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Project No. 70-197

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UNITED STATES GOVERNMENT
WASHINGTON, D.C.

INVESTIGATION OF PRESENT AND FUTURE VIBRATION ENVIRONMENT

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MAY 1971

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FIGURES (Phase II)

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INVESTIGATION OF PRESENT AND FUTURE VIBRATION ENVIRONMENT

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PREFACE

In January 1971, [redacted] STAT was awarded a contract to investigate the vibration environment [redacted] STAT [redacted] STAT as it affects operation of specialized equipment. The equipment is of a nature that present ambient vibration levels are limiting its performance capabilities. In some areas of the structure, operations have been completely disrupted and have had to be relocated to other portions of the building. The sources of the detrimental vibrations include mechanical equipment [redacted] STAT street traffic, railroad traffic, people walking in the building and normal activities such as opening and closing doors. To further complicate the environment, a new subway system is proposed immediately adjacent [redacted] STAT Therefore, in view of the known sensitivity of the present equipment to existing vibration levels, it appeared likely that the operations of the new subway may very well produce a vibration environment which would severely limit the operating capabilities of certain instruments.

Phase I of this investigation, which deals with ambient vibration conditions, is completed and summarized herein. The data and detailed descriptions of each group of measurements will be submitted in Appendix A, as a separate volume. Phase II, which deals with the effects of the subway, is essentially completed, except for a few additional computer runs which are considered important. These will be reported in Appendix B as a separate volume. Nevertheless, sufficient analysis has been completed to draw firm predictions regarding the vibration environment [redacted] STAT after the subway is completed.

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PHASE I

INVESTIGATION OF PRESENT AND FUTURE VIBRATION ENVIRONMENT

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[redacted]

PHASE I
INVESTIGATION OF PRESENT AMBIENT VIBRATION LEVELS

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INTRODUCTION

Phase I of the total investigation deals with the existing vibration environment [redacted] As mentioned in the Preface, [redacted] houses vibration-sensitive process equipment which is presently being disturbed by several sources of vibration. The objective of Phase I is to define accurately these sources and suggest modifications to the structure, auxiliary equipment or the process equipment itself to alleviate the situation and to provide a basis for assessing the effects of a proposed subway system which is treated in Phase II.

As an initial step in Phase I, a measurement program was conducted throughout [redacted] in the early part of 1971. Vibration measurements were taken at the locations of various types of sensitive equipment and at numerous vibration sources using velocity-type transducers, a storage oscilloscope, and an oscilloscope camera. The oscilloscope is capable of displaying two traces, which permits simultaneous comparison of amplitude and frequency at two locations. In most cases, the vibrations producing the greatest effects are usually at a predominant frequency which can be readily determined from the type of measurements obtained for this investigation. Therefore, a complete amplitude-frequency spectra was not necessary.

The subsequent sections of this report briefly discuss the main sources of disturbing vibrations, with comments regarding their probable effect on the process equipment, and the results of measurements on a particularly

[redacted] STAT

sensitive work bench. The details of vibration traces for all groups of measurements are contained in Appendix A.

SOURCES OF VIBRATION

Many sources of vibration exist within the [redacted] area, and no single piece of equipment or system can be considered as the major source. For the most part, the vibration levels are relatively low, but usually with a predominant frequency. Generally, it is possible to identify a single source of vibration only in a few instances. [redacted] immediately adjacent [redacted] houses the equipment for heating, ventilation, water and other utilities [redacted] This includes air-compressors, cooling towers and the air-handling units. The characteristics of the vibrations produced by each of these are summarized in the following paragraphs.

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Air-Handling Units

Vibration measurements were taken on the ducts and junctions as well as the housing of the fans that make up the air-handling units. The vibration measurements are most meaningful in terms of the frequency characteristics. The amplitudes of the motions are not of particular importance. This occurs because the vibrations produced in the duct work within [redacted] [redacted] are of an acoustical or high frequency nature and are transmitted by pressure fluctuations rather than mechanically. Mechanical transmissions were observed only in a few instances and these were for short distances along sheet metal ducts. The data shown below in Table I-I are a summary of vibrations measured in [redacted] related to air handling units.

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In many instances, the vibration frequencies are within the 15 to 25 Hz range which coincides with the natural frequency of the floor slabs

[redacted]

and is also within the range of the natural frequency of the work benches described below.

TABLE I-I
SUMMARY OF AIR-HANDLING UNIT VIBRATION DATA

Unit	Frequency Hz*	Amplitude Inches
Air Handling Unit No. 1	26.3	5.7×10^{-5}
	25.0	2.7×10^{-4}
	37.0	4.5×10^{-5}
Air Handling Unit No. 2	8.5	1.5×10^{-3}
	17.4	2.1×10^{-3}
	16.7	1.1×10^{-3}
	17.7	6.2×10^{-4}
	33.3	8.1×10^{-5}
	37.0	4.0×10^{-4}
Air Handling Unit No. 2E	12.1	1.6×10^{-3}
	72.2	6.9×10^{-4}
Air Handling Unit At South End [redacted]	21.1	2.1×10^{-4}
	28.6	1.6×10^{-3}
	33.3	1.4×10^{-4}

*Hz is the abbreviation for Hertz and has dimensions of cycles per second.

[redacted] STAT

Cooling Tower No. 1

Measurements were made on the circular frame at the top of the cooling tower to record the frequency of steady-state vibrations generated by the fan. Frequencies varied between 33 and 38 Hz and the amplitudes of motion were less than 10^{-3} inches. Steady vibrations at these frequencies were not observed at any location within [redacted] Thus, the cooling tower is dismissed as a significant source of vibration.

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Air Compressors

Two types of air compressors are located in the north end of [redacted] and run intermittently. The vibrations transmitted to the surrounding floor are summarized in Table I-II.

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TABLE I-II

SUMMARY OF AIR COMPRESSOR VIBRATION DATA

Item	Frequency Hz	Amplitude inches
DeVilbiss Compressors	10	1.1×10^{-3}
	20	1.2×10^{-3}
Worthington Compressors	8.7	3.2×10^{-4}
	18.2	1.1×10^{-4}

These vibrations are again within the natural frequencies of floor slabs and the work benches. However, if they were a significant contribution, it would be easy to correlate the periods of high vibration with the on cycle of the compressors since they produce a steady vibration as opposed to a random vibration. Vibrations of this nature were not observed at any location within

[redacted] STAT

Vibrations From Rock Crushers

The U. S. Geologic Survey occupies the south half of the sixth floor [redacted] where several rock crushers are housed. When in operation, one of two types of crushers generates vibrations through reciprocating action of a pair of jaws. Vibrations of 13 to 16 Hz at an amplitude of 1.3×10^{-4} inches were observed. The second type uses rollers rather than reciprocating action, and as a result, negligible vibrations are produced.

In some cases, larger pieces of rock must be broken with a sledge hammer so that they may be fed into the jaw crusher. The sledge hammer impact excited the natural frequency of the floor system (15 to 17 Hz) at a peak transient amplitude of approximately 4×10^{-5} inches. The natural frequency of the floor was determined by measuring the response caused by suddenly applying one's weight to his heels. This produced a peak amplitude of 6.8×10^{-4} inches at 14.7 cycles per second. The rock crusher and sledge hammer operations are definitely a problem for sensitive equipment located within several bays of the source. However, the periods of operation are relatively short and, if need be, a coordination of operations could possibly be worked out.

Vibrations From M-Street Traffic

Vibrations from traffic running along M-Street were measured at the ground surface approximately 30 feet west of the northwest corner of [redacted] A summary of the significant vibration levels recorded at this point are given in Table I-III.

[redacted] STAT

TABLE I-III

SUMMARY OF GROUND MOTION
RESULTING FROM M-STREET TRAFFIC

Description	Frequency Hz	Amplitude* Inches
Cars and Bus~~s~~es:		
Vertical	10.0	3.2×10^{-6}
"	11.8	5.4×10^{-6}
"	14.3	4.5×10^{-6}
Radial	12.9	2.5×10^{-5}
"	11.1	1.9×10^{-5}
"	11.1	4.1×10^{-5}
Trucks & Bus~~s~~es:		
Vertical	11.1	5.4×10^{-5}
"	11.8	4.1×10^{-5}
"	10.9	1.2×10^{-4}
Radial	11.0	1.5×10^{-5}
"	12.5	1.2×10^{-5}
"	11.0	2.6×10^{-5}

* Measured 30 feet west of the northwest corner [redacted] STAT

The significance of these vibrations is that the frequencies are predominantly 10 to 12 Hz which is typical of the frequency of motion that is most readily transmitted through the ground in the vicinity [redacted] STAT The amplitudes of motion are relatively low and occur at infrequent intervals--more in the form of an impact rather than a steady input since the vibrations are produced only when the vehicles are passing over a rough spot.

[redacted]

Vibrations From Railroad on First Street

A railroad siding to the west of [redacted] is intermittently used to switch cars located in the Navy Yard south [redacted] [redacted] During the passage of this train, ground vibrations are produced from impact as the train passes over joints on the rails. Ground vibrations were measured at the same location as for the traffic on M-Street. The vibrations produced by the train are listed in Table I-IV.

TABLE I-IV

SUMMARY OF GROUND MOTION
RESULTING FROM RAILROAD TRAIN

Description	Frequency Hz	Amplitude Inches
Vertical	11.1	6.2×10^{-5}
"	29	3.5×10^{-5}
Radial	25	8.1×10^{-6}
"	28	4.6×10^{-6}

These vibrations are similar to those caused by vehicles on M-Street except for some of the higher frequency contents.

Vibration Measurements of the Work Bench on the Fifth Floor

Measurements were made of the vibrations of the newest model work bench which was set up in an office on the fifth floor at the north end of the building. On this particular instrument, the problem arises from relative movement between the bench top and the mechanism cantilevered from the beam across the back of the instrument. Typical vibration records associated with this instrument are shown on Figs I-1, I-2 and I-3. Figure I-1 shows

[redacted]

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the vertical and horizontal motions of the floor supporting the work bench at the center of the bay between columns. Record No. 42 represents an impact from a light thump on the floor which produces free oscillations of the floor system at its natural frequency. The traces on Record No. 42 indicate that the floor frequency is approximately 15 cycles per second, and that it is easy for a person to produce a vibration which exceeds the ambient level by a factor of 4 or 5. Record No. 43 shows that the predominant ambient vibrations of motion are 18.5 to 20 Hz at an amplitude of 3.2×10^{-5} in the vertical direction and 4.5×10^{-6} in the horizontal direction.

Record No. 45 on Fig I-2 is a comparison of the vertical vibration of the floor with the vertical vibration of the glass on the work bench. Both traces are at the same scale and, thus, a direct comparison can be made. It is seen that the table vibrations are slightly greater and that both vibrations, of course, contain the same predominant frequencies.

Record No. 46 shows the vertical motion of the cantilevered instrument in comparison to the vertical motion of the work bench. The scale settings are the same for both traces, and therefore, the amplification of motion on the cantilevered instrument is readily seen. The frequency of the top trace represents the natural frequency of the instrument which is about 16 Hz. The amplitude of the motion of the instrument is 3.1×10^{-4} inches and represents an amplification of approximately 5 times the amplitude of the top of the work bench. Record No. 47 compares the horizontal motion of the cantilevered instrument with the light table. It is this movement which is causing the problem associated with using the instrument at its full capacity. The frequency of

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horizontal vibration of the cantilevered portion is 16.7 Hz at an amplitude of of 2.3×10^{-4} inches. The amplitude of horizontal vibration of the work bench is approximately one-tenth this value, and thus, the top trace is approximately equal to the relative motion between the cantilevered instrument and the light table. These results show that the natural frequencies of the instrument coincide with the natural frequency of the floor. Since the mass of the instrument is relatively small compared to the mass of the floor, high amplification factors are produced in the cantilevered system.

Ambient Floor Vibrations

Ambient floor vibrations were measured at many locations within [redacted] and the results are plotted in terms of peak displacement versus frequency on Fig I-4. Vibrations recorded during the passage of trucks, busses and a train are plotted on the same figure. The data are considered to represent a statistical collection since measurements were made over a wide range of locations within the structure. The floor vibrations show an increasing trend at higher floors with frequencies between 15 and 25 Hz. This frequency range corresponds to the natural frequencies of the floor [redacted]; [redacted] Ground vibrations predominantly occur between 10 and 12 Hz. To provide a physical reference for the amplitudes of motion, levels of human perception are indicated. It is common practice to assume that for an ordinary structure, vibrations below the limit of "barely noticeable to persons" represent a "vibration-free" environment.

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Vibrations from Ventilation Ducts

Vibration measurements were conducted on the fourth floor at the north end [redacted] to determine the amount of floor vibration contributed STAT

[redacted]

by the ventilation ducts. Measurements were taken on the floor with the air ventilating system on and off. A summary of the measurements is given in Table I-V.

TABLE I-V

COMPARISON OF VERTICAL FLOOR VIBRATIONS WITH VENTILATION SYSTEM ON AND OFF

Description	Frequency Hz	Amplitude Inches
Ventilation On	20.0	12.0×10^{-6}
	25.0	8.9×10^{-6}
	28.6	14.0×10^{-6}
Ventilation Off	20.0	11.0×10^{-6}
	23.8	8.4×10^{-6}
	26.7	6.0×10^{-6}

The conclusion to be drawn is that 100 percent effective corrective measures to the ventilating system would reduce the amount of vibration by less than 50 percent. From a practical standpoint, this would produce marginal improvements and only within the immediate vicinity of the ventilation duct. The reduction would probably be unnoticeable several bays from the ventilation duct.

CONCLUSIONS

For most areas [redacted] the vertical vibration levels of the floor are within a range generally considered to be "vibration free" for ordinary structures. The vibrations are random in nature and occur at the natural frequency of the floor system. The vibration levels are of the order

[redacted] STAT

of magnitude that are easily produced by the normal office type operation of people working within a building. Certain isolated areas near ventilation ducts and the rock crushers on the sixth floor vibrate at amplitudes somewhat greater than the average.

It is concluded from the above that a reduction in the vibration level of the floors [redacted] would not be eonomically feasible. STAT Corrections to the vibration problems associated with equipment can be most effectively produced by modifications to the equipment. If a more vibration free environment is required, a completely different type of structural system, specifically designed to minimize vibrations, should be considered. This, of course, means that a new structure would have to be built.

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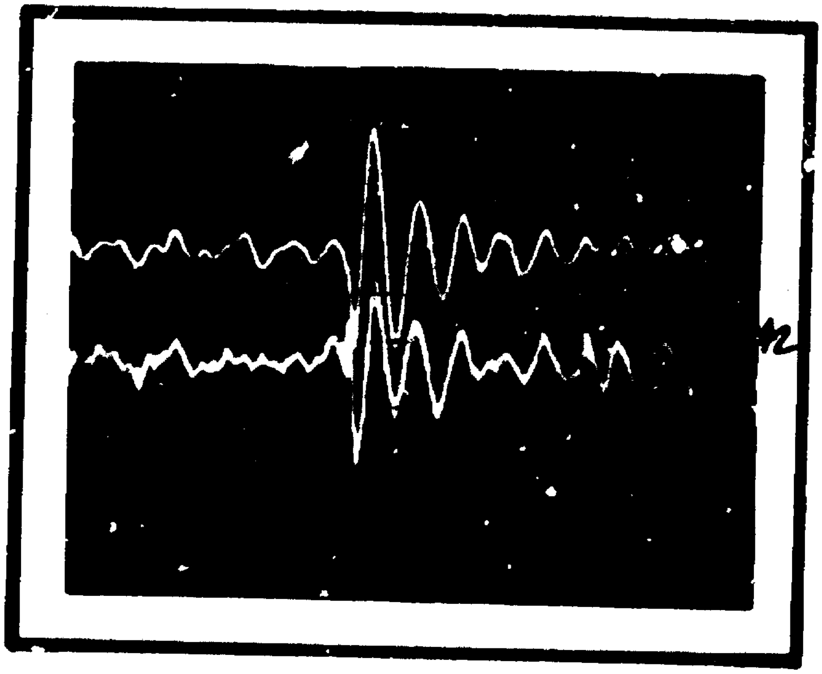
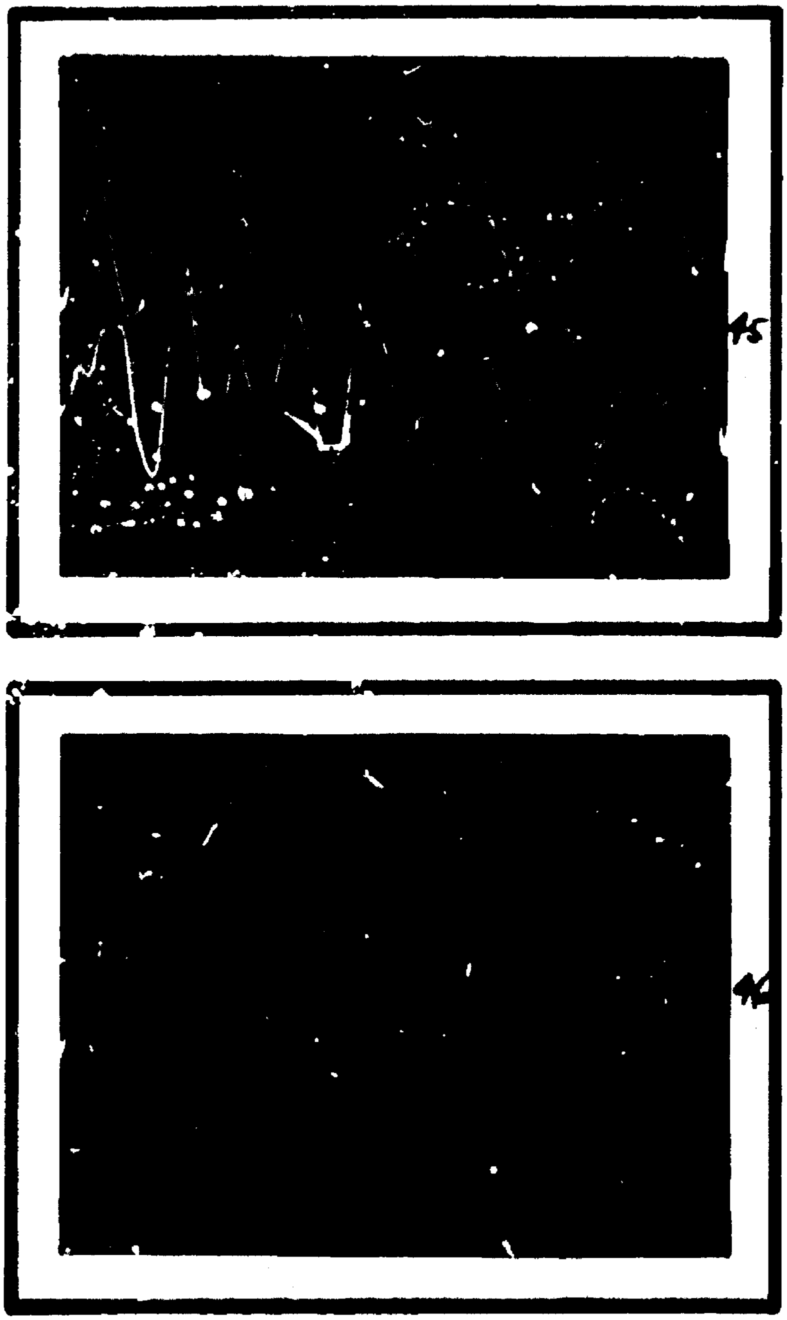
FIGURES

PHASE I



SCALES			
RECORD NO.	VERTICAL (IN./SEC.)/CM.		HORIZONTAL MILLISEC./CM.
	TOP	BOTTOM	
42	0.02	0.002	100
43	0.005	0.0005	50

FIGURE I-1

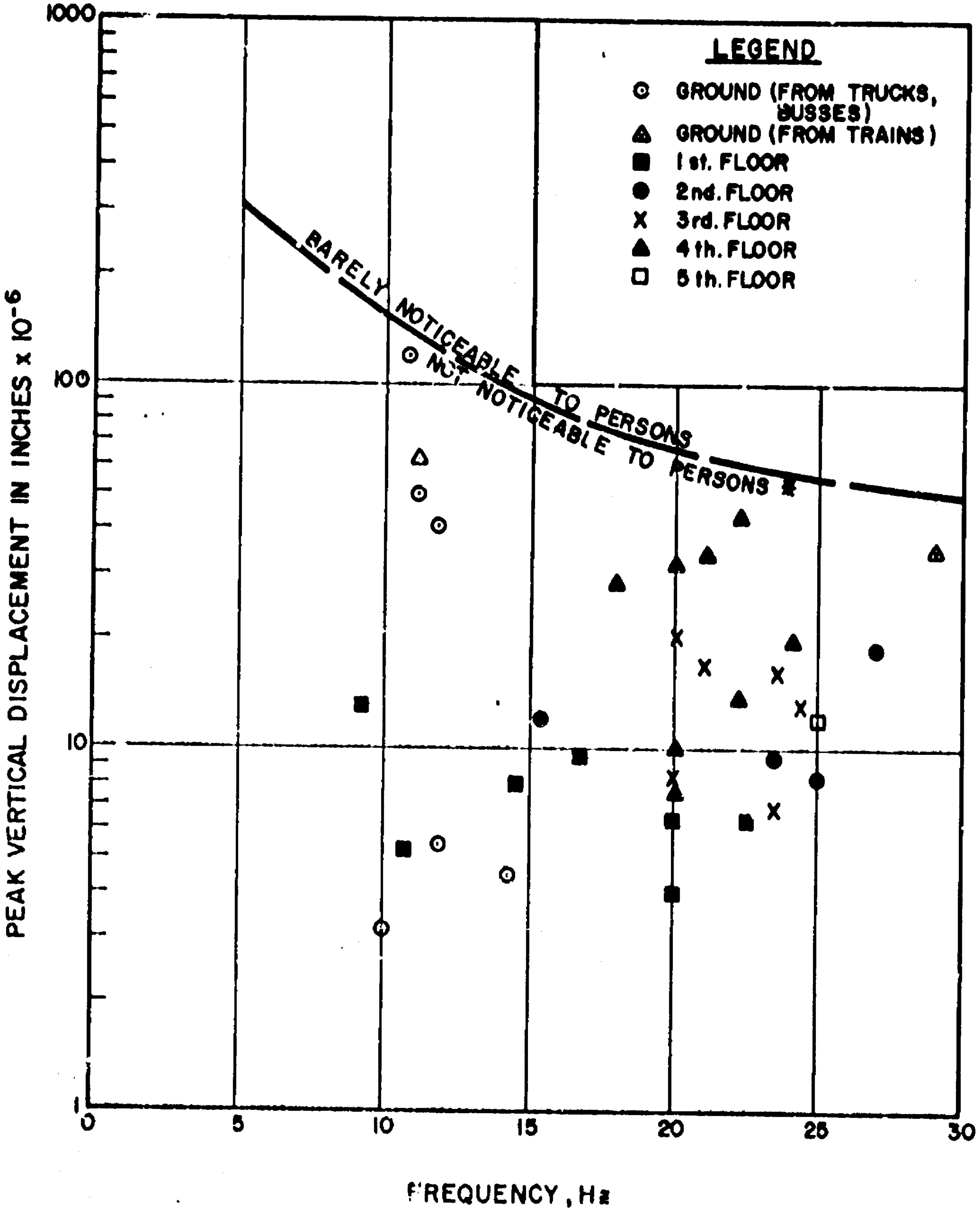


SCALES			
RECORD NO.	VERTICAL (IN./SEC.)/CM.		HORIZONTAL MILLISEC./CM.
	TOP	BOTTOM	
45	0.005	0.005	50
46	0.02	0.02	50

FIGURE I-2

RECORD NO.	VERTICAL (IN./SEC.)/CM.		HORIZONTAL MILLISEC./CM.
	TOP	BOTTOM	
47	0.02	0.02	50

FIGURE I - 3



* REFERENCE: REIHER AND MEISTER (1931)

[redacted]

SUMMARY OF AMBIENT VERTICAL VIBRATIONS IN AND AROUND [redacted]

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U.S. GOVERNMENT
WASHINGTON, D.C.

DWN BY	c/b	5-11-71	DRAWING NO.
CKD BY	JRH	5-11-71	70-197-A6
APP'D BY			

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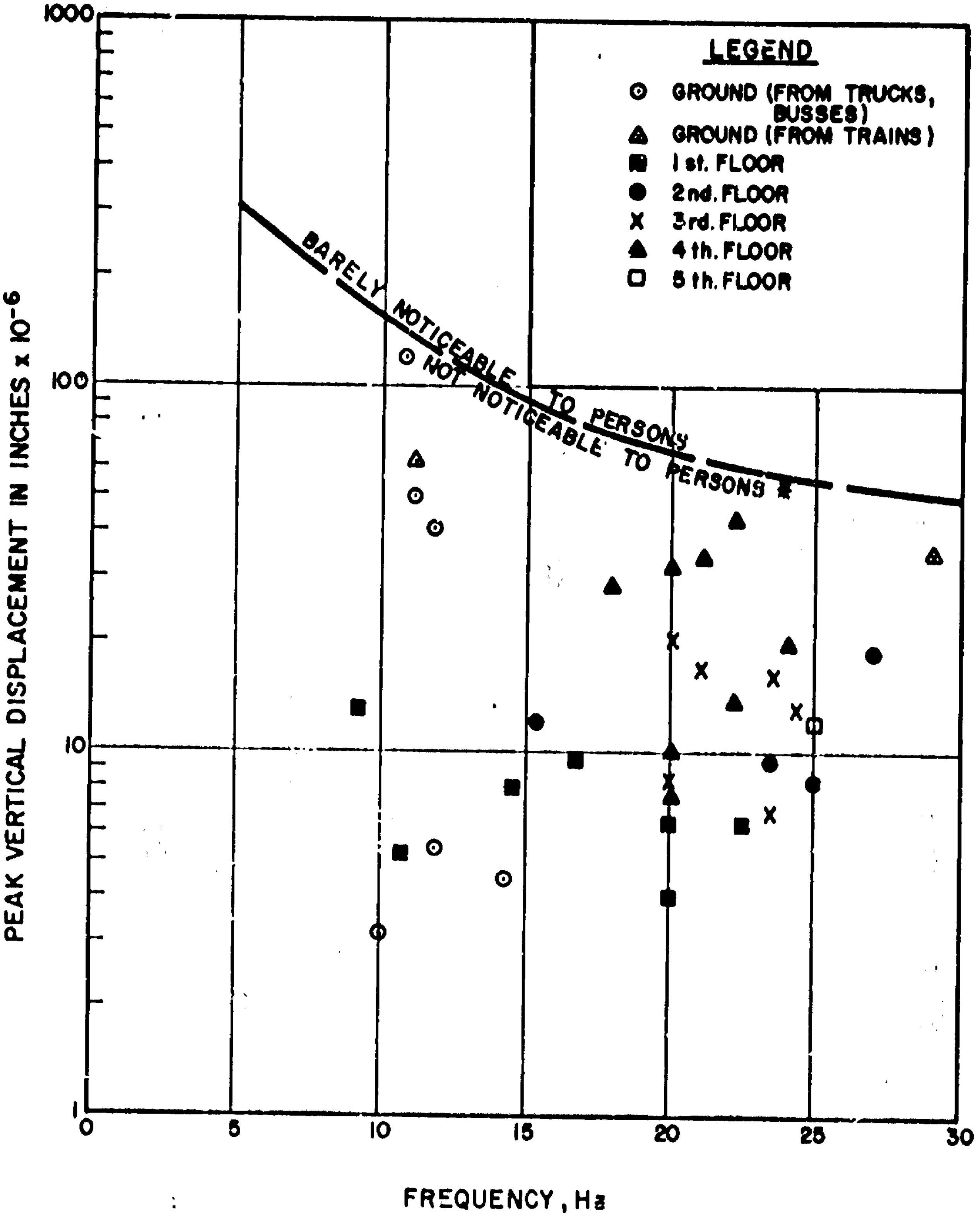
PR 3751-870

FIGURE I-4



* REFERENCE: REIHER AND MEISTER (1931)

[redacted] STAT

SUMMARY OF AMBIENT VERTICAL VIBRATIONS
IN AND AROUND [redacted] STAT

U.S. GOVERNMENT
WASHINGTON, D.C.

DWN. BY	c/b	5-11-71	DRAWING NO.
CKD. BY	JCH	5.11.71	70·197·A6
APPR'D. BY			

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FIGURE I - 4

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PHASE II

INVESTIGATION OF THE EFFECTS OF THE CONSTRUCTION AND OPERATION OF THE SUBWAY SYSTEM

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PHASE II

LIST OF TABLES

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PHASE II

LIST OF FIGURES

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LIST OF FIGURES
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PHASE II
INVESTIGATION OF THE EFFECTS OF THE CONSTRUCTION AND OPERATION OF THE SUBWAY SYSTEM

INTRODUCTION

This portion of the report deals with the investigation of the effects of the construction and subsequent operation of the proposed subway beneath M-Street, approximately 45 feet from the north face [redacted] STAT Construction activities are of concern because of possible settlement due to loss of soil during tunneling and due to consolidation during dewatering. Vibrations caused by construction activities are of lesser concern because the tunneling will take place through soil and no blasting or high-speed construction equipment will be involved. After the subway is completed, high-speed trains will be operating, and these will produce vibrations which are considered to be of a more serious nature.

To carry out this investigation, several disciplines were drawn upon to perform the analysis. Soil dynamics and geophysics were required to determine the dynamic characteristics of the soil with respect to vibration transmission while structural dynamics principles were required to formulate the structural model. Finally, a specialist in dynamic finite element computer techniques was used for an analytical solution of the soil response to the subway input.

After determining the site's characteristics and the general nature of the structure, a mathematical model representing the complete system from the subway to the structure was formulated. The model was then subjected to a vibrating input motion characteristic of the proposed subway system as predicted by WMATA consultants. The output from the model consisted of floor response motion which provides a direct basis for assessing the effect of the subway on the operation of equipment resting on each floor.

To simplify the presentation of the Phase II investigation, the various portions that make up the total system are described individually in the following sections of this report.

SUBWAY SYSTEM

The general location and cross-section of the subway system in the vicinity [redacted] STAT are shown on Figs II-1 and II-2. The building [redacted] STAT STAT Trains traveling between these two stations are scheduled to operate between 60 and 65 miles per hour traveling east and 55 to 60 miles per hour traveling west. The system is scheduled so that the direction of travel may vary from time to time in any one particular tube. The tunnels will be constructed using earth boring techniques immediately in front and west [redacted] STAT while a cut and cover technique will be used to the east. Since sands and gravels will be encountered during the tunneling operation, it will be necessary to provide a dewatering system to prevent flow of material into the face during construction. The present schedule is to begin construction in July of 1974 and to finish by August 1976; actual operation of the subways is scheduled for October of 1977.

The Nature of Vibrations Caused by Subway Trains

The nature of vibrations caused by the operation of a system of subway trains has been reviewed by considering data obtained by Wilson, Ihrig, and Associates, consultants to the WMATA. They have measured subway vibrations at two locations in Toronto, Canada, where the subway structure is

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located in soil.(1) Their data were analyzed with octave band filters and reduced to a motion spectra which plot root mean square acceleration versus octave band center frequencies as shown typically on Fig II-3. The data shown on this figure have been converted to displacements to be compatible with the computations in our investigation. For purposes of analysis of structural response, the vibrations below 30 cycles per second are significant, whereas the higher frequencies represent acoustical vibrations which are relatively unimportant.

<u>Factors Which Will Cause Lower Vibrations at the Washington Facilities</u>:

There are several differences between the Toronto System measured by the WMATA consultants and the system proposed by WMATA. Some of these differences will create a more stable vibration environment while others (as discussed in the next section) will tend to create a more adverse environment.

The rail fasteners for the Washington System will have a lower spring coefficient which should reduce the magnitude of vibration at frequencies above 30 Hz but will not significantly reduce the magnitude of motion between 8 and 30 Hz which is of concern in structural response analysis. Secondly, the Washington trains will be equipped with non-slip automatic braking systems which will reduce the formation of flat spots caused by wheel slippage. Flat spots, which are common to the Toronto System, are a principal source of vibrations on poorly maintained systems. Hence, their absence should tend to improve the situation [redacted]

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Factors Which Will Cause Higher Vibrations at the Washington Facilities:

The WMATA trains in the vicinity [redacted] will operate at 60 to 65 miles per hour; whereas, the Toronto System operates at 40 to 45 miles per hour. Vibration levels are generally found to be proportional to velocity, and therefore, Fig II-4 was prepared with approximate correction factors for conditions at WMATA. Another factor which may produce a slightly higher vibration environment is the use of two separate tunnels, as proposed by WMATA, rather than a single, wide tunnel. However, within the frequency range of interest to this investigation, it is believed that this effect will be minimal. STAT

In both subway systems, continuous welded rails are used, thus eliminating a potential source of vibrations at rail joints. Some consideration is being given by WMATA to specifying a concrete liner as opposed to a steel liner for the subway tubes to reduce acoustical vibrations. From a structural response standpoint, it appears that the concrete tube would probably be the better choice; but for frequencies less than 30 Hz, the difference between the two liners will probably be insignificant.

SUBSURFACE INVESTIGATION

During the period from February 23 to March 22, 1971, a series of five test borings were drilled at the site [redacted] The purpose of these borings was to determine the soil profile and dynamic soil properties. A plan of the borings in relation to [redacted] is shown on Fig II-5. Figures II-6 and II-7 show interpreted soil profiles based on the logs shown on Figs II-8 and II-9. STAT STAT

[redacted] STAT

Borings Nos. 1 and 2 which were intentionally drilled as a pair were placed 19 feet apart and drilled to a depth of 105 feet while Borings Nos. 3 and 4, also a predetermined pair, were placed 40 feet apart and drilled to a depth of 85 feet. To complete the soil profile under [redacted] STAT [redacted] STAT Boring No. 5 was drilled in front of the building to a depth of 81 feet. The dynamic soil properties were measured in the field using a cross-hole velocity measurement technique with each of the two pairs of borings mentioned above. As the drilling progressed, seismic tests were conducted on five-foot intervals and at approximately the same depth in a pair of two adjacent borings. In conjunction with the seismic study, split-spoon and undisturbed piston samples were pushed at various intervals to obtain samples for laboratory testing.

The soils underlying [redacted] STAT consist of 8 to 10 feet of fill material underlain by terrace deposits down to El -30 to El -40. According to the WMATA consultants,(2) the terrace deposits are of Pleistocene age, and the lower boundary at El -30 to El -40 marks the location of Creteaceous sediments. The uppermost terrace deposit is a layer of soft to medium hard silty clay that extends to approximately El -7, where a four-foot thick layer of soft organic clay occurs. Underlying this clay are medium dense to very dense interbedded layers of sand, gravel, and silt which extend to approximately El -70. A very hard silty clay was encountered at El -70 and extended to the bottom of the boring at El -85.

The seismic study was undertaken to determine the dynamic soil properties necessary for the prediction of the vibrations which will be transmitted

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(2) Report No. 14, Contract MOD. No. 3Z7021-009, Building 213, Washington Navy Yard, Section F003, Branch Route, Subsurface Investigation, by Mueser, Rutledge, Wentworth and Johnston, dated April 20, 1971.

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from the proposed subway [redacted] Figure II-10 shows a schematic diagram of the method and equipment used to measure the cross-hole compression wave (P-wave) and shear wave (S-wave) velocities in the various layers. Basically, P and S waves were generated by a hammer striking the drill rods attached to the split-spoon sampler. An electrical pulse is generated at the time of impact, and the time interval from impact to the arrival of P and S waves in the adjacent borehole was measured with a storage oscilloscope. This type of oscilloscope is advantageous since several records may be made and stored for comparison of the generated wave forms. Figure II-11 shows a typical recording of the P and S wave arrivals along with typical calculations of the wave velocities. The S wave varies from 800 feet per second near the ground surface to 3000 feet per second at 100 feet. Correspondingly, the P-wave increased from 5000 feet per second to 7700 feet per second. Since there is only a slight variation in the soil profile and measured velocities at each end of the building, the plot on Fig II-12 was used for the dynamic properties of the soil profile.

DEVELOPMENT OF THE STRUCTURAL RESPONSE MODEL

The system under consideration, as shown on Fig II-12, consists of a frame structure resting on a half space with two tunnels running normal to the direction of the structure. For purposes of computation, the structural frame was separated from the half space and modeled as an independent lumped parameter system. Then, a finite element analysis was conducted on the underlying half space with due account given to the presence of a structure at the surface. The output for the finite element program then served as

[redacted] STAT

input for the frame analysis. This section discusses the formation of the frame model while subsequent sections describe the finite element analysis of the soil response.

[redacted] is basically a reinforced concrete structure, 180 x STAT 400 feet in plan dimension, six stories high and supported by spread footings on 20-foot centers. The first floor is a slab supported on grade. The structure in its original form was only four stories high, but two additional floors were added at a later modification. The two top floors are a steel frame with reinforced concrete slabs.

A lumped-parameter model for the dynamic analysis of the structure was formulated with node points at the column floor intersections and midway between the columns. With this model, it was possible to analyze both the horizontal and vertical response of the floor system from inputs at the foundation. The natural frequencies for horizontal and vertical motion were calculated using a digital computer. Six horizontal modes of vibration were computed ranging from 1.0 to 11.6 Hz. The natural frequencies for vertical motion were computed and compared with the natural frequencies measured during the vibration investigation for Phase I. The model was then adjusted so that the natural frequencies in the vertical direction agreed with those actually measured. This provided an accurate model for vertical motion of the floor system.

The measurements made of the ambient vibrations indicated that the horizontal and vertical motions were quite similar in frequency content. These frequencies coincided closely with the computed vertical natural frequencies of the floor slab, indicating that the vertical natural frequencies are an important factor in the analysis of vibrations.

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DEVELOPMENT OF SOIL DYNAMIC RESPONSE MODEL

In computations of ground vibrations caused by industrial operations, subways and similar man-made sources, it has been experimentally verified that the soil can be considered as an elastic medium. Vibrations are transmitted through the soil essentially in two body wave forms and one surface wave form. One body wave, the compression or P-wave, generates particle motion in the direction of wave propagation while the shear wave or S-wave causes particle motion perpendicular to the direction of wave propagation and produces shear distortions in the medium. The surface wave, or Rayleigh wave, is characterized by the concentration of energy near the surface of the soil. They are analogous to the waves forming concentric rings when an object is thrown into a body of water.

For saturated soils, the compression wave velocity is about equal to the velocity of propagation of a compression wave in water, which varies from 4800 to 5500 feet per second. The shear and Rayleigh wave velocities are, for practical purposes, equal and are slower than the compression wave velocity. The two main factors, which influence the velocity of propagation of waves, are density and confining pressure. Thus, at increasing depth, wave velocities increase as shown on Fig II-12.

The computation of the dynamic response of a soil system differs considerably for similar computations for a structural system. When a vibration is produced in the soil, the energy propagates in the form of waves and is reflected at free surfaces and changes in materials. The wave energy is eventually lost either by propagation to infinity or by the generation of heat from internal soil damping. When formulating a finite element model

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to represent the soil, only a limited portion of the real system can be included in the model, and artificial boundaries must be created. Conventional finite element computer programs can only handle free, roller or fixed supports along the boundaries. Boundaries of this type cannot be used for the solution of dynamic soil-structure interaction problems since waves will be reflected at these boundaries. This, in effect, causes resonant frequencies which are dependent upon the size of the model used to represent the soil. There are two methods of treating the boundary to overcome this problem. Both methods have been developed by Dr. John Lysmer at the University of California who was retained as a special consultant on this aspect of the problem. The methods are described in the following paragraphs.

<u>Type A Boundary Conditions</u>

One method utilized to prevent reflection of wave energy at the artificial boundaries of the model is shown at the top of Fig II-13. The model consists of three zones. Zone I is composed of a finite element grid that includes the source of vibrations. Waves generated by the source are propagated to the left and right boundaries of Zone I. Beyond these boundaries, the soil is considered to be a layered system extending to infinity so that the waves propagate outward and are not reflected back to Zone I. The soil underlying Zone I is treated as a fixed boundary to represent bedrock or a very stiff soil.

The advantage of this modeling technique is that a relatively limited finite element mesh may be used to represent the system being analyzed. Once the conditions at the boundary of Zone I are computed, it is possible to compute the displacements at any point within Zones L and R through a closed form set of simultaneous equations.

Type B Boundary Conditions

A second technique used to represent the infinite extent of the soil is to provide an impedance matched boundary as shown on the lower portion of Fig II-13. It can be shown mathematically that a wave propagating through soil produces stresses directly proportional to particle velocity. Thus, a non-reflecting boundary may be produced by using viscous dampers which develop stresses in direct proportion to particle velocities at the boundaries. The viscous dampers shown on Fig II-13 generate the normal component of stress and a similar series of dampers must be included to generate the shear component of stress. With this technique, the stresses and displacements may not be computed outside the zone of the finite element model as they could using the other technique.

ANALYSIS OF SOIL RESPONSE DUE TO SUBWAY INPUT

Vibrations caused by the subway system are of a random nature that can be mathematically transformed from a displacement-time relationship to an amplitude frequency relationship through a Fourier transform. A coefficient of this transform, multiplied by the steady-state amplification factors of the response of the structure to a unit input at the subway, gives a coefficient of the Fourier transform of the response of the structure. The vibration measurements of the subway tunnel taken by the WMATA Consultants were reduced by octave band filters which provide the equivalent of a Fourier transform. The amplitude-frequency spectra of floor motion in the structure were obtained

[redacted]

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by multiplying the subway spectra by the amplification factor for steady-state motion at each frequency. As briefly discussed in a previous section, the first step in the analysis was to determine the ground response at the elevation of the footings [redacted] using the finite element analysis. The model for this portion included the subway tunnel, the soil and a mass loading on the soil to represent the inertia of the structure. Computations were also made without the mass loading of the structure to assess its influence. The results of these computations provided the necessary data for input to the structural response frame model.

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The steady-state frequency response of the finite element system, shown on Fig II-12, was computed using four, slightly different models to consider the depth to the lower rigid boundary and to study the effect of the structure on the motion caused by the subway system. For convenience in the analysis, a unit displacement input was assumed at the subway to arrive at the magnification factors for each frequency. All models assumed that the soil damping was 1-1/2 percent of critical, a value considered appropriate for the site's soils and degree of excitation.

Model 1 - Symmetric Model With Building

For this model, rock was assumed to exist at a depth of 230 feet below the ground surface. Symmetry was taken about a centerline between the two tunnels with the edge of the building extending to infinity from points located 30 feet from the tunnel centerline. The mass loading of the building included in the model increased the unit weight of the upper 5.8 feet of soil by 136 pounds per cubic foot with no change in the wave velocity. The finite element mesh used for this model consisted of 234 elements.

[redacted] STAT

Model 2 - Symmetric Model Without Building

This model was the same as Model 1 except that no increase in unit weight was used to account for the mass of the building. These results were used to study the significance of the mass of the building on the ground motion from the subway.

Model 3 - Deep Symmetric Modal With Building

Model 3 was identical to Model 1 except that the location to bedrock was increased to a depth of 300 feet below the surface.

Model 4 - Large "Exact" Model

Model 4 most closely represents the real system, but it is much more complex. The results were used to verify the adequacy of Model 1, which assumes symmetry and considerably reduced the amount of computations on the computer. The edge of the building was located at the correct distance of 68 feet from the vertical line midway between the two tunnels. The finite element mesh extended from the left side of the left tunnel to the edge of the building. The depth to bedrock was taken as 230 feet.

LOAD CASES CONSIDERED FOR THE ANALYSIS OF GROUND MOTIONS FROM THE SUBWAY

Six load cases were considered for the various models described above. The results of the computations for these load cases are shown on Figs II-14 to 26. Each curve is labeled with a coding system. For example, a curve labeled 3V20/68 indicates Load Case 3, vertical motion at a distance 68 feet from the centerline of the model caused by 20 Hz of unit excitation at the tunnels. The conditions for each load case are described as follows.

As an aid to the reader in studying the results presented on Figs II-14 to 26, Table II-VI summarizes the various load cases and models used for the analysis.

[redacted]

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Case 1

Figures II-14 through II-17 show the resulting horizontal and vertical magnification factors at the foundation level versus distance from the tunnel centerline for Model 1 considering both tunnels excited by a unit vertical displacement vibrating in-phase. Computations were carried out at frequencies of 4, 6, 10, 12.5, 15, 17.5, 20, 22.5, 25, 27.5, and 30 cycles per second.

The data shown on Figs II-14 and II-15 indicate that the vertical subway motion will cause magnified horizontal motions in the 10 to 15 Hz range and that motion caused by exciting frequencies higher or lower than this range will be attenuated. As indicated on Figs II-16 and II-17, vertical motion at the base [redacted] as caused by vertical vibration at the two subway tunnels will be generally attenuated except for a small frequency range around 10 Hz.

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Figures II-18 and II-19 show the magnification factors for displacements on a vertical line beneath the edge of the building at a distance of 68 feet from the line of symmetry in the model. These are plotted for frequencies of 10, 15 and 20 cycles per second. The corresponding displacements beneath the center of the building (268 feet from the centerline of the model) are shown on Figs II-20 and II-21. Except for frequencies in the 10 to 15 Hz range, these results generally indicate that attenuation is occurring as the vibration is transmitted to the surface and to lower depths.

Case 2

Case 2 uses the same loading conditions as Case 1, but it is applied to Model 2 to show the effect of the building. As noted in the previous section, Model 2 assumes that the building does not exist.

[redacted]

The effect of the building can be readily seen by the data summarized in the following table. The magnification data have been taken from Figs II-14, II-15 and II-22 for points [illegible]ance of 68 feet or greater from the tunnels.

TABLE II-1

COMPARISON OF LOAD CASES 1 AND 2

Magnification Factor With Building (Max)	Magnification Factor Without Building (Max)	Motion* Type	Excitation** Frequency (Hz)
1.04	1.12	Horizontal	10
0.68	1.18	Horizontal	20
1.22	1.12	Vertical	10
0.68	0.40	Vertical	20

* Motion experienced at base [redacted]

** Exciting motion is a unit displacement in the vertical direction in both tunnels acting simultaneously and in phase.

These results suggest that the presence of the building causes an attenuation of horizontal motion and only a slight amplification of vertical motion at the frequencies considered. Therefore, in a practical sense, the building does not significantly alter the incoming motion to the foundation.

[redacted] STAT

Case 3

Case 3 also uses the same loading conditions as Case 1, but they are applied to Model 3, which assumes that bedrock, or "rigid boundary" in the finite element analysis, is at a depth of 300 feet instead of 230 feet. The purpose of this loading case is to investigate the sensitivity of the predicted soil response to assumptions regarding this lower boundary. Figure II-23 shows the computed magnification factors for frequencies of 10 and 20 Hz. Comparison of these results with Figs II-14, II-15, II-16 and II-17 yields the results summarized in the following table:

TABLE II-II

COMPARISON OF LOAD CASES 1 AND 3

Magnification Factor* (Max) ("rigid boundary" at depth 230')	Magnification Factor* (Max) ("rigid boundary" at depth of 300')	Motion** Type	Excitation*** Frequency (Hz)
1.04	1.52	Horizontal	10
0.68	0.60	Horizontal	20
1.22	0.92	Vertical	10
0.68	0.38	Vertical	20

* Magnification factors are for distances greater than 68 feet from the tunnel centerline.

** Motion experienced at base [redacted] STAT

*** The exciting motion is a unit displacement in the vertical direction in both tunnels acting simultaneously and in phase.

These results, plus those shown on Fig II-24, indicate that the predicted motion is slightly sensitive to the assumption regarding the lower boundary, particularly in the low-frequency range. However, with three of the four

[redacted] STAT

output parameters considered, the use of a 230-foot boundary is conservative; whereas, in the fourth case (low frequency horizontal motion), the use of this boundary is non-conservative. From an overall standpoint, it is believed that the use of the 230-foot boundary is appropriate, and therefore, the results developed for Case 1 are proper.

Case 4

Load Case 4, which again is the same as Case 1, is applied to Model 4. As previously discussed, Model 4 is larger, and theoretically more exact than Model 1, in that it assumes that the east edge of the superimposed load caused by the building is 66 feet from the tunnel centerline and only on one side of the tunnels; whereas, Model 1 assumes that the east edge of the building is 30 feet from the tunnel centerline and on both sides of the street, or subway system. The more exact model was not used throughout the analysis because of the much greater computer time and cost required. Comparing the maximum magnification factors shown on Fig II-25 with those listed in the previous tables indicates the following results.

TABLE II-III

COMPARISON OF LOAD CASES 1 AND 4

Magnification Factor* (Max) (Model 1)	Magnification Factor* (Max) ("Exact" Model)	Motion** Type	Excitation*** Frequency (Hz)
1.04	1.28	Horizontal	10
1.22	0.80	Vertical	10

* Magnification factors are for distances greater than 68 feet from the tunnel centerline.

** Motion experienced at base [redacted] STAT

*** The exciting motion is a unit displacement in the vertical direction in both tunnels acting simultaneously and in phase.

[redacted]

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The results in Table II-III indicate that the use of Model 1 and its associated assumptions are reasonably appropriate but not conservative for horizontal motion. Therefore, it would appear that some type of correction factor might be appropriate. For instance, one might use the ratio of the maximum magnification factors, i.e., correction factor = CF = 1.28/1.04 = 1.18. However, the use of such a factor is not practical for basically two reasons - (1) as seen in the following section on structural response, the horizontal motion is severely attenuated as it travels up through the structure, and (2) since the input motion at the subway is defined over such a great range as shown on Fig II-4, the application of a correction factor on the order of 18% does not improve the accuracy of the analysis. Therefore, the results of the Model 1 computer runs are being used directly in the structural response.

Case 5

The large and more exact Model 4, discussed in the previous paragraphs, was also used to analyze the soil response when only one tunnel was excited. The objective of this case was to determine the effect of two trains versus one train running simultaneously past Building 213. A review of the two cases as shown on Fig II-26 and the results in Table II-IV, indicates that, as expected, the excitation of only one tunnel produces displacements somewhat smaller than those when both tunnels are excited.

[redacted]

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TABLE II-IV

COMPARISON OF LOADING CASES 4 AND 5

Magnification Factor* (Max) (Two Trains Running)	Magnification Factor* (Max) (One Train Running in South Tunnel)	Motion Type**	Excitation Frequency, Hz
1.28	0.92	Horizontal	10
0.82	0.75	Vertical	10

* Magnification factors are for distances greater than 68 feet from the tunnel centerline.

** Motion experienced at base [redacted]

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It is interesting to note that even though the energy input is twice as great when two trains are running, the soil response is only about 1.4 times greater for the horizontal motion and only about 1.1 times greater for the vertical motion. This is attributed to the greater distance that the wave forms must travel when the north tunnel is excited. As suggested by Fig II-26, it appears that the wave combination that generates the horizontal motion is a result of two peaks, one from each tunnel, arriving at the same time and adding. For vertical motion, it appears that a peak from the south tunnel is arriving at the same time as a valley from the north tunnel with a resulting smooth curve which is slightly higher.

These results suggest that (1) the assumption of two trains passing simultaneously and in phase is not an over-conservative assumption, and (2) little is to be gained by restricting subway scheduling so as to preclude the possibility of two trains passing simultaneously in front of

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[redacted]

[redacted] STAT

Case 6

All the input motion at the subway tunnels considered up to this point has been vertical, rather than horizontal (transverse to the axis of the tunnel) primarily because vertical motions are generally considered to be at least an order of magnitude greater. Even though the horizontal input motion is much smaller, it is conceivable that it could be amplified considerably more than the vertical motion. Therefore, Case 6, which assumes that the south tunnel only is excited by a unit horizontal displacement was investigated with the larger and more exact Model 4. The results are shown on Fig II-27 and compared with Case 5, on Fig II-28 and in Table II-V, which assumes one tunnel excited vertically.

TABLE II-V

COMPARISON OF LOAD CASES 5 AND 6

Magnification Factor* (one train - Case 5) Vertical Excitation	Magnification Factor* (one train - Case 6) Horizontal Excitation	Motion** Type	Excitation Frequency
.93	1.0	Horizontal	10
0.75	1.22	Vertical	10

* Magnification factors are for distances greater than 68 feet from the tunnel centerline.

** Motion experienced at base [redacted] STAT

The data indicate that the vertical input portion will be transmitted about the same as the horizontal input motion. Therefore, if the horizontal input motion is an order of magnitude lower than the vertical motion, then it can be considered negligible in assessing structural response as discussed in the next section of this report.

[redacted]

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ANALYSIS OF STRUCTURAL RESPONSE FROM GROUND MOTION INPUT

A previous section under "Development of the Structural Model" discusses the development of the structural model which was used to obtain the response of the structure from the ground motion input computed using the dynamic soil model. To compute responses of each floor, the computer program 'ANSYS' was used.

The response of the lumped parameter structural model was obtained by defining the displacement conditions at the base of the model in terms of real and imaginary components obtained from the Case 1 loading conditions of the dynamic soil model. The real and imaginary parts represent the in-phase and 90 degrees out of phase components of displacement with respect to the unit real displacement input at the tunnels. The steady-state response of the lumped parameter model was computed for frequencies of 4, 6, 10, 12.5, 15, 17.5, 20, 22.5, 25, 27.5, and 30 Hz.

The vertical and horizontal motion spectra at typical nodes are shown on Figs II-29 to II-33. These curves were obtained by multiplying the amplification factors at each frequency by the displacement input of the subway at the the corresponding frequency on Fig II-4. The results indicate that the maximum component of vertical floor displacement will occur at 10 Hz. The peak vertical ground displacement also occurs at 10 Hz. This correlates well with the predominant ground motion frequencies produced by busses, trucks and trains summarized in Table I-III.

The peak horizontal and vertical displacements at 10 Hz have been plotted for each floor of the structure on Figs II-34 and II-35. Figure II-34 indicates that the vertical displacement is the same at each floor level and equals the vertical component of ground displacement. The cyclic variation of

[redacted] STAT

the vertical displacement is not considered as an accurately predictable phenomenon and should not be used as a guide to location of high and low vibration levels. Consequently, the dashed line through the peaks is included and should be considered as representative of the predicted magnitude at each location.

The horizontal component of floor displacement plotted on Fig II-35 illustrates the attenuation with increasing floor level. The attenuation is characterized for ground motions at relatively high frequencies compared to the natural frequencies of the structure.

Comparison of Predicted Vibrations From the Subway with Present Ambient Vibrations

Figure II-36 compares the motion spectra at Node 47 to the ambient vibrations presented on Fig I-4. Node 47 is representative of the maximum vibrations produced by the subway at the north end [redacted] Line "A" STAT represents an upper bound envelope on the present vibration environment excluding temporary disturbances caused by trucks, busses and trains, while Line "B" includes the effect of these temporary disturbances. It is noted that in the low frequency range, Line "B" is based on measurements made outside the structure on the ground and not on the floors per se. The justification for the use of these data lies in the fact that our computer analysis indicates that ground motion at this frequency is transmitted upward through the building virtually unchanged. Therefore, it is appropriate to compare the responses of Node 47 with either Line "A" or Line "B." Based on this comparison, it is concluded that the proposed subway system will

[redacted] STAT

significantly affect the present environment (Line "A"), but the disturbance will not be greater than that presently caused by trucks, busses and trains (Line "B"). This conclusion is undergoing further checks to determine the effects of input frequencies precisely equal to the natural frequencies of the floor systems throughout the structure as it is entirely conceivable that the extent of the temporary disturbance might be greater than predicted above if a resonant condition develops. The results of this portion of the study will be reported in Appendix B.

SETTLEMENT [redacted] STAT
DUE TO SUBWAY CONSTRUCTION

A comprehensive report(2) dealing with the problem of settlement during construction of the subway has been prepared by WMATA's soils consultant. This report has been reviewed by EDCE and the results and conclusions are reiterated below.

As discussed in the Subsurface Investigation section, [redacted] STAT is underlain by soils of the "25-foot" Pleistocene terrace which extends downward to the Cretaceous surface between El -30 and -40. Consolidation tests indicate that the terrace soils are overconsolidated by drying to approximately 7 tons per square foot in excess of the overburden pressure near the top of the layer and to approximately 2 tons per square foot in excess of the overburden at El -20.

The spread footings [redacted] STAT vary in size as shown on Fig II-12 and were designed for a net average pressure of 3 tons per square foot. The bearing area of the footings covers approximately 1/3 of the

[redacted] STAT

[redacted]

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entire building area, thus the average pressure is 1 ton per square foot over the building area. On the basis of a 30-foot drawdown in the water table, negligible settlements will occur during the dewatering of the subway system. The loading of the soil from drawdown is equivalent to the load that would be produced if the design live load of the building were applied to the first four floors.

The major problem related to settlement during construction will occur from loss of ground during tunnel excavation. Dewatering may be difficult in the lower Pleistocene soils and running or flowing sands may be encountered. It has been estimated that settlements of the North Building line might be on an order of 1/4 inch under the most unfavorable construction procedures. To minimize loss of soil by flowing conditions, dewatering of the Pleistocene layer should be completed prior to construction. Also, the North Tunnel construction should be completed prior to commencement of work on the South tunnel.

Settlements of the order of magnitudes predicted are not considered to be serious. The settlement from dewatering is expected to be of a relatively uniform nature and will not create any noticeable effects. The settlement from loss of soil during excavation of the tunnels is entirely a function of construction control. By constructing the North tunnel first, experience will be gained so that the ground loss during construction of the south tunnel may be kept to a minimum.

CONCLUSIONS

The conclusions of this report are based on computations predicting the vibrations that will be produced by the subway system to be constructed

[redacted] STAT

under M-Street. Supplementary computer runs are in progress and will be discussed in a forthcoming appendix. Based on present information, the following conclusions are drawn:

1. Present ambient vibrations are random in nature and are caused, for the most part, by normal office-type activity of persons in the building.
2. Vibrations are of the order of 50 percent greater than the average near ventilation ducts. Elimination of vibration from the ventilation ducts would not significantly improve the vibration environment.
3. Reduction of the floor vibrations by altering the structure is not economically feasible. Therefore, elimination of the work bench vibrations will require alteration to the instrument itself.
4. The proposed subway system will significantly affect the present vibration environment of [redacted] but the disturbance will not be greater than that presently caused by trucks, busses and trains passing outside the building. STAT
5. Settlement of the [redacted] structure during construction of the subway will be negligible. STAT

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May 1971

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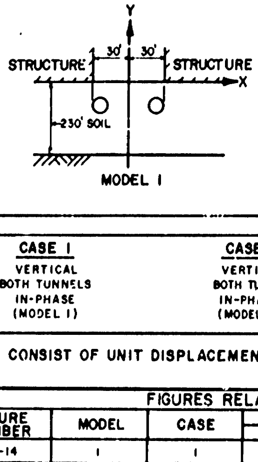
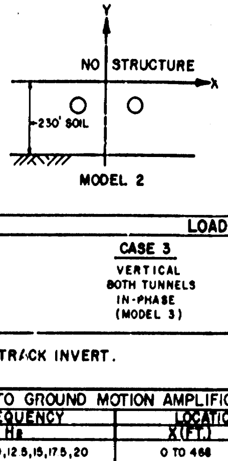
TABLE

PHASE II

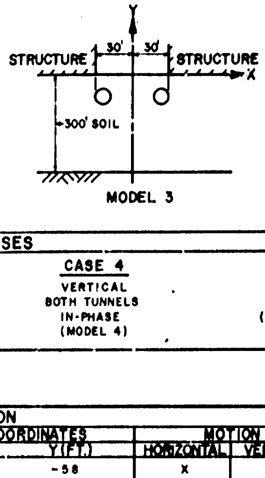
MODELS USED



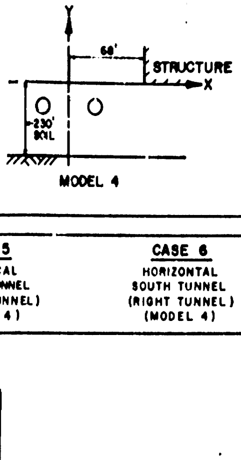
MODEL 1



MODEL 2



MODEL 3



MODEL 4

LOAD CASES

	CASE 1	CASE 2	CASE 3	CASE 4	CASE 5	CASE 6
* INPUT:	VERTICAL BOTH TUNNELS IN-PHASE (MODEL 1)	VERTICAL BOTH TUNNELS IN-PHASE (MODEL 2)	VERTICAL BOTH TUNNELS IN-PHASE (MODEL 3)	VERTICAL BOTH TUNNELS IN-PHASE (MODEL 4)	VERTICAL SOUTH TUNNEL (RIGHT TUNNEL) (MODEL 4)	HORIZONTAL SOUTH TUNNEL (RIGHT TUNNEL) (MODEL 4)

* ALL INPUTS CONSIST OF UNIT DISPLACEMENTS AT TRACK INVERT.

FIGURES RELATING TO GROUND MOTION AMPLIFICATION

FIGURE NUMBER	MODEL	CASE	FREQUENCY	LOCATION COORDINATES		MOTION	
			Hz	X(FT.)	Y(FT.)	HORIZONTAL	VERTICAL
II-14	1	1	4,6,10,12.5,15,17.5,20	0 TO 468	-5.8	X	
II-15	1	1	22.5,25,27.5,30	0 TO 468	-5.8	X	
II-16	1	1	4,6,10,12.5,15,17.5	0 TO 468	-5.8		X
II-17	1	1	20,22.5,25,27.5,30	0 TO 468	-5.8		X
II-18	1	1	10,15,20	68	0 TO-230	X	
II-19	1	1	10,15,20	68	0 TO-230		X
II-20	1	1	10,15,20	268	0 TO-230	X	
II-21	1	1	10,15,20	268	0 TO-230		X
II-22	2	2	10,20	0 TO 468	-5.8	X	X
II-23	3	3	10,20	0 TO 468	-5.8	X	X
II-24	3	3	10,20	58	0 TO -250	X	X
II-25	4	4	10	0 TO 468	-5.8	X	X
II-26	4	4,5	10	0 TO 468	-5.8	X	X
II-27	4	6	10	0 TO 468	-5.8	X	X
II-28	4	5,6	10	0 TO 468	-5.8	X	X

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FIGURES

PHASE II

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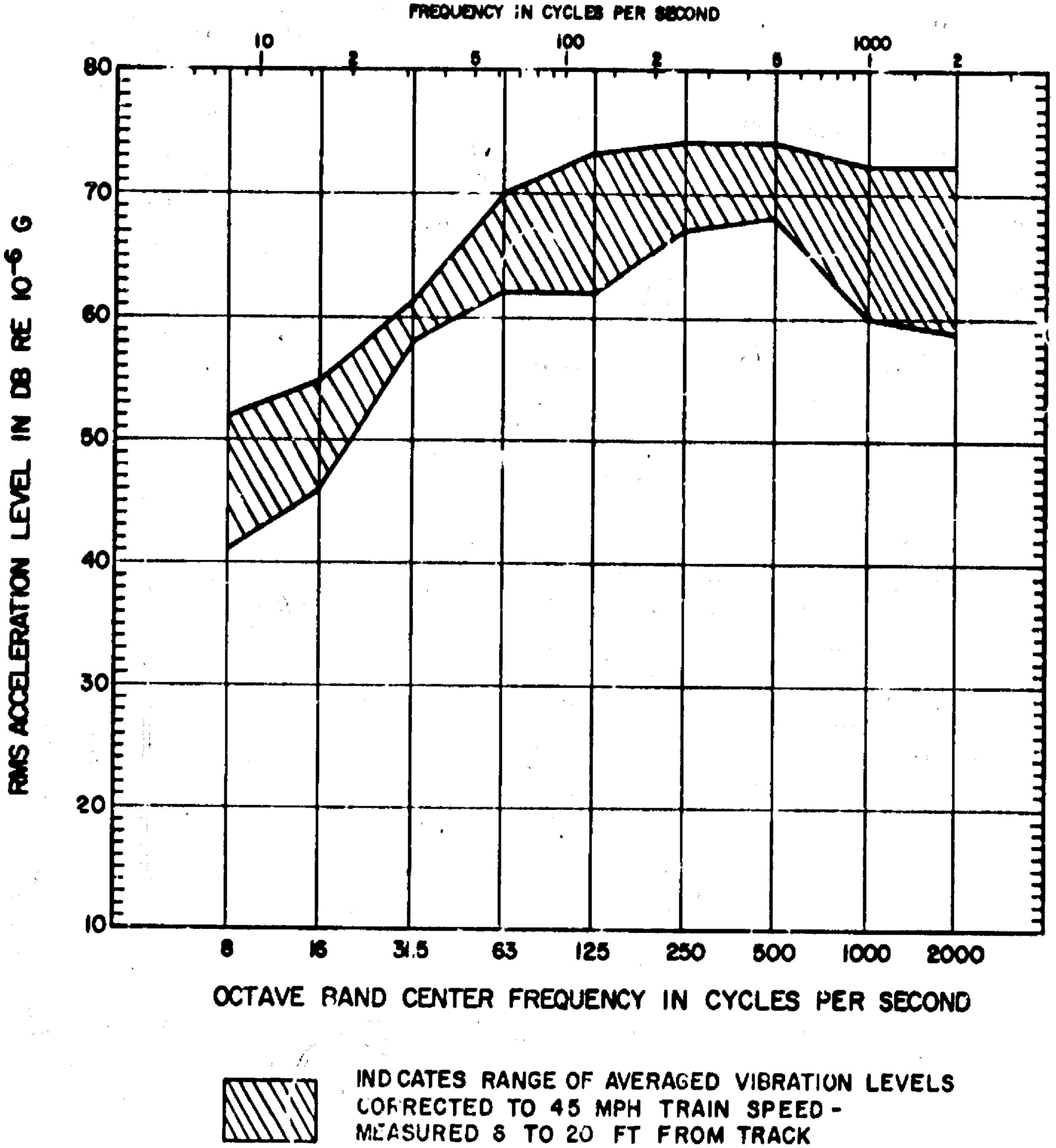

REFERENCE:
WILSON, IHRIG & ASSOC. - LETTER TO
DE LEUW, CATHER & CO. (MAR. 22, 1971)

FIGURE II - 3

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[redacted]

RMS ACCELERATION OF TUNNEL
VS. OCTAVE BAND CENTER FREQUENCY

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CKD BY	JLH	5-10-71	70-197-A4
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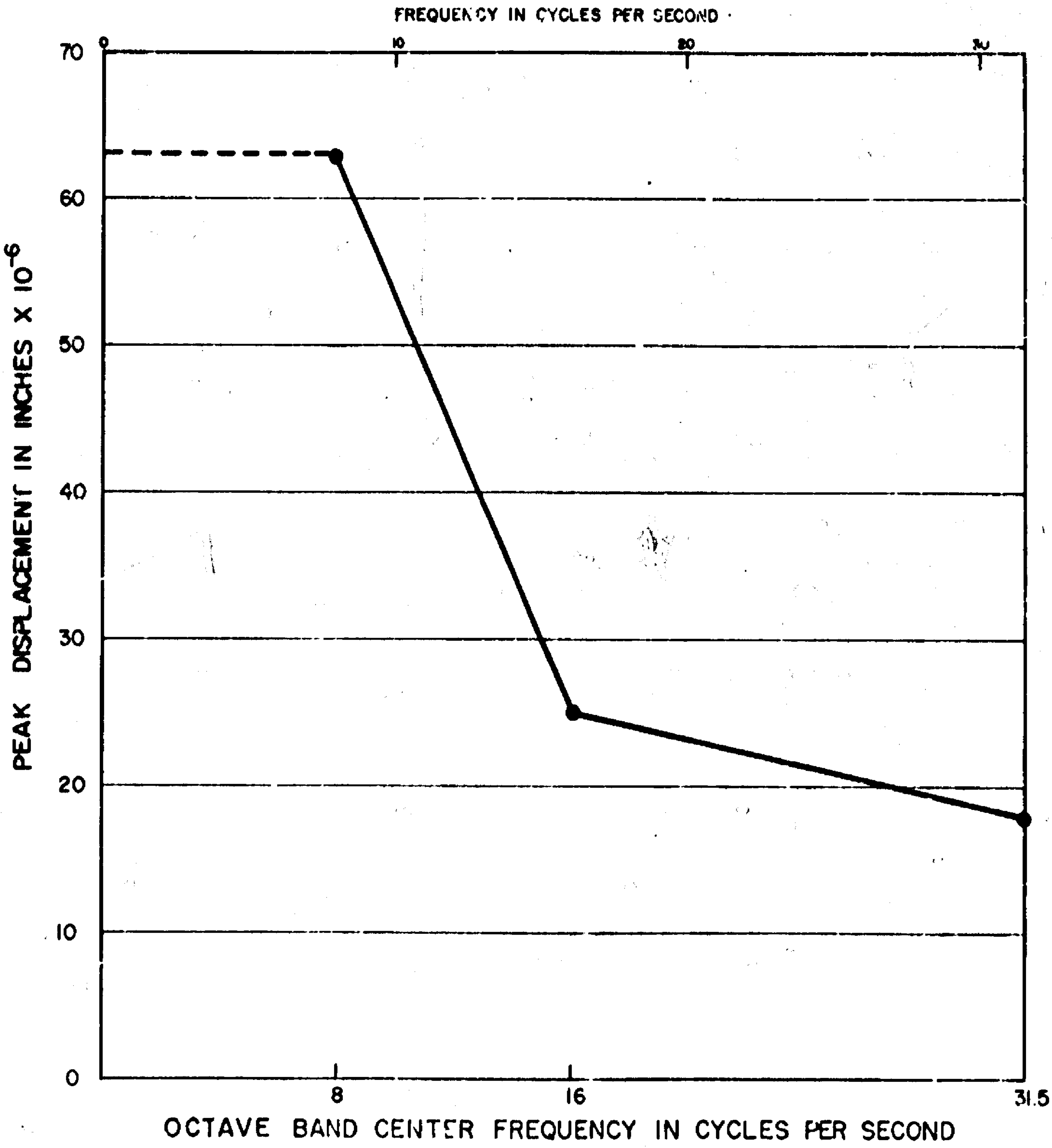

FIGURE II-4

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AVERAGED VIBRATION LEVELS CORRECTED TO 65 MPH TRAIN SPEED - MEASURED 6 TO 20 FT. FROM TRACK CENTERLINE

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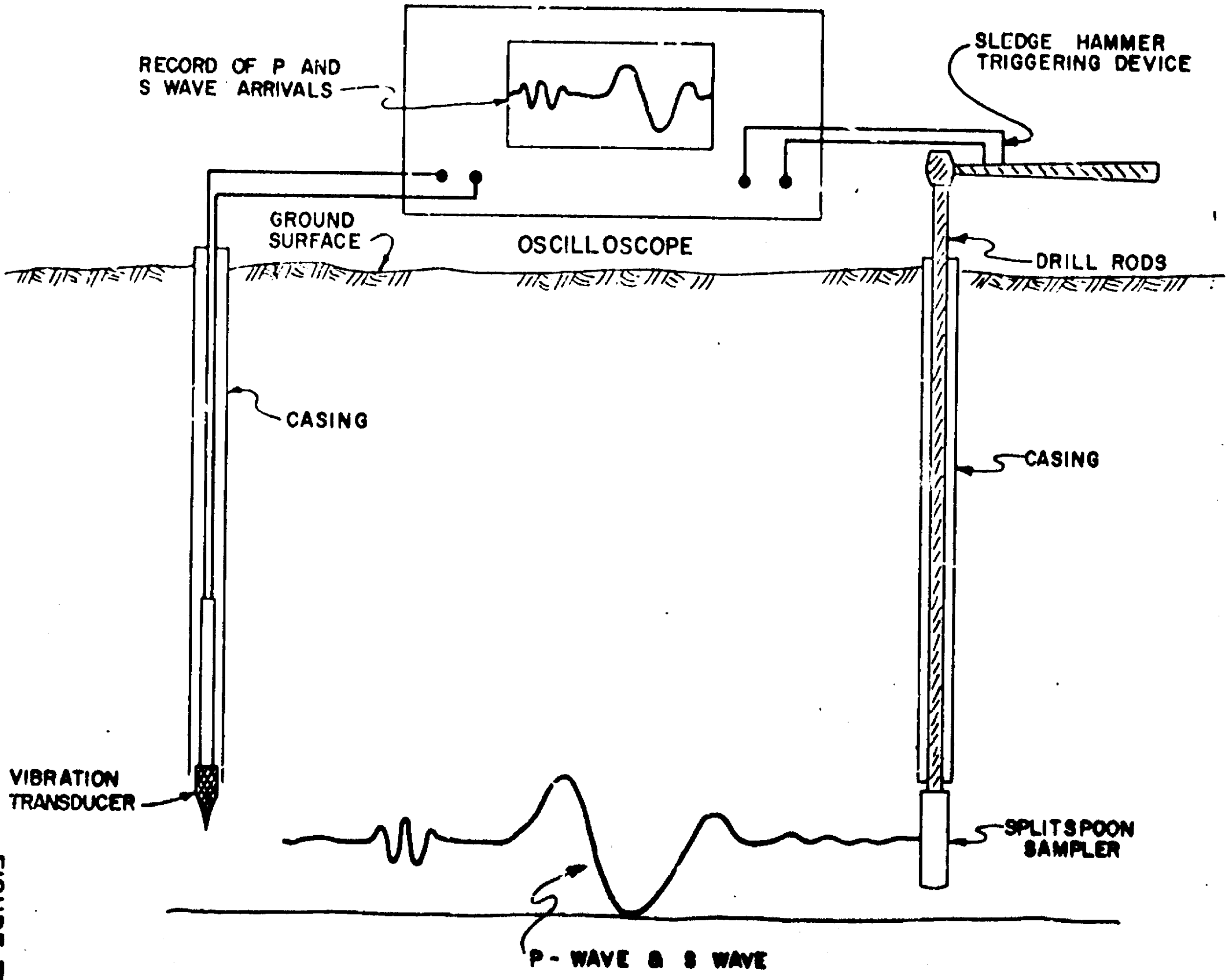

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SETUP FOR CROSS-HOLE
VELOCITY MEASUREMENT

DWN. BY	RGN	4-21-71
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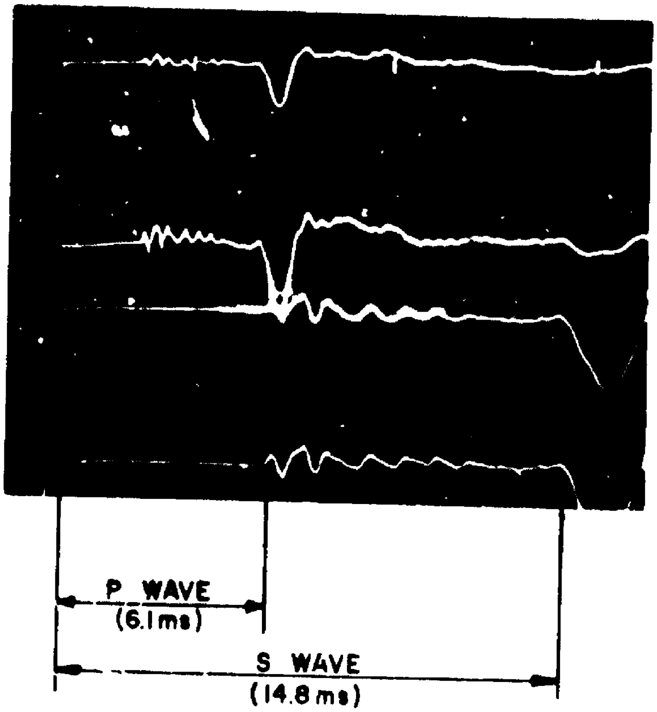
70-197-A1

FIGURE II-10



BORINGS 1 & 2
DEPTH 54'
HARD, SANDY CLAY
19.08' SPACING

TOP TRACE 5ms/cm
BOTTOM TRACE 2ms/cm

DRILL ROD CORRECTIONS

$$\frac{57'}{16{,}000'/\text{SEC.}} = -3.56\ \text{ms}$$

TRIGGERING DEVICE CORRECTION = +0.31 ms

$$-3.25\ \text{ms}$$

$$V_P = \frac{19.08'}{2.85\ \text{ms}} = 6690'/\text{SEC.}$$

$$V_S = \frac{19.08'}{11.55\ \text{ms}} = 1650'/\text{SEC.}$$

FIGURE II-11

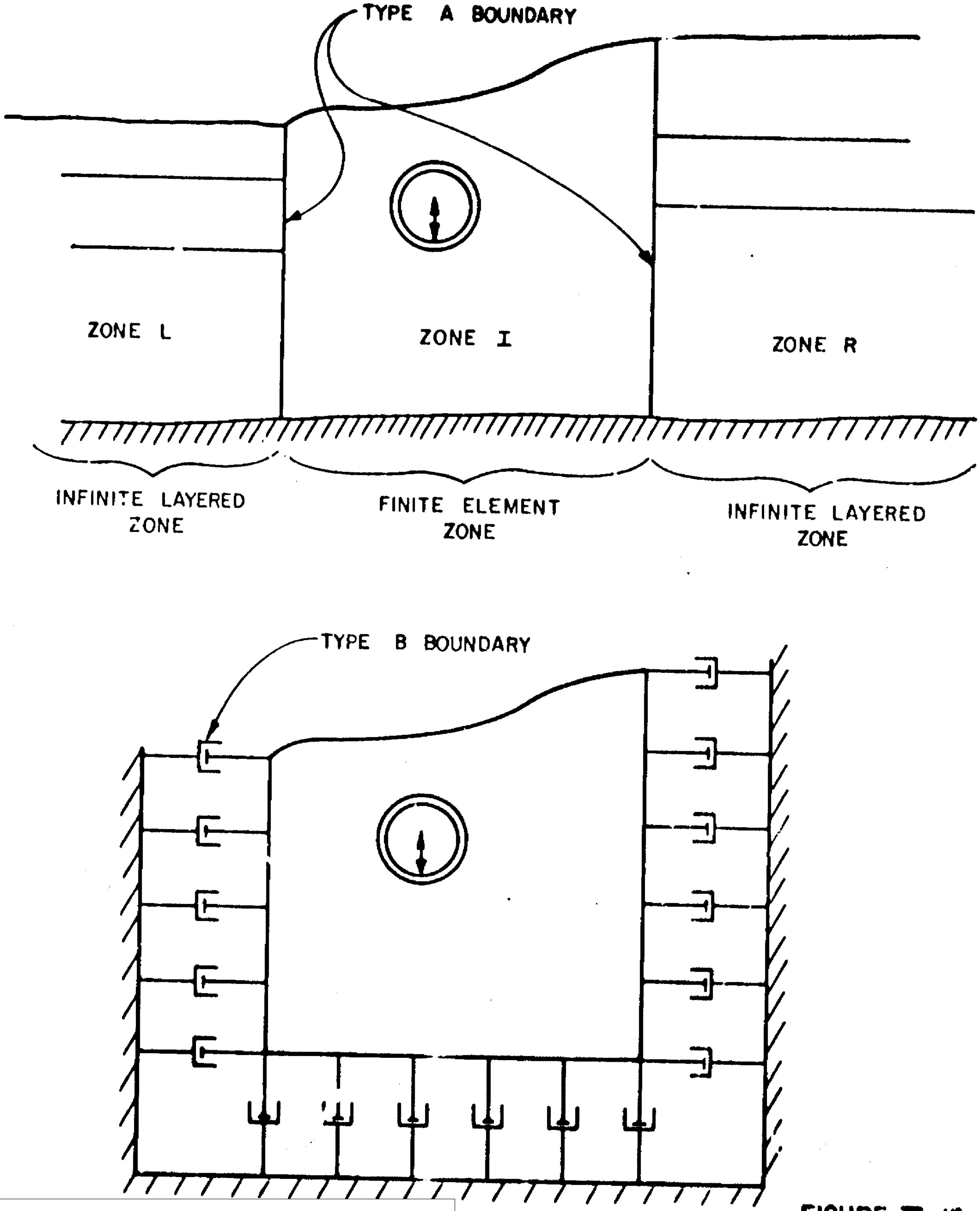


FIGURE II-13

BOUNDARY CONDITIONS USED TO REPRESENT A CONTINUOUS SYSTEM

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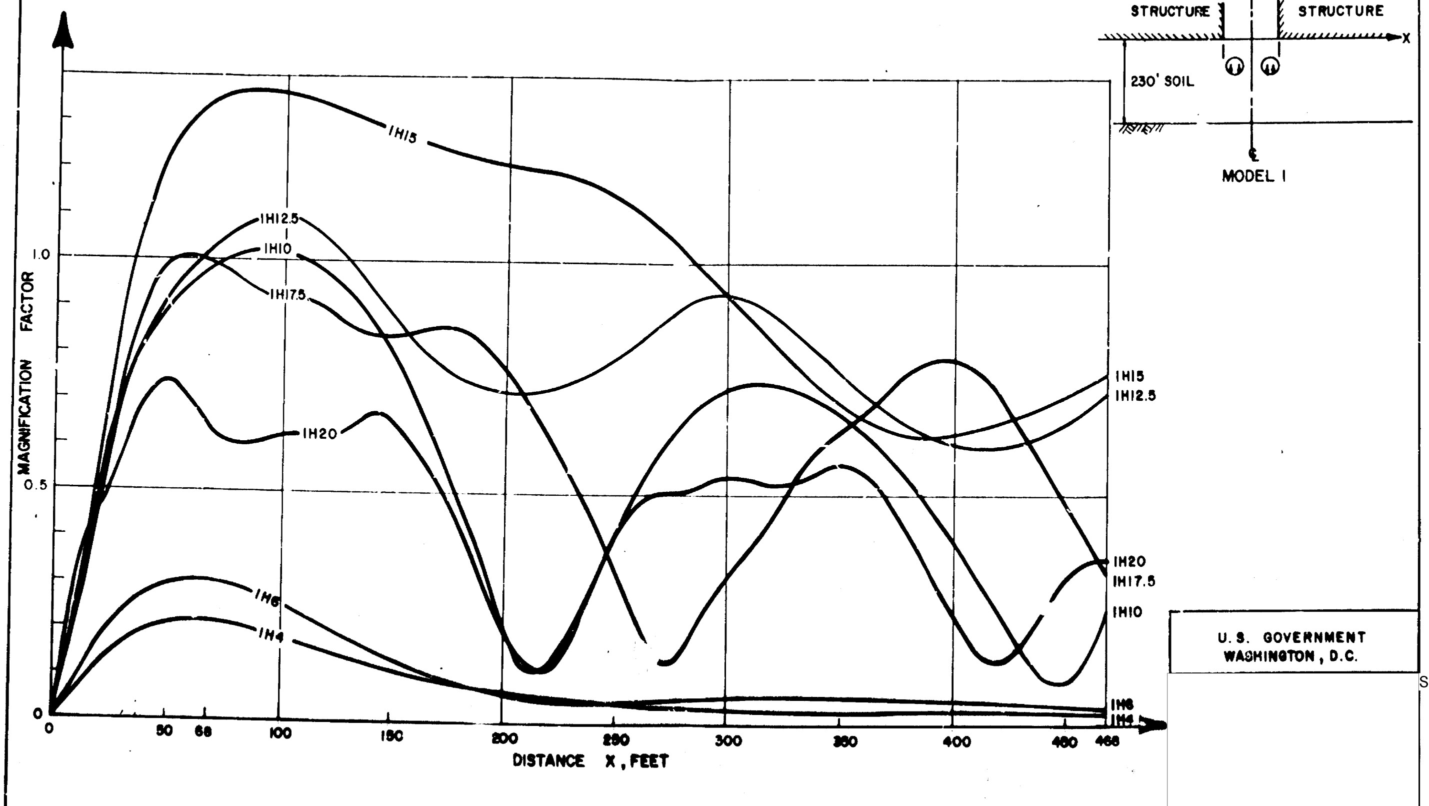

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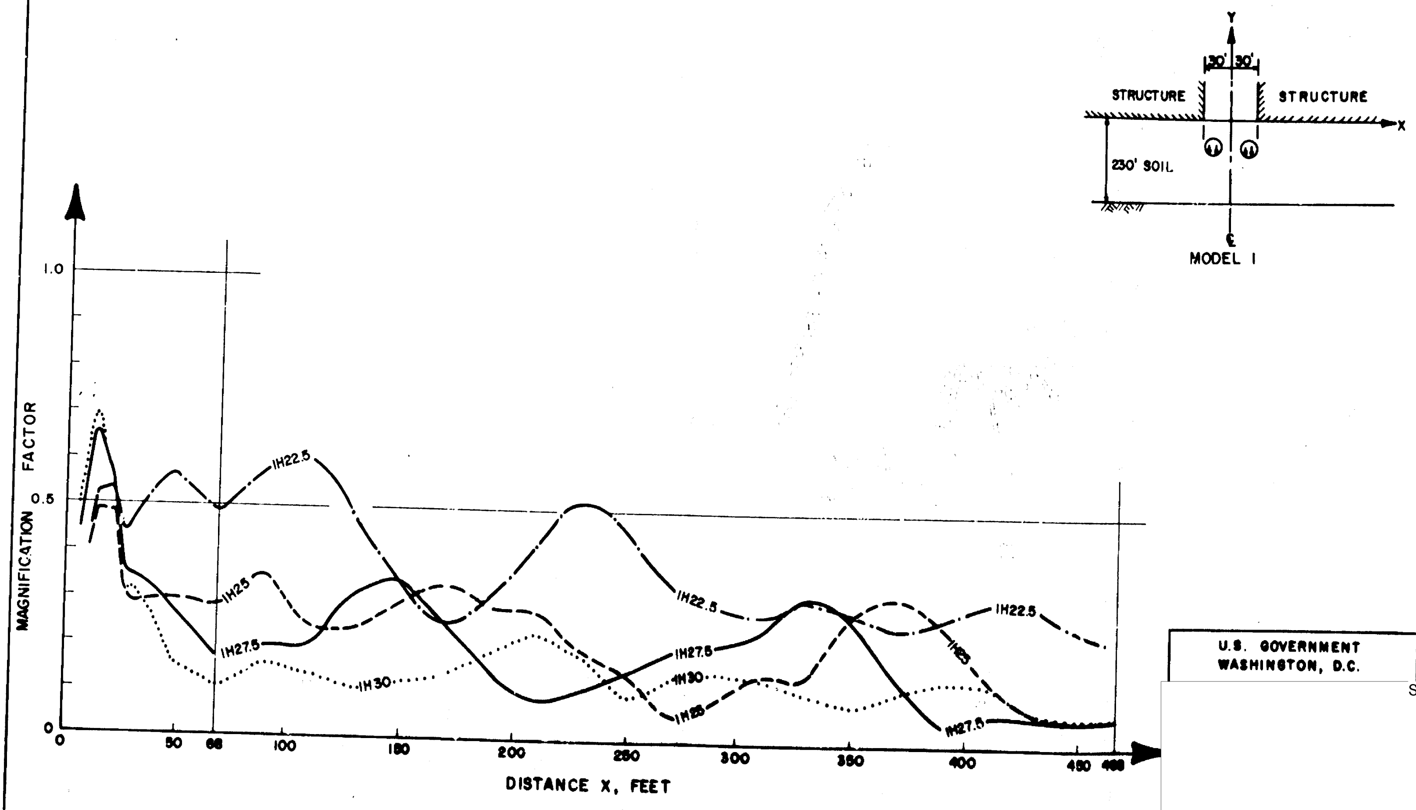
FIGURE II-14



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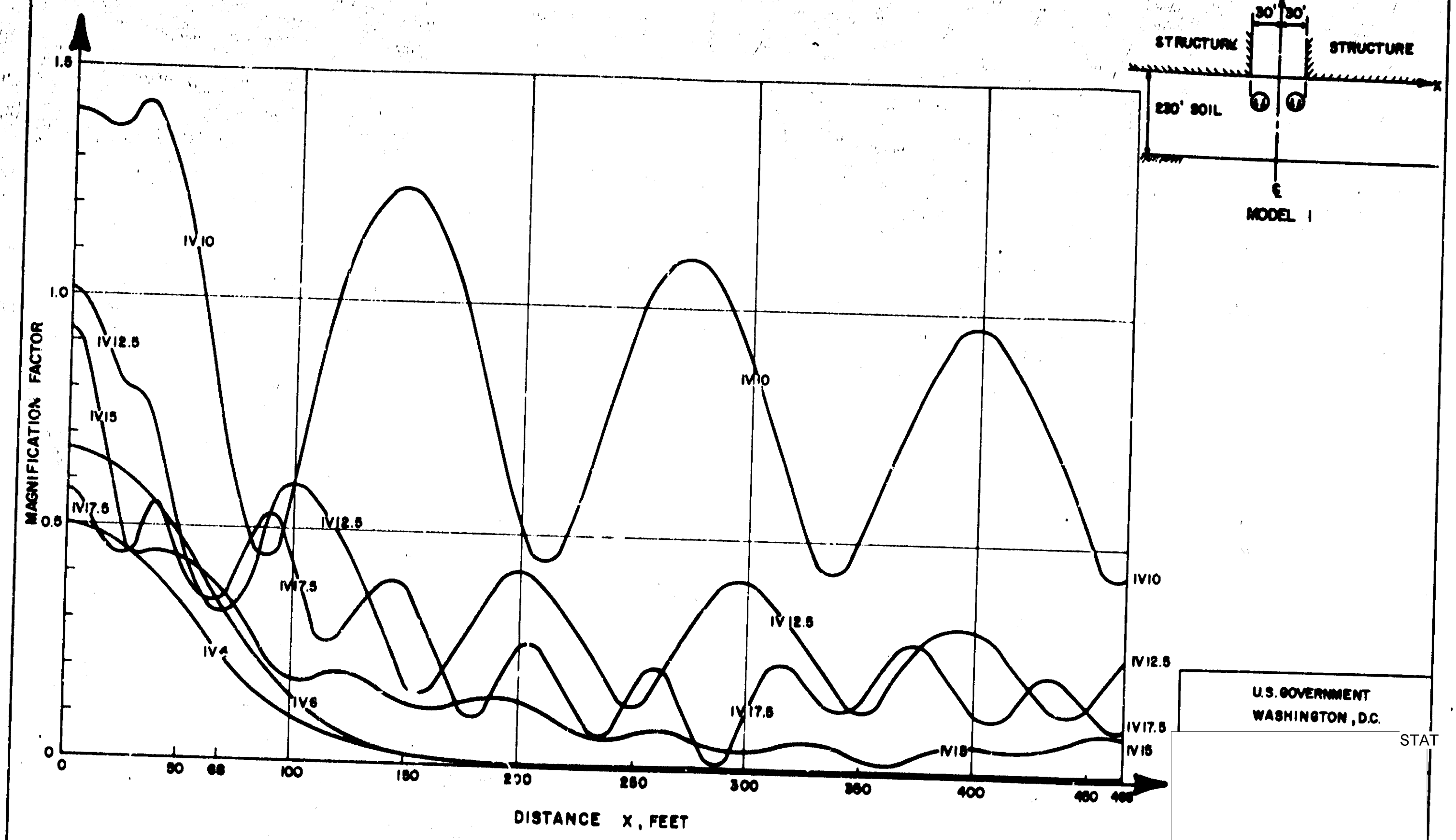
STAT [redacted]

FIGURE II-15

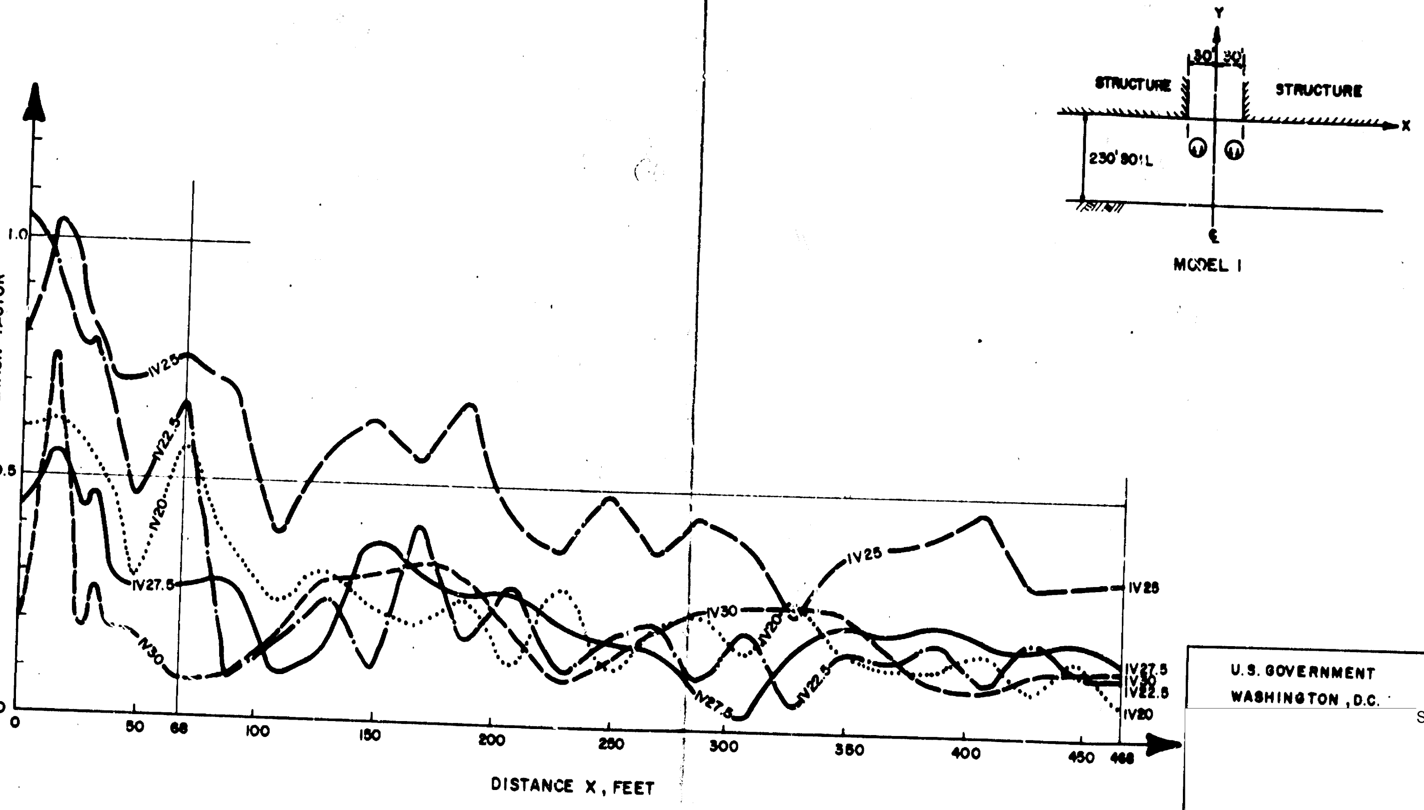


U.S. GOVERNMENT
WASHINGTON, D.C.

[redacted]

STAT

FIGURE II-16



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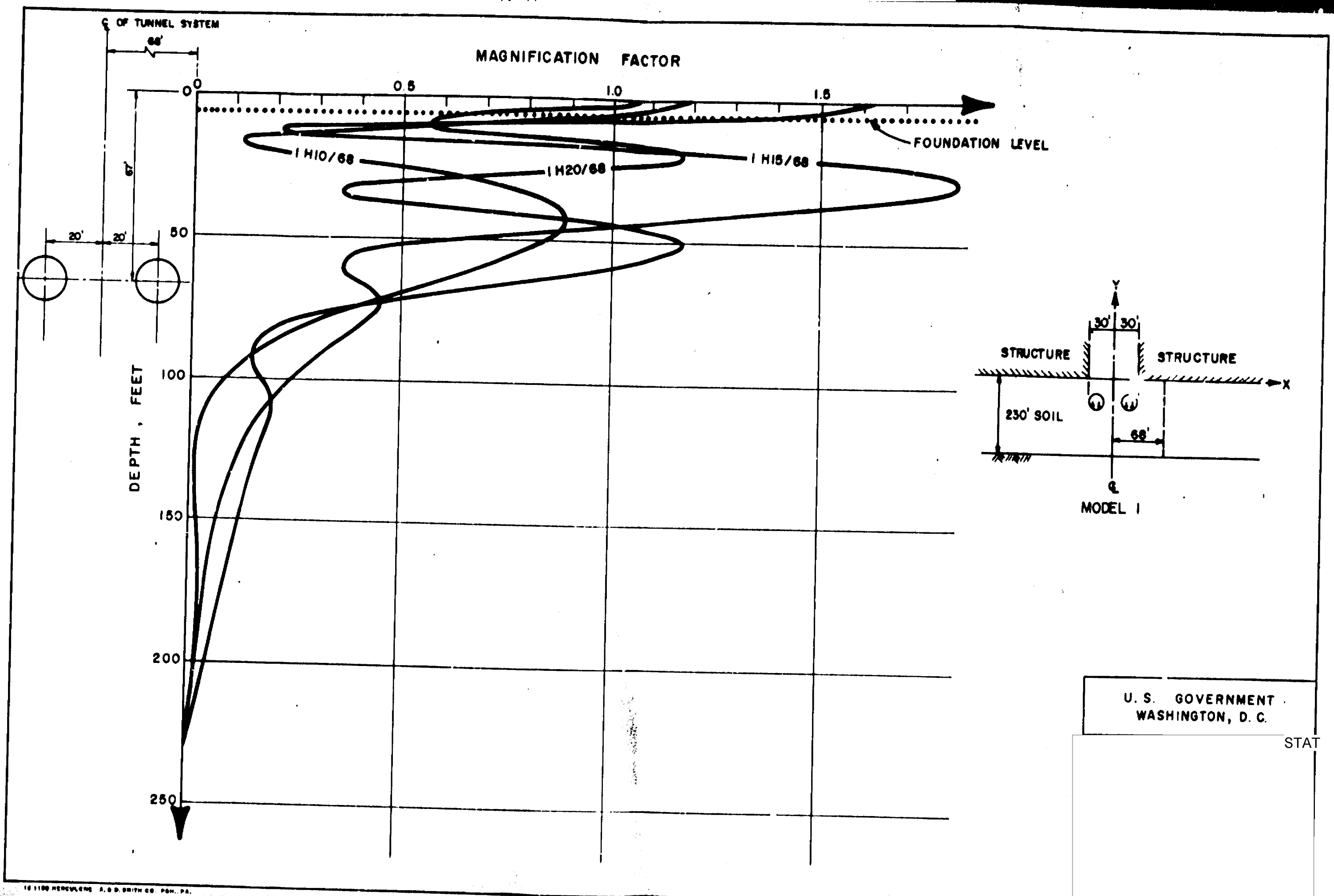
[redacted]

STAT

FIGURE II - 17

PR 9230-700



U. S. GOVERNMENT
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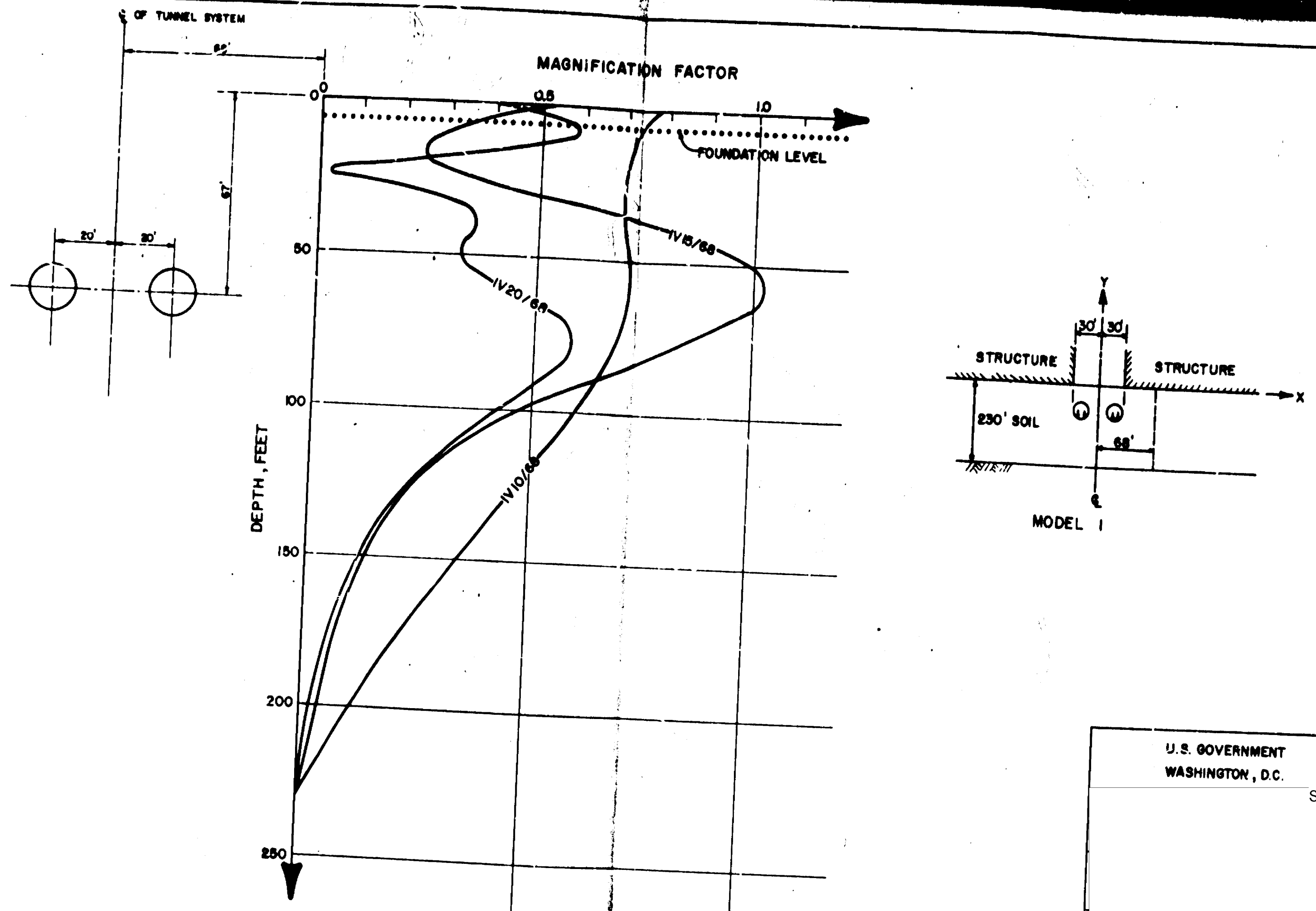
[redacted]

STAT

FIGURE II-16



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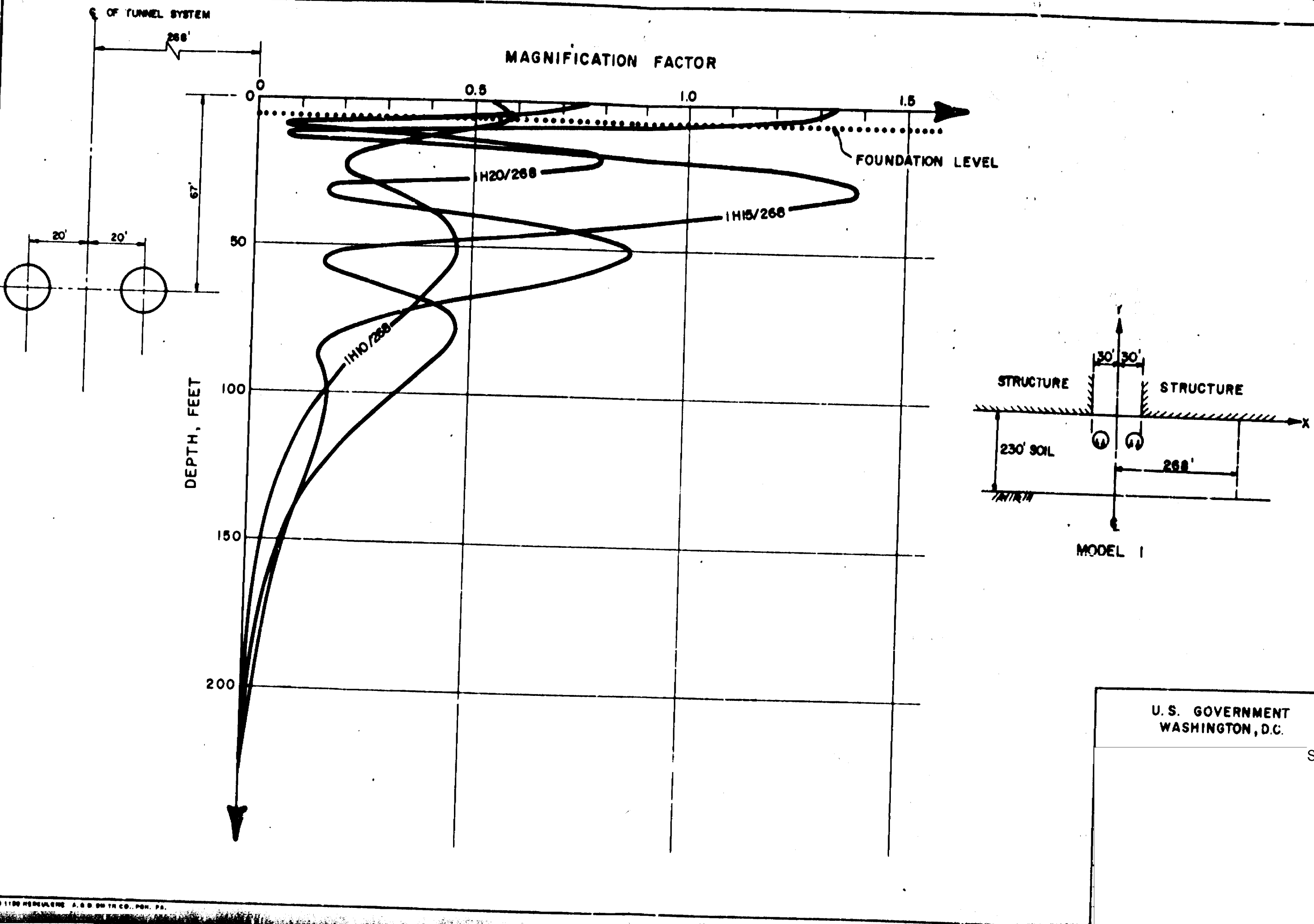
STAT

[redacted]



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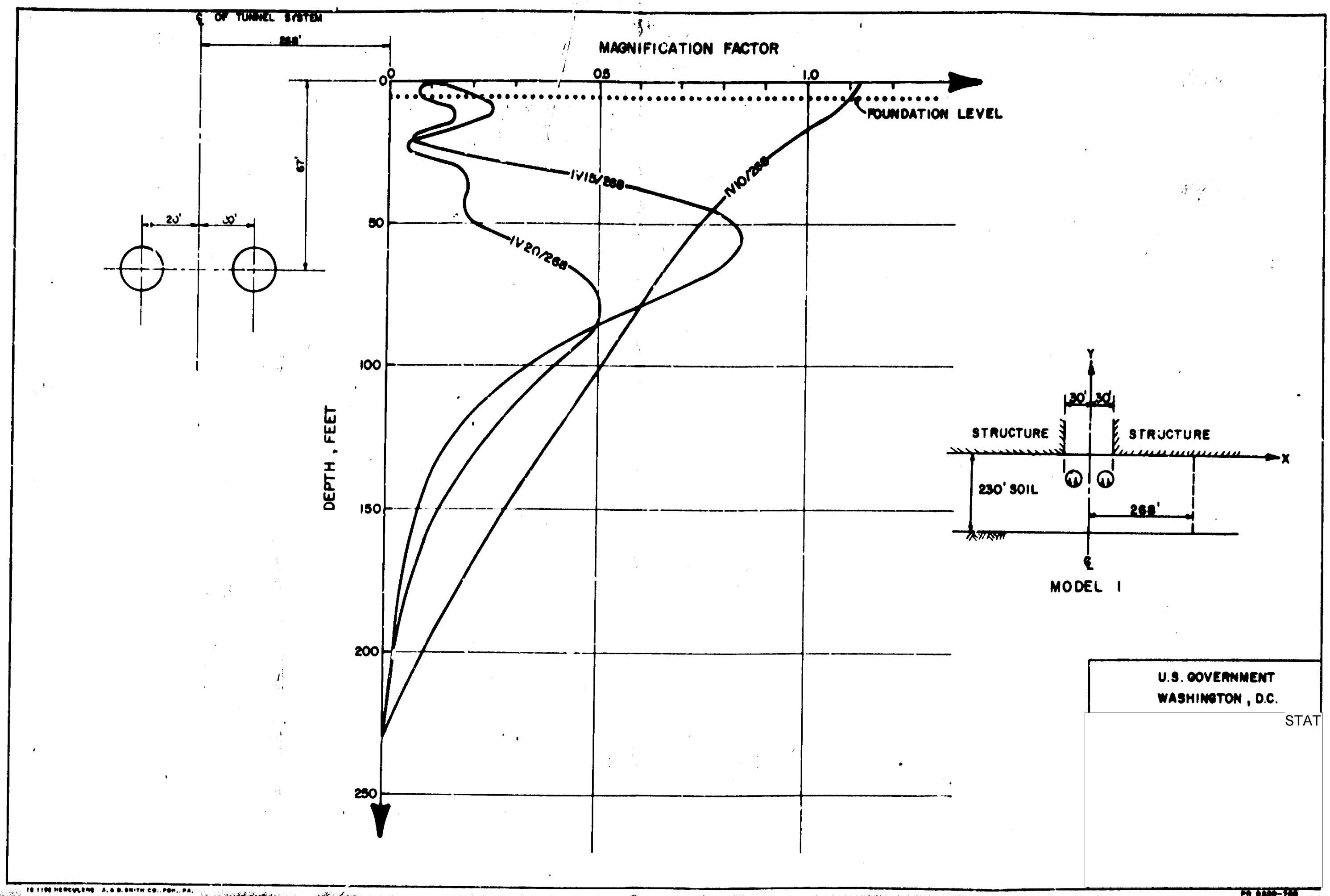
STAT

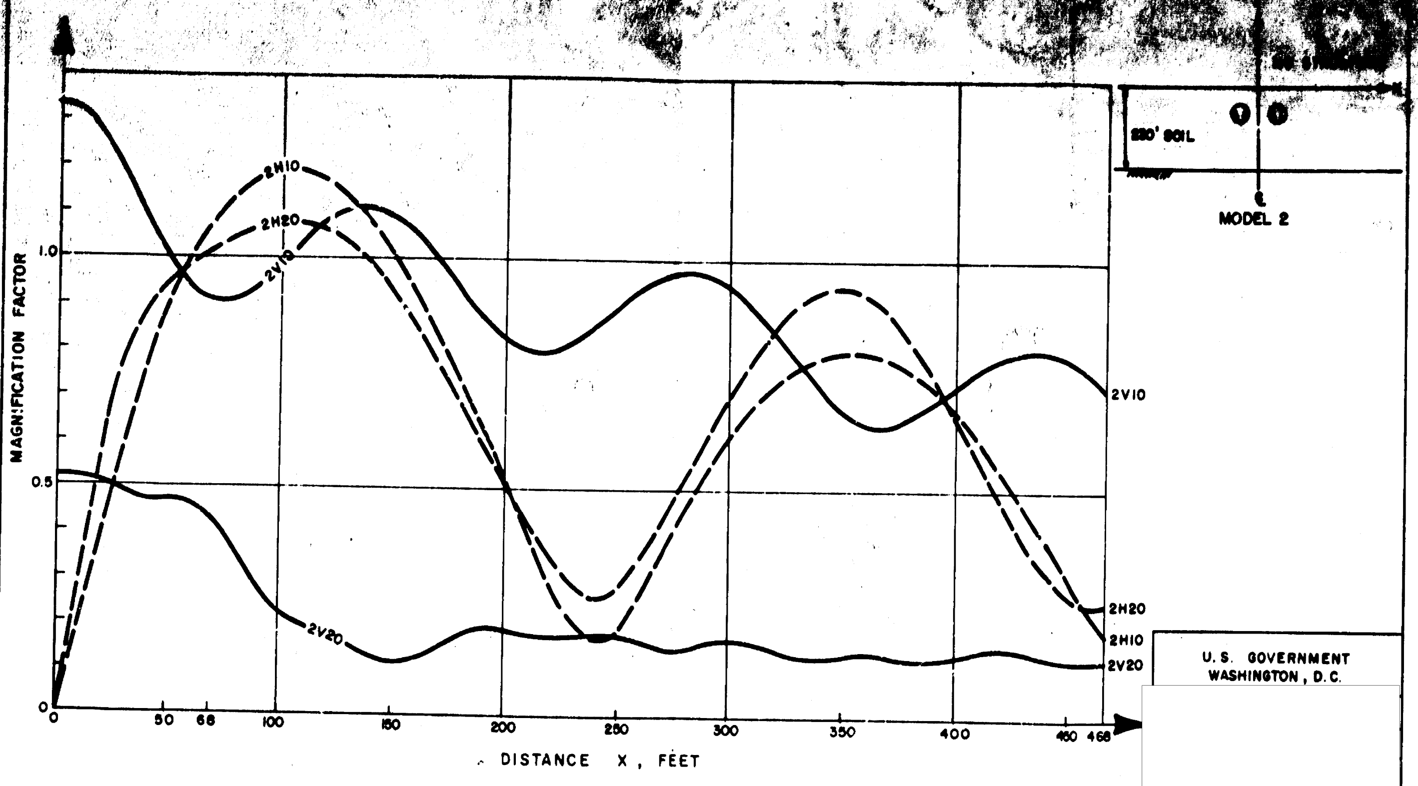
FIGURE II-20



U.S. GOVERNMENT
WASHINGTON, D.C.

STAT [redacted]

FIGURE II-21



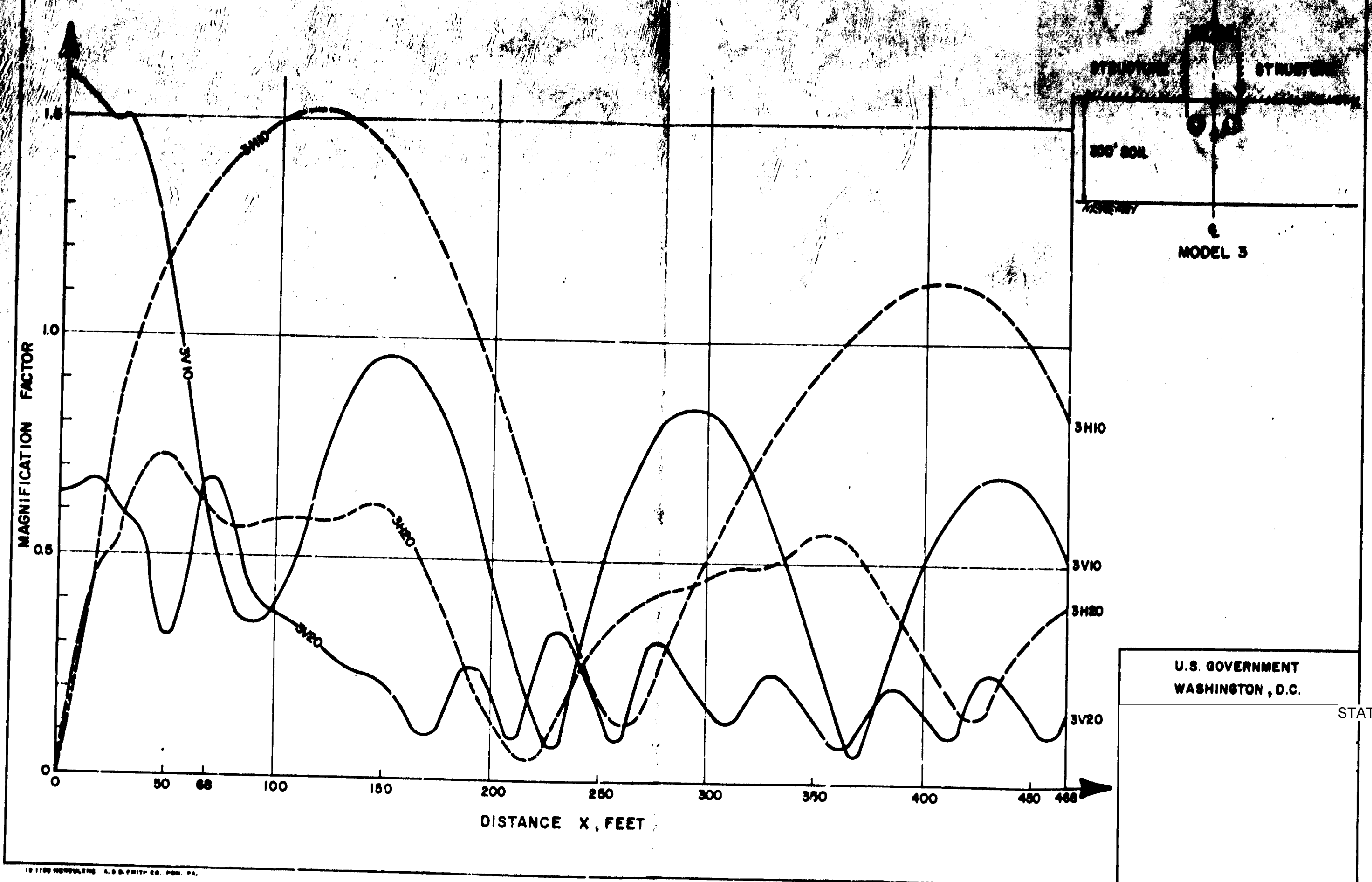
U. S. GOVERNMENT
WASHINGTON, D. C.

[redacted]

STAT

PR 9980-700

FIGURE II-22



U.S. GOVERNMENT
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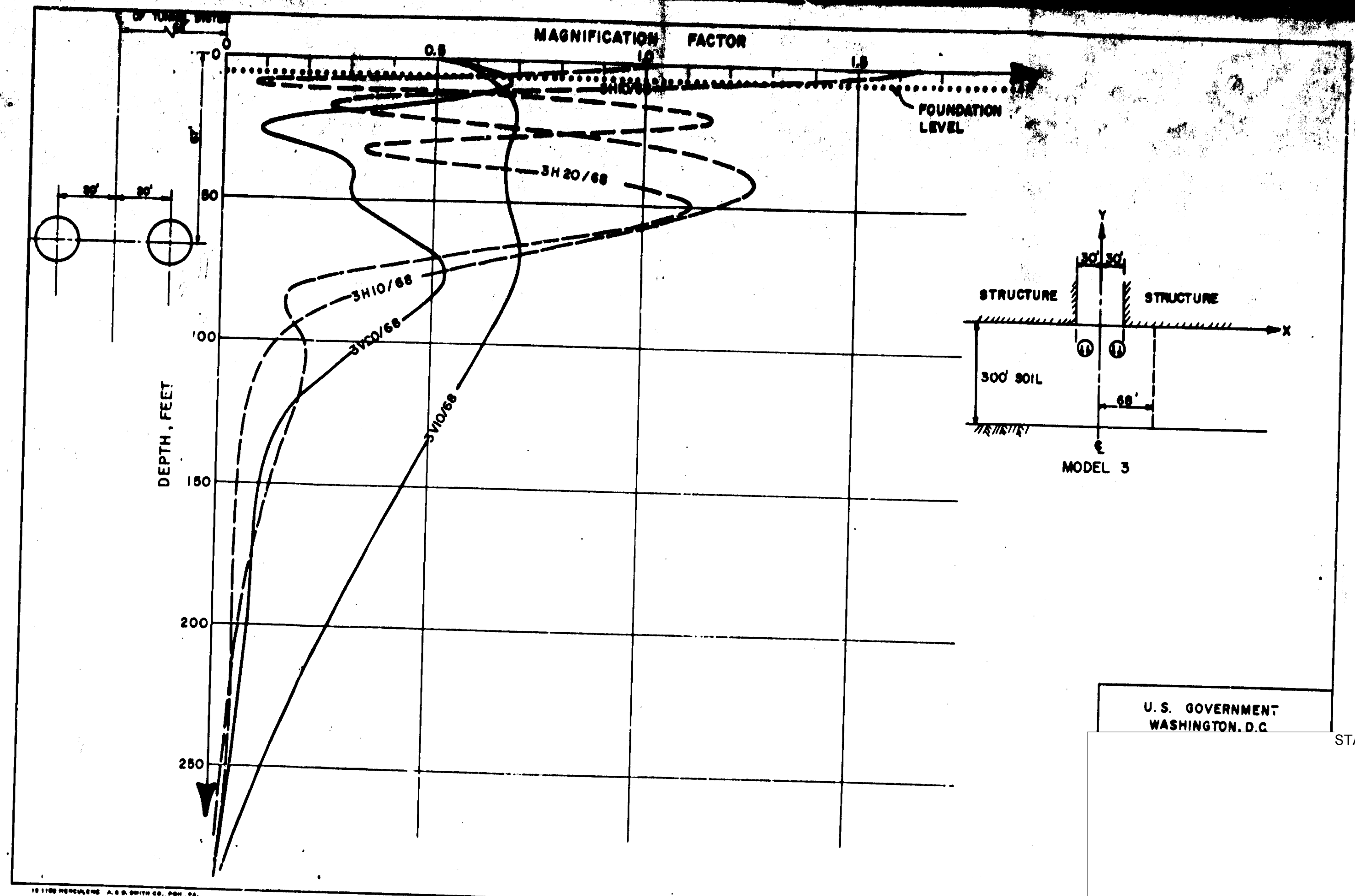
[redacted] STAT

FIGURE II-23



U.S. GOVERNMENT
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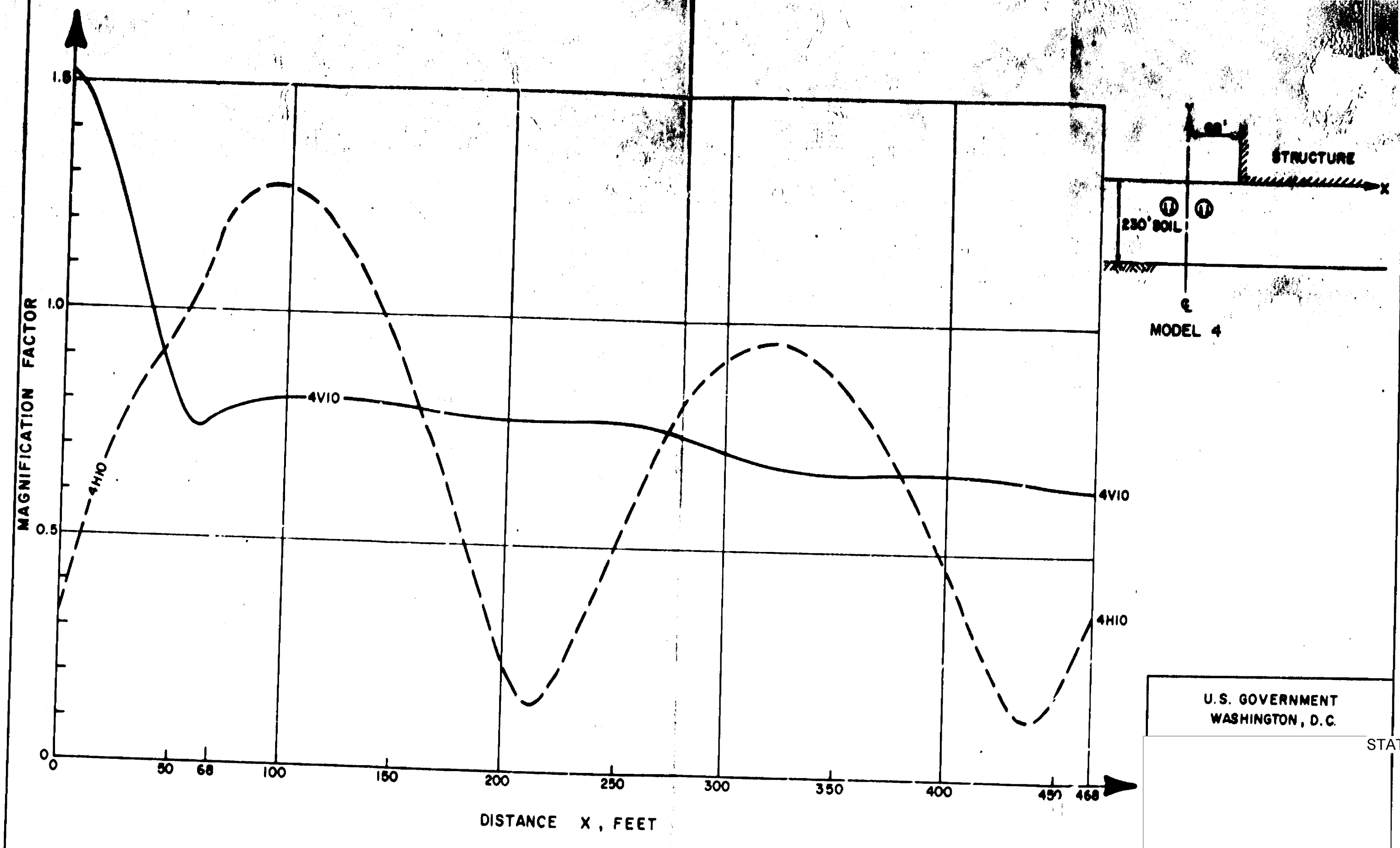
[redacted]

STAT

FIGURE II-24



U.S. GOVERNMENT
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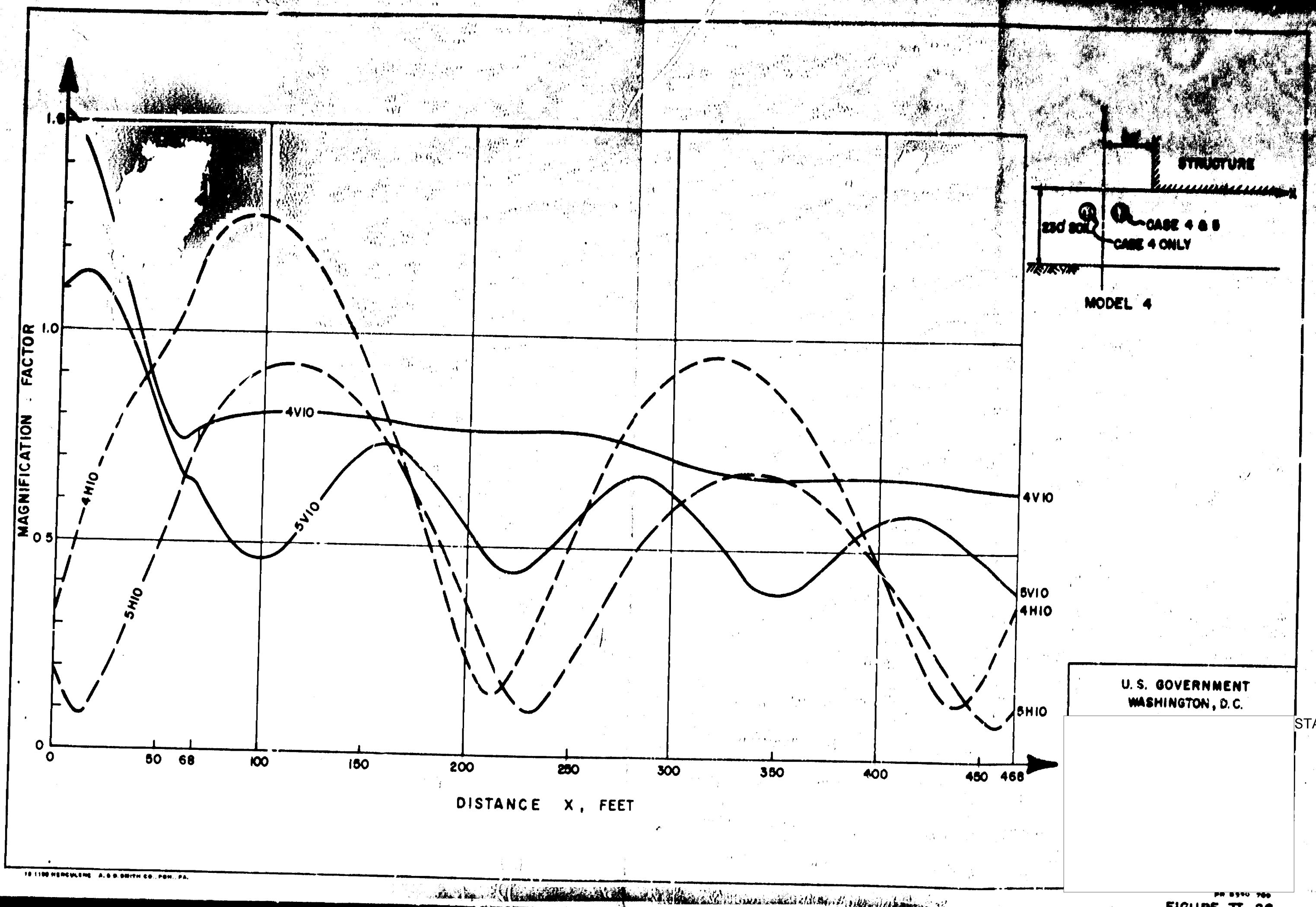
STAT

[redacted]

FIGURE II-25



U. S. GOVERNMENT
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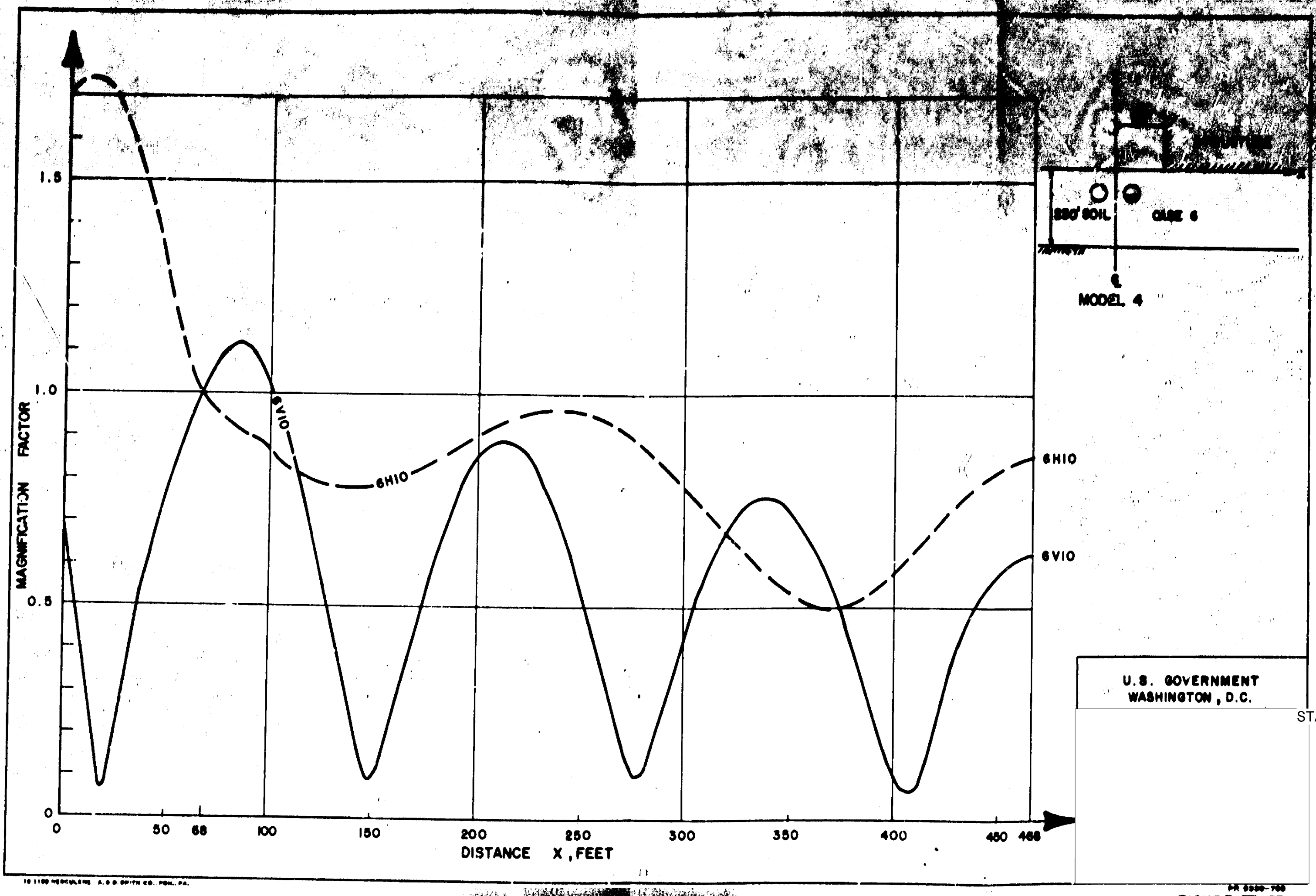
[redacted] STAT

FIGURE II-26



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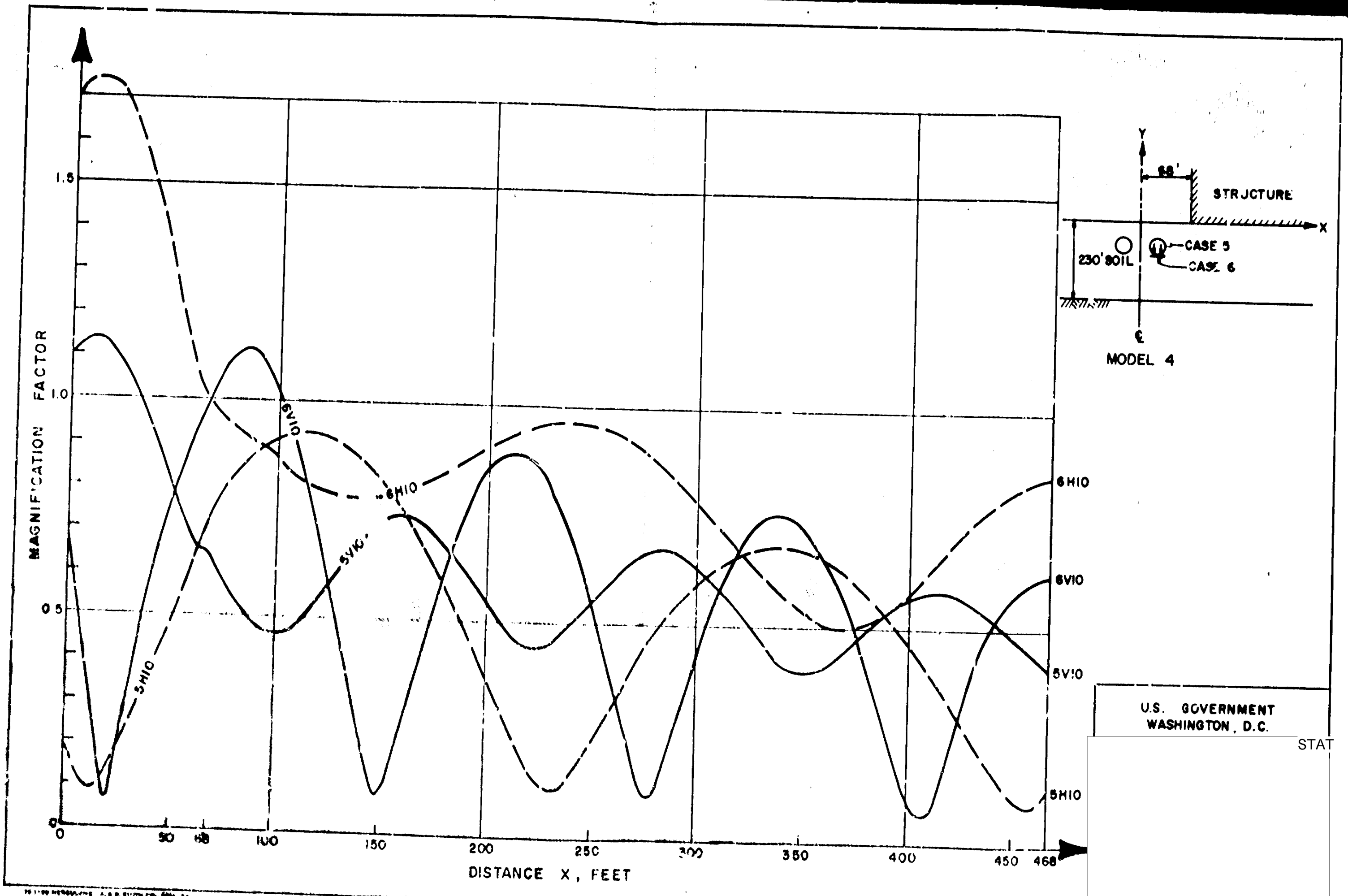
WASHINGTON, D.C.

[redacted] STAT

FIGURE II-27

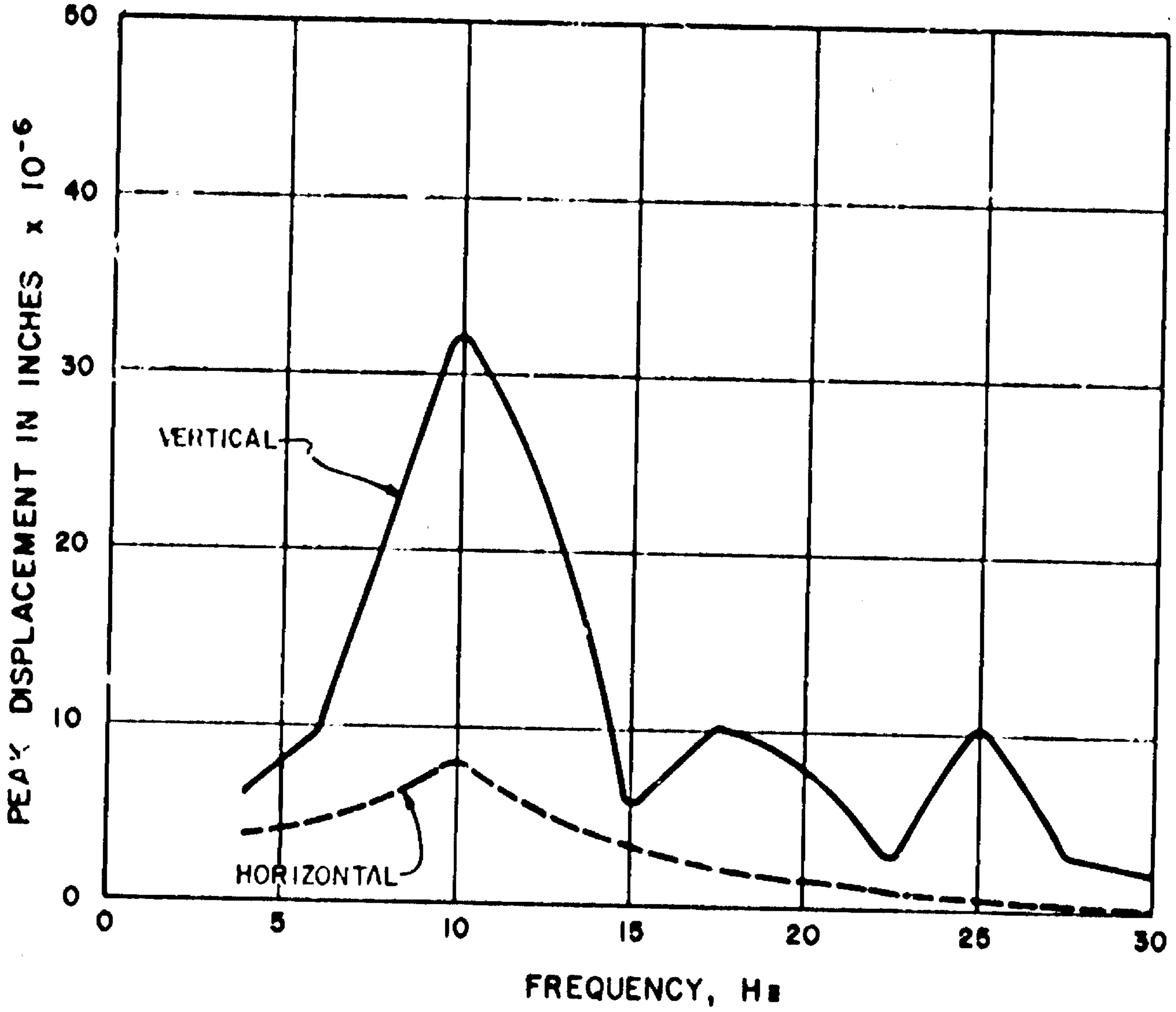


U.S. GOVERNMENT
WASHINGTON, D.C.

STAT

[redacted]

FIGURE II-28



NOTE: FOR LOCATION OF NODE 21, SEE DWG. 70-197-E6.

REV 5-24-71

[redacted]

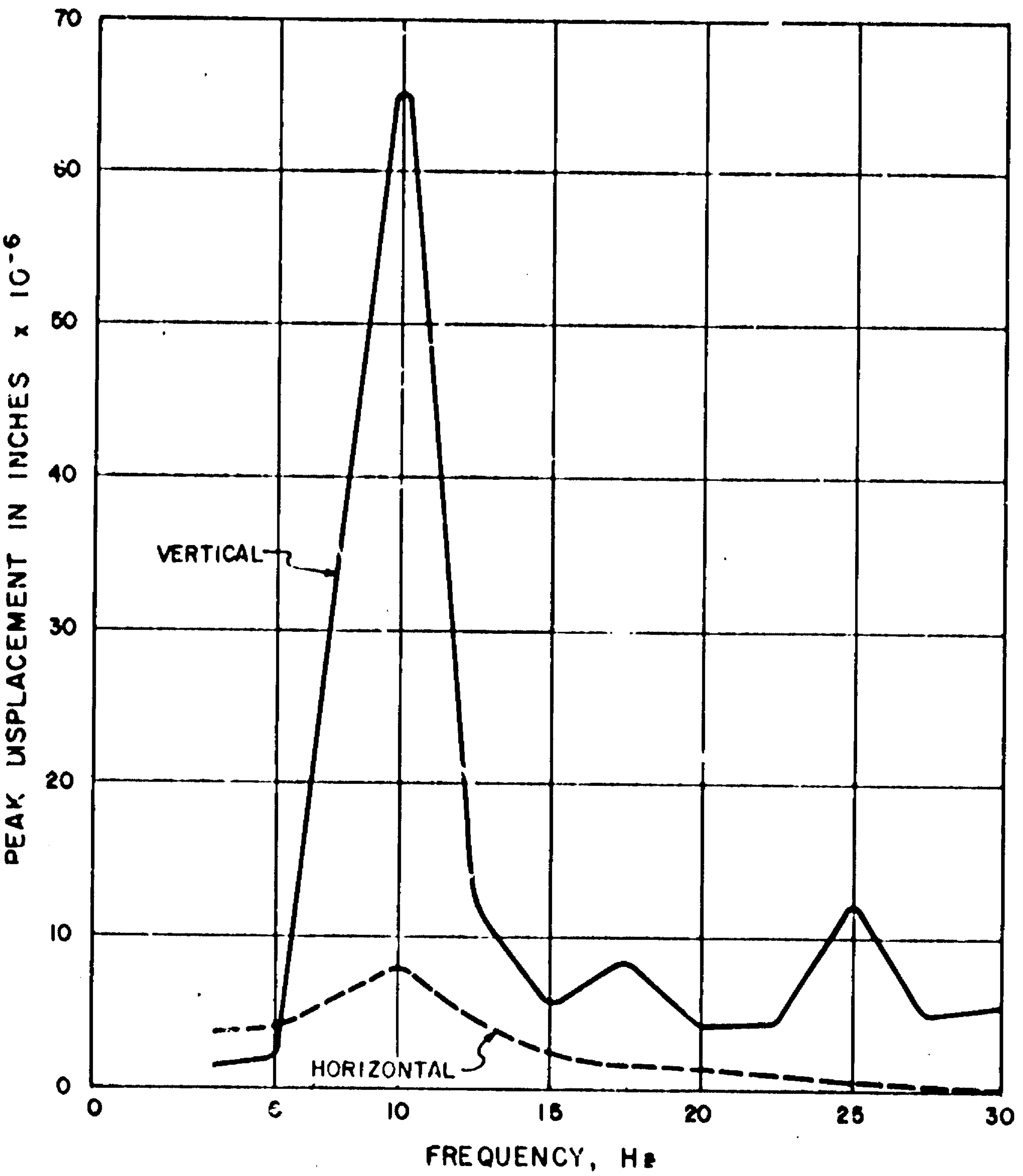
STAT

RESPONSES AT NODE 21 DUE TO
VERTICAL VIBRATIONS FROM TWO TUNNELS

U.S. GOVERNMENT
WASHINGTON, D.C.

DWN BY	RGN	5-10-71	DRAWING NO.
CKD BY	JCH	[illegible]	70-197-A7
APPR'D BY			

FIGURE II - 29



NOTE: FOR LOCATION OF NODE 47, SEE DWG. 70-197-E6.

REV. 5-24-71

[redacted]

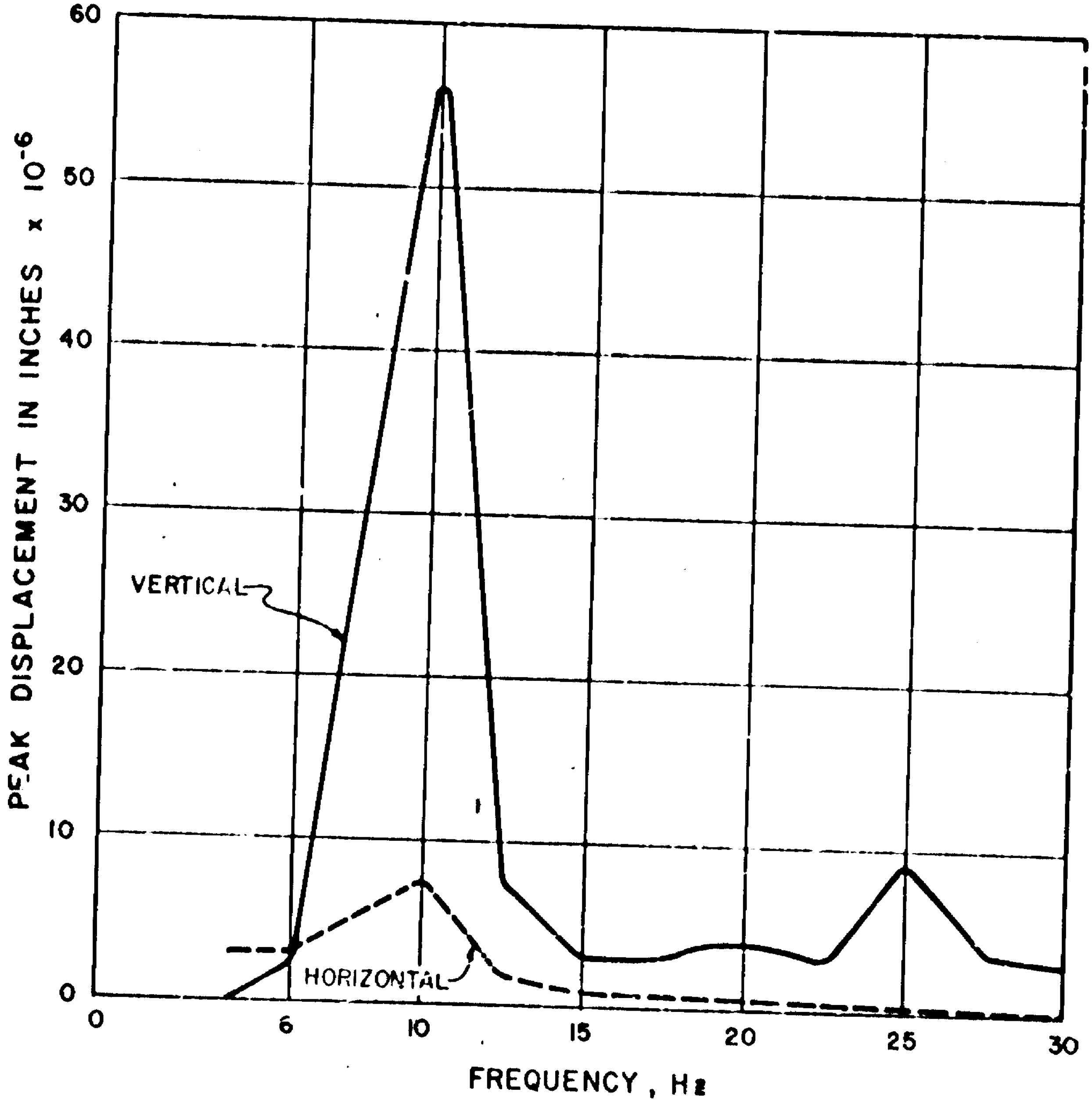
STAT

RESPONSE AT NODE 47 DUE TO VERTICAL VIBRATION FROM TWO TUNNELS

U.S. GOVERNMENT WASHINGTON, D.C.	DWN BY	RGN	5-10-71	DRAWING NO.
	CK'D BY	JAH	5-10-71	70-197-A8
	APPR'D BY			

FR 9791-870

FIGURE II - 30



NOTE: FOR LOCATION OF NODE 125, SEE DWG. 70-197-E6.

REV. 5-24-71

[redacted]

STAT

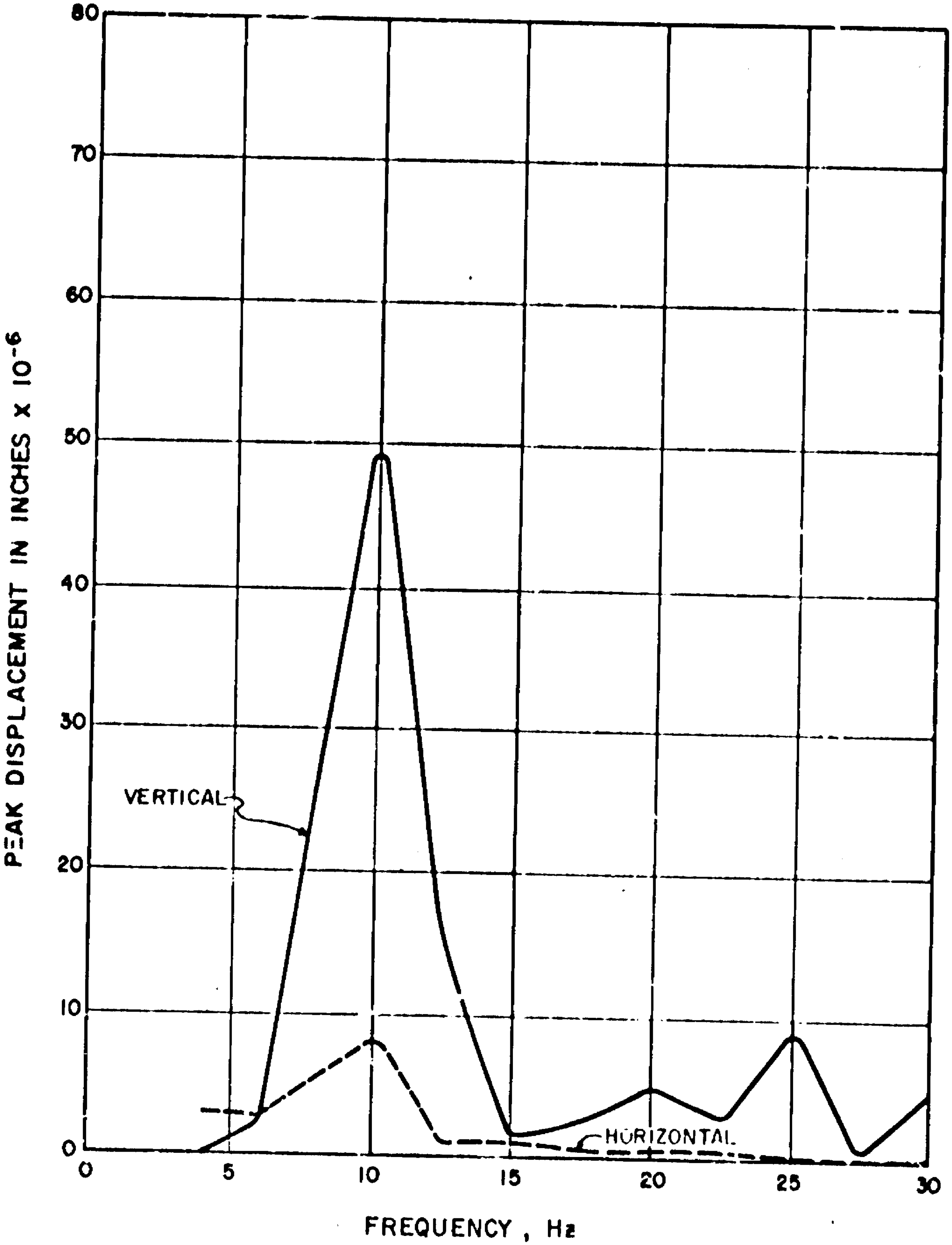
RESPONSE AT NODE 125 DUE TO VERTICAL VIBRATION FROM TWO TUNNELS

U.S. GOVERNMENT
WASHINGTON, D.C.

DWN BY	RGN	5-10-71	DRAWING NO.
CKD BY		5-10-71	70-197-A9
APPR'D BY			

PR 8781-870

FIGURE II-31



REV. 5-24-71

NOTE: FOR LOCATION OF NODE 151.
SEE DWG.70-197-E6.

STAT

[redacted]

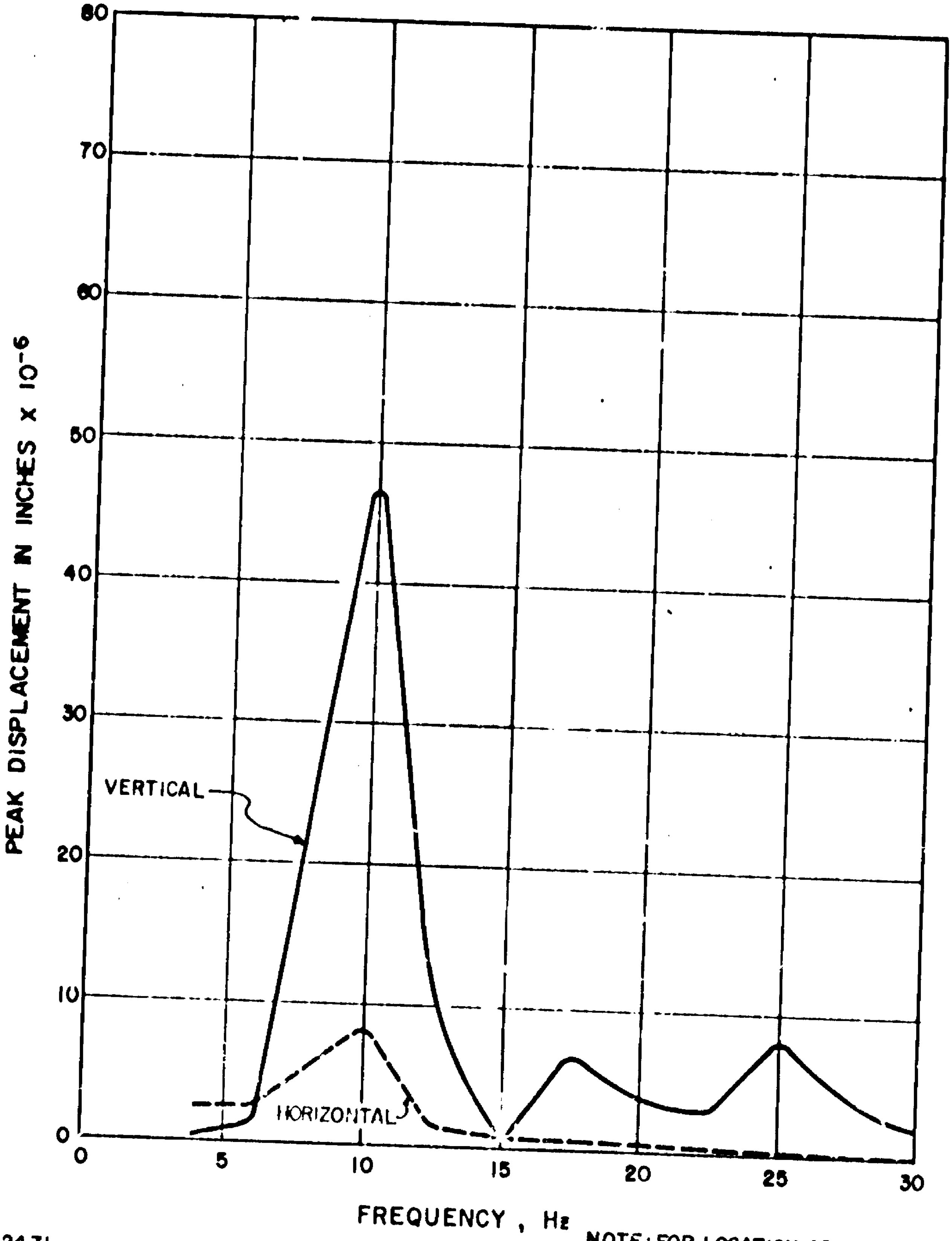
RESPONSES AT NODE 151 DUE TO VERTICAL VIBRATION FROM TWO TUNNELS

U.S. GOVERNMENT
WASHINGTON, D.C.

DWN BY	c/b	5-10-71	DRAWING NO.
CKD BY	JCH	5-10-71	70-197-A10
APPR'D BY			

FR 9791 870

FIGURE II-32



REV. 5-24-71

NOTE: FOR LOCATION OF NODE 203, SEE DWG. 70-197-E6.

[redacted]

STAT

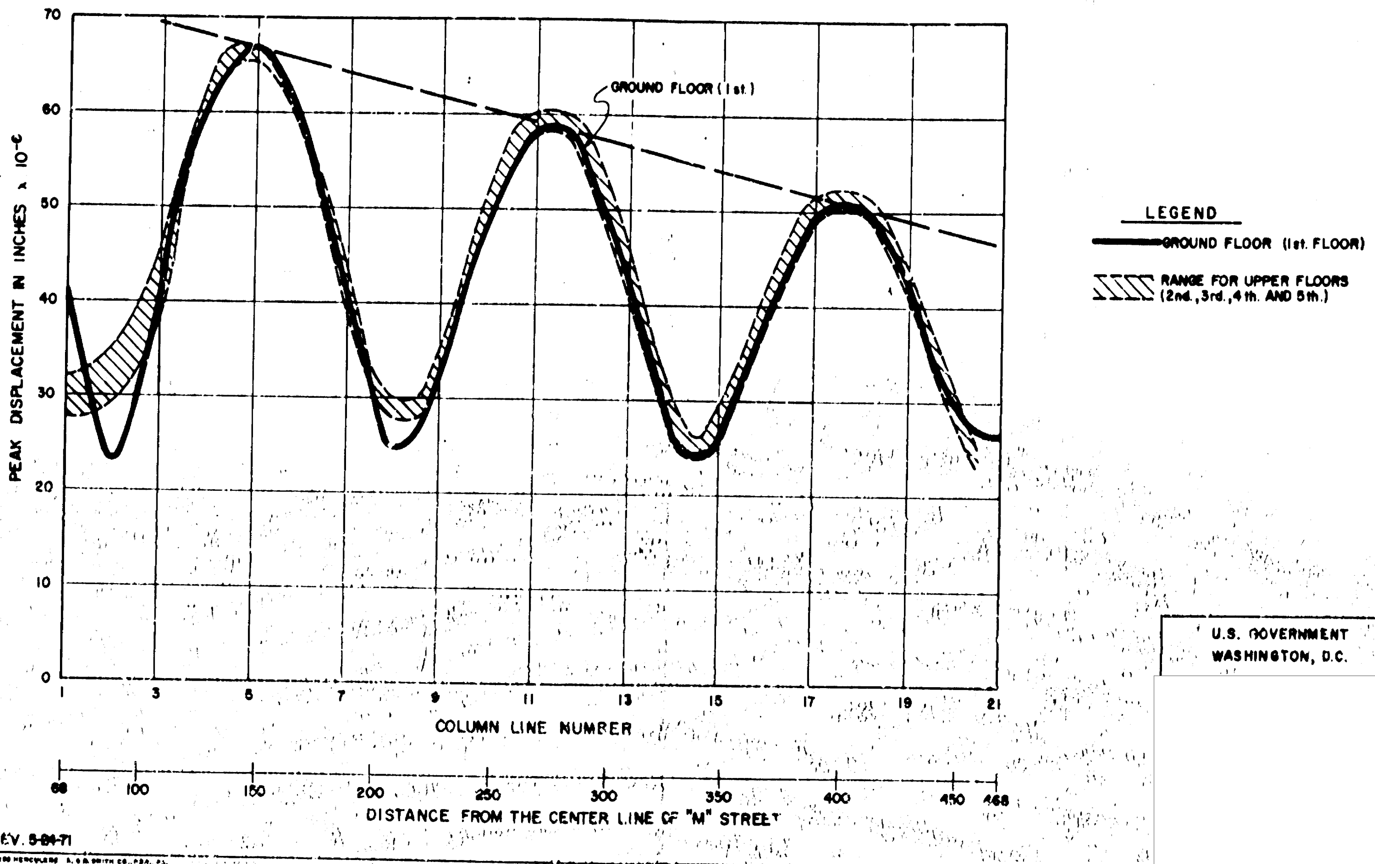
RESPONSES AT NODE 203 DUE TO VERTICAL VIBRATION FROM TWO TUNNELS

U.S. GOVERNMENT
WASHINGTON, D.C.

DWN. BY	c/b	5-10-71	DRAWING NO.
CKD. BY	JRH	5-10-71	70-197-A11
APP'D BY			

PR 8791-870

FIGURE II-33



REV. 5-24-71

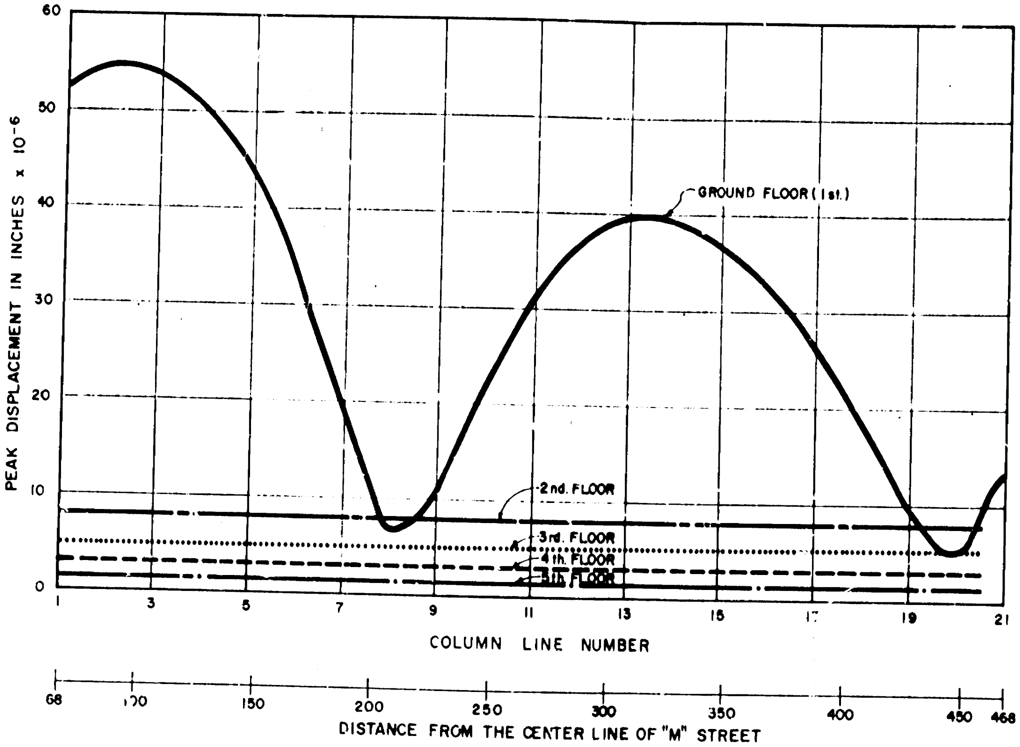
U.S. GOVERNMENT
WASHINGTON, D.C.

STAT

[redacted]

PR 9300-700

FIGURE II - 34



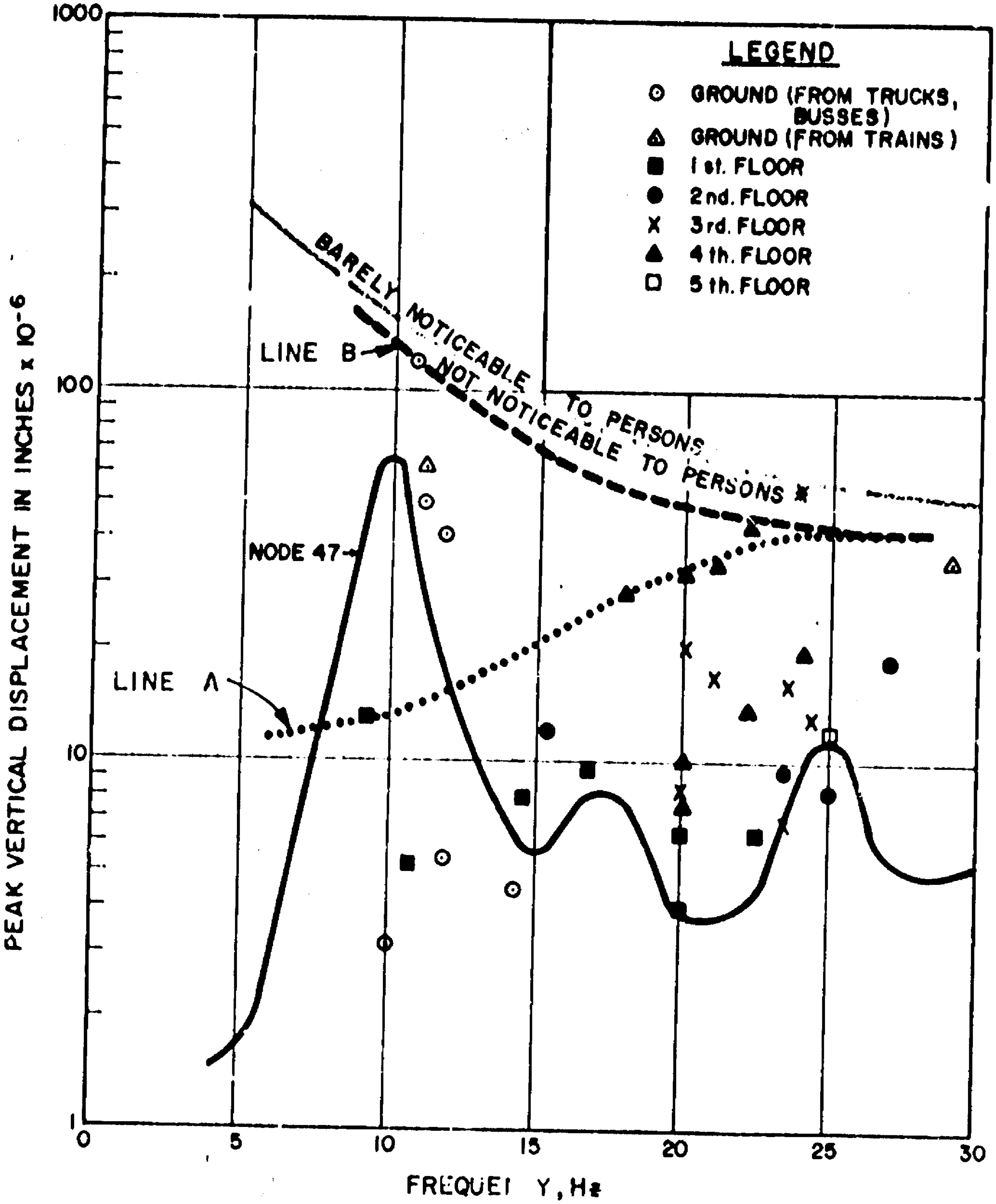
U.S. GOVERNMENT
WASHINGTON, D.C.

STAT

REV. 5-24-71

PR 9330-700

FIGURE II-35



LINE A: UPPER BOUND EXCLUDING TRUCKS, BUSSES AND TRAINS.
LINE B: UPPER BOUND INCLUDING TRUCKS, BUSSES AND TRAINS.

REFERENCE: REIHER AND MEISTER (1931)

[redacted]

REV. 5-24-71

STAT

COMPARISON OF MAXIMUM PREDICTED VERTICAL VIBRATION WITH PRESENT AMBIENT VIBRATIONS

U.S. GOVERNMENT
WASHINGTON, D.C.

DWN BY	c/b	5-11-71	DRAWING NO.
CKD BY	JRH	5/11/71	70-197-A12
APP'D BY			

FR 9791-870

FIGURE II-36