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STUDY OF GROUND MOTIONS FOR SIMULATION BY SHOCK TESTING MACHINE, by

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U. S. NAVAL CIVIL ENGINEERING LABORATORY
PORT HUENEME, CALIFORNIA

PREFACE

This study was performed in order to obtain criteria for the design of a testing machine that will simulate ground shock. Approximate methods were used in defining the motions encountered during ground shock in representative soil profiles, and the effect of various structures interposed between the free-field and equipment was estimated. Shock spectra were then obtained for typical equipment located in various types of structures placed in the assumed soil conditions. In this manner, a range of values was obtained that defined approximately the shock input and the shock response of equipment, which, it is believed, bounds the shock conditions for specified environments given in the Work Statement.

This study is the result of efforts of the staff of Agbabian-Jacobsen Associates. The principal contributors were

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The assistance of Dr. A. Paszyc and Mr. John Stephenson of the U. S. Naval Civil Engineering Laboratory is acknowledged.

Errata Sheet

Page	
7-5	Third line from bottom of page: change word equivalent to peak stress and notation c_{eq} to c_p
7-6	In equations defining a_v and v_{vz} change c_{eq} to c_p
	In equation defining d_v change c_{eq} to c_r and add following definition: c_r is average velocity of initial and peak stress disturbance.
7-7	In equation defining d_{vs} change the constant 7.5 to 30 and the notation c_{eq} to c_o and add the definition c_o is the seismic velocity at the surface itself
	In equation defining d_{rze} change c_{eq} to c_r
	In equation defining t_r change c_{eq} to c_p
7-9	In equations defining $(v_{max})_z$ and t_r change c_{eq} to c_p
7-10	In equation defining d_{vz} change the constant 7.5 to 30 and change the notation c_{eq} to c_o
7-18	After the heading Acceleration add Let $c_p = \frac{1}{2} c_{eq}$ where c_{eq} is specified seismic velocity
7-20	After the heading Displacement add Let $c_r = 0.75\ c_{eq}$ and $c_o = 0.5\ c_{eq}$

CONTENTS

1.0 INTRODUCTION

1.1 PURPOSE

The purpose of this study is to define ground shock motions and the response of equipment mounted in various types of structures so that criteria can be established for the design of a ground shock simulating testing machine.

The study considers shock motions due to nuclear effects for equipment mounted in six typical structures located at peak overpressure regions ranging from 15 psi to 500 psi. Weapon yields of 1 to 20 MT were considered. Three soil profiles were specified:

a)	Homogeneous with seismic velocity		:	c =	3000 fps
b)	Two layer -	Surface to 40 feet depth	:	c_1 =	1500 fps
		Below 40 feet	:	c_2 =	15000 fps
c)	Three layer -	Surface to 40 feet depth	:	c_1 =	1500 fps
		40 feet to 240 feet depth	:	c_2 =	3000 fps
		Below 240 feet	:	c_3 =	15000 fps

The three soil profiles and six structures are illustrated in Figure 1.1.

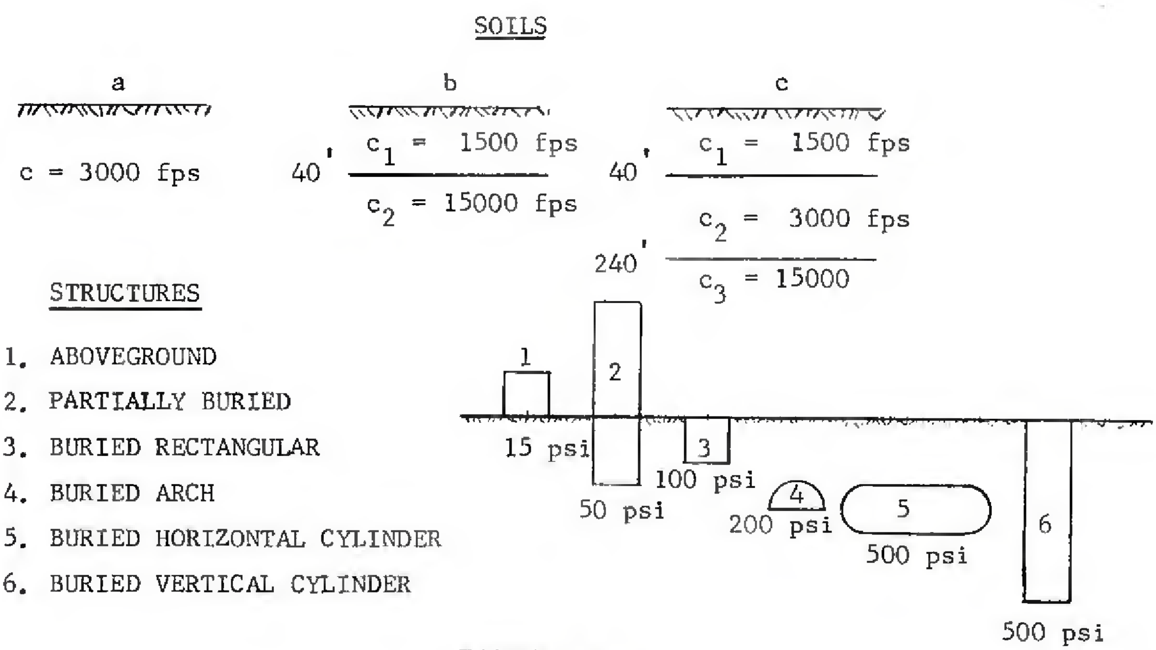

FIGURE 1.1

1.2 METHOD OF ANALYSIS

The shock motion for the equipment is evaluated in terms of shock response spectra and a qualitative description of velocity-time histories at the equipment support points. The shock spectra are derived from, (1) the peak ground motion intensities (displacement, velocity, and acceleration), (2) the peak structure motion due to the direct air blast slap (applicable for the aboveground structure and for the partially buried structure), and (3) the influence of the flexibility of the structure. For the velocity-time histories some specific characteristic values which are functions of the peak motions of the ground (or structure) and of the flexibility of the structure are given. For the purpose of this study, elastic behavior of equipment and attachments is assumed.

A brief description of each phase of the analysis is as follows:

(1) A preliminary design of the specified structures was performed in order to establish approximate stiffness and mass characteristics of each structure.

(2) The peak free-field ground motion intensities were calculated for the direct air slap for all the specified conditions over a range of depths in which the structure is located. An average free-field motion over the depth of burial for each structure was then calculated as an indication of the gross motion of the structure.

(3) The air slap induced seismic motions which may outrun (or occur simultaneously with) the surface shock front were considered by calculating arrival times of the refracted and reflected waves. The peak intensities were assumed to be related to the peak intensities of the direct air blast slap motion calculated in (2) by empirical formulas.

(4) The velocity-time history characteristics, for the motion defined in (2), were based on the compatibility of acceleration, velocity, and displacement and assumed to be a simple velocity pulse (Type I) with an initial downward motion and some permanent deformation at the higher overpressure levels.

(5) The velocity-time history for the motion calculated in (3) was related to the peak intensities and assumed to be an initially upward alternating motion (Type II) with time characteristics empirically related to field test data (see Reference 1.1).

(6) The air blast impact on structures 1 and 2 (aboveground and partially buried) induces rigid body and deformational motions to the structure. The peak displacement, velocity, and acceleration were calculated separately at the critical wall and floor positions for the rigid body and deformational motions.

(7) The peak motions calculated in items (2), (3) and (6) were combined as a single resultant motion imparted to each structure. The separate motions were combined by taking the square root of the sum of the squares of each peak value.

(8) The flexibility of the structure was considered in assessing the input to the equipment. The wall or floor supporting the equipment was modeled to a single-mass-spring system with the equipment and attachment as a second mass-spring system. The equipment mass, m_2, (as specified in Section 5) was found to be small with respect to the supporting element mass, m_1. Therefore the response of m_1 was calculated, assuming a single-degree-of-freedom system using a shock spectra response envelope.

(9) The response calculated in (8) becomes the input to the equipment and is used to obtain modified shock spectra (see Reference 9.1) describing the response of the equipment. The resonant responses which may occur between the equipment and the supporting structural element are shown on the equipment shock spectras.

2.0 SUMMARY OF RESULTS

2.1 SHOCK SPECTRA ENVELOPES

The shock spectral upper and lower envelopes for all the cases considered in this report are shown in Figure 2.1 for vertical and in Figure 2.2 for horizontal shocks. The specified equipment frequencies fall between 10 cps and 100 cps. At the frequency of 50 cps, shock input acceleration maxima of 1100 g vertically and 900 g horizontally are seen to occur. It should be remembered that both vertical and horizontal components may occur simultaneously.

The dashed line at the constant velocity of 300 ips is considered an upper bound of yield level for typical structural elements* (i.e. plastic deformation of structural elements usually will occur at velocities in this region). If this yield value at the constant velocity of 300 ips is an upper design limit, then the peak acceleration responses are 400 g both vertically and horizontally.

The structure to which the equipment is attached may be excited in many vibration modes due to the free-field motions. The structural vibration therefore may introduce high frequency accelerations (low amplitude) that can excite internal components of the equipment if the component frequencies are close to the structural frequencies. Only the lowest mode structural vibration frequencies have been considered in this study and they were found to vary between 5.5 and 165 cps. (Higher mode vibrations of the structure may also excite the fundamental as well as the higher modes in the equipment.)

The bumps on the two spectral upper envelopes account for a possible resonance condition of equipment with the lowest frequency mode of the structure. It is seen that if the two bumps are neglected the maximum response acceleration becomes 800 g vertically and 800 g horizontally over an extended frequency range to 1 KC.

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*See Appendix B for derivation of velocity bound for a typical structural element.

RANGE OF EQUIPMENT SHOCK SPECTRA VERTICAL SHOCK

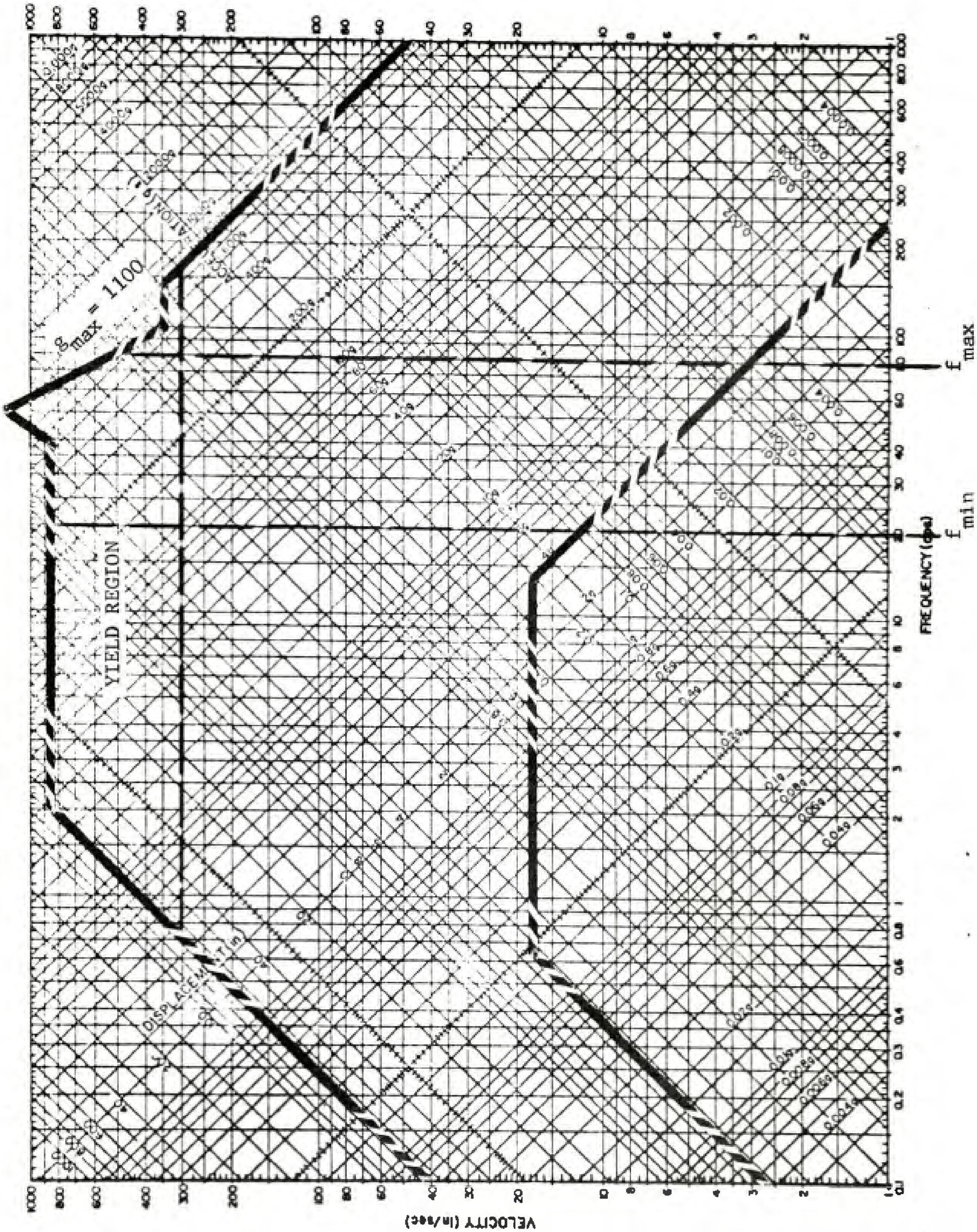


FIGURE 2.1

RANGE OF EQUIPMENT SHOCK SPECTRA HORIZONTAL SHOCK

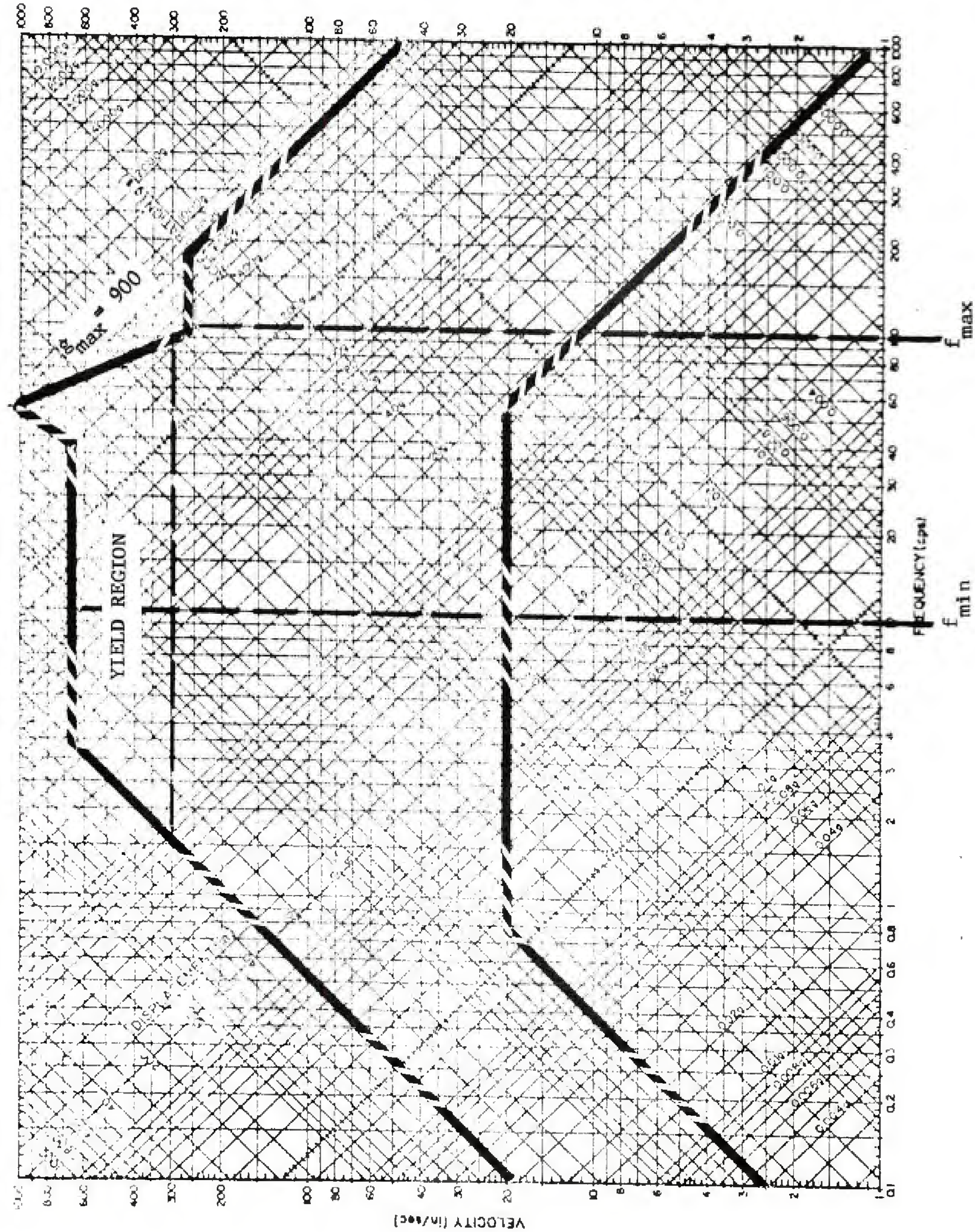


FIGURE 2.2

2.2 INPUT PULSE SHAPES

The input pulse at the base of the equipment is a combination of 1) the ground motion and the structure's rigid body motion due to the direct air blast slap (Type I pulse), and 2) the outrunning reflection and refraction waves induced through the soil (Type II pulse). As previously noted this motion may be modified due to the vibratory motion of the supporting structural element. The following paragraphs which are a summary of materials in Refs. 1.1 and 9.1 describe approximate characteristics of the Type I and Type II pulses.

The Type I pulse is an idealized motion with an initial downward velocity phase whose magnitude is a function of the overpressure pulse. The rebound of the structure (or ground) corresponding to the return phase of the velocity motion is based on elastic and plastic properties of the soil at the specific overpressure intensities. The peak horizontal motion is assumed to be related to the peak vertical motion by the factors $d_h = 1/3\ d_v$, $v_h = 2/3\ v_v$, $a_h = a_v$. Figure 2.3 lists the idealized velocity pulse bound for all the cases considered in this report.

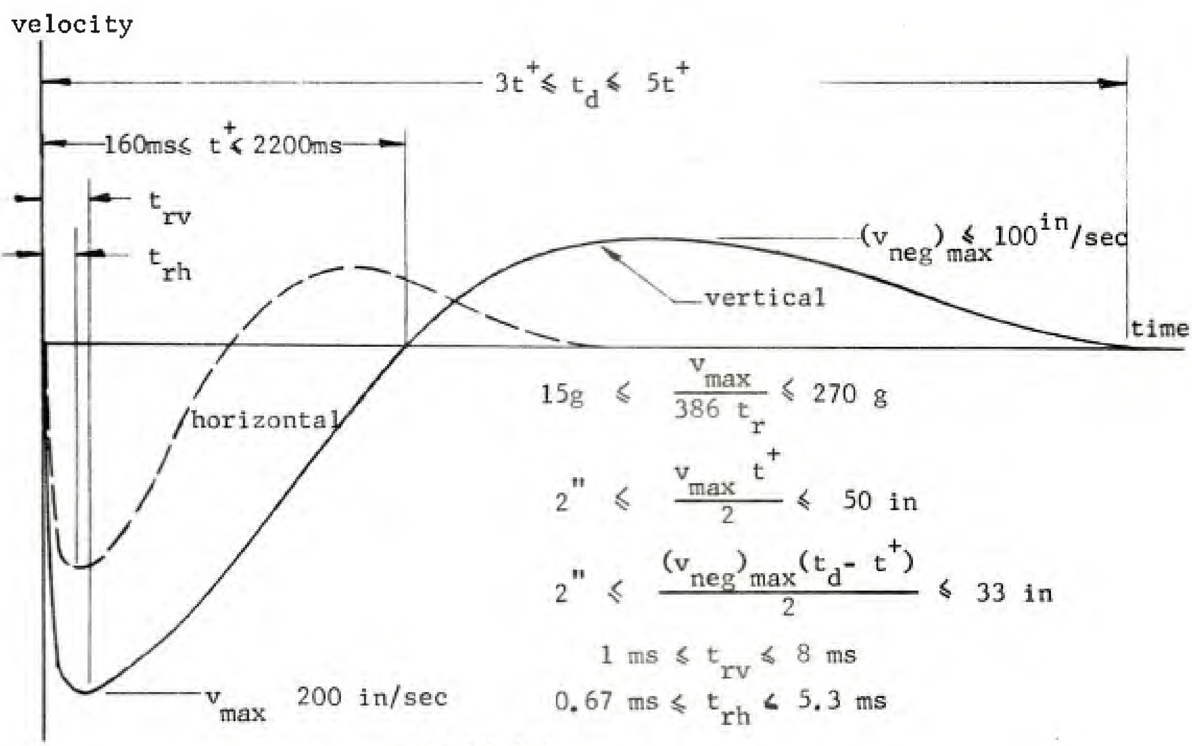


FIGURE 2.3

In some cases the Type II pulse (an alternating pulse) may arrive at the specified structure nearly simultaneously with the direct air blast slap. Arrival times of the pressure induced seismic motions (the G.Z. seismic motions were not considered) were calculated, and in some cases the seismic motion was found to arrive at the structure slightly before or slightly after the direct air slap on the structure as shown in Table 2.1.

TABLE 2.1

Structure	Overpressure Level	Soil Type	Phasing	
			Seismic lags	Seismic leads
2	50 psi	a	.03 to .1 sec	
3	100 psi	c		.3 to .4 sec
4	200 psi	b		.1 to .2 sec
5 and 6	500 psi	b and c	.05 to .4 sec	

The peak ground motion intensities of the Type II pulse are considered to be smaller than the ground motion due to the air blast slap. It has been suggested in Reference 9.1 that the peak vertical intensities of the alternating motion are related to the Type I idealized vertical pulse as follows:

$$d' = 0.3\ d, \qquad v' = 0.6\ v, \qquad a' = a$$

moreover, the horizontal components are assumed equal to the vertical components. Figure 2.4 shows the idealized Type II velocity pulse with characteristic peak intensities which are compatible with the critical cases considered in this report.

The Type II pulse characteristic time duration has been observed to increase as the distance between the outrunning seismic wave and the air blast shock front increases. This situation is illustrated in Section 7.

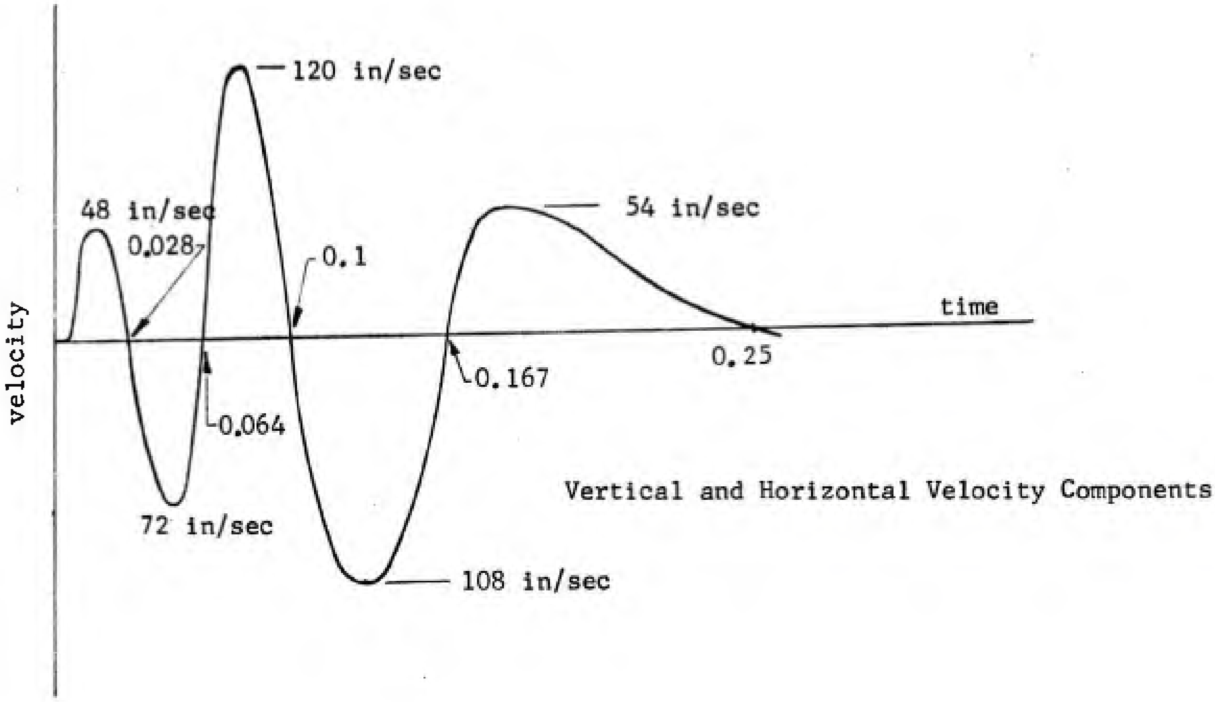

120 in/sec
48 in/sec
0.028
0.1
54 in/sec
time
velocity
0.25
0.167
0.064
Vertical and Horizontal Velocity Components
72 in/sec
108 in/sec

FIGURE 2.4

3.0 CONCLUSIONS

It can be concluded from this study that the critical free-field motion consists of an initial shock (i.e. an almost instantaneous velocity jump) with some alternating motion due to the outrunning phenomena, and this motion may be modified due to the resilience of the structure that is interposed between the soil and the equipment. (The low frequency displacement characteristic of the ground motion is not considered critical for hard mounted high frequency equipment.) If the structure moves essentially with the ground then the input to the equipment can be assumed to be the free-field motion. On the other hand if the structural element supporting the equipment is excited to the response predicted by shock spectra, then the critical input to the equipment will be a vibratory motion at the frequencies of the structure.

Therefore, a dual shock and vibration requirement is necessary for designing a ground shock simulating test machine. The machine must be capable of producing

(1) a transient excitation defined by a velocity jump of magnitudes found from Table 9.1 and durations defined by the rise times of Table 7.1 combined with a 3 or 4 cycle alternating motion defined by Figure 2.4 with amplitudes discussed in Section 2.2

and

(2) a quasi-steady state vibration with a frequency range of 5 to approximately 500 cps. The peak intensity of the motion of requirement 2 is very difficult to assess, but the spectrum envelope in Figure 3.1 may be considered as an upper bound to the vibratory motion peak intensity. The data points which appear on the figure are taken from the shock spectra of Figures 9.2 to 9.14. These points establish a constant displacement line of 4 inches, a constant velocity line of 340 ips, and a constant acceleration line of 300 g's. This envelope assumes that structure and equipment natural frequencies are greater than 5.5 cps and 10 cps respectively. For extremely low frequency equipment resonances the total ground motion as defined in Figure 2.3 and Table 9.1 should also be considered.

STRUCTURE RESPONSE ENVELOPE

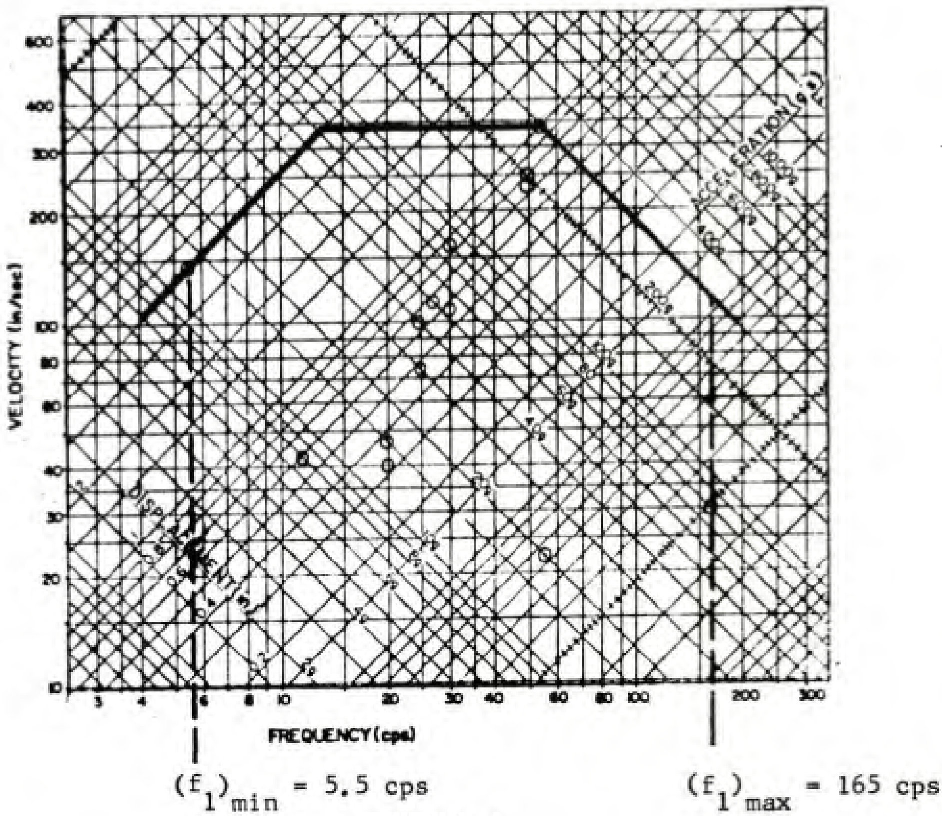


FIGURE 3.1

It must also be remembered that a vertical and horizontal input may occur simultaneously. For example if the machine is to be designed for the vector resultant peak input, then the input acceleration must be equal to the vector sum of the vertical and horizontal components.

4.0 LIMITATIONS OF THE STUDY AND RECOMMENDATIONS FOR FURTHER WORK

This study has attempted to identify the critical shock and vibratory components of the input motion and to ascertain the maximum response of the equipment. It should be remembered that the shock spectra technique of analysis is based on the response of a single-degree-of-freedom system exposed to the shock environment. The waveforms employed are based on simple empirical relationships (and one-dimensional soil theory) for peak ground motion intensities that have reasonable correlation with field test data. The real shock situation has complex characteristics which have to be idealized by gross assumptions so that shock spectra and smooth waveforms become applicable. This study has attempted to use these techniques to account for the real situation so that an upper bound free-field motion and a consequent input motion to the equipment could be established. Therefore the limitations of this method are

(1) The multi-degree-of-freedom response of the structure and of the equipment have not been determined.

(2) The effect of the random nature of the ground motion cannot be estimated with much confidence.

(3) The approximation that the structure motion is equal to some average free-field motion is very rough.

It is recommended that parallel with the preliminary design phase of the test machine, a more refined analysis be performed so as to ascertain if the herein specified shock and vibration requirements are sufficient to qualify the equipment. The structure and ground can be simulated by multi-degree-of-freedom models and exposed to shock effects. The analysis would take into account the higher modes of the structure (and equipment) and the complex properties of the soil media in a composite mathematical model without the need for superimposing the various effects. With the advent of computers this problem is now feasible for solution using numerical methods. A description of a proposed analysis and model is given in Section 10.

5.0 EQUIPMENT CHARACTERISTICS

Five equipments were considered to be mounted internally in each of the structures enumerated in Section 6. For analytical purposes the equipments were idealized to the extent that they were considered to be single-degree-of-freedom (SDOF) systems.

The natural frequencies which were specified have been assumed to be related to horizontal motions. Vertical frequencies have been taken as twice the horizontal frequencies.

All equipments are mounted at the lowest horizontal level in the structure except that the microwave transmitter and receiver is normally mounted on a vertical surface. All the equipments (identified below) are assumed to be mounted at the most responsive point on the particular section of the structure to which it is attached.

Equipment Name	Numeric Identifier	Weight lb.	Horizontal Natural Freq. cps
Microwave R and T	1	130	50
Base Station Transceiver	2	300	25
Tape Unit	3	800	10
Air Compressor	4	300	40
Generator Set	5	1170	20

Drawings of two of the equipments are included in Figures 5.1 and 5.2. It should be noted that each piece of gear can be approximated as a lumped mass supported on a spring element. In general, most pieces of equipment exhibit one strong resonant frequency and perhaps several additional but less significant resonances. For the purpose of determining the gross response of the equipment, consideration of the most pronounced resonant mode is usually sufficient. The report makes use of this assumption to obtain quantitative results. The effects of additional resonant modes are discussed qualitatively in Sections 2 and 3.

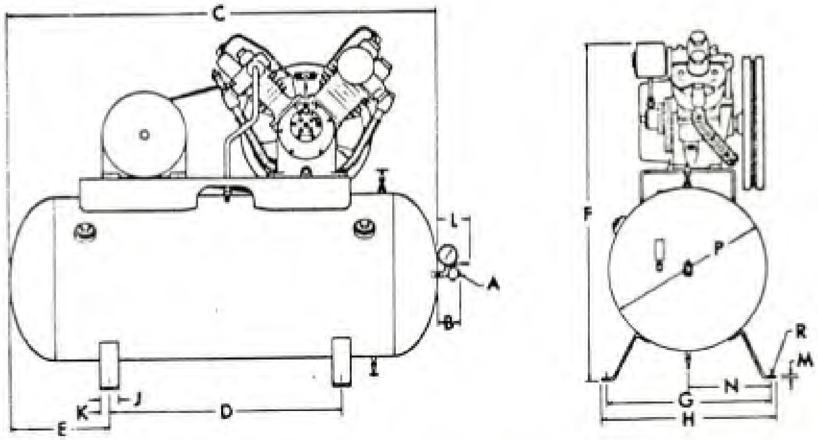


FIGURE 5.1 AIR COMPRESSOR

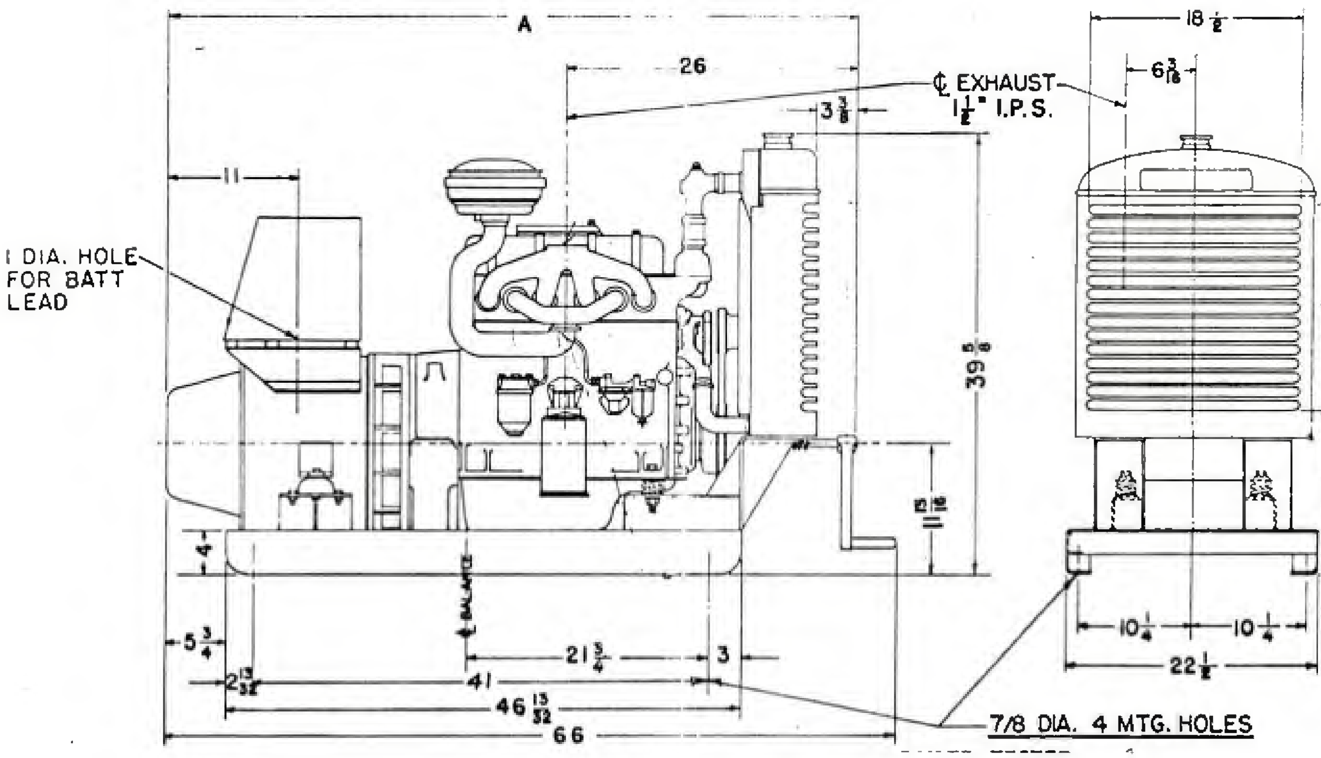


FIGURE 5.2 DIESEL-GENERATOR SET

6.0 STRUCTURES CONSIDERED

A preliminary design was performed for the six structural configurations specified in order that the influence of structural characteristics could be considered in calculating shock input (or shock spectrum response) parameters to the equipment internally mounted. The design was carried out in sufficient detail so that mass and stiffness characteristics could be assessed in an approximate manner. Only the larger yield weapon (20 MT) was considered because the structures are primarily pressure sensitive and the refinement of altering structural characteristics with yield (1 MT to 20 MT) would not be significant for this study.

6.1 ABOVEGROUND RECTANGULAR STRUCTURE

The overpressure region this type of structure must survive is specified as 15 psi. The walls are designed for the reflected air pressure treated as an impulse spike on the overpressure pulse. The roof slab and foundation slab are assumed to have identical response behavior to the overpressure pulse. The structure's dimensions and characteristics are shown in Figure 6.1.

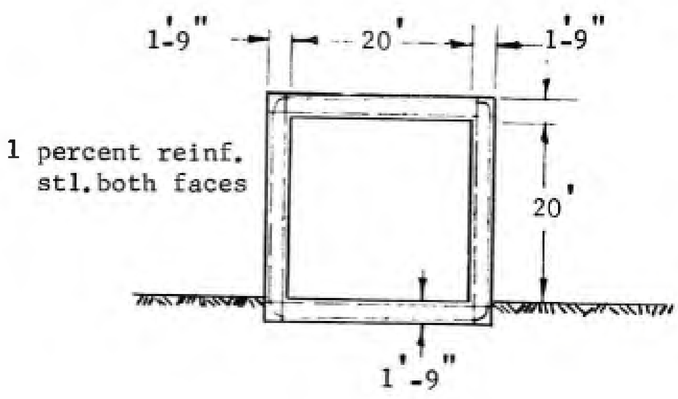


FIGURE 6.1

6.2 PARTIALLY BURIED STRUCTURE (TOWER)

A vertical cylindrical structure, 100 feet long, has a 60 feet projection above grade. This structure is designed to survive in the 50 psi overpressure region. (Its design is not performed to verify the tower's capability but only to establish approximate structural characteristics.)

The cylindrical structure is assumed to be of composite concrete-steel construction and is very sensitive to the drag effect associated with the dynamic pressure. In the 50 psi overpressure region the blast wave precursor (due to terrain effects) may produce an amplified dynamic pressure which is greater than the idealized dynamic pressure, but in this particular case the peak dynamic pressure is estimated to be 130 psi. This peak value with a drag coefficient of 0.4 was used in calculating the properties at the critical section. The structure's dimensions and characteristics are shown in Figure 6.2.

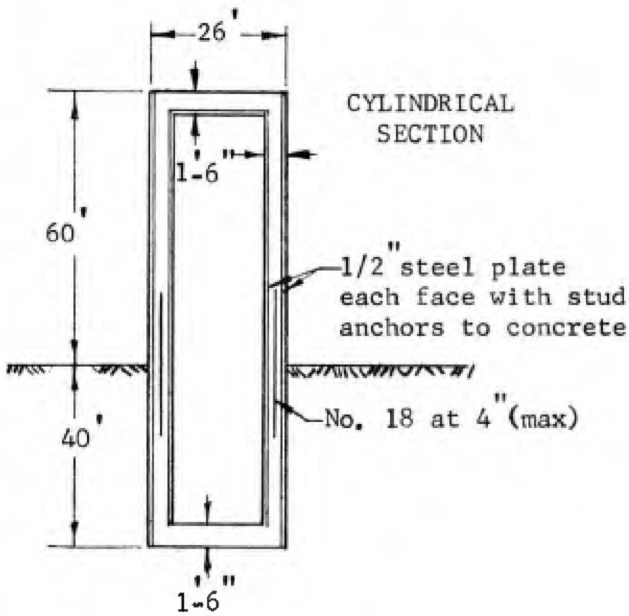


FIGURE 6.2

6.3 BURIED RECTANGULAR STRUCTURE

A rectangular structure flush with the surface will feel the peak overpressure effects with very little attenuation. The walls are therefore designed for a substantial fraction of the peak overpressure and for no reflected pressure. If the water table is near the surface the peak horizontal pressure on the walls is assumed to be equal to the peak overpressure. For the specified overpressure region of 100 psi the top and bottom slab and walls should be about 2'-6" thick,with tension reinforcing at mid-spans and supports equal to 1.3 percent as shown in Figure 6.3.

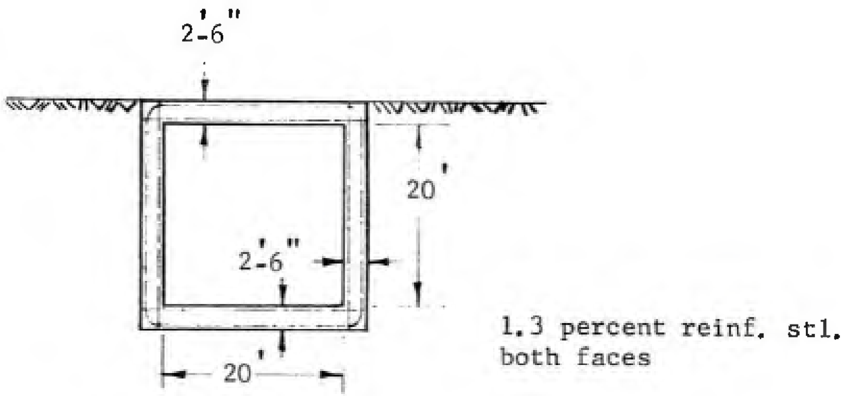


FIGURE 6.3

6.4 BURIED STEEL ARCH STRUCTURE

A steel arch structure supported by a concrete foundation, with a 25 feet burial depth above the crown has been specified. Since the arch has sufficient earth cover the compression mode will be the predominating mode of yielding. It was found that an arch thickness of 1 3/4' steel plate will be adequate for both dynamic and static loads for the 200 psi overpressure and earth cover. An equivalent ribbed or corrugated steel section may be assumed for the arch.

Moreover, a foundation thickness of 2'-6" is considered adequate for this condition. See Figure 6.4.

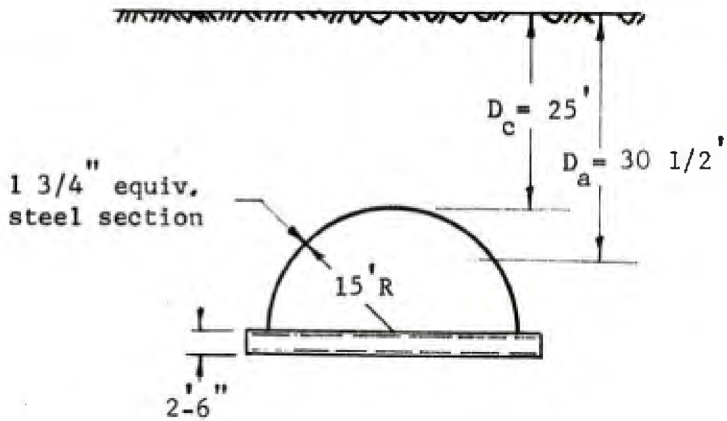


FIGURE 6.4

6.5 BURIED HORIZONTAL CYLINDRICAL STRUCTURE

A buried cylindrical structure designed to survive at the 500 psi overpressure region will most probably be a cast-in-place heavy reinforced concrete structure. The compression mode will be predominant and a 4 foot wall thickness will be adequate with 0.5 percent reinforcing steel on each face in both the circumferential and longitudinal directions. See Figure 6.5.

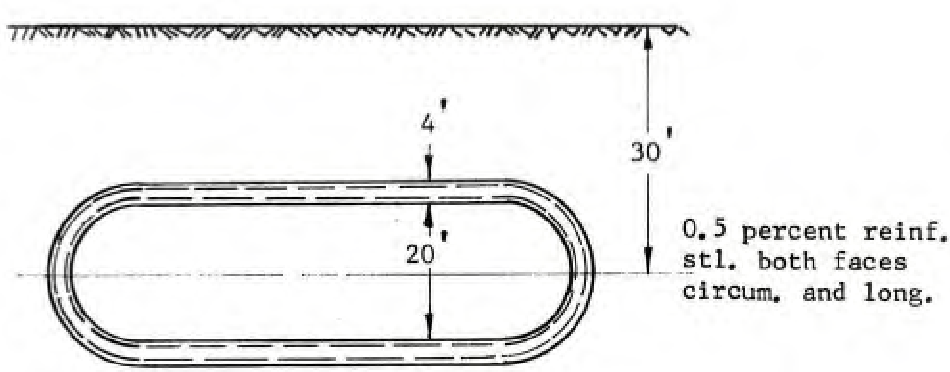


FIGURE 6.5

6.6 BURIED VERTICAL CYLINDRICAL STRUCTURE

The vertical cylindrical structure with the roof flush with the ground surface is required to survive at the 500 psi overpressure level. A reinforced concrete section is considered feasible for this structure. The upper portion of the cylinder wall must be thicker than the lower portion to account for the non-uniform horizontal pressure effect near the surface which produces bending as well as radial compression in the cylinder. The section assumed for this study is shown in Figure 6.6.

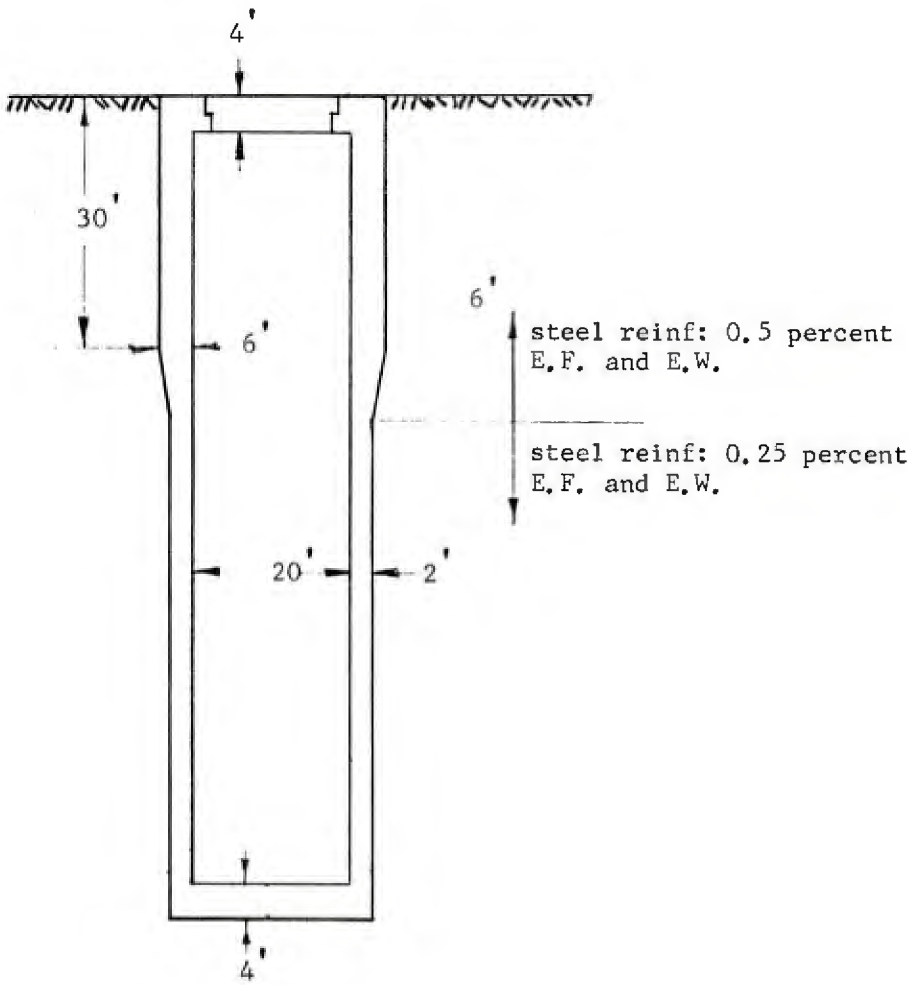


FIGURE 6.6

7.0 FREE-FIELD GROUND MOTIONS

7.1 INTRODUCTION

A nuclear burst occurring near the surface will initiate direct seismic disturbances as well as airblast disturbances. Moreover, the expanding air blast front's interaction with the ground will continue to generate additional air blast induced seismic disturbances as the blast front travels over the surface away from ground zero.

Since the peak overpressure of the blast front decays rapidly to a relatively low overpressure behind the front, the blast-pressure representation on the ground may be approximated by an expanding circular ring load followed by a less intense circular area load. This fact accounts for the relatively intense seismic disturbance that is excited or induced at the air blast front.

7.1.1 Blast Wave

Based mainly on field observations, the characteristics of the traveling blast front and its peak overpressure can be expressed approximately by:

$$r_p \approx 6\, W^{\frac{1}{6}}\, t_a^{\frac{1}{2}} \approx 43\, W^{\frac{1}{3}}/p_{so}^{\frac{1}{2}} \quad \text{, in kilo feet}$$

or identically by:

$$p_{so} \approx 51\, W^{\frac{1}{3}}/t_a \approx 1850\, W^{\frac{2}{3}}/r_p^{2} \quad \text{, in psi}$$

in which

r_p = distance from crater edge r_c to blastfront, in kilo feet

W = yield of blast, in M.T.

t_a = arrival time of front at r_p after detonation at G.Z., in seconds

p_{so} = peak overpressure at r_p, in psi

r_c = radius of crater edge = $W^{1/3}$ in kilo feet

7.1.2 Seismic Waves

Unlike the air disturbance of the nuclear blast, the induced seismic ground disturbances are much more involved since they depend on the seismic properties of the ground intervening between the bomb crater at G.Z. and the receiving station or target facility. If the facility is located on the surface it will experience three distinct types of seismic waves besides the air blast itself. The seismic waves are:

a) Dilatational or push-pull waves

b) Equivoluminal or shear waves

c) Surface waves

If the facility is deeply buried it will experience only a) and b) waves.

If the seismic properties of the ground are homogeneous and constant, a consideration of the direct and of the blast induced ground motions is relatively simple since in this case there will be neither refractions nor reflections to complicate the picture.

If the seismic velocity of the ground increases uniformly with depth, the refractive phenomenon by itself will tend to direct the major part of the seismic energy to the surface with the result that facilities located near the surface will then experience more ground motion than if the soil were uniform.

If horizontal stratifications or layering occur so as to present discontinuities in the soil's seismic properties, and if as before the seismic velocities increase with depth, (this is usually the case) then the reflective phenomenon together with the refractive will tend strongly to direct the seismic energy to the surface. In this case a refractive component designated as the "critical" one will be of especial consequence.

7.1.3 Outrunning Phenomena

If the seismic velocity increases with depth, and especially if horizontal stratifications exist it is possible that the direct seismic disturbance as well as the blast front induced disturbance may arrive at the target ahead of the blast front itself. This phenomenon is called outrunning. The importance of outrunning stems from two facts:

1) The initial motion of the outrunning phenomenon is directed upward while the motion induced by the direct blast constitutes an initially downward slap. Thus if a structural element, as a silo cover, is designed mainly for downward blast slap an outrunning, initially upward motion may have lifted the element off its support so that when the blast slap arrives it may cause a structural impact in addition to the air pressure impact.

2) Due to the generation patterns of the seismic waves as well as to the complexity of refractions and reflections from the non-uniform seismic characteristics of the ground, the outrunning waves, after their initial upward motion, will present alternations. While these alternations will vary in amplitude and frequency they nevertheless constitute a danger in regard to quasi-resonance build up in the structure. In other words, the alternating type of seismic motion is very different from the type due to direct blast slap.

Outrunning distances and times are determined for either direct or for air blast induced disturbances by superposing seismic distance-travel time curves on appropriate blast-front distance-travel time curves. Thus, if a facility, experiencing a stated overpressure p_{so}, is located on the surface at distance r_f from G.Z. the blast front will arrive at r_f in time t_f given by

$$t_f \approx 51\ W^{\frac{1}{3}}/p_{so} \approx r_f^{\ 2}/(36\ W^{\frac{1}{3}}) \text{ seconds}$$

If an outrunning motion, as determined by distance-travel time curves, occurs at distance r_o at time t_o that is $r_o < r_f$ and $t_o < t_f$, then the outrunning seismic motion will arrive at the facility at a time t_f' which is given by

$$t_f' \approx t_o + (r_f - r_o)/c_\ell \quad \text{seconds}$$

in which c_ℓ is an equivalent seismic velocity depending on the seismic characteristics of the layered ground intervening between r_f and r_o. Thus, if a relatively thin top layer's seismic velocity c_1 is considerably lower than c_2 of the second and relatively deep layer's velocity, c_ℓ will be approximately equal to c_2. If the second layer is also relatively thin with respect to a third layer whose seismic velocity $c_3 > c_2$, the equivalent c_ℓ will be approximately c_3.

Assuming that c_ℓ can be estimated, then the air blast (downward slap) disturbance will arrive at the facility at a phasing time

$$t_\phi = t_f - t_f' \text{ seconds}$$

later than that of the initially upward seismic disturbance generated either by a direct G.Z. disturbance or by an air blast induced (ring load) disturbance. Accordingly, if t_ϕ is positive outrunning will occur, but if r_o is greater than r_f then $t_o > t_f$ and the phasing time

$$t_\phi = t_f - t_o \text{ seconds}$$

will be negative. This means that the air blast (downward slap) disturbance will arrive ahead of any initially upward seismic disturbance. In many cases only the air blast induced disturbance will be of consequence.

7.2 QUANTITATIVE ASPECTS OF FREE-FIELD GROUND MOTIONS

It must be understood that the accuracy of the quantitative aspects of free field motions is open to question and to future improvement as the knowledge gained by tests and theory expands.

Nearly all our information is obtained from Reference 7.1 and relates to the types of motion characterized by the term Type I or systematic motion that exhibits a rather short duration of acceleration, a considerably longer duration of velocity, and a still longer duration of displacement with or without complete return to the original location, see Figure 7.1. The alternating type of motion, Reference 1.1 is shown in Figure 7.2 and is referred to as Type II; it is less well-known and therefore is still debatable.

7.2.1 Peak Vertical Motions of Directly or G.Z. Induced Disturbances

For distances of r_f from G.Z. less than or equal to 2.5 $W^{1/3}$ kilofeet, the peak vertical values for a W megaton surface burst are approximately given by

$$a_v = 0.4\, W^{5/6}\, r_f^{-3.5}\, c_{eq}^{2} \quad \text{in g units acceleration}$$

$$v_v = 12\, W^{5/6}\, r_f^{-2.5}\, c_{eq} \quad \text{in inches per second}$$

$$d_v = 4\, W^{5/6}\, r_f^{-1.5} \quad \text{in inches}$$

in which c_{eq} is an equivalent seismic velocity in kilofeet-sec depending on a subjective evaluation of the seismic properties intervening between the crater and the facility. For a layered soil an evaluation of an equivalent value of c_{eq} can only be guided by a stratigraphic plot. Great uncertainty will remain, especially in regard to the value of d_v in the expression of which c_{eq} does not even occur.

For distances of r_f equal to or greater than 2.5 $W^{\frac{1}{3}}$ kilofeet a slower attenuation rate is indicated, so that the peak values are given by

$$a_v = 0.081\, W^{\frac{1}{4}}\, r_f^{-1.75}\, c_{eq}^{2} \quad \text{in g units acceleration}$$

$$v_v = 4.8\, W^{\frac{1}{2}}\, r_f^{-1.5}\, c_{eq} \quad \text{in inches per second}$$

$$d_v = 3.2\, W^{\frac{3}{4}}\, r_f^{-1.25} \quad \text{in inches}$$

7.2.2 Peak Vertical Motions of Air Blast Slap Disturbances

In this case the overpressure p_{so}, creating the downward slap, and the equivalent seismic velocity c_{eq} are primarily involved, but obviously p_{so}, W, and r_f and t_f are related as already shown in the discussion of the blast wave.

The peak downward acceleration at the surface, i.e. when z is zero, is very much higher than a short distance below; it is approximately given by

$$a_v = 1.5\, p_{so}/c_{eq} \quad \text{in g units}$$

and the peak downward acceleration at depth z feet is given by

$$a_{vz} = 5 \times 10^{-3}\, p_{so}\, \alpha_z/z \quad \text{in g units}$$

in which

$$\alpha_z = \left(1 + \frac{z}{L_w}\right)^{-1}$$

and

$$L_w = 2300\, W^{\frac{1}{3}}/p_{so}^{\frac{1}{2}}$$

Since it is obvious that the above expression for a_{vz} will not hold for the surface where $z = 0$ and $\alpha_z = 1$, some judgement must be used at small depths where $\alpha_z \approx 1$. In other words

$$a_v \approx a_{vz}$$

when

$$c \approx 300\, z$$

Therefore, if $c_{eq} = 1.5$ kilo ft/sec, z must be at least 5 ft before the expression for a_{vz} can be used.

The peak downward velocity at the surface and at depth z feet is expressed by

$$v_{vz} = 0.5\, p_{so}\, \alpha_z/c_{eq} \quad \text{in inches per second}$$

The peak downward elastic and plastic displacement at the surface, $z = 0$, is expressed by

$$d_v = 0.9\, W^{\frac{1}{3}}\, p_{so}^{\frac{1}{2}}/c_{eq} \quad \text{in inches}$$

The residual downward displacement (plastic) at the surface is given very approximately by

$$d_{vs} = \frac{p_{so} - 40}{7.5\ c_{eq}^2} \qquad \text{in inches}$$

Consequently the peak elastic displacement at the surface is

$$d_{ve} = d_v - d_{vs}$$

The approximate value of the residual downward displacement at a depth z is given by

$$d_{vz} = d_{vs} \left(1 - \frac{z^*}{100} \right) \quad \text{in inches}$$

with

$$z^* = z \text{ ft., for } 0 < z \leq 100 \text{ ft}$$

$$z^* = 100 \text{ ft., for } z \geq 100 \text{ ft}$$

This means that no residual relative displacement will occur below z = 100 ft.

The approximate value of the downward elastic component of the relative displacement between the surface and a point at depth z is given by

$$d_{rze} \approx 2.4 \times 10^{-2}\ p_{so}\ c_{eq}^{-2}\ \frac{z^*}{100} \qquad \text{in inches}$$

A reasonable estimate of the peak vertical displacement at a point z is

$$(d_{vz})_{max} = d_{vz} + d_{ve} - d_{rze} \qquad \text{in inches}$$

The rise time t_r of the velocity from zero to peak value may be taken as

$$t_r \approx \frac{v_{vz}}{a_{vz}} \qquad \text{in seconds}$$

$$\approx \frac{100\ z}{c_{eq}\ g} = 0.26\ \frac{z}{c_{eq}} \quad \text{seconds at z}$$

$$\approx \frac{1}{3g} \approx \frac{1}{1000} \quad \text{seconds at } z = 0$$

The rise time t_e of the elastic and plastic displacement from zero to peak value or the duration of the positive velocity pulse may be taken as

$$t_e \approx \frac{2(d_{vz})_{max}}{v_{vz}} \qquad \text{in seconds at } z$$

$$\approx 3.6\, W^{\frac{1}{3}}\, p_{so}^{-\frac{1}{2}} \qquad \text{in seconds at } z = 0$$

7.2.3 Peak Horizontal Motions of Air Blast Slap Disturbances

Very little quantitative knowledge is available. However, it is considered to be good practice to assume that:

Horizontal accelerations = Vertical accelerations

" velocities $= \frac{2}{3}$ " velocities

" displacements $= \frac{1}{3}$ " displacements

7.2.4 Idealization of Systematic Type I Motions

Field tests show motions typically similar to the sketches in Figure 7.3.

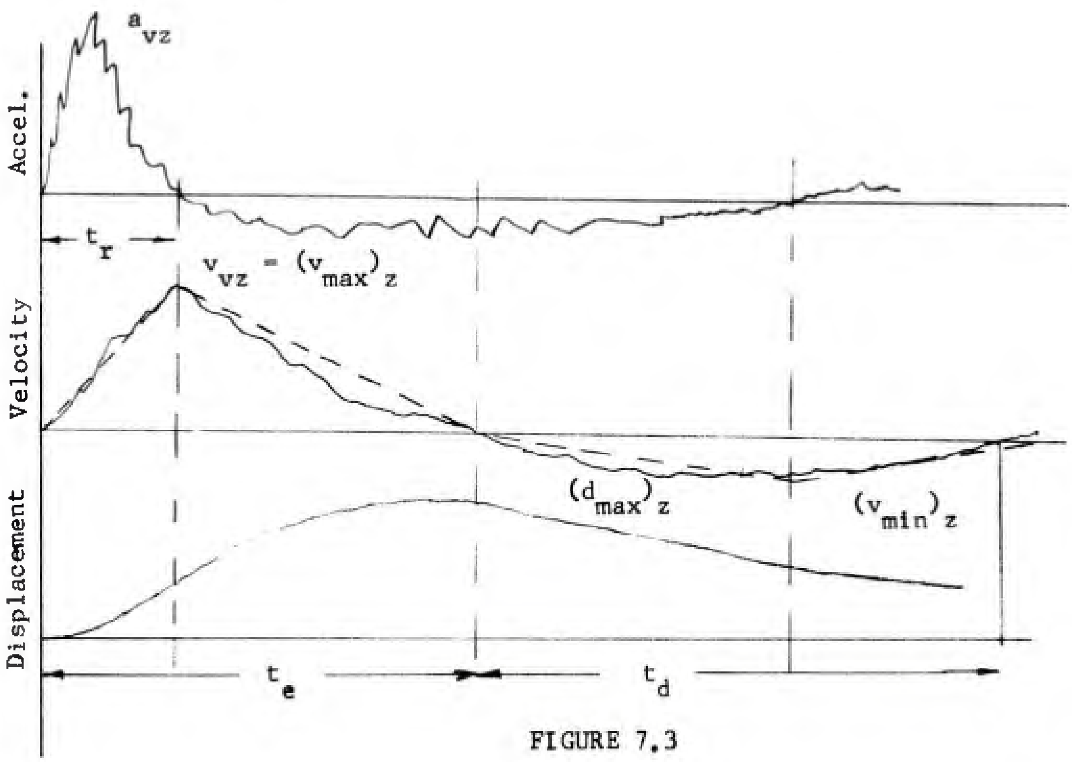


FIGURE 7.3

For computational purposes these curves are usually idealized in such a manner that the particle velocity curve is of triangular shape as indicated by the dotted lines. As a consequence of this, the acceleration curve becomes a block or step curve, and the displacement curve assumes parabolic shapes. For a complete return of displacement the area under the positive velocity curve therefore must equal the area under the negative curve, i.e.

$$(v_{max})_z \, t_e = (v_{min})_z \, t_d \qquad \text{inches/sec}$$

If a residual displacement, d_{vz} occurs, its magnitude will be given by:

$$d_{vz} = \frac{1}{2} (v_{max})_z \, t_e - \frac{1}{2} (v_{min})_z \, t_d \quad \text{inches}$$

In regard to the numerical values of Figure 7.3 we let

$$(v_{max})_z \approx \frac{p_{so}}{2c_{eq}} \alpha_z \qquad \text{in/sec}$$

$$t_r \approx 100 \frac{z}{c_{eq} g} = 0.26 \frac{z}{c_{eq}} \quad \text{seconds}$$

The questions of what values to assign to t_e, t_d, and $(v_{min})_z$ do not have simple answers.

Since the duration D_p^+ of the positive overpressure is relatively long, in the order of seconds, and since its attenuation with time is rapid, it must not be assumed that the expression for $(v_{max})_z$, involving only the peak overpressure p_{so}, will indicate a positive velocity for the entire duration D_p^+. Instead, since an impulse necessarily must be of relatively short duration with respect to the time element of the system it acts upon, it is justified to assume that the entire time integral of the positive overpressure at a specific location can be represented sufficiently well by a linear triangular pressure-time pulse involving p_{so} as a maximum and t_e as an effective duration so that in general

$$\text{Total Impulse} \int_0^{D_p^+} p \, dt = \frac{1}{2} p_{so} \, t_e \quad \text{psi seconds}$$

It can be shown that under these circumstances

$$t_e \approx \frac{2(d_{max})_z}{(v_{max})_z} \qquad \text{seconds}$$

or

$$t_e \approx 3.6\, W^{\frac{1}{3}}\, p_{so}^{-\frac{1}{2}} \qquad \text{seconds}$$

What happens later than t_e is even more problematic. It is obvious that the velocity will be reduced to zero and that a residual displacement may or may not result. Accordingly the time element t_d and the velocity element $(v_{min})_z$ are largely conjectural. It is often assumed that

$$2\, t_e \leq t_d \leq 4\, t_e \qquad \text{seconds}$$

Therefore since the residual displacement d_{vz} at z is given by

$$d_{vz} = \frac{p_{so} - 40}{7.5\, c_{eq}^{2}} \left(1 - \frac{z^*}{100}\right) \quad \text{inches}$$

the value of $(v_{min})_z$ can be expressed by

$$(v_{min})_z = \frac{(v_{max})_z\, t_e - 2\, d_{vz}}{t_d}$$

7.2.5 Idealization of Direct Seismic Motion from G.Z.

Test results are typically similar to those in Figure 7.3, and a triangular velocity pulse is assumed, but the numerical value of the maximum velocity is then taken as

$$v_{max} = \begin{cases} 12\; W^{\frac{5}{6}}\; r_f^{-2.5}\; c\,; & \text{if } r_f \leq 2.5\, W^{\frac{1}{3}} \text{ kilofeet} \\ 4.8\; W^{\frac{1}{2}}\; r_f^{-1.5}\; c\,; & \text{if } r_f \geq 2.5\, W^{\frac{1}{3}} \end{cases}$$

The duration of the positive velocity pulse t_e, Figure 7.3, is assumed to be related to the transit time t_f from GZ to r_f, namely

$$t_f \approx 51\, W^{\frac{1}{3}}\, p^{-1} \approx r_f^{\,2}/(36\, W^{\frac{1}{3}})$$

It is then common to take

$$\frac{1}{3}\, t_f < t_e < \frac{2}{3}\, t_f$$

However, unlike the idealization under 7.2.6, the rise time t_r is assumed not to depend on the ratio of maximum velocity to maximum acceleration but it is assumed to be about 1/3 of t_e. Finally the subsidence time t_d is ordinarily taken to be equal to t_r and the residual displacements are nearly always assumed to be zero.

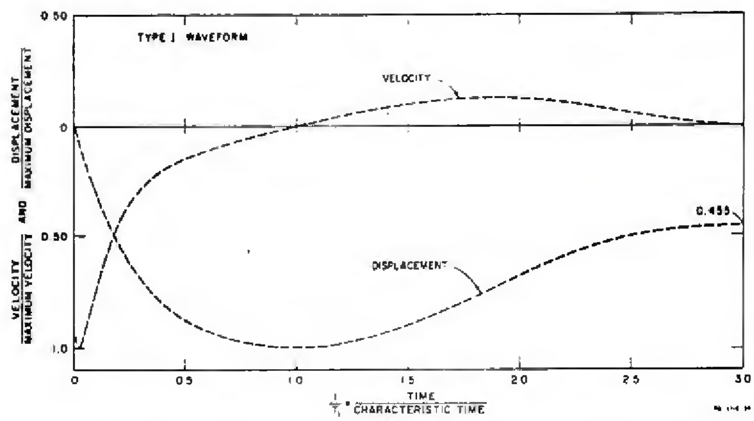
As stated before, the directly induced seismic wave from GZ is usually neglected except for very close in facilities subjected to very high overpressures.

7.2.4 Character of Seismic Motions Induced by Air Blast

It has already been mentioned that the downward air pressure slap motions of systematic Type I will induce seismic motions with an alternating character, Type II. Due to the complexity of crater and air blast loading geometries as well as due to the complexity of seismic refractions and reflections in the ground, it has been proposed to consider the alternating Type II a random motion, characterized by a mean power spectral density value within a limited frequency domain, but neither the mean power spectral density value nor the limited frequency domain has as yet been quantitatively defined.

An analysis of about a dozen field records by Stanford Research Institute Reference 1.1 have resulted in separating out and normalizing the velocity wave shape of the alternating Type II component. The two types of velocity wave form are shown in Figures 7.1 and 7.2.

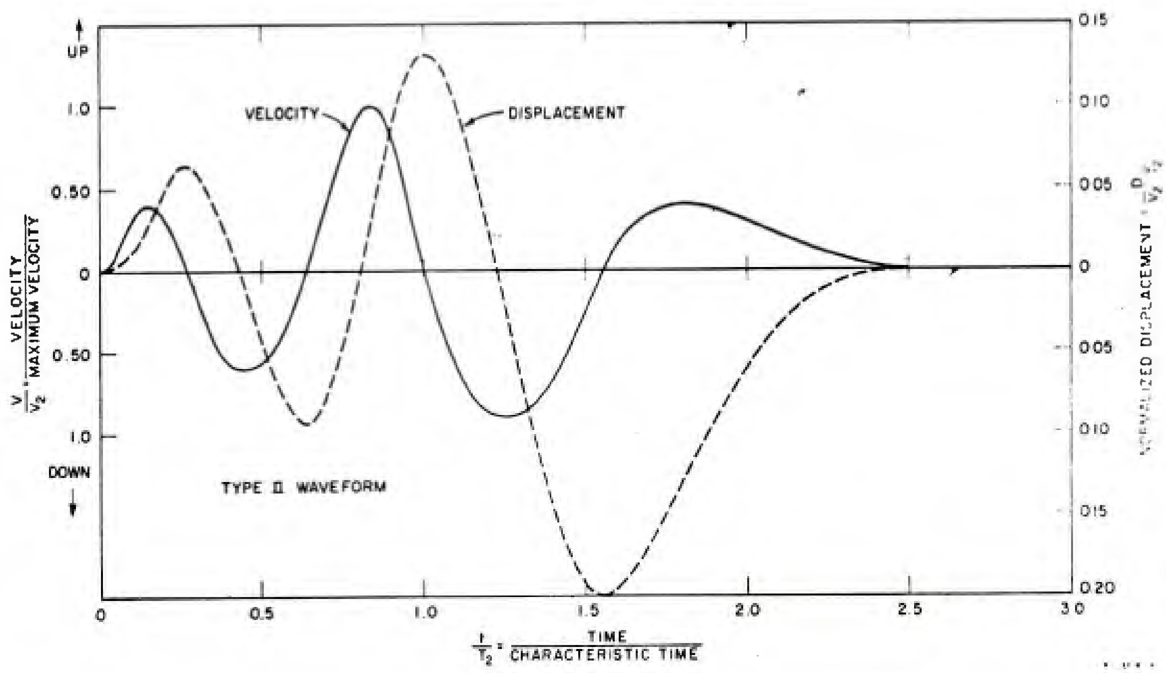
The characteristic duration D_2 of the Type II wave does not appear to scale with the cube root of the yield as does the duration D_p^+ of the



TYPE I (SUPERSEISMIC AIRBLAST) VERTICAL PARTICLE VELOCITY WAVEFORM AND ITS DISPLACEMENT

(taken from Reference 1.1)

FIGURE 7.1



TYPE II (OUTRUNNING GROUND MOTION) VERTICAL PARTICLE VELOCITY WAVEFORM AND ITS DISPLACEMENT

(taken from Reference 1.1)

FIGURE 7.2

positive phase of the overpressure wave. Tentatively the duration is given by

$$D_2 \sim 0.10 + \frac{r_f - r_o}{4} \qquad \text{in seconds}$$

when, as before, r_f and r_o are expressed in kilofeet. Figure 7.4 shows six phased diagrams of both types of motion relating to a specific yield (1 MT) and to a specific soil in which outrunning begins at r_o = 5720 ft. where p_{so} is 57 psi. Since the S.R.I. analysis is based on the relatively few field tests carried out on unconfined bursts before the test ban, only the shape and duration of the alternating motion is given; its magnitude has not been related to bomb yield, distance, and seismic properties of ground. However in many of the tests analyzed by S.R.I. the peak values of Type I and Type II motions were of the same order of magnitude. The six diagrams of Figure 7.4 arbitrarily show equal peak velocity amplitudes of the two types of motion. It should be noted that the six phasing phenomena range between $t_\phi = 0$, at $r_o = r_f$ = 5720 ft, with p_{so} = 57 psi, and t_ϕ = 0.324 sec, at r_f = 6820 ft, with p_{so} = 47 psi.

7.2.6 Intensity of Outrunning Air Blast Induced Ground Motion

If outrunning begins at r_o, t_o and $(p_{so})_o$, and if the facility is located at r_f, then as already stated the intensity of the outrunning seismic Type II motion at r_o is neither well understood nor quantitatively defined. Therefore it is customary for a design criterion bluntly to assume that the alternating Type II motion can be replaced by a fraction m of the systematic Type I motion (downward slap) occurring at the outrunning distance r_o and depending in magnitude on $(p_{so})_o$ so that:

$$(p'_{so})_o \approx m\,(p_{so})_o$$

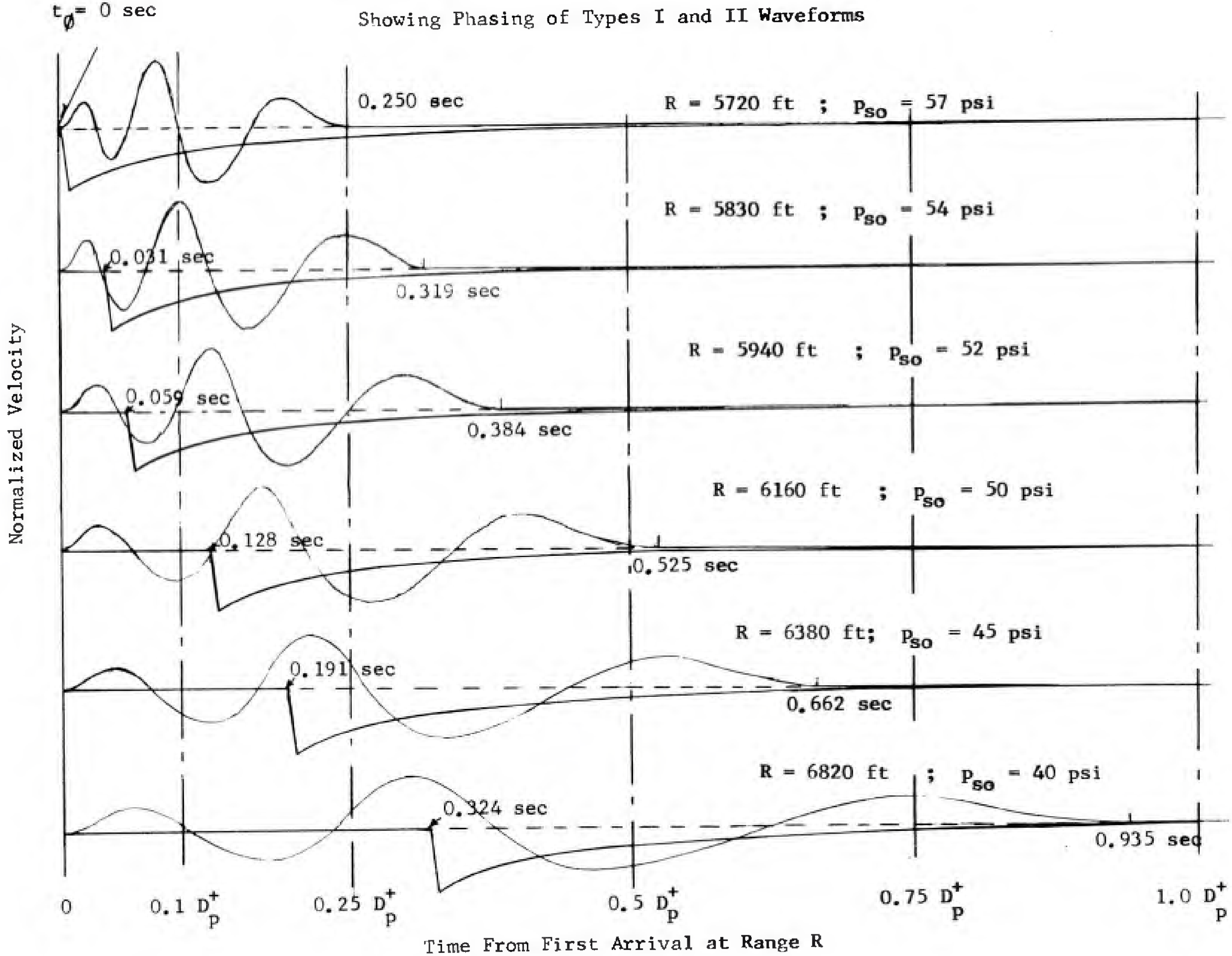
Under these circumstances the outrunning seismic motion of r_f may be thought of as related to a seismically attenuated equivalent negative overpressure $(p'_{so})_f$ which is given by

$$(p'_{so})_f \approx m\,(p_{so})_o \left(\frac{r_f}{r_o}\right)^{-n}$$

FIGURE 7.4

1 MT, Soil c Outrunning at 5720 ft from G.Z.

Showing Phasing of Types I and II Waveforms



in this expression the exponent n is taken as 2.5 if $(r_o + r_f)/2$ is less than 2.5 $W^{1/3}$ kilofeet, and for $(r_o + r_f)/2$ greater than 2.5 $W^{1/3}$ kilofeet n is taken as 1.5. Since the calculations are very rough, sufficient accuracy for engineering purposes may be obtained by assuming that n = 2. Which means that the p_{so} equivalent attenuation in the seismic medium is the same as the one in air. Accordingly, for n = 2, the equivalent negative overpressure at r_f becomes

$$(p'_{so})_f = m\ (p_{so})_f$$

Since the value of m is problematic, a parametric study should be made, and it will usually assign a variation of m from unity to 1/3.

Figure 7.5 shows the six phased diagrams of the Type I motion and the bluntly assumed design criterion (Type I) motion for the specific 1 MT yield and specific soil in which outrunning begins at r_o = 5720 feet. A fraction of m = 2/3 has been assumed; consequently at $r_o = r_f$ = 5720; $t_\phi = 0$; $(p_{so})_f$ = 57 psi; and $(p'_{so})_f$ = 38 psi, it can be concluded that the peak velocities at z = 0 will be

$$v_I = \frac{57}{2\ c_{eq}} \text{ in per sec. for the initially downward slap component}$$

$$v'_I = \frac{38}{2\ c_{eq}} \text{ in per sec. for the initially upward seismic component}$$

If c_{eq} = 1.5 kilofeet per second

v_I = 19 in/sec, and v'_I =-12.7 in/sec

at r_f = 6820 ft; t_ϕ = 0.324 sec; $(p_{so})_f$ = 40 psi; and $(p'_{so})_f$ = 27 psi

v_I = 13 in/sec , and v'_I = -9 in/sec

7.3 NUMERICAL RESULTS OF GROUND MOTION INPUTS TO THE SIX STRUCTURES LOCATED ON THE THREE SOILS

7.3.1 General Considerations

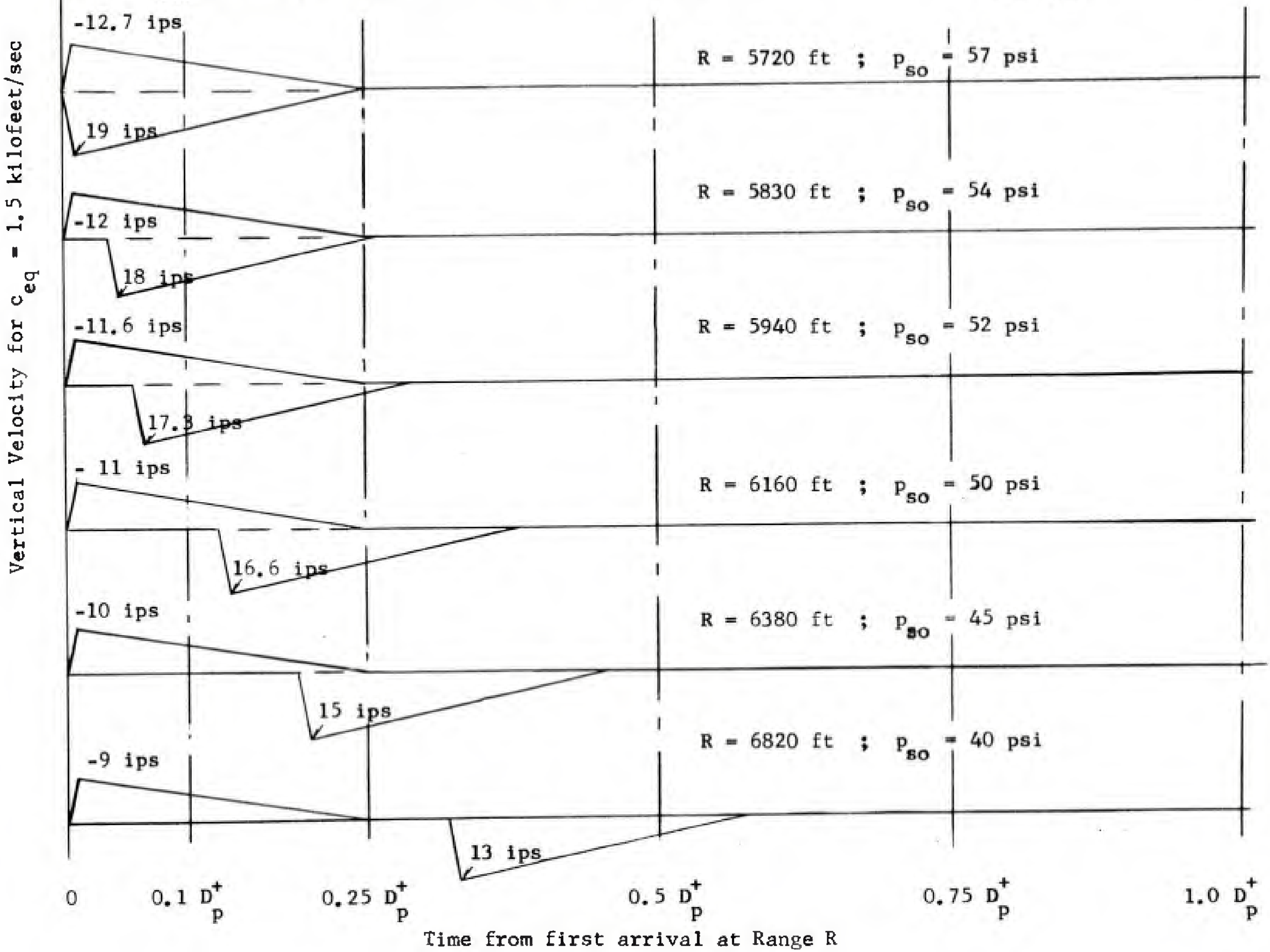
Due to the fact that two of the three soils are layered, the seismic

FIGURE 7.5

1 MT Yield, Soil c Outrunning at 5720 ft from G.Z.

Showing Phasing of Idealized Triangular Waveforms

Due to Downward Slap and to Air Blast Induced Outrunning Motion for m = 2/3



outrunning distances in them have been determined from a superposition of travel time-distance plots of 1) the blast waves for 1, 10, and 20 MT bursts, and 2) the seismic travel time-distance plots for the two and for the three layer soils. These outrunning distances and the consequent phasing times are therefore considered to be relatively accurate.

In regard to the use of equivalent seismic velocities c_{eq} the numerical results obtained by the expressions have been averaged subjectively, taking into account the soil stratigraphy, the location distance from G.Z., and the below surface depth of each of the six structures.

Since the accelerations at the surface, as given by the expression involving c_{eq}, do not agree when $z = 0$ with the accelerations as given by the expression involving only the depth z, the acceleration obtained by the latter expression for z = 5 ft has been averaged with the one at the surface involving c_{eq}.

A substitution of the numerical data in the expressions for the direct, G.Z. induced seismic motions shows that they are considerably smaller than the air blast induced seismic motions. Consequently the direct G.Z. induced motions have been neglected in this study.

7.3.2 Specific Calculations

As an example consider structure number 6 located in soils (a), (b), and (c) and exposed to a 10 MT burst.

Blast environment

Overpressure $= p_{so} = 500$ psi (specified)

Location $r_p = 43\, W^{\frac{1}{3}}/p_{so}^{\frac{1}{2}}$, kilofeet

$= 92.8/22.4 = 4.14$ kilofeet

$r_c = 1\, W^{\frac{1}{3}} = 2.16$ kilofeet

Distance $r_f = r_p + r_c = 6.3$ kilofeet $= 1.2$ miles

$$\text{Blast arrival time } t_a = 51\, W^{\frac{1}{3}}/p_{so}$$

$$= 110/500 = 0.22 \text{ sec}$$

Acceleration

$$\text{Max accel at surface} = a_o = \frac{3\, p_{so}}{c_{eq}} = \frac{1500}{c_{eq}} = \begin{cases} 500\text{g soil} & \text{(a)} \\ 1000\text{g} & \text{(b)} \\ 1000\text{g} & \text{(c)} \end{cases}$$

$$\text{" \quad " \quad " } z = 5' = a_{vz} = \frac{5\, p_{so}}{z}\, \alpha_z = \frac{2500}{5}\, 1 = 500\text{g}$$

$$\text{" \quad " \quad " } z = 50' \qquad = \frac{2500}{50}\, 0.9 = 45\text{g}$$

$$\text{" \quad " \quad " } z = 100' \qquad = \frac{2500}{100}\, 0.8 = 20\text{g}$$

If the max accel of structure number 6 is obtained simply by averaging accelerations at the three levels

$$(a_v)_6 = \frac{1}{3}\left[\frac{a_o + a_5}{2} + a_{50} + a_{100}\right]$$

$$= \frac{1}{3}\,(500 + 45 + 20) = 190\text{g soil (a)}$$

$$= \frac{1}{3}\,(750 + 45 + 20) = 270\text{g} \quad \text{(b)}$$

$$= \frac{1}{3}\,(750 + 45 + 20) = 270\text{g} \quad \text{(c)}$$

These values are undoubtedly high due to the extremely large acceleration of the soil at $z = 5$. They lead to very short rise times of the velocity. An alternative estimate leading to smaller values of acceleration will follow the calculations of maximum velocity below.

Velocity

$$\text{Maximum velocity } v_{rz} = \frac{p_{so}}{c_{eq}}\, \alpha_z$$

$$\text{Maximum velocity at } z=5' = \frac{500}{c_{eq}}\,1.0 = \begin{cases} 167 \text{ ips} & \text{soil (a)} \\ 333 & \text{(b)} \\ 333 & \text{(c)} \end{cases}$$

$$\text{" \quad " at } z=50' = \frac{500}{c_{eq}}\,0.8 = \begin{cases} 133 \text{ ips} & \text{(a)} \\ 27 & \text{(b)} \\ 133 & \text{(c)} \end{cases}$$

$$\text{" \quad " at } z=100' = \frac{500}{c_{eq}}\,0.6 = \begin{cases} 100 \text{ ips} & \text{(a)} \\ 20 & \text{(b)} \\ 100 & \text{(c)} \end{cases}$$

Max. velocity of structure number 6,

$$(v_v)_6 = \frac{1}{3}(v_o + v_{50} + v_{100})$$

$$= \frac{1}{3}(167 + 133 + 100) = 132 \text{ ips soil (a)}$$

$$= \frac{1}{3}(333 + 27 + 20) = 127 \text{ ips soil (b)}$$

$$= \frac{1}{3}(333 + 133 + 100) = 187 \text{ ips soil (c)}$$

A second indirect method of averaging acceleration will be presented. It is based on averaging the rise time of the velocity at the three levels. Thus at the

$$5' \text{ level, } t_r = \frac{v_5}{a_5 g} = \begin{cases} \frac{167}{500g} = 0.9 \text{ millisec,} & \text{soil (a)} \\ \frac{333}{750g} = 1.1 \quad , & \text{(b)} \\ \frac{333}{750g} = 1.1 \quad , & \text{(c)} \end{cases}$$

$$50' \text{ level, } t_r = \frac{v_{50}}{a_{50} g} = \begin{cases} \frac{133}{45g} = 3.8 \quad , & \text{(a)} \\ \frac{27}{45g} = 15.5 \quad , & \text{(b)} \\ \frac{133}{45g} = 3.8 \quad , & \text{(c)} \end{cases}$$

$$100' \text{ level, } t_r \quad \frac{v_{100}}{a_{100}g} = \begin{cases} \frac{100}{20g} & = 13 \text{ millisec, soil (a)} \\ \frac{20}{20g} & = 2.6 \quad , \quad \text{(b)} \\ \frac{100}{20g} & = 13 \quad , \quad \text{(c)} \end{cases}$$

Accordingly

$$(t_r)_{av} = \begin{cases} \frac{1}{3}(.9 + 3.8 + 13) & = 5.9 \text{ millisec, soil (a)} \\ \frac{1}{3}(1.1 + 15.5 + 2.6) & = 6.4 \quad , \quad \text{(b)} \\ \frac{1}{3}(1.1 + 3.8 + 13) & = 6.0 \quad , \quad \text{(c)} \end{cases}$$

and therefore

$$(a_v)_6 = \frac{(v_v)_6}{(t_r)_{av}} = \begin{cases} \frac{132}{5.9} \times \frac{1000}{g} & = 58g \quad , \quad \text{(a)} \\ \frac{127}{6.4} \times \frac{1000}{g} & = 51g \quad , \quad \text{(b)} \\ \frac{187}{6.0} \times \frac{1000}{g} & = 81g \quad , \quad \text{(c)} \end{cases}$$

It is believed that these accelerations being about one-fourth of the previous, are the more realistic ones, but for design purposes and for equipment testing the higher values should be given full consideration.

<u>Displacement</u>

Max elasto plast. displ. = $d_v = 75/c_{eq}$

$$= \begin{cases} 25 \text{ inches} & 25'' \text{ for soil (a)} \\ 50/2 \text{ to } 5 \times .8 = 4 \text{ or} & 15'' \quad \text{(b)} \\ 50 \text{ to } 25 \times .6 = 15 \text{ or} & 33'' \quad \text{(c)} \end{cases}$$

Residual plastic displ. at surf $d_{v_s} = \dfrac{P_{so} - 40}{7.5\, c_{eq}^2} = \dfrac{61}{c_{eq}^2}$

$$= \begin{cases} 6.8 \text{ in} & 7'' \text{ for soil (a)} \\ 27/2 \text{ to } 0 \text{ or} & 7'' \quad \text{(b)} \\ 27 \text{ to } 6.8 \text{ or} & 17'' \quad \text{(c)} \end{cases}$$

Velocity rise time using average acceleration

$$(t_r)_6 = \frac{(v_v)_6}{(a_v)_6} \frac{1}{386} = \begin{cases} 1.8 \text{ millisec} & \text{for soil (a)} \\ 1.2 \quad '' & \text{(b)} \\ 1.8 \quad '' & \text{(c)} \end{cases}$$

7.3.3 Result

Table 7.1 gives the above calculated intensities and time characteristics for all six structures and the three soil profiles. Figure 7.6 shows the critical parameters on the acceleration, velocity and displacement time-history curves.

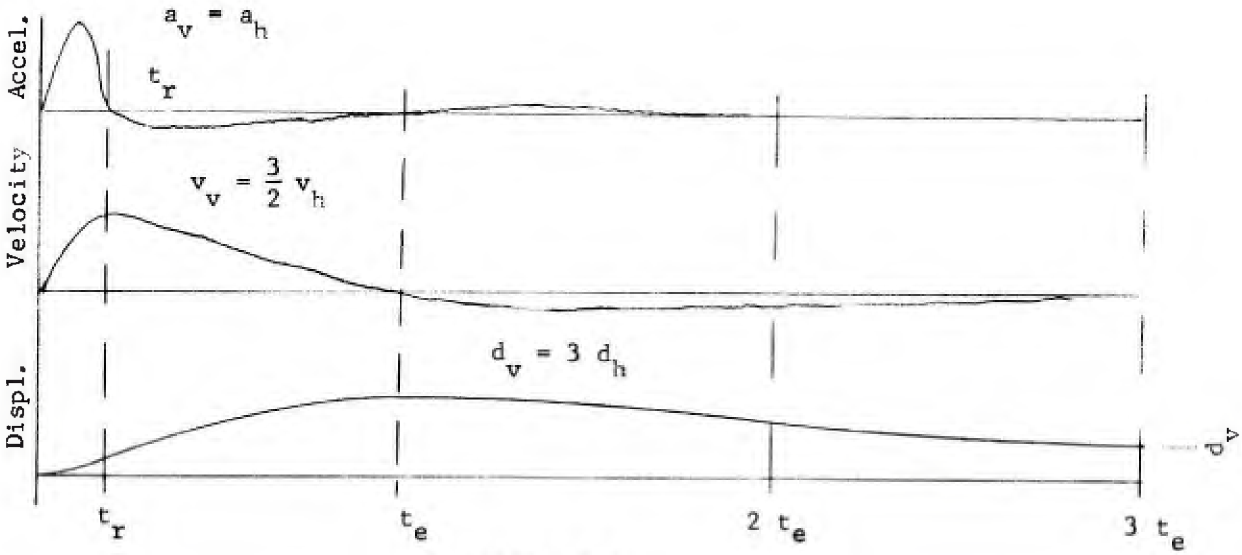


FIGURE 7.6

7.4 BLAST FRONT AND SEISMIC TRAVEL TIME-DISTANCE RELATIONSHIPS IN SOILS (a), (b) and (c)

Blast front distances from r_c are given by

$$r_p \approx 6\, W^{\frac{1}{6}}\, t_a^{\frac{1}{2}} = \begin{Bmatrix} 6 \\ 8.8 \\ 9.9 \end{Bmatrix} t_a^{\frac{1}{2}} \quad \text{kilofeet for} \begin{cases} 1 \ \text{MT yield} \\ 10 \quad '' \\ 20 \quad '' \end{cases}$$

The travel time-distance curves for the blast fronts are shown in Figure 7.7.

TABLE 7.1

Structure		1	2	3	4	5	6
P_{so}		15	50	100	200	500	500 psi
	(a)	15g	23g	62g	24g	50g	190g *
	(b)	23g	31g	86g	26g	63g	270g *
a	(c)	23g	31g	86g	27g	65g	270g *
	(a)	5	17	30	55	110	132 in/sec
	(b)	10	19	60	86	50	127 "
v	(c)	10	26	60	97	180	187 "
	1 MT	2	4	5	7	12	12 inches
	10	4	8	11	16	25	25 "
$d_{(a)}$	20	6	10	14	20	31	31 "
	1 MT	4	4	7	8	6	7 "
	10	9	9	15	17	15	15 "
$d_{(b)}$	20	11	14	20	23	17	18 "
	1 MT	4	6	8	11	15	17 "
	10	9	12	18	23	34	33 "
$d_{(c)}$	20	11	15	23	29	42	50 "
	(a)	0	0.2	0.9	2.4	7	7 "
	(b)	0	0.6	2.5	4.6	8	7 "
d_{vs}	(c)	0	0.6	2.6	5.8	17	17 "
	(a)	0.9	1.9	1.3	6.0	5.7	1.3 millisec *
	(b)	1.2	2.8	1.8	8.6	20	2.8 " *
t_r	(c)	1.2	2.8	1.8	9.3	7.0	2.5 " *
	(a)	800	420	280	230	200	150 "
	(b)	1700	950	600	370	450	300 "
t_e	(c)	2200	1270	700	600	550	400 "
	1 MT	2.3	1.3	1.0	0.75	0.55	0.55 miles
	10	4.9	2.9	2.2	1.6	1.2	1.2 "
$r_p + r_c$	20	6.4	3.7	2.7	2.1	1.5	1.5 "
	1 MT	3400	1000	510	260	100	100 millisec
	10	7300	2200	1100	560	210	210 "
t_a	20	9200	2700	1400	700	270	270 "

* These values obtained by simply averaging accelerations at the 5, 50, and 100 ft levels Smaller a values and larger t_r values result by indirect method of averaging t_r at the three levels.

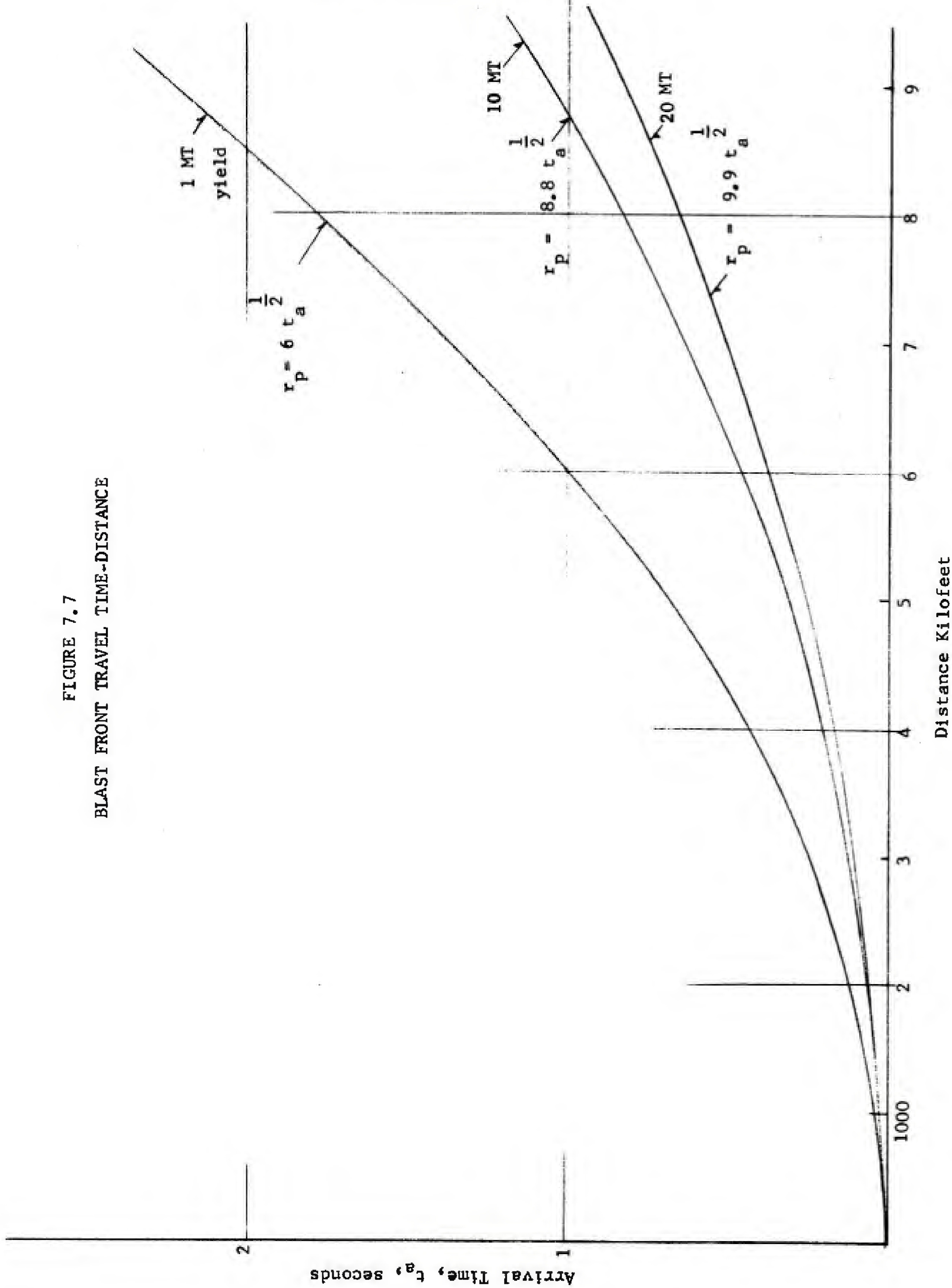


FIGURE 7.7

BLAST FRONT TRAVEL TIME-DISTANCE

Seismic fronts

Soil (a); $c_{eq} = 3$ kilofeet/sec

Since the soil is uniform, outrunning will begin when the blast front velocity

$$U = \frac{dr_p}{dt} \approx c_{eq} \approx 3\, W^{\frac{1}{6}}\, t_a^{-\frac{1}{2}}$$

$$U = \begin{Bmatrix} 3.0 \\ 4.4 \\ 5 \end{Bmatrix} t^{-\frac{1}{2}} \approx 3, \text{ or } t_o = \begin{cases} 1.00 \text{ sec} \\ 2.15 \\ 2.72 \end{cases} \text{ for } \begin{cases} 1 \text{ MT} \\ 10 \text{ MT} \\ 20 \end{cases}$$

$$r_p = \begin{cases} 6.0 \text{ kilofeet} \\ 13.0 \quad '' \\ 16.3 \quad '' \end{cases}, \text{ and } r_p + r_c = \begin{cases} 7.0 \text{ kilofeet} \\ 15.2 \\ 19.0 \end{cases}$$

$$p_{so} = 51\, W^{\frac{1}{3}}/t_o = \begin{Bmatrix} 51 \\ 110 \\ 140 \end{Bmatrix} t_o^{-1} \approx 51 \text{ psi for all yields}$$

Soil (b); two layered soil

$$c_{eq} = \begin{cases} c_1 = 1.5 \text{ kilofeet for } 0 < z < 40' & h_1 = 40' \\ c_2 = 15.0 \quad '' \quad \text{for } z > 40' & h_2 = \infty' \end{cases}$$

The critical time T_1 at which transmission to the surface will occur by way of the second or lower layer is

$$T_1 = 2h \frac{\sqrt{c_2^{\,2} - c_1^{\,2}}}{c_1\, c_2} = 0.053 \text{ sec}$$

The critical distance r_1 at which transmission to the surface will occur by way of the second layer is

$$r_1 = \frac{T_1}{\frac{1}{c_1} - \frac{1}{c_2}} = 88 \text{ ft.}$$

Accordingly the time travel-distance curve for soil (b) is shown by the two straight lines marked (b) in Figure 7.8.

The minimum distances at which seismic outrunning from the blast front will occur are determined by superposing the curve for the (b) soil, Figure 7.8 on each of the three blast front curves of Figure 7.7. Translations of the origin of the (b) curve along the blast front curves will give the following minimum outrunning distances

TABLE FOR SOIL (b)

	1 MT	10 MT	20 MT
r_p, kilofeet	2.7	4.7	5.6
$r_p + r_c$, kilofeet	3.7	6.9	8.3
P_{so}, psi	254	389	435
t_o, sec	0.1	0.28	0.39

Soil (c), three layered

$$c_{eq} = \begin{cases} c_1 = 1.5 \text{ kilofeet for } 0 < z < 40' \;;\; h_1 = 40' \\ c_2 = 3.0 \quad \text{"} \quad 40 < z < 240' \;;\; h_2 = 200' \\ c_3 = 15.0 \quad \text{"} \quad z > 240' \;;\; h_3 = \infty' \end{cases}$$

$$T_1 = 2h_1 \frac{\sqrt{c_2^2 - c_1^2}}{c_1 c_2} = 0.0462 \text{ sec} \;; \qquad r_1 = \frac{R_1}{\frac{1}{c_1} - \frac{1}{c_2}} = 139 \text{ ft.}$$

$$T_2 = 2h_1 \frac{\sqrt{c_3^2 - c_1^2}}{c_3 c_1} + 2h_2 \frac{\sqrt{c_3^2 - c_2^2}}{c_3 c_2} = 0.182 \text{ sec}; \quad r_2 = \frac{T_2}{\frac{1}{c_2} - \frac{1}{c_3}} = 511 \text{ ft.}$$

Accordingly the time travel-distance curve for soil (c) is shown by the three straight lines marked (c) in Figure 7.8, and superposition of this curve with Figure 7.7 gives

FIGURE 7.8

SEISMIC TRAVEL TIME-DISTANCE

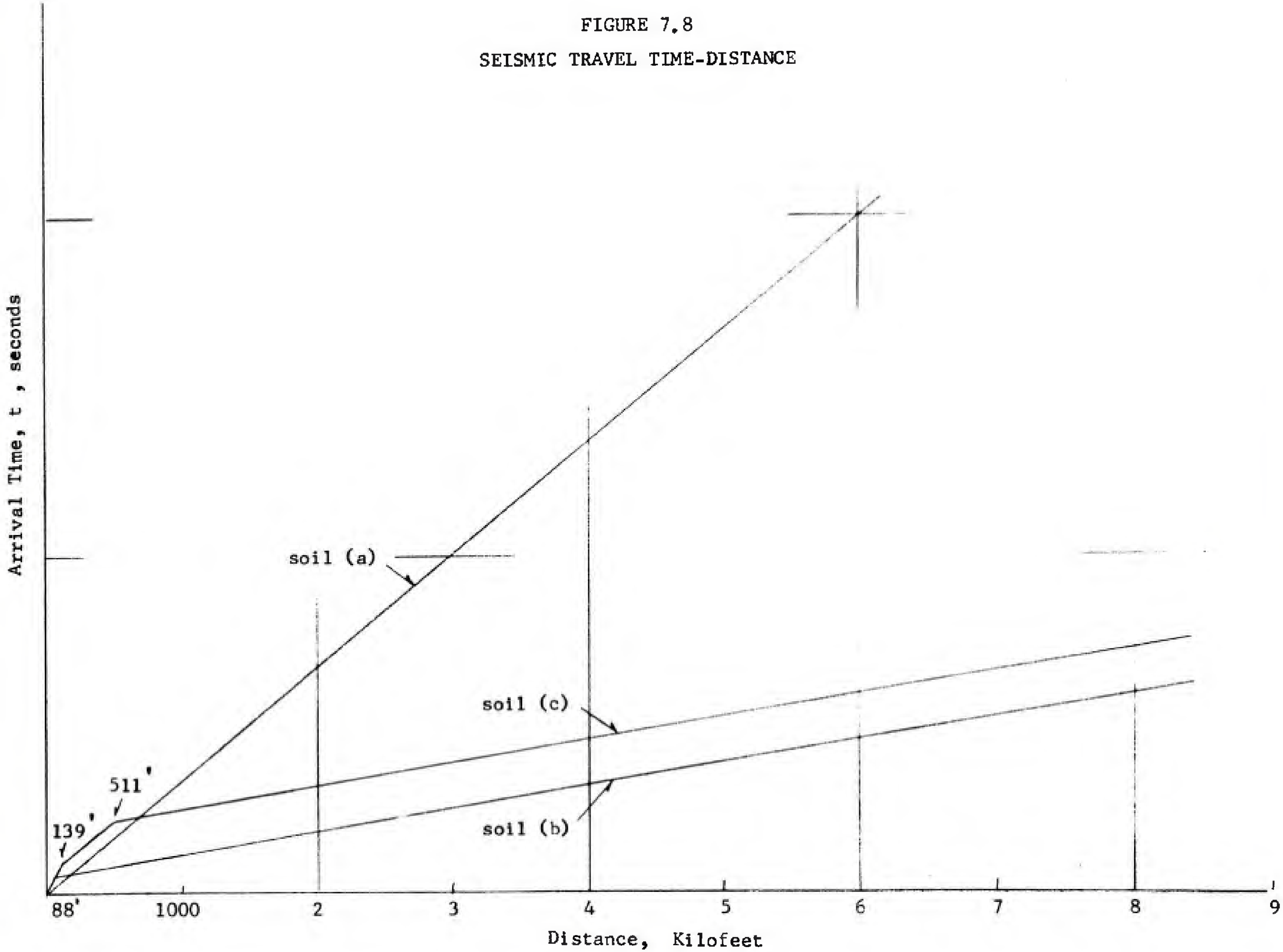


TABLE FOR SOIL (c)

	1 MT	10 MT	20 MT
r_p kilofeet	3.9	6.5	7.6
$r_p + r_c$, "	4.9	8.65	10.3
p_{so}, psi	122	204	235
t_o, sec	0.15	0.43	0.58

Phasing Times at Facility Located at $r_p + r_c$ when Outrunning Occurs at r_o

$$\text{Outrunning distance} = r_p + r_c - r_o = r_f - r_o$$

$$\text{Phasing time } t_\emptyset = t_f - t_o - \frac{r_f - r_o}{c}$$

$$c \quad \text{is seismic velocity} = \begin{cases} c_1 \\ c_2 \\ c_3 \end{cases}$$

in whichever layer is responsible for the outrunning phenomenon. Thus, for soil (a) c = 3 kilofeet/sec, for soils (b) and (c) it is 15 kilofeet/sec.

The equivalent but fictitious "overpressure" p_{eq} associated with the arrival of the outrunning motion at the structure's location r_f is assumed to be related to the overpressure p_o at the location r_o where outrunning is initiated. Thus

$$p_{eq} \approx p_o \left[\frac{r_o}{r_p + r_c}\right]^n \quad , \text{ let } n = 2$$

$$\approx p_o \left[\frac{r_o}{r_p + r_c}\right]^2$$

The following table shows the phasing times $t_\emptyset$ and equivalent, fictitious overpressures p_{eq} at the six structures.

TABLE 7.2

Structure number		1	2	3	4	5,6
Overpressure, p_{so}, psi		15	50	100	200	500
Equiv. press, p_{eq}, psi		12	37	65	115	215
		Phasing time t_ϕ in seconds				
Soil (a)	1 MT	0.7	0	-2.1	-1.7	-2.3
	10	1.4	-0.1	-2.8	-3.6	-5.0
	20	1.5	-0.1	-2.8	-5.0	-7.9
Soil (b)	1	2.7	0.7	0.3	0.2	0
	10	5.7	1.1	0.5	0.2	-0.1
	20	7.1	1.6	0.6	0.1	-0.2
Soil (c)	1	2.7	0.8	0.3	-1.7	-0.2
	10	5.7	1.4	0.3	0	-0.4
	20	7.0	1.5	0.4	-1.7	-0.5

The negative phasing times signify that the seismic motion arrives at the facility after the blast front.

8.0 AIR BLAST MOTIONS

Aboveground structures as well as above grade portions of partially buried structures are exposed to the full brunt of the nuclear air blast impact. If the structure is aboveground or only slightly buried, the air blast may move the structure horizontally and at the same time press the structure into the soil. This type of motion is referred to as the air blast induced rigid body response of the structure. In addition, bending and shearing of the exposed portion of the structure can be expected as the result of high surface wind velocities. Finally, local deformations of the exposed portions of the structure must be anticipated due to the overpressure in the blast front.

8.1 RECTANGULAR ABOVEGROUND STRUCTURE (SEE FIGURE 6.1)

The air blast induced motions of the aboveground rectangular structure whose foundation is at or near grade level is composed primarily of two components: rigid body motion and local deformation.

1) Rigid Body Motions Due to Air Blast on the Exposed Structure

a) Horizontal

When the steep positive pressure gradient of the air blast front, moving at high velocity over the surface of the ground, encounters the aboveground structure it exerts an initial positive force on the structure equal to the peak reflected pressure times the area of the structure facing the blast front. If the pressure is large, the structure may begin to slide along the surface with an acceleration dependent on the mass of the structure, the magnitude of the pressure, and the resisting force between the structure and ground.

As the pressure front envelops the structure, the structure is pressed vertically into the soil thus increasing its frictional resistance to sliding and almost simultaneously, a positive pressure will act on the back face of the structure, so that the net forward pressure acting on the structure will be sharply reduced. Consequently, the acceleration is rapidly decreased.

The character of the pressure gradient is estimated as shown in Figure 8.1. The peak acceleration is the initial force divided by the structure mass.

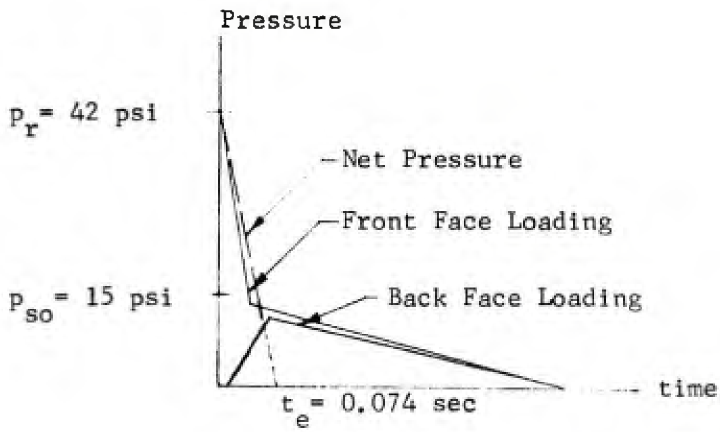


FIGURE 8.1

By analytical or graphical means the velocity-time and displacement-time responses can also be found. Generalized analyses of rigid body motion are given in Reference 8.1.

b) <u>Vertical</u>

The vertical rigid body motion due to the slap of the air blast on the structure will depend on the soil stiffness. As a first approximation, the structure may be considered as a rigid mass resting on an elastic foundation. Analytical and graphical means are available (Reference 8.1) for determining the acceleration, velocity and displacement response of the structure when the pressure-time history of the air blast is known or can be estimated. However, when the pressure duration is long in comparison with the natural period of the elastically supported structure the peak acceleration can be obtained conservatively by dividing the total vertical overpressure force by the mass of the structure. Peak velocity and displacement are very roughly related to the acceleration by the natural circular frequency ω, of the structure i.e.

$$\text{velocity} \approx \frac{\text{acceleration}}{\omega}$$

$$\text{displacement} \approx \frac{\text{acceleration}}{\omega^2}$$

The computed air blast rigid body motions are tabulated in Table 8.1 for the aboveground structure. Detailed analyses are included in the appendix.

2) Local Deformation Due to Impact of the Air Blast

a) Walls

The direct impingement of the air blast on the structure will induce bending of the wall elements. Assuming that the rise time to the peak pressure is essentially instantaneous, then the peak acceleration is simply the total horizontal reflected pressure force divided by an equivalent mass of the wall element.

When the deformation of the wall remains in the elastic region, the velocity and displacement are related to the acceleration by the natural circular frequency of the fundamental mode of response of the wall element (as shown under 1). If yielding occurs, the ductility factor μ can be determined from dynamic response charts (Reference 8.1) for which case the maximum displacement is then the product of the ductility factor and the yield deflection of the wall element.

b) Roof and floor

The analysis of the roof element is carried out in the manner described for the wall elements except that the appropriate applied pressure is the overpressure instead of the reflected pressure. In this structure the floor or bottom slab is monolithic with the walls. Therefore the floor response is identical to the roof response which is the customary assumption. The secondary deformational response of the aboveground structure is summarized in Table 8.1. Detailed analyses are included in the appendix.

8.2 PARTIALLY BURIED STRUCTURE (SEE FIGURE 6.2)

With 40 percent of the structure below grade, it can be assumed that the direct air blast will not result in appreciable horizontal rigid body motion. Therefore, the air blast motion of the structure consists almost entirely of bending and shearing due to the horizontal drag force plus local deformations due to the overpressure. (The vertical structure motion is assumed to be the same as the free-field ground motion.)

1) Bending and Shearing

The high wind velocities associated with the air blast will result in a dynamic drag that will cause bending and shear deformations of the structure in the exposed region. The peak effective force acting on the structure is dependent on the peak dynamic pressure and the drag coefficient for the structural shape (cylindrical). If this peak force is considered to have an instantaneous rise time then the method of analysis discussed for the local deformation of the aboveground rectangular structure is applicable.

The peak acceleration will occur at the top of the structure with a magnitude equal to peak drag force divided by an equivalent mass of the structure. The peak velocity and displacement response is related to the peak acceleration through the circular frequency of the fundamental bending and shear mode of the structure.

It can be seen that primary deformation plays a major role when personnel or equipment are located in the upper portions of the structure. In regions near and below grade, there is little effect from this response. Table 8.2 summarizes the primary deformation for a "high wall" position. See the appendix for detailed analyses.

2) Local Deformation

As the blast front envelops the structure it is compressed due to the uniform overpressure. Since the natural periods of the compression or ringing modes of vibration are small in comparison with the pressure duration, the structure responds with a radial vibration of the cylindrical walls. The peak acceleration, velocity and displacement are determined by computing the equivalent mass of the structure, the natural frequency of vibration, and the yield deflection.

Local deformations occur over most of the exposed portion of the cylindrical structure except near the top where the end cap inhibits large radial motion and near grade level where the restraining effects of the soil must be considered. It is customary to ignore end effects in fairly long cylinders and to account for soil effects by modifying the natural period of vibration due to the increased mass and stiffness afforded by the soil. Local deformations for the aboveground portion of the structure, referred to as a "med. wall" location, are tabulated in Table 8.2. Detailed analyses appear in the appendix.

AIR BLAST MOTIONS

TABLE 8.1 RECTANGULAR ABOVEGROUND STRUCTURE

SHOCK	ELEMENT	RIGID BODY			BENDING OR SHEARING			LOCAL DEFORMATION		
		a_1 g	v_1 in/sec	d_1 in.	a_2 g	v_2 in/sec	d_2 in.	a_3 g	v_3 in/sec	d_3 in.
VERTICAL	WALL	1.94	10.6	0.15	-	-	-	-	-	-
	FLOOR	1.94	10.6	0.15	-	-	-	10.5	23.9	0.14
HORIZONTAL	WALL	4.53	39.4	3.22	-	-	-	29.6	67.2	1.11
	FLOOR	4.53	39.4	3.22	-	-	-	-	-	-

TABLE 8.2 PARTIALLY BURIED STRUCTURE

SHOCK	ELEMENT	a_1 g	v_1 in/sec	d_1 in.	a_2 g	v_2 in/sec	d_2 in.	a_3 g	v_3 in/sec	d_3 in.
VERTICAL	WALL	-	-	-	-	-	-	-	-	-
	FLOOR	-	-	-	-	-	-	-	-	-
HORIZONTAL	HIGH WALL	-	-	-	12.6	52.1	0.56	-	-	-
	MED. WALL	-	-	-	-	-	-	32	12.4	-
	LOW WALL	-	-	-	-	-	-	-	-	-
	FLOOR	-	-	-	-	-	-	-	-	-

Note: Dash indicates that the type of motion at the location on the structure was not considered because of insignificant effect.

9.0 SHOCK SPECTRA ANALYSES

The effects of the free-field ground motion and the air blast motion may be combined in such a way that the total response of the structure and equipment attached to the structure may be found. The induced ground and air blast motions must in general be modified to account for properties of the soil, structure, equipment, weapon yield etc.

9.1 IMPORTANT CONSIDERATIONS

The parameters that are most important in determining the total response of the equipment attached to the structure are enumerated below:

(1) depth of burial
(2) the amplitudes and durations of the free-field motions
(3) outrunning or phasing between seismic shocks and blast front down slap
(4) amplitudes of the three air blast induced motions: rigid body, primary deformation and secondary deformation of the structure
(5) relationship between the natural periods of the structure and equipment to the free-field pulse durations
(6) mass ratios between the structure and equipment
(7) relationship between the natural periods of the structure and equipment
(8) damping of the structure, equipment and soil

Considerable interaction exists between many of these effects. For example, the depth of burial (1) influences the natural periods of the structure in (5) and (7). It also determines the effective mass of the structure in (6) and is necessary in finding the free-field motions of (2).

9.2 COMBINATION OF FREE-FIELD GROUND MOTIONS AND AIR BLAST MOTIONS OF THE STRUCTURE

The structure which interfaces with the soil media is assumed to have motions equal to the peak ground motions determined in Section 7 and in the case of the aboveground structure the air blast slap on the structure amplifies the ground induced motion. Since the peak intensities for each effect are calculated separately the resultant effect can only be determined in an approximate manner by either combining the peak intensities as the algebraic sum or taking the square root of the sum of the squares. The square root approximation will in general give acceptable results (usually sufficiently conservative) and therefore this approximation is used in this study.

The ground induced motions in some cases consist of two components, the ground motion due to air slap at the structure (Type I) and the ground motion due to the refracted and reflected waves in the ground due to the moving air blast shock front (Type II). The Type II motion as discussed in Section 7 is considered effective when it arrives nearly coincident with or ahead of the Type I pulse. Based on the arrival time calculation results of Section 7, it is determined that the Type II pulse may occur nearly simultaneously with the Type I pulse for some of the cases considered and shown in Table 2.1, Summary of Results. When unfavorable outrunning or phasing between two pulses occurs (say $t_{\phi} < 0.5$) these motions are modified by the following factors which are recommended in Reference 9.1.

	VERTICAL	HORIZONTAL
a becomes	$1.4\ a_v$	$1.4\ a_v$
v becomes	$1.2\ v_v$	$0.9\ v_v$
d becomes	$1.3\ d_v$	$0.6\ d_v$

If the structure is aboveground or if it is partially buried additional air blast motions must then be considered. It is shown in Section 8 that three air blast motions are possible: rigid body motion, bending and shearing, and local deformation. Denoting these motions by subscripts, 1, 2, and 3, and denoting the ground induced motion by subscript f, all four motions are combined quadratically as follows:

$$a = \left[a_1^{\ 2} + a_2^{\ 2} + a_3^{\ 2} + a_f^{\ 2}\right]^{\frac{1}{2}}$$

This combination is also applied to the velocities and displacements, in the vertical as well as in the horizontal directions. Ground accelerations are

TABLE 9.1 GROUND MOTIONS

STRUCTURE	ACCELERATION						VELOCITY						DISPLACEMENT					
	SYSTEMATIC a_f			OUTRUNNING a_f			SYSTEMATIC v_f			OUTRUNNING v_f			SYSTEMATIC d_f			OUTRUNNING d_f		
SOIL CONDITION	a	b	c	a	b	c	a	b	c	a	b	c	a	b	c	a	b	c
1 VERTICAL	0	0	0	-	-	-	5	10	10	-	-	-	6	11	11	-	-	-
1 HORIZONTAL	0	0	0	-	-	-	3.3	6.6	6.6	-	-	-	2	3.5	3.5	-	-	-
2 VERTICAL	-	31	31	32	-	-	-	19	26	20	-	-	-	14	15	13	-	-
2 HORIZONTAL	-	31	31	32	-	-	-	13	17	15	-	-	-	4.5	5	6	-	-
3 VERTICAL	62	86	-	-	-	120	30	60	-	-	-	72	14	20	-	-	-	30
3 HORIZONTAL	62	86	-	-	-	120	20	40	-	-	-	54	4.7	6.7	-	-	-	13.8
4 VERTICAL	24	-	27	-	36	-	55	-	97	-	100	-	20	-	29	-	30	-
4 HORIZONTAL	24	-	27	-	36	-	37	-	65	-	77	-	6.7	-	9.6	-	14.7	-
5 VERTICAL	50	-	-	-	88	91	110	-	-	-	60	216	31	-	-	-	22	55
5 HORIZONTAL	50	-	-	-	88	91	73	-	-	-	45	162	10.3	-	-	-	10	25.2
6 VERTICAL	190	-	-	-	378	378	132	-	-	-	132	224	31	-	-	-	23	65
6 HORIZONTAL	190	-	-	-	378	378	88	-	-	-	114	168	10.3	-	-	-	10.8	30

often neglected in the aboveground structure. Table 9.1 summarizes the peak intensities of the ground motion to be used in the above equation.

Occasionally, structural resonances* of walls, floors, roofs etc. may fall in the bandwidth $1/2t_d \leq f \leq 2/t_d$ where t_d is the ground pulse duration defined in Section 7, (Figure 7.5). When that occurs, the total input motion is usually modified by multiplying the ground displacements and velocities by the factor 1.5. Thus, if a wall is laterally resonant in the critical bandwidth, the motions will be:

VERTICAL	HORIZONTAL
a_v	a_h
v_v	$\left[v_1^2 + v_2^2 + v_3^2 + 2.25\ v_f^2\right]^{\frac{1}{2}}$
d_v	$\left[d_1^2 + d_2^2 + d_3^2 + 2.25\ d_f^2\right]^{\frac{1}{2}}$

9.3 STRUCTURE INTERACTION

The local response of the structural element which supports the equipment may alter the resultant free-field input to the equipment especially for hard mounted equipment. The flexibility of the structural element supporting the equipment may introduce two significant effects, (1) the input acceleration to the equipment may be attenuated if the structural element frequency is much lower than the equipment frequency or (2) the input acceleration to the equipment may be amplified if the equipment frequency is close to the structural element frequency.

The effect of the structural element response is determined by considering the 2-mass 2 DOF model shown in Figure 9.1. For the given equipments

*Structural resonances of partially and fully buried structures may require modification due to soil effects. See Reference 9.2 for additional information.

Isolated $f_2 = \frac{1}{2\pi}\sqrt{\frac{k_2}{m_2}}$

Isolated $f_1 = \frac{1}{2\pi}\sqrt{\frac{k_1}{m_1}}$

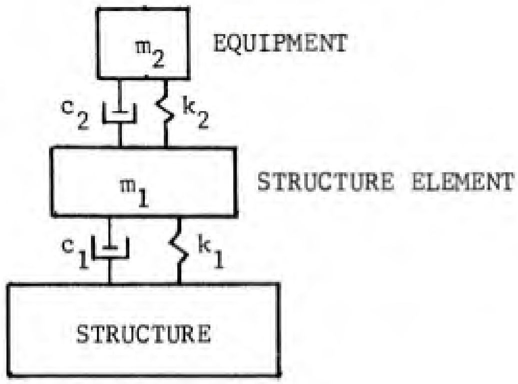


FIGURE 9.1

(see Section 5) the mass m_2 is very small with respect to the structural element m_1 (i.e. $m_2 < m_1/10$). Therefore as the structure moves with the ground the response of m_1 is not significantly influenced by the attachment of mass m_2. The response of the structural element can be calculated as a single DOF system and this response in turn becomes an input to the equipment.

The shock spectra technique has been suggested in Reference 9.1 for the analysis of a two-mass system where m_1 is much larger than m_2. The response of m_1 is bounded by the trapezoidal shock spectra envelope defined by the following factors

$$A_{v,h} = 2\ a_{v,h}$$

$$V_{v,h} = 1.5\ v_{v,h}$$

$$D_{v,h} = d_{v,h}$$

Therefore using the resultant inputs defined in Section 9.2, the shock spectras for the structural elements of interest are shown in Figures 9.2 - 9.14. The first mode frequencies (f_1) are noted on each spectra which defines the

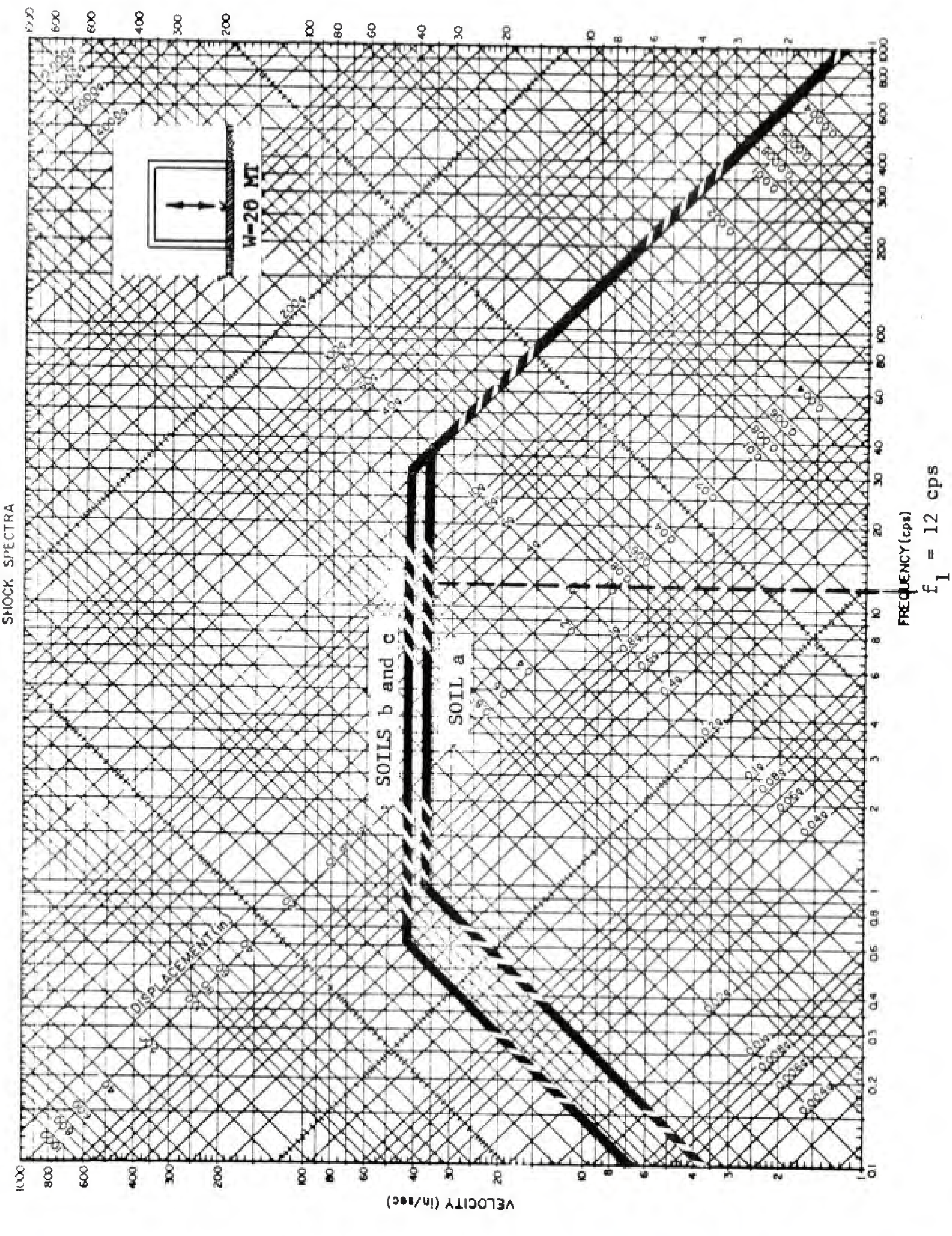

SHOCK SPECTRA
W-20 MT
SOILS b and c
SOIL a
DISPLACEMENT (in)
VELOCITY (in/sec)
FREQUENCY (cps)
f_1 = 12 cps

FIGURE 9.2

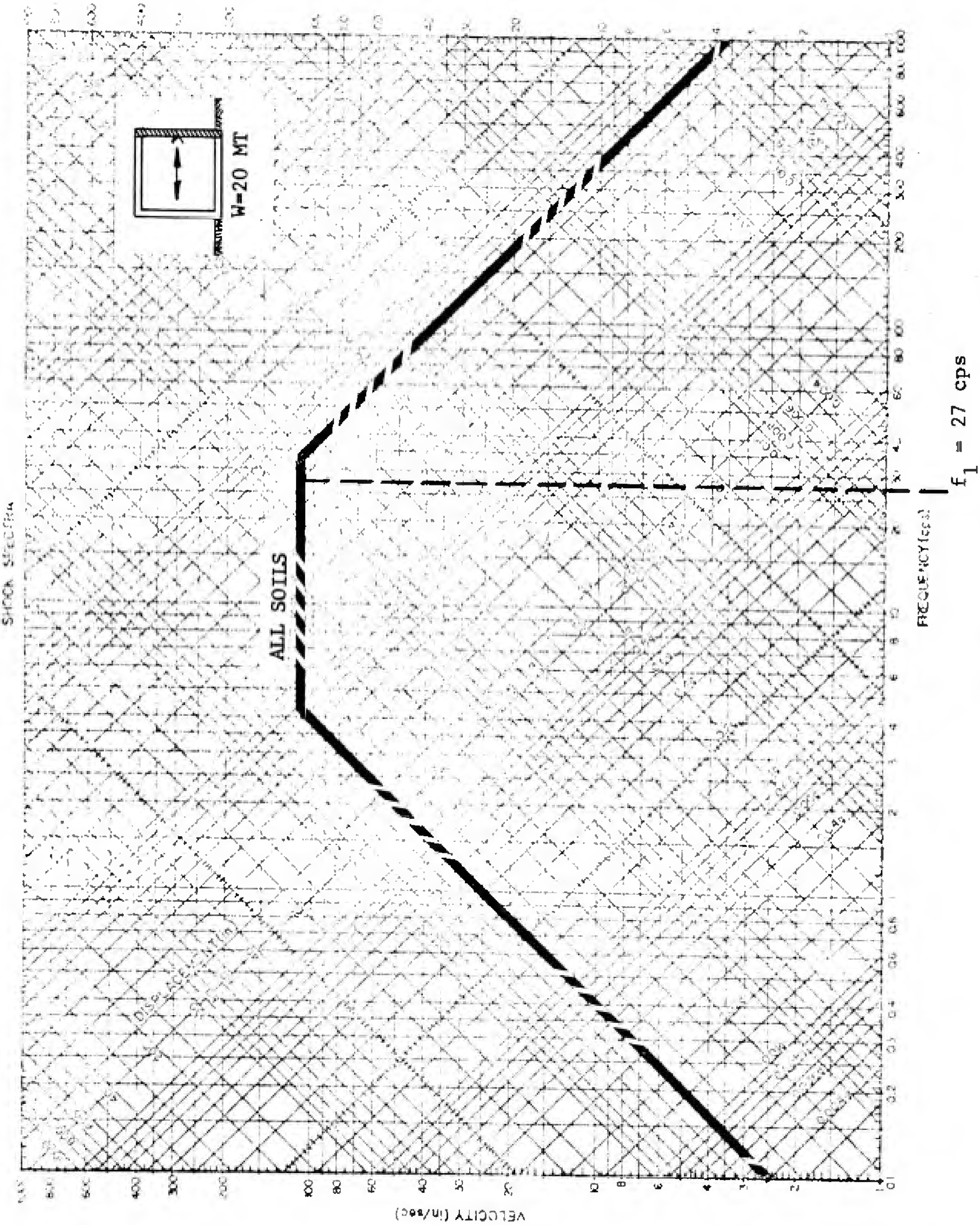

SHOCK SPECTRA
W=20 MT
ALL SOILS
f_1 = 27 cps
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.3

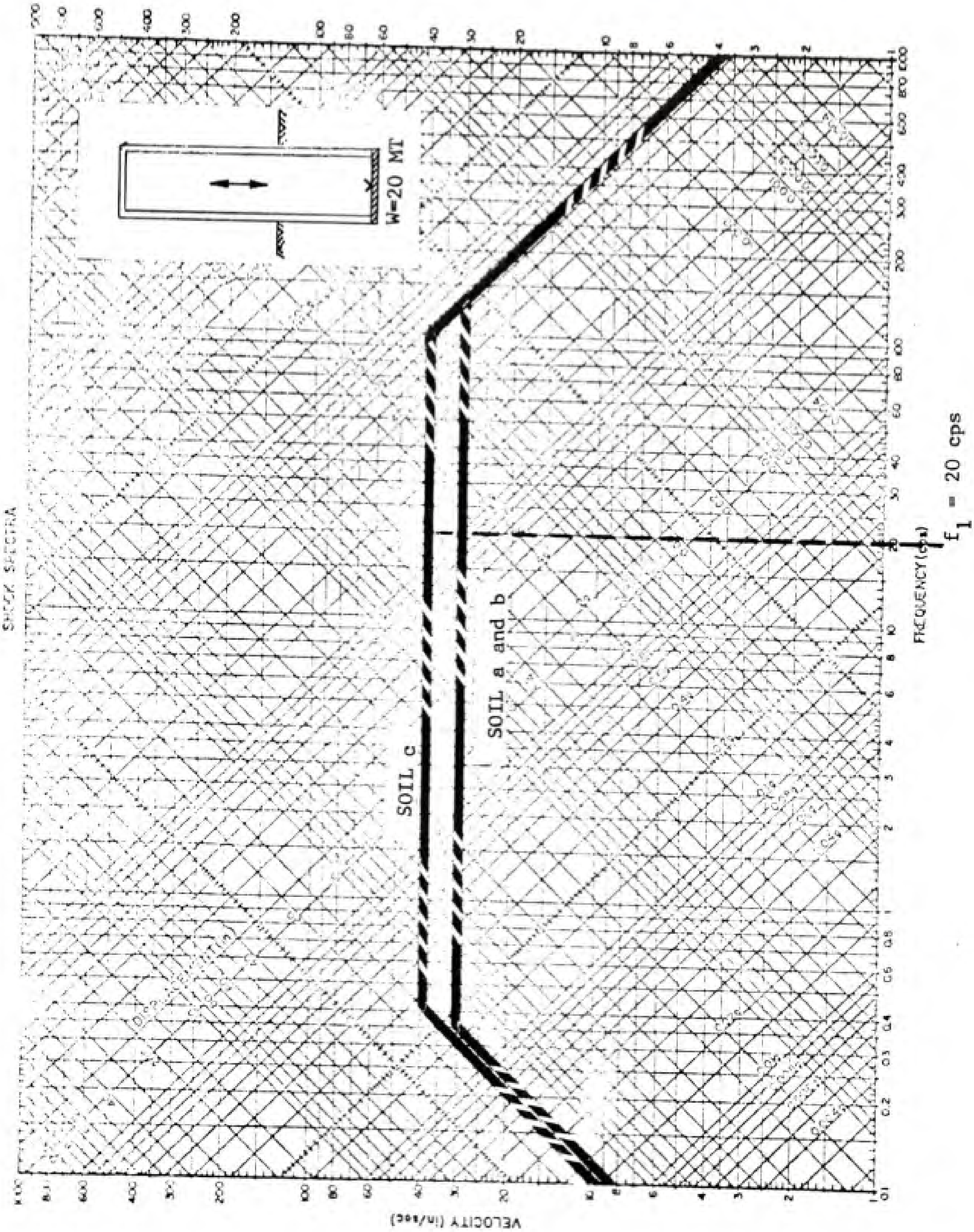
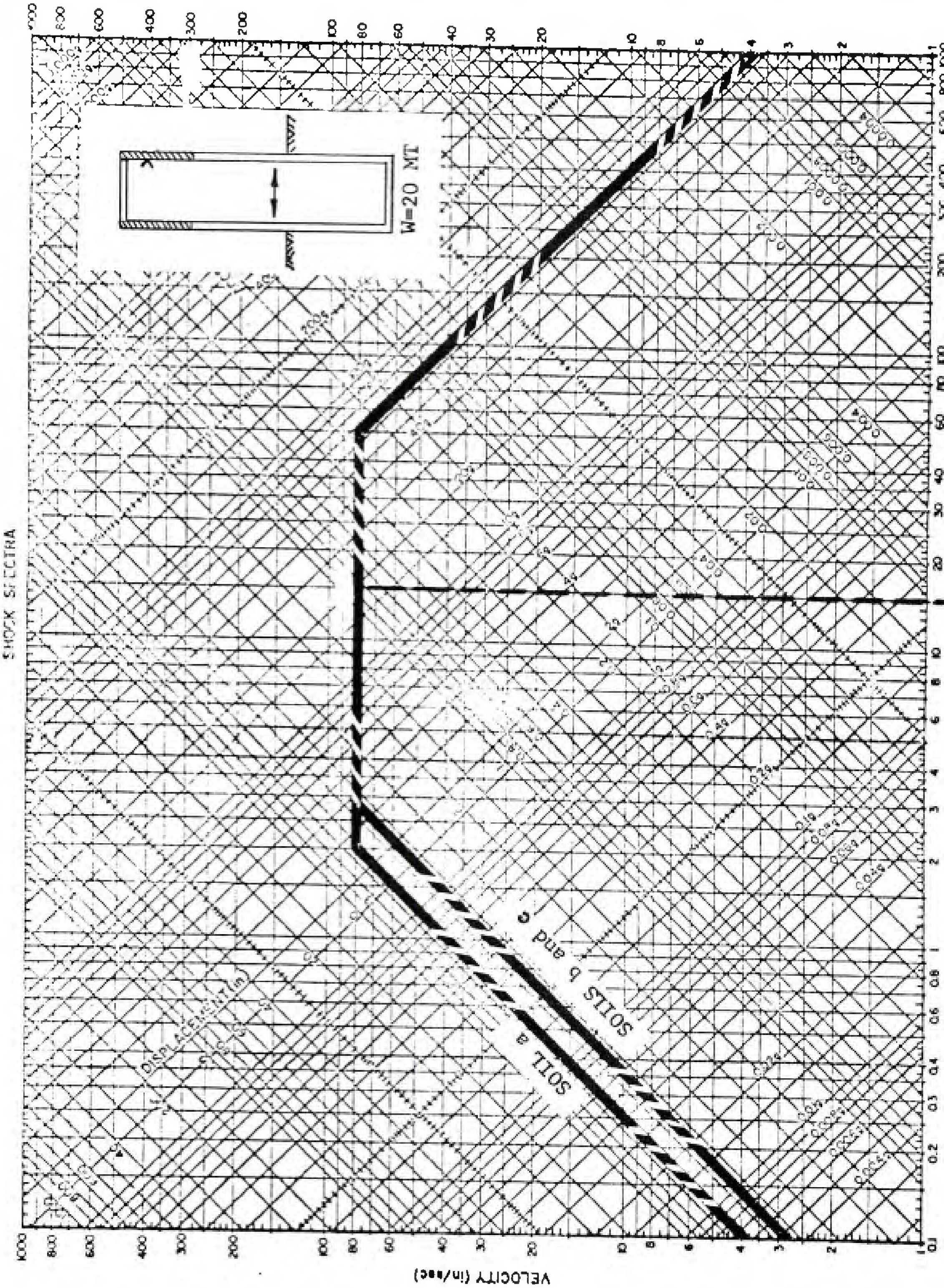

SHOCK SPECTRA
VELOCITY (in/sec)
FREQUENCY (cps)
W=20 MT
SOIL c
SOIL a and b
f_1 = 20 cps

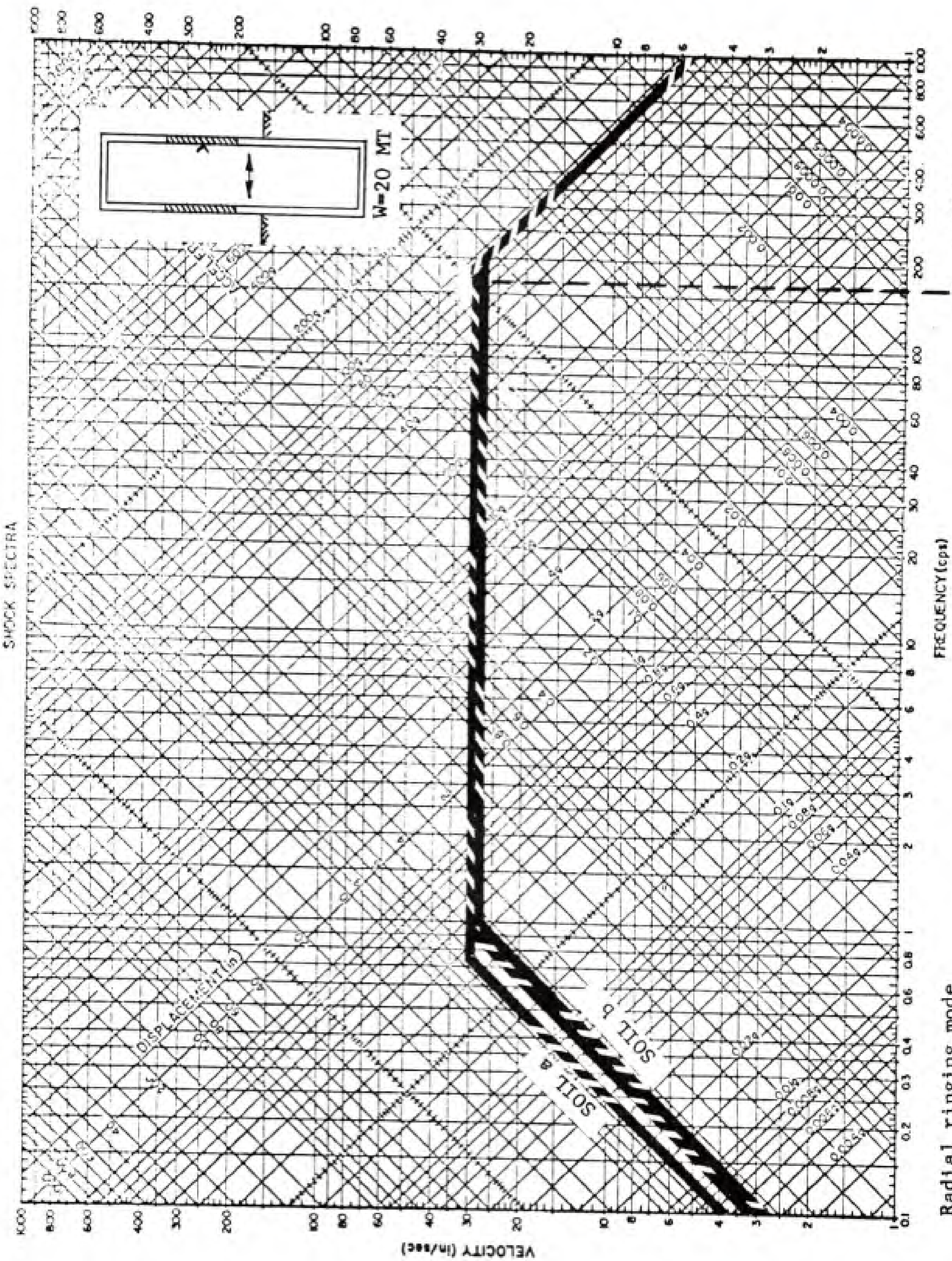
FIGURE 9.4



Cantilever bending mode

f_1 = 15 cps

FIGURE 9.5



$f_1 = 165$ cps

Radial ringing mode

FIGURE 9.6

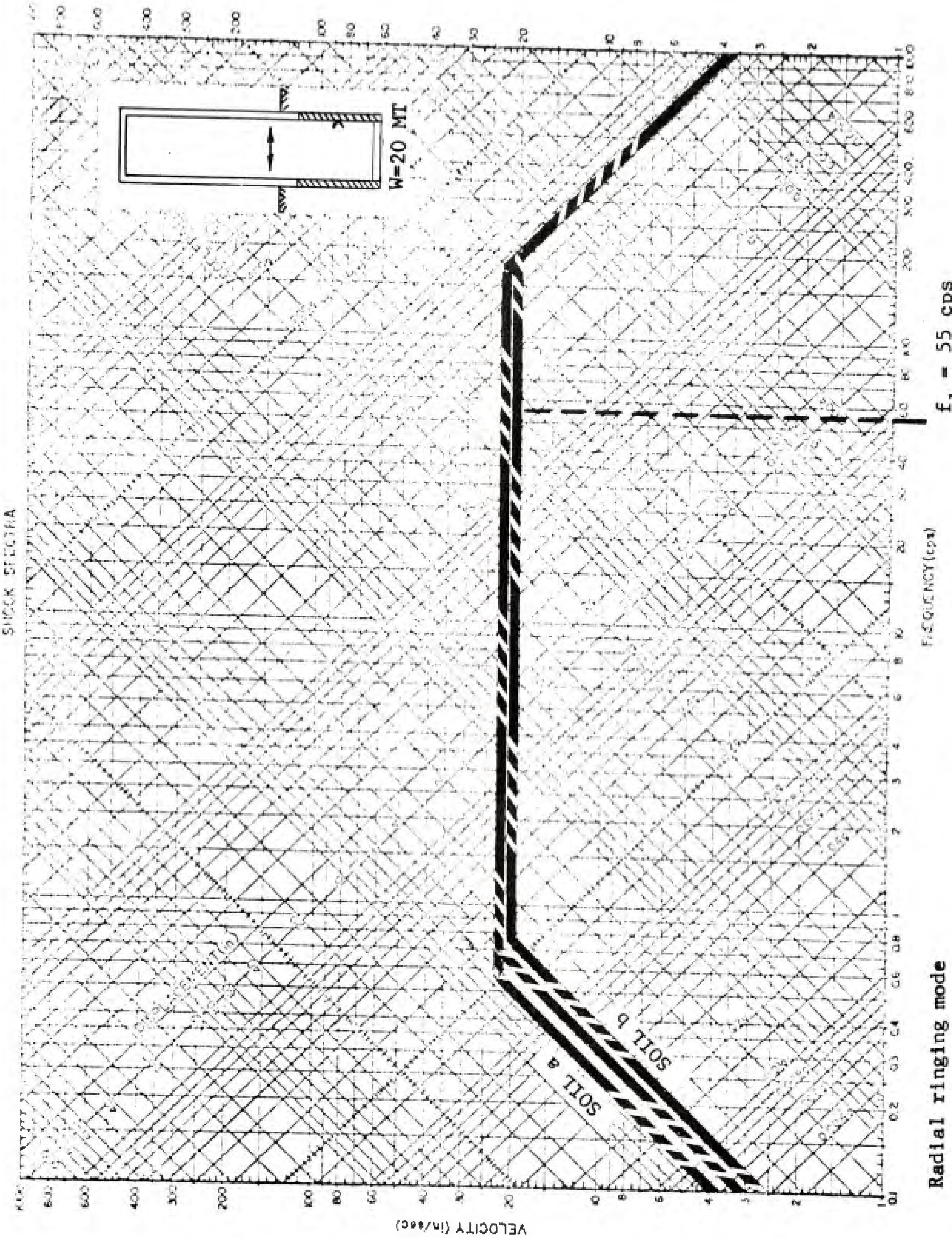

SHOCK SPECTRA
W=20 MT
SOIL a
SOIL b
f_1 = 55 cps
Radial ringing mode
VELOCITY (in/sec)
FREQUENCY (cps)

FIGURE 9.7

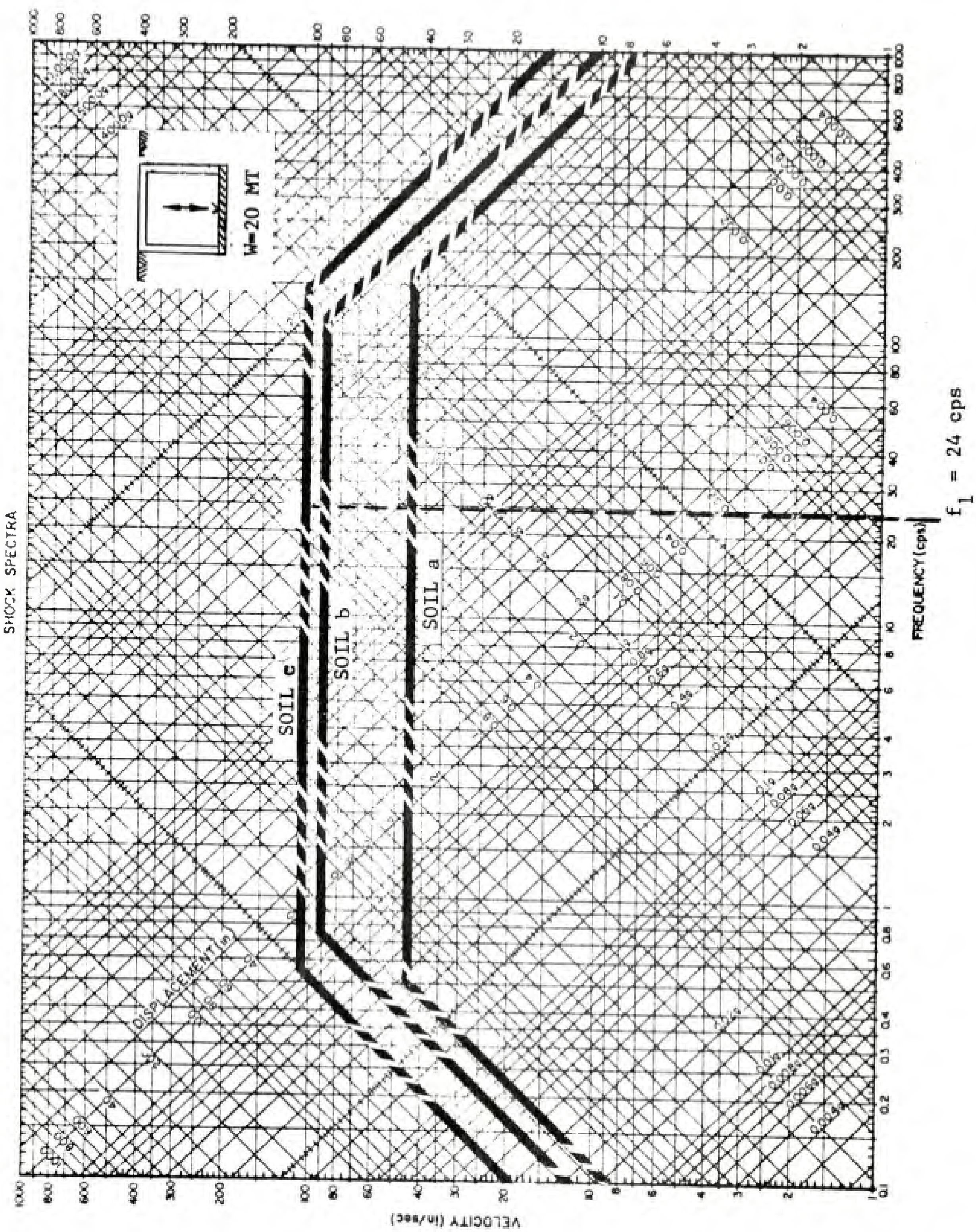

SHOCK SPECTRA
W-20 MT
SOIL c
SOIL b
SOIL a
DISPLACEMENT (in)
VELOCITY (in/sec)
FREQUENCY (cps)
f_1 = 24 cps

FIGURE 9.8

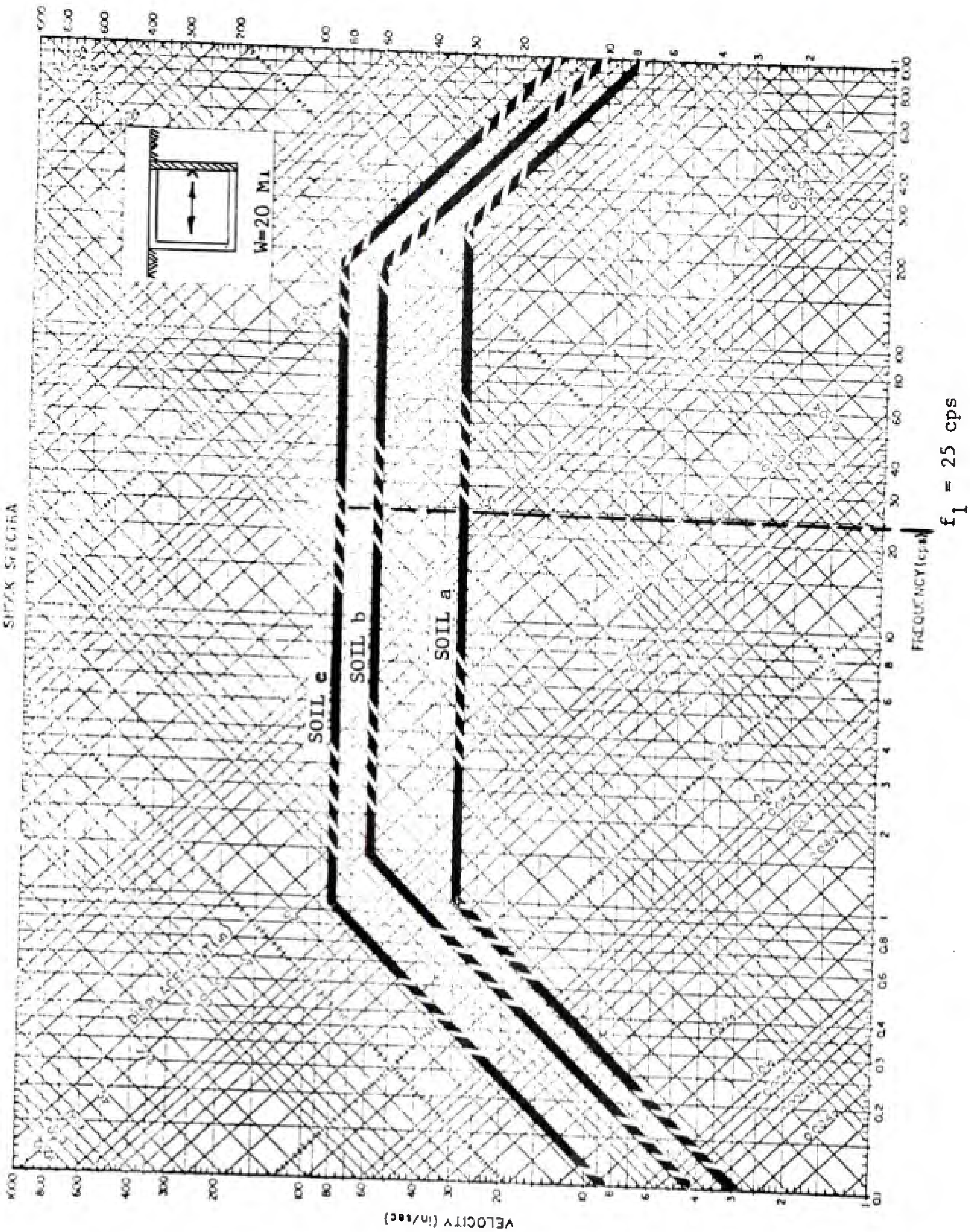

SHOCK SPECTRA
W=20 Mt
SOIL c
SOIL b
SOIL a
f_1 = 25 cps
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.9

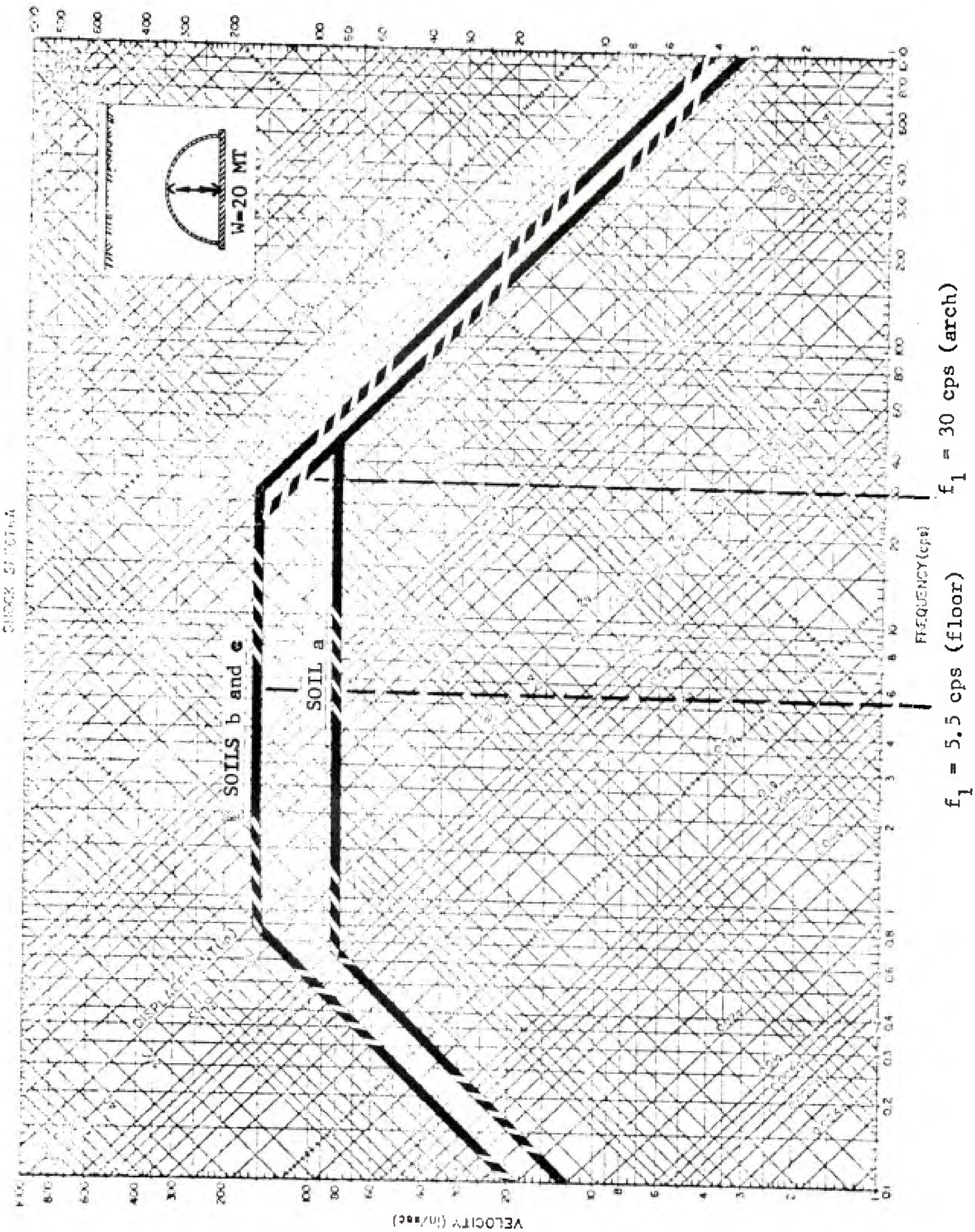
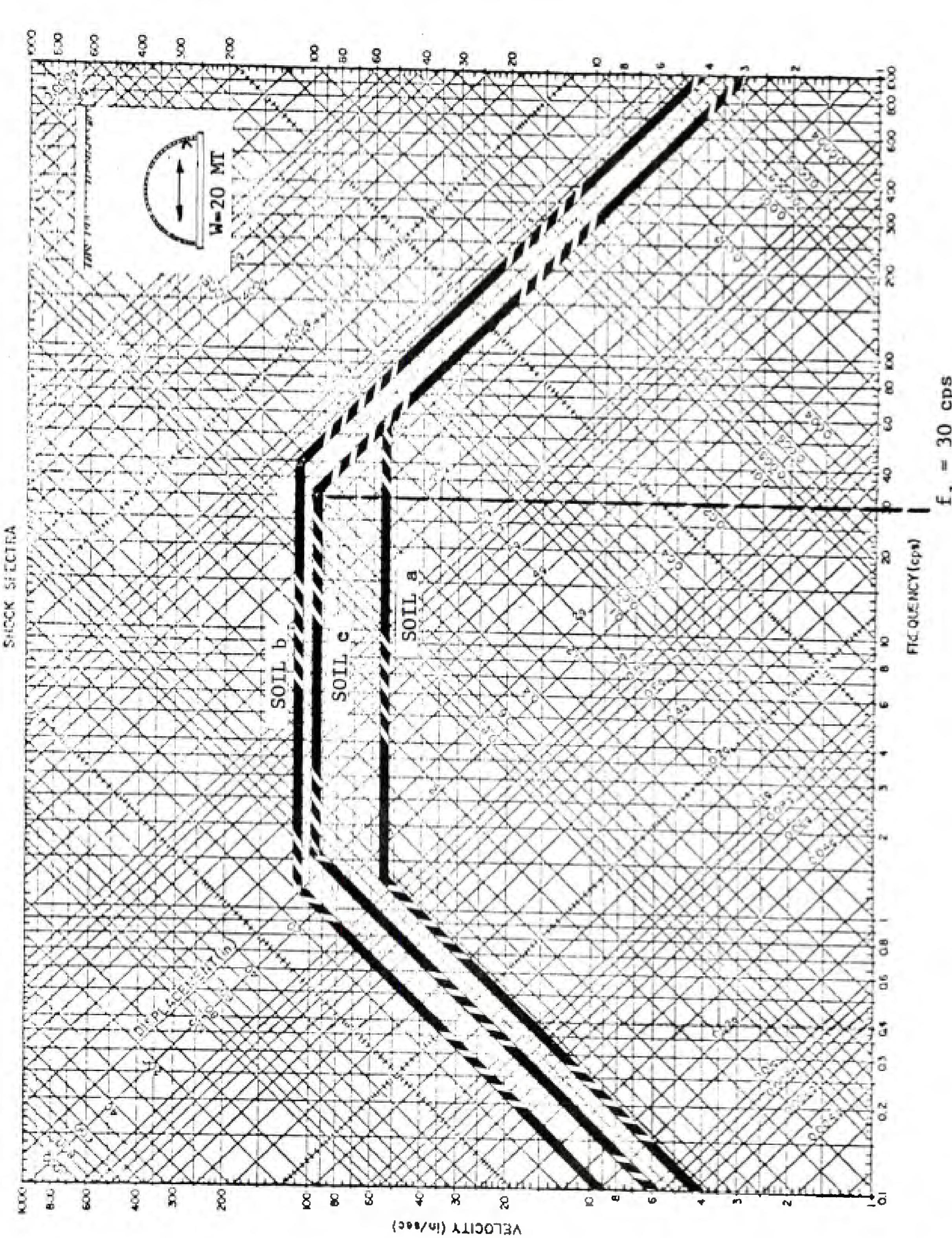

W=20 MT
SOILS b and c
SOIL a
f_1 = 30 cps (arch)
f_1 = 5.5 cps (floor)
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.10



SHOCK SPECTRA
W-20 MT
SOIL b
SOIL c
SOIL a
FREQUENCY (cps)
VELOCITY (in/sec)
f_1 = 30 cps

FIGURE 9.11

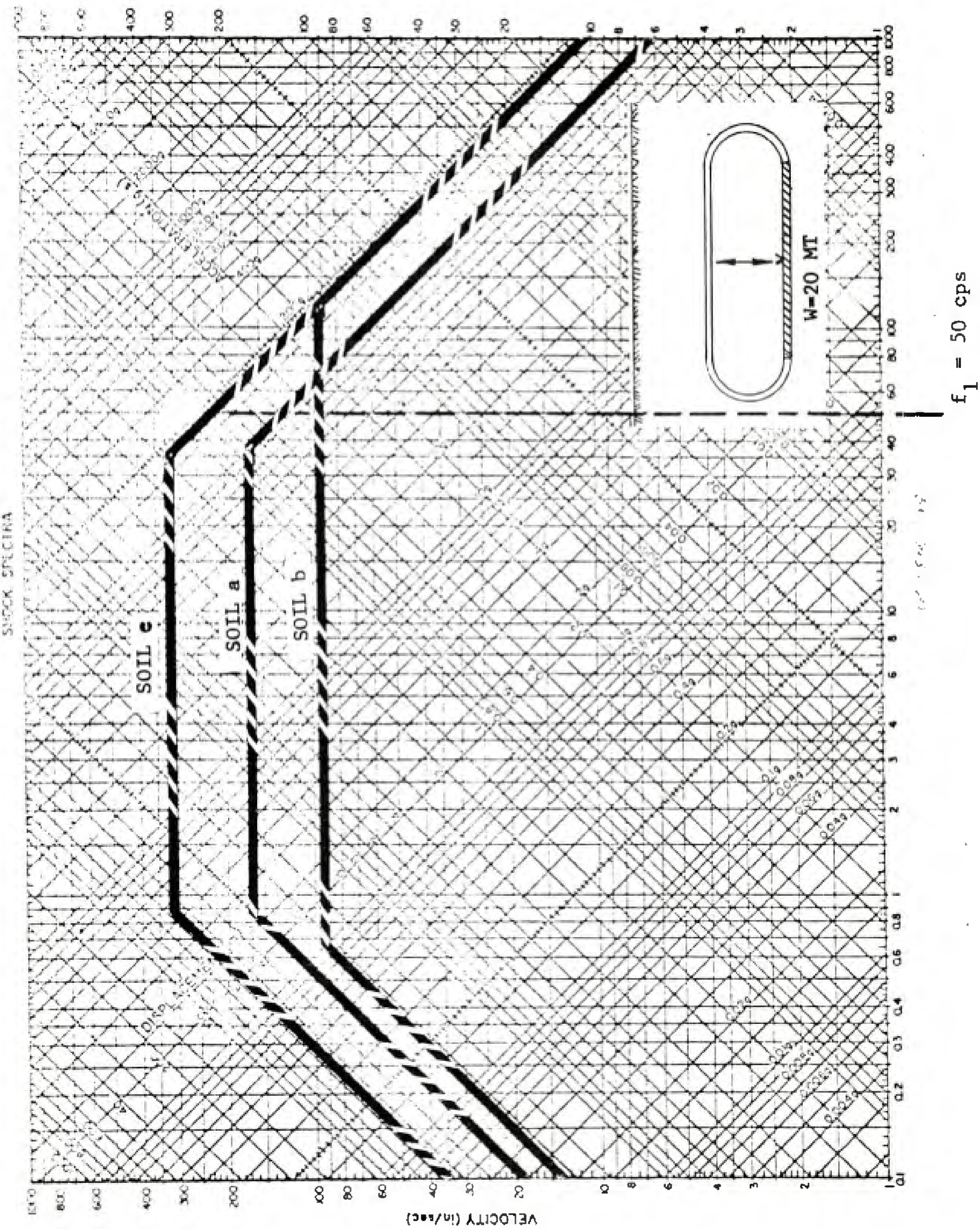

SHOCK SPECTRA
SOIL c
SOIL a
SOIL b
W-20 MT
f_1 = 50 cps
VELOCITY (in/sec)

FIGURE 9.12

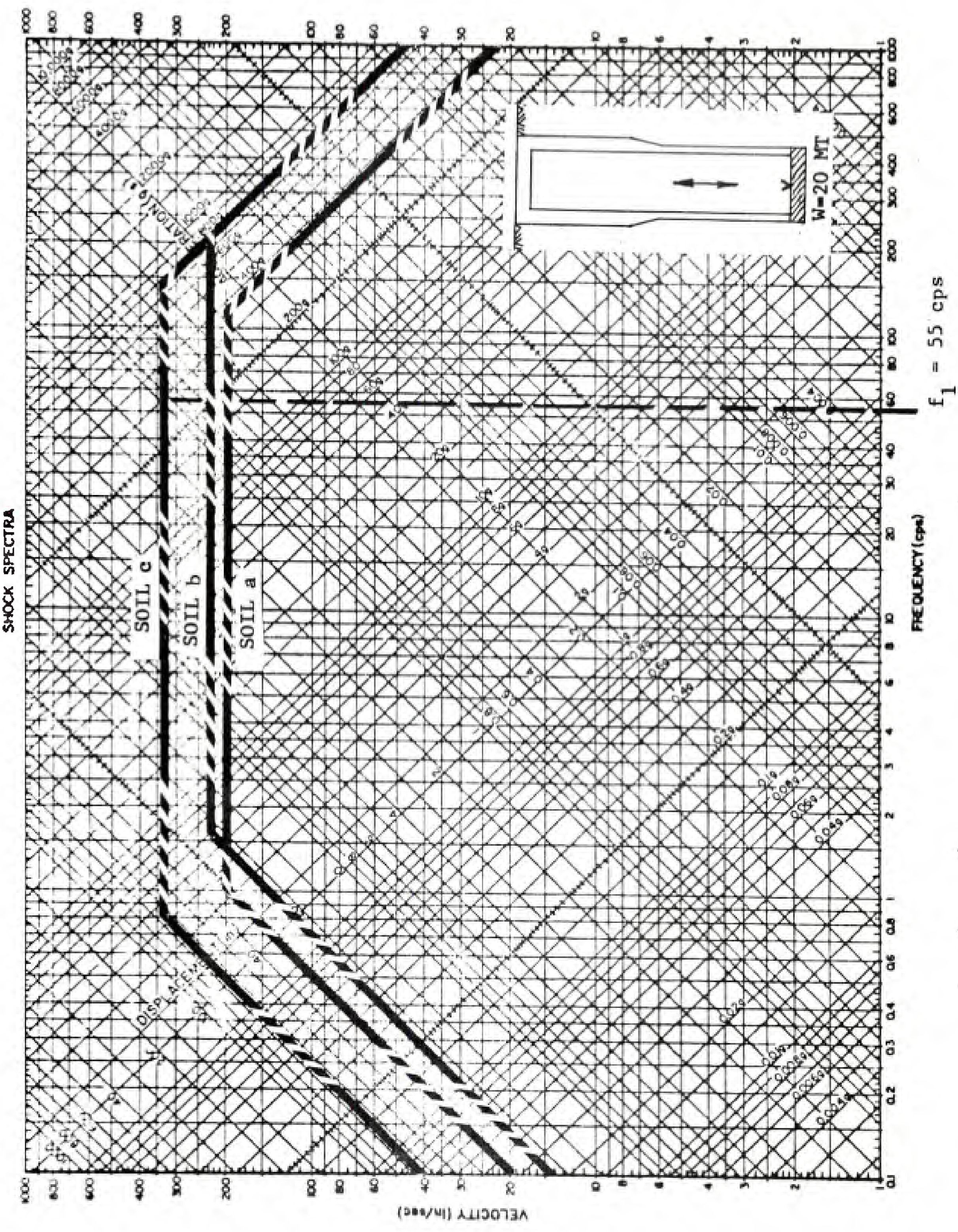

SHOCK SPECTRA
SOIL c
SOIL b
SOIL a
W-20 MT
FREQUENCY (cps)
VELOCITY (in/sec)
$f_1 = 55$ cps

FIGURE 9.13

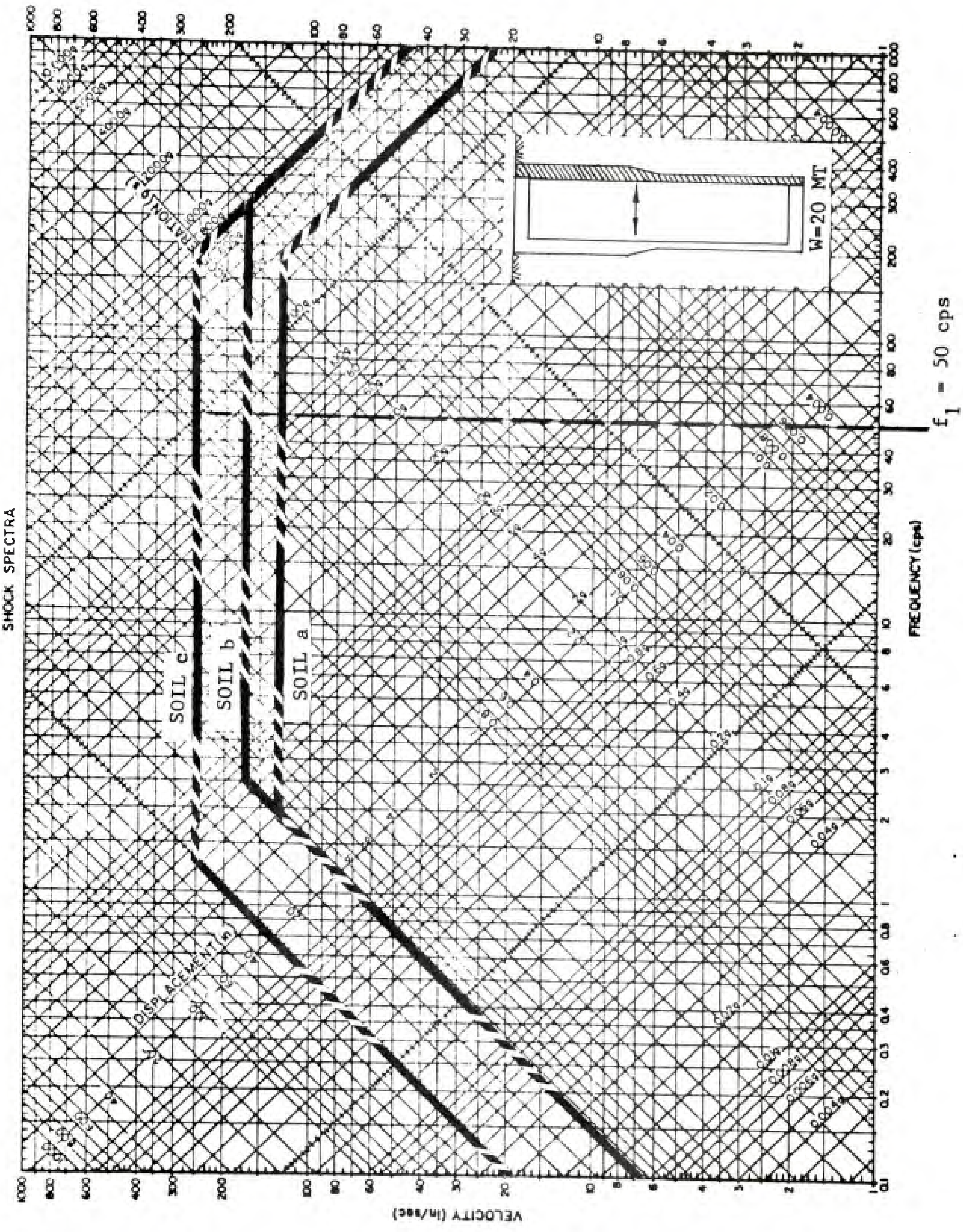

SHOCK SPECTRA
SOIL c
SOIL b
SOIL a
W=20 MT
f_1 = 50 cps
FREQUENCY (cps)
VELOCITY (in/sec)
DISPLACEMENT

FIGURE 9.14

peak input intensity to the equipment. For example in Figure 9.2, the bottom slab frequency, f_1, is approximately 12 cps and the peak response at this frequency is: (for soil condition a)

$$A_1 = 7g$$

$$V_1 = 37 \text{ in/sec}$$

$$D_1 = 0.5 \text{ inches}$$

9.4 EQUIPMENT RESPONSE

The shock spectra for equipment attached to structural elements such as walls and floors are derived from the structural element (mass m_1 in Figure 9.1) response spectra. As was noted in Section 9.3, the mass loading effects of the equipment on the structural elements is negligible i.e., the motion of the elements is essentially the same whether equipment is attached or not. Therefore the input to the equipment was modified by deriving a shock response spectra of the structural element which supports the equipment. At the frequency of the structural element, f_1, the peak intensity of the input to the equipment is defined as A_1, V_1, and D_1 as shown in Figure 9.15. The equipment response spectra bound defined by the usual three straight lines are: (see Reference 9.1 for further explanation of this method)

$$A = 2\ A_1$$

$$V = 1.5\ (v + V_1)$$

$$D = d + D_1$$

where v and d are the peak velocity and displacement inputs to the structural element (mass m_1). The modified equipment response spectra is shown in Figure 9.15. Occasionally, the ratio of the equipment's natural frequency (f_2) and the structural element's deformational natural frequency (f_1) may fall in the same range $0.3 \leqslant f_1/f_2 \leqslant 3$, then the response of the equipment will be greater than predicted by the above method. The degree of amplification is dependent on the frequency ratio f_1/f_2 and the damping in both the

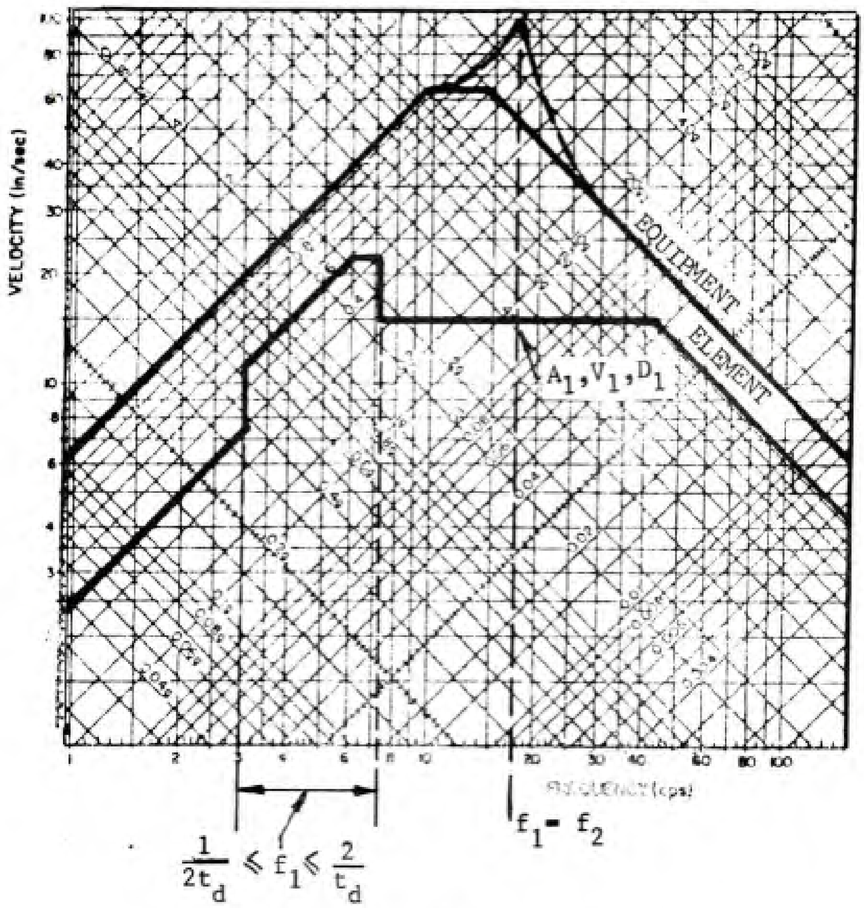


FIGURE 9.15

structure and the equipment. An idea of the amplification can be determined from response charts (e.g. Reference 9.3) relating these parameters. A typical resonance condition appears in Figure 9.15 as a dashed line.

Shock spectra for equipment attached to walls and floors of the six structures are presented in Figures 9.16 - 9.45. Each spectra has a pictorial inset which indicates the structure type and direction of shock. In addition, the structural element and the equipment location are identified by cross hatching. One percent structural damping is assumed except for structural elements which are backed by soil in which case 5 percent damping is used. Five percent damping is also assumed for all equipments. Triangles are used to indicate the natural frequencies and hence the responses for equipments.

The response of equipments not considered in this report can be obtained at the intersection of their natural frequencies and the applicable equipment shock spectra.

The large quantity of data presented in the shock spectra plots has been summarized in Table 9.3. The fragility levels of equipment are most frequently specified in terms of acceleration levels over a frequency band. Therefore, Table 9.3 contains the maximum acceleration response of each of the equipments as a function of structure type (numerically 1 through 6 as per Section 1), soil condition (a, b, and c as described in Section 1) and direction of shock.

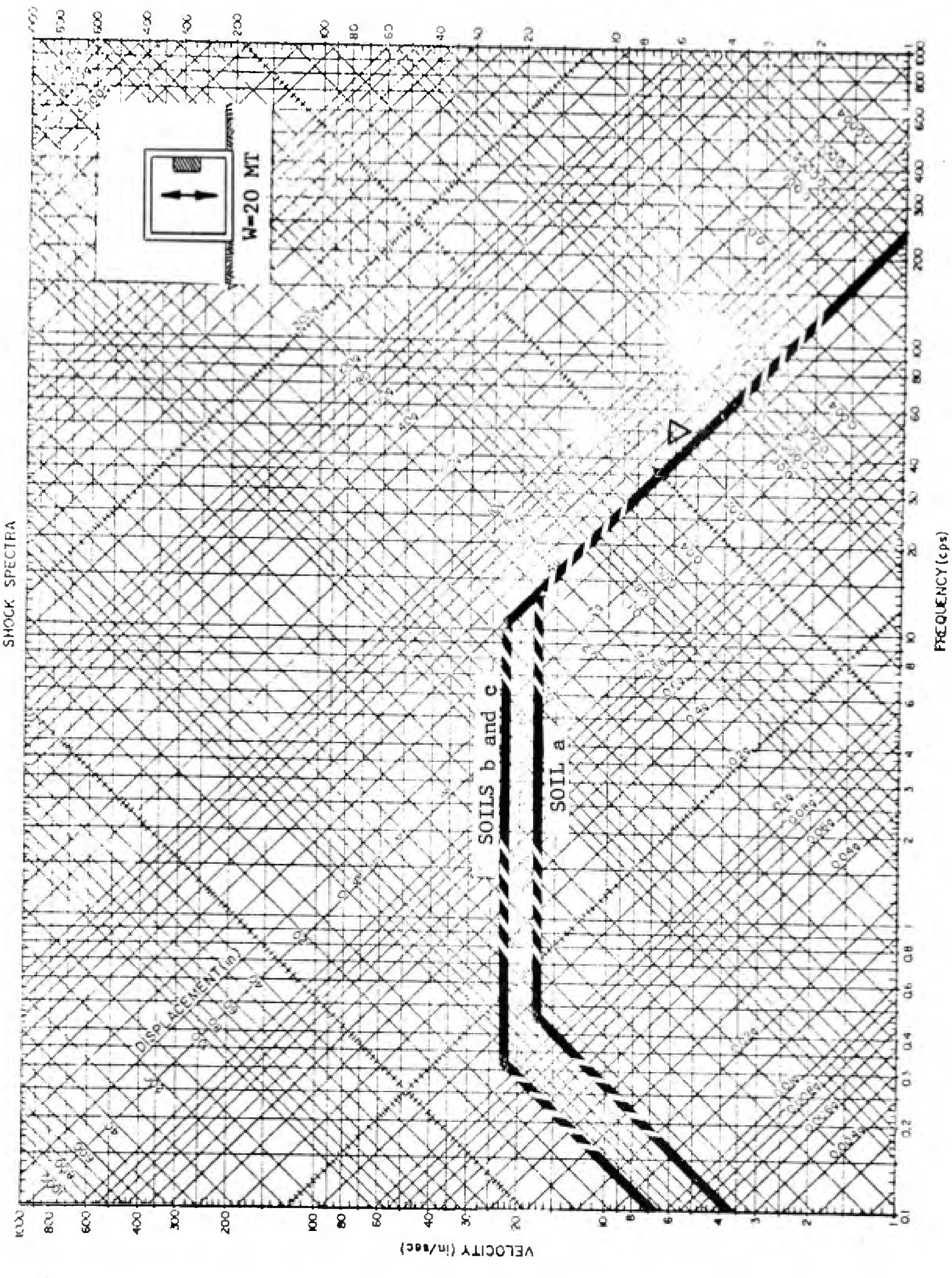


FIGURE 9.16

STL FORM 2221A

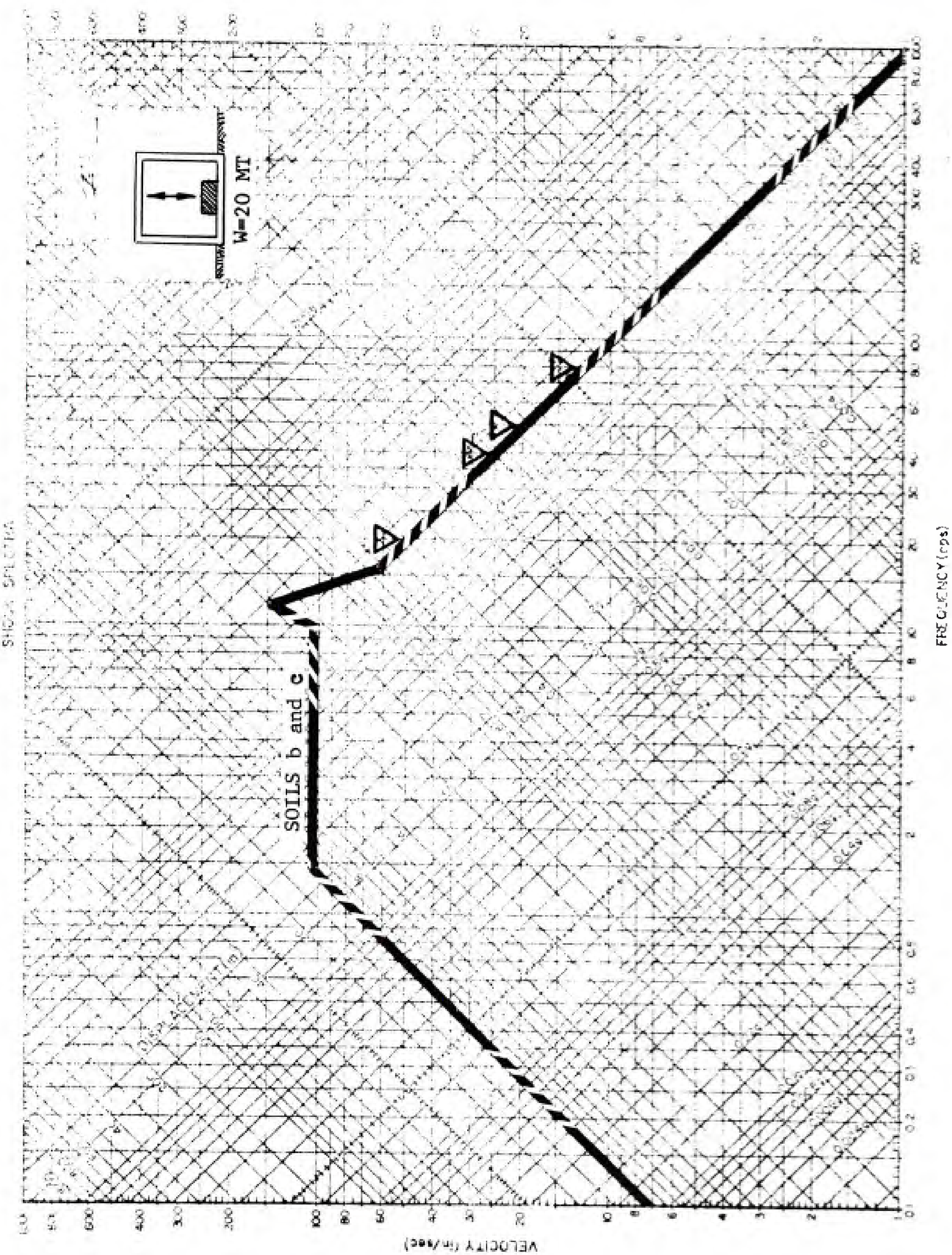

SHOCK SPECTRA
W=20 MT
SOILS b and c
VELOCITY (in/sec)
FREQUENCY (cps)

FIGURE 9.17

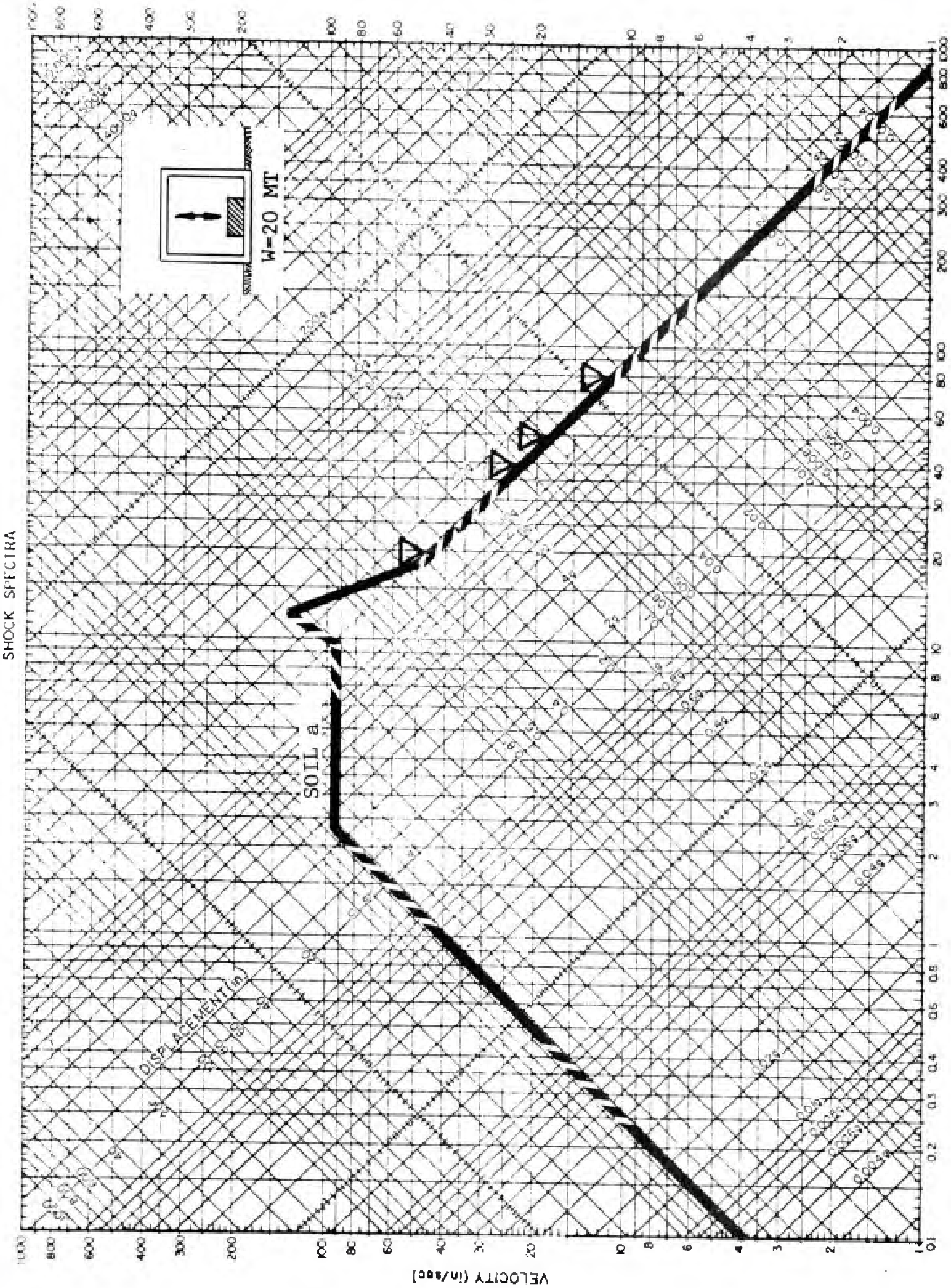

SHOCK SPECTRA
W=20 MT
SOIL a
DISPLACEMENT (in)
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.18

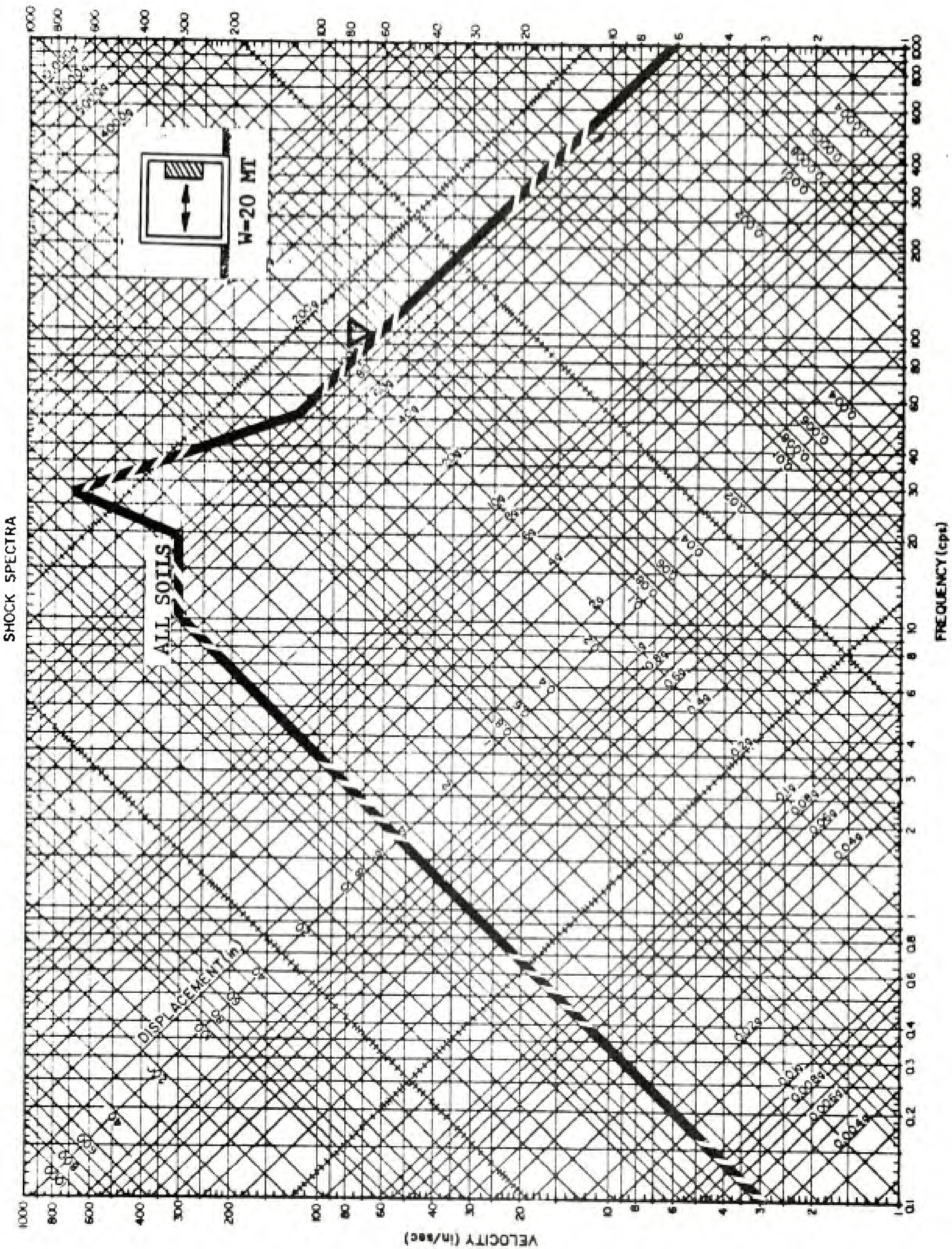

SHOCK SPECTRA
ALL SOILS
W-20 MT
DISPLACEMENT (in)
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.19

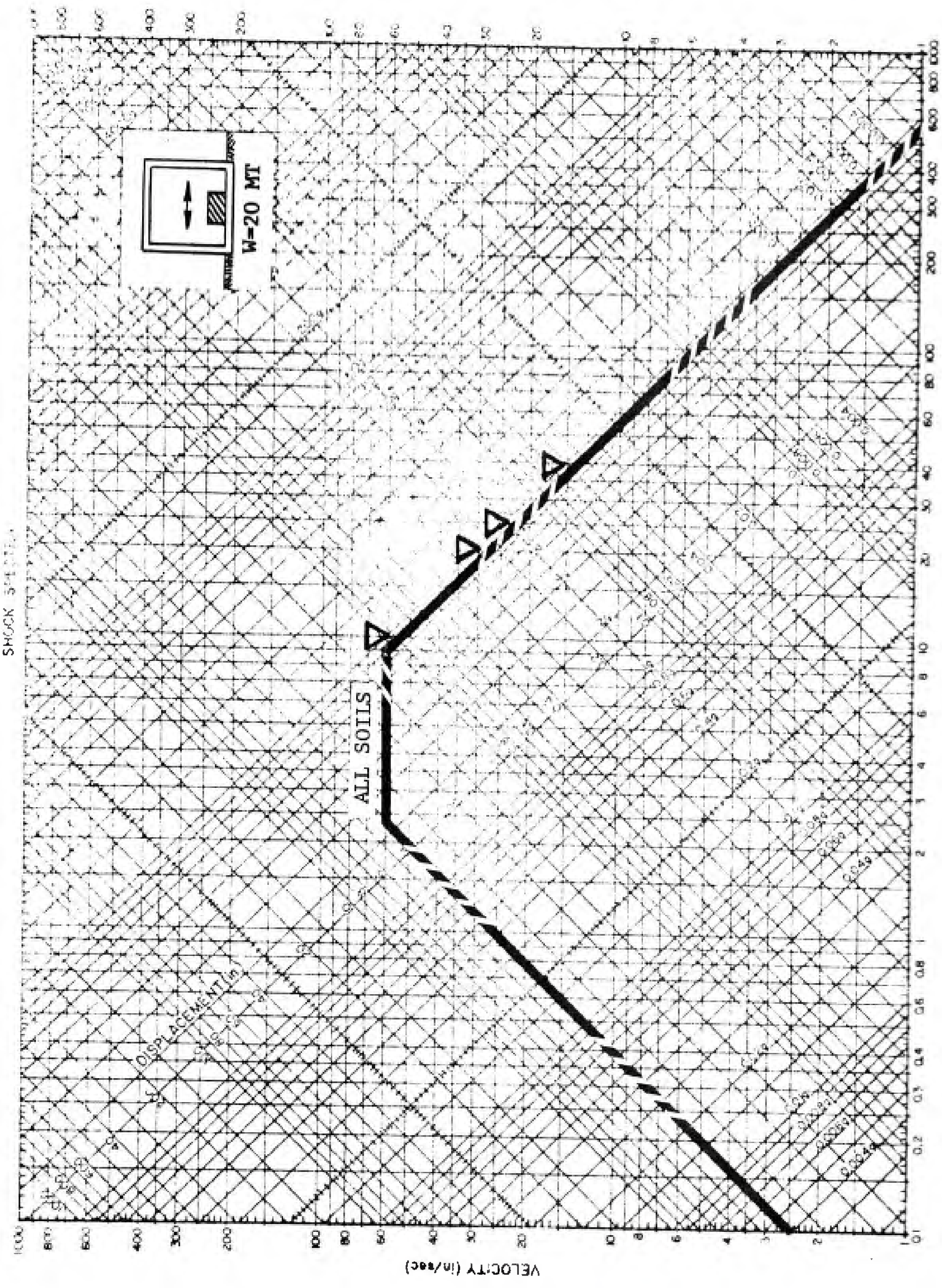

SHOCK SPECTRA
W=20 MT
ALL SOILS
DISPLACEMENT (in)
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.20

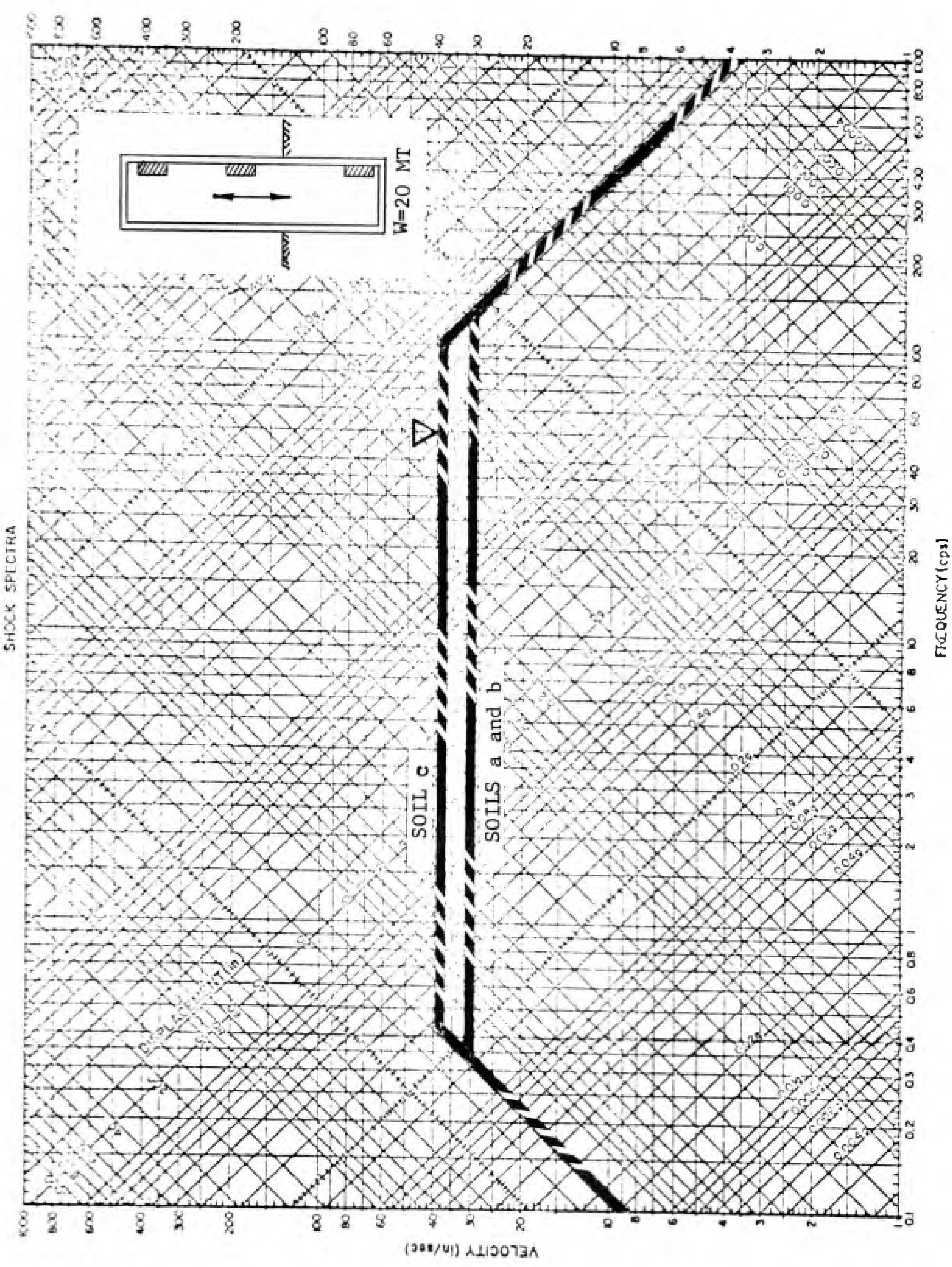

SHOCK SPECTRA
W=20 MT
SOIL c
SOILS a and b
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.21

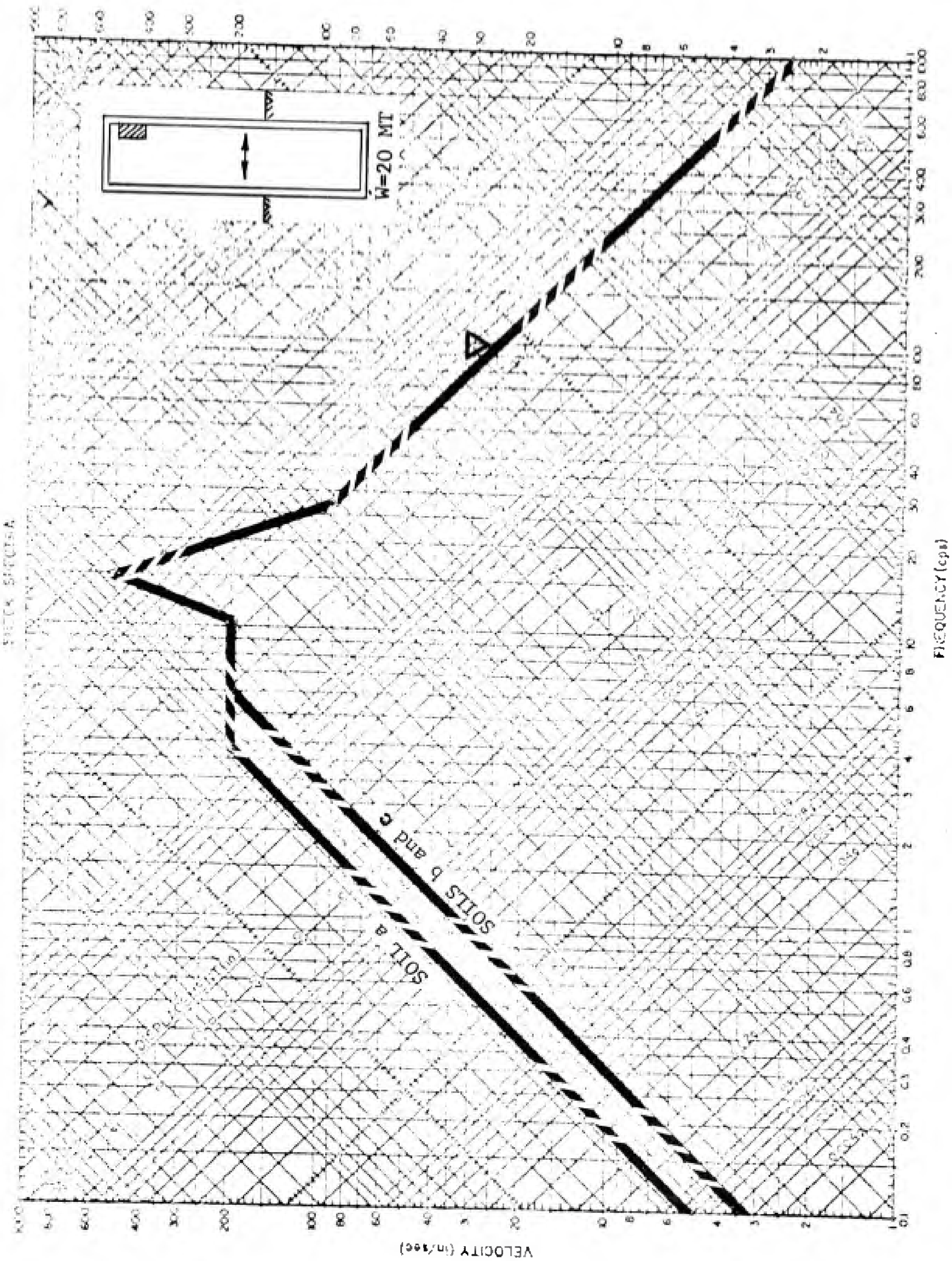


FIGURE 9.22

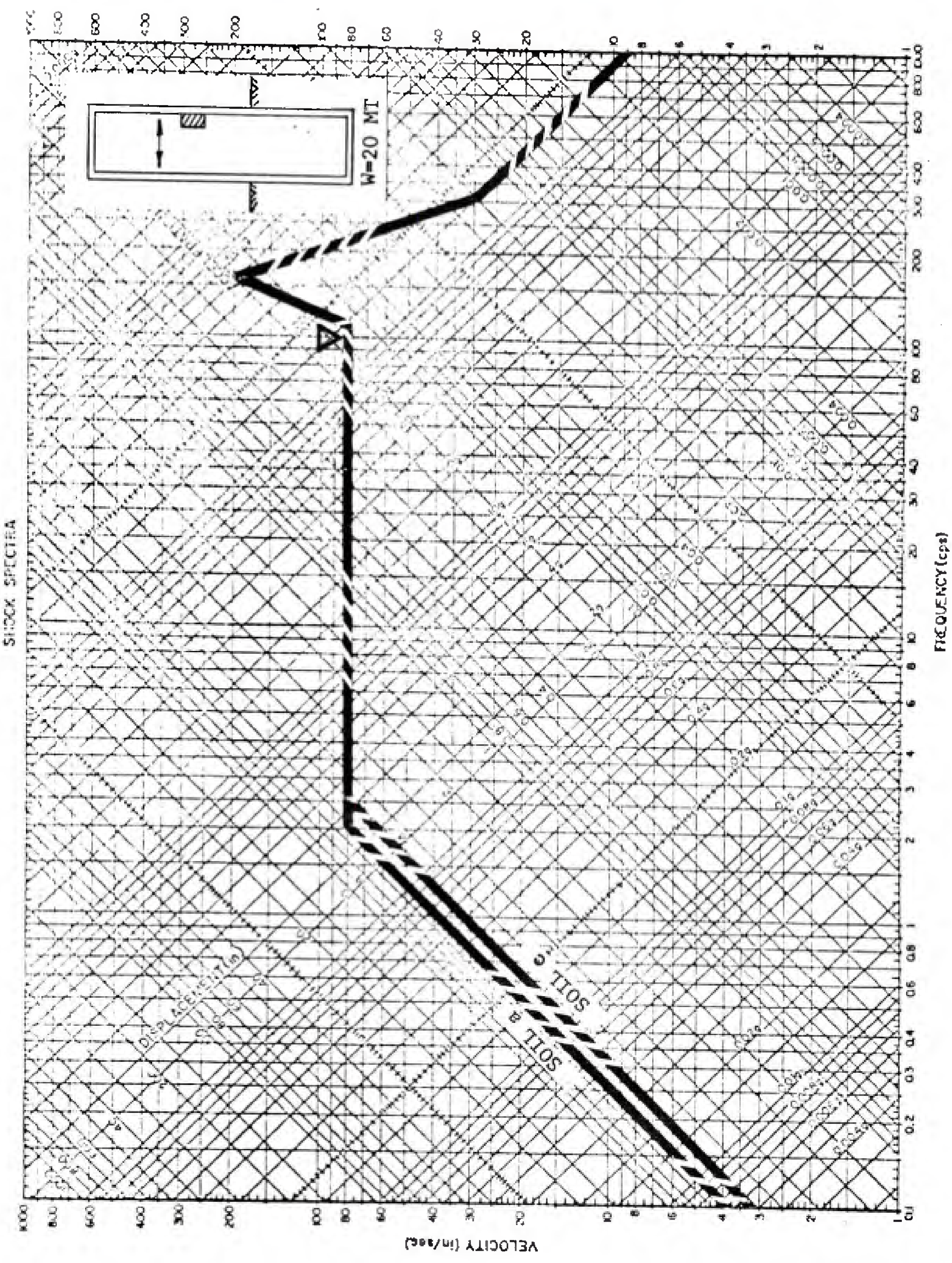

SHOCK SPECTRA
W=20 MT
SOIL B
SOIL C
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.23

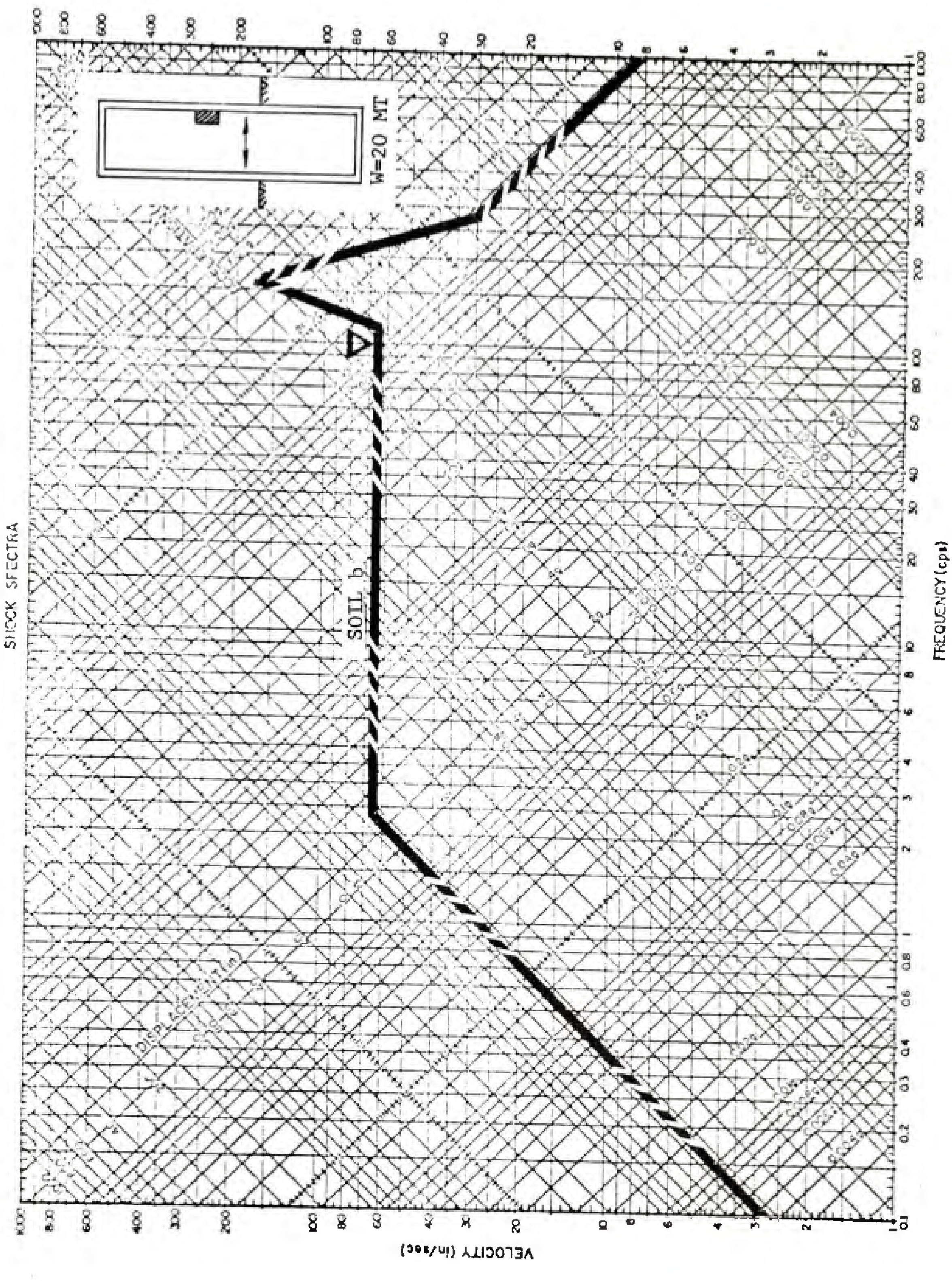


FIGURE 9.24

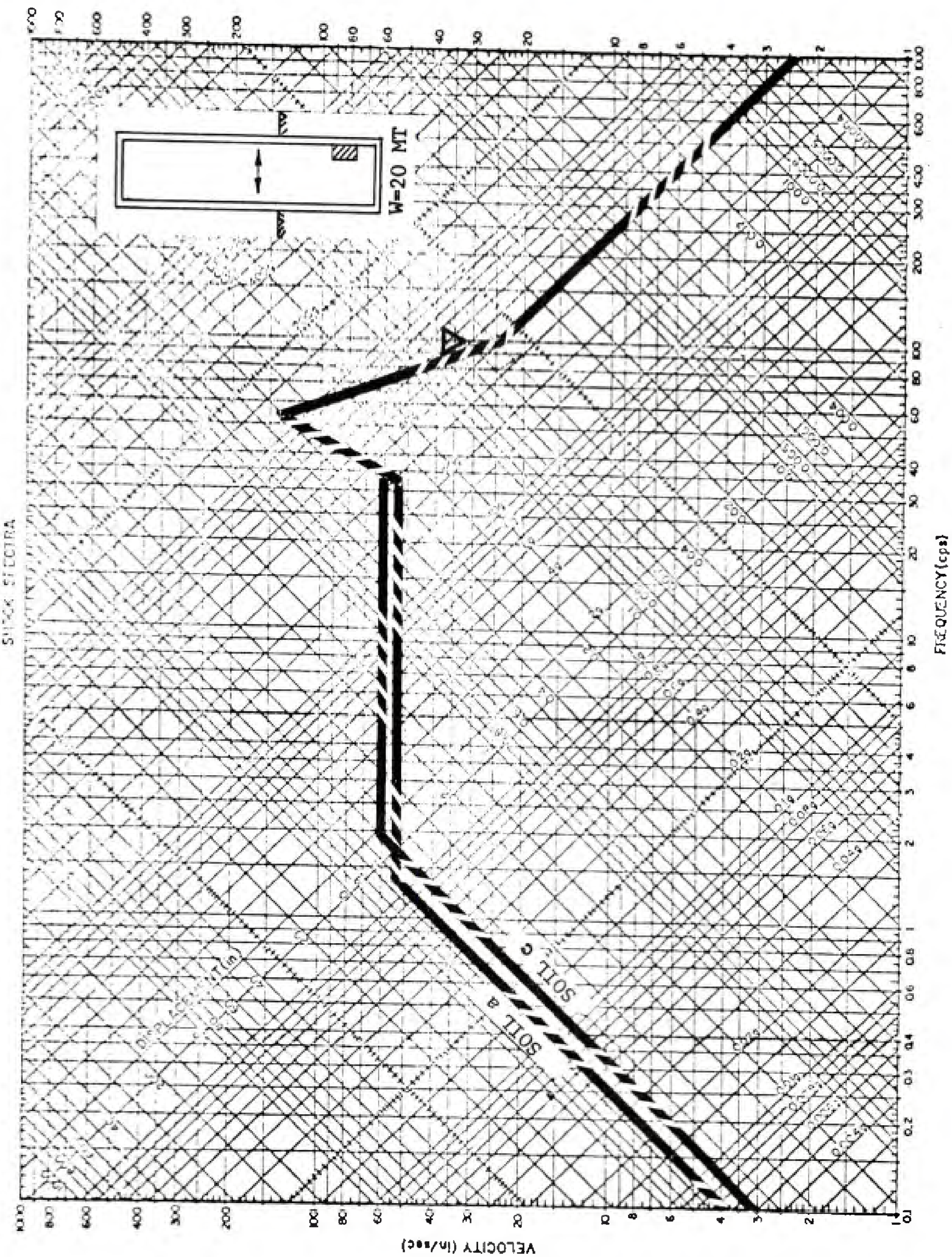


FIGURE 9.25

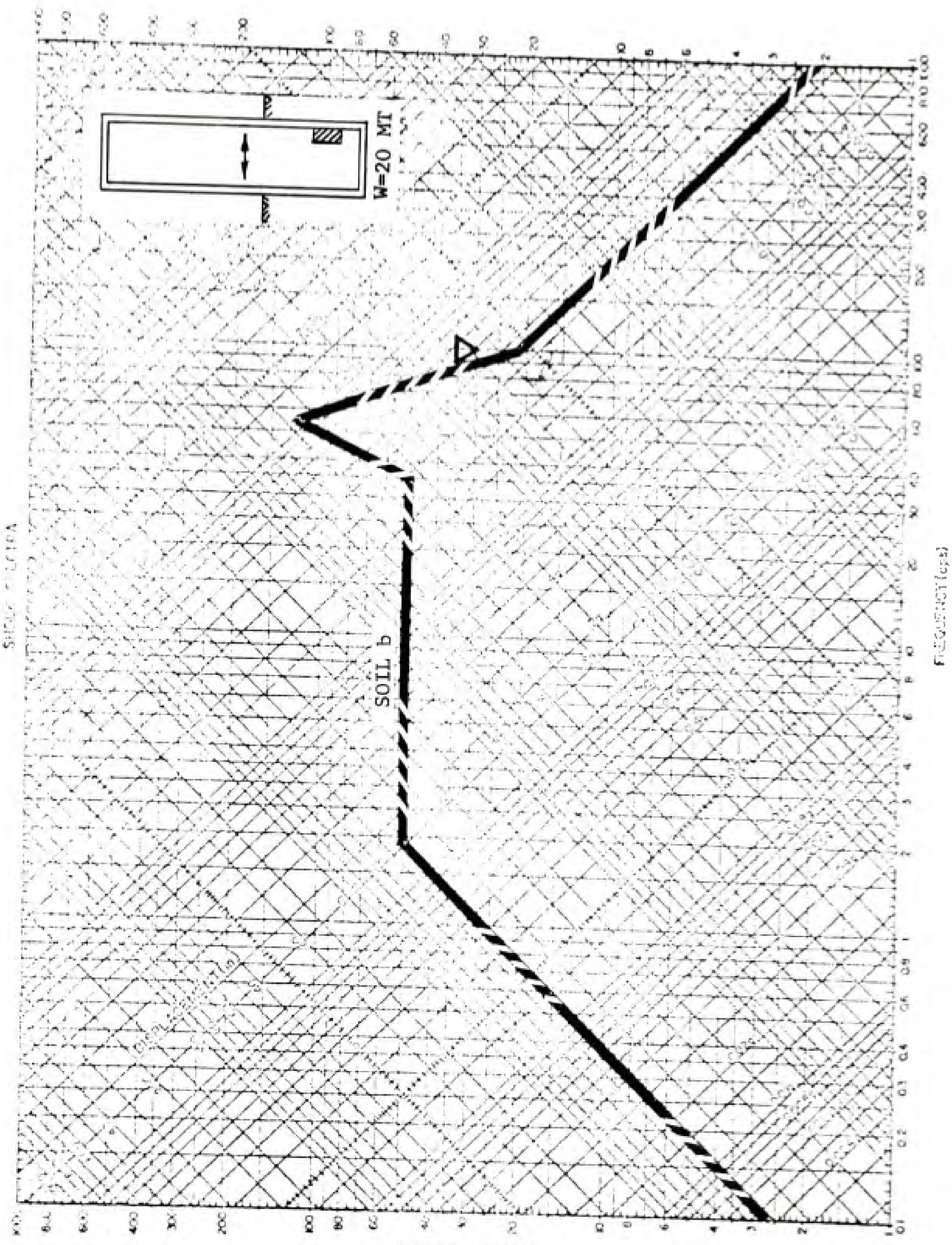


FIGURE 9.26

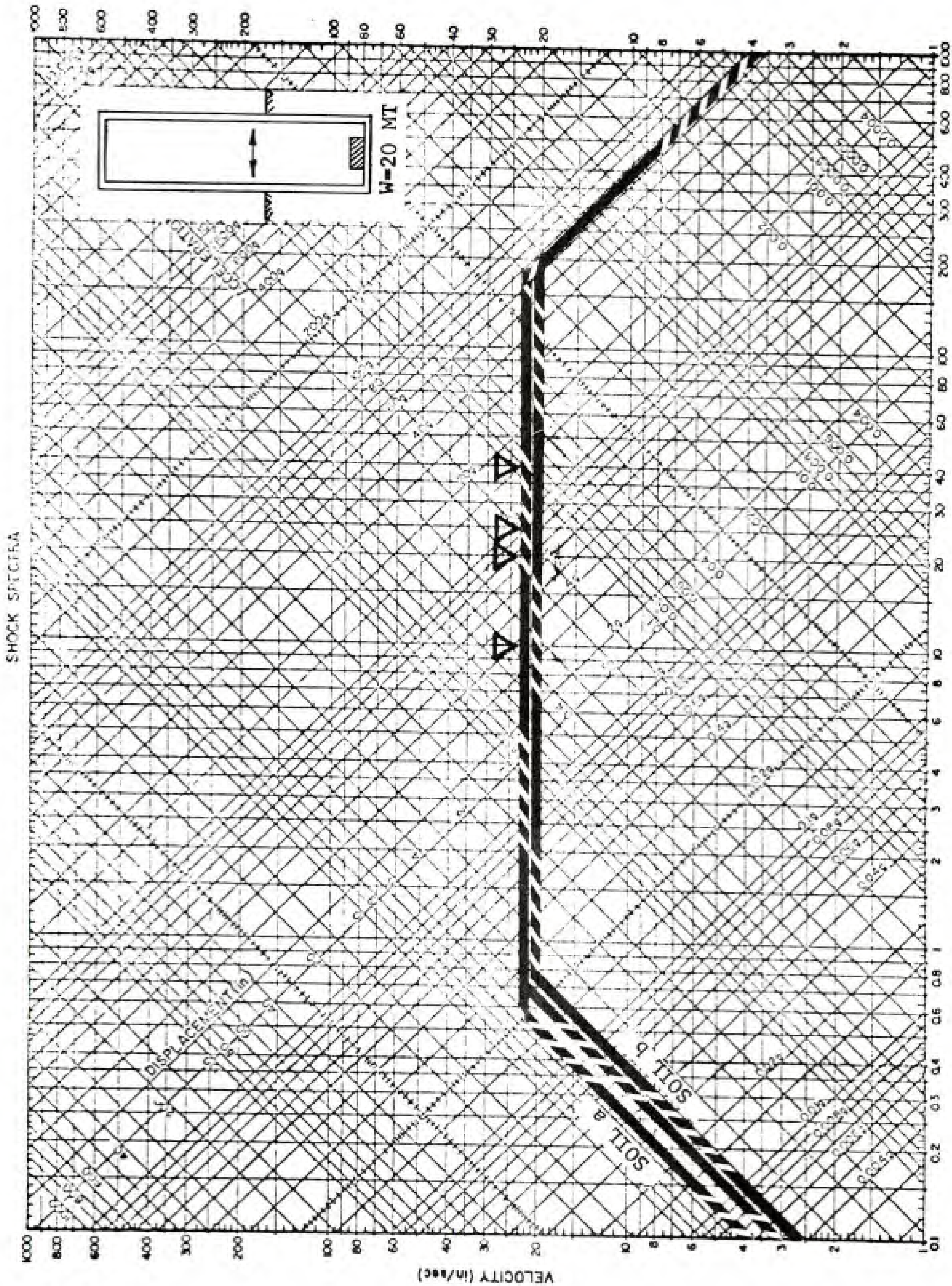

SHOCK SPECTRA
W=20 MT
SOIL B
SOIL D
DISPLACEMENT (in)
VELOCITY (in/sec)
FREQUENCY (cps)

FIGURE 9.27

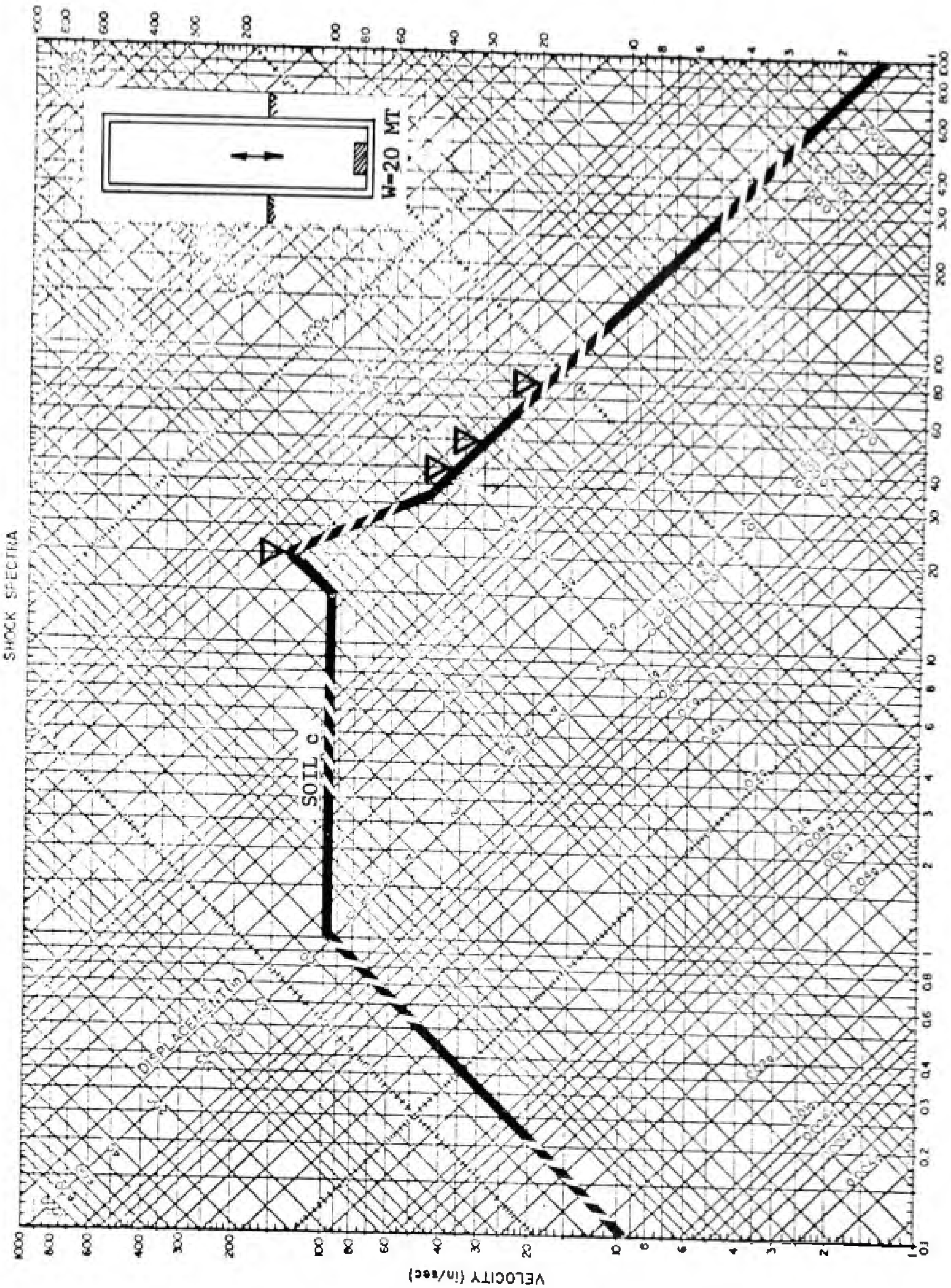


FIGURE 9.28

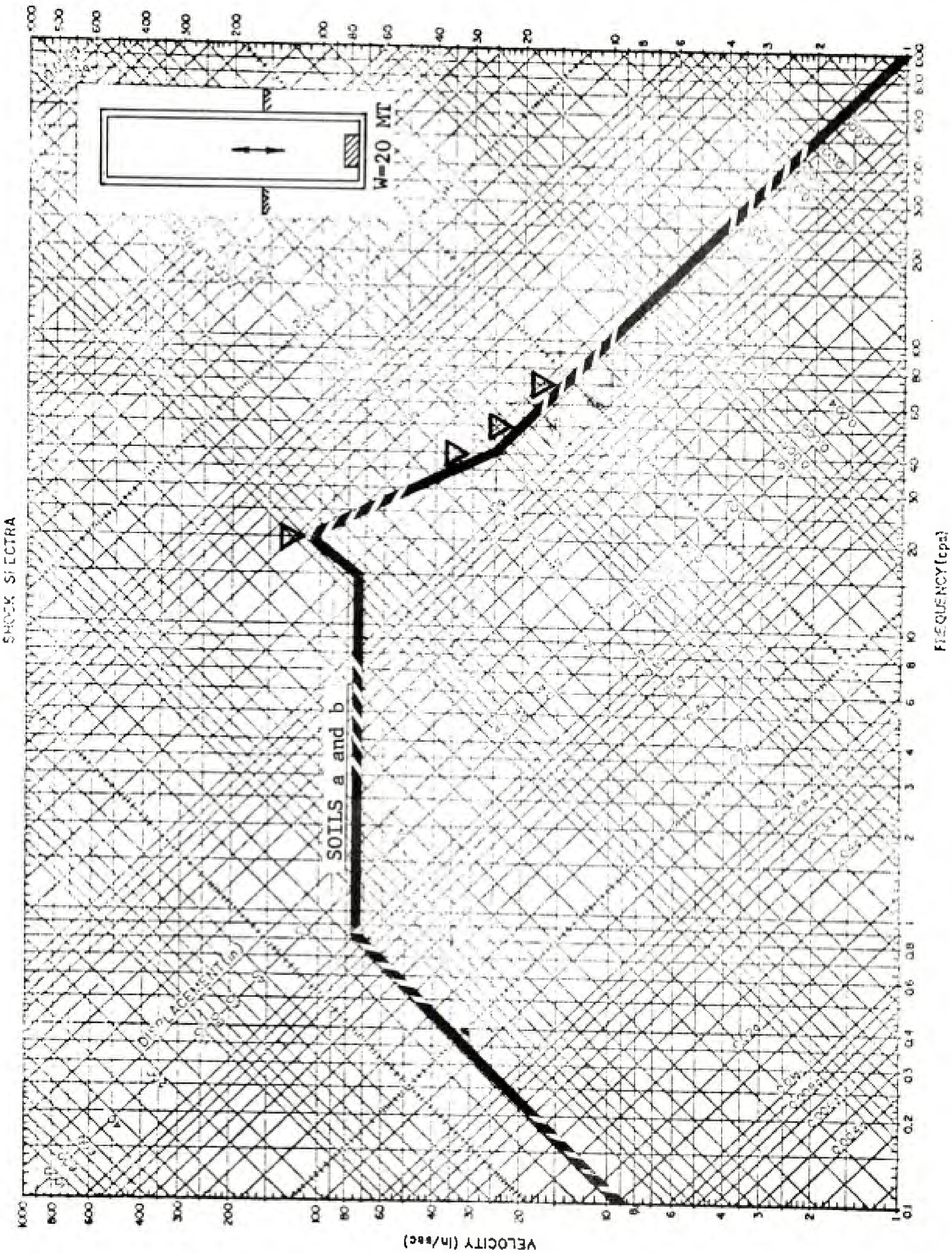

SHOCK SPECTRA
W=20 MT
SOILS a and b
VELOCITY (in/sec)
FREQUENCY (cps)

FIGURE 9.29

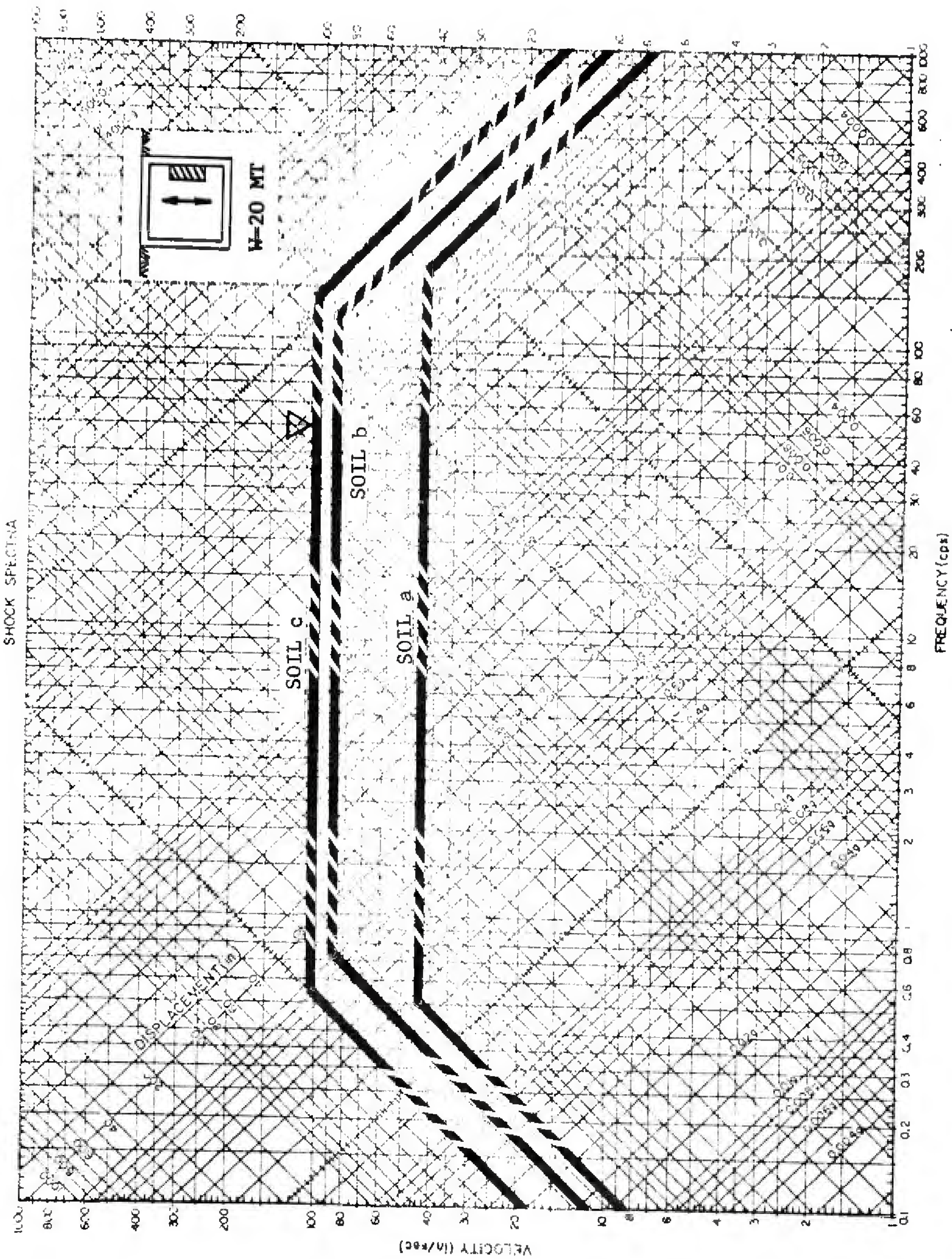

SHOCK SPECTRA
W=20 MT
SOIL c
SOIL b
SOIL a
DISPLACEMENT (in)
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.30

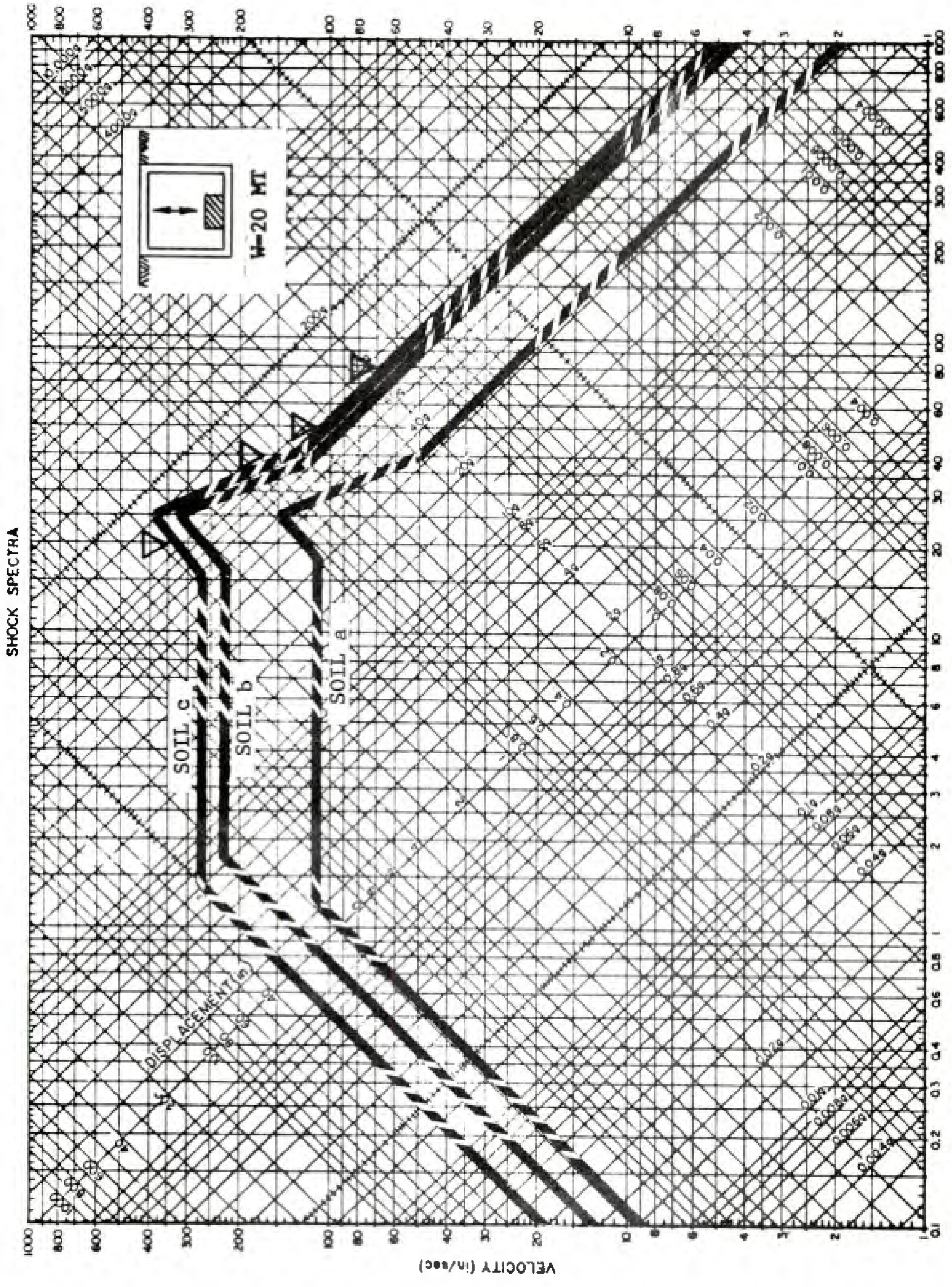

SHOCK SPECTRA
W-20 MT
SOIL c
SOIL b
SOIL a
DISPLACEMENT (in)
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.31

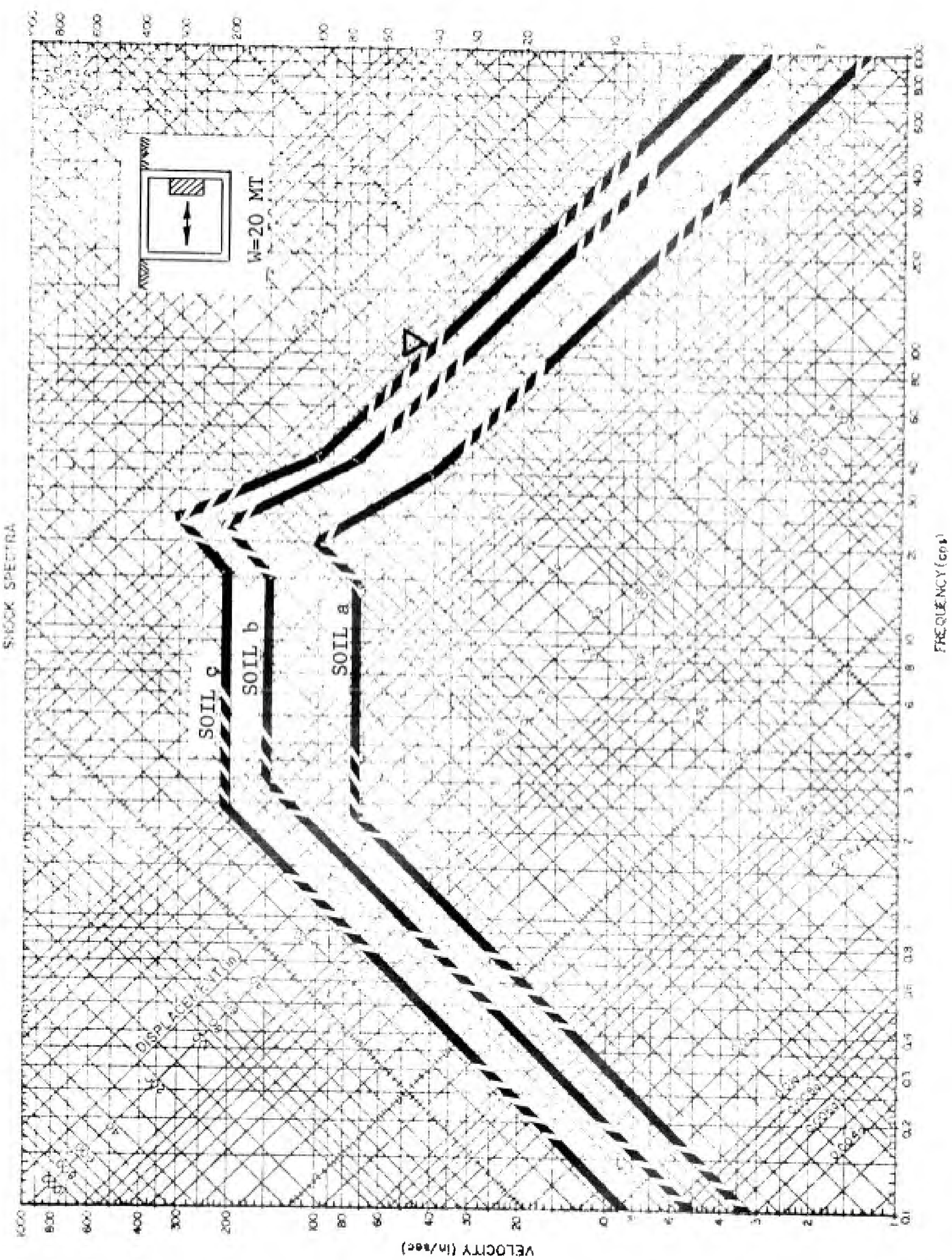

SHOCK SPECTRA
W=20 MT
SOIL c
SOIL b
SOIL a
VELOCITY (in/sec)
FREQUENCY (cps)

FIGURE 9.32

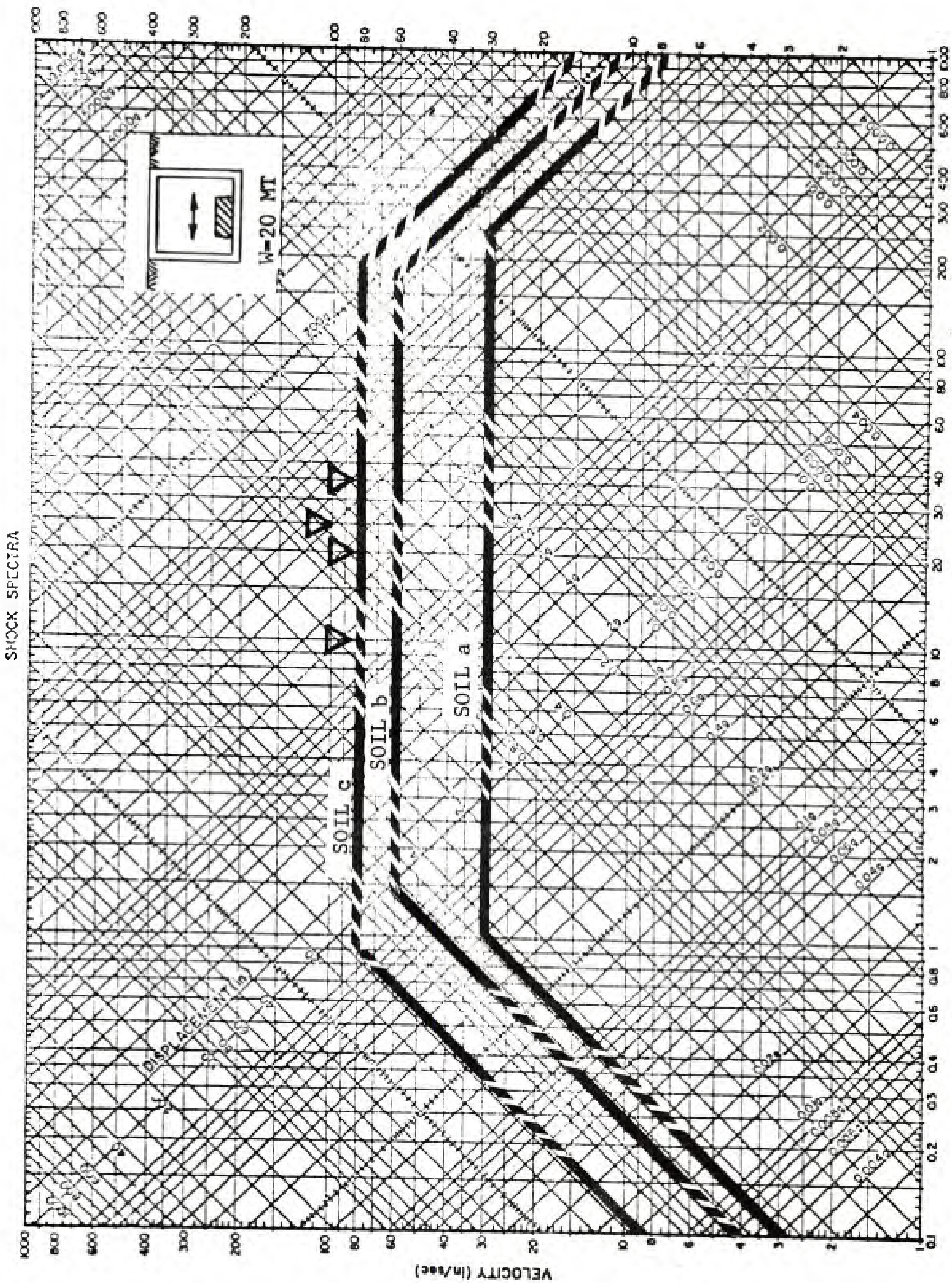

SHOCK SPECTRA
W-20 MT
SOIL c
SOIL b
SOIL a
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.33

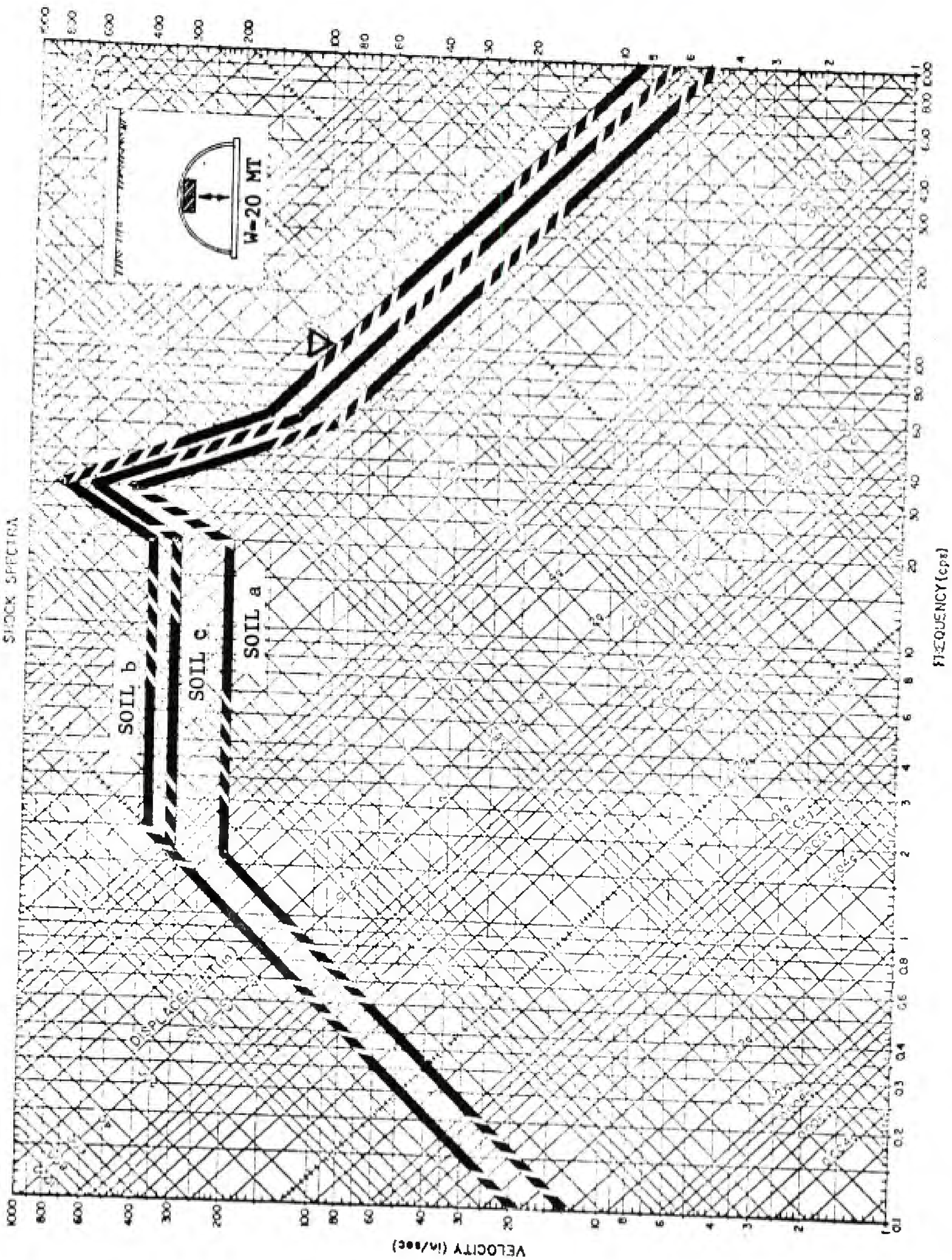

SHOCK SPECTRA
SOIL b
SOIL c
SOIL a
W-20 MT
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.34

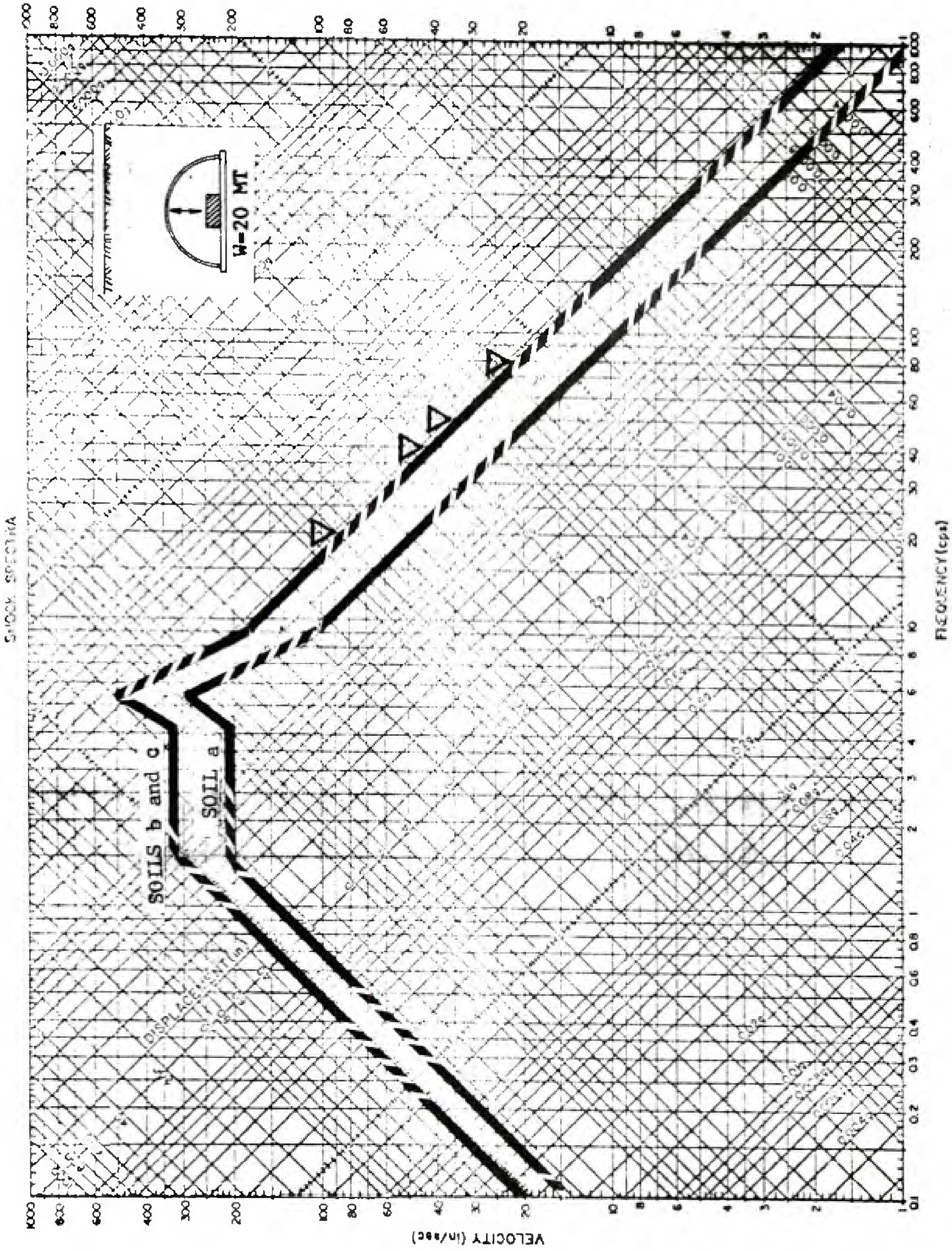

SHOCK SPECTRA
W=20 MT
SOILS b and c
SOIL a
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.35

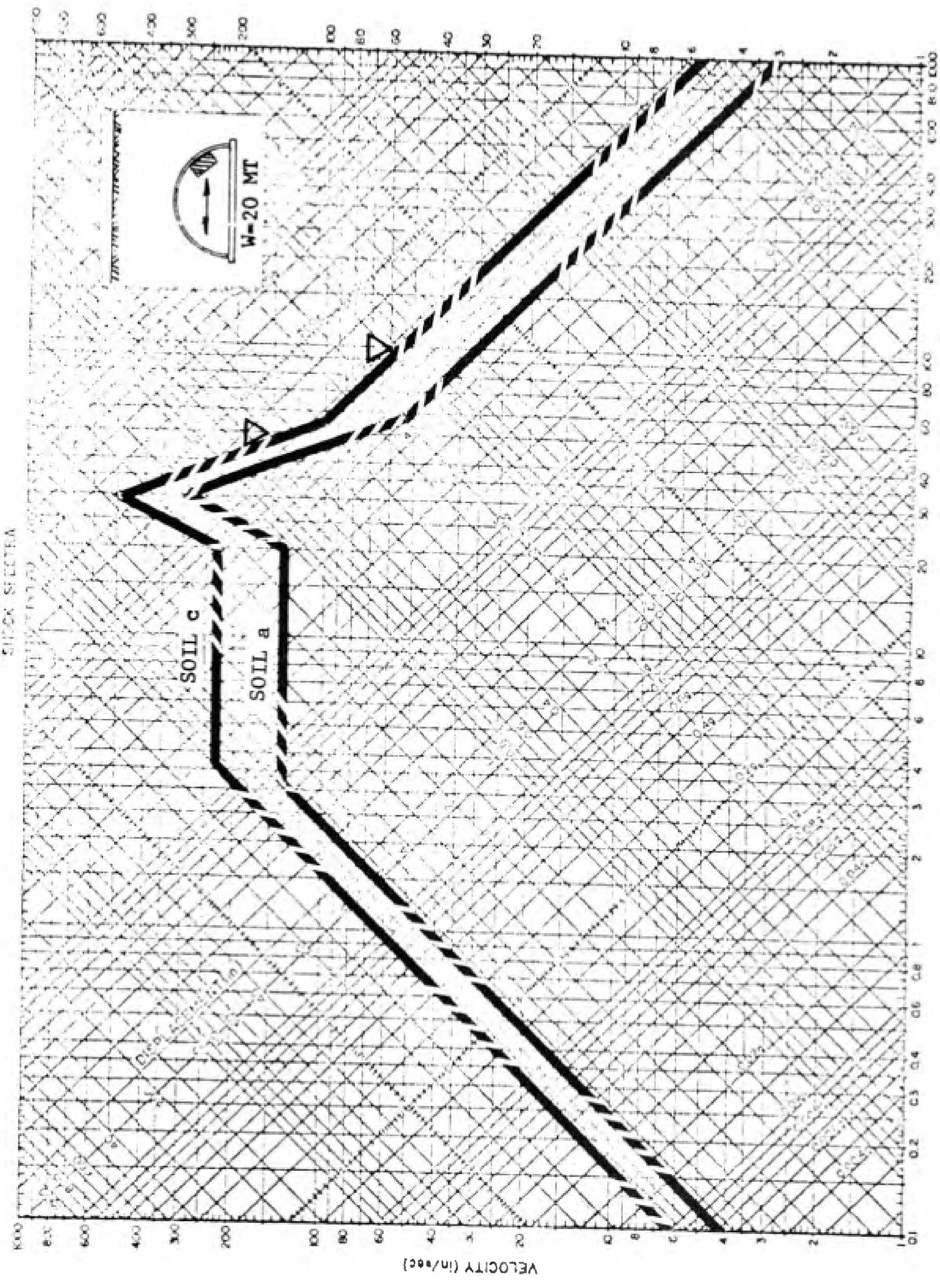

SHOCK SPECTRA
W-20 MT
SOIL c
SOIL a
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.36

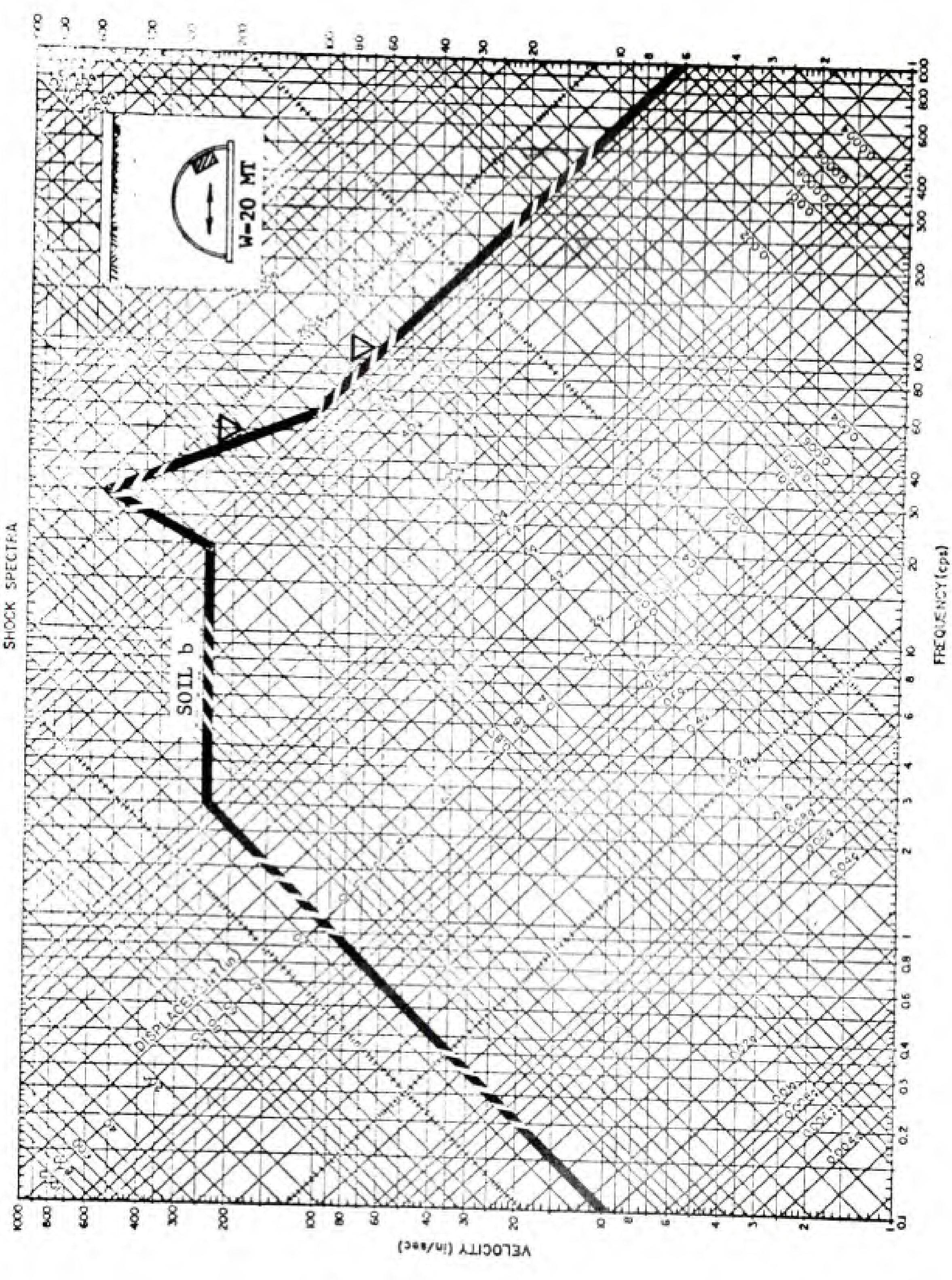


FIGURE 9.37

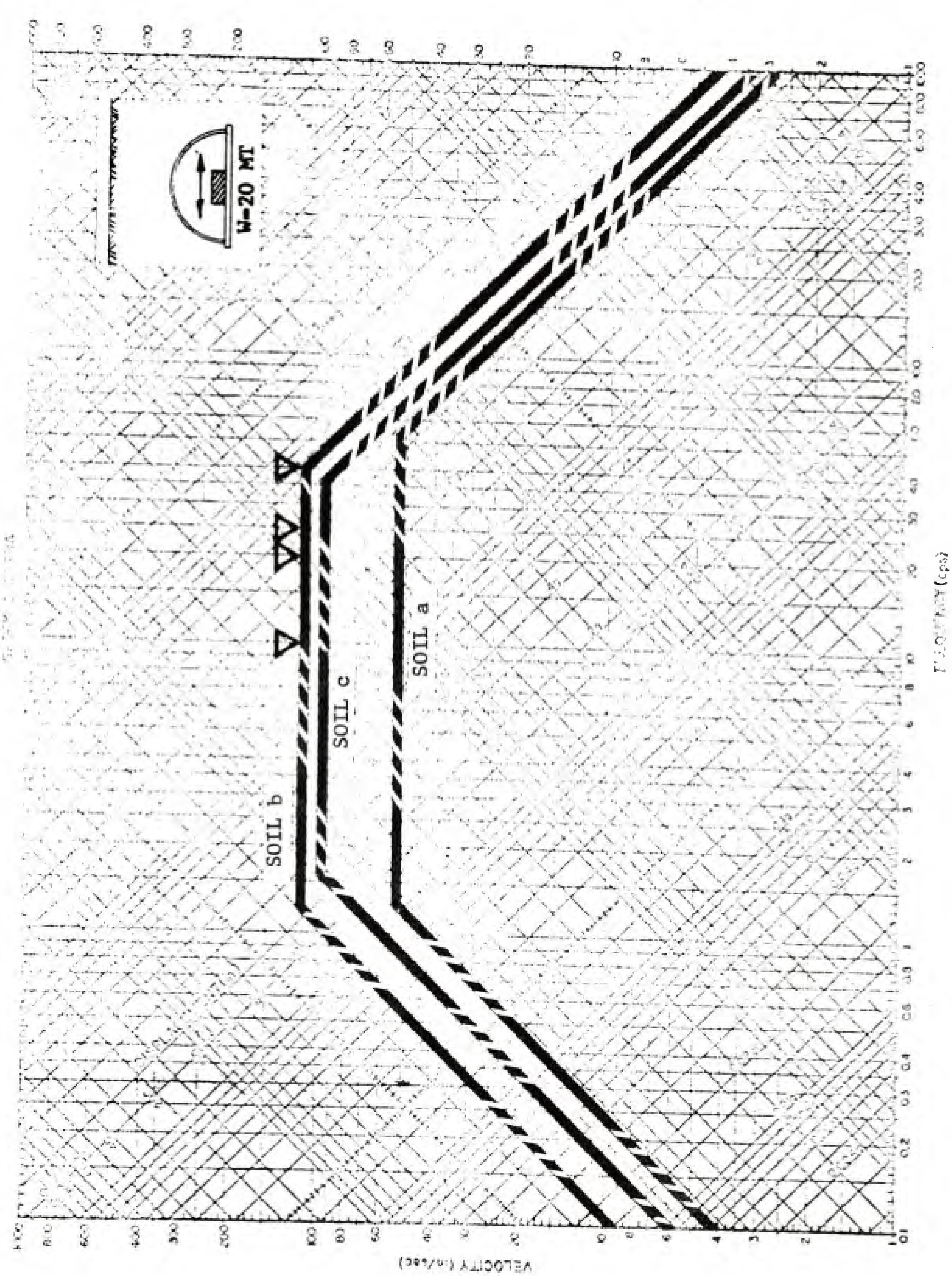


FIGURE 9.38

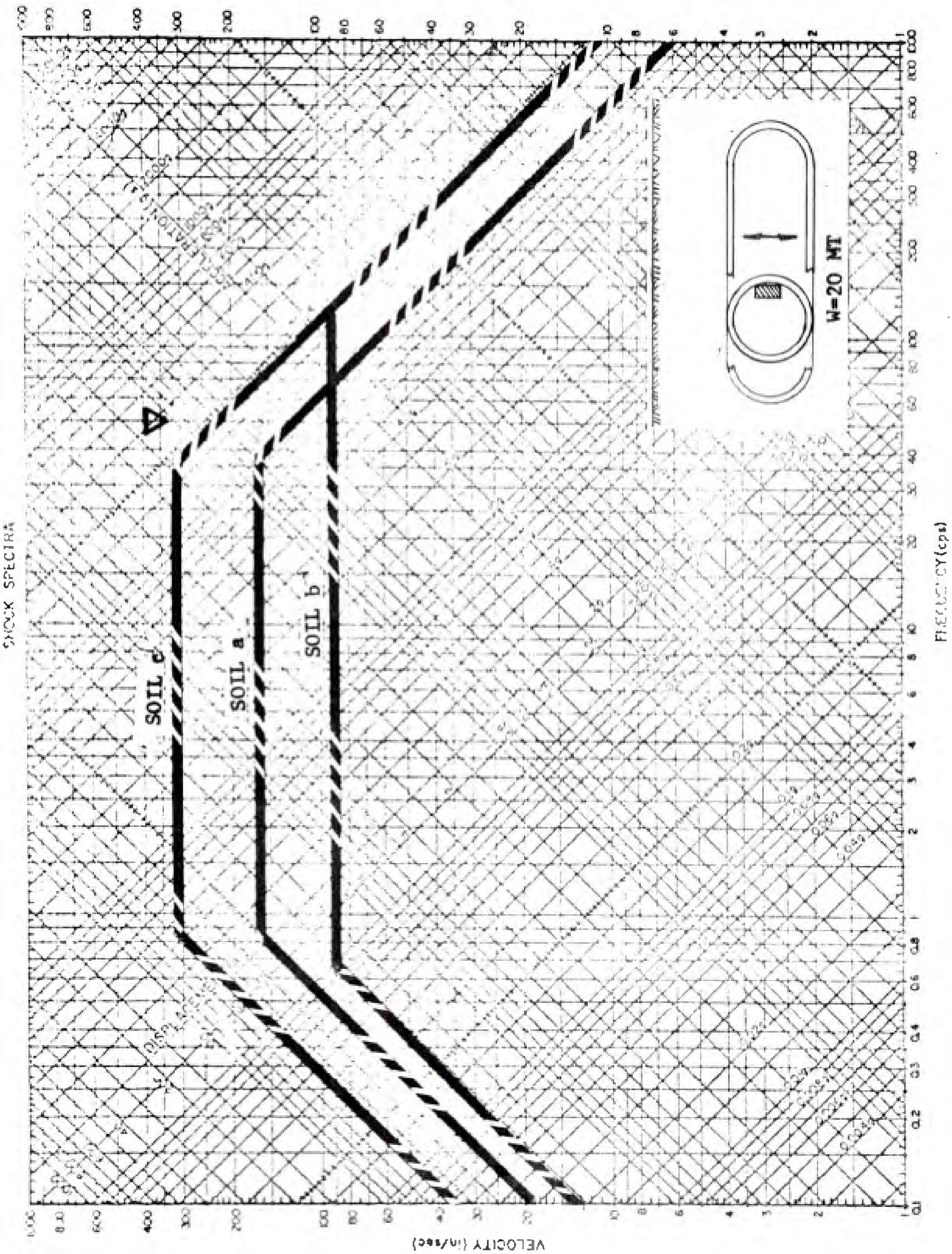


FIGURE 9.39

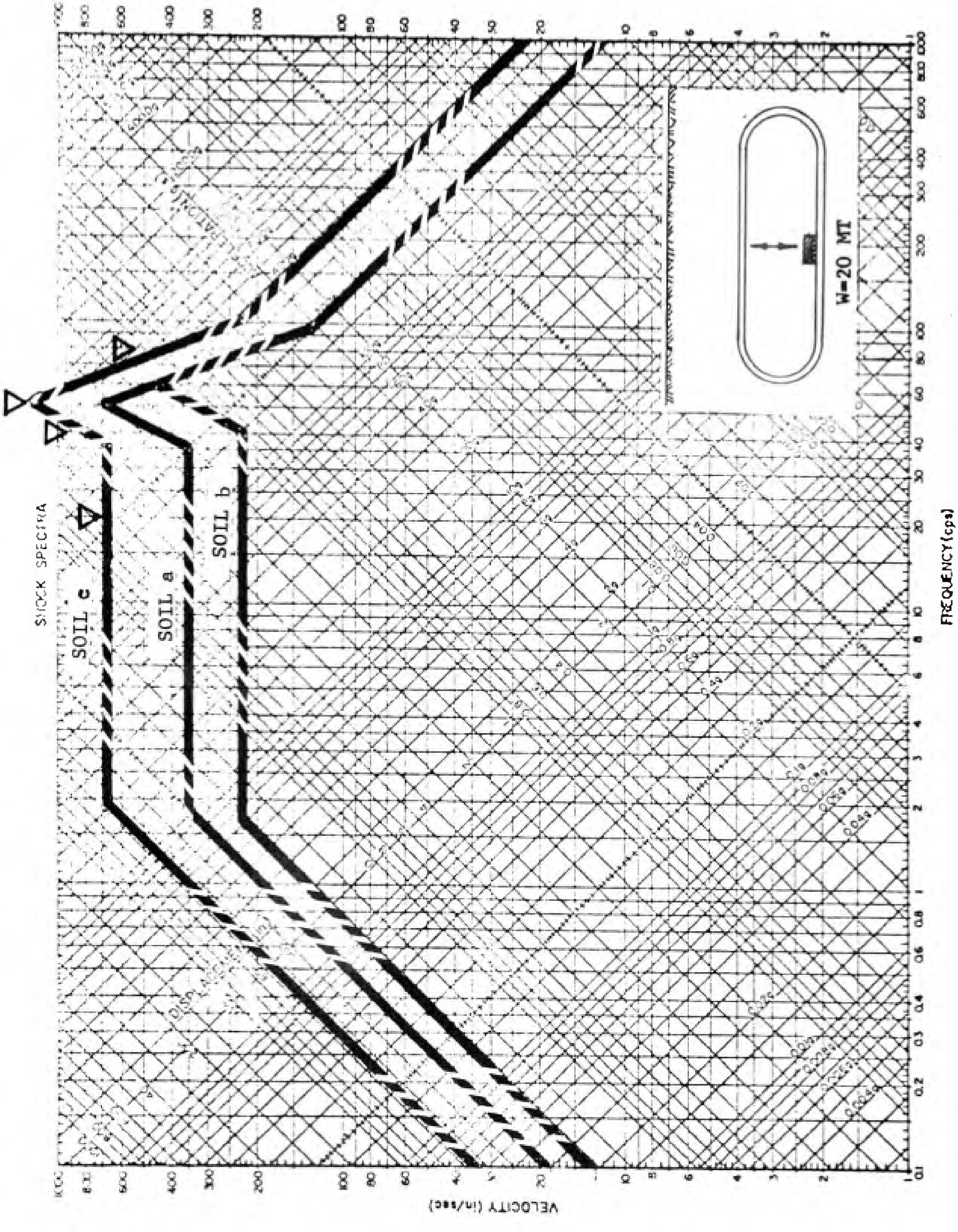

SHOCK SPECTRA
SOIL c
SOIL a
SOIL b
W=20 MT
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.40

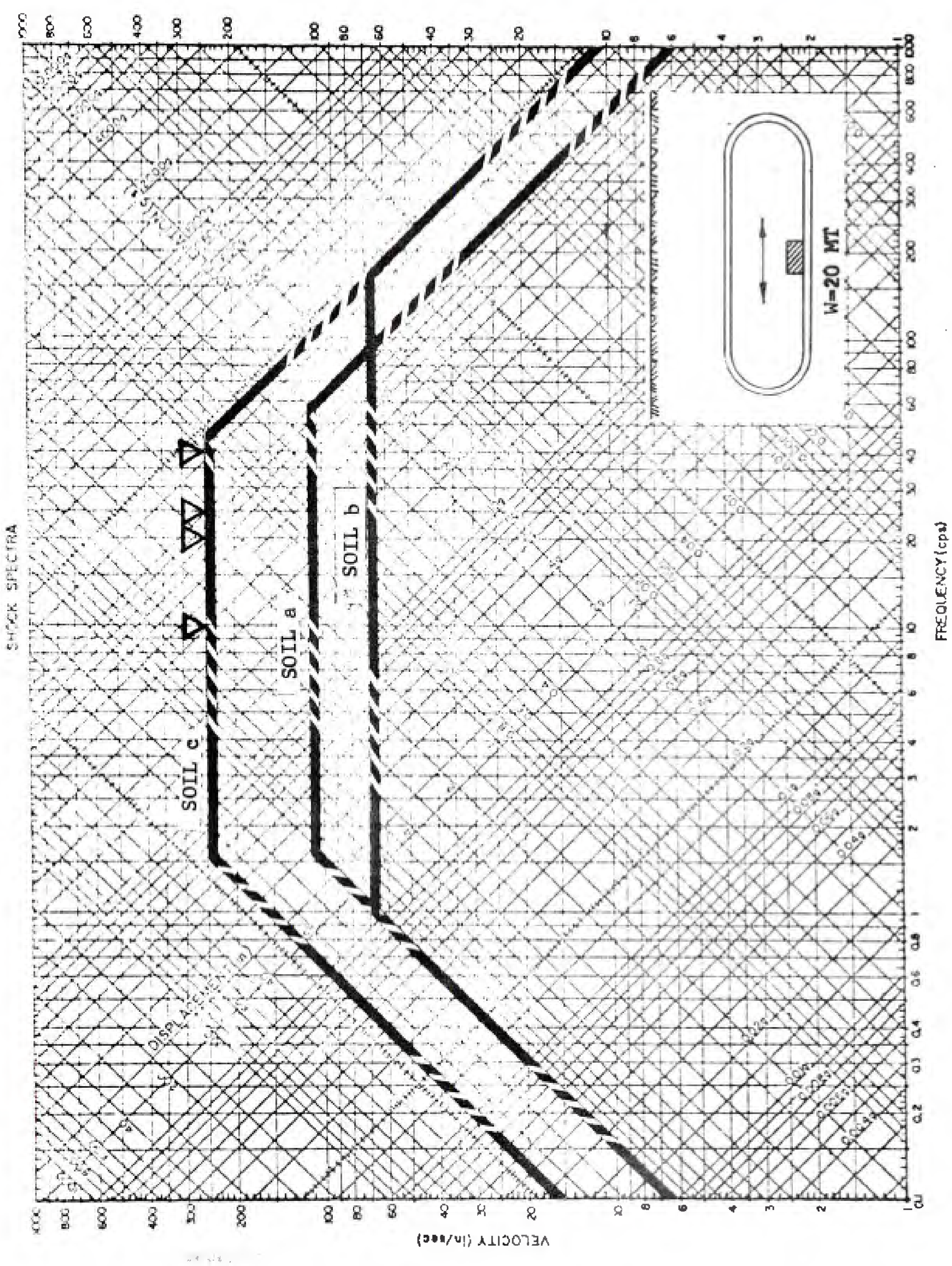

SHOCK SPECTRA
SOIL a
SOIL b
SOIL c
W=20 MT
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.41

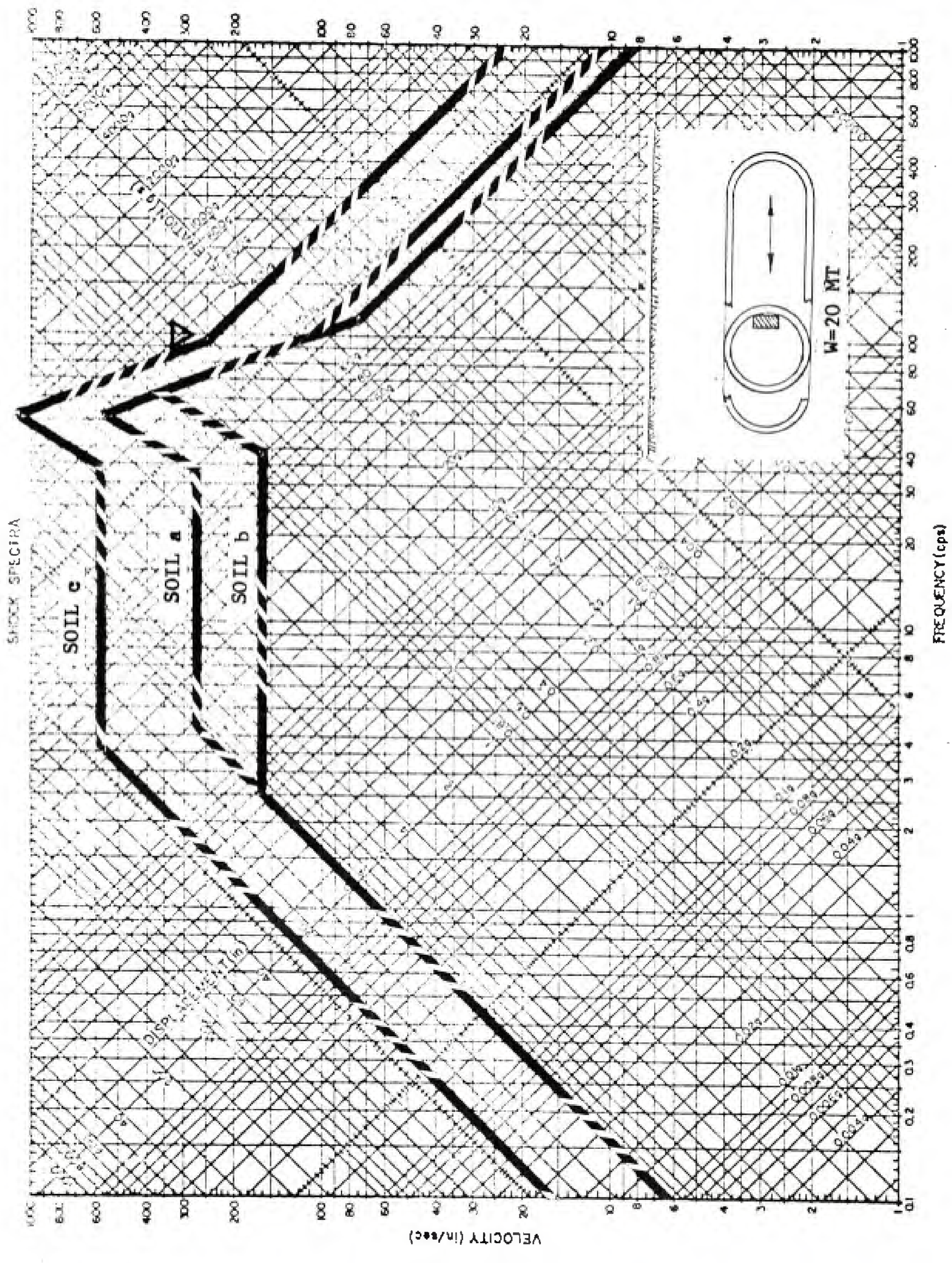

SHOCK SPECTRA
SOIL c
SOIL a
SOIL b
W=20 MT
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.42

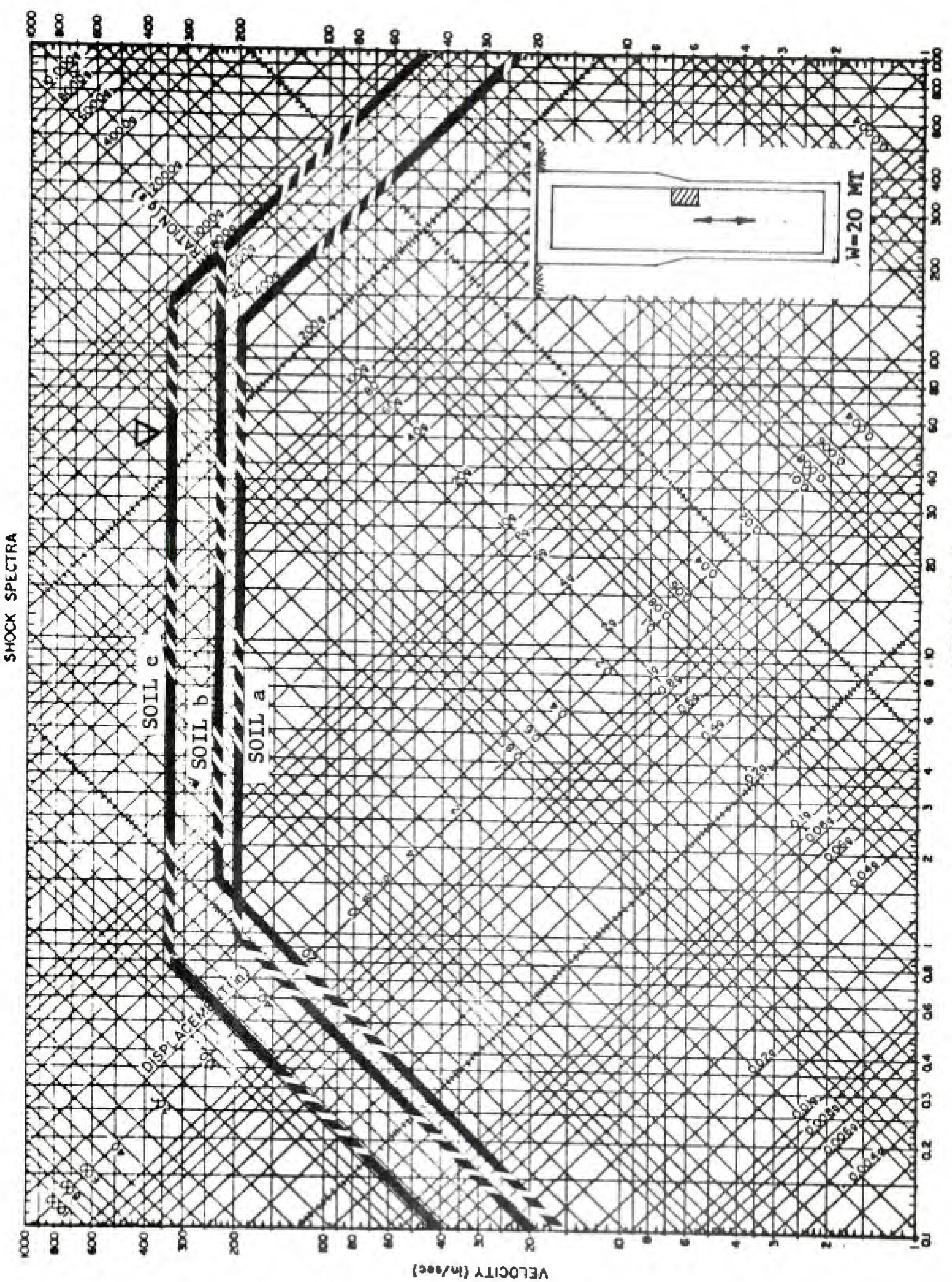

SHOCK SPECTRA
SOIL a
SOIL b
SOIL c
W=20 MT
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.43

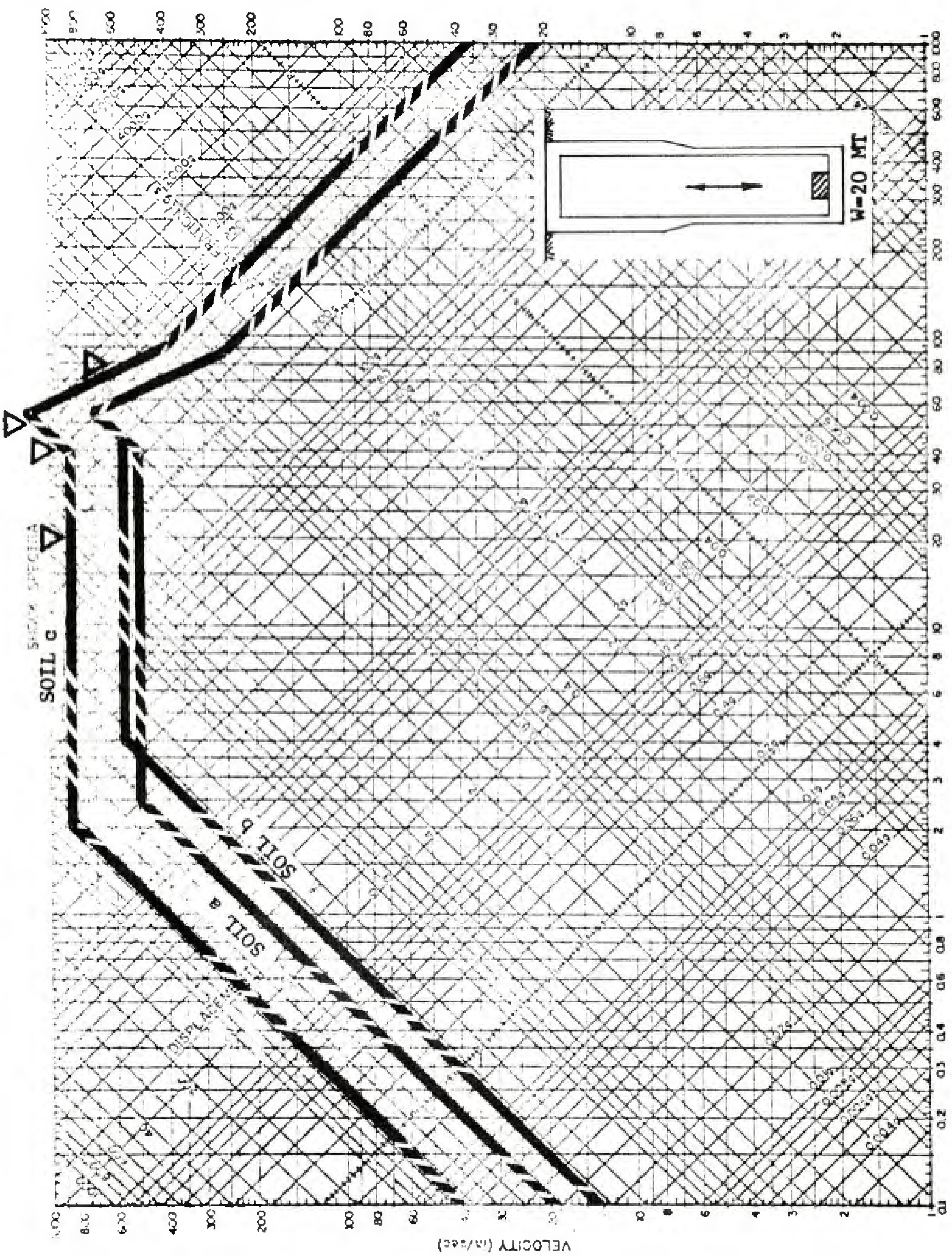


FIGURE 9.44

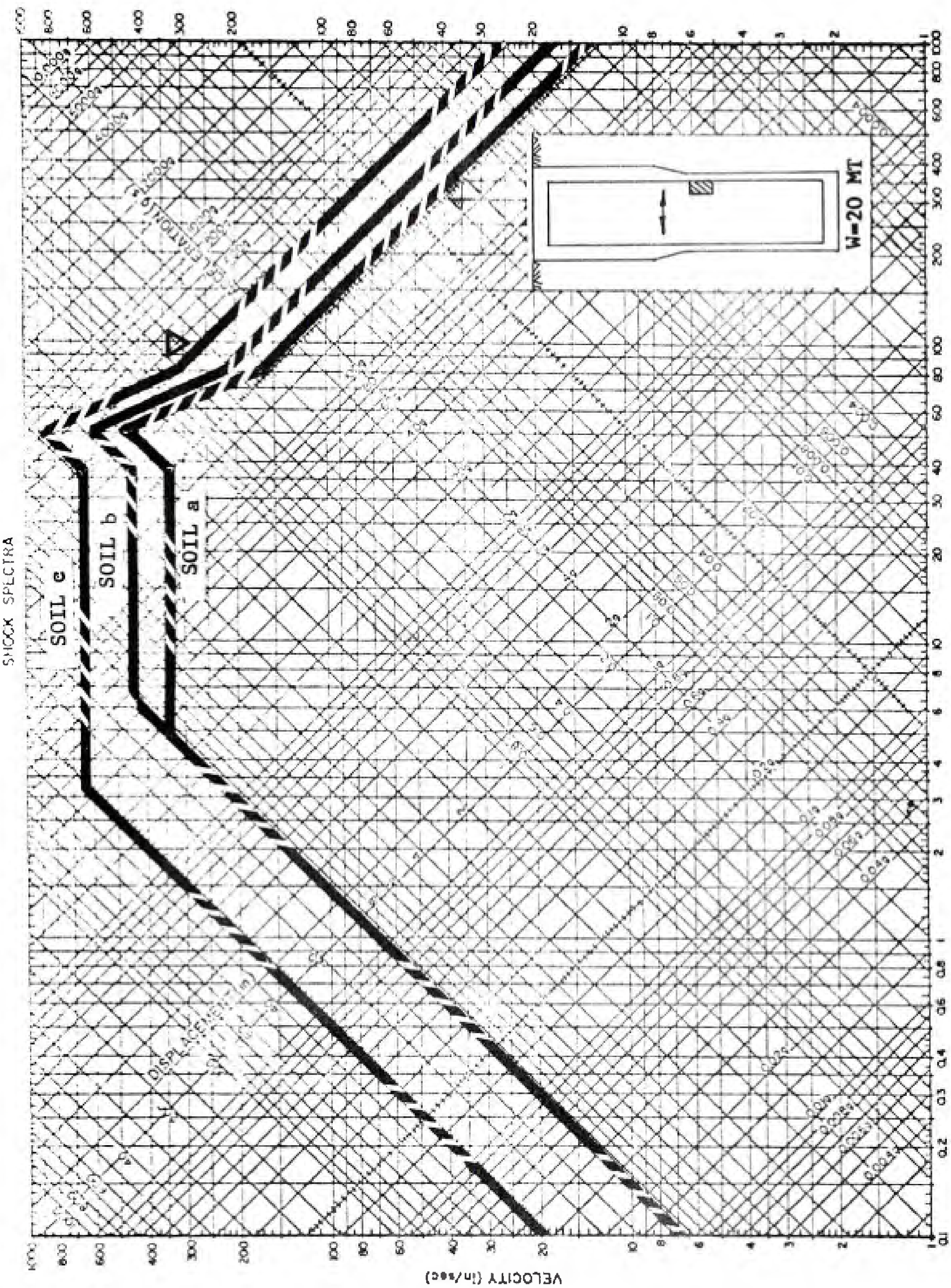

SHOCK SPECTRA
SOIL a
SOIL b
SOIL c
W-20 MT
DISPLACEMENT
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.45

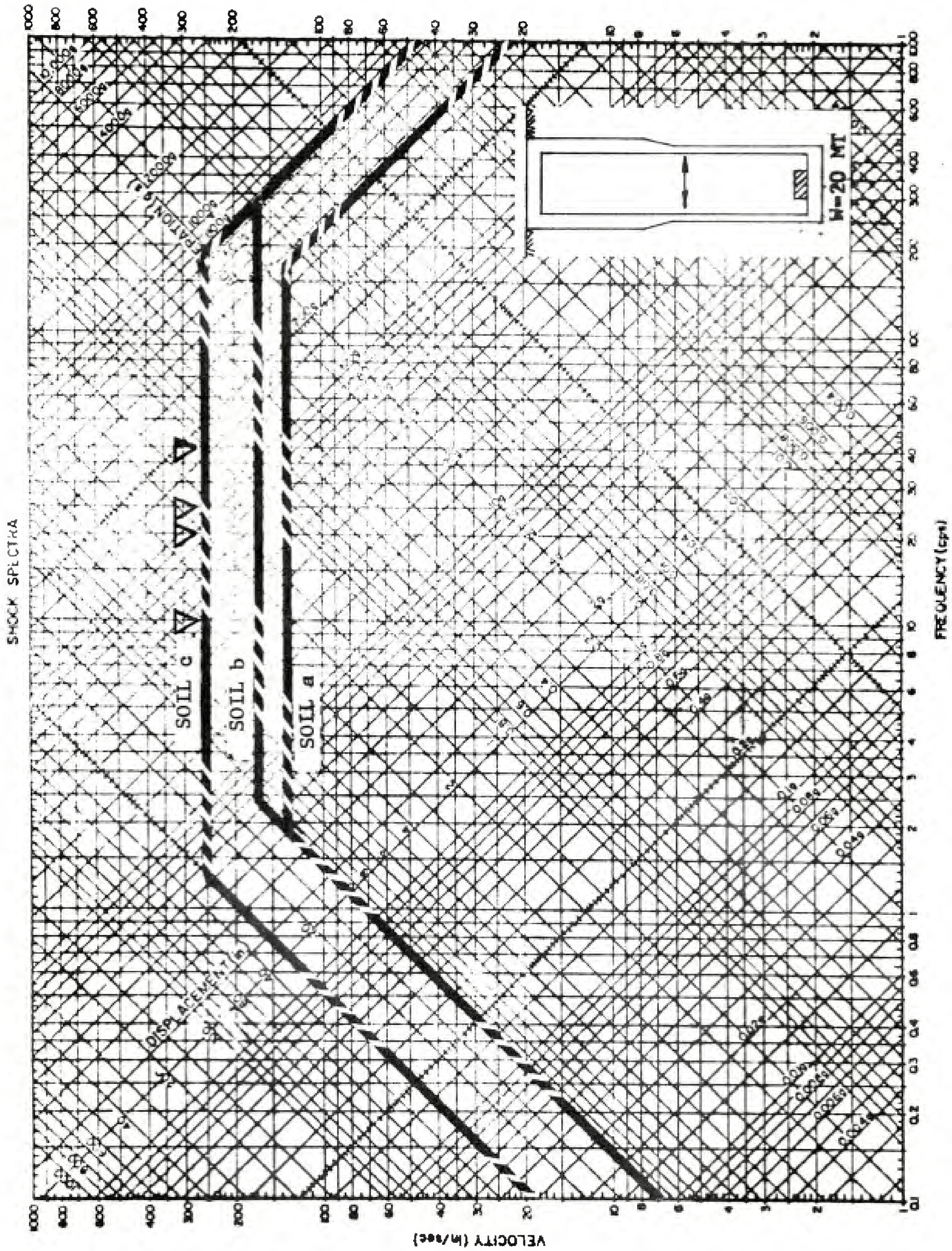

SHOCK SPECTRA
SOIL c
SOIL b
SOIL a
W-20 MT
FREQUENCY (cps)
VELOCITY (in/sec)

FIGURE 9.46

TABLE 9.3a EQUIPMENT RESPONSE DISTRIBUTION

VERTICAL SHOCK

(Note: The numbers on each bar refer to the response for the equipment in the specified structure, 1-6)

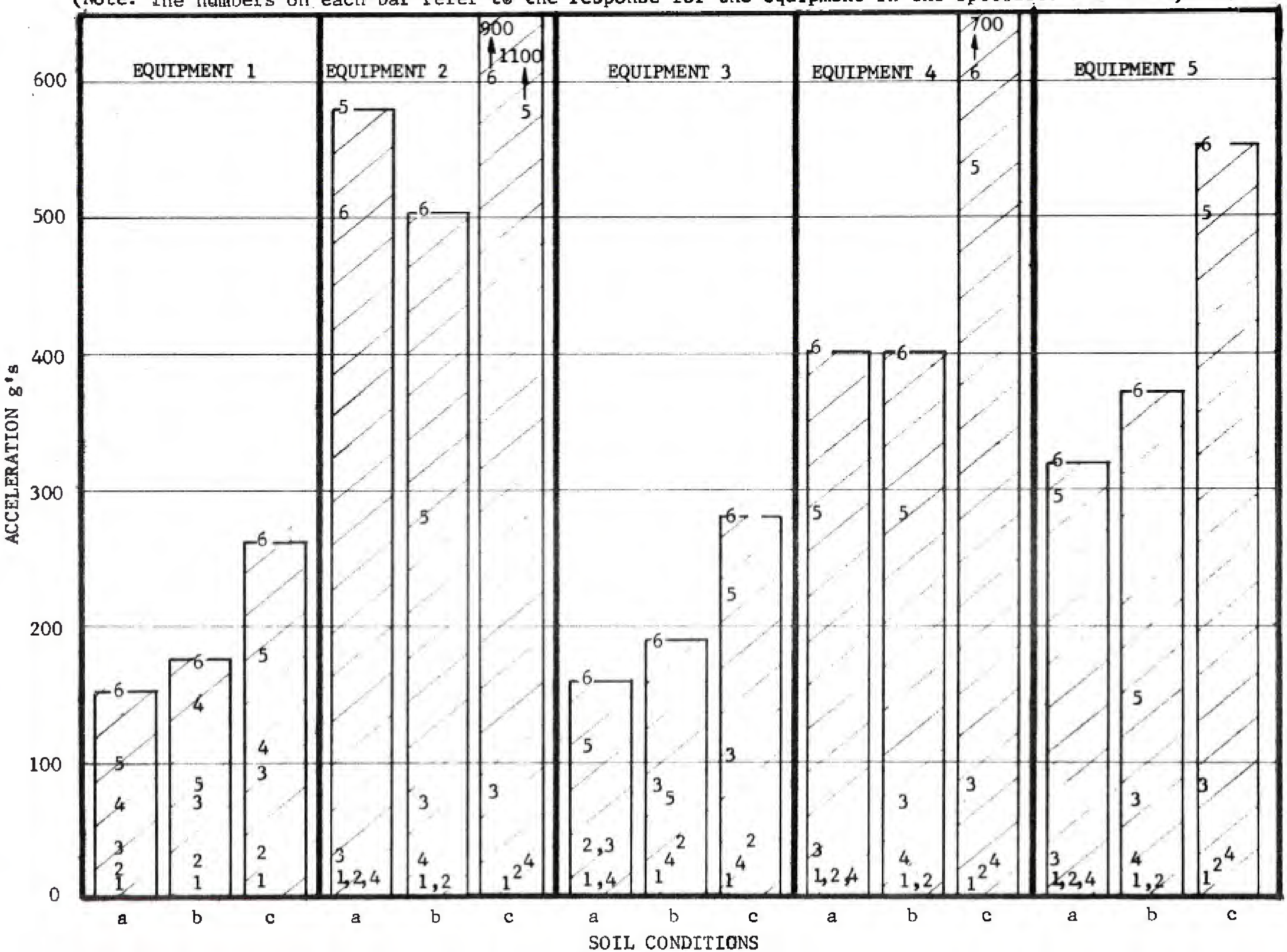
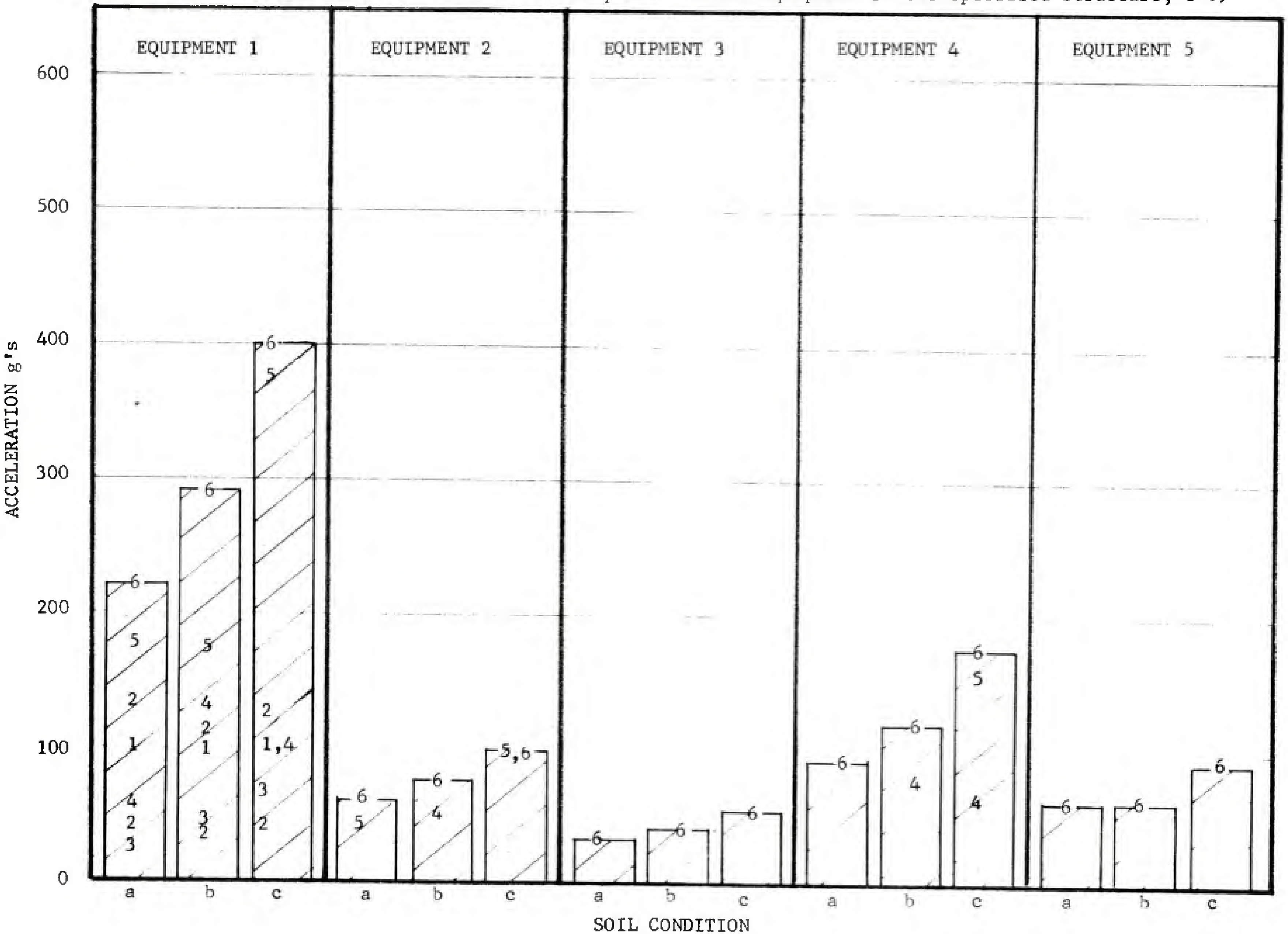


TABLE 9.3b EQUIPMENT RESPONSE DISTRIBUTION

HORIZONTAL SHOCK

(Note: The numbers on each bar refer to the response for the equipment in the specified structure, 1-6)



10.0 FOLLOW-UP ANALYSIS

The study reported here may be described as a step-wise procedure in which the response of the structure interposed between the free-field and the equipment is determined separately and is then applied to the equipment mounting as the modified or transmitted input. It will be more rational from a physical point of view, to analyze the soil-structure-equipment complex as a total system responding to groundshock. A satisfactory procedure would be to specify an isolated region around the soil-structure complex and to assume that any input motions which occur at the soil boundaries some distance away from the structure are not affected by the presence of the structure. The area of study for the most general case is illustrated in Figure 10.1 where ground motions, structure-soil interactions and direct air blast effects are all considered.

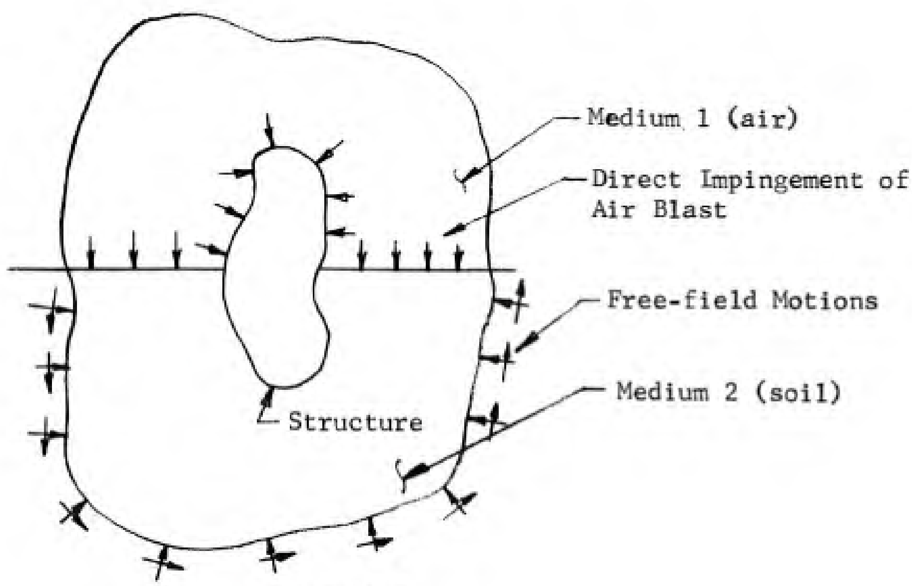


FIGURE 10.1

The structure will be capable of deforming into various shapes depending on the finite number of loads and soil interactions at the interface and the lumped configuration of the structure. For practical purposes only those deformations need be considered which will either induce significant stresses in the structure or provide an unfavorable environment for equipments attached internally to the structure. In effect, this limits the distortions in the structure to a finite number of modes bounded by the natural frequencies of the structure-soil combination which are capable of producing significant responses in the equipment. Most equipments, even highly fragile ones, are assumed to be unresponsive at frequencies beyond 2 KC. A more practical limit might be a 500 to 600 cps upper limit.

The stress-strain relationships for many soils are markedly non-linear. In layered media the propagated waves may be reflected and refracted a number of times so that the motion at any point in the soil may be quite complex. The result is a situation which is not amenable to investigation by the classical continuous media method of analysis.

As complex as the soil mechanics and structure-soil interactions are, the motion in the isolated region can be investigated to any degree of accuracy desired by the proper definition of the soil and structural properties and by considering the entire region to be composed of a matrix grid of mass particles and resilient and energy absorbing interconnections. The particle density in the grid should be sufficient to insure proper response of all modes of deformation in the aforementioned frequency band.

A typical soil-structure matrix is illustrated in Figure 10.2 where each square indicates a parcel of mass and the interconnections signify extensional and shear spring and energy absorbing elements. Note that initially, only a two dimensional case is considered; this is probably sufficient since the shock motion can be considered as unidirectional.

Since in Figure 10.2 there are 24 discrete masses representing the soil and 8 representing the structure there will be 32 coupled second order differential equations to describe the motion of the soil-structure complex in terms of the free-field ground motions and the overpressure and dynamic pressure.

It should be noted that each individual soil-structure problem is unique in regard to the grid arrangement. However, the equations of motion will always be of second order so that one problem resembles another mathematically. With high speed computing techniques available, the difference between problems manifests itself principally as a difference in computing times.

It is important here to note that such an analysis will give the response of the equipment and the structure in the form of time-dependent functions, and the qualitative waveforms described in this report can be supplemented or improved by obtaining a quantitative time-history of the response motions of the structure elements. The proposed method of analysis is especially useful when buried structures extend into different layers of the soil or otherwise when soil motions are very sensitive to the distance below grade.

Analytical studies are underway to define the ground motions in a more realistic manner so that more definitive soil motions around buried structures will be available. Analytical models of the soil (a 2-dimensional half space with non-linear and hysteretic characteristics) have been developed and computer programs have been written to determine the soil motions for a time dependent point load and moving load at the surface (see Reference 10.1). Any meaningful results from these studies can be used in helping to define the free-field motion at the model boundary for the proposed follow-up analysis.

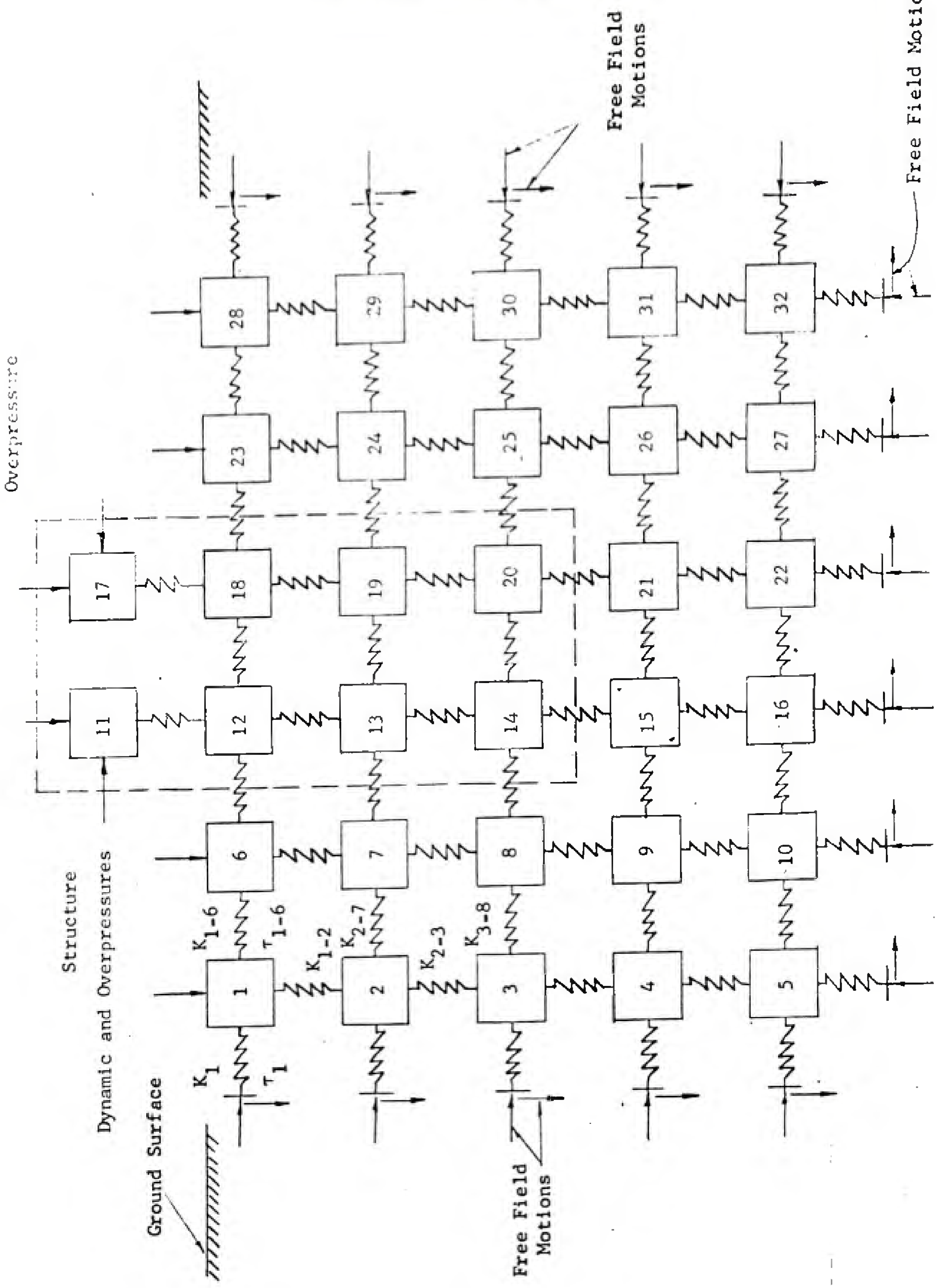


FIGURE 10.2 TWO DIMENSIONAL STRUCTURE RESPONSE MODEL

REFERENCES

1.1 "Nuclear Geoplosics" DASA-1285(IV), For the Defense Atomic Support Agency, under Contract Nos. DA-49-146-xz-030 and DA-49-146-xz-121, May 1964

7.1 "Principles and Practices for Design of Hardened Structures", Air Force Design Manual, T. D. AFSWC-TDR-62-138, December 1962

8.1 "Design Manual AEC Test Structures" December 1961, Volume II, Structural Response Characteristics Under Dynamic Loads

9.1 "Computations of Free-Field Motion and Shock Spectra for Linear and Non-Linear Systems", For Ralph M. Parsons Company under Subcontract 7-SC-3362-12 by N. M. Newmark, 15 July 1964 (SECRET)

9.2 "Design Manual AEC Test Structures", Volume III, Design of Blast Resistant Structures, December 1961

9.3 "Dynamics of Package Cushioning", R. D. Mindlin, Bell Telephone Laboratories

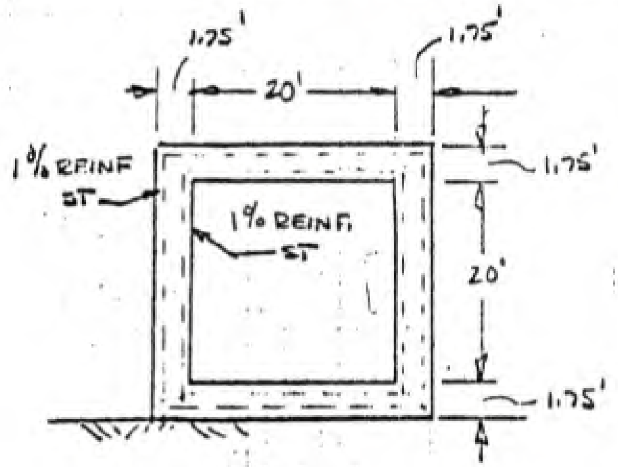
10.1 "Model for Wave Motions in Axi-Symmetric Solids", A.H.-S, ANG and J. H. Rainier, Journal of the Engineering Mechanics Division, ASCE Volume 90, No. EM2, April 1964

APPENDIX A

Calculations of Air Blast Motions

AGBABIAN-JACOBSEN ASSOCIATES ENGINEERING CONSULTANTS	SUBJECT Navy – Ground Motion Study		SHEET 1 OF
	JOB NO. 6524	BY JAM	DATE 10/4/65

DESIGN OF ABOVEGROUND RECTANGULAR STRUCTURE.



DEPTH 60 FT.

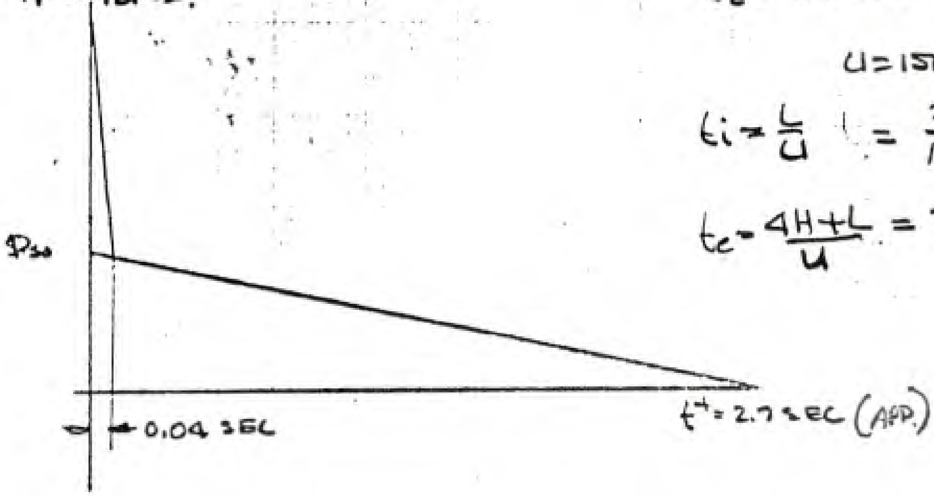
$H = 21.75'$

$L = 23.5'$

$B = 63.5'$

$P_{so} = 15$ PSI

$P_r = 42$ PSI.



$t_c = 0.04$ SEC

$U = 1500$ FPS.

$t_i = \frac{L}{U} = \frac{23.5}{1500} = 0.0157$ SEC

$t_c = \frac{4H+L}{U} = \frac{4 \times 21.75 + 23.5}{1500} = 0.074$ SEC

COMPUTE WEIGHT OF STRUCTURE

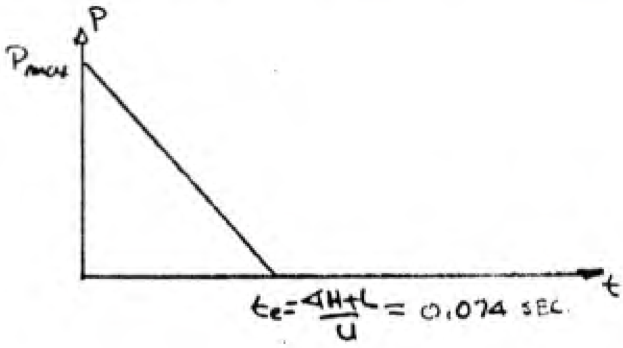
$$W = \left[2 \times 1.75 \times 23.5 \times 63.5 + 2 \times 20 \times 1.75 \times 63.5 + 2 \times 20 \times 20 \times 1.75\right] 150 \frac{LB}{FT^3}$$

$$W = 150\left[5220 + 4440 + 1400\right] = 1.66 \times 10^6 \text{ LB.}$$

ASSUME A CONCRETE TO SOIL COEFFICENT OF FRICTION

$C_f = 0.5$

THE EQUIVALENT TRIANGULAR PULSE IS



COMPUTE RIGID BODY MOTIONS IN HORIZONTAL AXIS

$$P_{max} = 144 P_r BH = 144 \frac{IN^2}{FT^2} \times 42 \frac{LB}{IN^2} \times 63.5\ FT \times 21.75\ FT = \underline{8.35 \times 10^6\ LB}$$

$$R_i = W C_f = 1.66 \times 10^6 \times 0.5 = \underline{8.3 \times 10^5\ LB}$$

$$R_{max} = \left[W + 144 P_{so} BL\right] C_f = \left[1.66 \times 10^6 + 144 \times 15 \times 63.5 \times 23.5\right] 0.5$$

$$= \left[1.66 \times 10^6 + 3.22 \times 10^6\right] 0.5 = \frac{4.88 \times 10^6}{2} = \underline{2.44 \times 10^6\ LB} \quad t > 0.0157\ SEC$$

$$\frac{P_{max}}{m} = \frac{8.35 \times 10^6}{1.66 \times 10^6} g = \underline{5.03\ g}$$

$$R_i/m = \frac{8.3 \times 10^5}{1.66 \times 10^6} g = \underline{0.5 g}$$

$$R_{max}/m = \frac{2.44 \times 10^6}{1.66 \times 10^6} = \underline{1.47 g}$$

THE PEAK HORIZONTAL ACCELERATION IS

$$a_h^1 = 5.03 - 0.5 = \underline{4.53\ g}$$

CONSTRUCT THE ACCELERATION DIAGRAM.

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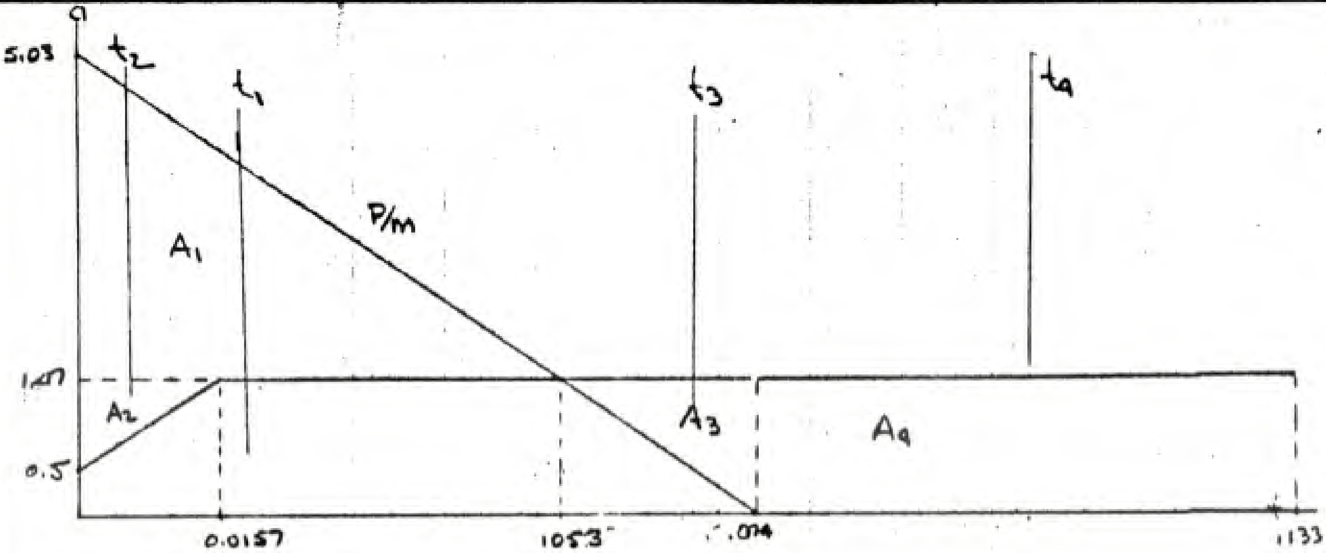


$$A_1 = \frac{0.053 \times (\overset{3.56}{5.03 - 1.07})}{2} = 0.0944 \text{ g SEC}$$

$$A_2 = \frac{0.0157 \times (\overset{.97}{1.47 - 0.5})}{2} = 0.0076 \text{ g SEC}$$

$$A_3 = \frac{1.47 \times (\overset{.021}{.074 - 0.053})}{2} = 0.0154 \text{ g SEC}$$

$$A_4 = 1.47 \times (t_d - 0.074) - A_1 + A_2 - A_3 = 0.0866$$

$$t_d = \frac{0.0866}{1.47} + 0.074 = 0.059 + 0.074 = 0.133 \text{ SEC}$$

Also

$$t_1 = \frac{.053}{3} = 0.0177 \text{ SEC}$$

$$t_2 - \frac{0.0157}{3} = .00523 \text{ SEC}$$

$$t_3 = 0.074 - \frac{0.074 - 0.053}{3} = 0.074 - .007 = 0.067 \text{ SEC}$$

$$t_4 = 0.074 + \frac{0.133 - 0.074}{2} = 0.074 + 0.030 = 0.104 \text{ SEC.}$$

THE PEAK HORIZONTAL VELOCITY IS

$$V_h' = A_1 + A_2 = 0.102\ g\,sec = 0.102 \times 386 = \underline{39.4\ in/sec}$$

$$A_1\tau_1 = 0.0944\,(\overbrace{0.133 - 0.0177}^{.1153}) = .0109\ g\,sec^2$$

$$A_2\tau_2 = 0.0076\,(\overbrace{0.133 - 0.00523}^{0.128}) = 0.000971\ g\,sec^2$$

$$A_3\tau_3 = 0.0154\,(\overbrace{0.133 - 0.067}^{.066}) = 0.001015\ g\,sec^2$$

$$A_4\tau_4 = 0.0866\,(\overbrace{0.133 - 0.104}^{.029}) = 0.00251\ g\,sec^2$$

$$d_h' = A_1\tau_1 + A_2\tau_2 - A_3\tau_3 - A_4\tau_4 = 0.0084\ g\,sec^2$$

$$d_h' = 0.0084 \times 386 = \underline{3.22\ in}$$

COMPUTE DEFORMATIONAL HORIZONTAL MOTION

$H' = 23.5$

CONSIDER DEFORMATION OF WALL $B = 63.5'$, $H = 21.75'$, $L = 23.5'$

$$W = 63.5 \times 23.5 \cdot 1.75 \times 150 = 39.2 \times 10^4\ LB$$

$$m_e = \alpha m = \frac{0.75 \times 39.2 \times 10^4}{g} = \frac{29.4 \times 10^4}{g}$$

$$a_h'' = \frac{P_{max}}{m_e} = \frac{8.35 \times 10^6}{29.4 \times 10^4}\,g = \underline{28.4\,g}$$

$$R = 0.075\,(\rho + \rho_s)\, f_y \frac{B\,d^2}{H'}$$

$$R = \frac{0.075 \times 2 \times 5.2 \times 10^4}{23.5} \times 63.5 \times (18)^2 = 6.82 \times 10^6\ LB$$

COMPUTE DEFORMATIONAL HORIZONTAL MOTION

$B=60$, $H=L=20$

$W = 60 \times 20 \times 1.75 \times 150 = 31.5 \times 10^4$ LB.

$$m_e = \alpha m = \frac{0.78 \times 31.5 \times 10^4}{g} = \frac{24.6 \times 10^4}{g}$$

$$a''_h = \frac{P_{max}}{m_e} = \frac{144 \times 42 \times 60 \times 20}{24.6 \times 10^4} g = \underline{29.6\,g}$$

COMPUTE TOTAL RESISTANCE AVAILABLE

$$R = \frac{0.075\,(\phi + \phi_s)\,f_y\,Bd^2}{H}$$

$$R = \frac{0.075 \times 2 \times 5.2 \times 10^4 \times 60 \times (18)^2}{20}$$

AND SPECIFIC RESISTANCE

$$R = \frac{0.075\,(\phi + \phi_s)\,f_y\,\cancel{B}d^2}{H^2\,\cancel{B}}$$

$$R = 0.075 \times 2 \times 5.2 \times 10^4 \times \left(\frac{1.5}{20}\right)^2 = 43.9 \text{ PSI}$$

COMPUTE THE PERIOD

$$T = \frac{1}{7200}\,\frac{H^2}{d\sqrt{Q}} = \frac{(20)^2}{7200 \times 1.5} = 0.037 \text{ SEC.} \qquad f = \frac{1}{T} = 27 \text{ CPS}$$

FROM ASCE No 42 PG 110-114 AND FROM PG 1 (HERE)

DEFINE

$P_1 = 27$ PSI $t_1 = 0.04$ SEC

$P_2 = 15$ PSI $t_2 = 2.7$ SEC

$\frac{t_1}{T} = \frac{0.04}{0.037}$ 1.08

$\frac{t_2}{T} = \frac{2.7}{0.037} =$ 73

Assume $\mu = 2$.

From AEC Design Man Vol II

$F_1 = 1.01$

$F_2 = 0.73$

Compute required resistance

$R_R = \frac{P_1}{F_1} + \frac{P_2}{F_2} = \frac{27}{1.01} + \frac{15}{0.73} = 26.7 + 20.5 = 47.2$ PSI

Try $\mu = 2.5$

$F_1 = 1.11$

$F_2 = 0.8$

$R_R = \frac{P_1}{F_1} + \frac{P_2}{F_2} = \frac{27}{1.11} + \frac{15}{0.8} = 24.3 + 18.8 = 43.1$ PSI

$\omega = \frac{2\pi}{T} = 170$ RAD/SEC

$d_y = \frac{1}{7600} \frac{H^2}{d} = \frac{(20)^2}{7600(1.5)} = 0.037$ FT $= 0.45$ in

$d''_h = \mu d_y = 2.5 \times 0.45 = \underline{1.11 \text{ in}}$

$V''_h = \frac{a''_h}{\omega} = \frac{29.6 \times 386}{170} = \underline{67.2 \text{ IN/SEC}}$

RIGID BODY VERTICAL MOTION

ASSUME $k = 100$ LB/IN³

$$K = 144 k B L = 144 \frac{IN^2}{FT^2} \times 100 \frac{LB}{IN^3} \times 63.5\,FT \times 23.5\,FT$$

$$K = 21.5 \times 10^6\ LB/IN$$

$$\omega_V = \sqrt{\frac{Kg}{W}} = \sqrt{\frac{21.5 \times 10^6 \times 386}{1.66 \times 10^6}} = \sqrt{5000} = 70.7\ RAD/SEC.$$

$$P_{MAX} = 144\, p_{so} B L = 144 \times 15 \times 63.5 \times 23.5 = 3.22 \times 10^6\ LB$$

$$a_V' = \frac{P_{MAX}}{m} = \frac{3.22 \times 10^6}{1.66 \times 10^6} g = \underline{1.94\,g}$$

$$V_V' = \frac{a_V'}{\omega} = \frac{1.94 \times 386}{70.7} = \underline{10.6\ IN/SEC}$$

$$d_V' = \frac{a_V'}{\omega^2} = \frac{10.6}{70.7} = \underline{0.15\ IN}$$

FLOOR DEFORMATIONS

$B = 60'$, $H = L = 20'$

$W = 60 \times 20 \times 1.75 \times 150 = 31.5 \times 10^4$ lb.

$$M_e = \delta m = \frac{24.6 \times 10^4}{g}$$

$$a_V'' = \frac{P_{MAX}}{M_e} = \frac{144 \times 15 \times 60 \times 20}{24.6 \times 10^4} g = \underline{10.5\,g.}$$

$$R = \frac{0.075(\Phi + \Phi_s) f_y d^2}{L^2} = 43.9\ PSI.$$

$$T = \frac{L^2}{7200\, d\sqrt{\Phi}} = 0.037\ SEC.$$

$P_1 = 15$ PSI $\quad t_1 = 2.7$ SEC

$$\frac{t_1}{T} = \frac{2.7}{0.037} = 73$$

ASSUME $\mu = 2$

$F_1 = 0.73$

$$R_R = \frac{P_1}{F_1} = \frac{15}{0.73} = 20.6 \text{ PSI}$$

ASSUME $\mu = 1$

$F_1 = 0.5$

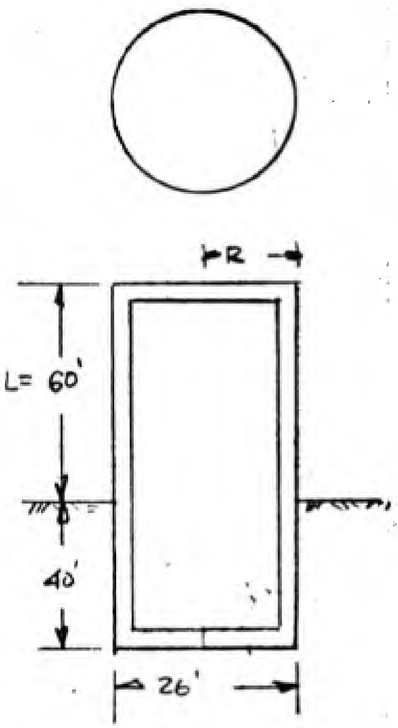
$$R_R = \frac{P_1}{F_1} = \frac{15}{0.5} = 30 \text{ PSI}$$

STRUCTURE REMAINS IN ELASTIC REGION

$$V_V'' = \frac{a_V''}{\omega} = \frac{10.5 \times 386}{170} = 23.9 \text{ IN/SEC}$$

$$d_V'' = \frac{a_V''}{\omega^2} = \frac{23.9}{170} = 0.14 \text{ in}$$

Partially Buried Structure

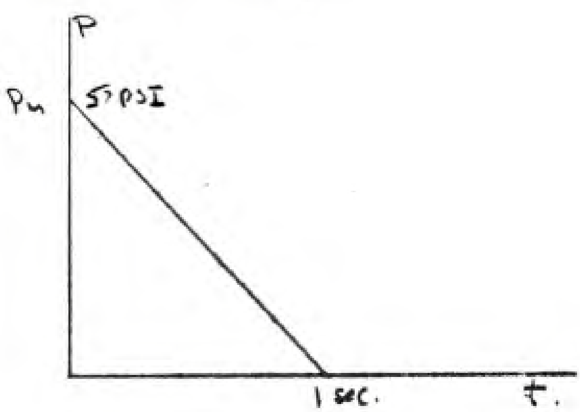


Rigid Body Motions

Identical to ground motion

Horizontal Deformation

1. Dynamic Drag.



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$$W = \pi D_s t_s L \gamma_s + \pi D_c t_c L \gamma_c$$

$$W = \pi \left[26 \times 12 - 2\right] 2 \times 60 \times 12 \times 0.283 + \pi \left[26 \times 12 - 22\right] 18 \times 60 \times 12 \times \frac{130}{1728}$$

$$W = \pi \times 310 \text{ IN} \times 2 \text{ IN} \times 60 \times 12 \text{ IN} \times 0.283 \frac{\text{LB}}{\text{IN}^3} + \pi \times 290 \text{ IN} \times 18 \text{ IN} \times 60 \times 12 \text{ IN} \times \frac{130}{1728} \frac{\text{LB}}{\text{IN}^3}$$

$$W = 3.96 \times 10^5 + 8.88 \times 10^5 = \underline{12.84 \times 10^5} \text{ LB.}$$

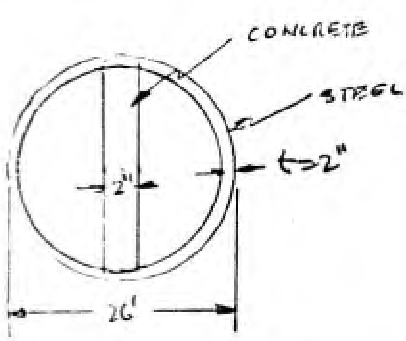
$$P_{max} = 144 \times 2R \times L \times P_m.$$

$$= 144 \times 26 \times 60 \times 52 = 11.7 \times 10^6 \text{ LB.}$$

THE SPECIFIC RESISTANCE TO CAUSE YIELDING IS, FOUND FROM THE FOLLOWING.

ASSUME $E_s/E_c = 15$

THEN THE TRANSFORMED SECTION IS APPROXIMATELY



$$I = \pi R^3 t + \frac{b 8 R^3}{12} = (13 \times 12)^3 \left[\pi \times 2 + \frac{16}{12}\right]$$

$$I = 38 \times 10^6 \times 7.61 = 29 \times 10^6 \text{ IN}^4$$

$$\sigma = \frac{Mc}{I}$$

$$M_{CANT} = \frac{1}{2} R L^2$$

$$\sigma_y = 5.2 \times 10^4 \text{ PSI}$$

$$\therefore \quad \sigma = \frac{rRL^2R}{I} = \frac{rR^2L^3}{I}$$

$$r = \frac{\sigma I}{R^2L^2} = \frac{5.2\times10^4 \times 29\times10^6}{(13\times12)^2\times(60\times12)^2} = \underline{119\ \text{PSI}}$$

COMPUTE THE FREQUENCY OF A CANTILEVER.

$$\omega = 3.52\sqrt{\frac{EI}{ML^4}}$$

$$\omega = 3.52\sqrt{\frac{30\times10^6 \times 29\times10^5 \times 386}{12.84\times10^5\times(60\times12)^3}} = 3.52\sqrt{702}^{\,26.5}$$

$\omega = 93.2$ RAD/SEC

$f = 14.8$ CPS.

$$T = \frac{1}{14.8} = 0.068 \text{ SEC}$$

FIND EQUIVALENT MASS

$$\delta_m = \frac{wL^4}{8EI} = \frac{wL\,L^3}{8EI} = \frac{PL^3}{8EI} \quad \text{in}$$

$$K_b = \frac{P}{\delta} = \frac{P\,8EI}{PL^3} = \frac{8EI}{L^3}$$

$$\omega = \sqrt{\frac{K_b}{K_{LM}M_b}} = \sqrt{\frac{8EI}{L^3K_{LM}M_b}} = 3.52\sqrt{\frac{EI}{M_bL^3}} \quad 1/\text{sec.}$$

$$\frac{8\cancel{EI}}{\cancel{L^3}K_{LM}\cancel{M_b}} = 11.05\frac{\cancel{EI}}{\cancel{M_bL^3}}$$

$$11.05\,K_{LM} = 8$$

$$K_{LM} = .725$$

$$M_e = \frac{0.725\times12.84\times10^5}{386} = 2410\ \frac{\text{LB SEC}^2}{\text{IN}}$$

$$a_n'' = \frac{P_{max}}{M_e} = \frac{11.7 \times 10^6}{12.84 \times 10^5 \times 0.725}\, g = \underline{12.6\, g}$$

$$t_1/T = 1/0.068 = 14.8$$

Assume $\mu = 4$

$$F_1 = 0.91$$

$$R_R = \frac{P_1}{F_1} = \frac{52}{0.91} = 57 \text{ PSI}$$

Assume $\mu = 1.5$

$$F_1 = 0.65$$

$$R_R = \frac{52}{0.65} = 80 \text{ PSI}$$

Assume $\mu = 1$

System is elastic

$$V_n'' = \frac{a_n''}{\omega} = \frac{12.6 \times 386}{93.2} = \underline{52.1 \text{ IN/SEC}}$$

$$d_n'' = \frac{52.1}{93.2} = \underline{0.56 \text{ in}}$$

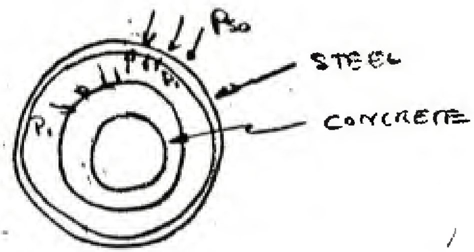
2. Cylindrical Breathing

ASSUME THAT THE OVER PRESSURE ACTS ON THE ENTIRE OUTER SURFACE OF THE CYLINDER.

$W = 12.84 \times 10^5$ LB

$P_{max} = 144 \times \pi \times 26 \times 60 \times 50 = 3.52 \times 10^7$ LB

THE RESISTANCE AT YIELD IS FOUND FROM.



IN THE STEEL

$$\sigma_{s\,MERID} = \frac{(P_{so} - P_1) R_s}{2 t_s}$$

$$\sigma_{s\,HOOP} = \frac{(P_{so} - P_1) R_s}{t_s}$$

$$\delta_s = \frac{R_s}{E_s}\left(\sigma_{sH} - \mu \sigma_{sM}\right)$$

IN THE CONCRETE

$$\sigma_{c\,MERID} = \frac{P_1 R_c}{2 t_c}$$

$$\sigma_{c\,HOOP} = \frac{P_1 R_c}{t_c}$$

$$\delta_c = \frac{R_c}{E_c}\left(\sigma_{cH} - \mu \sigma_{cM}\right)$$

But $\delta_s = \delta_c$

$$\frac{R_s}{E_s}\left[\frac{(P_{so}-P_1)R_s}{t_s} - \mu_s \frac{(P_{so}-P_1)R_s}{2t_s}\right] = \frac{R_c}{E_c}\left[\frac{P_1 R_c}{t_c} - \mu_c \frac{P_1 R_c}{2t_c}\right]$$

$$\frac{R_s^2}{E_s t_s}(P_{so} - P_1)\left[1 - \frac{\mu_s}{2}\right] = \frac{R_c^2 P_1}{E_c t_c}\left[1 - \frac{\mu_c}{2}\right]$$

$$P_1\left\{\frac{R_c^2}{E_c t_c}\left(1 - \frac{\mu_c}{2}\right) + \frac{R_s^2}{E_s t_s}\left(1 - \frac{\mu_s}{2}\right)\right\} = \frac{R_s^2}{E_s t_s}\left(1 - \frac{\mu_s}{2}\right) P_{so}$$

THE ELASTIC BUCKLING IS FOUND FROM.

$$P_{scr} = (P_{so} - P_1)_{scr} = 0.807\ \frac{E_s t_s^2}{L R_s} \sqrt[4]{\left(\frac{1}{1-\mu_s^2}\right)^3 \frac{t_s^2}{R_s^2}}$$

$$P_{ccr} = P_{icr} = 0.807\ \frac{E_c t_c^2}{L R_c} \sqrt[4]{\left(\frac{1}{1-\mu_c^2}\right)^3 \frac{t_c^2}{R_c^2}}$$

$E_s = 30 \times 10^6$ | $E_c = 2 \times 10^6$
$R_s = 13 \times 12 - 1 = 155$ | $R_c = 13 \times 12 - 11 = 144$
$t_s = 2$ | $t_c = 18$
$\mu_s = 0.3$ | $\mu_c = .25$

$L = 100 \times 12$

$$P_1 \left\{ \frac{2.07 \times 10^4}{2 \times 10^6 \times 18} \left(1 - 0.1\right)^{.9} + \frac{2.04 \times 10^4}{30 \times 10^6 \times 2} \left(1 - .15\right)^{.85} \right\} = \frac{2.04 \times 10^4}{30 \times 10^6 \times 2} \left(1 - .15\right) P_{so}$$

$$P_1 \left\{ 5.17 \times 10^{-4} + 3.9 \times 10^{-4} \right\} = 3.4 \times 10^{-4} P_{so}$$

$$P_1 = \frac{3.4}{8.57} P_{so} = \underline{0.396\ P_{so}} \quad \text{IN THE ELASTIC REGION}$$

$$P_{icr} = \frac{0.807 \times 2 \times 10^6 \times (18)^2}{100 \times 12 \times 144} \left\{ \left(\frac{1}{1-.06}\right)^3 \frac{(18)^2}{(144)^2} \right\}^{1/4}$$

$$P_{icr} = 3.03 \times 10^3 \left[1.2 \times 1.57 \times 10^{-2} \right]^{1/4} = \underline{3.03 \times 10^3} \left(.0188\right)^{1/4}_{.37} = 1120\ \text{PSI}$$

$$(P_{so} - P_1)_{scr} = \frac{0.807 \times 30 \times 10^6 \times 4}{100 \times 12 \times 155} \left\{ \left(\frac{1}{1-.09}\right)^3_{1.13} \left(\frac{2}{155}\right)^2 \right\}^{1/4}$$

$$= 520 \left[1.33 \times 1.65 \times 10^{-4} \right]^{1/4}_{2.19}$$

$$= 520 \times 1.21 \times 10^{-1} = \underline{63\ \text{PSI}}$$

$$P_{socr} = 63 + 1120 = \underline{1183\ \text{PSI}} \quad \text{in buckling}$$

FOR CONCRETE $\sigma_{c\,yield} \cong 3000$ PSI

$\sigma_{c\,shear} \cong 600$ PSI

HENCE $\sigma_{c\,hoop} = 2400$ PSI

$$\frac{P_1 R_c}{t_c} = 2400$$

$$P_1 = 2400 \frac{t_c}{R_c} = \frac{2400 \times 18}{144} = 300 \text{ PSI}$$

FOR STEEL $\sigma_{s\,yield} = \frac{(P_{so} - P_1) R_s}{t_s} = \frac{(P_{so} - P_1)\, 155}{2} = 5.2 \times 10^4$

$$(P_{so} - P_1) = \frac{2 \times 5.2 \times 10^4}{155} = \underline{670 \text{ PSI}}$$

HENCE $P_1 \leq 300$ PSI

$P_1 = 0.396\, P_{so}$

$\therefore P_{so} = \frac{300}{0.396} = 750$ PSI

THEREFORE, WHEN $P_{so} = 750$ PSI $P_1 = 300$ PSI

P_{1cr} IS NOT REACHED

$(P_{so} - P_1)_{scr} = 63$ PSI

$(750 - 300) = 450 \text{ PSI} \leq 670$ PSI STEEL CANNOT BUCKLE BECAUSE CONCRETE DOES NOT FAIL.

HENCE $r = 750$ PSI

THE FREQUENCY OF THE CYLINDER IS

$$\omega = \sqrt{\frac{Eg}{\gamma R^2}}$$

THE STATIC DEFLECTION DUE TO THE EXTERNAL LOAD P_{so} IS

$$\delta = \frac{R_c^2 P_1}{E_c t_c}\left[1 - \frac{\mu_c}{2}\right]$$

where $P_1 = 0.396 P_{so}$

$$\delta = \frac{R_c^2 (0.396) P_{so}}{E_c t_c}\left[\frac{2-\mu_c}{2}\right]$$

THE TOTAL LOAD IS $P = 2\pi R L P_{so}$; $P_{so} = \frac{P}{2\pi R L}$

$$\delta = \frac{R_c^2 (0.396) P (2-\mu_c)}{4 E_c t_c \pi R L}$$

$$\frac{P}{\delta} = \frac{4 E_c t_c L \pi R}{R_c^2 (0.396)(2-\mu_c)} = K$$

$$\omega = \sqrt{\frac{K}{m_e}} = \sqrt{\frac{Eg}{\gamma R^2}}$$

$$\omega_{STEEL} = \sqrt{\frac{30 \times 10^6 \times 386}{0.283 \times (155)^2}} = \sqrt{1770 \times 10^3} = \sqrt{1.77 \times 10^6} = 1330 \ \frac{RAD}{SEC}$$

$$\omega_{CON} = \sqrt{\frac{2 \times 10^6 \times 1728 \times 386}{130 (144)^2}} = \sqrt{494 \times 10^3} = 703 \ RAD/SEC$$

SINCE THE TWO CYLINDERS ARE ABOUT EQUALLY STIFF ASSUME AN AVERAGE FREQUENCY.

$$\sqrt{\frac{K}{m_e}} = 1000$$

$$m_e = \frac{K}{10^6} = \frac{4E_c t_c \pi R l}{R_c^2 (0.396)(2-\mu_c)\, 10^6} = \frac{4E_c t_c \pi l}{R_c (0.396)(2-\mu_c)\, 10^6}$$

$$m_e = \frac{4 \times 2 \times 10^6 \times 18 \times \pi \times 60 \times 12}{13 \times 12 \times 0.396 \times 1.75 \times 10^6} = 3000$$

$$K_{Lm} = \frac{m_e}{m} = \frac{3 \times 10^3 \times 386}{1289 \times 10^5} = 0.9$$

$$a_h'' = \frac{P_{max}}{m_e \times g} = \frac{3.52 \times 10^7}{3 \times 10^3 \times 386} = 32g$$

$$V_h'' = \frac{a_h'' g}{\omega} = \frac{32 \times 386}{1000} = 12.4 \text{ in/sec}$$

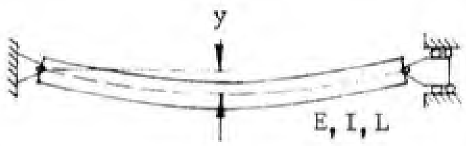
$$d_h'' = \frac{V_h''}{\omega} = 0.0124 \text{ in.}$$

APPENDIX B

Derivation of Velocity Bound

APPENDIX B

Consider a pin ended beam excited by a transverse shock or vibration in the first mode where the beam remains in the elastic region



w = weight of beam per unit length

Natural circular frequency $= \omega_n = 9.86 \sqrt{\frac{gEI}{wL^4}}$ (1)

Static deflection at center $= y = \frac{5wL^4}{384\ EI}$ (2)

Moment at center $= M = \frac{wL^2}{8}$ (3)

Stress at center $= \sigma = \frac{Mc}{I}$ (4)

Combining (2), (3) and (4)

$$y = \frac{5}{48} \frac{\sigma L^2}{Ec}$$

Then the maximum velocity at the center of the beam is

$$\dot{y} = \omega_n y = 0.587 \sqrt{\frac{gI\sigma^2}{w\ Ec^2}}$$

If the beam for example is rectangular

$$I = \frac{bh^3}{12}$$

$$w = \rho bh$$

$$c = \frac{h}{2}$$

ρ = density of beam

So that

$$\dot{y} = 0.339 \sqrt{\frac{g\sigma^2}{\rho E}}$$

Therefore, the velocity of the beam is dependent only on material properties and other constants and is independent of physical properties.

Taking for example high strength aluminum alloy to be used in a shock environment where yield stress is applicable as a definition of failure criterion

$$\sigma_y = 40{,}000 \text{ psi}$$

$$\rho = 0.1 \text{ lb/in}^3$$

$$E = 10 \times 10^6 \text{ psi}$$

Hence, at yield

$$\dot{y} = 266 \text{ ips}$$

For structural steels

$$\sigma_y = 36{,}000 \text{ psi}$$

$$\rho = 0.283 \text{ lb/in}^3$$

$$E = 30 \times 10^6 \text{ psi}$$

and, at yield

$$\dot{y} = 82.5 \text{ ips}$$

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13. ABSTRACT

This study was performed in order to obtain criteria for the design of a testing machine that will simulate ground shock. Approximate methods were used in defining the motions encountered during ground shock in representative soil profiles, and the effect of various structures interposed between the free-field and equipment was estimated. Shock spectra were then obtained for typical equipment located in various types of structures placed in the assumed soil conditions. In this manner, a range of values was obtained that defined approximately the shock input and the shock response of equipment, which, it is believed, bounds the shock conditions for specified environments given in the Work Statement.

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Free field ground motions						
Shock spectra for equipment inside structures						

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