATEGORY 100
PRELIMINARY

Protection Of Existing Structures

Description—This work shall consist of the protection of existing structures due to open excavation, pile placement, sheet pile installation, blasting, removal of existing bridge, or any other item which may effect the existing structures. Existing structures include, but not limited to the Wilson Mill Building located in the northwest quadrant of the bridge site.

Construction

Preconstruction Survey—The Contractor shall retain an experienced seismologist for the purpose of monitoring and registering vibrations in adjacent and nearby structures which are deemed necessary, and approved by the Engineer. This protection shall be for any structure which is liable to any damaging effects of any construction activity operations. The seismologist shall be approved by the Engineer.

The Contractor shall retain a commercial photographer, who shall be approved by the Engineer.

The Contractor shall retain a professional engineer registered in Maryland who is experienced in the field of building inspection surveys. This professional engineer will be referred to hereinafter as "Building Inspector". The building inspector shall be approved by the Engineer.

Prior to beginning any work, the Contractor, the insurer, photographer, building inspector, seismologist, and the Engineer, shall make a detailed inspection of each structure to record the condition of all walls, and other structural elements, as well as its contents and equipment that may be in place, and pavements and sidewalks that may become subject to possible damage claims. The record shall consist of a written report including measurements, sketches, and photographs as required to fully delineate the extent of or lack of deficiencies.

Photographs shall be 8 X 10 in. size and shall be in color and include views inside and outside of the existing structures.

A notarized statement certifying the dates this preconstruction survey was made shall be furnished by the Contractor, the building inspector, the seismologist, and the photographer to the Engineer. This certification shall include a statement that the preconstruction survey was made in the presence of and to the satisfaction of the respective owners.

The written report and photographs shall be furnished to the Engineer. The written report shall state acceptable levels of vibrations at the various existing structures together with the Contractor's procedures proposed for use for the various construction activities so as not to exceed the acceptable levels of vibrations.

Before any inspections are performed, the Contractor shall notify the owners of the structures involved, which have been approved by the Engineer, requesting their permission to enter upon the properties for the purpose of making these inspections for the protection of the owner.

In the event that access for the purpose of determining the condition of the buildings and structures is refused by any owner, the Contractor shall notify the Office of Bridge Development in writing and may thereupon be relieved of the responsibility for making the survey with respect to the property to which access is denied.

The Contractor shall, where possible, have the owner or a representative of the owner present during these inspections and secure the signature of the owner/representative on the completed documents, submitting a copy to the owner/representative.

A copy of all data relative to existing conditions of each respective property as found by the preconstruction survey, shall be forwarded to each property owner. Two identical copies shall be submitted to the Engineer.

Upon completion of the work on this project, and prior to final acceptance of the work, the Contractor with his insurer, building inspector, and with the Engineer, shall re-examine each property to determine any changes from the original conditions established by the preconstruction survey.

Vibration Surveillance—The seismologist shall record vibrations, during pipe pile and any sheet piling operations, blasting, removal of existing structure, or any other activity that may cause excessive vibrations near any adjacent structure. The seismologist shall submit to the Engineer the proposed methods of monitoring construction activities, effects on construction activities, effects on adjacent structures, including work plans that indicate the type and layout of sensing devices. The seismologist shall record the vibrations and direct the occurrence of damage due to the construction activities. The proposed methods and plans shall be approved by the Engineer prior to any construction activity.

If any construction activity has an adverse effect on adjacent structures as determined by the seismologist or the Engineer, the construction activity operations may be suspended while corrective action is being taken. The surveillance shall continue as long as required by the Engineer.

The Contractor shall not exceed the acceptable vibration levels contained in the preconstruction written report.

Measurement And Payment

The protection of existing structures including all of the costs of the preconstruction survey, seismologist, building inspector, photographer, preparation and submission of written reports complete, will not be measured but will be paid for at the Contract lump sum price on the Protection of Existing Structures item.

GP-7.11 PRESERVATION AND RESTORATION OF PROPERTY

B. The Contractor shall be responsible for all damage or injury to property of any character during the prosecution of the work, resulting from any act, omission, neglect or misconduct in his manner of method of executing said work, or at any time due to defective work or materials, and said responsibility shall not be released until the work shall have been completed and accepted. When or where any direct or indirect damage or injury is done to public or private property by or on account of any act, omission, neglect or misconduct in the execution of the work or in consequence of the non-execution thereof on the part of the Contractor, he shall restore, at his own expense, such property to a condition similar to, or equal to, that existing before such damage or injury in an acceptable manner. In case of the failure on the part of the Contractor to restore such property or make good such damage or injury, the procurement officer may, upon forty-eight (48) hours notice, proceed to repair, rebuild or otherwise restore such property as may be deemed necessary and the cost thereof will be deducted from any moneys due or which may become due the Contractor under this contract.

GP-7.13 RESPONSIBILITY FOR DAMAGE CLAIMS

A. The Contractor shall indemnify and save harmless the State and all of its representatives from all suits, actions, or claims of any character brought on account of any injuries or damages sustained by any person or property in consequence of any neglect in safeguarding the work or through the use of unacceptable materials in the construction of the improvement, or on account of any act or omission by the said Contractor, or as a result of faulty, inadequate or improper temporary drainage during construction, or on account of the use, misuse, storage or handling of explosives, or on account of any claims or amounts recovered for any infringement of patent, trademark, or copyright, or from any claims or amounts arising or recovered under the Workmen's Compensation Laws, or any other law, bylaw, ordinances or decree. The property of any character during the prosecution of the work resulting from any act, omission, neglect or misconduct, in the manner or method of executing said work satisfactorily or due to the non-execution of said work or at any time due to defective work or materials and said responsibility shall continue until the improvement shall have been completed and accepted.

MINNESOTA

Vibration Specification Examples

Four levels of vibration control can be provided on a project, depending on things such as structure susceptibility to damage, proximity to vibration producing activities, local concerns, or district policy. The "levels" can briefly be defined as follows:

Level 1—No specific mention in contract of possible problems or controls. (On a state-wide basis, this is most common for minor or small quantities of pavement breaking or pile driving).

Level 2—Alert contractor to possible problems and make him/his insurance company totally responsible for any courses of action appropriate.

Level 3—Detail concerns and require the contractor to do a prescribed condition survey and to employ a qualified vibration specialist to establish a safe vibration level and monitor the vibrations. The contractor is still responsible for any problems.

Level 4—State takes lead role and has consultant(s) do a damage susceptibility study to establish vibration control limits, and a preconstruction condition survey for each structure. The State also takes responsibility for vibration monitoring during construction to insure compliance with vibration control limits. At this level, the State assumes some responsibility for damage to structures if the established vibration limits are not exceeded by the contractor. The degree of responsibility depends on the vibration specification - most vibration specifications are aimed at avoiding structural damage, leaving the contractor responsible for any cosmetic damage (e.g. plaster cracks, broken windows, etc.) and keeping residents/occupants informed and "happy".

Examples of Level 2 through Level 4 specifications are given below. Each of these was produced for a specific project and needs to be personalized or fine tuned for other projects. There may be levels between those shown, but care must be taken to keep the specifications consistent, for example, it would be inconsistent to expect the contractor to take total responsibility for vibrations and then put a vibration specification in the contract.

Level 2

S- Construction Vibrations

Vibration producing activities (such as blasting, pile driving, vibratory compaction, pavement breaking or operation of heavy construction equipment) may be required for construction of this project. The Contractor is advised that structures are located close to the proposed work and that construction activities shall be conducted so as to preclude damage to these structures and undue annoyance to occupants. The contractor shall be responsible for all damage caused by his activities.

Level 3

S - Construction Controls And Monitoring

Vibration producing activities (such as blasting, pile driving, vibratory compaction, pavement breaking or operation of heavy construction equipment) may be required for construction of this project. The Contractor is advised that structures are located close to the proposed work and that construction activities shall be conducted so as to preclude damage to same. The Contractor shall be responsible for any damage caused by his activities.

At least 30 days prior to start of such work, the Contractor shall provide a pile driving plan to the Engineer, which shall include, but not be limited to the following: proposed pile driving method, vibration monitoring plans (including the format for reporting the vibration readings), anticipated vibration levels at the closest building(s), condition survey format, and public relations activities. A copy of all reports shall be provided to the Engineer.

S- .1 Condition Survey

A preconstruction building Condition Survey shall be conducted by the Contractor on the _______________ building(s), prior to the commencement of any vibration producing activity.

The survey will include a documentation of interior subgrade and above grade accessible walls, ceilings, floors, roof and visible exterior as viewed from the grade level. It will detail (by engineering sketches, video tape, photographs, and/or notes) any existing structural, cosmetic, plumbing or electrical damage. The survey will be conducted by a Professional Engineer, registered in the State of Minnesota.

A report shall be issued that will summarize the preconstruction condition of the building(s) and will identify areas of concern, including potential personnel hazards (falling debris) and structural elements that may require support or repair.

Crack displacement monitoring gages will be installed as appropriate across any significant existing cracks to help verify any additional building distress if it should develop. The appropriate location, number, and type of gages will be established by the Contractor and/or the Project Engineer. The gages will be read prior to vibration producing activities, as well as during these activities. Data shall be obtained on a weekly basis for as long as vibration producing activities are being conducted. A report shall be submitted which summarizes the data. The Engineer shall be alerted if any significant movement is detected by the monitoring gages.

S- .2 Vibration Controls

The Contractor shall employ a qualified vibration specialist to establish a safe vibration level for the ___________ building(s). This specialist shall also supervise the Contractor's vibration monitoring program. During all vibration-producing activities, the Contractor shall monitor vibration levels at _____________ building(s), and shall not exceed the safe level established to preclude damage to this structure(s).

The vibration monitoring equipment shall be capable of continuously recording the peak particle velocity and providing a permanent record of the entire vibration event. Copies of all vibration records and associated pile driving data shall be provided to the Engineer in a format approved by the Engineer.

S- .3 Public Relations

The Contractor shall maintain a complaint log and make this available to the Engineer on request. Occupants/owners of adjacent buildings shall be notified by the Contractor at least 2 weeks prior to commencement of any vibration producing activity that might affect the structure or inhabitants.

Level 4

S- Vibration Monitoring And Control

The following provisions do not relieve the Contractor of any responsibility for damage caused by his operations, nor do they relieve the Contractor from compliance with all applicable federal, state, county and city codes relative to the use and storage of explosives. In the event that a conflict occurs between this specification and other codes, it shall be resolved by the Engineer.

S- .1 Susceptibility Study

A detailed document titled _______________________, has been prepared for Mn/DOT by (Name of consulting firm), and a copy of this report is available for inspection at the District ________ Headquarters, (address and contract). This report includes an evaluation of buildings and structures in proximity to the project and an evaluation of their susceptibility to construction vibration damage. The vibration criteria for this project are based on this study.

S- .2 Condition Survey

A condition survey will be performed for buildings in proximity to the project. This survey will document the existing exterior and interior conditions of these buildings.

The survey will include a documentation of interior subgrade and above grade accessible walls, ceilings, floors, roof, and visible exterior as viewed from the grade level. It will detail (by engineering sketches, video tape, photographs, and/or notes) the existing structural, cosmetic, plumbing and electrical damage, but will not necessarily be limited to areas in buildings showing existing damage.

Crack displacement monitoring gages will be installed as appropriate across any significant existing cracks to help verify any additional building distress, should it develop. The gages will be read prior to commencement of vibration producing activities, as well as during these activities. Results of this monitoring will be made available to the Contractor.

S- .3 Ground Vibration Controls

The following vibration control limits are applicable for all construction work, including but not limited to blasting, pile driving, compaction, ripping and hauling activities.

The Contractor is advised that the ground vibration control limits defined herein may restrict his construction practices, that he should consider these limitations in preparing his bid.

If the Contractor exceeds 80% of the ground vibration limit as given below, for any construction activity, he shall cease that activity and submit a report. The report shall give the construction parameter data and include a proposal for corrective action necessary to ensure that the specified limit is not exceeded for future activities. This report shall be submitted to the Engineer, and his permission must be obtained prior to the continuation, or beginning of any future vibration producing construction activities.

If the Contractor exceeds the ground vibration limit for any construction activity, the Engineer will direct that all activities related to those causing the vibration to be stopped. The Contractor shall submit to the Engineer a report giving the construction parameter data and include the proposed corrective action for future construction events. In order to proceed with any future vibration producing activities, written permission must be obtained from the Engineer.

A. Definitions

Following definitions shall apply to the vibration controls:

- Peak particle displacement—the peak particle displacement is the maximum movement induced by the vibration. The displacement amplitudes are in units of mils (0.001 inch) zero to peak amplitude.
- Peak particle velocity— The peak particle velocity is the maximum rate of change with respect to time of the particle displacement. The velocity amplitudes are in units of millimeters per second (mm/s) zero to peak amplitude.

Frequency— the frequency of the vibration is the number of oscillations which occur in one second. The frequency units are given in Hertz (Hz) where one Hz equals one cycle per second.

B. Ground Vibration Control Limit

The ground vibration controls are applicable to external locations adjacent to affected buildings or structures. The maximum single component peak particle velocity resulting from construction activity shall not exceed ________________.

S- .4 Instrumentation

The Contractor shall furnish, maintain and operate three vibration monitor (amplitude and frequency sensitive) during any vibration producing activities that could, in the judgment of the Engineer, produce measurable ground vibrations. In the event that the Contractor chooses to have concurrent vibration producing activities at more than one location on the construction site, he shall notify the Engineer in writing at least two weeks prior to the commencement of such activities. The Engineer may require additional vibration monitoring instruments at each location depending on site parameters. No vibration producing activities may be started until the appropriate instrumentation is provided by the Contractor and approved by the Engineer.

All vibration instruments shall be powered with rechargeable batteries, and the Contractor shall supply extension geophone and microphone cables so that the instruments can be placed within structures if outside temperatures drop below zero degrees Celsius.

All vibration instruments shall be supplied with current calibration documents and shall be re-calibrated on approximately a six month use interval. At a minimum, instrument specific calibration curves of peak particle velocity input to peak particle velocity output shall be provided over the specified frequency ranges at both 12 mm/s and 25 mm/s for each instrument.

The Contractor shall be responsible for instrument maintenance. If the Contractor does not maintain a sufficient number of instruments to monitor the buildings/structures adjacent to the vibration producing activity, the Engineer may direct that all vibration activities cease until a sufficient number are working. Recording tape shall be supplied by the Contractor and at least a two-week supply maintained.

The Contractor shall designate an individual in his organization or under contract to him, who will be responsible for instrument coordination. The Contractor will be responsible for placing the instruments at measuring locations designated by the Engineer, and reading and recording the pertinent vibration event data. The Contractor will report the data to the Engineer at the completion of each vibration event.

The amplitude and frequency sensitive recording instrument shall be a SSU 1000D or equivalent, available from Philip Berger and Associates, P.O. Box 779, Warrendale, PA 15095. This instrument shall be capable of measuring and recording the frequency and peak particle velocity in three mutually perpendicular axes. ("Vector sum" instruments are not acceptable). This instrument shall also have the following features:

1. Self-triggering
2. Permanent record of the time history of the vibration event readable in the field, and the ability to process the data to determine the frequency of all three peak vibration levels. The permanent field record shall be of a quality to permit ready interpretation of the frequency content up to 150 Hz.
3. A digital display or printout which will yield immediate results of the three components of vibration and air blast.
4. Sensitivity ranges to resolve peak particle velocities from 0.25 to 100 mm/s.
5. Frequency range from 5 - 200 Hz + 3dB.

S- .5 Public Relations

The Contractor is required to have both letter and personal contact with resident and owners or operators of the buildings that are adjacent to the construction area or near enough to it for ground vibrations from construction operations to affect the personal property, displays or merchandise of these buildings. This contact will be made prior to the beginning of any vibration producing activity. The Contractor will furnish a list of those contacted to the Engineer.

As described elsewhere in these provisions, the ground vibration resulting from construction work will be monitored by the Contractor. The Contractor will measure the magnitude of each vibration event with at least two vibration instruments, generally located adjacent to the closest or most critical structures.

6. Record Keeping

The Contractor shall maintain a log of all vibration producing activities at which ground vibrations were measured. The log shall include the maximum peak particle velocity and its associated frequency, type and location of the vibration producing event, location of the geophones and closest distance from the vibration producing event to the geophone(s). When vibration producing activities are in progress, the Contractor shall submit daily reports to the Engineer which include all the vibration log data from the day. These reports shall be submitted at the end of each day, and no further vibration producing activity will be allowed until such reports are received by the Engineer.

The Contractor shall be responsible for removing all vibration records produced by the vibration instruments and attaching them to the corresponding Blast Log for submittal to the Engineer as part of the daily report.

The Contractor shall maintain a complaint log of all vibration related complaints, contacts and actions, and shall furnish copies to the Engineer on request.

NEW YORK

Vibration Criteria Notes

"The Contractor's attention is directed to the need to minimize vibrations due to his construction activities. The Contractor shall govern his methods of operations, such that the peak particle velocities measured from the driven sheeting or pile locations to the closest building shall not exceed the following values:

____ mm/s peak particle velocity for buildings
_____ mm/s peak particle velocity for utilities

The maximum hammer energy for driving sheetpiling shall not exceed _____ N-m

These criteria will be strictly enforced, and the Contractor is advised that he will be required to limit hammer energy and take all measures necessary, such as hand excavation, to keep vibrations within acceptable levels."

Commentary

The velocities are determined on a site specific basis. The energy is also determined by proximity of utilities and buildings and site conditions.

Present <u>NOTE!</u>

4. Building Condition Survey Notes

"Building condition surveys will be performed by the Engineer or his representatives within the contract limits prior to the commencement of work, after completion of work under this contract, and at locations and times during construction as ordered by the Engineer. The condition survey shall include, but is not limited to:

a. Photographic and videotape documentation of the interior and exterior conditions of the buildings.
b. Extent and location of existing signs of building distress.

The Contractor shall accompany the Engineer during each building condition survey performed by the Engineer to verify the data recorded. The Contractor shall provide labor and equipment, such as ladders, as required by the Engineer to assist the Engineer's representative in carrying out these surveys. The cost of this shall be included in other items of work.

NORTH DAKOTA

622.03 B. Pile Driving—Pile shall not be driven within 80 ft of concrete which has cured less than 3 days or a greater distance if determined necessary by the Engineer. (North Dakota, Florida, and Nova Scotia have clauses relating to pile driving in the vicinity of fresh concrete, but recent data indicates that curing of concrete is not vulnerable to vibrations.)

SOUTH DAKOTA

The following plan note instituted 1988:

> *Preconstruction Condition Survey*—The Contractor shall arrange for a preconstruction survey of any nearby buildings, structures, or utilities which may potentially be at risk from pile driving and other related construction activities. The survey method used shall be acceptable to the contractor's insurance company. The contractor shall be responsible for any damage resulting from pile driving and other related construction activity. Occupants of local buildings shall be notified by the contractor prior to the commencement of pile driving or similar operations. The contractor shall monitor vibrations caused by construction activities such as pile placement.

GERMANY

DIN 4150

Figure 1 plot of frequency versus velocity of three cases is shown below.

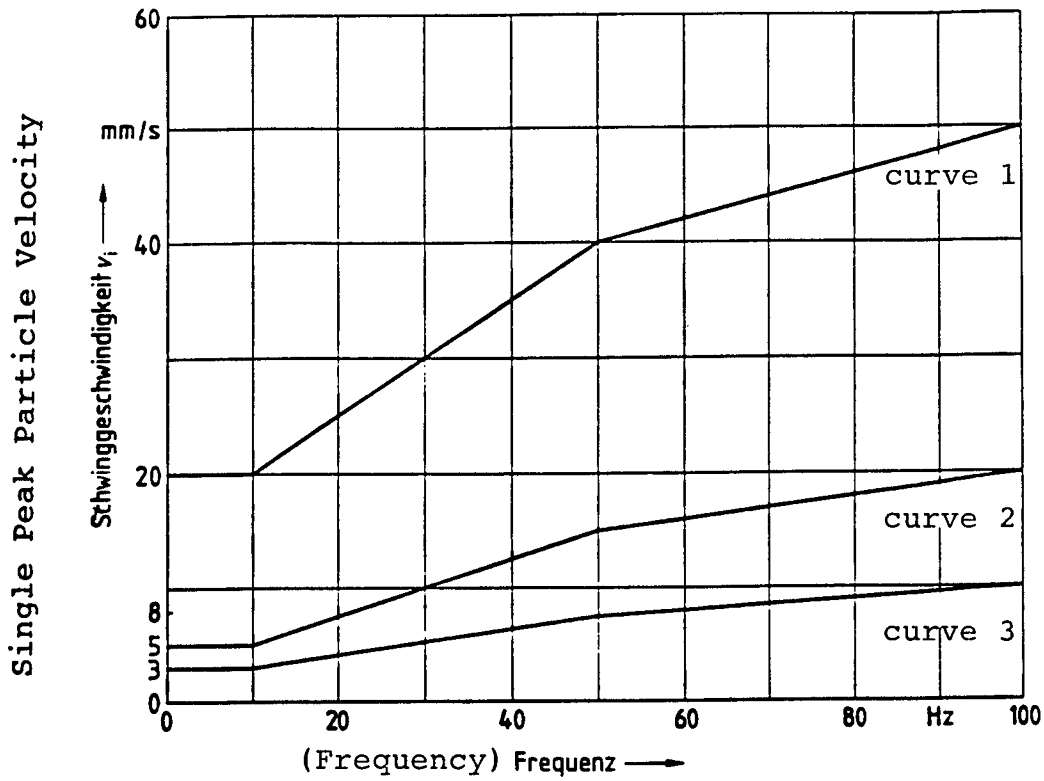

FIGURE D-1 Particle velocity versus frequency for three cases.

TABLE D-1
THREE CASES FOR DIN 4150, FIGURE D-1

		Foundation (Amplitude velocities in mm/sec) Frequencies			Ceiling of Highest Floor (Amplitude velocities in mm/sec) All Frequencies
Curve	Type of Building	< 10 Hz	10 to 50 Hz	50 to 100* Hz	
1	Structures used for business, industrial buildings and similarly designed structures	20	20 to 40	40 to 50	40
2	Living quarters, family house and building used for housing purposes	5	5 to 15	15 to 20	15
3	Structures who are particularly sensitive to vibrations and don't belong to category 1 or 2 and additionally are worthy of preservation	3	3 to 8	8 to 10	8

*For frequencies above 100 Hz use the values for 100 Hz.

CANADA

Nova Scotia 88-32

Pile driving not permitted within 15 meters of concrete or grout within 3 days of placement.

CANADA

Ontario Metric OPSS 120 General Specs for use of Explosives

120.04.02 Blast Design

Where explosives are to be used within 100 m of any utility, residence, structure of facility, the following is required:

a. Two weeks prior to the use of explosives the name of the Professional Engineer responsible for the blast design including a record of their experience and a statement of their qualifications shall be submitted to the Contract Administrator.
b. A blast design which shall include the design peak particle velocity, peak sound pressure level, number of holes; pattern, orientation and size of drill holes; depth of drilling, collar and toe load; mass and type of charge per delay; and number and time of delays. A copy of the blast design shall be provided when requested by the Contract Administrator.
c. The following shall be submitted to the Contract Administrator forty-eight hours prior to the use of explosives:

   1. A letter signed by the Professional Engineer responsible for the blast design indicating the areas for which a blast design was completed.

   2. A letter signed by the blaster indicating that he is in receipt of the blast design and will carry out the blast according to the design.

120.04.03 Pre-blast Survey

A pre-blast survey is required for all residences, utilities, structures and facilities likely to be affected by the blast within 100 m of the location where explosives are to be used. A copy of the pre-blast survey shall be provided when requested by the Contractor Administrator.

Forty-eight hours prior to the use of explosives a certificate, signed by an independent specialist, indicating that a pre-blast survey has been carried out for those areas within 100 m of where the explosives will be used, shall be submitted to the Contract Administrator.

APPENDIX E

Example Vibration Criteria

The following taken from Dowding (21), *is intended to provide information to an agency developing pile driving vibration monitoring specifications. It is not necessarily inclusive of a particular agency's total needs and should be customized as appropriate.*

A) GENERAL PROVISIONS

This specification is intended to establish controls for pile driving in the interest of life, health, and safety of employees and the public, as well as the protection of nearby structures, property, and soils that remain in place. All of the contractor's responsibilities apply equally to any subcontractor involved in pile driving activities. Pile driving shall be allowed only during specific periods of time, as determined by the engineer on site, according to locally applicable codes and necessary operational restrictions.

Public awareness—The contractor is required to have both letter and personal contact with residents, institutional operators, and business establishments that are within the construction area and near enough for ground vibrations from pile driving to be easily perceptible. This contact shall be made prior to the beginning of any pile driving activity. The contractor is required to furnish the engineer with a list of those contacted prior to the pile driving operations, and include on that list all pertinent information as approved by the engineer.

Permanent displacement—A line (location) and grade (elevation) survey will be performed by a surveyor licensed by the state in which the construction occurs. It will establish control and gradelines to detect movements along the exterior faces of the buildings. This survey will be conducted on all buildings within a 25 m radius of the construction site and all historic buildings or structures within a 125 m radius. Reports shall be delivered monthly to both engineer and contractor.

All control lines and grades shall be referenced to existing benchmarks, which shall be established far enough from the construction site to be preserved for all surveys. Reference points are generally taken at a distance greater than 225 m from the site so they are well beyond the reach of pile driving operations. The precision required can vary, but in general should be accurate to 1.5 mm.

Tilting of the nearest walls of structures will be established by measurement with a portable tiltmeter or other suitable method.

Buildings included in this survey are those that could experience permanent deformation because of their proximity to the pile driving. The amount of deformation expected therefore needs to be quantified, so measurements shall be made at intervals determined by the engineer, but at least once a month.

Existing Building Cracks—Permanent deformation of buildings will be monitored with crack monitoring gauges. Their sensitivity shall be to 1.5 mm. The type of gauges shall be determined by type of potential distress (plaster cracks, movement, etc.). A minimum of six crack monitoring gauges will be placed on strategic structures, within a radius of 25 m from the nearest pile driving operations, and on buildings or structures of particular concern, such as historical monuments, within a radius of 125 m from piledriving activities.

Surveys and gauge readings are generally obtained monthly. A report must be issued to the contractor and engineer monthly that summarizes the survey and crack opening data. The area monitored by surveys and gauges will vary. A typical urban distance might be 120 m out from the piledriving activity.

B) PRECONSTRUCTION SURVEY

A preconstruction survey shall be undertaken prior to the start of any activity on the site, including the test pile program. The survey will include all buildings within a radius of 120 m of the pile driving activities, or out to a distance at which vibrations of 2 mm/sec occurs. The objective of this survey is to determine the buildings' susceptibility to disruption from pile driving vibrations. Disruption includes impact on sensitive equipment and operations, as well as cosmetic cracking and effects on the surrounding geological and/or geotechnical materials. The results of this study will be made available to the contractor.

Susceptibility ratings of structures—The engineer should classify the buildings inspected under the requirements of this specification into different categories, as a function of a building's susceptibility to cracking during blasting vibrations. Each building inspected should be placed into one of the following categories: low susceptibility, moderate susceptibility, and high susceptibility to cracking. Cracking is the threshold of cosmetic cracking, as defined below.

A building identified as having *high susceptibility* has already experienced a significant amount of degradation to its primary structural and/or nonstructural system. Pile driving vibrations may result in further degradation of these elements, possibly resulting in injuries to personnel in the building. Buildings with loose or unstable elements, such as loose bricks or structurally cracked terra-cotta cornices, are considered to fall into this category. Buildings with significant quantities of fragile, potentially unstable contents, which may be damaged during pile driving, are also included in this category.

A building identified as having a *moderate susceptibility* has not yet experienced a significant degradation to its primary structure, or its nonstructural systems, which would lead to further building degradation due to pile driving vibrations; however, some building deterioration has occurred prior to pile driving. Buildings identified as having bricks that may possibly be loose, as determined by visual inspection, are considered to fall into this category. Buildings with small to moderate quantities of fragile, potentially unstable contents, which may be damaged during pile driving, are also included in this category.

A building identified as having *low susceptibility* is not expected to experience cosmetic cracking when subjected to pile driving vibrations. Also, the contents of such a building will not suffer damage due to pile driving.

Microvibrations and sensitive equipment and/or operations—An important part of the preconstruction survey should deal with the possible nearby presence of sensitive equipment and/or operations, such as hospitals, computerized industries or banks, or industrial machinery. It is necessary to take this information into account for the establishment of the controls.

Surrounding soil densification: One of the objectives of the Preconstruction survey is to check for stability of the soils surrounding the pile driving site. Densification of loose material and slope movement can occur during pile driving vibrations, and this possibility must be considered when establishing control limits for ground motions.

Condition Survey

A condition survey shall be undertaken for all buildings within 120 m of the construction activity, and all historic buildings or structures within 400 m. This survey shall document the existing exterior and interior conditions of these buildings.

This survey shall include documentation of interior subgrade and above grade accessible walls, ceiling, floors, roof, and visible exterior as viewed from the grade level. It will detail, by videotape and/or photographs, the existing structural, cosmetic, plumbing, and electrical conditions, and shall include all walls, and not be limited to areas of building showing existing damage. Notes and sketches may be made to highlight, supplement, or enhance the photographic documentation.

The condition report shall present engineering notes and photographs or video records. The report shall also summarize the condition of each building and define areas of concern. Reports of the condition surveys shall be made available to the contractor for his review prior to the start of any construction or demolition activities.

A pre-pile driving (condition) survey, to be of real value, has to be conducted with care, ensuring that no observable defects are omitted. A poor inspection in which defects are omitted will be of little value to an operator. In many cases, homeowners are unaware of all the defects present in their homes, but they will inspect their homes more closely upon being startled by construction noise or vibration, and will notice preexisting defects for the first time. Such cases lead to complaints, litigation, and sometimes to court orders to halt construction operations. The presence of this survey in the contract considerably reduces the chances of such complaints, and if they do occur, provides information vital for assessment of the cracking and settlement of post construction claims. A blank field inspection report is presented as Figure E-1. The distance out to which this inspection is conducted will vary. A typical urban distance might be 120 m.

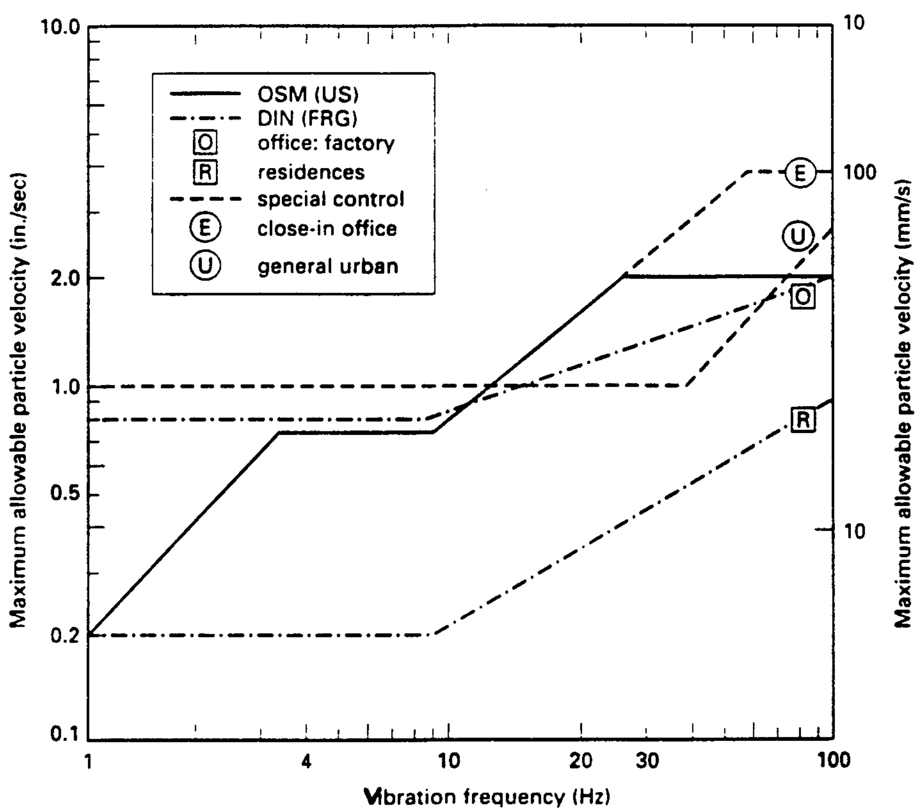

FIGURE E-1 Suggested field inspection report.

Cracking Deformation

Types of cracking—The engineer will distinguish different types of cracking in structures as follows:

– Cosmetic cracking includes 1) threshold damage: opening of old cracks, and formation of new plaster cracks; dislodging of loose structural particles such as loose bricks from chimneys and 2) architectural or minor damage: superficial, not affecting strength of the structure (broken windows, loosened or fallen plaster), hairline cracks in masonry.

– Structural cracking is minor damage resulting in serious weakening of the building (large cracks, shifting of foundations or bearing walls, major settlement resulting in distortion or weakening of the structure, wall put out of plumb.)

C) PARTICLE VELOCITY CONTROLS

Definitions

The *peak particle velocity* is the maximum rate of change of position with respect to time, measured on the ground. The velocity amplitudes are give in units of millimeters per second (mm/s), zero to peak amplitude.

The *frequency* of vibration is the number of oscillations that occur in 1 second. The frequency units given are in hertz (cycles per second).

The *dominant frequency* is usually defined as the frequency at the maximum particle velocity, which will be calculated visually from the seismograph strip chart for the half cycle that has as its peak, the maximum velocity.

The *scaled distance* is equal to the distance from the pile driving to a building or other target, measured along the path traveled by the vibrations, divided by the square root of the energy expended in each blow of the pile driving or each cycle of the vibratory pile driving. Common units are meters (m), and Newton-meters (N-m).

Controls

Pile driving shall be controlled by limiting ground particle velocity. Peak particle velocity shall be the measure of the level of vibration, and it should be measured with the instrumentation and methods described in Section E of this specification. Peak particle velocity shall satisfy one of the following controls:

– Maximum Peak Particle Velocity Independent of Frequency (**OPTION 1**)

The peak particle velocity shall be less than a specific control limit at the nearest structure. The type of structure and distance between this structure and the nearest pile will dictate the allowable value as described in Table E-1. Particle velocity shall be recorded in three mutually perpendicular axes.

The maximum allowable peak particle velocity shall be that of any of the three axes.

TABLE E-1

LIMITING PARTICLE VELOCITY

Structure and Condition	Limiting Particle Velocity (mm/sec)	Limiting Particle Velocity (in./sec)
Historic and some old structures	12	0.5
Residential structures	12	0.5
New residential structures	25	1.0
Industrial building	50	2.0
Bridges	50	2.0

– Maximum Peak Particle Velocity That Varies with Frequency (**OPTION 2**)

Frequency-based limits for the peak particle velocity shall be imposed as defined in Figure E-2, for distances of less than 25 m. The engineer shall choose the governing curve in Figure E-2. At all other distances, the maximum particle velocity shall be 25 mm/sec (1 in./sec).

For this option, a seismographic record, including both particle velocity time history and dominant vibration frequency, shall be provided by the contractor for each pile driven. The method for that analysis of the predominant frequency contained in the vibration time histories shall be approved by the engineer during the submittal of the blast plan.

Optional Additional Clause

The engineer reserves the right to adjust the values designated in the paragraphs above, if, in his or her opinion, the pile driving procedures being used are damaging the adjacent structures or soils.

Application of the Particle Velocity Control

If the contractor exceeds 80 percent of the ground vibration control limit for any single axis during a pile driving operation, he/she shall cease all pile driving activities and submit an additional written report to the engineer. This report shall give the vibration measurements data and include the corrective action for the next pile to be driven to ensure that the limit will not be exceeded. The next pile shall not be driven until the engineer acknowledges, in writing, that a driving process change has been implemented.

If the contractor exceeds 100 percent of the ground vibration control limit for any single axis during pile driving, he or she shall cease all pile driving related activities and submit a written report to the engineer. This report shall give the driving and vibration data and include necessary proposed corrective action for the next pile to be driven to ensure that the specified limit will not be exceeded.

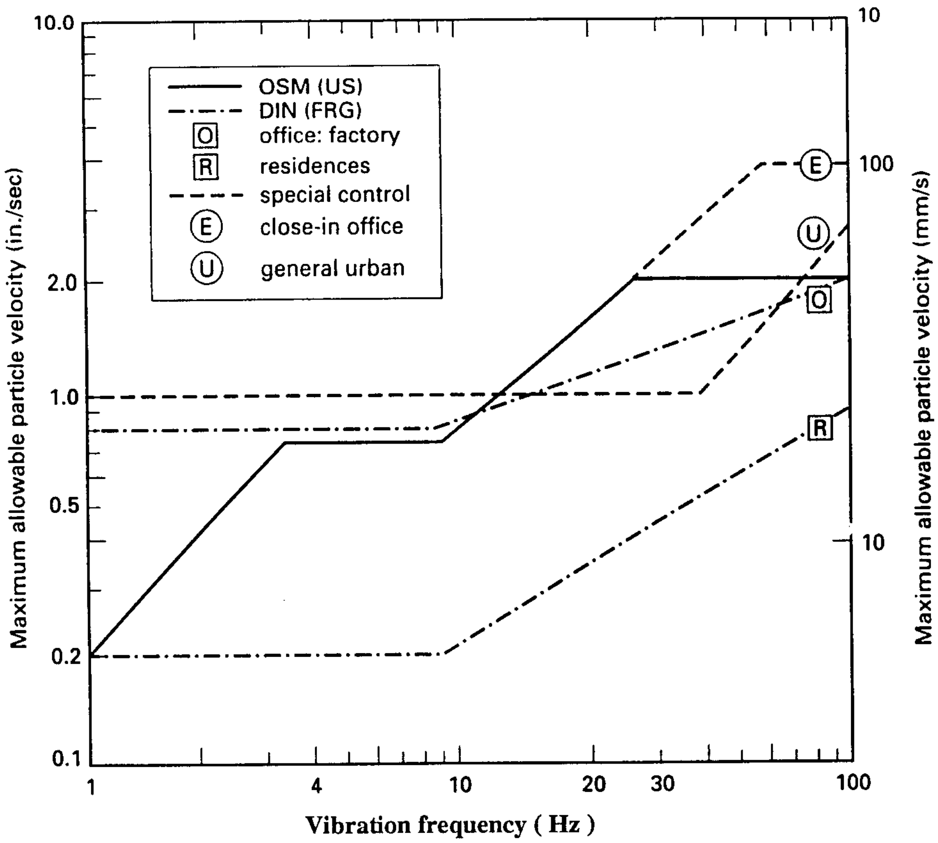

FIGURE E-2 Compariosn of frequency-based allowable particle velocity controls to show increasing allowable particle velocity with increasing dominant frequency *(21);* same as Figure 20.

D) MONITORING OF VIBRATIONS

Recorded Data

Peak Particle Velocity—All three components (longitudinal, transverse, vertical) of particle velocity will be measured on the ground at the location of the nearest and other strategic structures and/or at any locations the engineer deems necessary for any particular pile driving operations. These measurements shall be made on the ground adjacent to these structures as the pile driving is going on.

Pile Driving Log—The contractor shall maintain a pile driving log and shall submit daily reports to the engineer on piles driven and vibrations measured. These logs shall be in the form specified in the Driving Plan.

Instrumentation

The contractor shall provide the instrumentation agreed to in the pile driving plan to monitor the pile driving vibrations and permanent deformation of the strategic structures. On-site measurements will be made by the engineer. The engineer will provide any other additional instrumentation not defined herein.

Vibration Monitors—(Seismographs) Vibrations in the form of particle velocities shall be monitored by Type I and/or Type II monitors.

Type I is a waveform recorder. It provides a particle velocity wave form or time history of the recorded event, sometimes in conjunction with peak event information. (Type I must be used for Option 2 monitoring.) Independent chart recorders with separate motion transducers can be used in place of "stand-alone" monitors like seismographs when approved by the engineer.

Type II is known as a continuous peak particle velocity recorder and it provides no waveform and therefore no frequency information. Both Types I & II can be employed for Option 1. [Note: Acceptable monitors include those shown in Chapter 6 of the text. (Another table of acceptable monitor characteristics could be inserted at this point, as customized for a specific DOT.)].

Transducer Attachment (Coupling)

When the measurement surface consists of rock, steel (or other metal), asphalt, or concrete, the transducers shall be bolted to the measurement surface or bonded with high strength adhesive. [The transducer units of most portable seismographs can be removed from the measurement case and installed at an appropriate location on the ground or structure. It is only this transducer unit that needs to be coupled to the surface on which it is sensing vibration. The case with recording instrumentation can be set on any appropriate surface.] On other surfaces the mass of the seismograph and/or transducer package may be sufficient for good coupling. For significant accelerations (greater than 1.0g), adhesive or bolts shall be used on all solid surfaces. All transducers on vertical surfaces shall be bolted in place. In some locations burying the transducers will minimize air borne noise, while in other situations, sand bags over the transducers can aid with coupling and reducing air borne noise.

Number and Location

The number of instruments required is dependent on the specific site. However there shall be, as a minimum, two monitors of type I. One monitor will be used on site, while the second is held in reserve or used at a specific complaint or potential complaint site.

Archiving

The contractor will provide the engineer with all data necessary for record-keeping purposes. These data shall be kept by both parties for at least 3 years, and shall include, as a minimum, the following information:

- All monthly surveys conducted for vibration control purposes, including the preconstruction survey.
- The original driving plan, as well as any adjustments made to it during the course of the construction activities.
- All monitored data, relative to each and every pile installed. These driving records shall contain all information as required and approved in the pile driving plan, including all information concerning the type and characteristics of the monitoring instruments used, their locations and orientations.
- All driving records correlated with monitored data.
- All weather conditions occurring during the driving activities.

E) PILE DRIVING

Driving Plan

No less than three weeks prior to commencing the test pile program, or at the preconstruction conference (whichever is earliest), or at any time the contractor proposes to change the driving method, the contractor shall submit a driving plan to the engineer for review. The driving plan shall contain: (1) all information required under the general piling specifications, and (2) all information relative to vibrations and vibration controls, as described in the following sections.

Pile-Driving Equipment

Two types of pile drivers can be used: impact or vibratory hammers. The contractor shall be aware of the fact that ground vibrations induced by these machines are of different nature, and therefore utmost care shall be taken in the selection of the equipment and driving method.

Test Pile Program

Definition/Responsible Party

The contractor shall provide any necessary cooperation with the engineer for conducting a test pile program. While the engineer will take the lead role in this program, the contractor shall concur in the intent, design, and process of the testing. This program shall be performed prior to the start of any piling activities. It shall be performed to show how the vibrations decrease with increasing wave travel path distances from the pile and vary with the type of pile used. This program is intended to provide subsequent guidance for the choice of pile placement technique for this particular project, and not to define any envelope or relationship to be used as a control.

Monitoring

The number, type and location of the seismographs used to monitor the test pile program shall be determined by the engineer.

Analyses

Statistical analysis of the test data will be performed by the engineer. The results of these analyses will be transmitted to the contractor within three weeks after the completion of the test pile program. Three analyses are to be performed: 10 an attenuation analysis, 20 a frequency analysis, and 30 a response spectrum analysis.

F) AIR OVERPRESSURE

Preliminary Concerns and Definitions

Air overpressure as covered herein, is defined as airborne pressure waves, resulting from pile driving operations. Noise is the high-frequency audible portion of air overpressure.

The contractor shall be fully aware of the two different kinds of air pressure waves and of their possible adverse effects:

1) Low-Frequency Waves—These waves are inaudible but have the potential to crack buildings and break windows because they induce vibration in structures.
2) High-Frequency Waves—These waves are referred to as noise and cause community annoyance.

This specification does not establish acceptable noise levels and does not relieve the contractor from obeying all OSHA and community noise standards.

Noise Level Criterion for Impact Evaluation

The engineer should use a noise level criterion for impact evaluation. This criterion should give an appreciation of the noise level as a function of the percentage of time that this given level is exceeded.

This criterion allows for considerations of duration. Indeed, the effect of a continuous even, repeated, say more than 1,000 times a day, like the impact of a pile driver, will not be the same as that of a punctual event occurring fewer than five times a day.

Pile driving is considered to be a continuous event. The impact of the hammer on the pile is reproduced hundreds of times a day, thereby possibly inducing annoyance even at relatively low noise levels. Noise control measures can be applied to pile driving in a cost-effective manner, to reduce the objectionable off-site noise by about 10 dB.

General Provisions for Noise Control

The contractor shall take the necessary time and effort to understand and conform to the existing noise ordinances in the community where the construction site is located.

The contractor shall be informed of the noise radiation characteristics of his equipment (like impact pile drivers). This shall imply periodic measurements, despite the equipment manufacturer's noise specifications.

Equipment noise often increases with use, especially if maintained improperly. Furthermore, the resulting sound levels may rise in confined spaces or where the air waves are channeled by structures.

The contractor shall include noise as a factor in planning his operations. If the predicted noise emission is to be excessive for a particular period, the contractor shall discuss this problem with the local residents and officials to reach some type of agreement *before commencement of the work.*

With careful planning it is possible to maintain a highly efficient work output with a minimal noise output. It is necessary to identify, in chronological order, the various tasks with their associated equipment. This information, along with accurate sound level measurements, allow a prediction of the noise that will emanate from the construction site, and this noise level checked against local regulations.

The contractor shall use all of the information above to work with local officials and equipment manufacturers, in order to formulate rational noise reductions which can be implemented within a reasonable time frame.

Air Overpressure Control Limits

Air overpressures are not often a problem in pile driving operations except in confined spaces and in sensitive areas. However, overpressures shall not exceed the limits listed in Table E-2 measured at the location of any public building, residence, school, church, or community or institutional building. If necessary to prevent damage, the engineer will specify lower maximum allowable air pressure levels than those given in this section for use in the vicinity of specific structures.

TABLE E-2

MAXIMUM ALLOWABLE AIR-BLAST OVERPRESSURES *(17)*

Lower-Frequency Limit of Measuring Systems (Hz)	Maximum Air-Blast Overpressure [dB (3dB)]
0.1 Hz high-pass system	134
2 Hz high-pass system	133
5–6 Hz high-pass system	129
C-weighted (events less than 2 sec duration	105

Monitoring of Pile Driving Air Overpressure

Linear scaling of sound level instruments shall be used as default. In cases where no low-frequency overpressure is expected and only noise community annoyance is of concern, A or C weighted instruments can be used as approved by the engineer.