SEISMIC EFFECTS ON ELEVATED TRANSIT LINES OF THE NEW YORK CITY TRANSIT AUTHORITY

by

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PREFACE

The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 3, Lifeline Systems, and more specifically to the study of dams, bridges and infrastructures.

The safe and serviceable operation of lifeline systems such as gas, electricity, oil, water, communication and transportation networks, immediately after a severe earthquake, is of crucial importance to the welfare of the general public, and to the mitigation of seismic hazards upon society at large. The long-term goals of the lifeline study are to evaluate the seismic performance of lifeline systems in general, and to recommend measures for mitigating the societal risk arising from their failures.

In addition to the study of specific lifeline systems, such as water delivery and crude oil transmission systems, effort is directed toward the study of the behavior of dams, bridges and infrastructures under seismic conditions. Seismological and geotechnical issues, such as variation in seismic intensity from attenuation effects, faulting, liquefaction and spatial variability of soil properties are topics under investigation. These topics are shown in the figure below.

Program Elements and Tasks

Dams	Bridges	Infrastructures
• Fragility Curves • Computer Codes • Risk Assessment and Management	• Evaluate and Recommend Response Modification Factor (RMF) • Develop Probabilistic Load and Resistant Factor Design (LRDF) Format	• Inspection, Maintenance and Repair • Non-destructive Tests (NDT) and Inspection • Develop On-line System Identification Techniques (INTELAB) • Evaluate Seismic Effects on Metropolitan New York Transit Facilities

This study performed a broad assessment of the structural facilities of the New York City Transit Authority (NYCTA) system. The aim of the study was to determine those facilities, if any, which could most likely sustain significant structural damage when subjected to relatively low level seismic events which may be expected to occur in the New York City area. In this initial assessment, the types of primary structural damage considered were limited to overstressing of the moment connection at the column, bent girder joint at the top of the traverse frame, and the potential for overturning of the pedestal footings at the column base which are typical of the footings used along the lines when these are subjected to horizontal seismic motions. Based on this initial assessment, it was found that the elevated structure typical of the NYCTA system is in fact sensitive to the dynamic loads that would be imposed by such events. These structures were also found to have relatively low capacity to sustain such lateral load inputs.

ABSTRACT

The objectives of this study were to first perform a broad assessment of the structural facilities of the New York City Transit Authority (NYCTA) system to determine those facilities, if any, which could sustain significant structural damage when subjected to relatively low level seismic events which may be expected to occur in the New York City area. Based on this initial assessment, it was found that the elevated structure typical of the NYCTA system is in fact sensitive to the dynamic loads that would be imposed by such events. These structures were also found to have relatively low capacity to sustain such lateral load inputs.

The types of primary structural damage considered in this study were limited to overstressing of the moment connection at the column/bent girder joint at the top of the transverse frame and the potential for overturning of the pedestal footings at the column base. Pedestal footings are typically used along the lines and were evaluated when subjected to horizontal seismic motions. No consideration was given to other major effects which would normally be included in such a detailed safety evaluation of the train/structure system. Items such as the potential for inducing derailments from the lateral and vertical motions of the frame, or the effects of combination of the seismic load with other design loads of interest (dead and live loads) were not evaluated. Detailed safety appraisals of the seismic response of such systems would normally consider these and other important effects.

Studies were conducted for both the Jamaica and Flushing elevated lines, and include the evaluation of both single level and double level bent structures carrying a variety of train load combinations. The level of seismic shaking was varied from a relatively low magnitude of 0.05 g peak bedrock acceleration to a value of 0.15 g . 0.15 g is the peak acceleration level currently recommended in the latest edition of the Uniform Building Code for design in the New York City region. Such levels of seismic shaking correspond to postulated earthquake magnitudes in the range of about 5 to 5.5, a level currently considered reasonable for this area. These bedrock motions were then "convolved" upward through the overburden soils to obtain ground surface seismic motions compatible with these bedrock inputs. These ground surface motions were then input into the bent structures, accounting for soil-structure interaction effects, to obtain peak responses of the elevated structures. The calculations were

performed station-by-station along these elevated lines to obtain site specific evaluations.

The results indicate that, in general, little damage would occur to the elevated structures if the event postulated was limited to earthquake levels corresponding to a 0.05 g acceleration peak, although some permanent strain effects in the structure can be expected at some locations. For the higher input acceleration, significant damage to the structure can be expected, with deflections reaching magnitudes of three to four times yield. Of course, further study is required to incorporate the effects of combined loadings due to dead and live loads as well as seismic, prior to determining damage estimates and proposed plans of action to mitigate these effects.

Other types of elevated structures were originally planned to be evaluated but must await further study. The structure of most interest for any follow-on study is the bent structure located near 125th Street on the Broadway/7th Avenue Line. This facility is very tall and slender and is also most likely to be extremely sensitive to lateral dynamic load inputs. It should be pointed out that this structural assessment does not imply that other potentially serious consequences of low level seismic shaking cannot occur in the system. Items such as failure of other sensitive systems ancillary to the TA facilities (adjacent gas lines, etc.) or failure of waterproofing in deteriorating tunnels could also seriously erode the capability of the TA to provide consistent transportation service.

ACKNOWLEDGEMENT

This effort was conducted at the City University of New York (CUNY) and was supported in part by the National Center for Earthquake Engineering Research (NCEER), located at SUNY Buffalo, under contract with the Research Foundation of the City University of New York as well as by the City University of New York. Information and technical support was provided by personnel of the New York City Transit Authority (NYCTA) as well by personnel of the Department of General Services, Soil Exploration Section, of the City of New York. Appreciation is expressed to these organizations as well as to both NCEER and to CUNY for their support, financial and otherwise, in the conduct of this study. In addition to the various colleagues associated with NCEER who contributed to various aspects of the study, the authors wish to thank Messrs. W. Haid and M. Kyriacou of the NYCTA for their assistance in making information available for this evaluation.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

SECTION 1

INTRODUCTION

Early in 1986, the City University of New York (CUNY), with the support of the New York City Transit Authority (NYCTA), undertook a study to assess the impact of a potential seismic event upon their transportation system, and in particular their rail system. This situation developed from several different initiatives, such as the relatively recent growth in interest in East Coast seismicity that has appeared in the open literature (particularly by the electric power industry and the U. S. Nuclear Regulatory Commission), as well as from reports of several small seismic events that occurred in the Westchester County area just north of the City immediately prior to that time. These developments all served to indicate that the New York City area is located not in a zone of benign seismicity, but rather in one of rather low seismic hazard, which, although low, should be suitably accounted for when considering the adequacy of the design of various systems of importance to the life of the area. In recognition of this new concern, the latest recommendations contained in the Uniform Building Code (Ref. 6) upgrades the New York City environs from a zone 1 seismic definition to a zone 2a, with the peak ground acceleration (PGA) of 0.15 g's recommended for the design of new facilities.

Unfortunately, the consideration of seismic hazards has not generally been included in system designs in this region to any significant degree in the past and is generally not considered to any significant degree presently. Applicable building codes have not as yet been upgraded for this area to include a seismic component, although some interest has recently been shown by local building officials. This report presents a summary of the beginnings of such an effort for the New York City Transit

Authority. The study was supported by the National Center for Earthquake Engineering Research (NCEER), centered at SUNY Buffalo, whose stated mission is to evaluate and alleviate the hazards faced by the national infrastructure system from potential seismic events.

The principal objectives of this study have been the following:

- to establish a broad assessment of the impact of the seismic hazard on the major components of the NYCTA transportation system,

- to perform a more detailed assessment of the response of the elevated structures of the system to this hazard since it is probably the structure most susceptible to such a hazard, and

- to establish the analytical framework within which detailed evaluations can be pursued after this initial assessment is completed.

A preliminary assessment of the components of the NYCTA system, which will be described in the following paragraphs, indicated that from a structural point of view, the elevated structure was most susceptible to seismic loadings. The study, conducted at CUNY with assistance of personnel from the NYCTA, has concentrated on this aspect of the investigation only, and is the subject of this report. The overall system wide evaluation was outside the scope of this project.

However, it is clear that a significant aspect of any seismic evaluation of the system must be concerned with the definition of the major components of that system as well as components of other systems nearby whose damage may impact on its timely operation. For example, even if small seismic events may not damage the rail system directly, they can make the transportation system virtually unusable if damage would occur to, say, generally vulnerable components of the electric power supply system, or a natural gas line or primary electrical component passing nearby, or cause flooding conditions to occur in the older subway tunnels of the system by leakage from nearby water supply conduits particularly susceptible to vibration damage. These system wide aspects,although originally included in the plan for this study, are not addressed in this report.

SECTION 2
NYCTA SYSTEM COMPONENTS

The NYCTA system is extensive, consisting of about 450 route miles of trackway, most of which are located below ground in various tunnel cross-sections, or on the surface in open cut sections, but with about 75 miles on elevated structures. A plot of these various lines is shown in Figure 2-1 for four of the five boroughs of the City. The rail lines in Staten Island are not shown in this Figure. The underground sections of the system have been constructed in both hard rock and soil using various drilling, blasting and/or cut-and-cover methods of construction. River crossings are made via major bridge connections or through river tunnel sections. These tunnel sections have been either placed along the river bottoms which have been cleared of the soft silts and muds of the river or drilled under the rivers through the foundation rocks and soils. Connection to the land tunnels are then made through portal sections which form the transition between the different tunnel sections used on land and under the rivers. To support this complex system, a variety of structures are required to house the various equipment and personnel required to operate and maintain this system.

In discussing the vulnerability of NYCTA facilities to the seismic hazard, several different categories of damage potential can be defined which are useful in a first cut evaluation of the system. Considering firstly structural behavior only, these damage categories are

- catastrophic structural damage that poses an immediate danger to personnel within;
- structural damage that causes the structure to be classified as potentially unsafe;

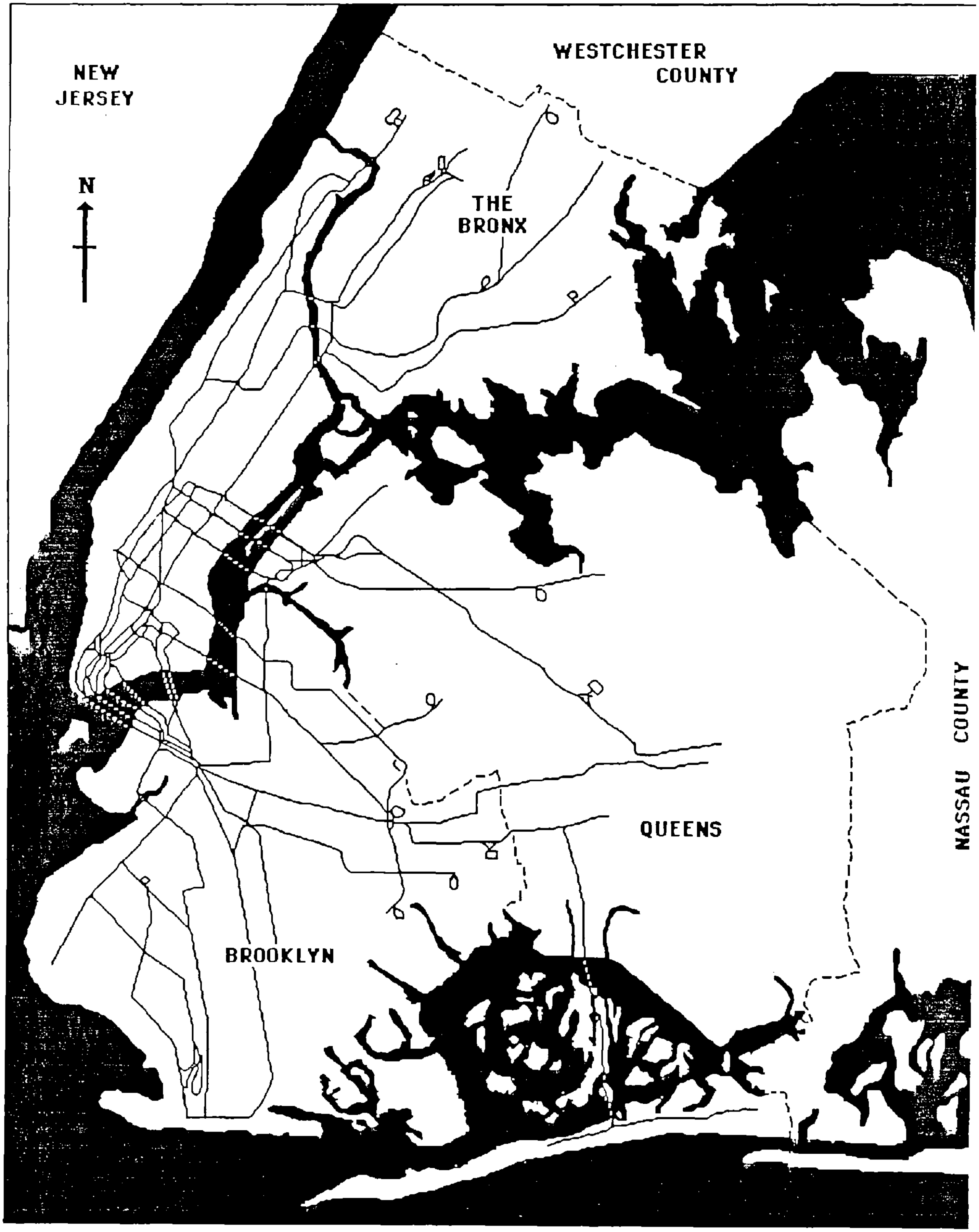


FIGURE 2-1 NYC TRANSIT AUTHORITY SUBWAY LINES

- noncatastrophic damage to the primary structures but significant damage to important subsystems within;
- noncatastrophic damage to the primary structure but serious damage to subsystems within or nearby.

With these categories of damage potential in mind, we can then proceed to assess the impact of a seismic event on the various primary facilities of the system. We have defined nine different structural types which possess fundamentally different structural response characteristics due either to their structural properties or to their method of construction. They will therefore tend to respond to seismic inputs in different ways. These generic structural types can be listed as follows:

- Cored tunnels in rock or soil
- Cut and cover tunnel sections
- River sections laid by trench methods
- River sections tunneled in place
- Track across large suspension bridges
- Small buried structures
- Surface steel structures
- Surface concrete structures
- Overhead elevated structures
- Track across small span bridges

A rather broad brush assessment of these structural types to a relatively low level earthquake input of about magnitude 5 has been made and is summarized in Table II-1. As indicated therein, it is not anticipated that such a low level seismic event will cause catastrophic damage to occur to any of the structural systems considered to date. This assessment is based on relatively crude evaluations, and assumes, of course, that the structure under consideration is performing as originally

TABLE II-1

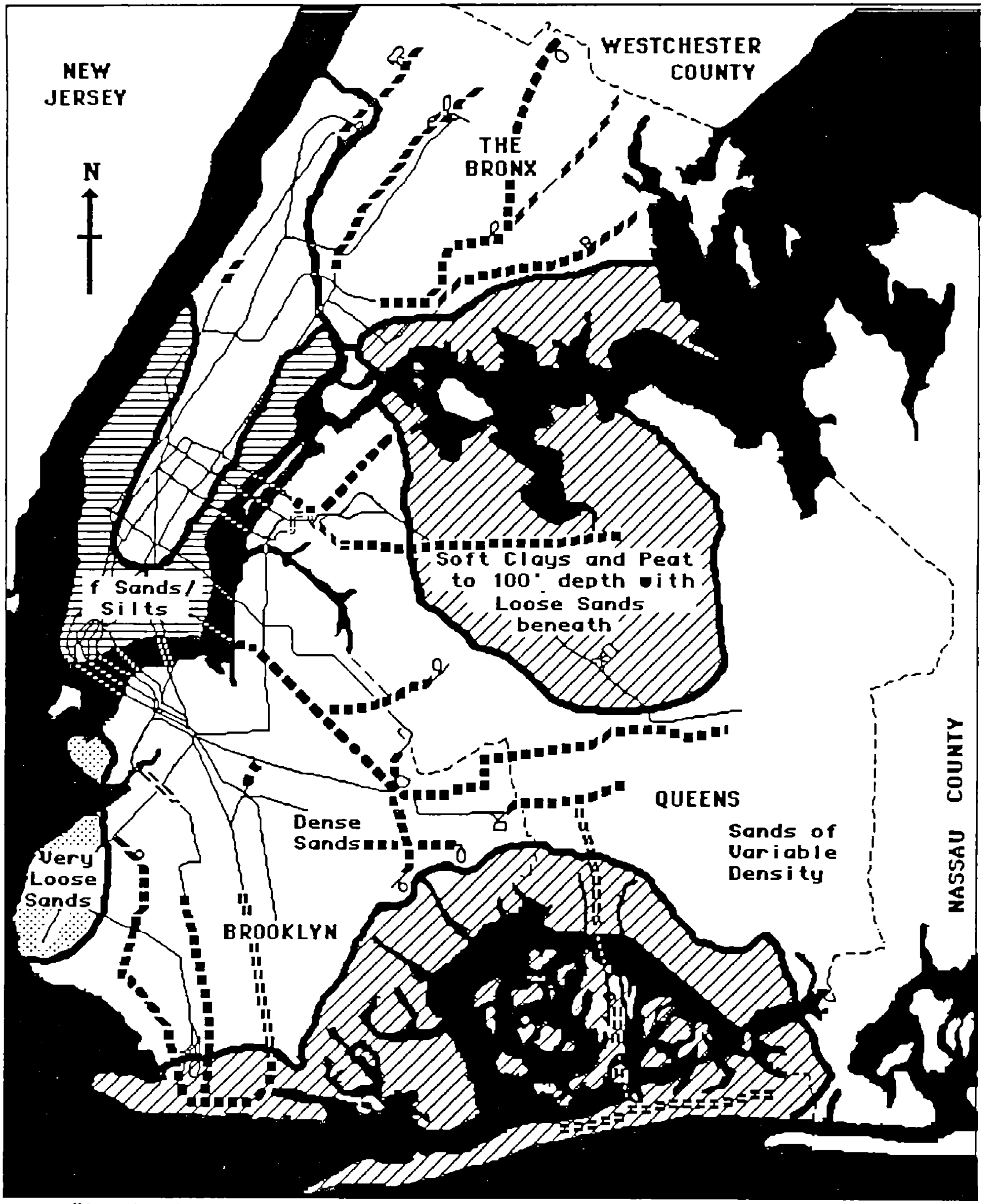
PRELIMINARY STRUCTURAL VULNERABILITY ASSESSMENT

Structural Category	Catastrophic Damage To Primary Structures	Serious Damage to Primary Structure That May Impact Safety	Subsystem Damage That May Threaten People	Subsystem Damage That May Lead To Safety Concern
Cored Tunnels in Rock	No	No	No	Yes (Controls, Water Proofing)
Cut & Cover Box Tunnels	No	No	No	Yes (Controls, Water Proofing)
River Sections	No	No	Yes (Water Proofing)	Yes (Controls)
Large Span Bridges	No	No	No	Yes (Controls)
Small Span Bridges	No	No	No	Yes (Supports)
Buried Box Structures	No	No	No	Yes (Elect. Equipment)
Steel Framed Structures	No	No	Yes (Overhead Cranes)	No
Concrete Structures	No	Yes (Flat Slab Structures)	Yes (Overhead Cranes)	No
Overhead El Structures	No	Yes (Joint Yielding)	Yes (Derail-ments)	Yes (Controls)

intended. Clearly, if significant deterioration of the system element has occurred over the years, even such a small earthquake as a magnitude 5 event may cause significant damage, particularly for those structures which do not have a significant lateral load carrying capability included in their original design.

For the overhead elevated structure, however, such low level events can cause serious damage to the primary structure. These elevated lines are designed as a two dimensional rigid frame structure in the transverse direction, connected in the longitudinal direction by individual girders supporting the train tracks. The transverse frames are supported on simple pedestal foundation elements located at the ground surface, resting either directly on the ground or on pile clusters. Most often, the elevated line supports one trackway level with three separate tracks, although some two level structures exist. To underscore the methods used for the structural evaluation of this system, the elevated structure for two specific lines of the system were evaluated and are the subject of this report.

A plot of the locations of these elevated lines within the NYCTA system is shown in Figure 2-2, together with general descriptions of the foundation conditions which can be anticipated at the various areas of the City. As will be described in the following paragraphs, the foundation conditions play a major role in altering the seismic motions which can be anticipated at a given location, even for a specified uniform bedrock input motion. The correspondence of the properties of the soil overburden with the properties of the surface structure then determine if significant damage will or will not occur during a given event.



■■■■ Elevated Lines

==== Embankments or Viaducts

FIGURE 2-2 ELEVATED LINES IN NYC TRANSIT AREA

SECTION 3

GENERAL FOUNDATION CONDITIONS IN THE METROPOLITAN AREA

As mentioned above, the foundation conditions encountered in the different areas of NYC are extremely variable, and must be included when evaluating response of either surface or buried structures. Several areas of significant interest to the NYCTA system are shown on the map of Figure 2-2, while the depths to bedrock in various parts of the City are shown in Figure 3-1. At the west end of Brooklyn, the upper soils encountered typically consist of very loose sands with standard penetration resistance (SPT) sample blow counts consistently less than 10. Much of this area has been reclaimed from the Bay by simple filling methods, usually with little attention paid to the state of the compaction of these soils. As can be imagined, these saturated loose sands are extremely susceptible to any vibratory loadings, particularly those associated with a seismic event. These loose soils can either lose their support capacity through the phenomena associated with soil liquefaction or could consolidate during shaking leading to significant settlements. Experiences with vibratory pile driving in this area of Brooklyn indicate that these soils are very susceptible to ground shaking, which could lead to large settlements and loss of capacity from even low levels of shaking. Such behavior indicates that structures founded on these soils, as are the two to four story residences typical in the area, would be susceptible to major damage for even mild seismic events. Away from this region, the soils in the Brooklyn area are for the most part dense sands and gravels with correspondingly high SPT blow counts and thus not particularly susceptible to loss of capacity during a seismic event. At the southern end of the borough, these sands extend to a depth of several hundred feet.

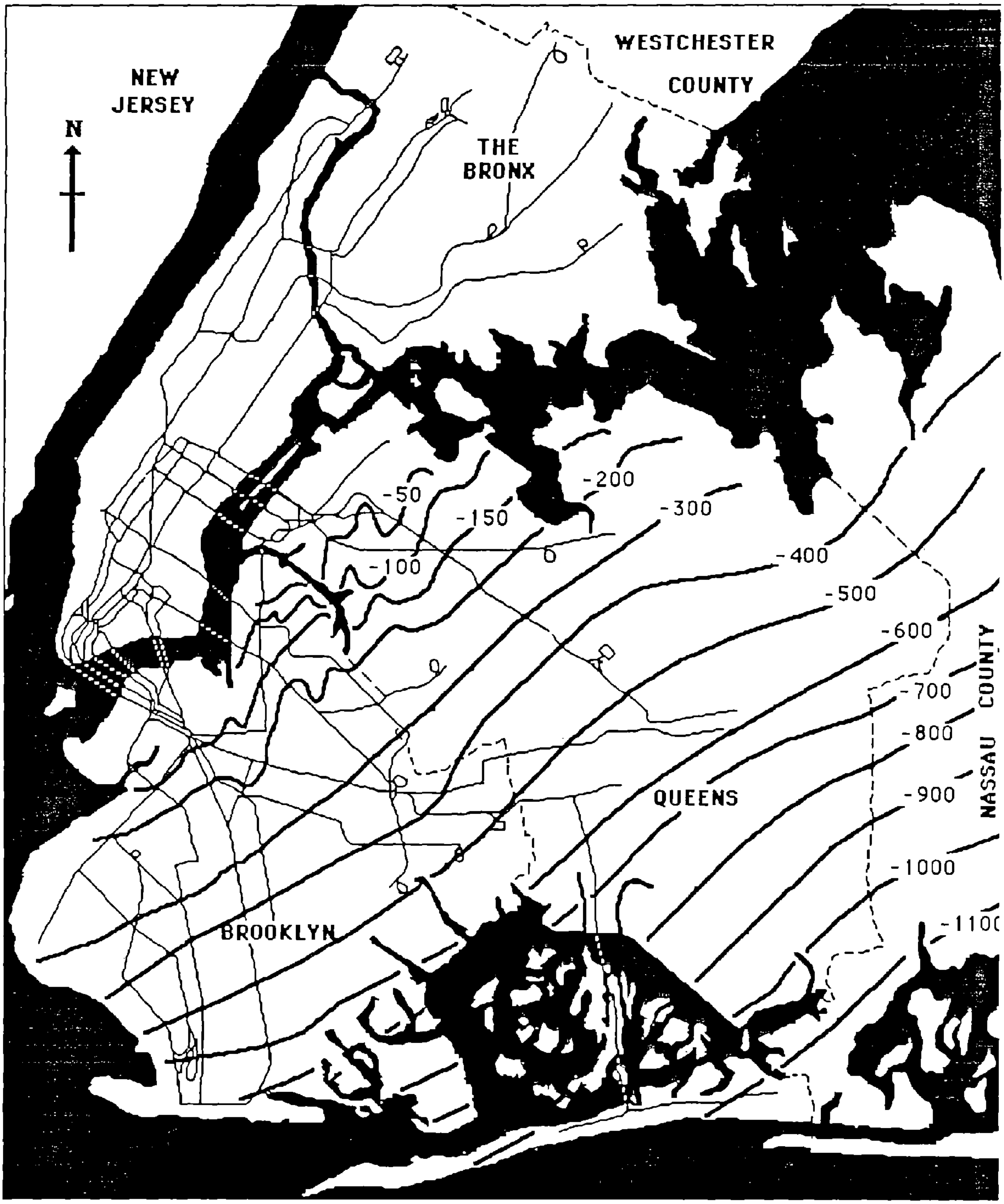


FIGURE 3-1 APPROXIMATE BEDROCK ELEVATIONS (MHW AT 0)

In both Queens and the Bronx, alongside the Long Island Sound, the upper soils are extremely soft silts, clays and peats (in some cases underconsolidated) which can extend to great depths. In these areas, the ground water table is near the ground surface. The depth of these soft soils can extend to as much as 200 feet. In addition, below these soft fine-grained soils exist fine sands which are of extremely variable density, with SPT blow counts, even at these great depths, varying from zero to about 30 bpf. Thus, these lower sandy soils may behave peculiarly during a low level seismic event. Since major structures in the area are often supported on friction piles driven to these sands for support, these structures may be affected by low level events. As will be discussed in a later section, the soft clays play a major role in modifying the ground motions which would be felt at the ground surface, given a particular motion at the bedrock. This in turn will influence the response of surface supported structures. In the Bronx, the soft silt/clays are typically not as thick as in Queens and are underlain by highly fractured bedrock.

In the remainder of both the Bronx and Manhattan, the vast majority of the area indicates bedrock at or near the ground surface, with the exception of relatively narrow zones along both river banks and shore areas. In these zones, the soils are again found to be highly variable, with silty soils of variable density being prevalent.

SECTION 4
SEISMIC HAZARD DEFINITION

The definition of the seismic hazard which must be included in any system evaluation for the eastern U.S (and in particular for New York City) has not as yet been completely defined, since major questions still remain about the specific seismogenic processes controlling in this region. Recent work in this area (Refs. 1 - 4) indicates, however, that this hazard is significant and if anything is being revised upward (Refs. 6 - 8). A summary of the seismic history in this area, presented in Ref. 3, indicates that several earthquakes in the range of magnitude 4 to 5 were centered in the NYC area or its immediate vicinity, with the latest being recorded in 1884. This event was centered near the mouth of New York harbor and caused minor structural damage throughout a region 200 km wide, from western Connecticut to eastern Pennsylvania. The impact of such an event on the current extensive and crowded facilities in the area has not been evaluated as yet. Seismicity studies performed for the licensing application of the Indian Point Nuclear Power Plant (Ref. 1) indicate that such low level events have a return period of from 50 to 100 years for earthquakes of magnitude 5 or thereabouts, based on a historical record of up to 250 years. Thus, an event of magnitude 5 is certainly a reasonable one for consideration as a lower bound estimate of the hazard that should be used for design purposes in this area.

The historical record is not as yet sufficient to establish a reasonable upper bound estimate of the seismic hazard which can be used for design purposes. Some discussion can be found in the literature (e.g., Ref. 10) which indicates that the large Charleston earthquake of 1886, with an estimated magnitude of about 7.3, occurred on an as yet unspecified fault, and, more importantly, that all such fault structures on the east coast should be considered capable of supporting such large earthquakes. However, in the absence of specific strong motion data for large earthquakes on the eastern seaboard, interpretation of strong motion amplitudes (or peak ground accelerations) must rely on the use of intensity data obtained elsewhere. Typically, almost all the available ground motion - intensity data have been obtained in tectonically active regions of the world that are usually characterized by far greater absorption of elastic wave energy than is expected in the eastern United States. This conclusion is reached by studying low amplitude seismographic data. Therefore, it is indicated that large earthquakes will have felt areas significantly larger on the east coast than those associated with seismically active zones. These larger, more distant events can be expected to produce local damaging ground motions of longer duration than the smaller, closer events.

4.1 Bedrock Motion Definitions

A lower bound hazard definition of a magnitude 5 to 5.5 seismic event can be associated with peak bedrock or outcrop accelerations which can reach values of as much as 0.2 g's. For example, data from 678 world earthquake records (Ref. 11), ranging in magnitude from less than 5 to greater than 8, was analyzed by Donovan to obtain approximate relations between peak ground shaking, earthquake magnitude and distance from the epicenter to the site. These attenuation relationships indicate that depending upon the distance of the epicenter to the ground surface, zones of peak shaking can be expected to reach an intensity range of from VI to VII, based on the definitions of the Modified Mercalli scale. Such a level of shaking is typically associated with some structural damage to unreinforced masonry and/or stone structures, some minor effects on soil slopes, etc., but no significant damage to relatively flexible structures which are already designed to resist a lateral wind load component, except for possibly some facade damage. However, even flexible structures which are not designed to withstand a lateral load component may be expected to incur some serious damage. Peak ground displacements for such events can also be expected to reach values of about 1.5 to 2.0 inches.

However, the peak ground displacement and acceleration levels sustained at sites where significant soil overburden is located can be expected to vary significantly from these approximate values. The existence of the soil modifies the characteristics of the surface motion, changing its frequency content, peak acceleration levels and even pulse durations. Significant study of this topic of site amplification has occurred in recent years which indicates that, although the process is still complicated except for the simplest of cases, it is clear from observations that the type of soil or subsoil has a major influence on the surface ground motions that are recorded. The controlling characteristics of the soil generally are its stiffness and damping parameters as well as its thickness to bedrock. The classic example of this motion modification has been the recent Mexico City event, in which relatively high frequency motions were transmitted through the basement bedrock for significant distances, which were in turn transformed upward at deep soil sites to very low frequency, long duration motions which caused considerable damage to structures founded in these soils. In general, for a given earthquake, where the local intensity of shaking is low, the measured accelerations can be expected to be higher on sediments than on rock. However, for higher intensity shaking associated with earthquakes of higher magnitude, the reverse can be expected, with the peak accelerations measured on sediments being less than the corresponding rock outcrop motions.

For the New York City area, the 1988 version of the Uniform Building Code (Ref. 6) now recommends a peak ground acceleration of 0.15 g's for input to any seismic hazard assessment, which is a significant upgrade in the size of the seismic hazard currently felt to be realistic for design of structures in this area. This peak ground acceleration level is defined as a bedrock or outcrop motion. The UBC recommends an additional parameter to modify this acceleration depending upon general site characteristics. For this study, a relatively broad-banded seismic input was used as the definition of the bedrock seismic motion, with a peak acceleration varying from 0.05 g's to 0.15 g's, that is, peak accelerations which cover the range of inputs corresponding to a magnitude 5 to 5.5 event. The acceleration-time history associated with this acceleration level was selected to possess a wide range of frequencies, from about 0.5 to 20 hz, which covers the range of interest for most structures. It should be noted that the accelerogram selected for the study envelopes the recommended design response spectrum currently used by the U.S. Nuclear Regulatory Commission to define seismic input to safety analyses of nuclear power plants (Ref. 5).

This response spectrum was generated from studies of strong motion recordings taken from events of significantly larger magnitude than considered herein. The time duration of this pulse is longer than would normally be associated with a magnitude 5 event, since it contains lower frequency content included for safety in the structural analysis. Such long duration pulses may develop, even for low magnitude events, when deep soft soil overlays bedrock. This type of soil configuration exists in the NYC area. The pulse duration plays a role in assessing nonlinear response or amount of damage that will occur to the surface structures of interest to this study. Further study of this aspect of the seismic hazard definition must be included if additional evaluations are to be conducted.

4.2 Ground Surface Motion Inputs

To assess the impact of the variable soil conditions that exist throughout the metropolitan area on response of structures founded at the ground surface, a series of convolution analyses were performed for the different soil conditions known to exist at the sites of interest. The concept of convolution is indicated schematically in Figure 4-1. The specific accelerogram described above is used as the horizontal seismic input to the lower bedrock surface. This motion is then allowed to propagate upward, through the overburden, or soil column, and a modified ground surface time history is determined, which is specific to the site of interest at a given location. As can be anticipated, the soil overburden significantly modifies the ground motion which was input at the bedrock and which is sustained by the surface structure. This in turn may have a significant impact on its response and susceptibility to damage. Considering the ground surface response at or near the primary frequency of the soil column, the input motion can be expected to be significantly amplified, while motions at higher frequencies can be expected to be reduced, depending upon the specific properties of the soil. At frequencies much less than the soil layer frequency, the surface motions will be essentially unaffected.

The impact of the soil overburden can play a major role in defining damage potential to surface structures. For example, all studies of the recent Mexico City earthquake indicate that at sites with deep soft soil columns, the role of the soil was to

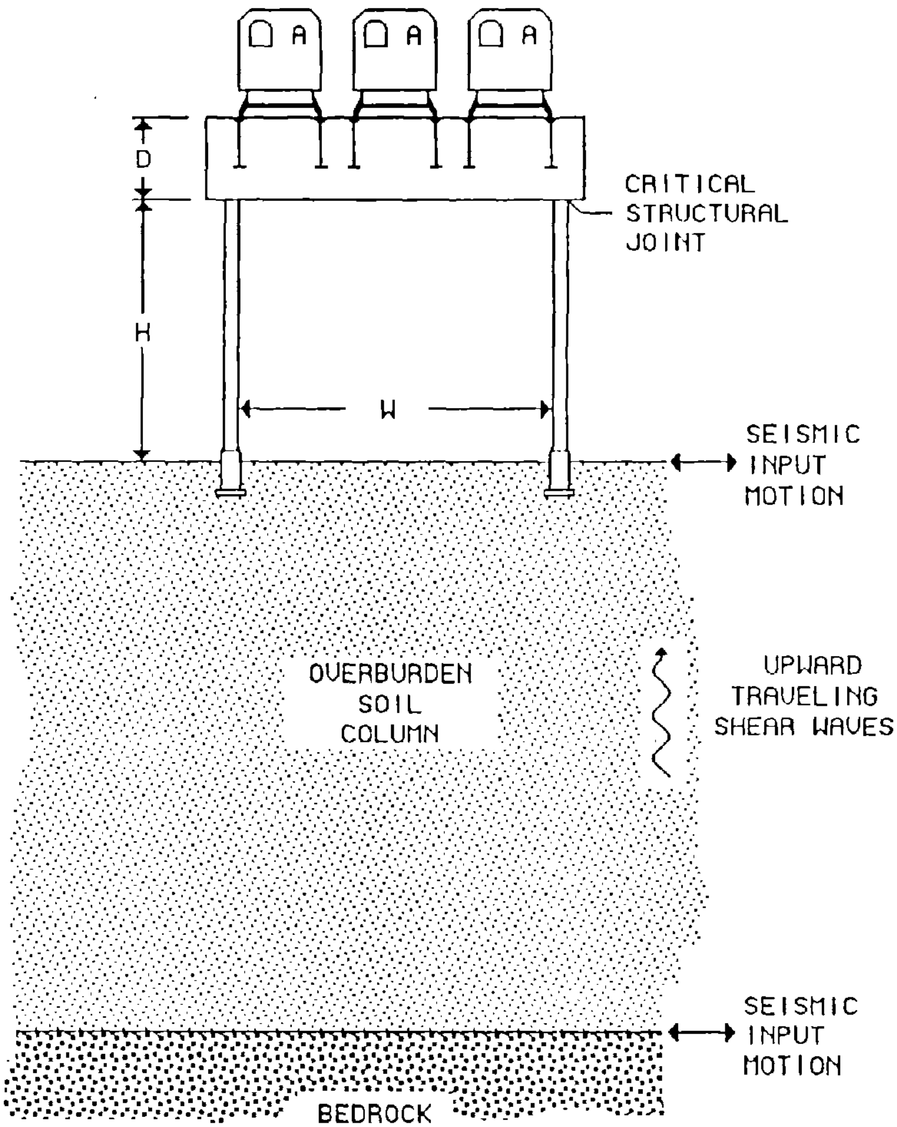


FIGURE 4-1 TYPICAL CONFIGURATION FOR ANALYZING ELEVATED STRUCTURES

magnify surface response at frequencies of about 0.5 to 1.0 cps, just those frequencies which corresponded to building natural frequencies. The event then caused significant damage to such low frequency surface structures at these locations. At other locations where the soil column was not as deep or as soft, such magnitudes of damage did not occur.

In fairness, it should be mentioned that this simplified concept of upward propagating shear waves, although attractive due to its simplicity, may not be representative of the actual processes through which surface motions are developed from basement bedrock motions, and in some cases is contrary to observations. For example, it is noted by Hanks in Ref. 12 that for the lower frequency motion components, where the frequencies are one hz or lower, the surface motions are due primarily to Rayleigh or Love surface wave motions and not to upward propagating shear waves. Even for higher frequency motions of interest to structural engineers, surface waves may have an important effect on the resulting response at the ground surface. In addition, this simplified approach does not suitably incorporate the potential effects of the soil column on vertical motions developed at the ground surface. Thus, more complex motions must be considered other than those indicated by this simplified concept. On the other hand, measurements taken at some instrumented locations (Refs. 13 through 15) indicate that the general concept of the upward traveling shear wave may be a reasonable approach to determining the gross characteristics of the effects of the soil overburden on surface wave motions.

As part of this study, therefore, where specific responses of elevated structures were to be calculated, surface ground motions were computed to determine the modified ground accelerograms which were then to be input into the structures.

The procedure used is termed a "convolution analysis", and assumes that the seismic bedrock motions are propagated upward through the soil column via horizontal shear waves whose magnitudes are frequency dependent. The computation is a standard one in seismic response analysis, but requires detailed computer analysis at each location of interest. Corollary to this computation, however, is the fact that at specific locations along each elevated line of interest, specific information had to be developed to properly define the overburden soil configuration together with its properties of interest. Therefore a detailed soil data base had to be developed for this study from boring information available from a variety of sources to suitably define the properties of the soil overburden at the various locations of interest. Soil boring data was obtained primarily from two agencies, the NYCTA and the Department of General Services, Soil Exploration Section. The boring logs were used to obtain standard penetration data, soil descriptions and depth to bedrock along both the Flushing and Jamaica Lines.

An example of the accelerogram used as input to the bedrock at the bottom of a soil column is shown in Figure 4-2a. This particular time history has a peak acceleration of 0.05 g's and duration of 20 seconds, although the period of primary shaking lasts only about 10 seconds. The accelerogram is a typical one, indicating a gradual buildup to the period of strong shaking, followed by a decay to low level motions which may continue for significant durations. From a structural damage point of view, the period from about 3 seconds to 13 seconds is the interval of primary interest as being the period for causing damage to the structure. The 5% damped response spectrum of this motion is shown in Figure 4-2b and indicates the relatively broad banded nature of the motion, that is, significant energy exists in the motion in the 1 hz to 10 hz range of most interest to structural

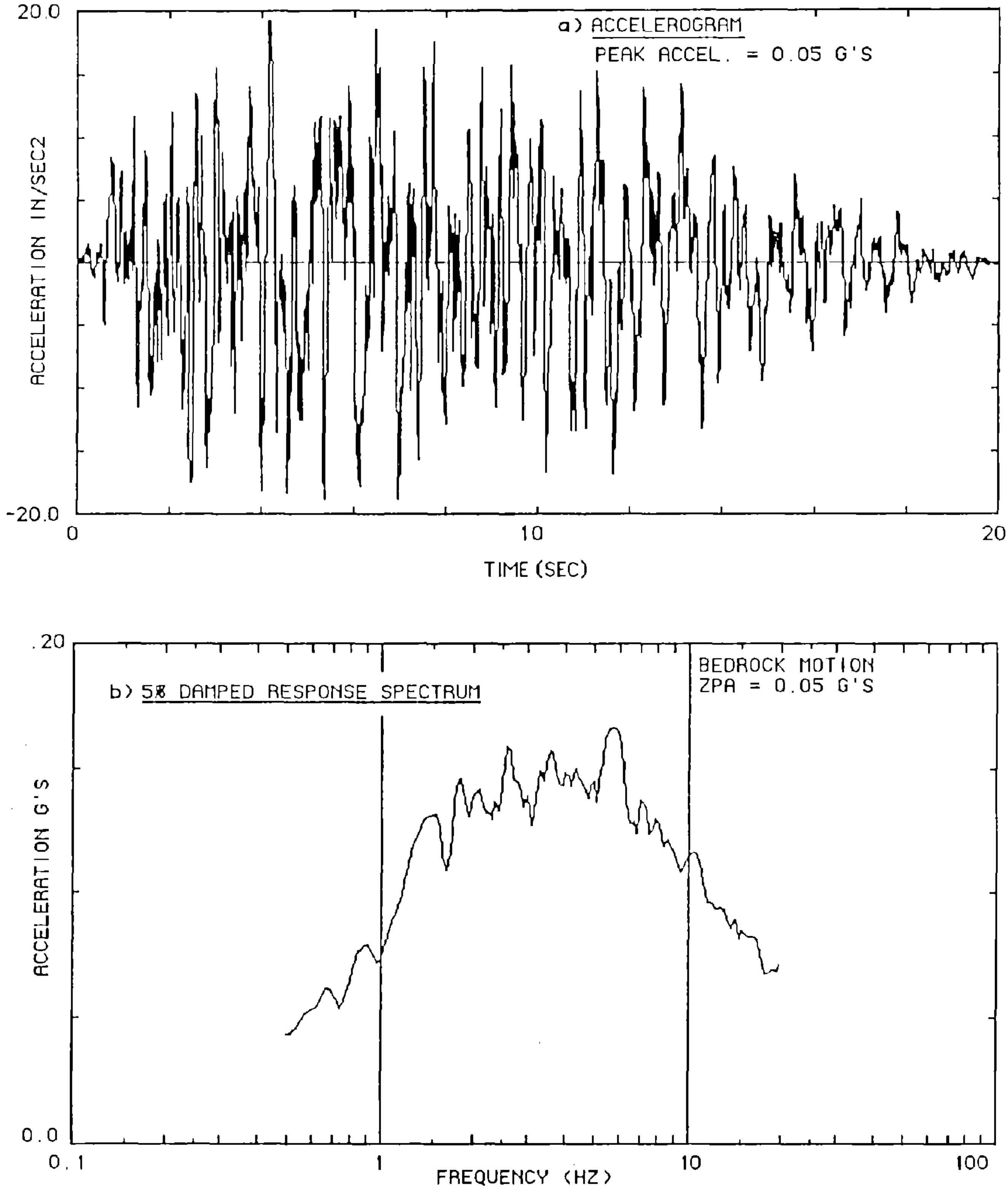


FIGURE 4-2 SPECIFIED BEDROCK SEISMIC MOTION FOR 0.05G PEAK ACCELERATION

engineers. To simulate higher peak bedrock acceleration levels for this study, the accelerogram of Figure 4-2a was simply scaled linearly with peak acceleration and used as input to the soil column.

Using fast Fourier transform methods, this time history is broken into its Fourier components, the components convolved with the soil column (that is, propagated upward through the soil, frequency by frequency), and recombined to yield the time history of the motion at the ground surface compatible with the input bedrock motions. This surface accelerogram is then used, in turn, as input to the elevated structure at that particular location. To obtain site specific structural responses along the entire structure of a long elevated line, this calculation had to be performed at the various locations of interest along that line.

4.2.1 Surface Accelerograms Along the Jamaica Line

Surface ground motions were generated for various locations along the two elevated lines investigated for this study, the Jamaica Line and the Flushing Line. For the Jamaica Line, it was found that the elevated portion of the line traverses an area consisting predominantly of sands. The soils are primarily of a medium dense character throughout the entire line from Marcy Avenue to the end of the line. A cross section of the soil profile along the entire line is shown in Figure 4-3. Borings available at Marcy Ave. (near the Williamsburg Bridge area), the East New York train storage yard, and borings at 121 Street and Jamaica Avenue, were used to obtain the required soil properties. A summary of the soil properties and the borings from which they were obtained are presented in Table IV-1. The primary variability in soil profile along the entire line is the depth of overburden to bedrock, which, as can be noted from Figure 4-3,

TABLE IV-1

JAMAICA LINE BORING SUMMARY

BORING LOCATION	BORING NUMBER	BORING DEPTH (FT)	NO. OF SAMPLES	AVERAGE BLOWS/FT	UNIT WGT (PCF)	SHEAR MOD. (KSF)
Marcy Ave (No. 1)	MB1	52.0	8	32	129	212.1
	MB2	37.0	3	20	119	169.4
	MB3	52.0	9	39	134	237.9
	MB4	52.0	6	32	127	203.4
	MB5	52.0	10	25	118	175.3
	MB6	52.0	9	31	129	205.6
East NY Stor. Yd (No. 2)	ENYCQ-1	51.0	5	14	113	136.2
	ENYCQ-4	55.5	4	17	118	154.9
	ENYCQ-5	51.0	5	6	105	95.7
	ENYCQ-6	82.0	8	49	136	307.7
	ENYCQ-7	52.0	5	41	131	280.2
	ENYCQ-8	47.5	2	5	102	91.2
	ENYCQ-9	50.0	2	20	113	171.0
	ENYCQ-10	40.0	3	23	122	182.1
	ENYCQ-12	52.0	7	33	125	238.1
	ENYCQ-13	44.0	1	50	140	443.5
	ENYCQ-14	52.0	5	29	125	214.3
Jam. Ave at 121 St (No. 3)	RHSS-1	52.0	10	46	137	308.3
	RHSS-2	70.0	17	33	128	239.7
	RHSS-3	52.0	10	42	133	286.4

TABLE IV-2

JAMAICA LINE SOIL COLUMN FREQUENCY

DEPTH OF SOIL COLUMN (FT)	FUNDAMENTAL SOIL COLUMN FREQUENCY (CPS)
Bedrock	-
100	0.599
200	0.299
300	0.200
400	0.150
500	0.120
600	0.100

Assumed Average Soil Properties:

- Shear Modulus (Ksf) = 226.0
- Unit Weight (Psf) = 127.0
- Poisson's Ratio = 0.2
- Shear Wave Velocity (Fps) = 239.0
- Soil Damping (%) = 5.0

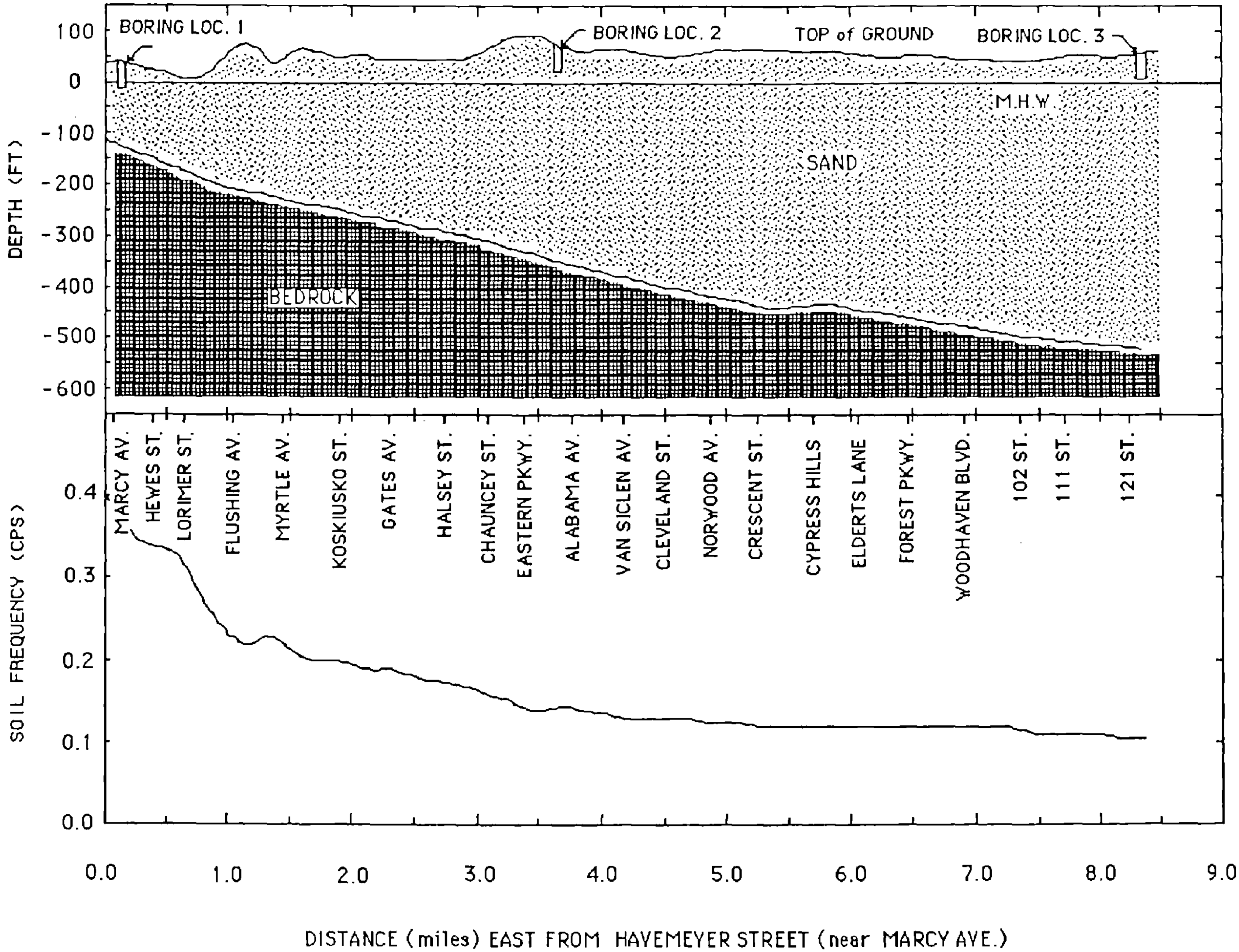


FIGURE 4-3 SOIL PROFILE ALONG JAMAICA LINE

was found to vary from a little more than 100 feet at the Marcy Avenue stop to about 500 feet at the end of the line. Using the average soil properties computed from the sample blow counts, the fundamental frequency of the soil column was computed as a function of depth of the soil overburden, and are shown in Table IV-2. These results are also plotted in Figure 4-3. As may be noted, the primary column frequency decreases significantly as the depth to bedrock increases, with the column frequency varying from about 0.4 hz near Marcy Avenue to about 0.1 hz at the end of the line. This frequency can then be expected to have a significant impact on the surface motions calculated along the route.

Amplification functions are plotted in Figure 4-4 for different thicknesses of soil overburden along the Jamaica Line which indicate that the thicker this overburden becomes, the lower the frequency at which seismic motions will be amplified as shear waves propagate up the soil column. If the surface structure is susceptible to these low frequencies, it will respond with large excursions and have the potential for greater damage. As may be noted from these functions, some additional amplification may occur at the secondary frequencies of the soil column indicated by the secondary peaks in these plots. In these calculations, an average soil hysteretic damping ratio of 5% was assumed for the sands, a reasonable value for the low levels of shaking considered. The value of the damping ratio used in the calculations has a significant impact on the magnitudes of the peaks of these amplification plots, which in turn play a major role in selecting those frequency components amplified by the soil column.

Using the specified ground motion indicated by the accelerogram of Figure 4-2 as input to the bottom of the soil

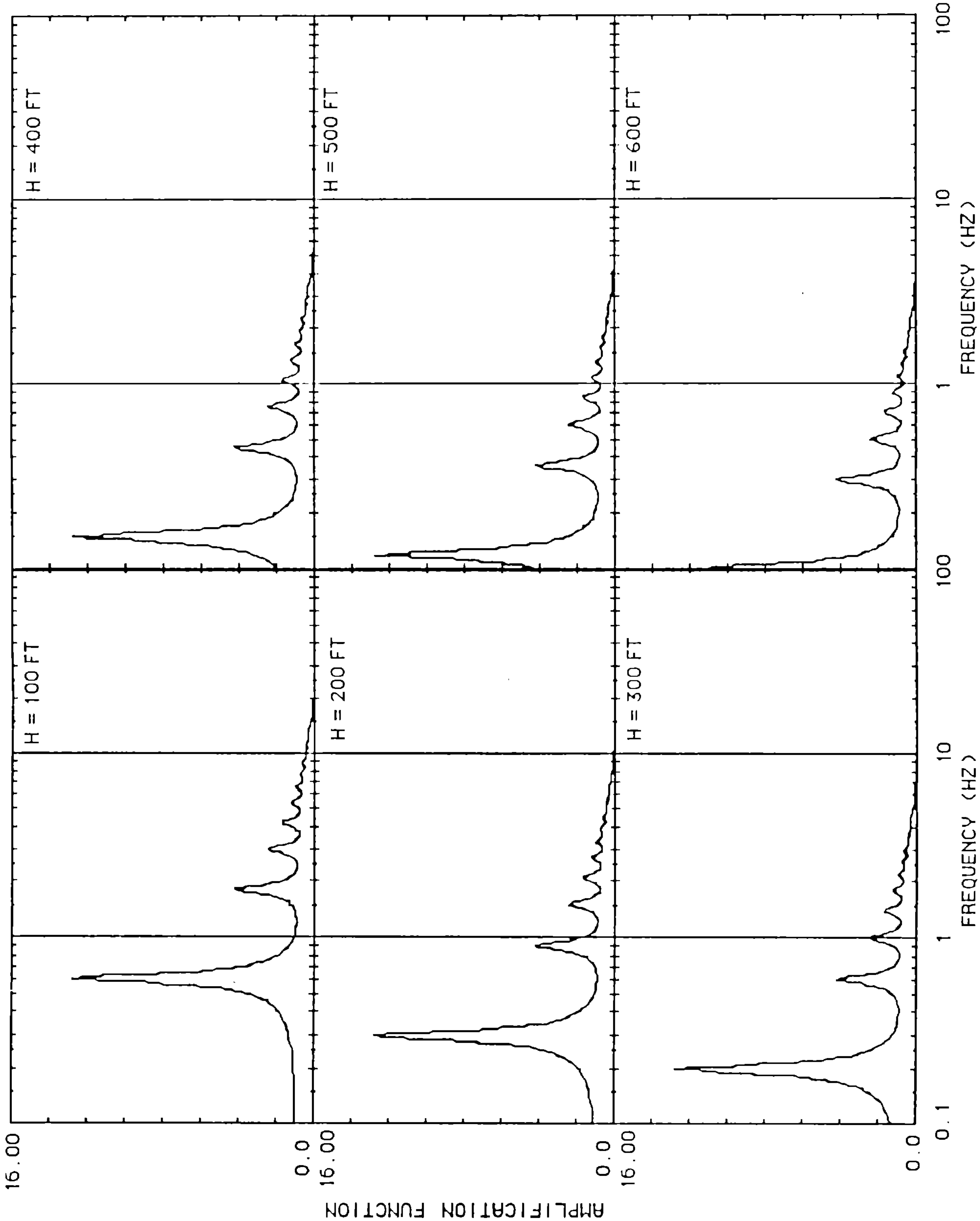


FIGURE 4-4 AMPLIFICATION FUNCTIONS ALONG THE JAMAICA LINE
(FOR 5% SOIL DAMPING)

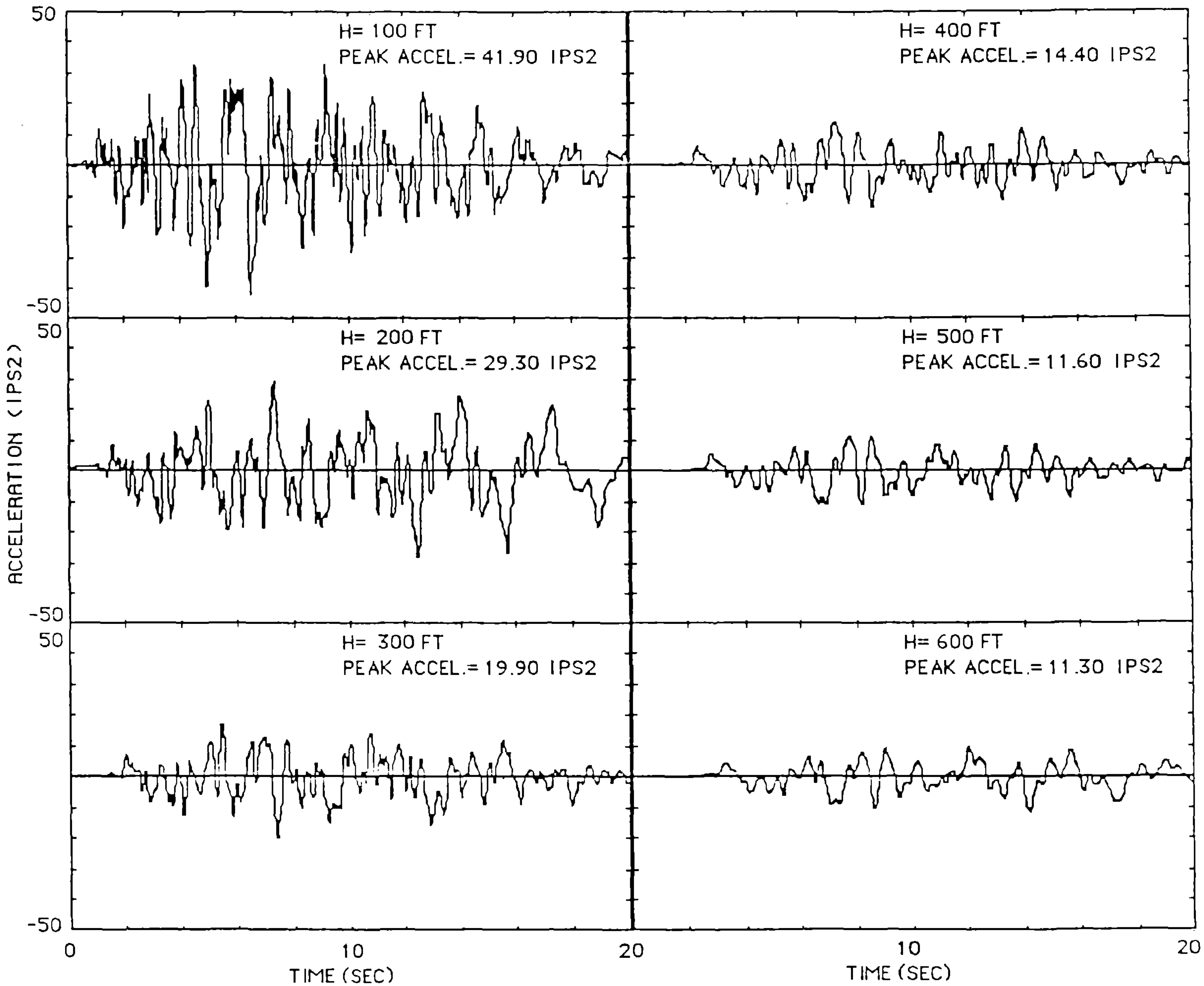


FIGURE 4-5 SURFACE ACCELEROGRAMS ALONG THE JAMAICA LINE
(ASSUMING 5% SOIL DAMPING)

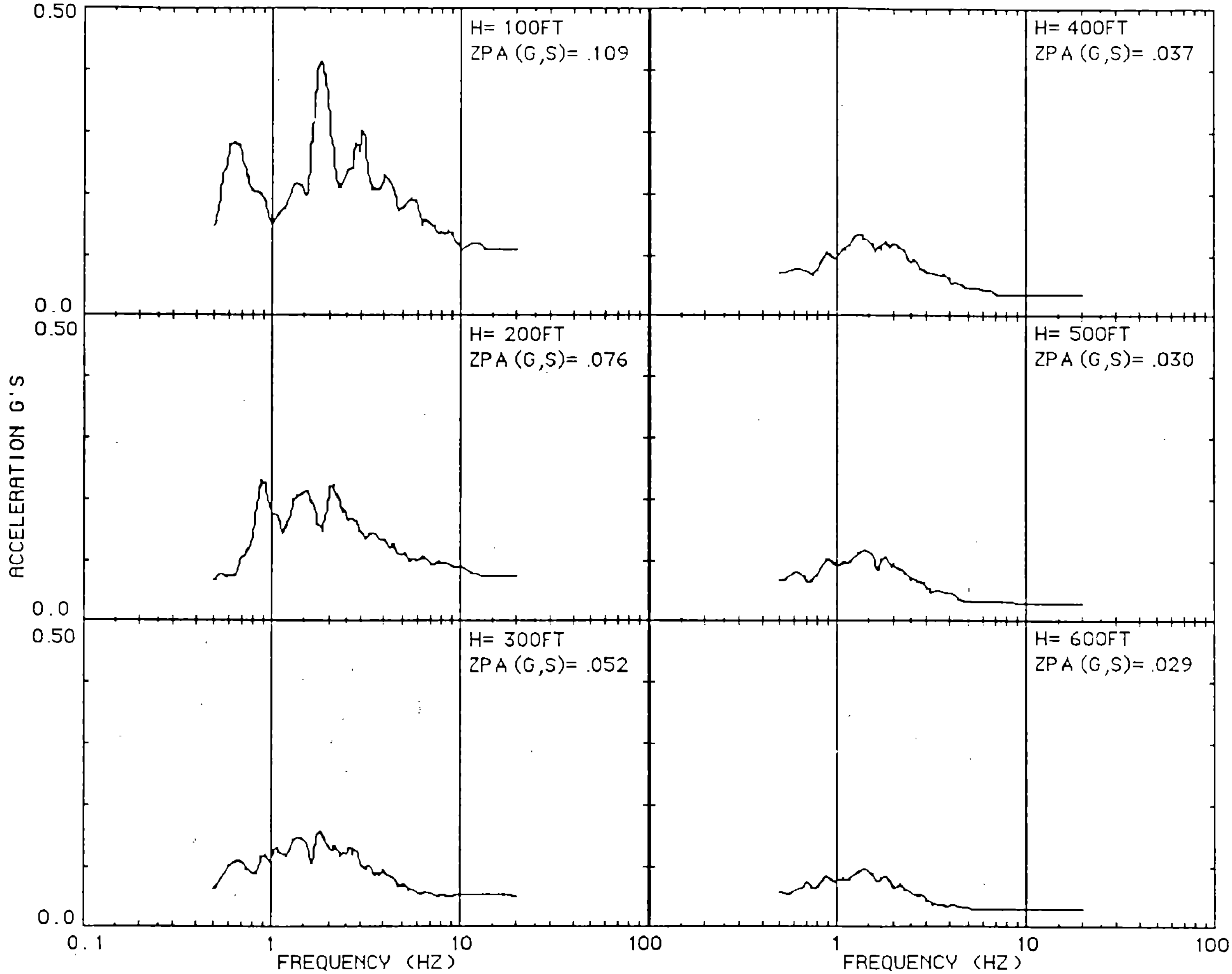


FIGURE 4-6 5% DAMPED RESPONSE SPECTRA FOR THE JAMAICA LINE column, plots of the calculated time histories at the ground surface are shown in Figure 4-5 for varying thicknesses of the soil overburden. As may be noted, the character of the surface motion is significantly impacted by the thickness of the soil column. The shallower the column, the more high frequency components are retained in the wave motion, and the larger the peak acceleration sustained at the ground surface. The deeper the soil column, the lower the frequencies noted in the surface response and the lower the peak ground accelerations sustained. Response spectra for these surface wave motions were calculated and these are shown in Figure 4-6. The shift in both frequency content and magnitude is clearly shown by these figures as a function of depth to bedrock.

4.2.2 Surface Accelerograms Along The Flushing Line

For the Flushing Line, the character of the soil profile at various locations along the line from Hunters Point Avenue to Main Street can be fundamentally different from that along the Jamaica Line. The elevated portion of the Flushing Line traverses a soil area that was divided into four major sections as indicated in Figure 4-7, namely, the Long Island City, the Sunnyside, the Jackson Heights and the Willets Point areas. This soil data was obtained from a variety of boring logs available along the line, a summary of which is presented in Table IV-3 for these four areas. In particular, the borings at the Willets Point area indicate extensive deposits of both cohesive and loose granular soils, while the other boring locations indicate generally granular soils. The soft clays at Willets Point show significant thicknesses and in many ways are similar to the soft clays of Mexico City, which are known to have significantly altered the response of the ground surface to seismic bedrock motions.

TABLE IV-3

FLUSHING LINE BORING SUMMARY

BORING LOCATION	MAT'L DESCR.	LAYER THICK. (FT)	UNIT WEIGHT (PCF)	ASSUMED POISSON'S RATIO	SHEAR MODULUS (KSF)	SHEAR WAVE VELOCITY (FPS)
LIC (No. 1)	Sand	to Bedrock	123	0.3	100.2	162.0
Sunnyside (No. 2)	Sand	to Bedrock	137	0.3	252.0	243.4
Jackson Heights (No. 3)	Sand	to Bedrock	129	0.3	212.7	230.4
Willets Point (No. 4)	Sand	25	111	0.3	124.3	189.9
	Clay	32	81	0.4	36.8	121.4
	Sand	23	115	0.3	110.4	175.4
	Clay	16	81	0.4	67.3	164.0
	Sand	to Bed.	107	0.3	72.9	148.1

Average Soil Damping in Sand (%) = 5
Average Soil Damping in Clay (%) = 2

TABLE IV-4

FLUSHING LINE SOIL COLUMN FREQUENCIES

LOCATION	DEPTH OF SOIL COLUMN (FT)	FUNDAMENTAL SOIL COLUMN FREQUENCY (CPS)
Long Island City	50	0.810
	100	0.405
Sunnyside	50	1.217
	100	0.609
	150	0.406
	200	0.304
Jackson Heights	150	0.384
	200	0.288
	250	0.230
	300	0.192
Willets Point	200	0.181
	250	0.142
	300	0.117
	350	0.099

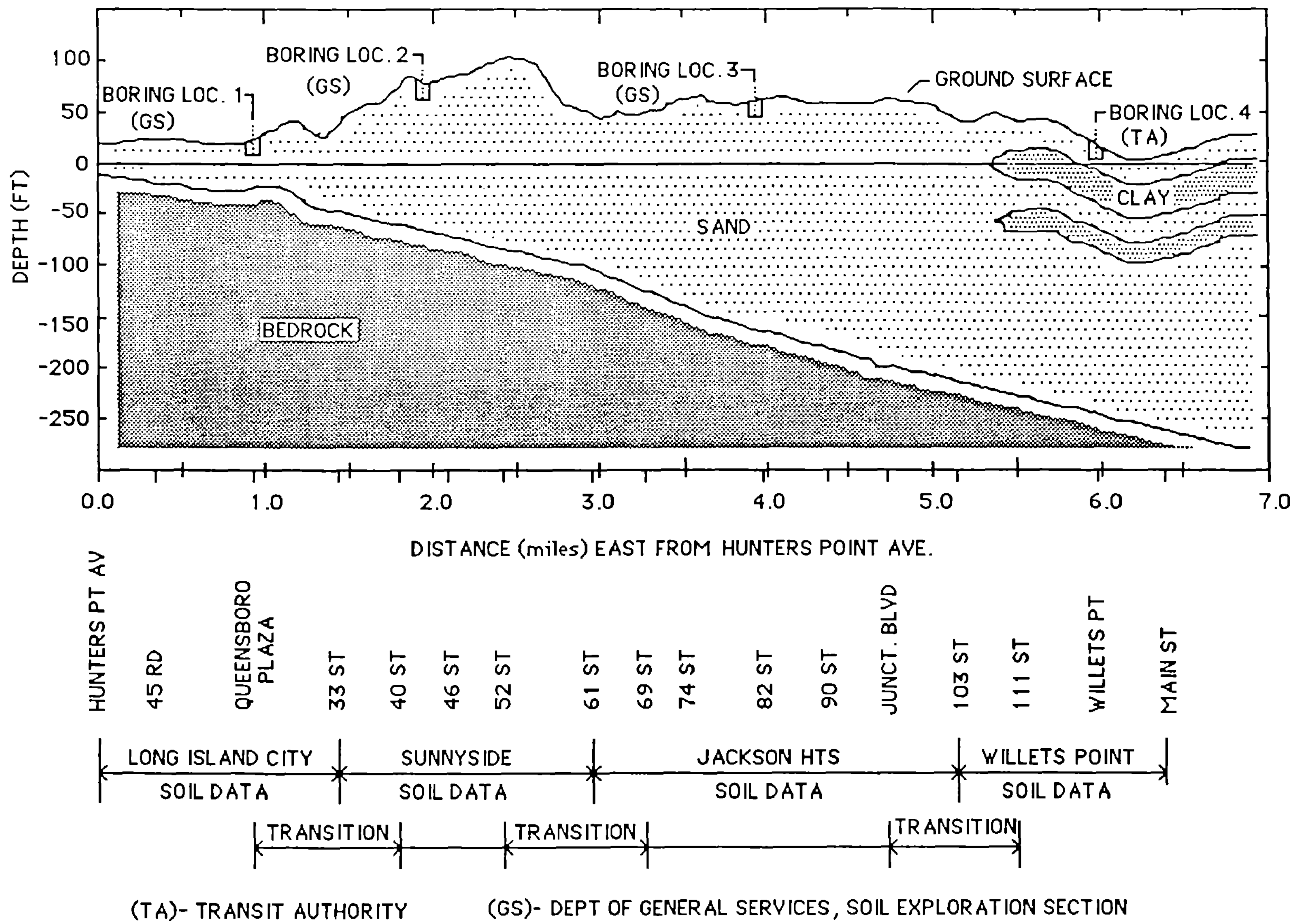


FIGURE 4-7 SOIL PROFILE ALONG FLUSHING ELEVATED LINE

Again, surface ground motions were calculated using the representative soil properties of the four different soil profiles along the line and varying the bedrock depth within each section. A summary of the fundamental column frequencies obtained for the various soil columns in these areas is shown in Table IV-4. Amplification functions for these locations are presented in Figures 4-8 through 4-11, and again indicate the characteristic primary and secondary frequencies of these soil columns. Surface accelerograms for the various areas along the Flushing line are shown in Figures 4-12 through 4-15 which indicate the impact of the soil column on the seismic motions. Again, the softer and deeper the column, the lower the frequency content and magnitude of the acceleration at the ground surface. In the Willets Point area where the very soft surface clays are present, the duration of relatively strong shaking may in fact be longer than that expected at sand sites since the saturated clays may not significantly damp out the surface motions with time. This was also noted at Mexico City, where the shaking continued for long durations. Response spectra at these four sites are presented in Figures 4-16 through 4-19.

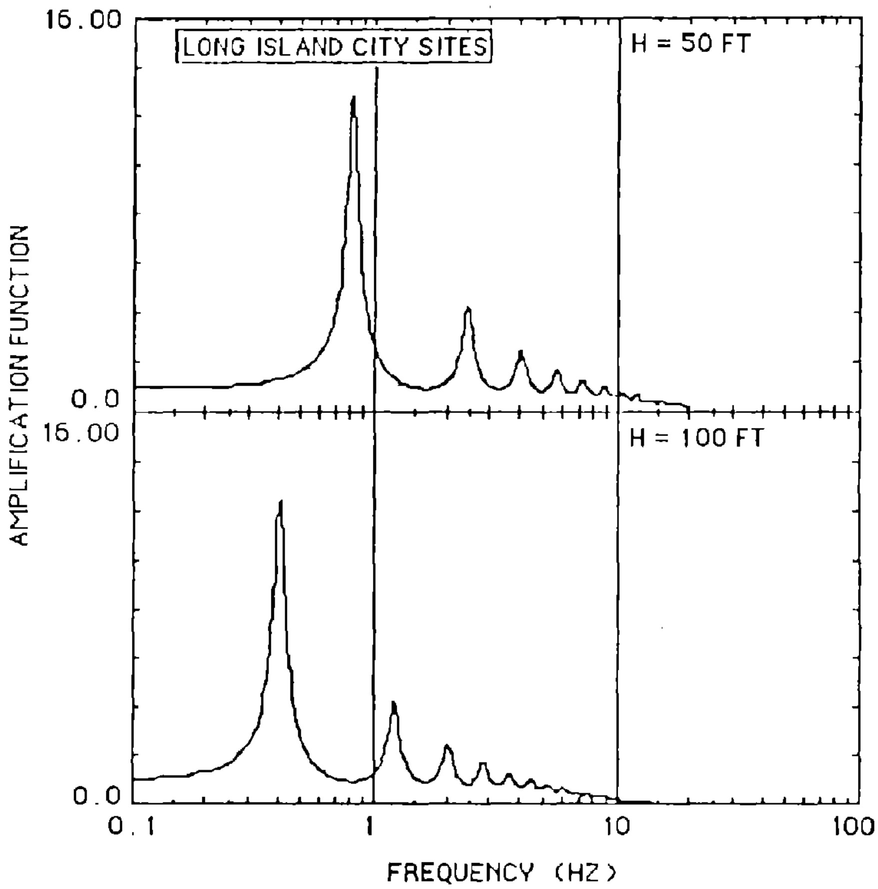


FIGURE 4-8 AMPLIFICATION FUNCTIONS ALONG THE FLUSHING LINE AT THE LIC SECTION (FOR 5% SOIL DAMPING)

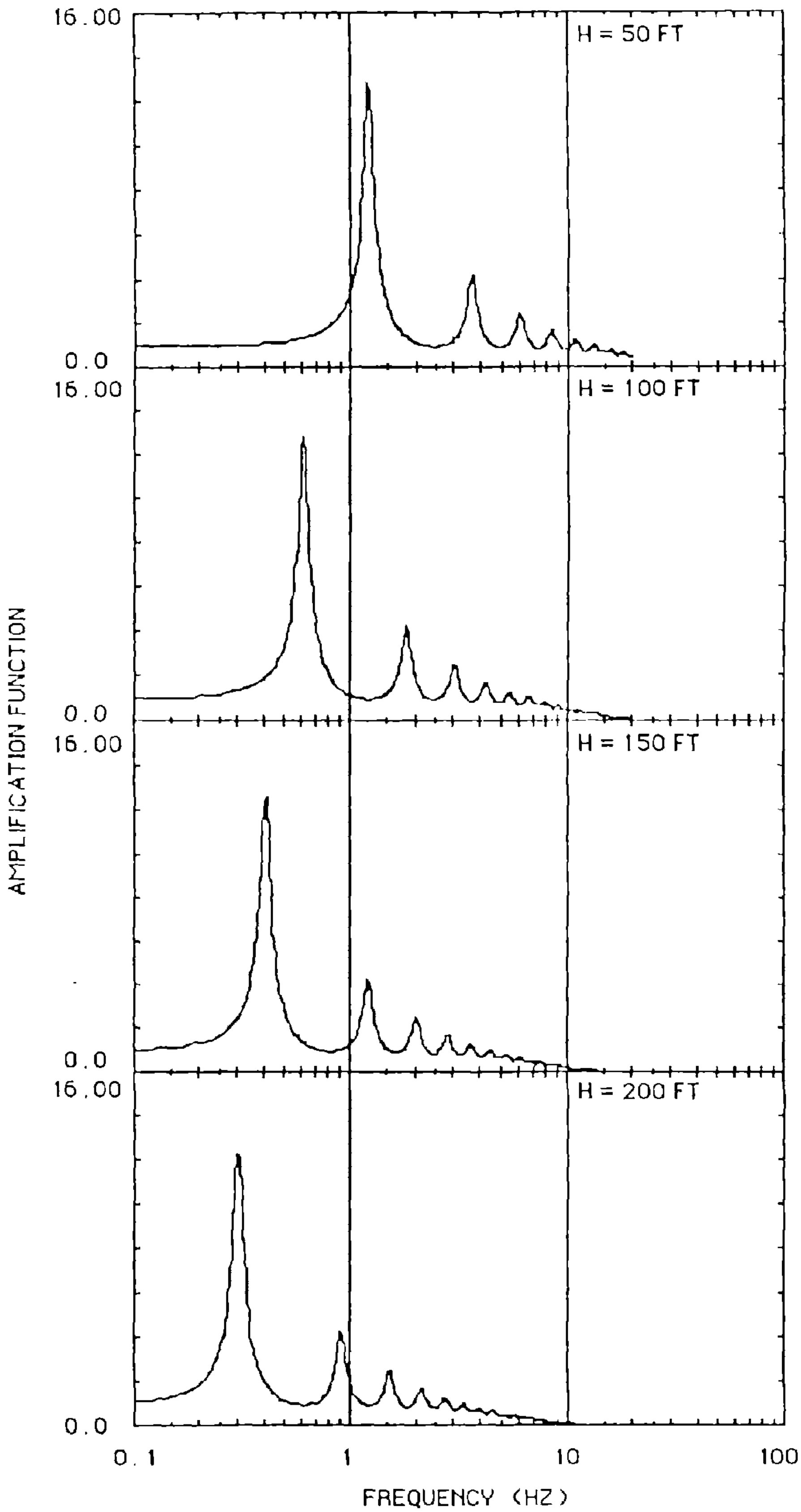


FIGURE 4-9 AMPLIFICATION FUNCTIONS ALONG THE FLUSHING LINE AT THE SUNNYSIDE SECTION (FOR 5% SOIL DAMPING)

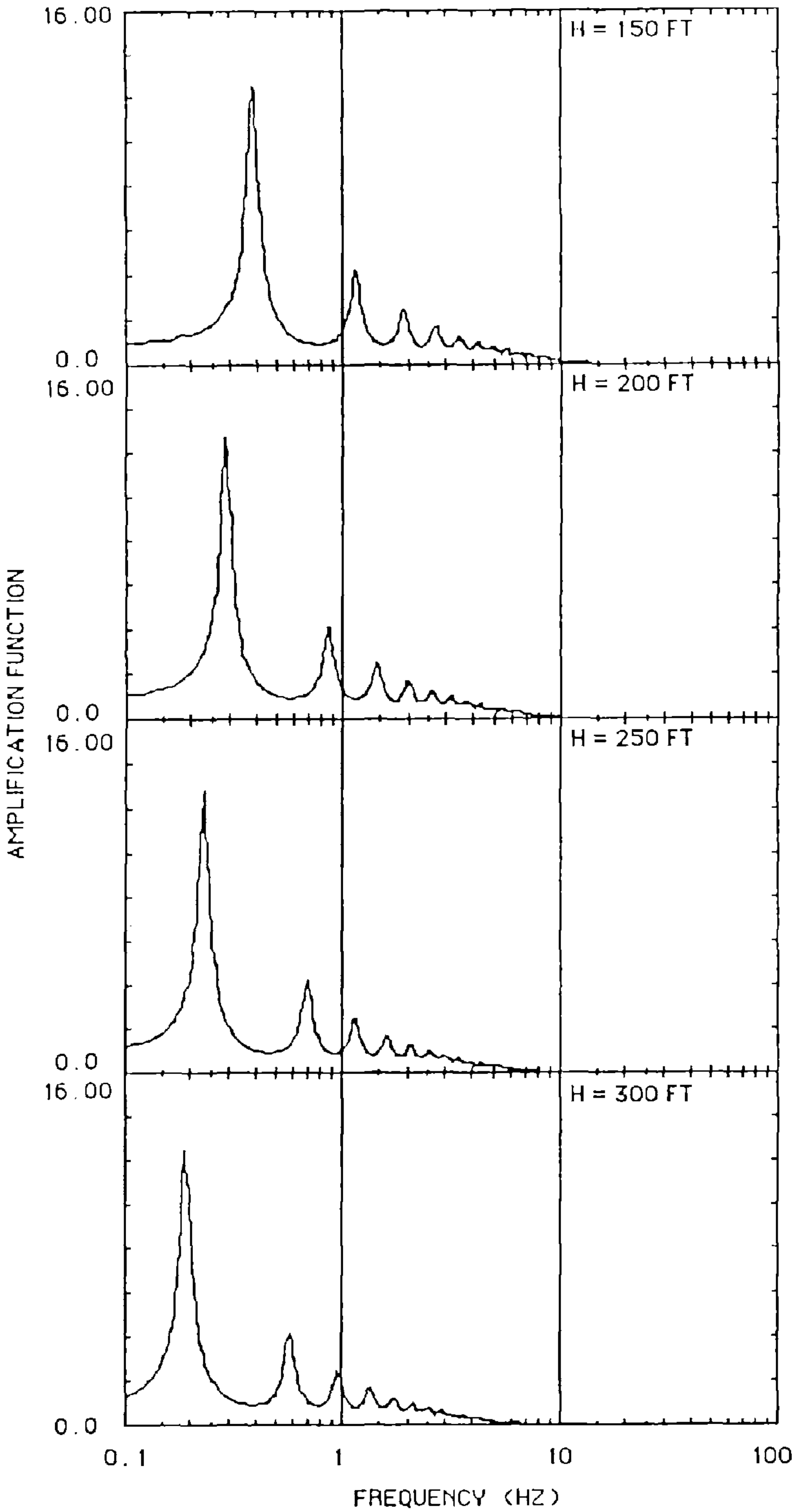


FIGURE 4-10 AMPLIFICATION FUNCTIONS ALONG THE FLUSHING LINE AT THE JACKSON HEIGHTS SECTION (FOR 5% SOIL DAMPING)

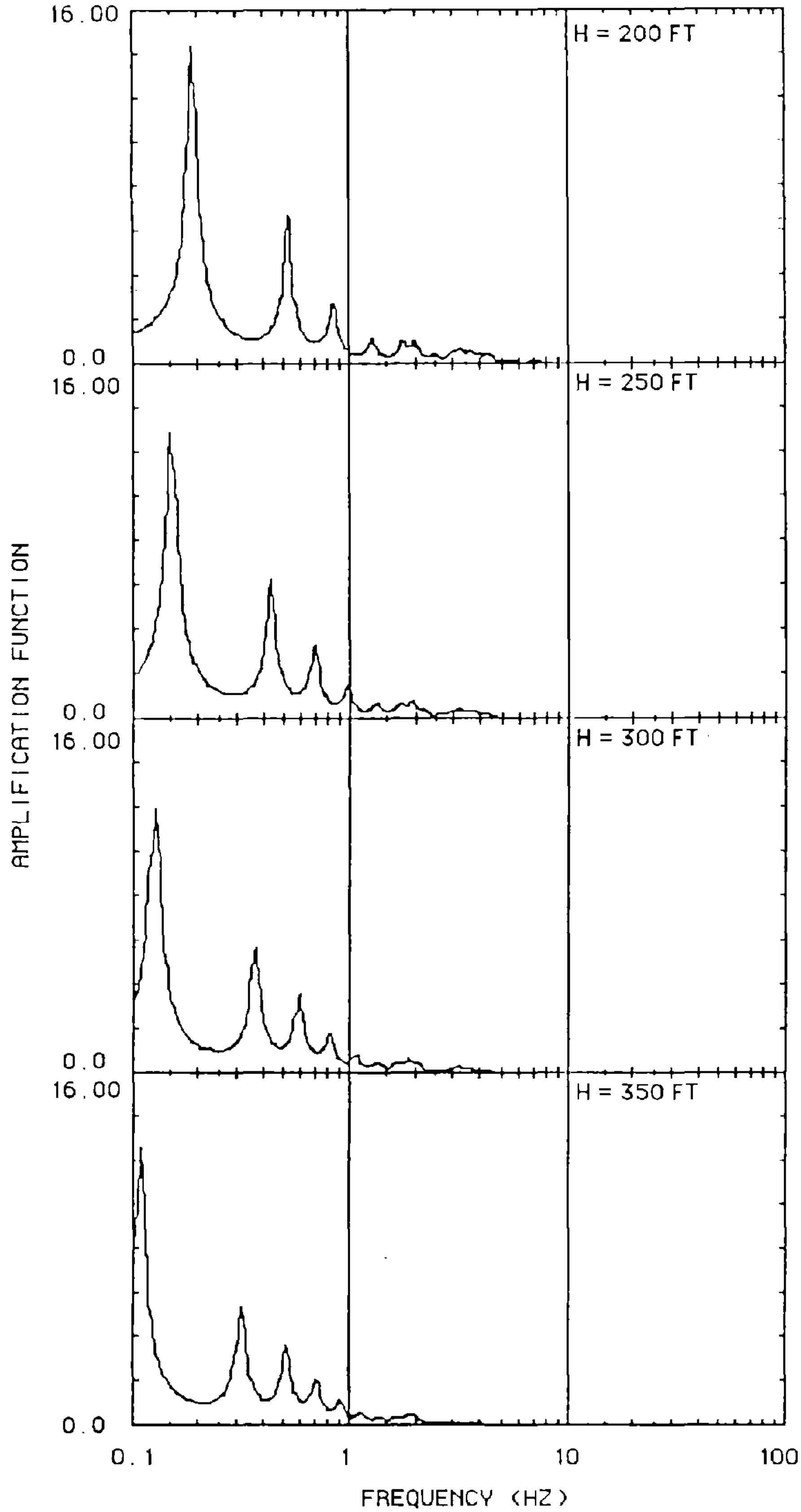


FIGURE 4-11 AMPLIFICATION FUNCTIONS ALONG THE FLUSHING LINE AT THE WILLETS POINT SECTION (FOR 5% SAND AND 2% CLAY DAMPING)

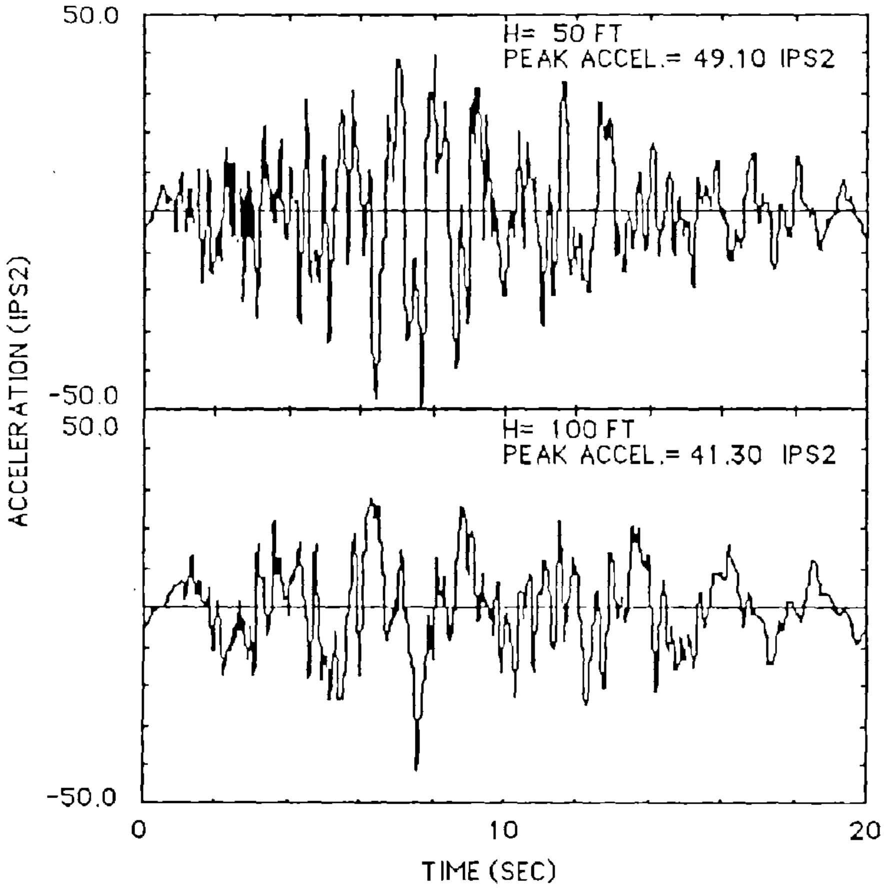


FIGURE 4-12 SURFACE ACCELEROGRAMS ALONG THE FLUSHING LINE AT THE LIC SECTION (FOR 5% SOIL DAMPING)

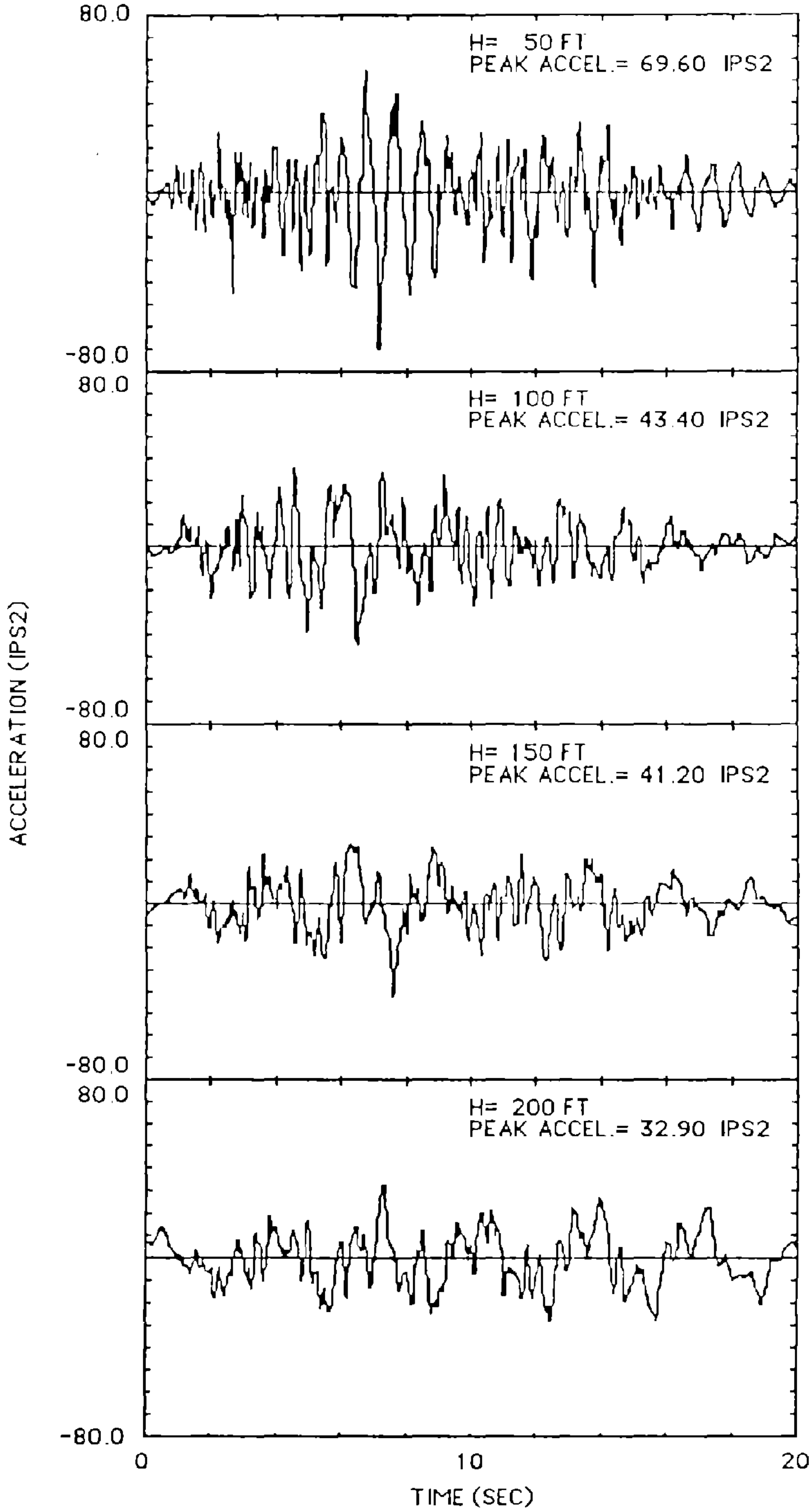


FIGURE 4-13 SURFACE ACCELEROGRAMS ALONG THE FLUSHING LINE AT THE SUNNYSIDE SECTION (FOR 5% SOIL DAMPING)

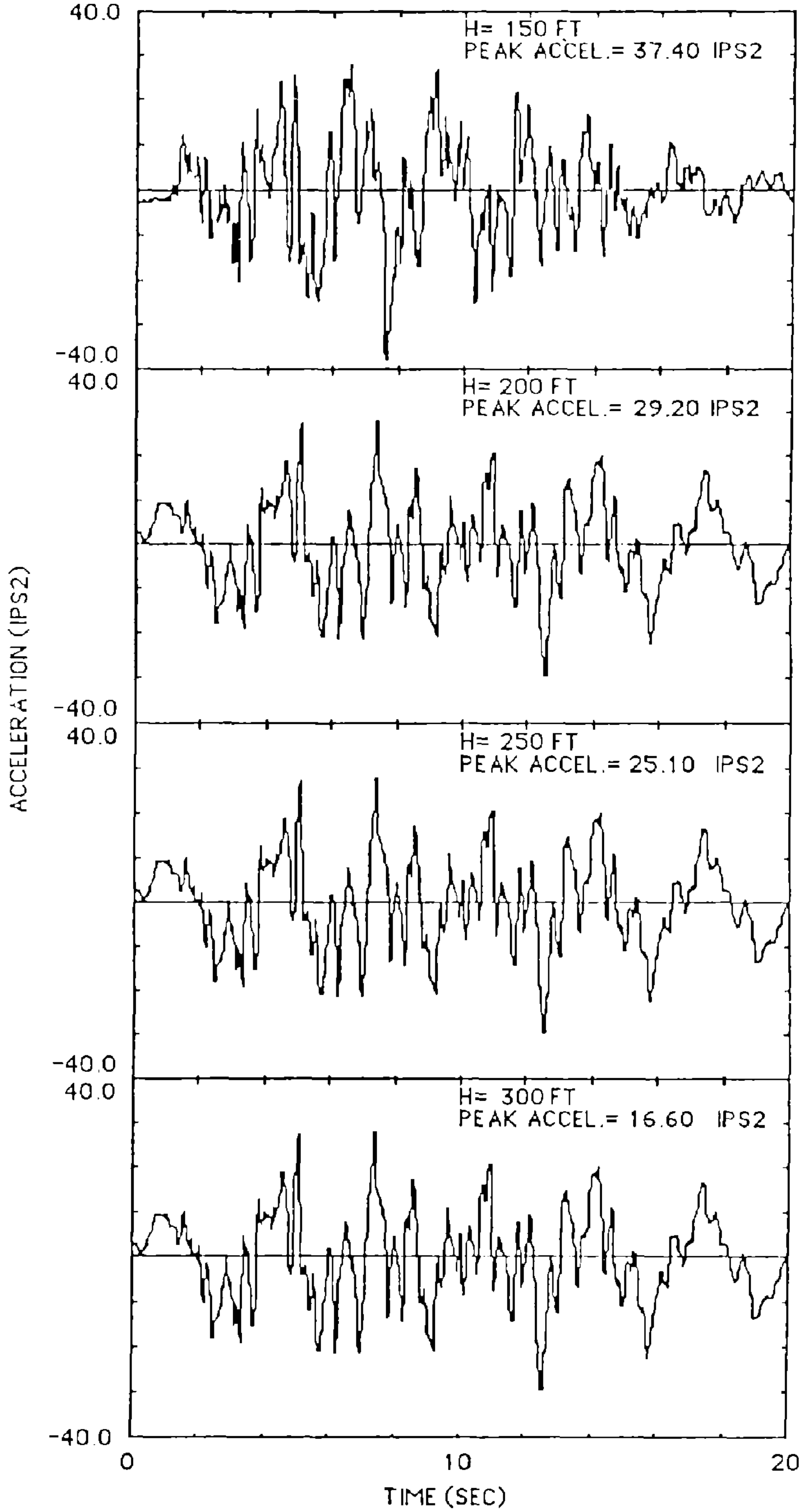


FIGURE 4-14 SURFACE ACCELEROGRAMS ALONG THE FLUSHING LINE AT THE JACKSON HEIGHTS SECTION (FOR 5% SOIL DAMPING)

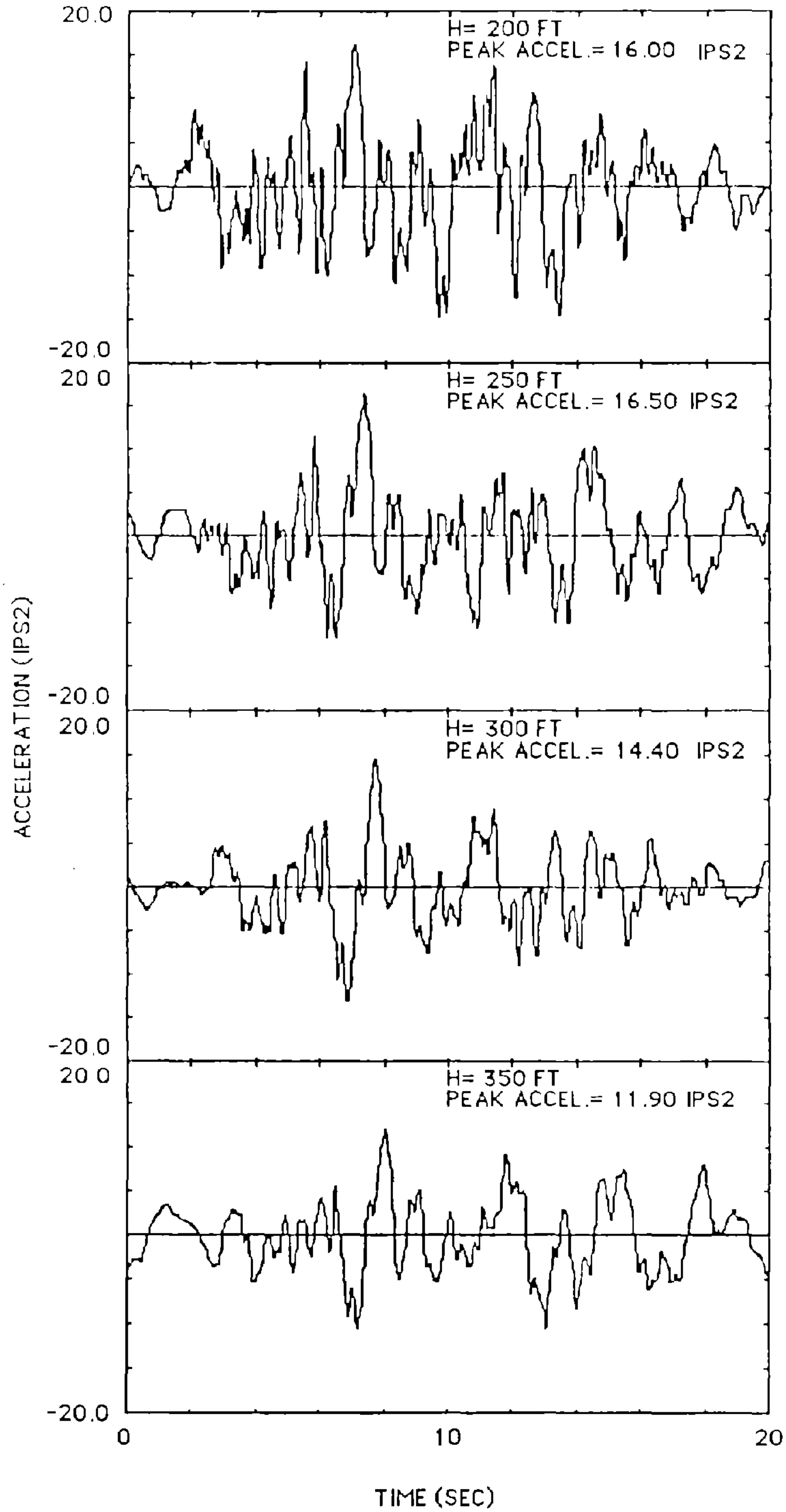


FIGURE 4-15 SURFACE ACCELEROGRAMS ALONG THE FLUSHING LINE AT THE WILLETS POINT SECTION (FOR 5% SAND AND 2% CLAY DAMPING)

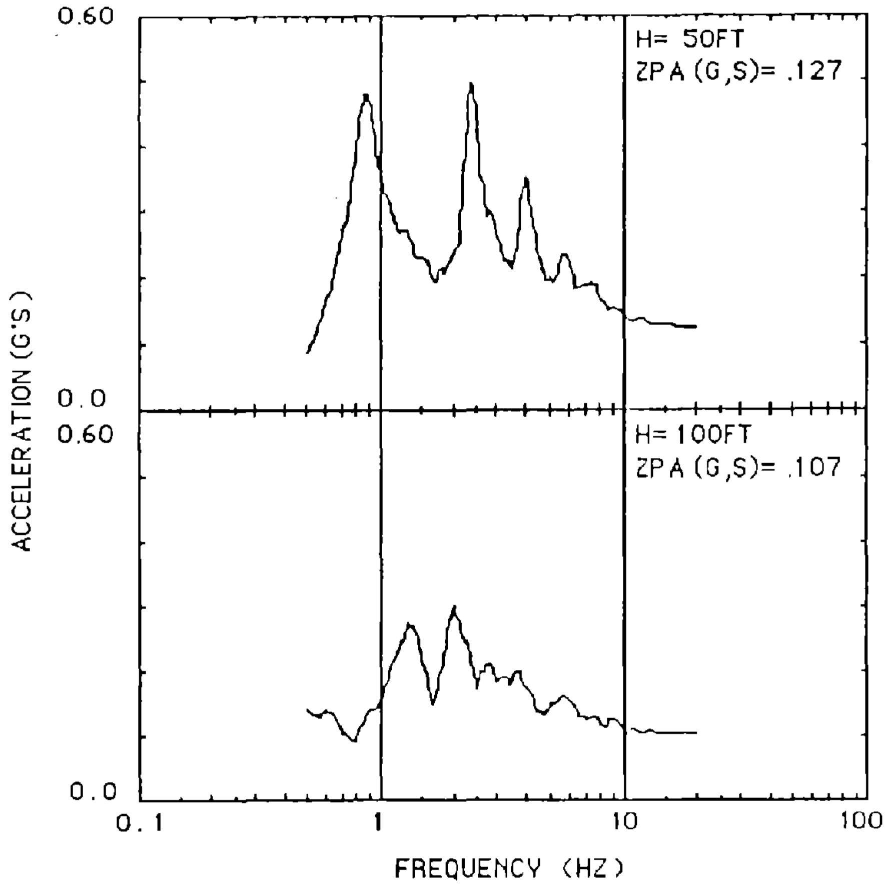


FIGURE 4-16 5% DAMPED SURFACE RESPONSE SPECTRA ALONG THE FLUSHING LINE AT THE LONG ISLAND CITY SECTION

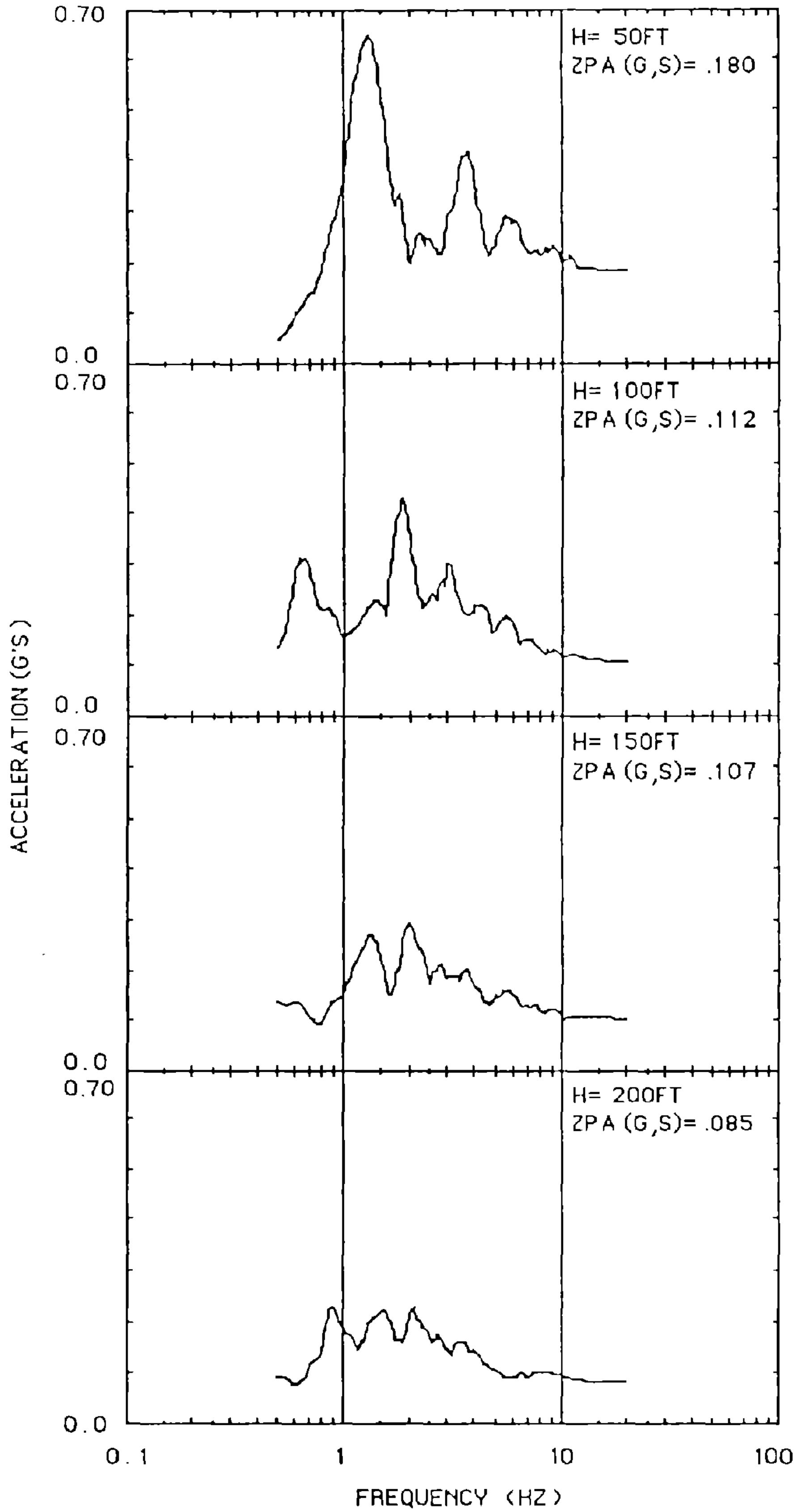


FIGURE 4-17 5% DAMPED SURFACE RESPONSE SPECTRA ALONG THE FLUSHING LINE AT THE SUNNYSIDE SECTION

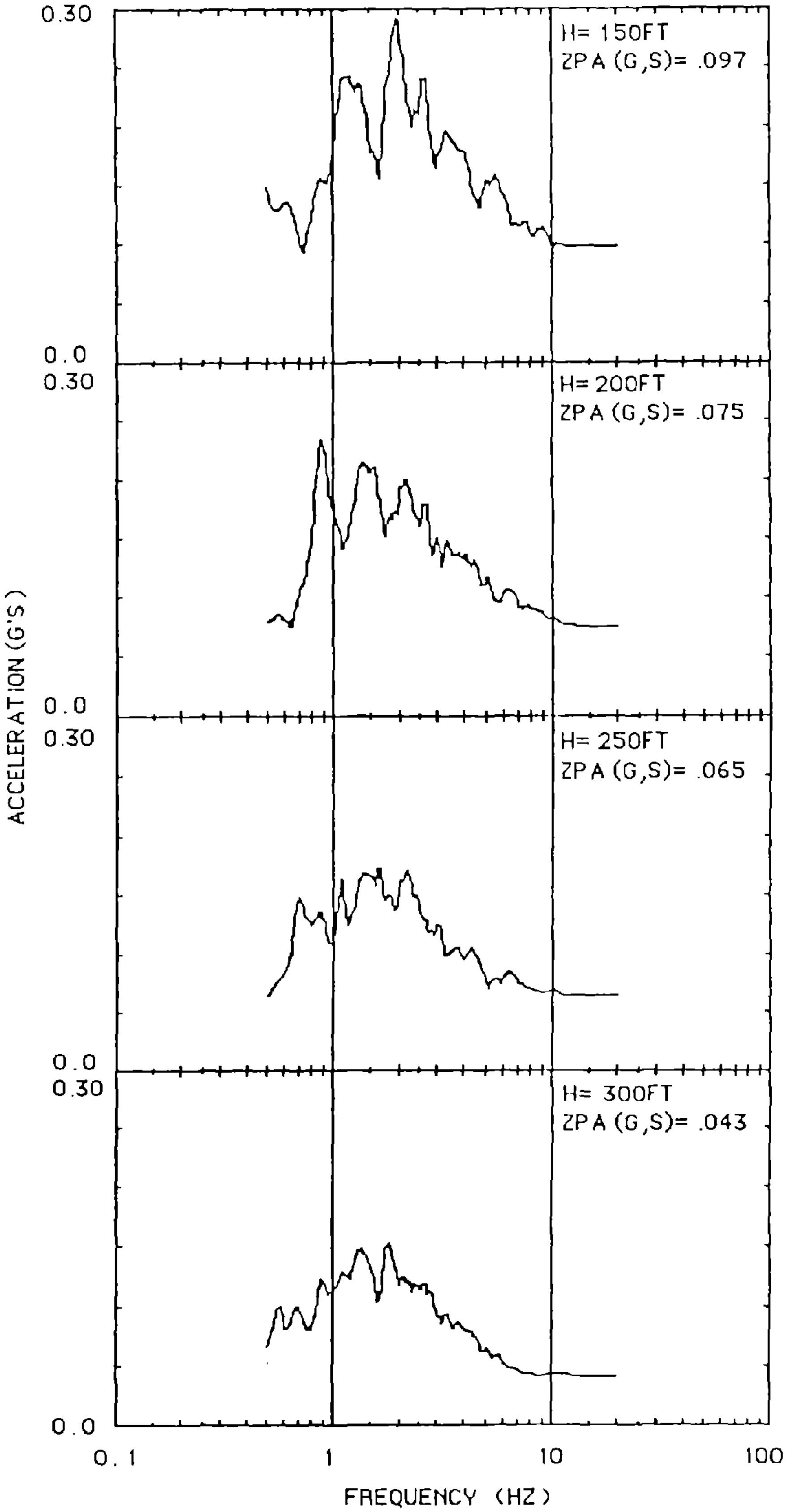


FIGURE 4-18 5% DAMPED SURFACE RESPONSE SPECTRA ALONG THE FLUSHING LINE AT THE JACKSON HEIGHTS SECTION

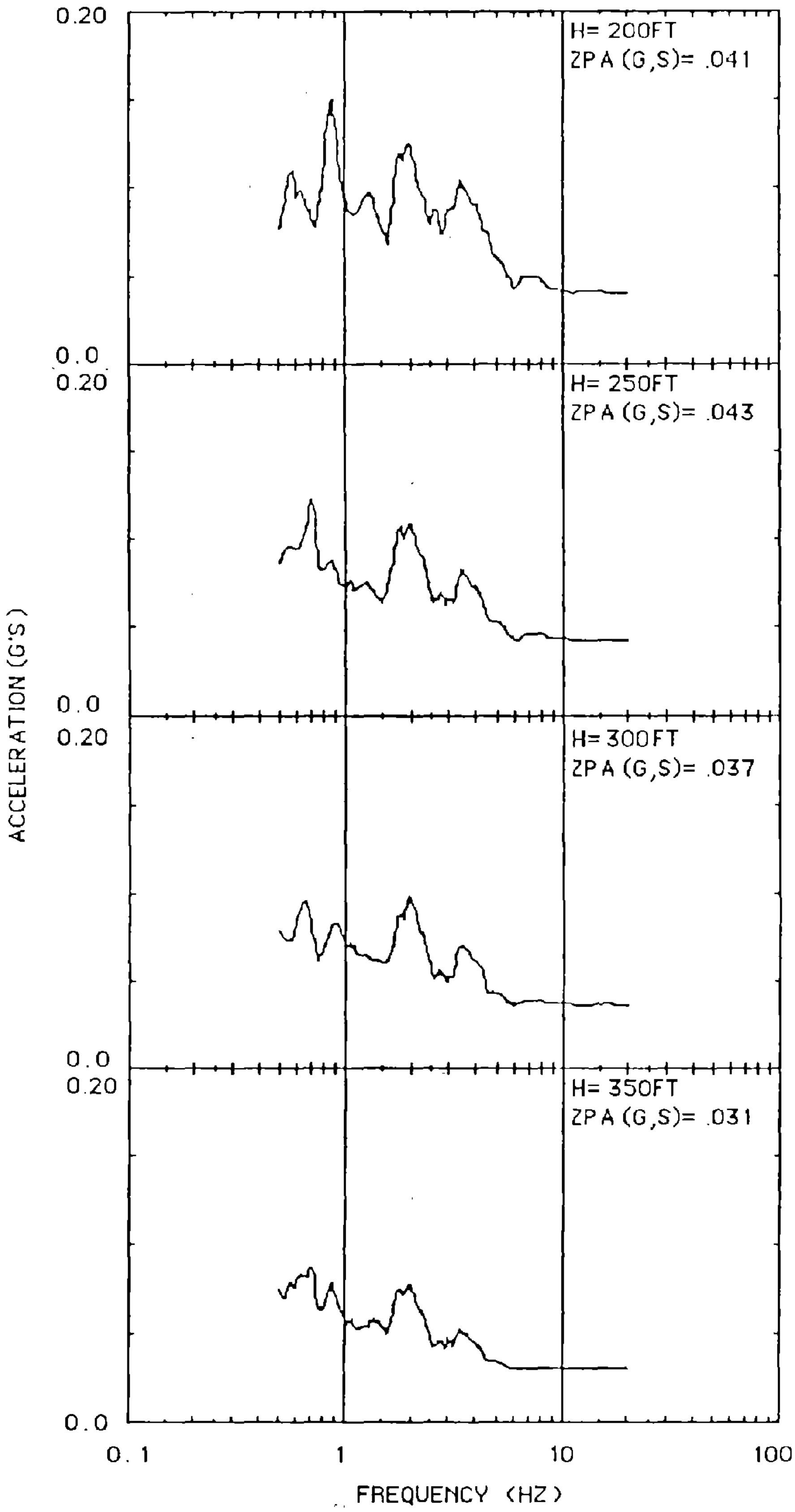


FIGURE 4-19 5% DAMPED SURFACE RESPONSE SPECTRA ALONG THE FLUSHING LINE AT THE WILLETS POINT SECTION

SECTION 5

RESPONSE OF ELEVATED STRUCTURE

In the following paragraphs, the calculated seismic response of two lines evaluated is presented. The Jamaica line crosses the Williamsburg Bridge and traverses Brooklyn and Queens along Broadway, Fulton St. and Jamaica Ave. Both the "J" and "Z" trains operate along this line. Typically, the line allows for two train operation, a local train in each direction. A third track is available between Marcy Ave. and Myrtle St. and allows for express service between these two stations. A third track has also been included along certain other portions of the line. The Flushing elevated line services the northern part of Queens along Roosevelt Ave. and Queens Boulevard with the "#7" train. A third track on this line is available for express service.

In assessing the response of the elevated lines in the system, the structural properties of typical transverse bents along each of the two lines were developed. The information required for this was made available by the NYCTA. From as-built drawings, stiffness, mass and strength properties of the structural models were determined. These structural models were then subjected to the surface ground motions generated from the convolution analyses previously discussed. As described previously for the general case of a soil layer lying above bedrock, the criteria horizontal motion described in Figure 4-2 was input at the bedrock level, the response of the ground surface calculated from the convolution analysis, and the peak structural response then determined. For the case of the structure resting directly on the bedrock (with no soil overburden), the criteria motion was input directly to the structure. It should be noted that in each single level bent response calculation, soil-structure interaction effects were included. That is, the ability of the soil around the column footings to move differently from the "free-field" surface motions was included in the evaluation. As can be anticipated for these structural types, soil-structure interaction effects were found to be small and therefore neglected in the two level bent calculations. The details of the analyses developed for both single and double level bent structures are presented in the appendices to this report.

For each problem considered, the peak lateral deflection of the upper girder was calculated using a step-by-step time integration procedure, assuming the structure to behave in an elastic-perfectly plastic manner. That is, the structure was considered to behave linearly until the yield moment at the upper column joint is reached. For displacements greater than this yield displacement, the column-girder joint is assumed to maintain its moment capacity with no increase in strength incorporated in the calculations. The ratio of the final bent displacement to the yield displacement is termed the bent ductility. If this ductility exceeds a value of unity, then the stresses developed at the upper column joint exceeds the yield stress of the steel, with permanent strains remaining in the steel after completion of the dynamic response. Such behavior indicates overstressing of the joint occurring from the dynamic effects.

It should be pointed out that the analyses conducted were not meant to include all important details of the structural response in the evaluations. Rather, the purpose of the analyses was to determine in an approximate fashion the extent of the problem that can be anticipated for these structural elements. Thus, in these calculations, no account is taken of the following items known to play an important role in arriving at detailed structural assessments of damage potential.

- No influence of vertical motion components from a given seismic event was included in the response calculation, although such inputs could cause significant stresses in the columns and increase the amount of yielding developed from the horizontal motions.

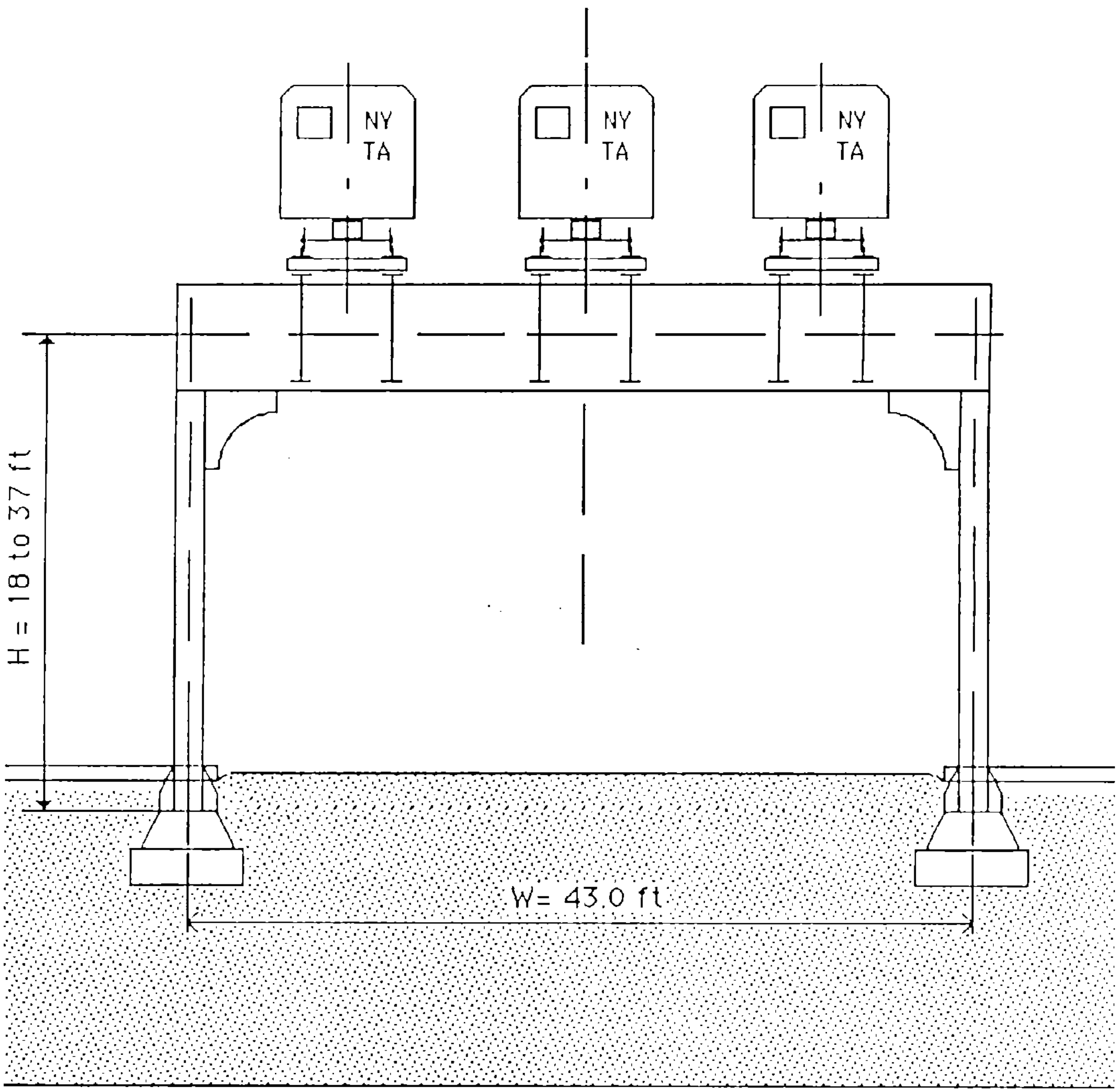
- The stresses caused by other load components which contribute to the development of stresses in the columns were not included in the response calculations, although these can clearly use up part of the capacity of the column sections and lead to increased ductility.

- No account was taken of the potential effects of decreased section capacities caused by deterioration of the column and girder properties developed over the years, although these properties have deteriorated.

Calculations of the peak lateral girder response were made for the case of peak bedrock motions varying from 0.05 g's (corresponding approximately to a magnitude 5 seismic event input at bedrock levels) to 0.15 g's (corresponding to the input levels currently recommended by the Uniform Building Code) in increments of 0.025 g's. Calculated bent responses were determined for various cases of train configurations operating along the two lines, since the train masses contribute significantly to the total mass of the system and therefore to the final ductility achieved at the column joint. The train configurations were varied from the case where no trains were located on the tracks to the case where as many trains as possible were located on the tracks, which depends on the particular line and bent location being considered.

For the studies along the Jamaica Line, the structural model used in the calculations is shown in Figure 5-1. For this structure, the bent height varies along the line from about 18 feet to as much as 37 feet to the centerline of the bent girder. The width of the bents is 43 feet to the column centerlines. Evaluations of the peak lateral response for the case of an input bedrock acceleration level of 0.05 g's are shown in Figure 5-3. As can be noted, the response remains essentially elastic (no permanent strains in the column joint) for all cases along the line except for the case where three trains are on the tracks near the Marcy Avenue section of the line, and where two trains are located on the tracks near the 115th Street location near the end of the line. For both these two cases, the ductility ratio slightly exceeds 1, indicating some small exceedance of the capacity of the sections. However, it is most likely that no significant damage would occur for this level input to the structure. The primary differences between these two ends of the line where some exceedances develop is the thickness of the soil overburden and the height of the bent columns. Both of these parameters impact the primary frequency of transmission of waves up the soil column and the frequency of the bent itself. The relation between these frequencies determines the capability of the input motion for causing damage to the bent columns. The frequencies of both the bent structures and the soil column are shown in Figure 5-2 for the various locations along the Jamaica line.

Figures 5-4 through 5-7 present the same data for the Jamaica line for increasing levels of seismic input to the bedrock levels. As may be expected, increased input levels cause an increase in the ductility levels reached by the bent structures. For the case of 0.15 g's input at bedrock level (the level currently recommended by the UBC) shown in Figure 5-7, the



Mass of Top	=	3.33 k-sec²/ft
Mass of Foundation	=	1.03 k-sec²/ft
Mass of 1 Train	=	4.16 k-sec²/ft
A Column	=	44.1 in.²
I Column	=	1,950.9 in.⁴
I Cross Girder	=	102,254.0 in.⁴
Moment Capacity	=	848.3 ft-k

FIGURE 5-1 JAMAICA LINE BENT CROSS-SECTION

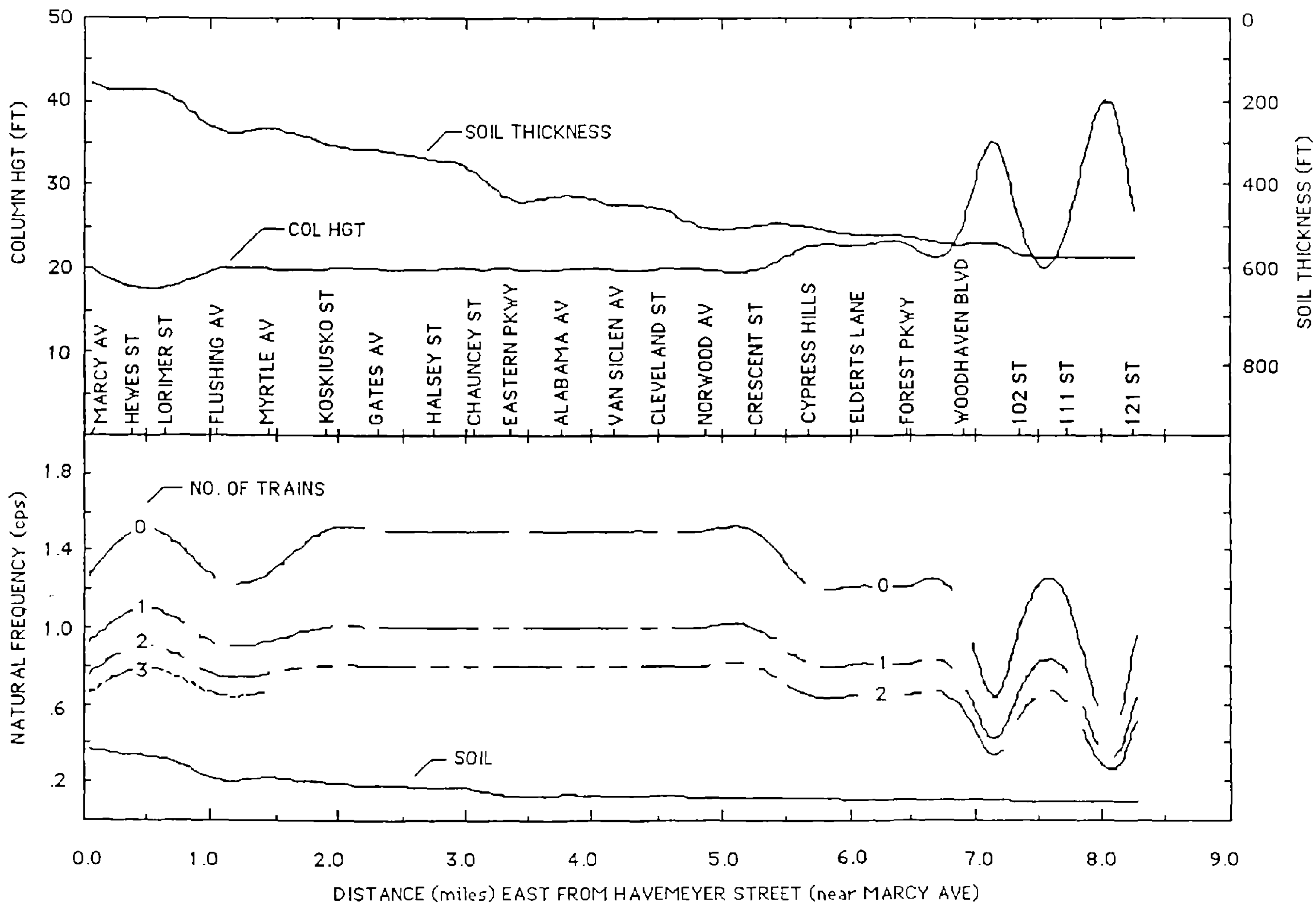


FIGURE 5-2 STRUCTURAL AND SOIL FREQUENCIES ALONG THE JAMAICA LINE

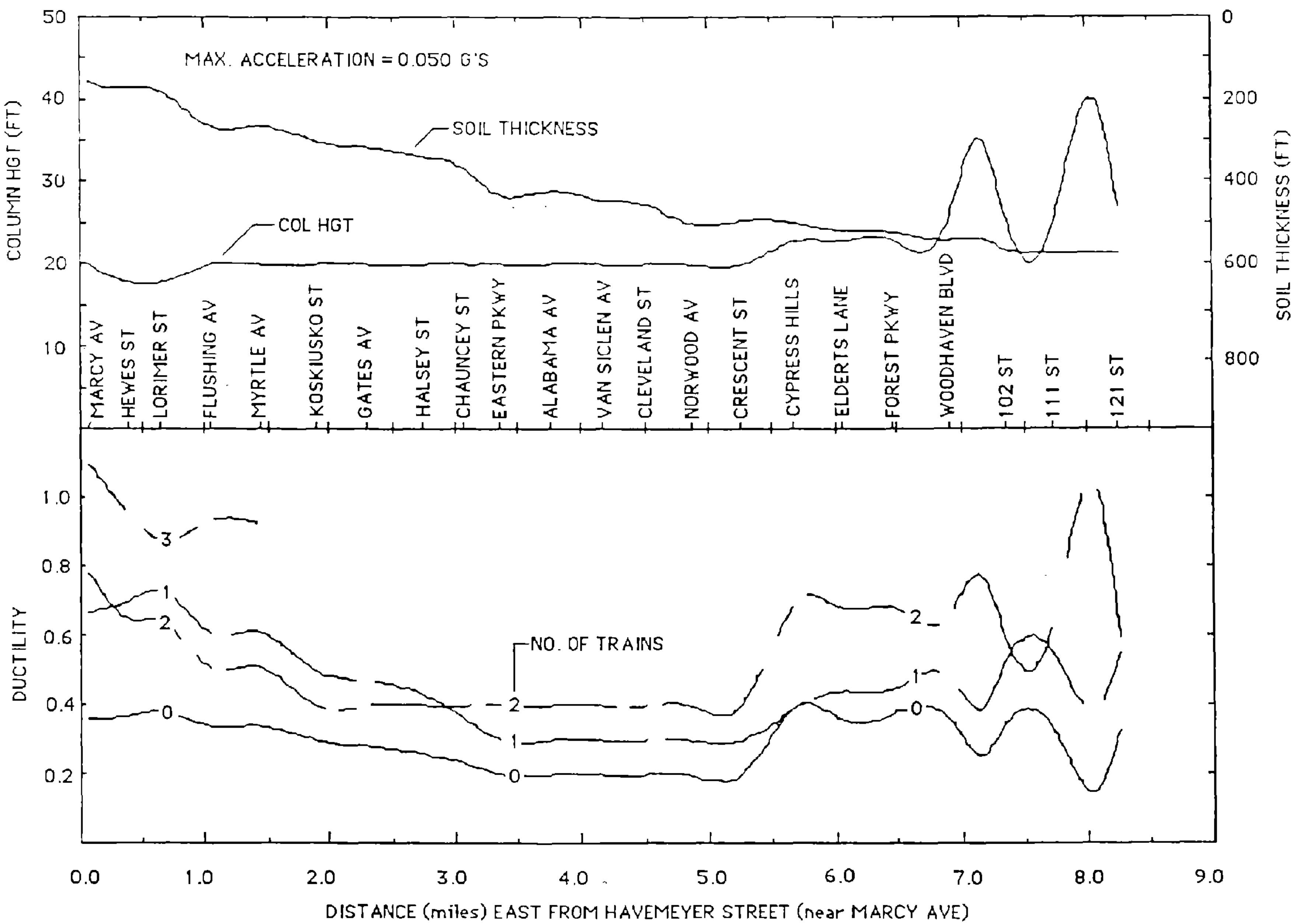
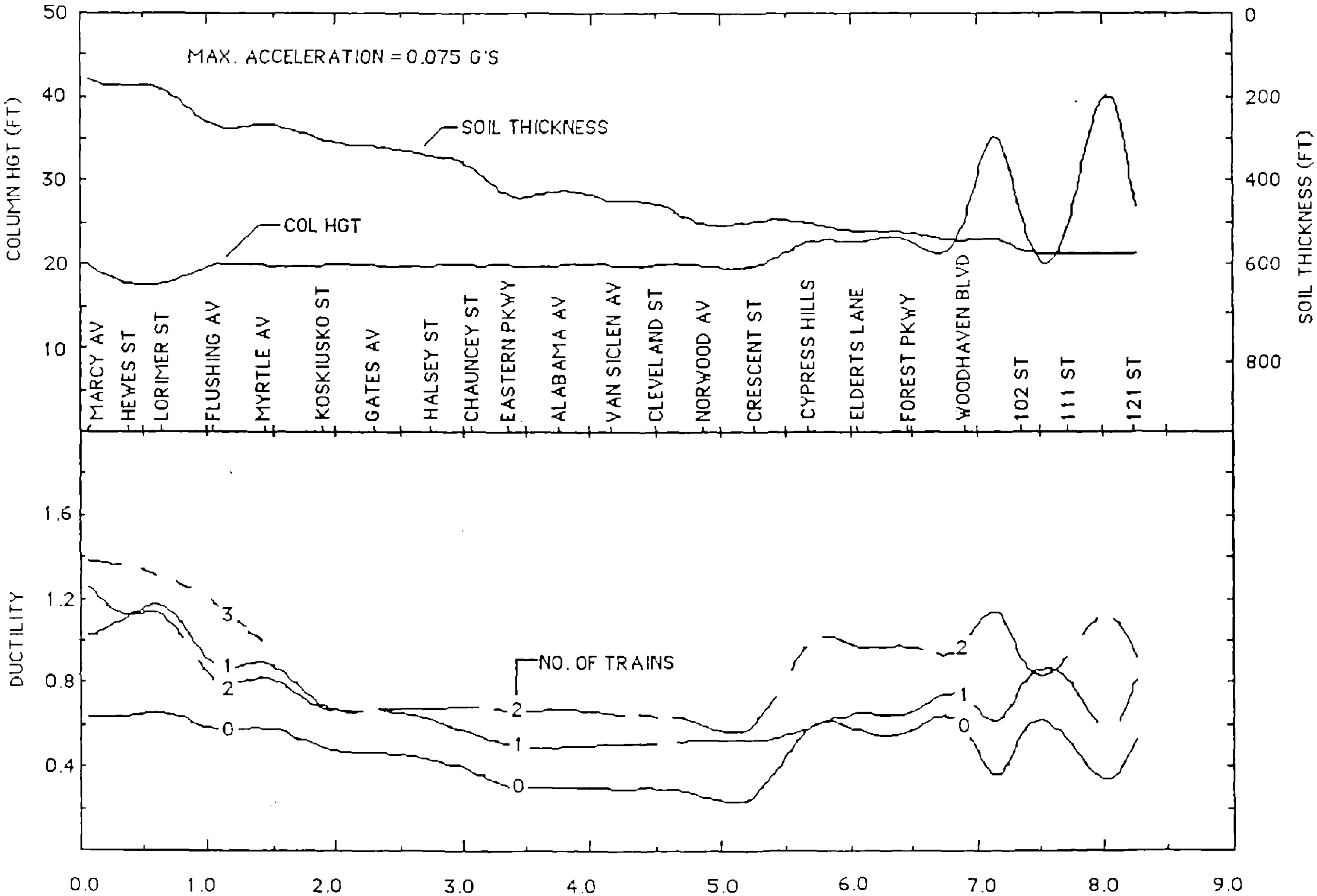


FIGURE 5-3 DUCTILITY ALONG JAMAICA LINE FOR 0.05G BEDROCK SEISMIC MOTION



DISTANCE (miles) EAST FROM HAVEMEYER STREET (near MARCY AVE)

FIGURE 5-4 DUCTILITY ALONG JAMAICA LINE FOR 0.075G BEDROCK SEISMIC MOTION

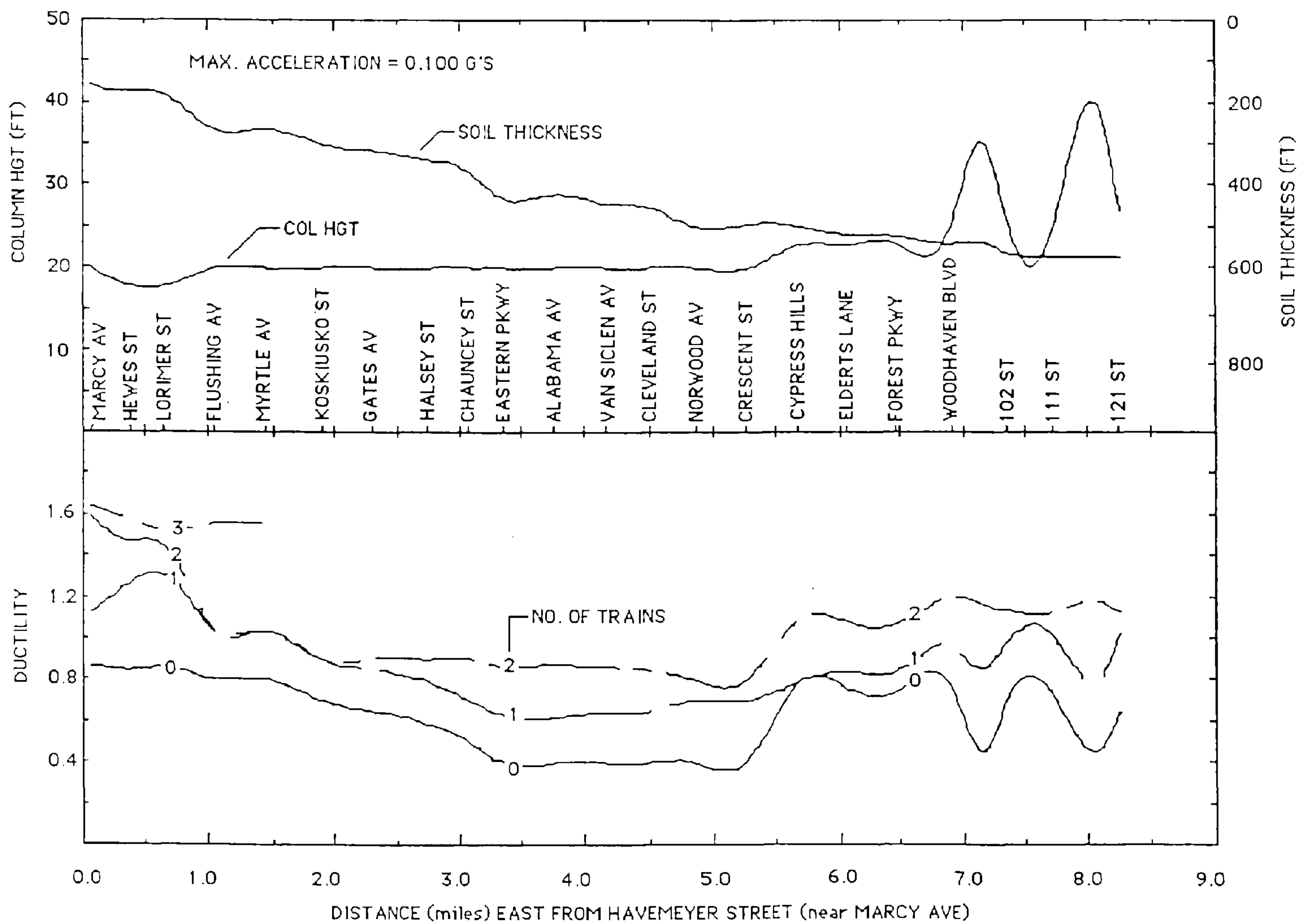


FIGURE 5-5 DUCTILITY ALONG JAMAICA LINE FOR 0.100G BEDROCK SEISMIC MOTION

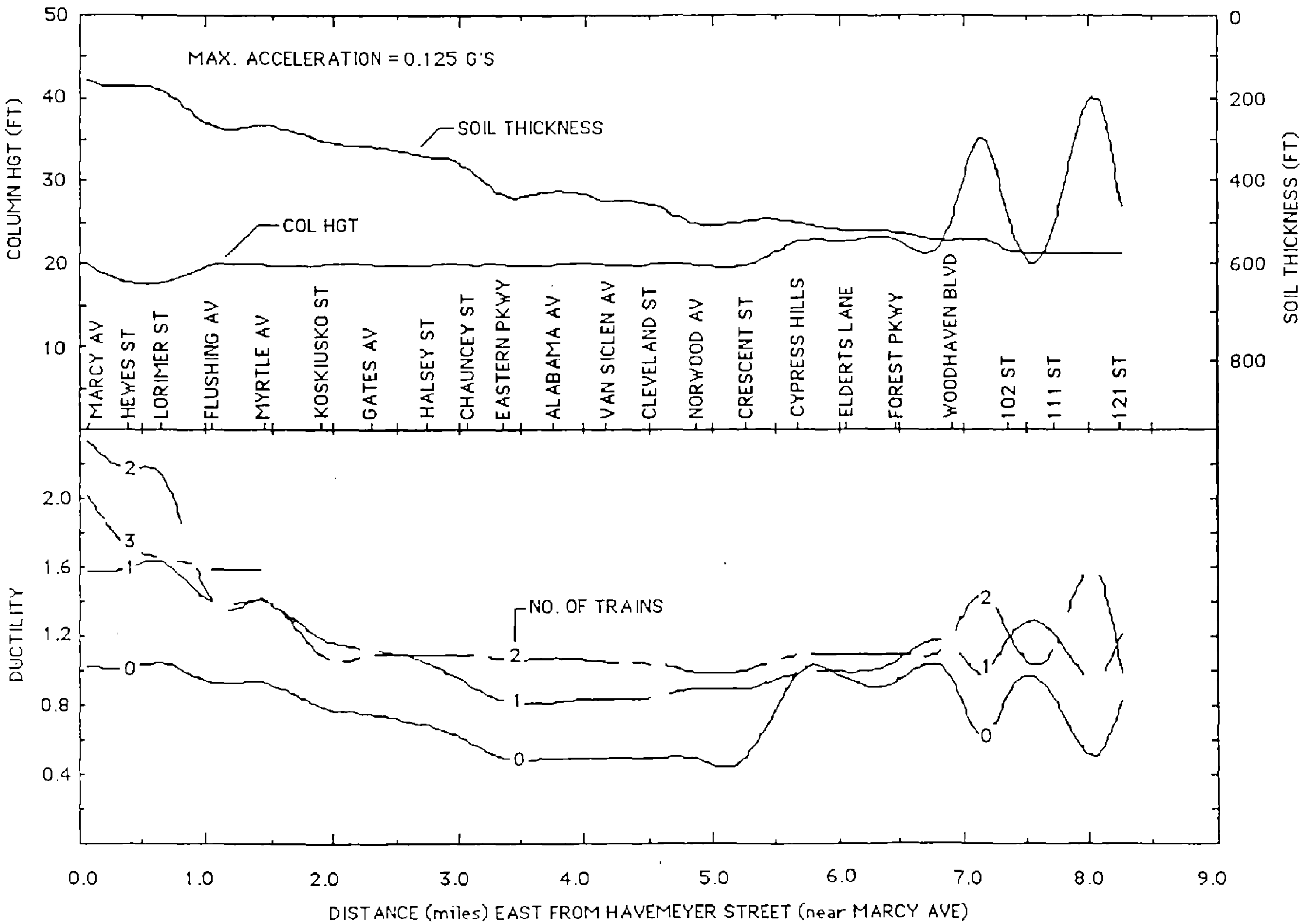


FIGURE 5-6 DUCTILITY ALONG JAMAICA LINE FOR 0.125G BEDROCK SEISMIC MOTION

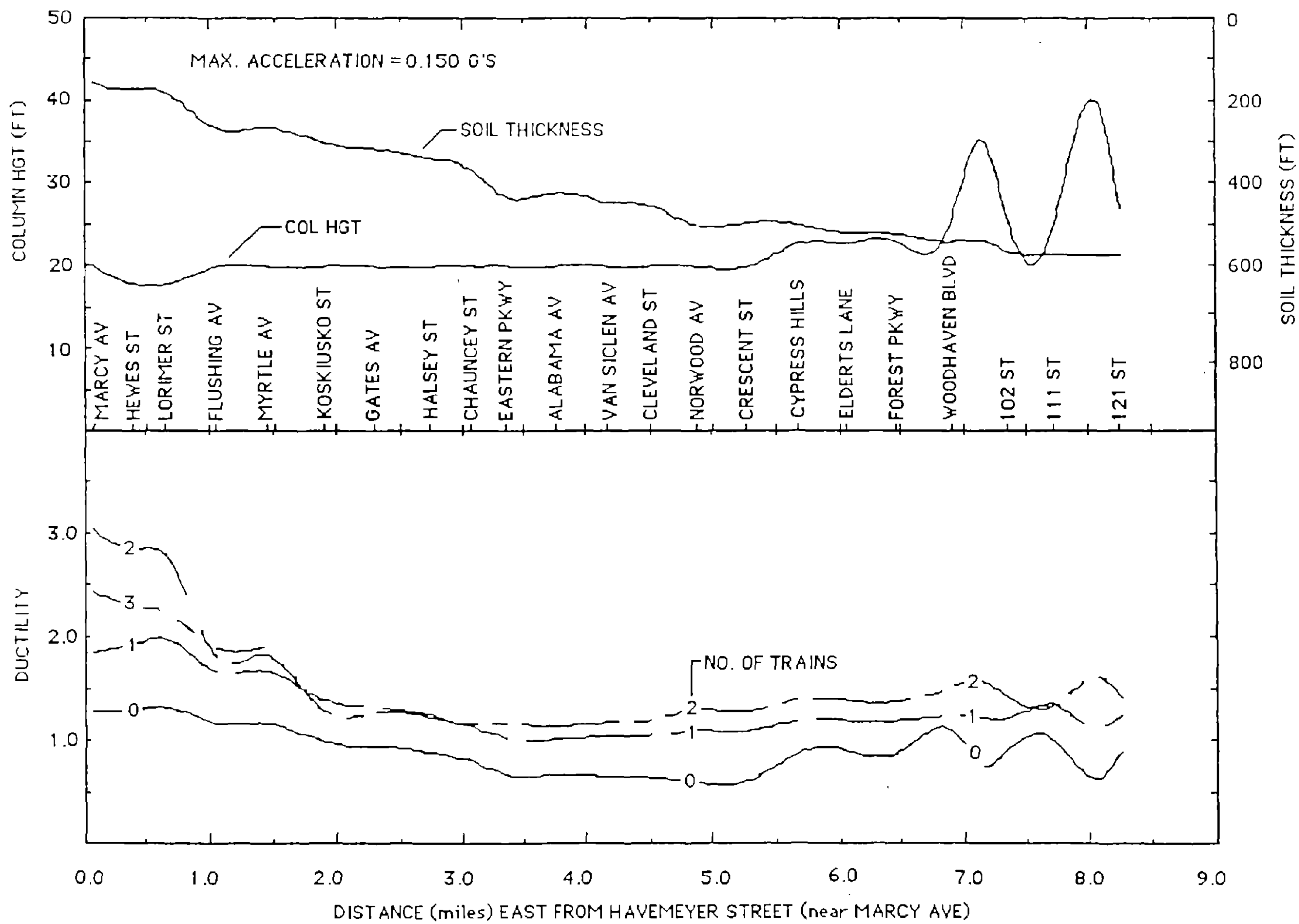


FIGURE 5-7 DUCTILITY ALONG JAMAICA LINE FOR 0.150G BEDROCK SEISMIC MOTION

peak response significantly exceeds a value of unity over a major portion of the line, even if no trains are positioned along the trackway. Near Marcy Avenue, the ductility ratio exceeds a value of three, even when only two trains are located along the line, indicating that such a situation will lead to extensive damage to the structure. When three trains are placed on the bent in this area, the peak ductility (or exceedance of capacity) is actually less than that for only two trains. This effect illustrates the impact of the differences between frequencies of structural response and soil column on the final calculated responses. Again, it should be mentioned that the amount of damage or exceedance that would be sustained would most likely even exceed these values if all the effects mentioned above were included in the evaluations.

In addition to the potential for yielding developing at the top of the vertical columns of the bents, the potential for overturning of the column footing was evaluated. This was included in the evaluation since it was found that many of the footing pedestals were not constructed with vertical steel connecting the pedestal to the footing. A typical footing along the Jamaica line is shown in Figure 5-8 which presents the column base plate, concrete pedestal and footing configuration. Such a foundation design is relatively typical along many of the elevated lines of the NYCTA system. Overturning forces required to retain the bent were compared with the passive capacity of the foundation soils to restrain the pedestal, as indicated in Figure 5-8, for each combination of bent location, seismic input level, number of trains on the bent, etc. The safety factor against overturning is defined in these calculations primarily as the passive capacity of the resisting soils divided by the applied maximum overturning force induced by the seismic event. When this factor becomes less than unity, potential overturning of the

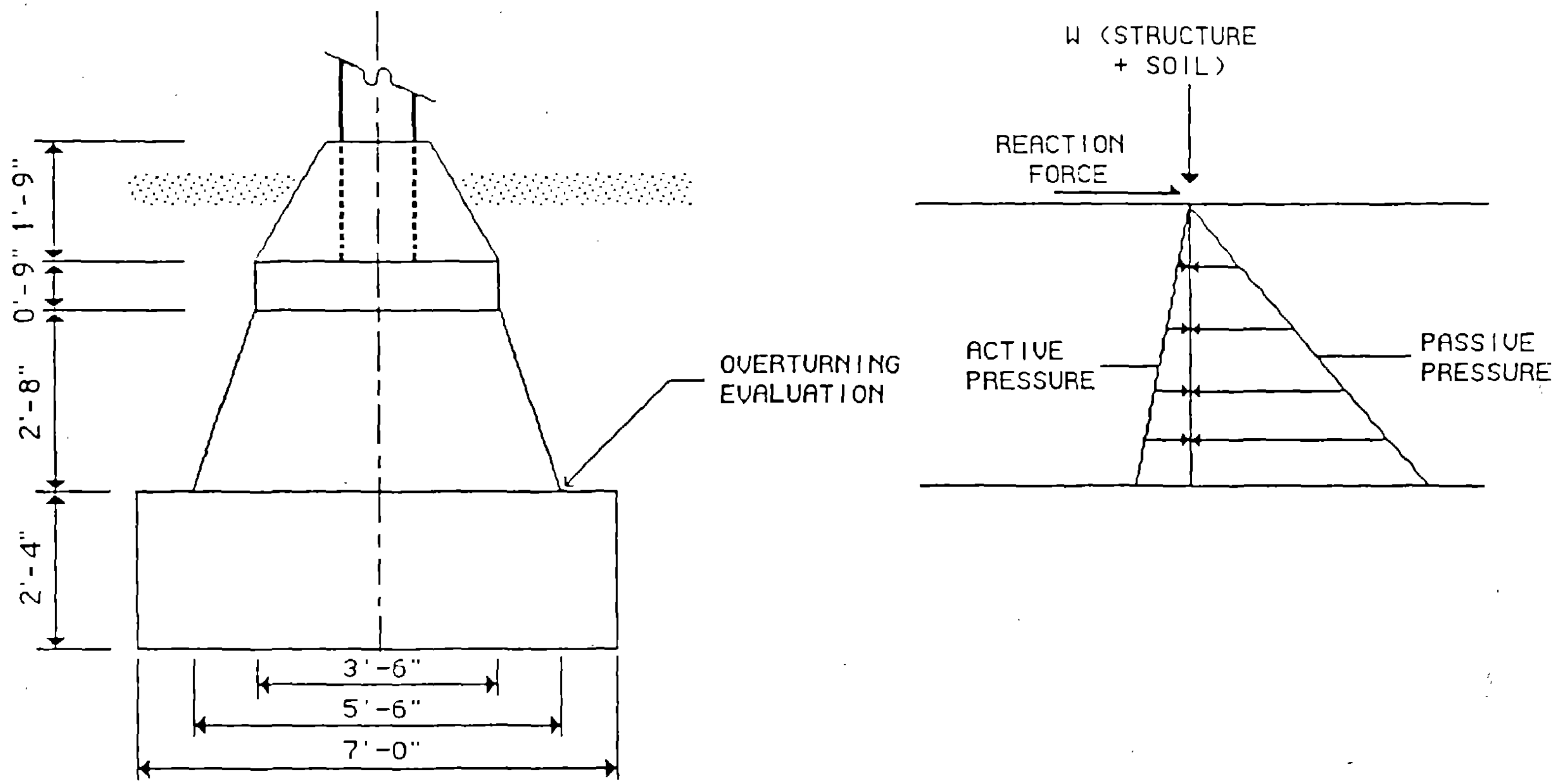


FIGURE 5-8 JAMAICA LINE COLUMN FOUNDATION
SAFETY FACTOR COMPUTATION

footing pedestal can occur and lead to failure of the bent. The results for the various levels of seismic input at the bedrock level are presented in Figures 5-9 through 5-13. For the lower level of seismic input of 0.05 g's shown in Figure 5-9, no particular problem is indicated since the safety factors calculated are greater than unity along the entire line. For the case of a peak acceleration level of 0.15 g's input to the bedrock, however, significant portions of the line show exceedance of the footing's lateral capacity. As shown in Figure 5-13, it may be noted that at some locations, this factor is significantly less than one, particularly for the case with no or one train located on the trackway. If no vertical steel exists in the pedestals at these locations to connect the pedestal to the footing, overturning could be a problem. Such an occurrence would lead to the effective failure of the bent, since the column would be displaced laterally from its normal position.

For the evaluation of the Flushing elevated line, two separate bent structures were evaluated in this study. The first bent cross-section is a single story bent section, applicable for the majority of the line, and is shown in Figure 5-14. This section can support as many as three trains along the trackway. In the Willets Point area, a double bent cross-section exists as shown in Figure 5-15. This structure can carry as many as four trains on the lower level in some locations of the line and two trains on the upper level. The natural frequencies of both the structure (with a variety of train load combinations) and the the soil column are shown in Figure 5-16. Similar data are shown in Figure 5-17 for the double bent structure. However, since so many train load combinations are possible for this configuration, the structural frequencies are shown only for those cases which were found to be the controlling cases for the double bent problem. The properties of the double bent sections of the Flushing line

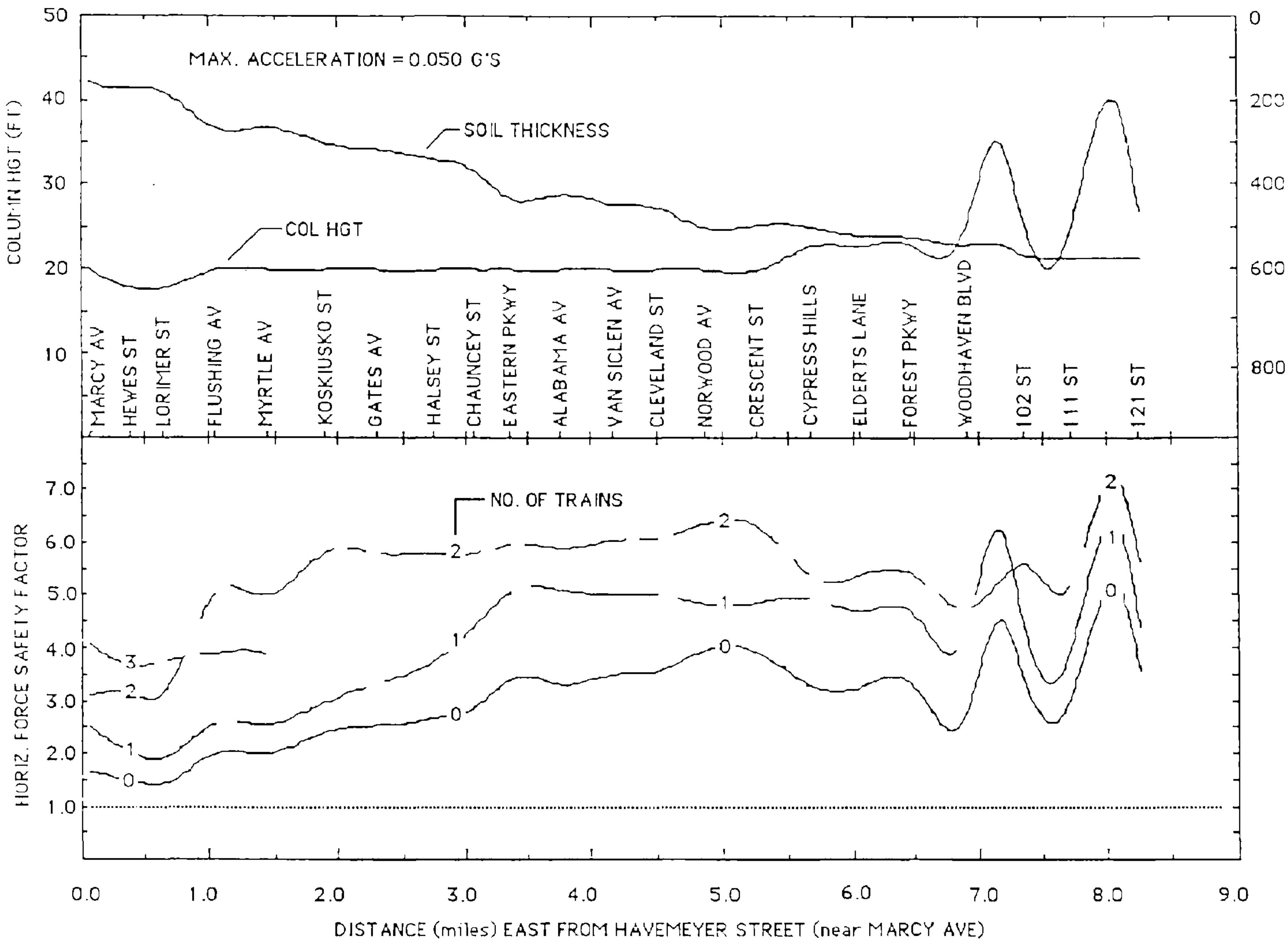


FIGURE 5-9 OVERTURNING SAFETY FACTOR FOR FOOTING ALONG THE JAMAICA LINE FOR A 0.050G SEISMIC INPUT AT BEDROCK

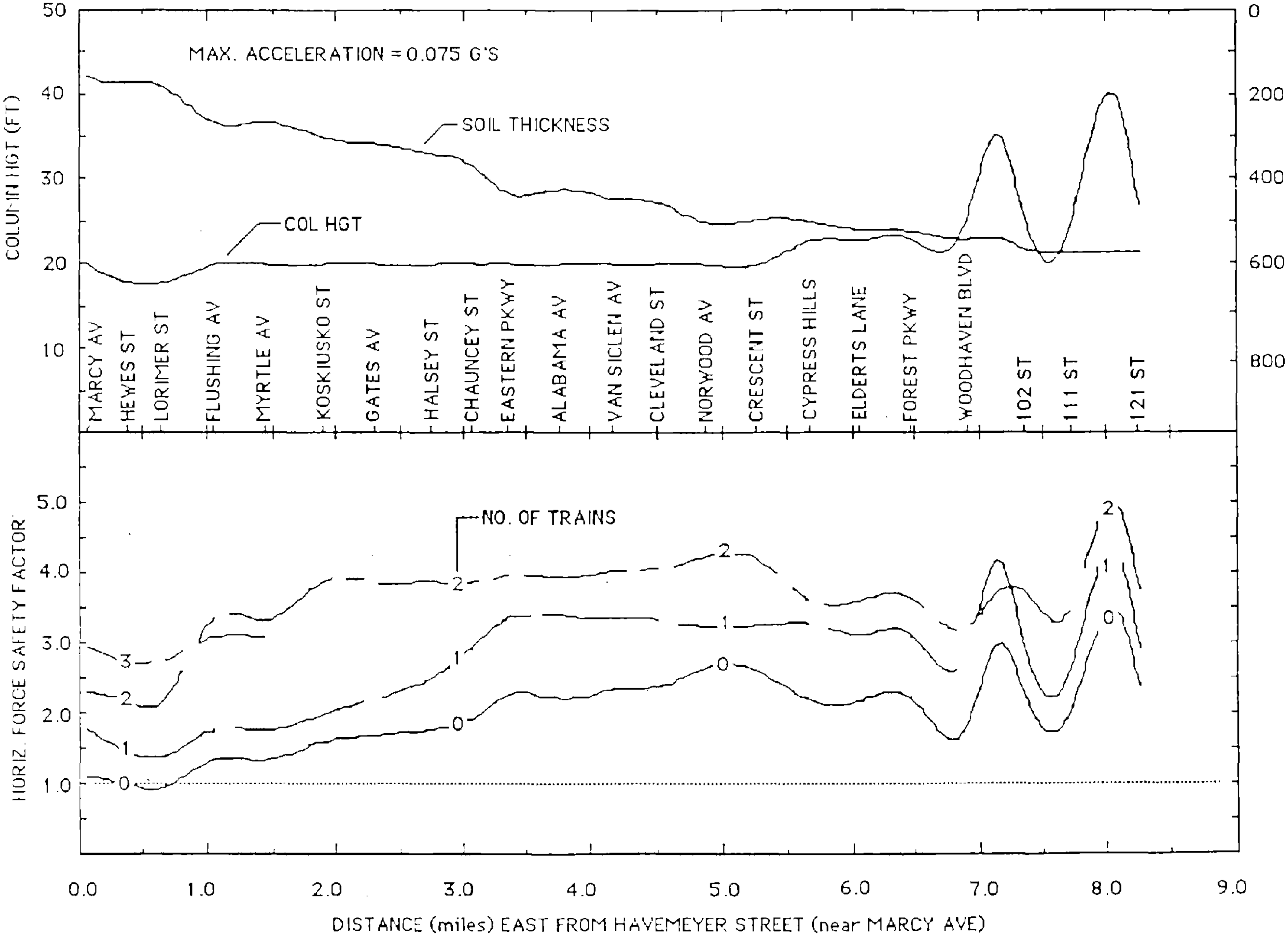


FIGURE 5-10 OVERTURNING SAFETY FACTOR FOR FOOTING ALONG THE JAMAICA LINE FOR A 0.075G SEISMIC INPUT AT BEDROCK

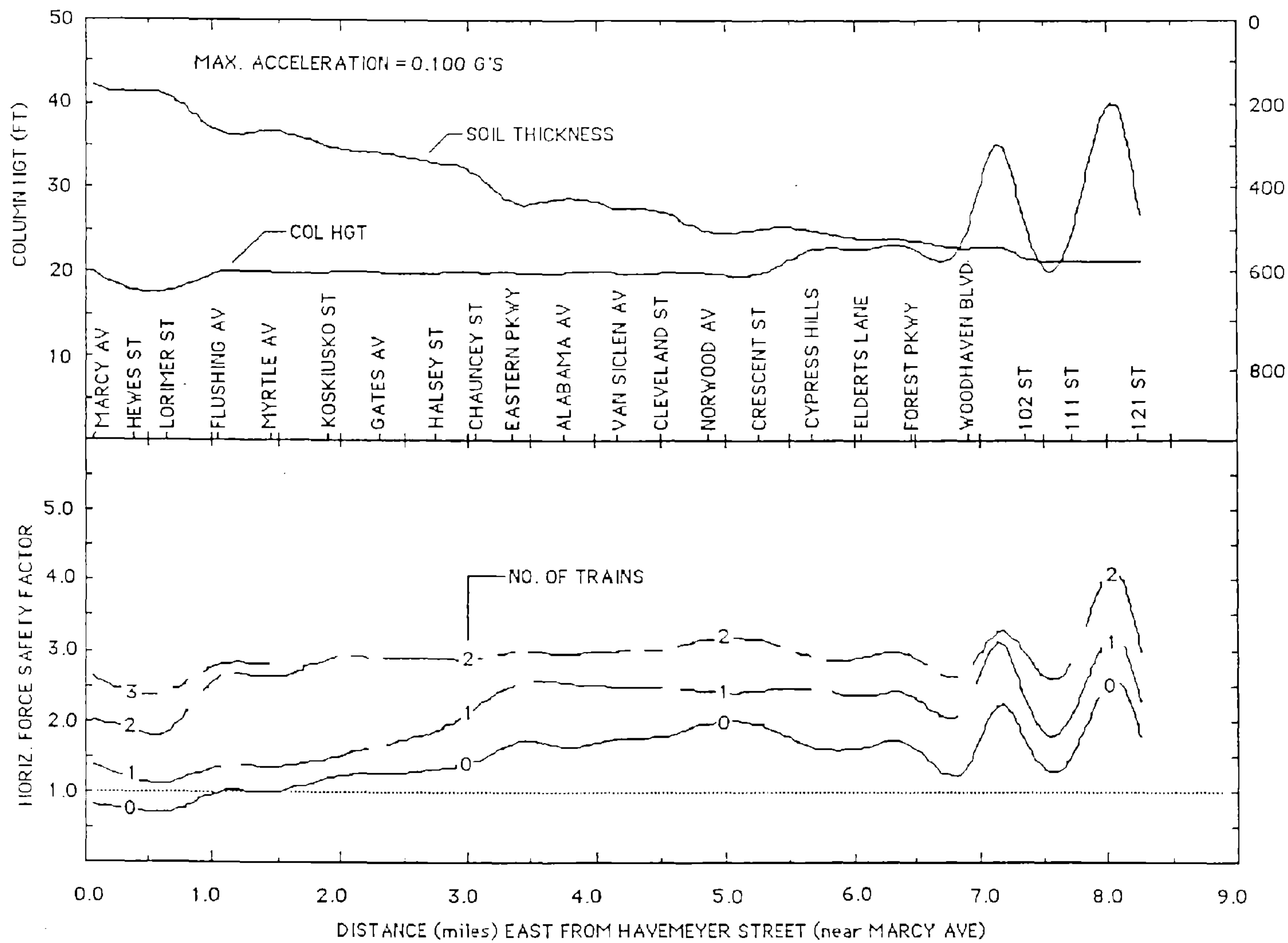


FIGURE 5-11 OVERTURNING SAFETY FACTOR FOR FOOTING ALONG THE JAMAICA LINE FOR A 0.100 G SEISMIC INPUT AT BEDROCK

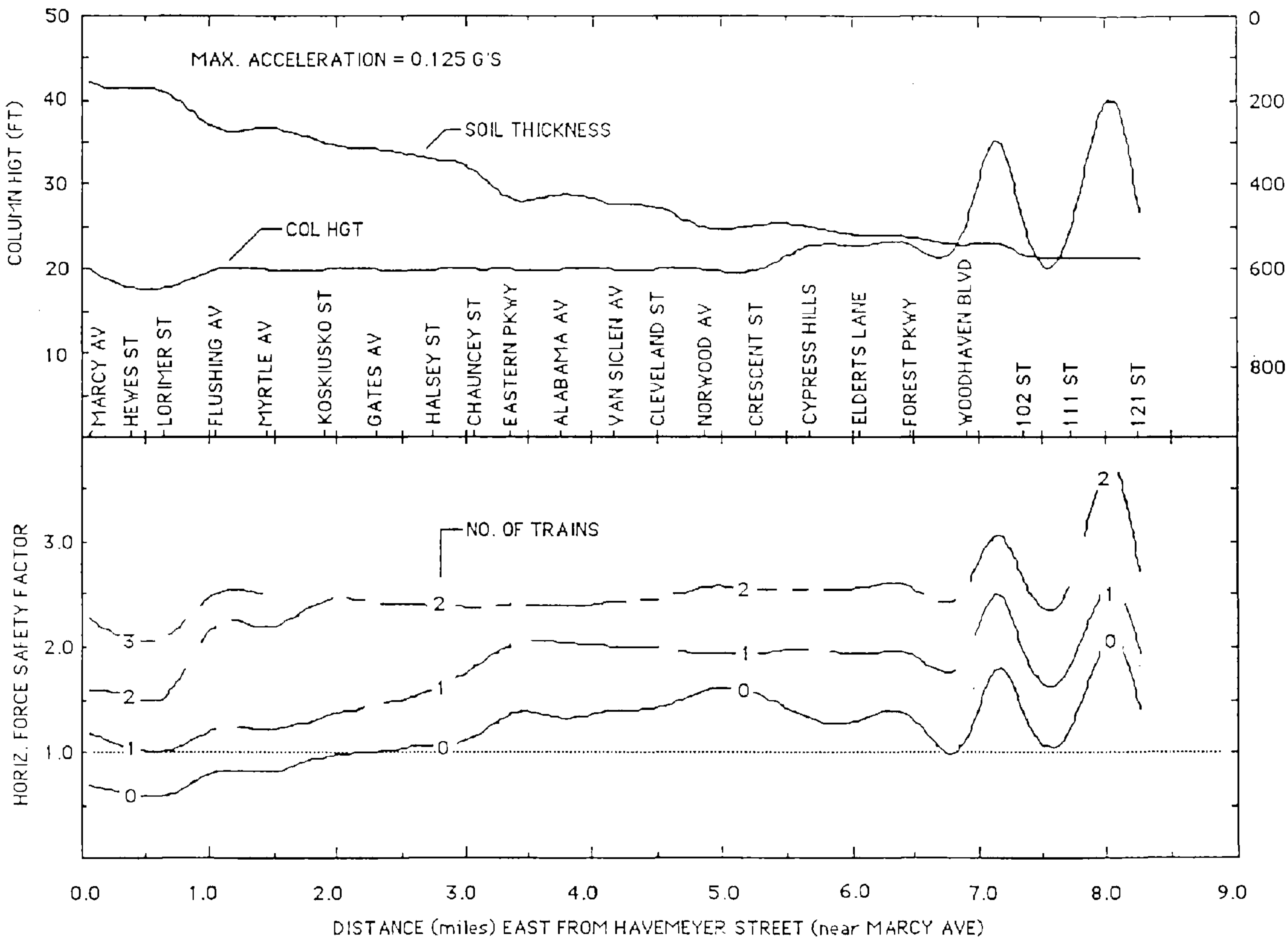


FIGURE 5-12 OVERTURNING SAFETY FACTOR FOR FOOTING ALONG THE JAMAICA LINE FOR A 0.125G SEISMIC INPUT AT BEDROCK

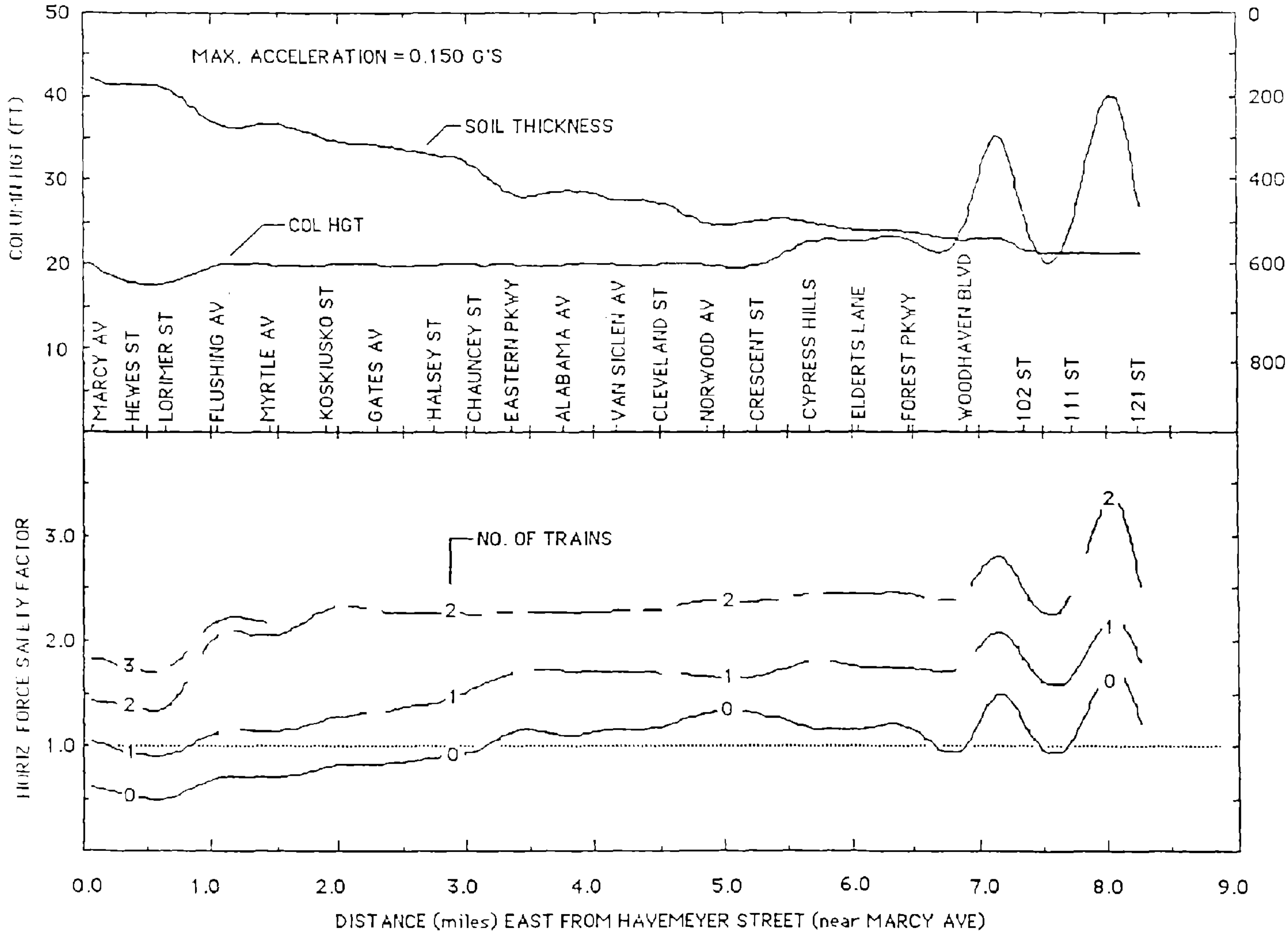
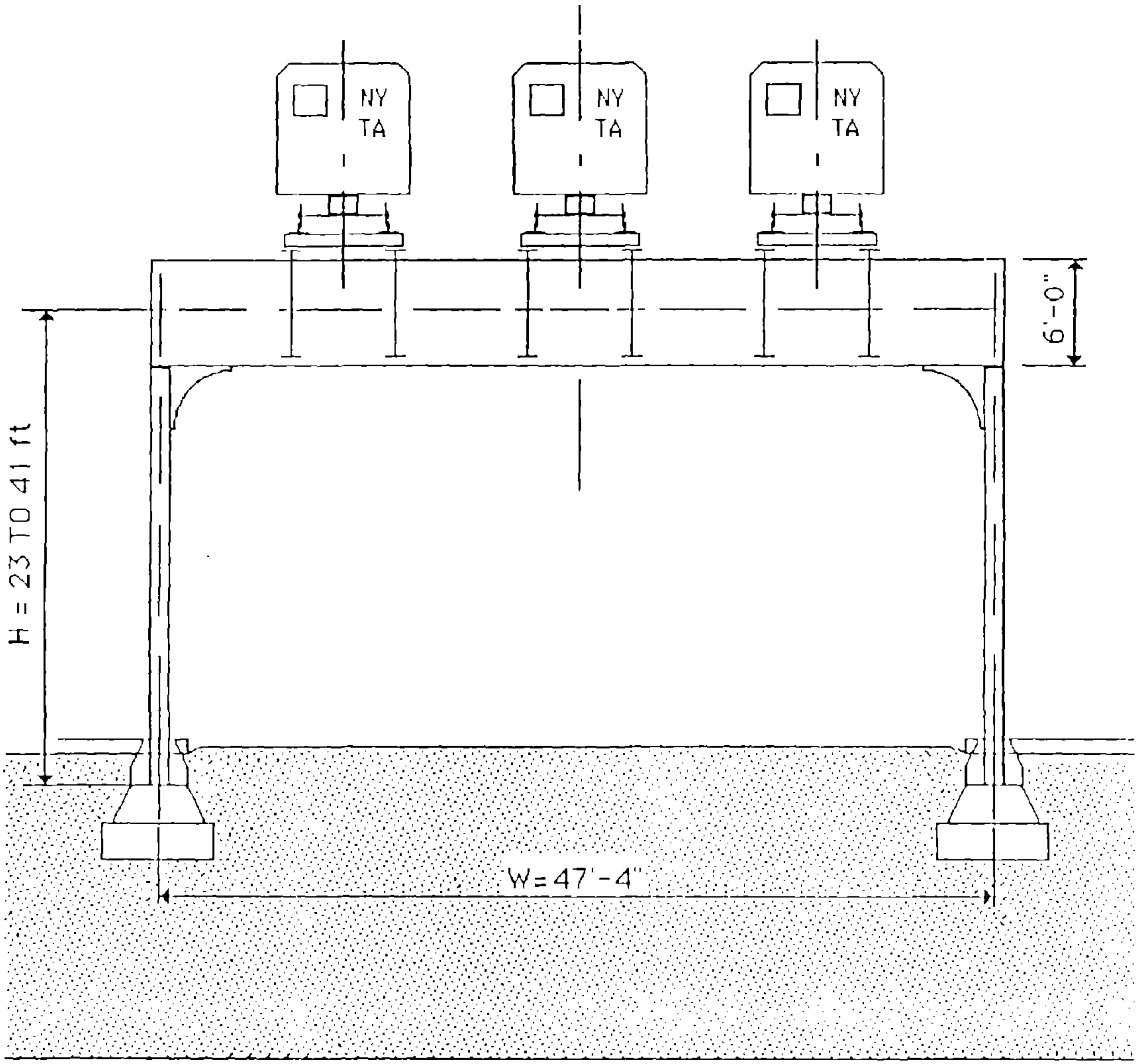


FIGURE 5-13 OVERTURNING SAFETY FACTOR FOR FOOTING ALONG THE JAMAICA LINE FOR A 0.150 G SEISMIC INPUT AT BEDROCK



Mass Top (0 Trains)	=	4.14 $k\text{-}sec^2/ft$
Mass Foundation	=	0.92 $k\text{-}sec^2/ft$
Mass of 1 Train	=	3.61 $k\text{-}sec^2/ft$
A (Column)	=	44.1 $in.^2$
I (Column)	=	2,087 $in.^4$
I (Cross Girder)	=	92,238 $in.^4$
Column Capacity	=	885 ft-k

FIGURE 5-14 FLUSHING LINE SINGLE STORY BENT PROFILE

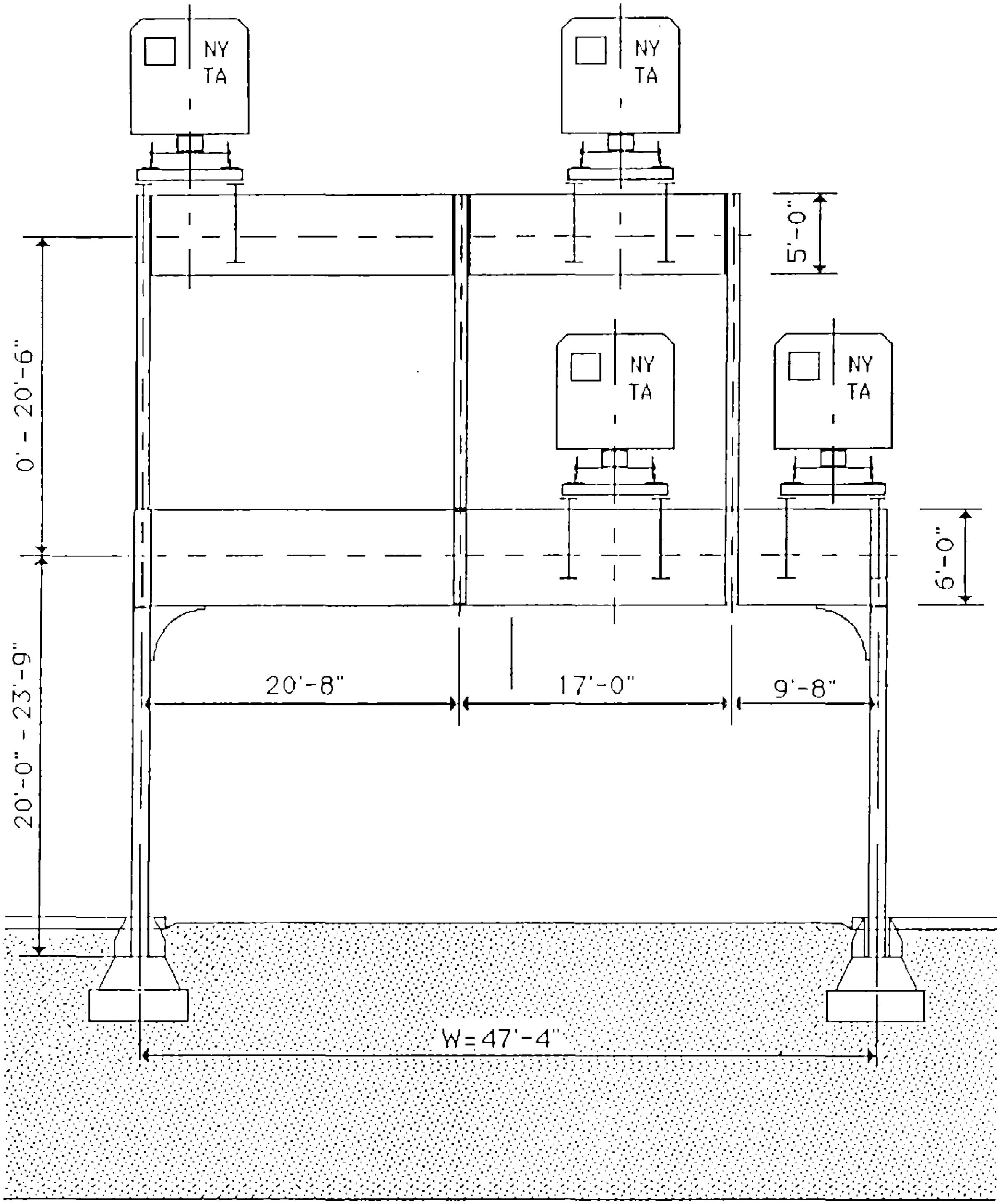


FIGURE 5-15 FLUSHING LINE TWO STORY BENT PROFILE IN THE WILLETS POINT SECTION

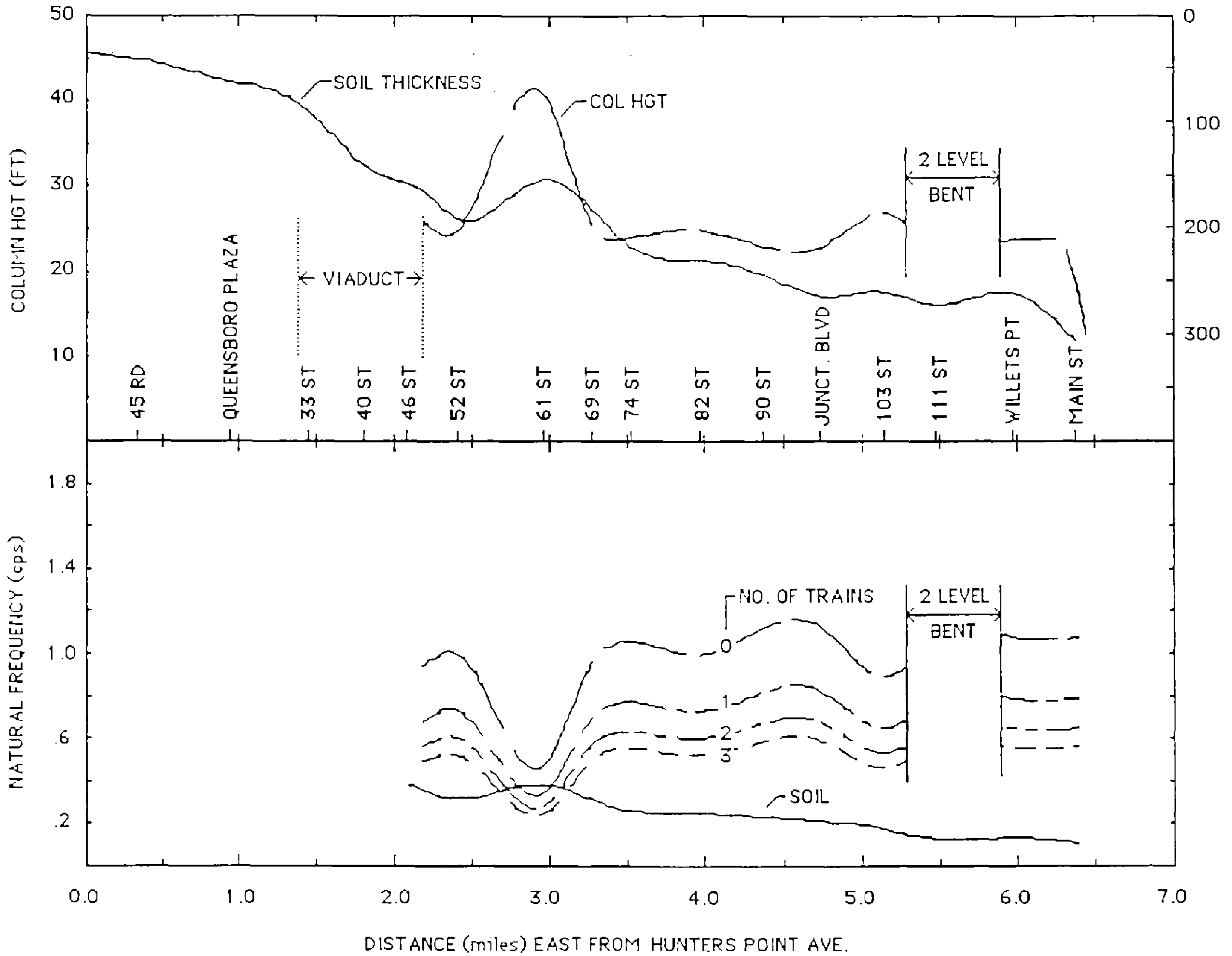


FIGURE 5-16 NATURAL FREQUENCIES ALONG FLUSHING LINE SINGLE LEVEL BENT

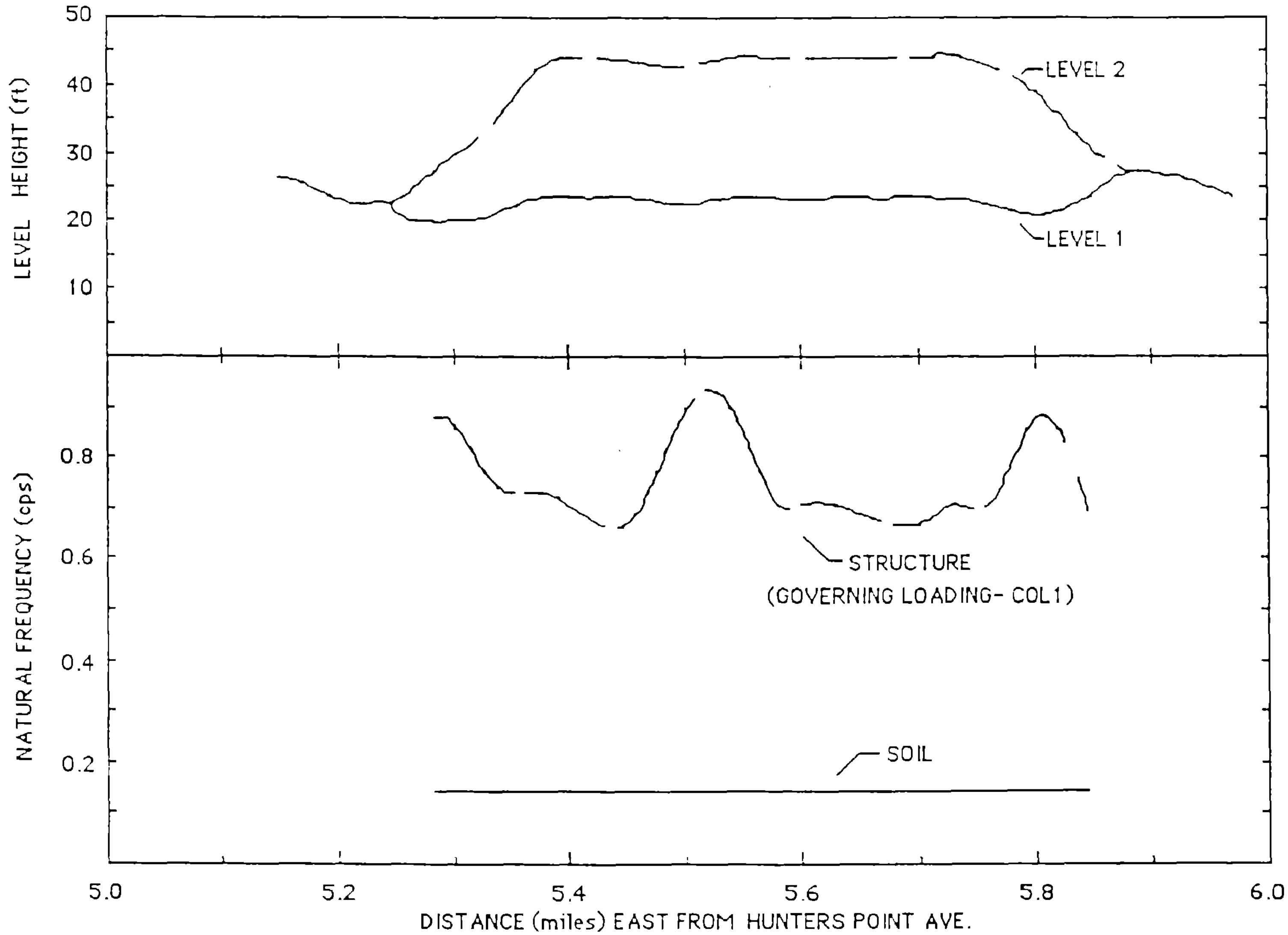


FIGURE 5-17 NATURAL FREQUENCIES ALONG FLUSHING LINE DOUBLE LEVEL BENT

are presented in Table V-1, while the critical train load combinations for these bents for each location and acceleration level considered are presented in Tables V-2 through V-6. Peak ductility ratios developed at both the first story and second story column-girder connections are also included in these tables.

The results for permanent displacements of the single bent structures along the Flushing line are shown in Figures 5-18 through 5-22. Again, for peak bedrock acceleration levels of 0.05 g's, no significant exceedances were noted in joint ductility. Ductility ratios were generally less than unity, except for some minor overstress indicated at several bent locations. For larger inputs, the picture changes, however, until for a peak acceleration level of 0.15 g's input at bedrock levels and shown in Figure 5-22, the ductility ratio of the single level bent system shows significant exceedances over a large portion of the line.

For the two level bents,the results for various acceleration input levels are presented in Figures 5-23 through 5-27. An envelope of maximum ductilities is plotted for each input level as a function of the bent location. The top sketch in these figures indicates the specific column height for both levels at the various locations, while the second indicates the maximum number of trains on each level. The lower sketch indicates the maximum ductility reached for each input level, from 0.05 g's to 0.15 g's peak acceleration at bedrock. Surprisingly, the results are not significantly different from those found in the single bent cases. At lower input levels (Figure 5-23), some exceedances in ductility occur with ductility ratios slightly exceeding unity. At the higher input levels (Figure 5-27), ductility ratios reach a value of about three. For the double bent structures, the

TABLE V-1

FLUSHING LINE - PROPERTIES FOR TWO LEVEL BENTS

BENT NO.	DISTANCE (MILES)	MASS 1 0 TRAINS (KS^2/FT)	MASS 2 0 TRAINS (KS^2/FT)	I_1 EQUIV. (FT^4)	I_2 EQUIV. (FT^4)	MOMENT CAPACITY COLUMN 1 (FT-K)	MOMENT CAPACITY COLUMN 2 (FT-K)
519	5.282	4.20	1.27	0.3089	0.0557	2532	630
520	5.291	4.20	1.27	0.3089	0.0557	2532	630
524	5.333	4.20	1.27	0.3368	0.0557	2730	630
528	5.373	4.20	1.27	0.4564	0.0883	3552	972
532	5.414	4.84	1.27	0.4882	0.0727	3762	816
536	5.455	4.84	1.27	0.4882	0.0727	3762	816
540	5.497	4.84	1.27	0.5552	0.0727	4194	816
544	5.537	4.84	1.27	0.7733	0.0807	5554	894
548	5.580	4.84	1.27	0.7209	0.0807	3972	894
550	5.601	4.84	1.27	0.5209	0.0807	3972	894
563	5.713	3.00	2.54	0.4253	0.1614	3342	1878
564	5.723	3.00	2.54	0.4723	0.1209	3657	1341
568	5.764	3.00	2.54	0.4253	0.1161	3342	1194
572	7.807	3.00	2.54	0.3947	0.1454	3132	1632
576	5.845	3.00	2.54	0.3947	0.0963	3132	1104

TRAIN MASS = 3.61 KS^2/FT PER TRAIN

TABLE V-2

FLUSHING LINE - MAXIMUM COMPUTED DUCTILITY FOR TWO LEVEL BENT FOR 0.05 G SEISMIC INPUT MOTION

BENT	MAXIMUM DUCTILITY COLUMN 1			MAXIMUM DUCTILITY COLUMN 2		
	No. of Trains		Ductility	No. of Trains		Ductility
	Level 1	Level 2		Level 1	Level 2	
519	3	0	0.81	2	1	0.18
520	3	0	0.81	2	1	0.22
524	3	1	0.93	0	1	0.37
528	3	1	1.14	3	1	0.46
532	4	1	1.18	3	1	0.64
536	4	1	1.18	3	1	0.64
540	4	0	0.69	2	1	0.64
544	3	1	0.60	3	1	0.57
548	4	1	1.14	4	1	0.64
550	4	1	1.14	4	1	0.64
563	2	2	1.21	2	2	0.53
564	2	2	1.11	2	2	0.93
568	2	2	1.13	2	2	1.00
572	2	1	0.70	0	2	0.32
576	2	2	1.14	2	2	0.25

TABLE V-3

FLUSHING LINE - MAXIMUM COMPUTED DUCTILITY FOR TWO LEVEL BENT FOR 0.075 G SEISMIC INPUT MOTION

BENT	MAXIMUM DUCTILITY COLUMN 1			MAXIMUM DUCTILITY COLUMN 2		
	No. of Trains		Ductility	No. of Trains		Ductility
	Level 1	Level 2		Level 1	Level 2	
519	2	1	1.23	2	1	0.25
520	2	1	1.22	2	1	0.30
524	3	1	1.37	0	1	0.56
528	3	1	1.24	3	1	0.64
532	4	1	1.25	3	1	0.77
536	4	1	1.25	3	1	0.77
540	4	0	1.04	2	1	0.96
544	3	1	0.90	3	1	0.85
548	4	1	1.27	4	1	0.79
550	4	1	1.27	4	1	0.79
563	2	2	1.25	2	2	0.59
564	2	2	1.29	2	2	1.05
568	2	2	1.39	2	2	1.20
572	2	1	1.04	0	2	0.48
576	2	2	1.25	0	2	0.28

TABLE V-4

FLUSHING LINE - MAXIMUM COMPUTED DUCTILITY FOR TWO LEVEL BENT FOR 0.100 G SEISMIC INPUT MOTION

BENT	MAXIMUM DUCTILITY COLUMN 1			MAXIMUM DUCTILITY COLUMN 2		
	No. of Trains		Ductility	No. of Trains		Ductility
	Level 1	Level 2		Level 1	Level 2	
519	3	0	1.38	2	1	0.25
520	3	0	1.38	2	1	0.31
524	3	1	1.32	0	1	0.74
528	3	1	1.35	3	1	0.72
532	4	1	2.00	1	1	0.92
536	4	1	2.00	1	1	0.92
540	4	0	1.16	2	1	1.16
544	3	1	1.08	3	1	1.01
548	4	1	1.63	4	1	1.01
550	4	1	1.63	4	1	1.01
563	2	2	1.73	2	2	0.67
564	2	2	1.32	2	2	1.18
568	2	2	1.39	2	2	1.30
572	2	1	1.23	0	2	0.64
576	2	2	1.67	0	2	0.37

TABLE V-5

FLUSHING LINE - MAXIMUM COMPUTED DUCTILITY FOR TWO LEVEL BENT FOR 0.125 G SEISMIC INPUT MOTION

BENT	MAXIMUM DUCTILITY COLUMN 1			MAXIMUM DUCTILITY COLUMN 2		
	No. of Trains		Ductility	No. of Trains		Ductility
	Level 1	Level 2		Level 1	Level 2	
519	3	0	1.42	1	1	0.30
520	3	0	1.42	1	1	0.35
524	3	1	1.45	0	1	0.82
528	3	1	1.63	1	1	0.80
532	4	1	2.59	1	1	1.08
536	4	1	2.59	1	1	1.08
540	2	1	1.52	2	1	1.19
544	3	1	1.38	3	1	1.14
548	4	1	1.86	4	1	1.09
550	4	1	1.86	4	1	1.09
563	2	2	2.50	2	2	0.75
564	2	2	1.85	2	2	1.36
568	2	2	1.69	2	2	1.37
572	2	1	1.57	0	2	0.75
576	2	2	1.96	0	2	0.38

TABLE V-6

FLUSHING LINE - MAXIMUM COMPUTED DUCTILITY FOR TWO LEVEL BENT FOR 0.150 G SEISMIC INPUT MOTION

BENT	MAXIMUM DUCTILITY COLUMN 1			MAXIMUM DUCTILITY COLUMN 2		
	No. of Trains		Ductility	No. of Trains		Ductility
	Level 1	Level 2		Level 1	Level 2	
519	3	1	1.52	1	1	0.33
520	3	1	1.50	1	1	0.38
524	3	1	1.57	0	1	0.88
528	3	1	1.54	1	1	0.88
532	4	1	2.76	1	1	1.31
536	4	1	2.76	1	1	1.31
540	2	1	1.41	2	1	1.41
544	4	1	1.28	1	1	1.32
548	4	1	2.10	4	1	1.20
550	4	1	2.10	4	1	1.20
563	2	2	2.73	2	2	0.79
564	2	2	2.18	2	2	1.31
568	2	2	1.74	2	2	1.40
572	2	1	1.52	0	2	0.79
576	2	2	2.18	0	2	0.39

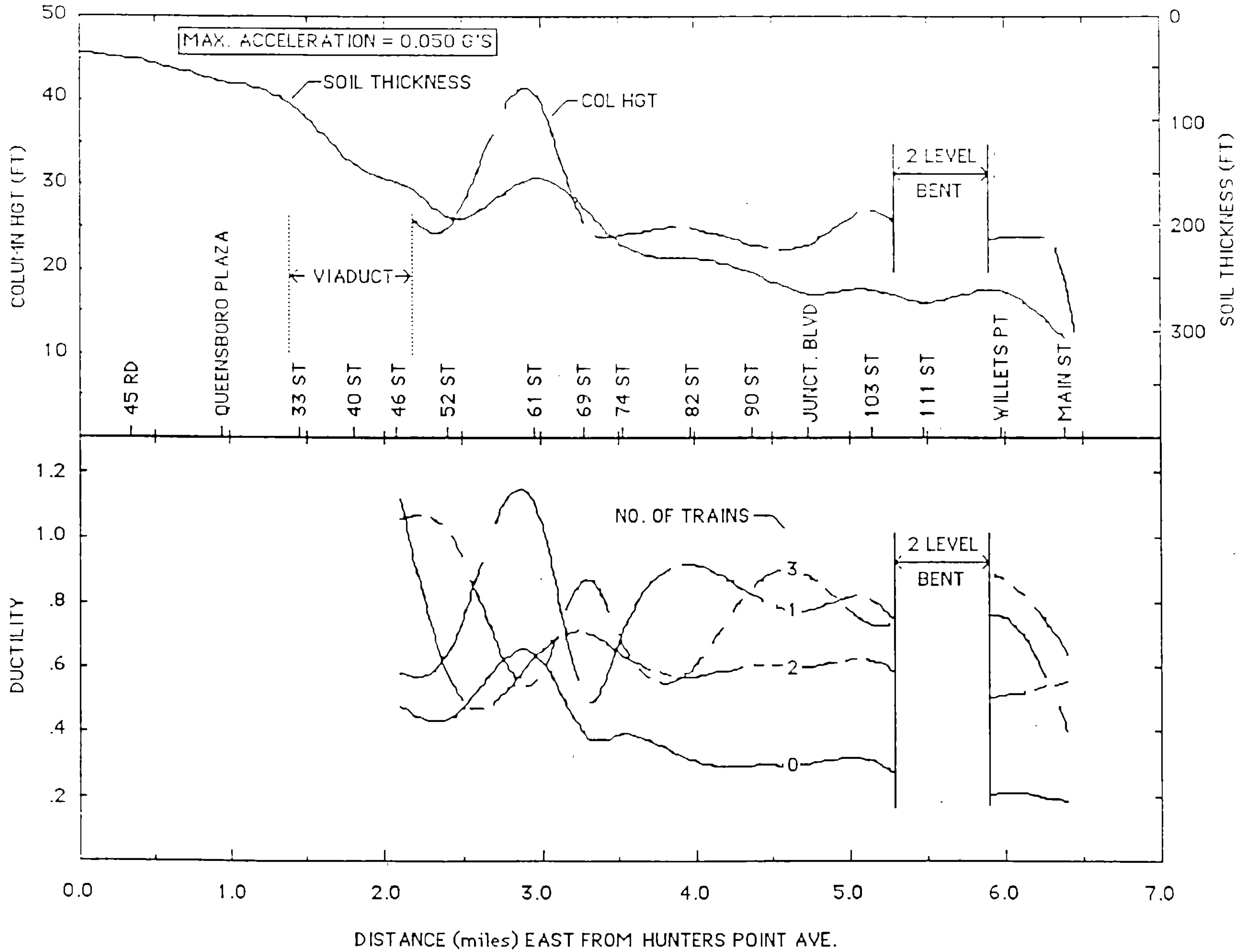


FIGURE 5-18 DUCTILITY ALONG THE FLUSHING LINE (SINGLE BENT) FOR 0.050G BEDROCK SEISMIC MOTION

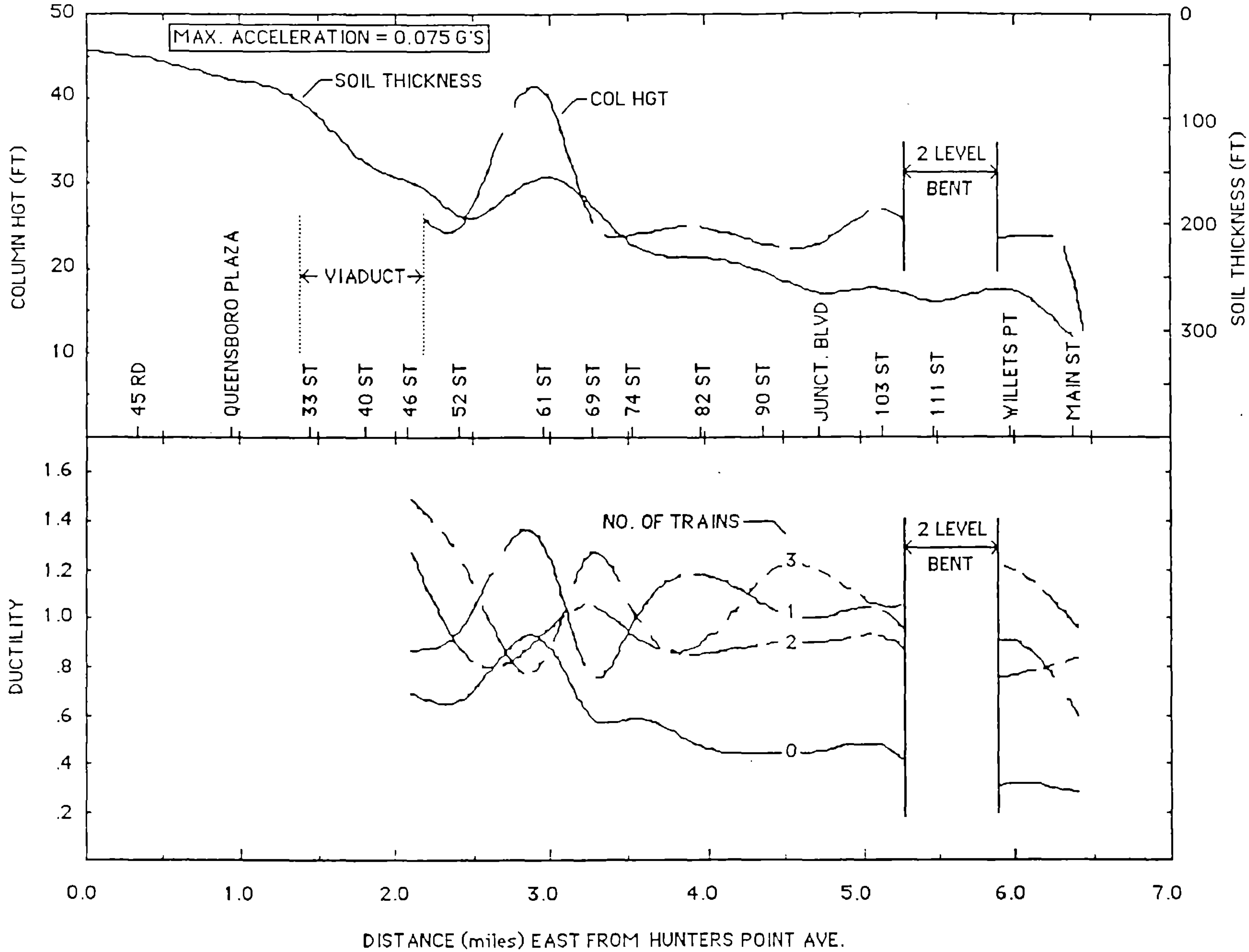


FIGURE 5-19 DUCTILITY ALONG THE FLUSHING LINE (SINGLE BENT) FOR 0.075 0G BEDROCK SEISMIC MOTION

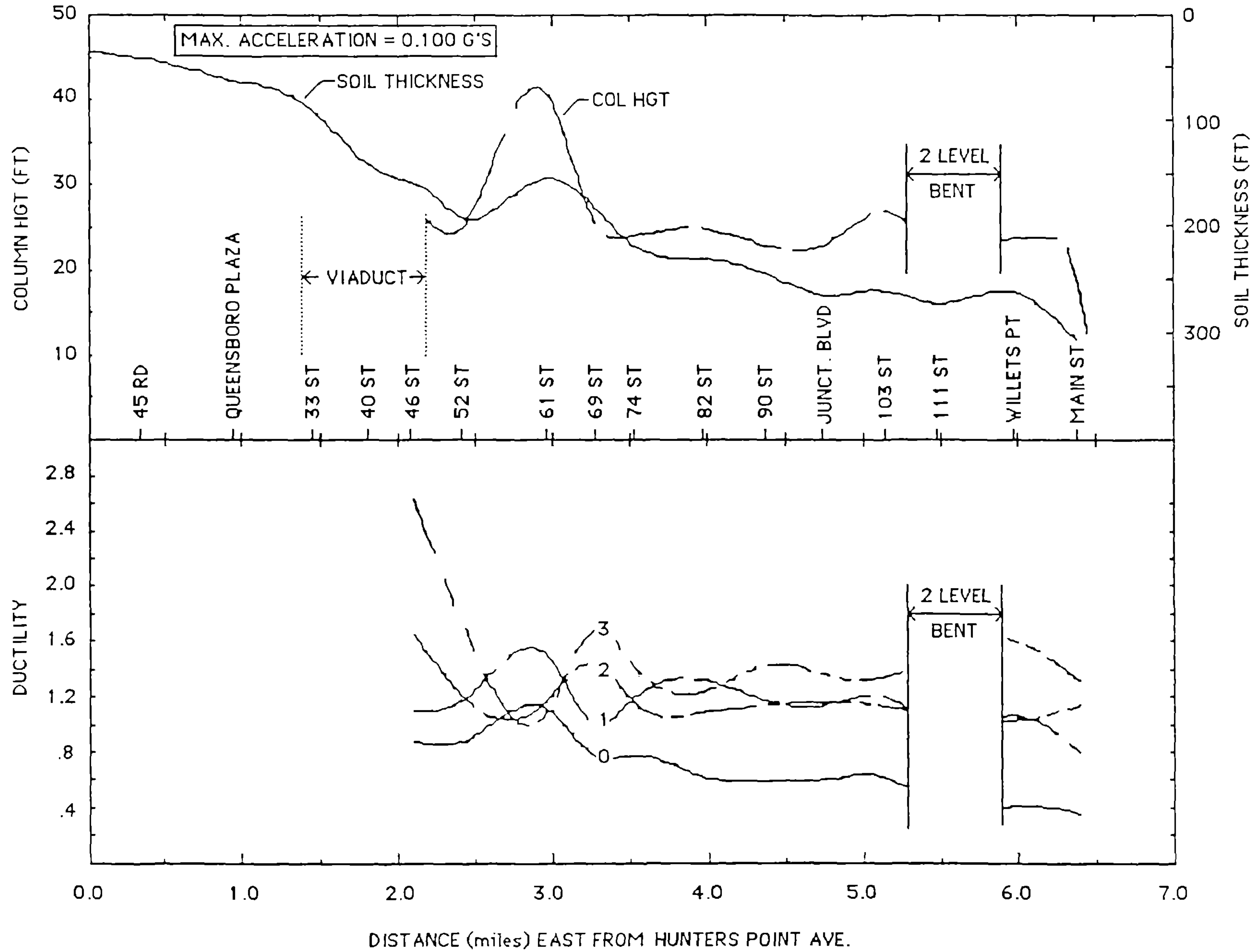


FIGURE 5-20 DUCTILITY ALONG THE FLUSHING LINE (SINGLE BENT) FOR 0.100G BEDROCK SEISMIC MOTION

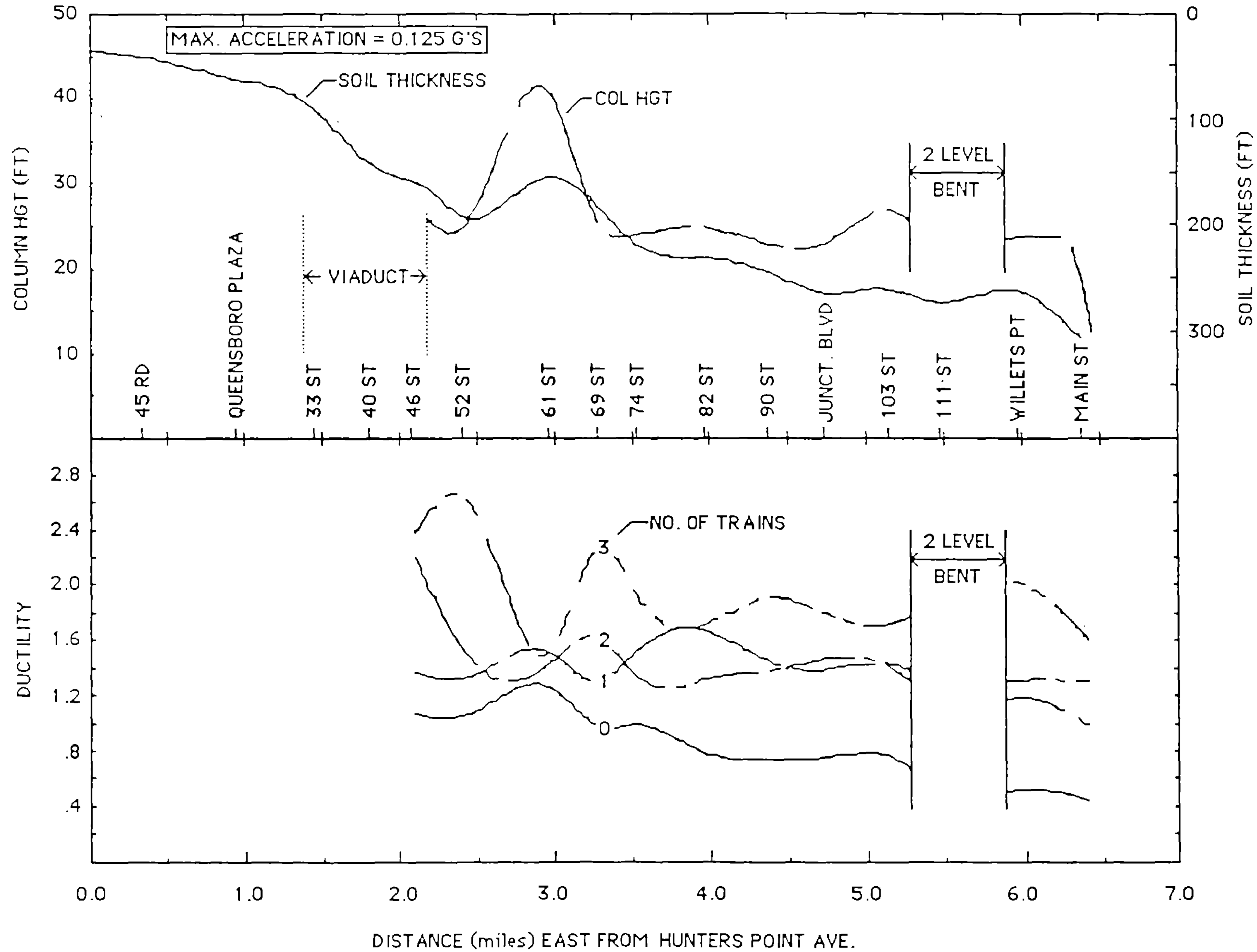


FIGURE 5-21 DUCTILITY ALONG THE FLUSHING LINE (SINGLE BENT) FOR 0.125 G BEDROCK SEISMIC MOTION

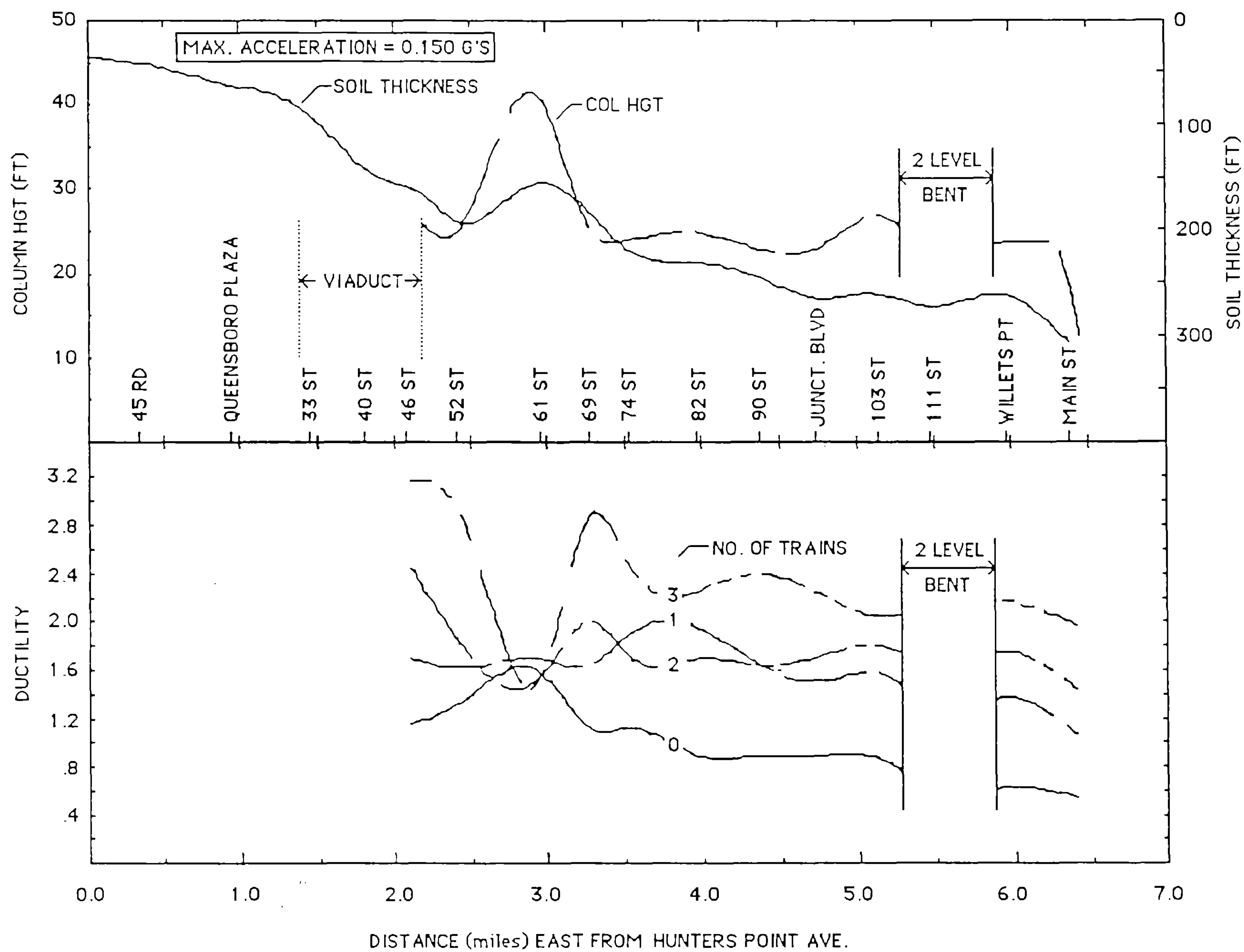


FIGURE 5-22 DUCTILITY ALONG THE FLUSHING LINE (SINGLE BENT) FOR 0.150 G BEDROCK SEISMIC MOTION

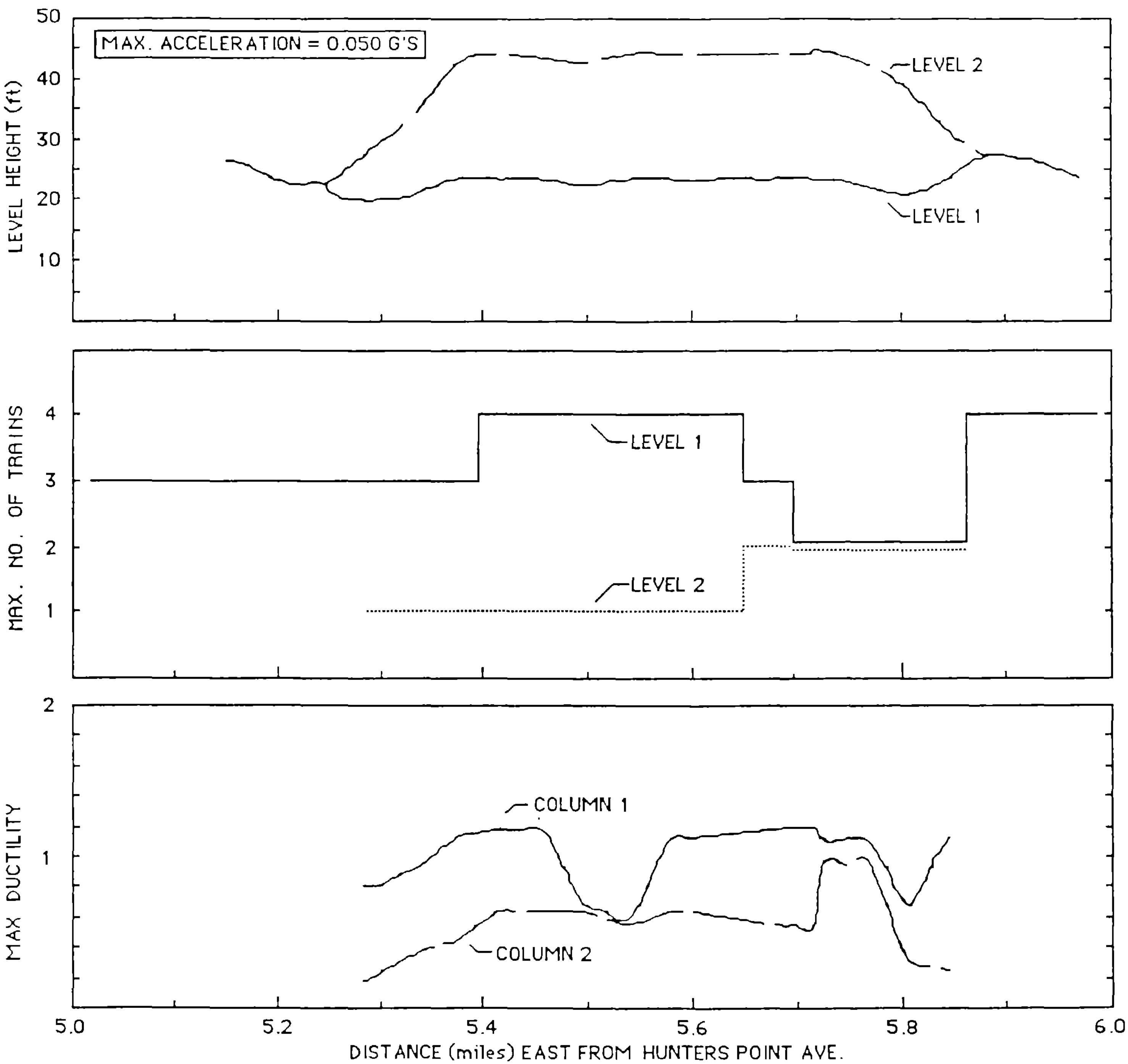


FIGURE 5-23 DUCTILITY ALONG THE FLUSHING LINE (DOUBLE BENT) FOR 0.050G BEDROCK SEISMIC MOTION

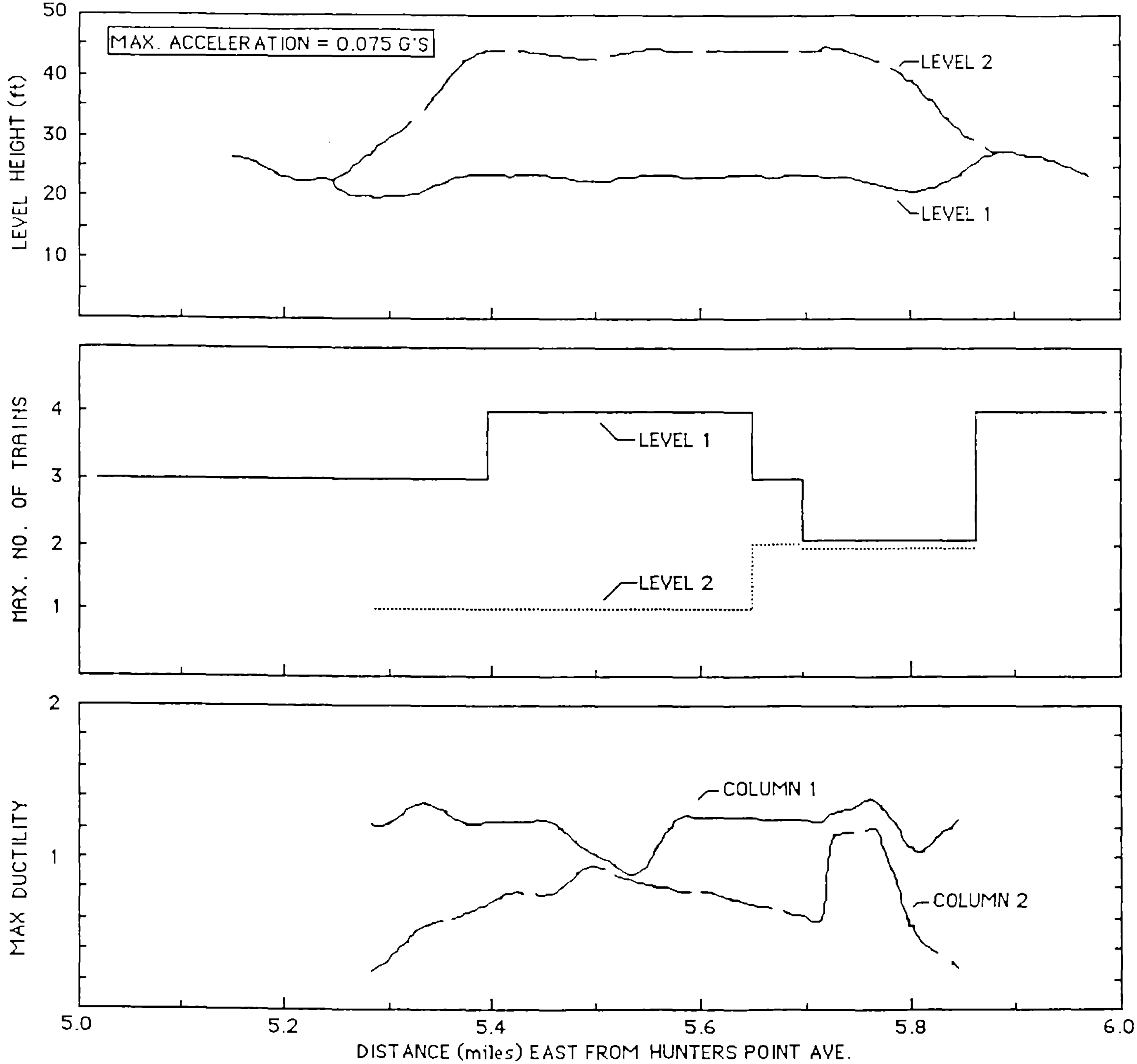


FIGURE 5-24 DUCTILITY ALONG THE FLUSHING LINE (DOUBLE BENT) FOR 0.075 G BEDROCK SEISMIC MOTION

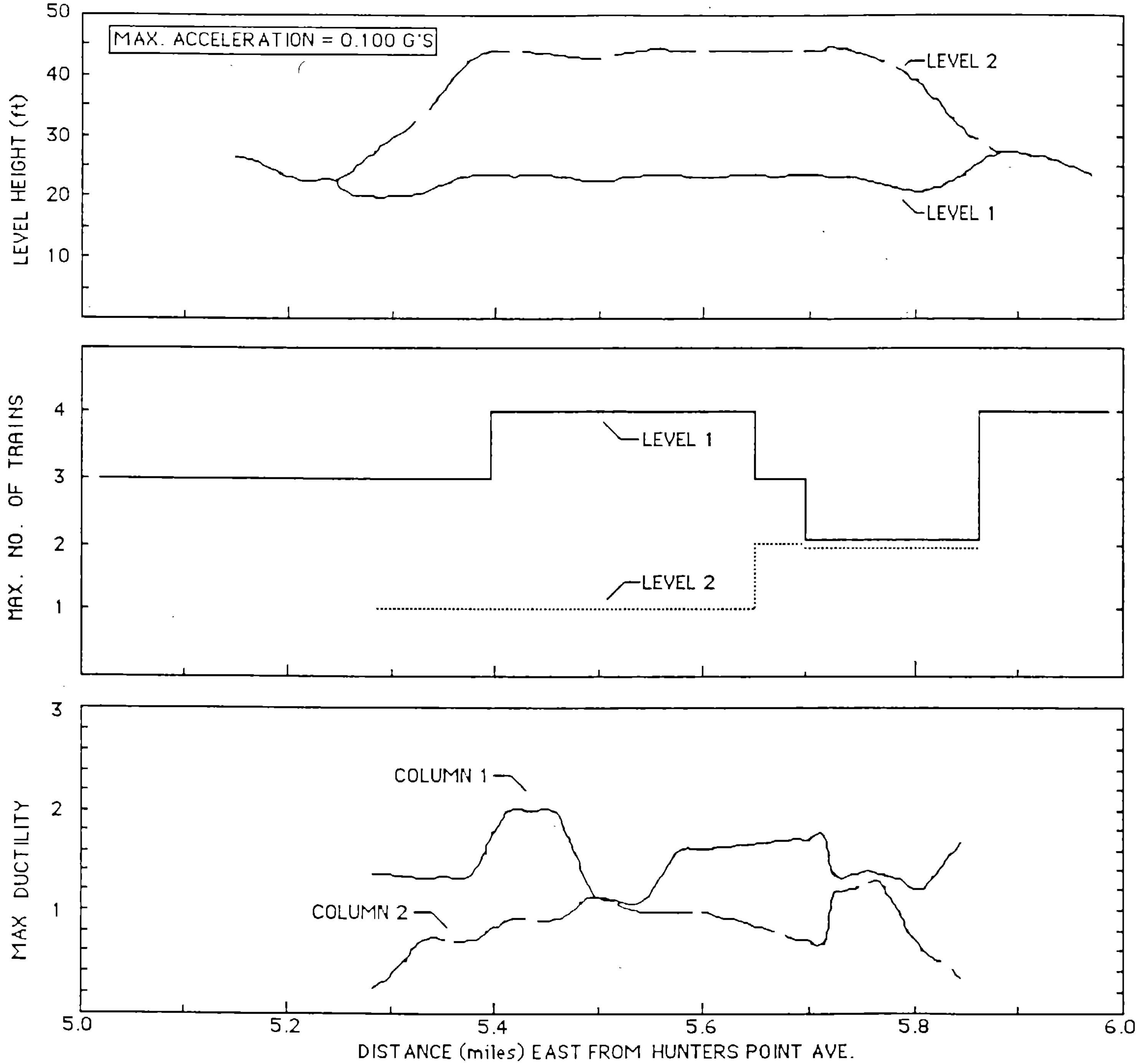


FIGURE 5-25 DUCTILITY ALONG THE FLUSHING LINE (DOUBLE BENT) FOR 0.100G BEDROCK SEISMIC MOTION

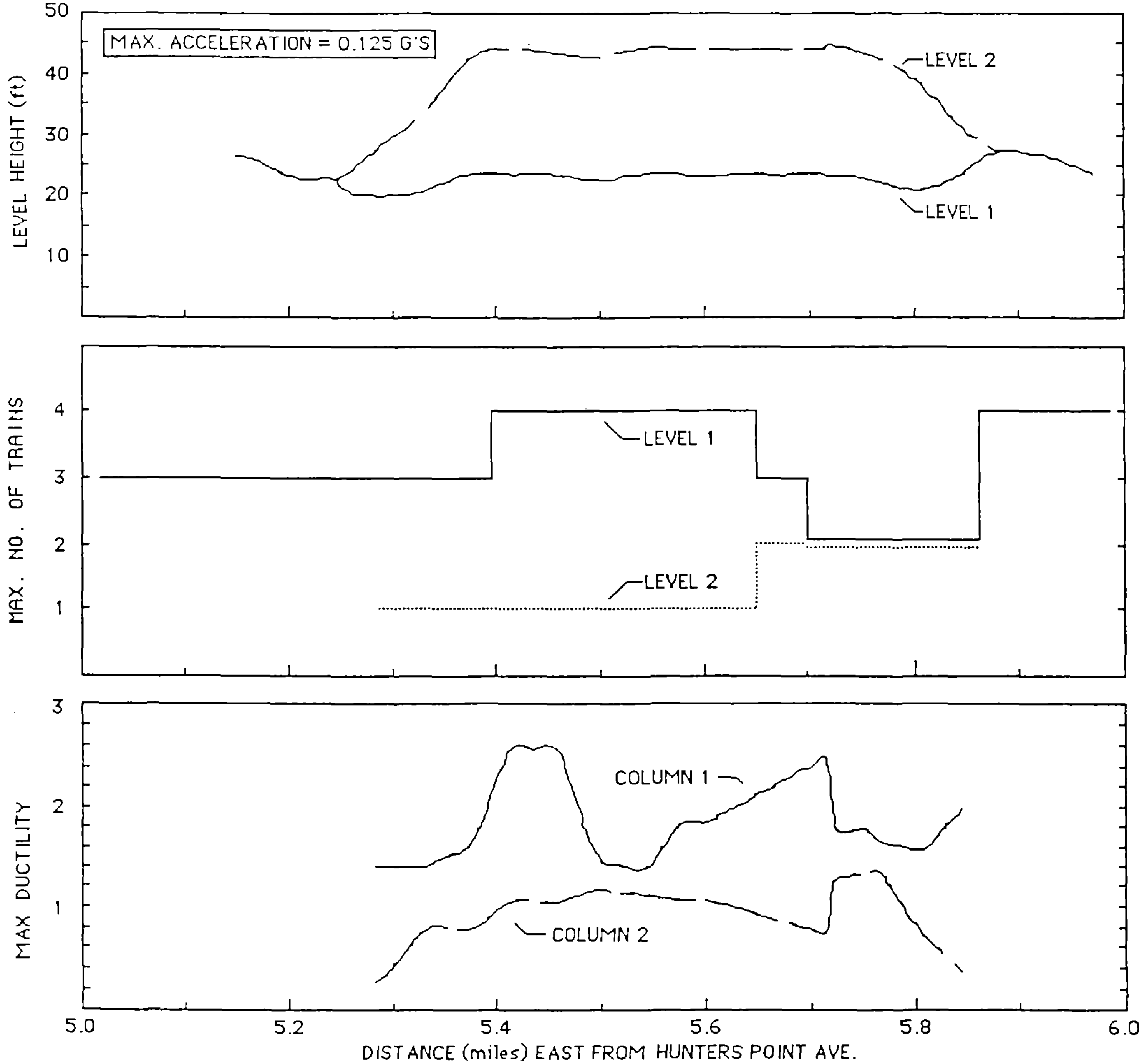


FIGURE 5-26 DUCTILITY ALONG THE FLUSHING LINE (DOUBLE BENT) FOR 0.125 G BEDROCK SEISMIC MOTION

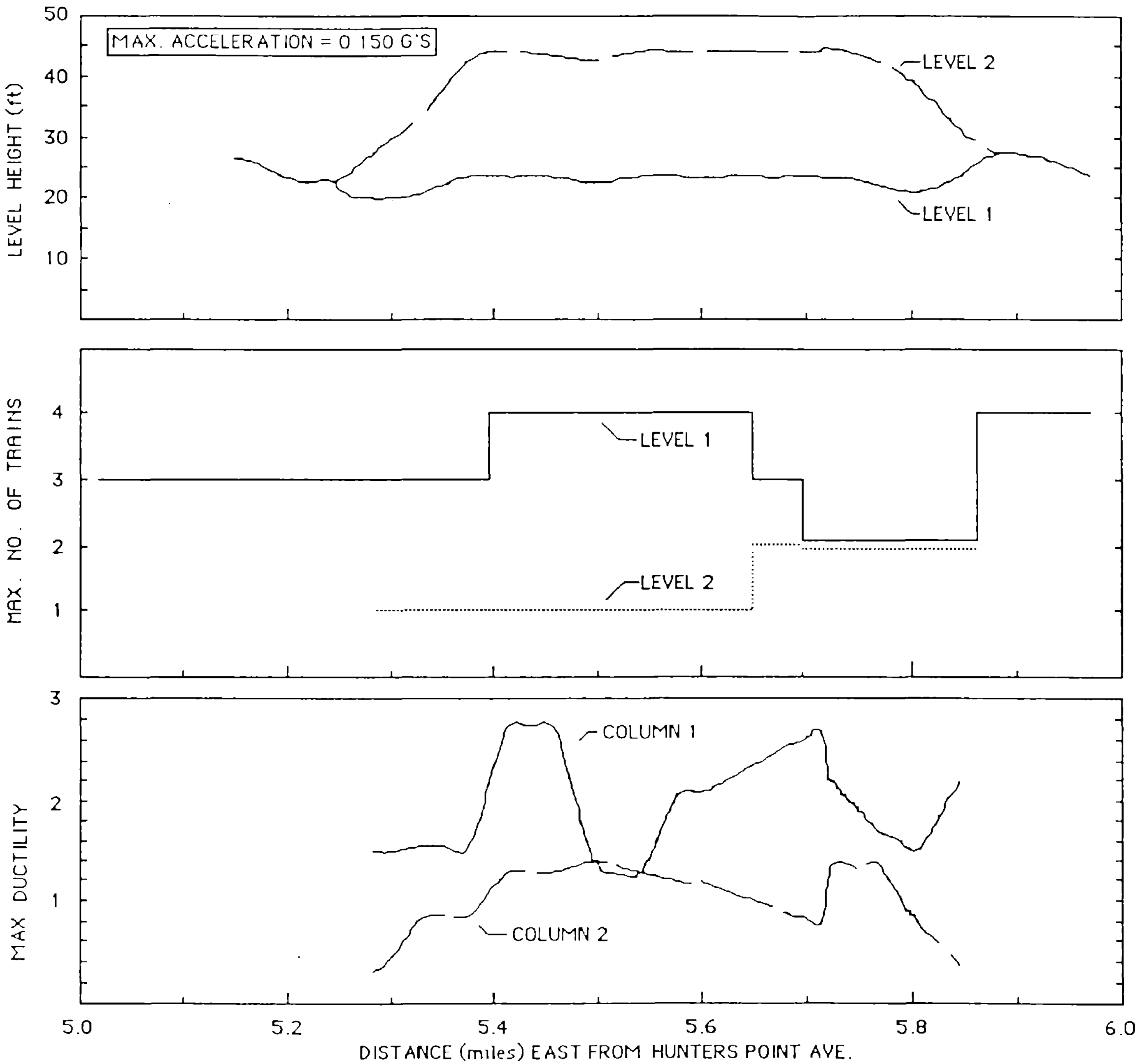


FIGURE 5-27 DUCTILITY ALONG THE FLUSHING LINE (DOUBLE BENT) FOR 0.150 G BEDROCK SEISMIC MOTION

column cross-sections are significantly larger than those of the single bent structure. Even though the total train mass leading to peak responses is significantly higher, this added capacity of the column keeps exceedance levels to about the same value as the single bent structure. However, since two track levels are involved, the peak displacements at the second level are significantly increased over the single bent case, probably increasing the probability of derailments for these input levels.

For the two level bent case, an estimate of the amount of column stress developed from some of the other load components (dead and live train loads) was calculated and is shown in Table V-7. As may be noted, these reach values of approximately 4 to 5 ksi. If the steel used in these structures has a yield capacity of 36 ksi, these load components use up about 14% of the column capacity available to resist the seismic load components. No impact factor was included in these calculations for the train loads, nor were wind effects considered. Such effects would normally not be included in such seismic evaluations. In any case, it can be expected that the inclusion of the dead and live load components would cause these peak ductility estimates to increase somewhat.

TABLE V-7

FLUSHING LINE - AXIAL STRESSES UNDER CRITICAL TRAIN LOAD COMBINATIONS
(NEGLECTING WIND, BRAKING, AND OTHER BENDING LOADS) FOR DOUBLE BENT SECTION
(FOR 0.150 G BEDROCK INPUT)

BENT NO.	DISTANCE (MILES)	LEVEL 1 COLUMNS					LEVEL 2 COLUMNS				
		Dead Load (kips)	No. Trains Levels 1 and 2	D/L Load (kips)	Area (in2)	Stress (ksi)	Dead Load (kips)	No. Trains Level 2	D/L Load (kips)	Area (in2)	Stress (ksi)
519	5.282	176.1	3	524.8	118.2	4.44	40.9	1	157.1	42.4	3.71
520	5.291	176.1	3	524.8	118.2	4.44	40.9	1	157.1	42.4	3.71
524	5.333	176.1	4	641.1	125.7	5.10	40.9	1	157.1	42.4	3.71
528	5.373	176.1	4	641.1	155.7	4.12	40.9	1	157.1	63.2	2.49
532	5.414	196.7	5	777.9	163.2	4.77	40.9	1	157.1	58.9	2.67
536	5.455	196.7	5	777.9	163.2	4.77	40.9	1	157.1	54.5	2.88
540	5.497	196.7	4	661.7	178.2	3.71	40.9	1	157.1	54.5	2.88
544	5.537	196.7	3	545.4	218.2	2.50	40.9	1	157.1	58.9	2.67
548	5.580	196.7	5	777.9	170.7	4.56	40.9	1	157.1	58.9	2.67
550	5.601	196.7	5	777.9	170.7	4.56	40.9	1	157.1	58.9	2.67
563	5.713	178.4	4	643.4	140.7	4.57	81.8	2	314.3	133.6	2.35
564	5.723	178.4	4	643.4	159.5	4.03	81.8	2	314.3	88.2	3.56
568	5.764	178.4	4	643.4	140.7	4.57	81.8	2	314.3	73.4	4.28
572	5.807	178.4	3	527.1	140.7	3.75	81.8	2	314.3	109.0	2.88
576	5.845	178.4	4	643.4	140.7	4.57	81.8	2	314.3	75.8	4.15

SECTION 6

CONCLUSION

This report has presented preliminary results attempting to assess the potential impact of seismic input motions on two particular elevated lines of the NYCTA rapid transit system. Two potential damage mechanisms were considered for these structural types, namely the potential for exceeding the yield stress at the column-girder joint of the bent and the potential for overturning of the pedestal footings typical in these designs. The results indicate that indeed the elevated structure is one which is susceptible to horizontal seismic input motions, and for which damage can be expected, even for the relatively low seismic input levels anticipated to be realistic for the New York City area. The amount of the damage developed is a function of the location of the particular bent along the line, since the soil overburden at the site significantly influences the motions delivered to the ground surface from the bedrock inputs, and modifies the amount of damage that can develop.

It should be noted that the calculations completed to date do not take into account a variety of effects, namely, the impact of the vertical motion component associated with the seismic environment input to the structure, as well as the effects of other load components acting simultaneously with the seismic load. Both considerations can be expected to make the effects of the seismic load more pronounced. However, much more detailed and difficult calculations would have to be performed to truly determine the effects of these additional components on anticipated damage levels to the bent structures.

It should be pointed out that the results of these calculations can be considered from a different point of view. In one sense, these results can be considered to be an evaluation of the "fragility" of the elevated lines studied. The primary damage results have been organized into a form of structural damage as a function of peak input acceleration level, a classic formulation of structural fragility. These results can then be combined with the results of the seismic hazard evaluation for the New York City area. The seismic hazard is typically presented in the form of the annual frequency of exceedance of a given peak bedrock acceleration level (Refs. 7 and 8). The results of the fragility analysis can then be convolved with the seismic hazard definition to yield the probability of exceeding a particular damage level, or a quantitative definition of the seismic risk for these structures. Such a probabilistic formulation can be used to arrive at a quantitative means for assessing the most susceptible component in the system and the component of most importance to the continued operation of the line following a seismic event. Such information, relatively routinely used currently in evaluating major structural components, is essential when planning retrofitting or upgrading programs system wide.

Finally, it should be noted that other important elements of the system may sustain damage to ancillary systems either housed within the structure or on the outside (water proofing, other systems, etc.) when subjected to such low level earthquakes. These failures may be important from an operational point of view, in that effective failure of the element has occurred even though serious structural damage has not occurred to the element itself. These effects have not been evaluated for this report.

SECTION 7
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APPENDIX A

STRUCTURAL RESPONSE ANALYSIS
FOR
SINGLE BENT STRUCTURE

The dynamic response of a single story bent can be computed from the following simplified analysis. The primary assumption made is that the cross girder connecting the two columns of the bent is essentially rigid as compared to the much more flexible columns. A schematic diagram of the bent is shown in Figure A-1, with the corresponding analytic model presented in Figure A-2. The mass at the top of the bent consists of the sum of the mass of the girder, the mass of one-half of each column, the mass of the connecting beams (and trackway) between bents, and the mass of any trains on the girder (which may vary anywhere from none to three). The mass of the train is obtained as the mass per foot of train times the distance between bents. The mass at the bottom of each column is obtained from one-half the column mass plus the mass of the footing.

In the analysis, interaction between the bent structure and the supporting soil is included by using "interaction springs and dampers" in both the horizontal and vertical directions, as is typical in seismic analyses. These springs/dampers account for flexibility of the soil immediately around the foundation as well as the ability of the soil to carry seismic energy away from the foundation. For most problems, this representation of soil-structure interaction effects is adequate. For the single story bent, the equation of motion in the vertical direction is given simply by

$$\left[m_t + 2m_f\right](\ddot{v}) + 2C_v(\dot{v} - \dot{v}_g) + 2K_v(v - v_g) = 0 \qquad \text{(A.1)}$$

where m_t and m_f are the masses of the bent top and footings, K_v and C_v are the soil-structure interaction coefficients shown in Figure A-2, v is the average vertical displacement of the bent, and v_g is the vertical motion of the ground surface due to the seismic input, assuming that the structure was not there; that is, v_g is the "free-field ground motion" defining the seismic motion at the ground surface that would occur were no structure present. Note that the columns are assumed to be rigid in the vertical direction in this simplified analysis.

In the horizontal direction, the relation between the column stress resultants (moment and shears) and the lateral displacements of the column tops and rotation of the rigid girder is defined by

$$\begin{Bmatrix} M_t \\ V_t \\ V_b \end{Bmatrix} = \frac{3EI}{H^3} \begin{bmatrix} H^2 & -H & H \\ -H & 1 & -1 \\ H & -1 & 1 \end{bmatrix} \begin{Bmatrix} \theta_t \\ u_t \\ u_b \end{Bmatrix} \qquad (A.2)$$

where M and V are the moment and shears top and bottom of the column, θ is the rotation of the girder and u is the lateral displacement of the columns, top and bottom.

Considering horizontal and rocking equilibrium of the columns and girder, the following two relations are obtained

$$m_t \; (\ddot{u}) + \frac{6EI}{H^3} \left[- H\theta_t + u_t - u_b \right] = 0 \qquad (A.3)$$

$$\left[J + \frac{m_f \; L^2}{2} \right] (\ddot{\theta}) + \left[\frac{K_v \; L^2}{2} \right] (\theta) + \frac{6EI}{H^2} \left[H\theta - u_t + u_b \right] = 0 \qquad (A.4)$$

Considering equilibrium of the foundation elements, the fourth equation required is derived as

$$[m_f](\ddot{u}) + [C_h](\dot{u} - \dot{u}_g) + [K_h](u - u_g) + \frac{3EI}{H^3}\left[H\theta - u_t + u_b\right] = 0 \qquad (A.5)$$

Equations A.1, A.3, A.4 and A.5 constitute a set of four equations governing the response of the bent, which can be written in classical matrix form as

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + [K]\{X\} = \{F\} - \{R\} \qquad (A.6)$$

where the matrices M, C and K are defined by the non-zero terms as

$M_{11} = m_t + 2m_f$, $M_{22} = m_t$, $M_{33} = 2m_f$,
$M_{44} = J + (m_f\ L^2)/2$

$C_{11} = 2C_v$, $C_{33} = C_h$, $C_{44} = (C_v\ L^2)/2$

$K_{11} = 2K_v$, $K_{22} = 6EI/H^3$, $K_{23} = -K_{22}$, $K_{24} = -K_{22}H$,

$K_{32} = 2K_{23}$, $K_{33} = K_h + 3EI/H^3$, $K_{34} = 3EI/H^2$

$K_{42} = K_{24}$, $K_{43} = 2K_{34}$, $K_{44} = (K_v L^2)/2 + 6EI/H$

and the applied force vector of equation A.6 is defined by

$$\{F\} = \begin{Bmatrix} 0 \\ 0 \\ C_h\dot{u}_g + K_h u_g \\ 0 \end{Bmatrix}$$

The vector {R} in equation A.6 represents the correction from linearity caused by yielding developed at the connection between the column and the girder and is defined by

$$\{R\} = \begin{Bmatrix} 0 \\ \dfrac{2M_c}{H} \\ -\dfrac{M_c}{H} \\ -2M_c \end{Bmatrix}$$

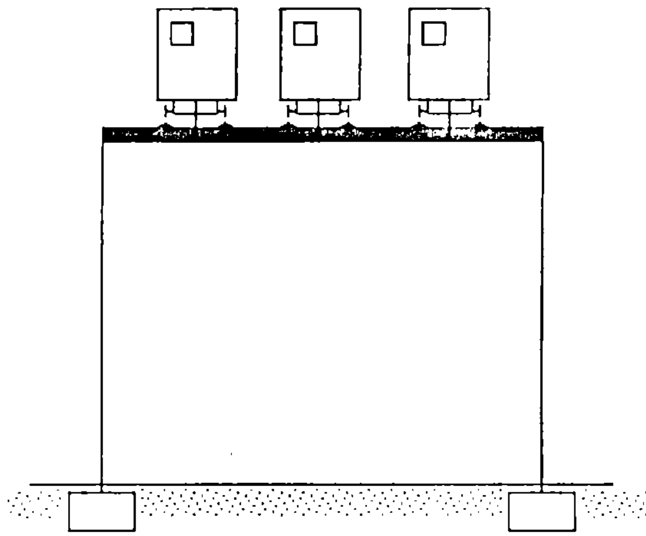
where M_c is the correction moment defined in Figure A-5 and H is the column height. The corresponding displacement vector is given by

$$\{X\} = \begin{Bmatrix} v \\ u_t \\ u_b \\ \theta \end{Bmatrix} = \begin{Bmatrix} \text{average vertical displacement} \\ \text{top horizontal displacement} \\ \text{bottom horizontal displacement} \\ \text{rotation of cross girder} \end{Bmatrix}$$

where the displacements u and v represent the total displacements of the bent.

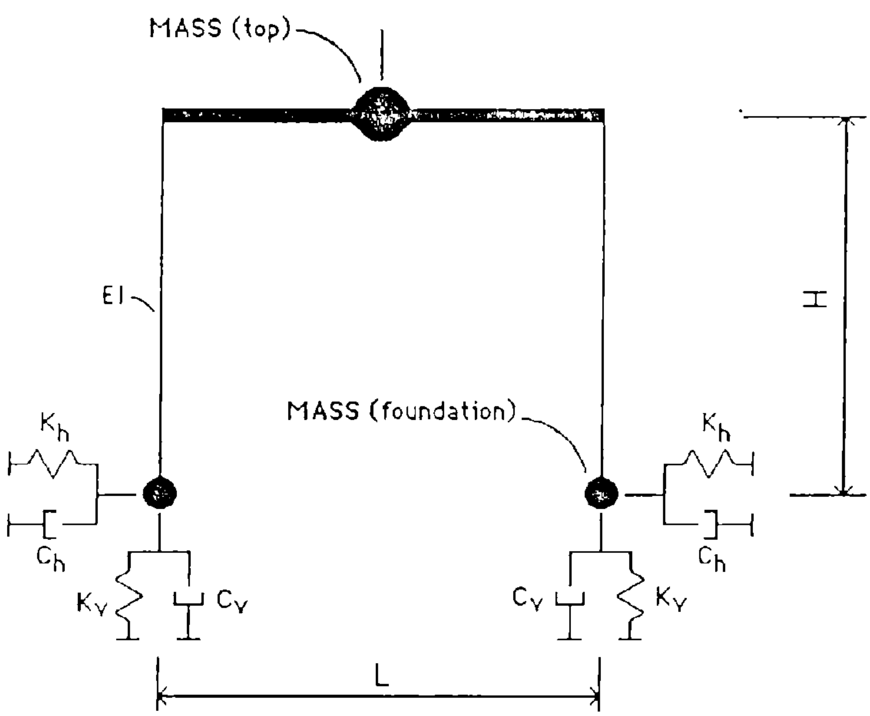
To determine the response of the bent structure when subjected to a seismic input loading, solutions to equation A.6 are obtained using the Wilson-Theta integration method (Ref. 9), in which accelerations are assumed to vary linearly during a small time interval of $\theta * \Delta t$. The parameter θ is taken to be 1.4 to assure the stability of the numerical integration. The equations of motion are numerically integrated to obtain the velocities and displacements at each time step in the calculation and these in turn are used to calculate column bending moments and curvatures.

The moment-curvature relationship for the vertical bent columns is assumed to be elastic-perfectly plastic as shown in Figure A-5. As illustrated, the ductilities of the columns are determined by comparing the calculated curvature at the end of each time step with the yield curvature. The correction moment used in the integration algorithm is computed as shown. For this simplified nonlinear model, the moment curvature relation is assumed to be independent of the direction of bending and its previous history; that is, Bauschinger stress effects are neglected.



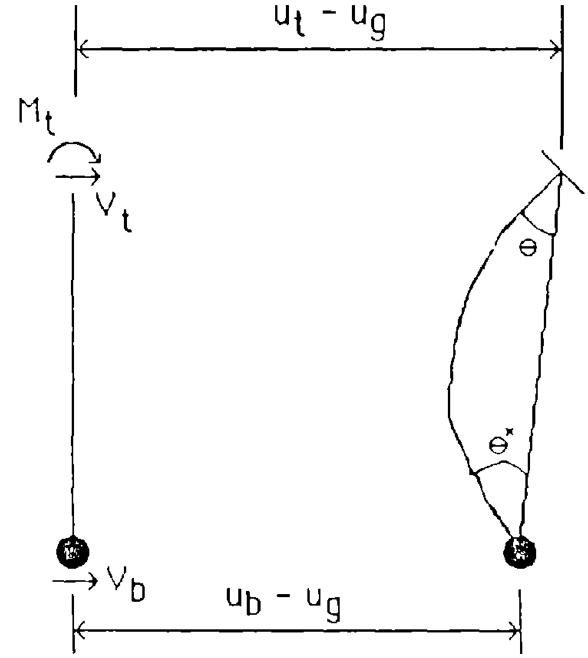
TYPICAL BENT CROSS-SECTION

FIGURE A-1

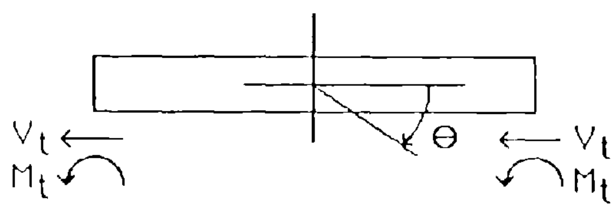


SCHEMATIC OF BENT MODEL

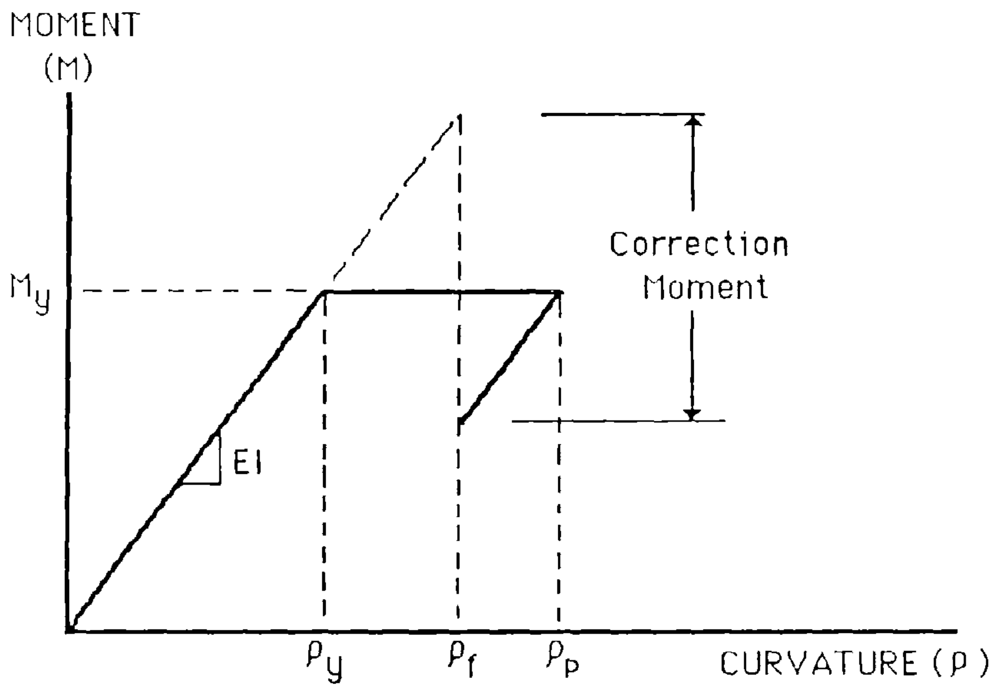
FIGURE A-2



BENDING DEFORMATION OF COLUMN
FIGURE A-3



FORCES ON GIRDER
FIGURE A-4



ρ_y = Curvature at yield

ρ_f = Curvature at end of time interval

ρ_p = Maximum previous curvature

$$\text{Ductility} = \frac{\rho_f}{\rho_y}$$

FIGURE A-5 MOMENT/CURVATURE RELATIONSHIP ASSUMED FOR COLUMN/GIRDER JOINT

APPENDIX B

STRUCTURAL RESPONSE ANALYSIS FOR DOUBLE BENT STRUCTURE

The dynamic response of the two story bent structure was determined from the following simplified analysis. The primary assumptions made are that the cross girders connecting the two columns of the bent are essentially rigid as compared to the much more flexible columns, and that the columns are rigid in the axial direction. The analysis of the single story bent (see Appendix A) indicated that soil structure interaction effects are not significant for these relatively light structures. As a result, these effects were omitted in the two story bent analysis, and the seismic motions are directly input to the base of the bent columns. Finally, the base of the columns are assumed to be pin connected; that is, the footings are assumed to offer no resistance to rotation.

A schematic diagram of the bent is shown in Figure B-1, with the corresponding analytic model presented in Figure B-2. The mass at the center of each girder of the bent (m_1 and m_2) consists of the sum of the mass of the girder, the mass of one-half of each column, the mass of the connecting beams (and trackway) between bents, and the mass of any trains on the girder (which may vary anywhere from none to three). The mass of the train is obtained as the mass per foot of train times the distance between bents. The moments of inertia (I_1 and I_2) of the columns in the analytic model are taken as the sum of the moments of inertia of each of the two columns shown in Figure B-1.

In the horizontal direction, the relation between the bottom column shear forces and the lateral displacements is defined by

$$V_T = - V_B = \frac{3EI_1}{L_1^3} (x_1) \qquad \text{(B.1)}$$

where V_T and V_B are the shears at the top and bottom of the lower columns. Since the girders are assumed to be rigid in flexure and the columns are assumed to be rigid in the axial direction, the rotation of the girder and column at the top end must be zero.

In the horizontal direction, the relation between the top column shears and the lateral displacements is defined by

$$V_T = - V_B = \frac{12EI_2}{L_2^3} (X_2 - X_1) \tag{B.2}$$

Considering horizontal equilibrium of the top and bottom girders, the following relations are obtained

$$m_2 (\ddot{x}_2) + \frac{12EI_2}{L_2^3} (X_2 - X_1) = - m_2 \ddot{Z} \tag{B.3}$$

$$m_1 (\ddot{x}_1) + \frac{12EI_2}{L_2^3} (X_1 - X_2) + \frac{3EI_1}{L_1^3} = - m_1 \ddot{Z} \tag{B.4}$$

Equations B.3 and B.4 constitute a set of two equations governing the response of the bent, which can be written in classical form as

$$[M] \{\ddot{X}\} + [K] \{X\} = \{F\} - \{R\} \tag{B.5}$$

where the matrices M and K are 2x2 matrices with nonzero terms given by

$$M_{11} = m_2, \qquad M_{22} = m_1$$

$$K_{11} = 12EI_2/L_2^3, \qquad K_{12} = -\ 12EI_2/L_2^3, \qquad K_{21} = K_{12}$$

$$K_{22} = 12EI_2/L_2^3 + 6EI_1/L_1^3$$

and the applied force vector of equation B.5 is defined by

$$\{F\} = -\ \ddot{Z} \begin{Bmatrix} m_2 \\ m_1 \end{Bmatrix}$$

As described in Appendix A, the vector {R} in equation B.5 again represents the correction from linearity caused by any yielding developed at the connection between the columns and the girders. Solutions to equation B.5 are once again obtained by using the Wilson-Theta method.

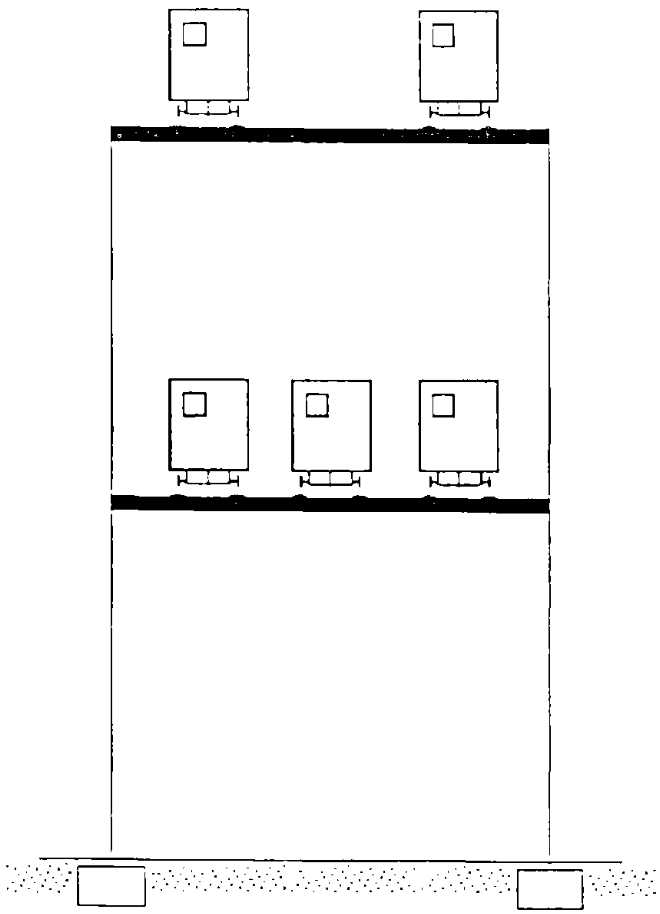


FIGURE B-1
TYPICAL BENT CROSS-SECTION

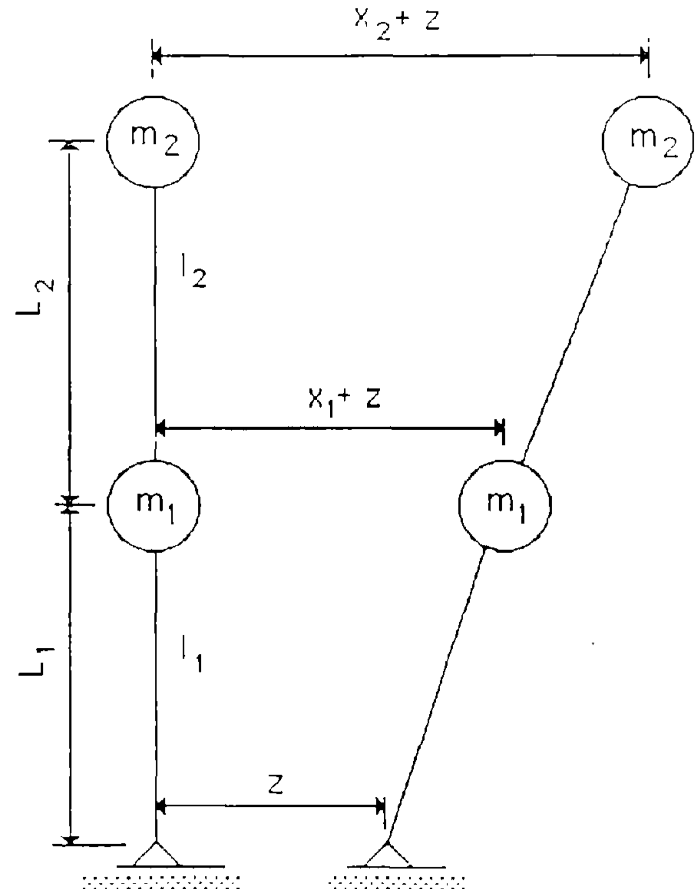


FIGURE B-2
DISPLACEMENT PROFILE

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