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FLEXIBLE SLEEVED-PILE FOUNDATIONS FOR ASEISMIC DESIGN

(Final Report)

by
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March 1982

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ABSTRACT

Reported herein is an investigation of the feasibility of constructing buildings on horizontally flexible foundations to mitigate the effects of earthquakes. The particular concept studied involves the use of slender steel piles enclosed in sleeves to permit flexural distortion.

Piles are designed by a simple procedure using smoothed response spectra. The performance of building-foundation systems so designed are then studied using time histories of actual ground motions. It is shown that the simple design procedure is adequate and that the concept achieves the desired result of greatly reducing seismic forces.

By this means, the maximum seismic forces on the building may be reduced to a level which is so low that the forces probably do not affect the design of the superstructure. The forces are less than those required by current codes even though the building structure and the piles would remain elastic during the maximum design earthquake. In many cases the lateral seismic forces would be less than the design wind forces.

The use of energy-absorbing devices to further decrease earthquake effects is also studied. It is shown that such devices are feasible and that their use can reduce the cost of the foundation system.

The economic feasibility of the concept is studied briefly. It is believed that the additional foundation cost can be justified on the basis of savings in initial superstructure cost and in probable future damage costs. However, additional and more detailed economic studies are required to fully evaluate the concept.

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The opinions, findings and conclusions or recommendations expressed in this report are those of the author and do not necessarily reflect the views of the National Science Foundation.

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CHAPTER I - INTRODUCTION

The purpose of the research project for which this is the final report has been to investigate the effectiveness and feasibility of constructing buildings on horizontally flexible foundations to mitigate the effects of earthquakes. This concept permits the design of the building for very modest lateral forces and yet limits the earthquake response to essentially elastic behavior. Although other types of isolating systems might be used, these studies are limited to the sleeved-pile system in which flexibility is achieved by the bending of slender steel piles within a sleeve (or caisson) below grade.

In general, the approach has been to design pile systems for various values of design and site-related parameters using a simple dynamic model, and then to investigate the behavior by subjecting the building-foundation systems to real ground motions. The results clearly indicate that the proposed concept reliably achieves the desired result.

This report contains a summary of previously reported studies and the results of certain additional investigations of behavior and the effect of parametric variations. Practical design considerations are also discussed, but the assessment of economic benefits must be studied on a case-by-case basis and only general comments in that regard are made herein.

1.1 Background

The aseismic design of conventional buildings under current codes is based on relatively small equivalent static loads and anticipates considerable yielding and hence serious damage in a severe earthquake. Most of the structural systems being utilized, even in areas of high seismicity, were developed in the past without regard to seismic requirements. There is an obvious need to develop new systems which will more effectively and economically provide earthquake protection.

The general concept of isolating a building from earthquake effects by lengthening the natural period is well understood. Because of the relatively high frequency content of earthquake ground motion, this has the effect of reducing the inertia forces and hence the required strength of the structure. There is, however, a corresponding increase in horizontal displacements which must be accommodated.

Among the systems which have been investigated is the "soft story" concept in which yielding (or flexibility) is concentrated in the bottom story, thus reducing the seismic forces on the upper stories. Other concepts include those in which the building is placed on sliding bearings or laminated rubber bearings which are very flexible in shear. All of these systems have substantial disadvantages and have rarely been attempted in practice, except for a few soft-story buildings.

In the concept investigated here, the desired flexibility is introduced below grade as part of the foundation. Because the seismic response is restricted to elastic behavior of the steel piles, isolation can be provided with a high degree of reliability. The concept of course requires an increase in the cost of the foundation which must be balanced by a decrease in the cost of the superstructure and, more importantly, a decrease in the probable cost of future damage.

1.2 The Sleeved-Pile Concept

Basically, the system studied herein involves the insertion of a soft spring between the building superstructure and the soil foundation, thus producing a very long period of the total system (10 or more seconds). Devices other than sleeved piles might be used for the spring. However, it was believed desirable to concentrate the study on this one practicable means of implementation.

The sleeved pile is shown schematically in Fig. 1.1. It consists of a cylindrical steel pipe within a thin steel sleeve of sufficient diameter to accommodate the horizontal displacements of the pile top during the design earthquake. The desired flexibility is provided by bending of the pipe over its free length. The top can be either fixed or pinned to the bottom of the building structure, but the latter condition is probably more desirable. The pipe piles are designed to remain elastic under the maximum earthquake predicted for the site. Because the pipes are quite slender, the lateral stiffness is appreciably affected by the axial load, and the ratio to the Euler load may be as high as 0.75. However, collapse due to instability, which could possibly occur if displacements exceeded the design value, is prevented by the retained soil around the building basement, or by pile-to-sleeve contact. The subgrade structure shown in Fig. 1.1 is not necessary, and the pile top could be at grade.

The sleeve permits transfer of horizontal forces from pile to soil well below the ground surface where resistance to such forces is great. The pipe pile is supported by a concrete infill. Some reinforcement at the top of the concrete might be required. The vertical pile load is carried by bearing at the bottom of the sleeve, if a bearing layer exists, or by the sleeve acting as a friction pile.

For a building supported on sleeved piles we may visualize the piles as providing a horizontal shear spring of predictable, elastic, stiffness. Because this force-displacement relationship is not associated with friction, or with large distortions of a nonlinear material, but rather with flexural distortion of the piles, it is not only predictable, but can be age-independent. The fundamental period of the total system may be made very long, perhaps as long as 20 seconds, although this may not be the optimum solution.

In summary, it is believed that the sleeved-pile concept has several advantages over other isolating concepts. These include:

1. A greater length is available to achieve the desired flexibility, and any desired natural period could be achieved.

2. Because only elastic behavior is involved, the properties of the system can be accurately predicted.

3. Although seismic stresses in the pile would be high, safety against collapse (due to larger than design earthquake motions and/or the P-Δ effect) is ensured because the ground provides a limit to horizontal displacement. Under gravity loads alone, the pile axial stress is small and, with the pile restrained at its top by a wind-resisting device, the factor of safety is large.

4. It should be easier to accommodate the necessarily large deflections at ground level than in the bottom story of the superstructure, as in the soft-story concept.
5. It would be possible to build damping devices into the system at the piles and thus achieve an optimum value. Much higher damping than normally found in structures would be beneficial and possibly economically feasible.

1.3 Summary of Previous Studies

Previous investigations of the sleeved-pile concept under this project were reported in References 1 through 3. The last utilizes the results of the first two to develop a simple yet adequate design procedure for establishing the required length and section properties of the pipe pile. It also reported time-history analyses of the response in detail which validated the design procedure. These results are summarized below.

1.3.1 Ground Motions. The instruments and techniques used in the past for recording and processing earthquake ground motions are inadequate for long-period systems; i.e., responses calculated from the resulting accelerograms are unreliable. To circumvent that difficulty, a "new standard processing scheme" was developed for the purposes of this project. This is developed in Ref. 2, and the procedure is summarized in Ref. 3.

A search was made for strong-motion records which, after being processed by the new scheme, would be reliable for systems having periods up to 12 seconds. Four such pairs of records (simultaneous vertical and horizontal motions) were located and adapted for the time-history analyses of this study.

These same records were used to establish the smoothed design spectra shown in Figs. 1.2 and 1.3 for 0.5 and 10% damping. The ordinates, when multiplied by the design peak ground displacement, give the peak response displacement. These spectra were used for the design of the pile foundation systems. The higher damping value implies the use of special damping devices (see Sect. 2.5). The spectra shown are based on limited data and are not being proposed for general use. However, they are adequate for the purposes of this study.

1.3.2 Sleeved Pile Design. The pile design procedure developed in Ref. 3 is based on the following assumptions: (1) The building superstructure is assumed rigid, and its rotational inertia is ignored; (2) the effect of vertical ground motion is ignored; and (3) the flexibility of the soil surrounding the sleeve is ignored; i.e., the pipe pile is assumed fixed at the top of the concrete infill. Thus the problem is reduced to the response of a one-degree system.

The pile design procedure may be summarized by the following equations:

Given: P = vertical load per pile = $M_t g$

T_p = desired fundamental period

β_p = pile damping ratio

f_y = yield point of pipe steel

t = selected thickness of pipe steel

y_{max} = peak response displacement for T_p and the peak ground displacement (Figs. 1.2 or 1.3).

For a cylindrical pile (constant diameter and thickness) pinned at the top:

$$\ell = \frac{4g}{\omega_p^2 \pi^2 (\frac{P}{P_E})} \frac{a^3}{\tan a - a} \tag{1.1}$$

where ℓ = required free pile length

ω_p = desired design frequency

P_E = Euler buckling load

$a = \pi\sqrt{P/P_E}$

Also,

$$I = \frac{4P\ell^2}{\pi^2 E(\frac{P}{P_E})} \tag{1.2}$$

$$f_{max} = f_y = \frac{P}{8I} D^2 + \frac{P}{2I} (D+t) \left\{ y_{max} - \frac{S_{max}}{M_t} \frac{\ell}{g} \right\} \tag{1.3}$$

$= $ maximum pile stress

where D = average pipe diameter

$S_{max} = \omega_p^2\, y_{max}\, M_t$ = maximum pile shear

and
$$t = \frac{8I}{\pi D^3} \tag{1.4}$$

An iterative procedure is used as follows: (1) Assume a value of P/P_E; (2) compute ℓ by Eq. (1.1); I by Eq. (1.2); D by Eq. (1.3); and t by Eq. (1.4). The process is cycled until Eq. (1.4) yields the desired wall thickness. This produces a pipe design in terms of a unique combination of ℓ, D and t. Note that the maximum pile stress is made equal to the yield stress. No factor of safety is required because design would be based on the maximum credible earthquake.

Design charts were constructed for the parametric combinations of $\beta_{\hat{p}}$, f_y, P and t shown in Table 1.1. These are shown in Figs. 1.4 through 1.7, where required free pile length and diameter are plotted against design period. These are based upon an assumed peak ground displacement of one foot (30.48 cms). The following observations can be made.

There is an advantage in using the longest design period possible. The limit of 12 seconds used in these plots resulted only from the belief that the earthquake records and design spectra were not reliable above this point.

When the vertical load per pile increases, the required length and diameter also increase, but not nearly in proportion to the load. This suggests that a large pile spacing, and hence a smaller number of piles, is desirable.

The use of higher strength steel appreciably decreases pile length and diameter. Although the 100 ksi value is attainable, it may be beyond the current fabricating capabilities in the U.S.

An increase in damping to a value such as 10%, which requires the addition of special damping devices (see Sect. 2.5), appreciably reduces the required pile length and diameter.

1.3.3 Time-History Analyses. In order to verify the simple design procedure, and to further investigate the response of the building-foundation system, time-history analyses were executed using the processed ground motions referred to in Sect. 1.3.1. Whereas the pile design procedure was based on a one-degree model, the model used in these analyses included the flexibility of the superstructure, represented by its fundamental mode, and the flexibility of the soil surrounding the sleeve. Thus two uncoupled three-degree systems were created, simulating both horizontal and vertical motions, as shown in Fig. 1.8. Although the models were uncoupled, the effect of the response to vertical motion on pile axial load, and hence horizontal stiffness, was included in the calculation of horizontal response. The analysis technique is fully described in Ref. 3.

Time-history analyses were made for buildings with piles designed as described in Sect. 1.3.2. Typical results are shown in Tables 1.2 and 1.3. For this analysis, the building superstructure (if on a rigid foundation) was assumed to have horizontal and vertical fundamental periods of 1.0 and 0.2 secs. respectively, and a damping ratio of 0.02. In order to eliminate the earthquake strength as a variable, each of the records was scaled so as to have the same response spectrum ordinate at the design period as the smoothed design spectrum.

The results are in close agreement with the responses predicted by the simple design model as indicated in Table 1.4. It is therefore concluded that the design procedure, based on the smoothed design response spectra and the one-degree model, produces reasonable designs.

As expected, the building base shear coefficients are quite small. They vary from approximately 0.02g to 0.05g (for different design periods), whereas the same building on a rigid foundation (T = 1 sec.) would have an elastic coefficient of approximately 1.2g. If the same building with a rigid foundation were designed by Code (UBC - Zone 4), the base shear coefficient might be approximately 0.09. Furthermore, the seismic base shears for a typical building are approximately the same as the design wind shear. In many cases the wind loading would control the design.

1.3.4 Parametric Studies. Additional time-history analyses were made to further investigate the behavior of the building-pile system.

It was found that, even with extremely soft soil, the additional flexibility around the sleeve had very little effect and could be ignored.

The effect of vertical ground motion on the horizontal response did not prove to be significant. In general, it caused a modest increase in building acceleration and base shear, but had little effect on peak pile stress or displacement.

Increasing the building period from one to three seconds had little effect on the design forces although, of course, the internal building distortion increased.

The design concept being studied requires a wind-resisting device at the foundation to prevent excessive building motion due to wind. This would be designed for brittle failure under a modest seismic force. Time-history analyses were run with an initial rigid foundation support having a breaking strength equal to the maximum design wind load. It was found that this initial restraint had little effect on the performance of the system during the design earthquake.

Time-history analyses were also performed for systems with 10% pile damping provided by devices at the pile top. Again, these indicated that the simple pile design procedure was adequate.

1.3.5 Conclusions. The previous studies summarized above led to the following tentative conclusions: (1) The sleeved-pile concept is an effective means of isolating a building and is theoretically feasible; (2) the simple pile design procedure developed is reliable and adequate for design purposes; and (3) for maximum efficiency, a long design period, the use of high-strength steel, and the introduction of damping devices are desirable, although there are practical limits to these parameters. Additional studies were considered necessary to investigate certain effects on behavior which had been ignored and to obtain a better indication of economic feasibility. These are reported in the following chapters.

CHAPTER II - FURTHER INVESTIGATIONS

Following the studies summarized in Chapter 1, additional investigations were undertaken to determine the effects of further refinements in those analyses. In addition, the feasibility of increasing the damping in the system by the use of energy-absorbing devices was investigated. These results are presented below.

2.1 Effect of Higher Building Modes

In the previous time-history analyses, only the fundamental mode of the building superstructure was modeled (Sect. 1.3.3). In order to investigate possible effects of higher modes on system response, the model was expanded to include five degrees of freedom in the superstructure as shown in Fig. 2.1. It was modeled as a shear building with a linear first-mode shape. The parameters K_f and C_f represent the stiffness and damping of the piles including the effect of the soil surrounding the sleeve. Again, the horizontal and vertical responses were computed simultaneously and the effect of the latter on the former was included. A complete desciption of the model and the method of analysis is given in Ref. 4.

Typical results are shown in Table 2.1, where computed peak responses considering only the first mode are compared with those considering all five modes of the superstructure. For these calculations the horizontal fundamental period of the building was taken to be 1.0 secs. and the damping ratio as 0.02 in all modes. The small differences in the one-mode results and those given in Table 1.3 are due to slight differences in modeling.

It may be observed that the only significant effect of the higher modes is in the peak acceleration at the top of the building. The building base shear and pile stresses are essentially the same. This effect on the top acceleration is also commonly found in buildings on rigid foundations, and suggests that the same kind of allowance currently made in codes (i.e., a concentrated inertia force at the top) might also be appropriate here. It is believed that a simple analysis procedure for the building frame, based on equivalent static loads dependent only on the period of the one-degree, pile-building model, could be developed.

The higher building modes do not significantly affect the pile response nor the validity of the pile design procedure previously outlined.

2.2 Pile Group Effect

In the earlier analyses, the flexibility of the soil surrounding the sleeve was determined on the basis of a single isolated pile. This flexibility was found not to be significant in that case. However, if the behavior of all piles under the building were considered, the flexibility and hence the effect would be increased. This question was investigated and the results reported in Ref. 4.

The interaction between piles in a group depends upon the size and spacing of piles as well as the soil properties. A thorough investigation of the dynamic lateral stiffness of a pile group connected by a rigid cap was reported in Ref. 5. It is concluded therein that the earlier work by Poulos and Davis (Ref. 6) for groups under static loads also gave reasonable results for dynamic stiffness. The results given in Ref.6 were therefore used for the investigation reported here.

Typical results are shown in Table 2.2, where response based on the stiffness of an isolated pile are compared with those including the group effect. For these calculations, the group was taken to be a rectangular array of 64 piles at 15-foot spacing, supporting a ten-story building with a period of 1 second. A soft soil with a shear wave velocity of 120 m/sec was assumed. The model shown in Fig. 2.1 was used with decreases in the pile stiffness and damping (K_f and C_f) to reflect the pile group effect.

In this case, as well as in others considered, the only significant effect is in the acceleration at the top of the building. Therefore, consideration of the pile group effect did not change any of the conclusions previously drawn.

2.3 Effect of Overturning

In all previous analyses the effect of the overturning moment due to seismic forces was ignored. The primary effect of overturning is an increase in the exterior pile axial stress which, of course, depends upon the dimensions of the building. For illustrative purposes, a 10-story building (120 ft in height) with plan dimensions of 120 x 120 ft and an 8 x 8 array of piles was investigated. Using the five-mass model of the building, the seismic overturning moment was computed at each step in the time-history analysis, and the additional stress in the exterior piles determined (assuming a linear variation in pile forces). It was assumed that the horizontal translational response of the building was not affected, i.e., that the horizontal stiffness of the entire pile group was not changed by the overturning moment.

The increase in the computed maximum pile stress is shown in Table 2.3. This depends upon the phasing of the pile displacement and the overturning moment, which do not necessarily reach a maximum simultaneously. Therefore, the increase varies between earthquake records. The average stress increase in Table 2.3 is 3.9%, with a maximum of 7.6%. While some allowance for this effect might be made in the pile design, it does not seem to be an important consideration for a building of these dimensions.

2.4 Effect of Site Condition

The pile designs generated previously (Sect. 1.3.2) were based on smoothed spectra for soil sites, i.e., locations where bedrock is at a considerable depth. Presumably, in these cases the sleeved piles would be either friction piles or would bear on a firm soil layer, but not rock. In other cases, the sleeved piles might bear on rock if it was near the surface, and hence the ground motions to be used for design might be somewhat different. In particular, one would expect the responses in the long-period range of interest here to be less.

To investigate this difference, new design spectra for rock sites were generated as shown in Figs. 2.2 and 2.3. These are based upon a rough averaging of four accelerograms recorded on rock, processed as described in Sect. 1.3.2. Note that there is a greater difference between the two spectra with small damping, but there is not a large difference in either case for the period range of interest. All of these spectra are based on limited data and are not being proposed for general use.

Required pile length and diameter for the postulated rock site are plotted in Figs. 2.4 and 2.5. These results may be compared with Figs. 1.4 and 1.5, which are for a soft soil site. Due to the smaller design displacements, both the required length and the diameter are less for the rock site. However, the differences are not substantial, there being little difference in diameters and only about 20% in lengths. It has been assumed in this comparison that the peak ground displacement is the same at both sites.

2.5 Use of Energy-Absorbing Devices

It has been demonstrated that an artificial increase in pile damping appreciably decreases the required length and diameter (Sect. 1.3.2). It also decreases the pile displacement and the building base shear. One way to provide this damping increase is to insert energy-absorbing devices between the building base at pile-top level and the retained soil around the base of the superstructure (see Fig. 1.1).

Several such devices have been proposed and investigated by others. Most of these utilize the plastic deformation of ductile metals (usually steel) for the absorption of energy. One possible device is shown schematically in Fig. 2.6. In this case energy is absorbed by torsional deformation of a circular rod. The rod is held between, and twisted by, two arms which are pin-connected to the building and retaining wall. Opposite ends of the two arms are connected to the same base, such that relative displacement between the building and ground results in a scissors-like motion which twists the rod. The device must be detailed so as to freely accommodate motion in the perpendicular direction. Two sets of devices would independently provide damping in two orthogonal directions.

For this investigation, the resistance of the device is idealized as an elastic, purely plastic function as shown in Fig. 2.7, where:

F = force between building and ground

F_p = maximum force provided by the plastic torsional strength of the rod

y = relative motion at pile top

y_m = maximum relative motion

y_e = elastic limit displacement

y_p = pseudo-elastic limit at full plastification.

It is assumed that rebound in either direction is at the initial elastic slope and that there is no strength degradation. Actual devices might have more complex functions, but this idealization is sufficient for the present purpose.

The investigation reported here is an attempt to establish a relationship between the hysteretic damping provided by the device and equivalent viscous damping. One way of doing this is to equate the energies absorbed in one complete cycle of motion. Assuming a complete cycle with amplitude equal to the maximum displacement during the earthquake (y_m), the energy absorbed by viscous damping is

$$W_v = 2\pi\beta_p M_t \omega^2 y_m \qquad (2.1)$$

where β_p is the pile damping ratio, M_t is the total mass supported by the pile, and ω is the fundamental, or design, frequency. The energy absorbed by one complete hysteretic loop is

$$W_h = 4F_p \left[y_m - \frac{F_p}{K_e}\right] \qquad (2.2)$$

where K_e is the initial stiffness defined by F_e/y_e (see Fig. 2.7). For a circular rod in torsion, $F_e = \frac{3}{4} F_p$, and it is assumed that the device will be designed such that $y_m = 15\ y_e$. Thus,

$$K_e = \frac{11.25\ F_p}{y_m} \qquad (2.3)$$

Equating Eqs. (2.1) and (2.2), the required plastic force per pile is given by

$$F_p = \frac{\pi}{1.82}\ \beta_p M_t \omega_p^2 y_m \qquad (2.4)$$

This will be referred to as the theoretical force required to produce a response equivalent to that of a system with viscous damping β_p. The time-history analyses discussed below are intended to investigate the behavior of hysteretic-damped systems and, in particular, the validity of Eq. (2.4).

The analyses of the system with a damping device were made using the model shown in Fig. 2.1, with the addition of a horizontal force between the ground and the foundation mass (M_f) following the function shown in Fig. 2.7. These results are compared with those for viscous damped systems.

Typical results are shown in Table 2.4. In the upper table, peak response values are listed for a range of viscous damping values. In the lower table, the responses are listed for various values of the peak force provided by the damping device. The center column (fraction = 0.1) is for a force determined by Eq. (2.4) with β_p = 0.10 and y_m taken to be the design pile displacement for that damping value. The other columns are for fractions of the force so determined. In general, it may be seen that hysteretic damping is less effective (i.e., the responses are larger) than the corresponding viscous damping. The pile displacements obtained in these analyses are plotted in Fig. 2.8. This indicates that, in order to obtain the same displacement as that resulting from a certain amount of viscous damping, the force given by Eq. (2.4) should be increased. In particular, if the equivalent of 10% viscous damping is desired, Eq. (2.4) should be multiplied by 1.6.

However, the results obtained were erratic. Results are shown in Figs. 2.9 and 2.10 for a ten-second design period and two different ground-motion records. In Fig. 2.9 the difference between the two types of damping is less than in Fig. 2.8, while in Fig. 2.10, hysteretic damping produced smaller displacements than did viscous damping. The probable reason for this erratic behavior is that the effect of different damping functions on peak response, which generally occurs early in the record, is not the same as for a steady-state response.

Therefore, the results of this limited study are inconclusive. However, it can be stated that the type of energy-absorbing device considered herein can be an effective means of reducing response. One way to eliminate the need to relate the two kinds of damping would be to construct design spectra (such as Figs. 2.2 and 2.3) for hysteretic rather than viscous damping.

For the purpose of designing devices (see Sect. 3.3), it will be tentatively assumed that the force given by Eq. (2.4) should be multiplied by 1.5. The required forces are quite modest. For example, for 10% equivalent viscous damping, a pile load of 500 kips, a twelve-second design period, and a peak pile displacement of 1.4 ft, the required damping force is only 1.5 kips per pile.

2.6 Effect of Peak Ground Displacement

All previously reported results were based on an arbitrary peak ground displacement of 30.48 cm (1 ft). The effect of variation in this design parameter is shown in Fig. 2.11, which is typical of all the cases investigated. As expected, the required pile length and diameter increase with increasing ground displacement. The relationship is almost linear, but the increase in pile size is less than proportional to the displacement. Under the assumptions made here, the maximum pile displacement and building base shear are exactly proportional to the peak ground displacement.

2.7 Summary

None of the additional investigations altered the conclusions drawn in the first report (Ref. 3). The sleeved-pile system accomplished its intended purpose and the simple design procedure originally proposed is quite adequate. The use of energy-absorbing devices, while not necessary, may prove to be economically desirable.

CHAPTER III - DESIGN CONSIDERATIONS

3.1 General

The proper design of a sleeved-pile foundation system is, of course, very site-dependent, and it is difficult to generalize the process. However, one can make some observations regarding desirable ranges of the design parameters and practical considerations in the design of such systems. No attempt will be made here to discuss the details of the design, e.g., the design of the sleeve, the transfer of forces between the pile and the sleeve, or that between the top of the pile and the bottom of the superstructure. Attention will be focussed on the design of the pipe pile itself.

The construction technique would be essentially the same as for a caisson foundation. The only difference is that the caisson (sleeve) would not be completely concrete-filled. Caisson lengths as large or larger than those required for the sleeves are not uncommon. The sleeves could be of the telescope type, which is convenient for construction, because the required clearance between pipe and sleeve decreases rapidly with depth. In most locations it would be difficult to keep the sleeve dry. However, this is not necessary providing that the pipe and sleeve are corrosion-protected. In fact, it might be possible to use water within the sleeve as a damping mechanism. If rock is near the surface, it would be necessary to place the pipe in a drilled hole. While this is certainly feasible, it may not be economically attractive.

Turning to the general design parameters, it is advantageous to adopt a long design period, i.e., ten seconds or more. This not only decreases the seismic forces on the superstructure; it also decreases the required length and diameter of the pile. Furthermore, the pile displacement decreases, thus reducing the required clearance within the sleeve and around the building, and the required flexibility of the utility connections into the building.

Having selected a design period, the designer must then select a pile spacing, which determines the load per pile and hence the properties of the pile itself. In some cases, the depth of bedrock may determine the pile length and the procedure would be applied in reverse. It is also possible that the make-up of the building base may affect pile spacing.

Steel pipes of the dimensions required are readily available from several suppliers at reasonable cost. High yield strength is desirable, but is limited to about 60 ksi by current U.S. manufacturing techniques. However, foreign manufacturers have produced pipes with considerably higher strengths. The ratio of the wall thickness to pipe diameter is also limited by the production process, and some of the plots herein may indicate unacceptable ratios. However, this ratio can be made any value desired, although decreasing thickness requires less load per pile or greater length.

In order to implement the sleeved-pile concept successfully, some modification of existing codes may be necessary. For example, the structural ductility requirements probably should not be applied, since the building remains elastic. Also, the concept of designing the piles for yield point stresses during the maximum expected earthquake, while quite logical, does not fit within the usual code framework.

3.2 Pile Dimensions vs. Spacing or Load

For design purposes, it is convenient to plot the previously given data in the form of Figs. 3.1 and 3.2. For given values of peak ground displacement, design period (T_p), soil condition, steel yield strength, pile damping ratio (β) and wall thickness (t), there is a unique relationship between pile load (or pile spacing) and the required pile free length and diameter. These relationships are shown in Fig. 3.1 (pile free length) and Fig. 3.2 (pile diameter). The plots are based upon a peak ground displacement of one foot. The damping ratio of 0.5% is intended to represent that inherent in the pile itself, while 10% is a possible equivalent viscous damping ratio when energy-absorbing devices are used. The wall thicknesses of 1.0 and 1.5 ins. are arbitrarily chosen. Additional curves for other parametric combinations can easily be generated (see Sect. 1.3.2).

For example, suppose a design is being made for the following parameters:

T_p = 12 secs

Soil Site

F_y = 60 ksi

β_p = 0.5%

t = 1.0 ins

P = 400 kips

Then, from Fig. 3.1(c), the required free length of the pile is 48 ft, and from Fig. 3.2(c), the required diameter is 1.6 ft. For this design, the peak pile displacement is predicted to be 2.0 ft (Fig. 1.2), and the predicted building base shear coefficient is

$$\frac{\omega_p^2 \, y_m}{32.2} = 0.017 \text{ g}$$

Consider now a hypothetical 10-story building with plan dimensions of 100 ft by 200 ft. The total weight of the superstructure might be about 34,000 kips. If the piles are regularly spaced at 20 ft in both directions (66 piles), the load per pile is about 515 kips. Assuming the same parameters as in the preceding example, except pile load, the required pile length is 52.5 ft and the required diameter 21.3 ins.

Variations on this design are shown in Table 3.1(a-f). Results for design periods of 10 and 12 secs, and for pile spacings of 10, 20 and 33.3 ft, are given. Also given are designs utilizing damping devices with β_p = 10% (see Sect. 3.3). All of these designs are for a soft soil site with peak ground displacement of 12 ins, and 60 ksi steel with a wall thickness of 1.0 in.

Considering the case of a 12-second design period without dampers, the length and diameter of the piles increase with spacing, but the total amount of steel required decreases:

Pile Spacing (ft)	No. of Piles	Length (ft)	Diameter (ins)	Total Steel (tons)
10	231	36.9	10.8	490
20	66	52.5	21.3	390
33.3	28	68.6	35.0	360

The optimum pile spacing obviously depends on many factors, such as the cost of installing sleeves to various depths, that cannot be considered here. However, it appears that neither a small nor a large spacing is attractive. Note that all of these designs result in the same seismic forces on the superstructure, since this depends only on period and damping.

To reverse the problem, suppose that site conditions dictated a pile free length of 40 ft. Then, by Fig. 3.1(c), the load per pile must be 210 kips and the pile spacing (and hence number of piles) must be selected accordingly. The diameter for this case would be 14.2 ins (Fig. 3.2(c)). It might be desirable to increase the pipe wall thickness, which would increase the load per pile in proportion, while the diameter remains unchanged.

Similar design results for a 20-story building are shown in Table 3.2(a-f). All other parameters are the same as for the 10-story building, the only difference being in the load per pile. This increases both the length and diameter. It would probably be appropriate to use a thicker pipe wall for the taller building.

The design procedure illustrated above is quite simple and straightforward. By changing the values of the several parameters, a variety of design solutions are possible for a given situation.

3.3 Design with Energy-Absorbing Devices

The use of energy-absorbing devices to add hysteretic damping forces between the pile top and the ground is discussed in Sect. 2.5. It is concluded there that the required maximum force given by Eq. (2.4) should be increased by approximately 1.5 to produce the same effect as viscous damping with ratio β_p. In this section, torsional devices as depicted in Fig. 2.6 are actually dimensioned for use with the 10- and 20-story buildings previously used for illustration.

Referring to Figs. 2.6 and 2.7, the fully plastic resistance of the device is given by

$$F_p = \frac{4}{3} \frac{\pi \tau_y r^3}{\ell} \tag{3.1}$$

where

r = radius of torsion bar
τ_y = shear yield strength
ℓ = length of device arms.

The elastic limit displacement is given by

$$y_e = \frac{\tau_y a \ell}{Gr} \tag{3.2}$$

where a = length of torsion bar
G = shear modulus

Letting, by design, $y_m = 15\, y_e$, solving Eqs. (3.1) and (3.2) yields

$$a = \frac{G}{20\,\pi\sigma_y^2} \cdot \frac{F_p\, y_m}{r^2} \tag{3.3}$$

$$\ell = \frac{4}{3}\,\pi\sigma_y \frac{r^3}{F_p} \tag{3.4}$$

where F_p is the desired force and y_m is the design pile displacement. These equations assume that the torque is always equal to $\frac{F}{2} \times \ell$, which is sufficiently accurate unless the ratio of y_m to ℓ is excessive. Thus, for a given material and bar radius, the dimensions of the device may be determined.

Shown in Tables 3.1 and 3.2 are the required pile sizes and possible dimensions of the devices for each case presented. These are arbitrarily based on an equivalent viscous damping ratio of 10%. For these calculations, the shear yield strength was taken to be 25 ksi and the shear modulus as 12×10^3 ksi. It is assumed that devices will be placed outside the building base opposite each row of piles. Thus n is the number of piles per device, first along the long sides of the foundation and then along the short sides. F_p is computed by Eq. (2.4), where M_t is the mass per pile, multiplied by 1.5 and the number of piles per device. Of course, any number of devices could be used.

The dimensions shown for the devices are merely illustrative. The length, ℓ and a, is computed by Eqs. (3.3) and (3.4) after selecting a bar radius which produces reasonable values of these dimensions.

It may be observed that the use of dampers significantly reduces the length and diameter of the piles. Furthermore, it reduces even further the seismic forces on the superstructure. In some cases (e.g., Table 3.1(b)), the ratio of pile diameter to wall thickness is probably too small, and a thinner pile should be used.

These results are intended to demonstrate that the use of these devices is indeed feasible, i.e., the design dimensions are reasonable, and that their cost may be more than balanced by the reduction in pile size and the decrease in superstructure displacement and acceleration.

3.4 Cost Considerations

It is not possible, within the limitations of this effort, to make a cost-benefit analysis for sleeved-pile systems. This would be greatly dependent on the soil conditions at the site as well as the type and size of the building to be constructed. It would also depend upon the seismicity of the area. However, it is possible to make some general observations.

The additional cost, as compared to a conventional design, lies primarily in the installation of the sleeves, including both the drilling operation and the steel in the shell. However, this is similar to the installation of caisson systems, which are frequently used, and the inherent cost does not preclude the use of sleeved piles. Another additional cost is the construction of a retaining wall, perhaps sheet piling, around the perimeter of the foundation (see Fig. 1.1). This cost would depend upon whether or not a basement was required. The cost of the pipe steel is probably not a major item. If the building site required piles in any case, the additional cost would be less.

The cost savings may be divided into two parts: (1) The reduced cost of the superstructure due to the virtual elimination of critical design seismic forces, and (2) the reduction in the cost of future earthquake damage, including loss of life or personal injury. In Ref. 7 it is estimated that the increase in initial cost due to seismic requirements for Zone 3 (1970 UBC) is in the order of 5% of the total construction cost. If no seismic requirements (including ductility requirements, etc.) are necessary for a building on sleeved piles, which might be debatable, this amount would be saved. It is also possible that certain economical building types (e.g., masonry or precast panel buildings), which otherwise might not be permitted in seismic regions, could be used if founded on sleeved piles.

The estimation of future damage costs is obviously more difficult. However, using Ref. 7 as a guideline (no precise figures are given therein), it appears that the present value of future losses for a conventional building, located in and designed for Zone 3, could easily be 10% or more of the initial total cost. Whatever the value, it would be greatly reduced by the use of sleeved piles.

Thus it may be speculated that the use of a sleeved-pile system would be justified if the additional costs were less than 10 - 15% of the total construction cost. For example, a building of approximately ten stories might cost $500 per sq ft of foundation area, and hence the justified cost of a sleeved-pile system would be $50 - $75 per sq ft. Without the benefit of cost estimates, it seems probable that a system could be installed within this cost limit. However, the decision to increase the initial cost obviously depends upon the owner's attitude toward the risk of future damages.

All of the above comments are highly speculative and are merely indicative of the considerations which should be taken into account in evaluating the proposed system. A more thorough investigation, including actual cost estimates for a particular project, is required.

CHAPTER IV - CONCLUSIONS

The analytical results presented herein clearly demonstrate the effectiveness of the sleeved-pile concept as a method of isolating buildings from the effects of earthquake ground motions. The seismic forces on the superstructure may be reduced to such low levels that they do not affect the design, and the structure remains elastic under the design earthquake.

The simple design procedure, based on a one-degree model, has been shown to be adequate for the design of the piles.

The higher modes of the building superstructure have little effect on pile behavior and base shear, but do moderately affect the vertical distribution of seismic force.

Consideration of the total pile group as opposed to an isolated pile, including interaction between piles and overturning moment, has little effect on the results.

The required pile length and diameter increase with increasing ground displacement, but less than proportionally.

It is desirable to design the pile system to have a very long natural period, since this reduces both pile length and diameter and peak relative displacement. Periods even longer than the 12-second limit imposed herein are probably appropriate.

An artificial increase in damping, by the use of energy-absorbing devices, is quite beneficial. The required size, and probable cost, of such devices is modest.

With respect to the piles alone, it is probably economical to use a large spacing and hence a smaller number of piles, even though this increases the required length.

Based upon very limited considerations, it appears that the added cost of a sleeved-pile system, as compared to a conventional design, can probably be justified on the basis of savings in superstructure cost and in the expected cost of future earthquake damage.

It is recommended that further design studies, including detailed cost estimates, be undertaken to evaluate the economic feasibility of the proposed concept.

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TABLES

Pile Damping Ratio	0.5% or 10%									
Steel Yield Stress (MPa)	414 (60 ksi)				689 (100 ksi)					
Pile Axial Force (MN)	1.11 (250k)		2.22 (500k)		1.11 (250k)		2.22 (500k)		3.34 (750k)	
Wall Thickness (cm)	2.54	3.81	2.54	3.81	2.54	3.81	2.54	3.81	2.54	3.81
Case Identification	A	B	C	D	E	F	G	H	I	J

Table 1.1 Combinations of Pile Design Parameters

Earthquake Record	Design Period (seconds) 8	10	12	Design Period (seconds) 8	10	12
EQ-1	50.0 (0.051g)	51.7 (0.053g)	41.8 (0.043g)	46.7 (0.048g)	33.3 (0.034g)	22.9 (0.023g)
EQ-2	49.8 (0.051g)	34.7 (0.035g)	26.4 (0.027g)	48.4 (0.049g)	30.0 (0.031g)	19.2 (0.020g)
EQ-3	54.0 (0.055g)	27.9 (0.028g)	16.5 (0.017g)	50.5 (0.051g)	27.0 (0.028g)	15.9 (0.016g)
EQ-4	48.3 (0.049g)	29.2 (0.030g)	20.4 (0.021g)	46.9 (0.048g)	27.4 (0.028g)	17.2 (0.018g)

Acceleration at Building Top (cm/sec^2) — Building Base Shear/Building Mass (cm/sec^2)

EQ-1	1.77	1.27	.73	46.6 (0.047g)	37.8 (0.039g)	27.0 (0.028g)
EQ-2	1.84	1.10	.85	48.2 (0.049g)	32.2 (0.033g)	21.0 (0.021g)
EQ-3	1.91	1.02	.60	51.1 (0.052g)	27.1 (0.028g)	16.0 (0.016g)
EQ-4	1.75	1.03	.62	47.1 (0.048g)	27.9 (0.028g)	18.2 (0.019g)

Building Displacement (cm) — Pile Shear/Total Mass (cm/sec^2)

EQ-1	75.1	71.3	61.2	372	442	450
EQ-2	77.3	69.7	59.8	387	419	432
EQ-3	78.8	67.1	57.0	411	385	381
EQ-4	74.0	67.6	56.8	409	422	444

Pile Displacement (cm) — Pile Stress (MPa)

Table 1.2 Time-History Analysis Results; Case A, β_p = 0.5%, f_y = 414 MPa, P = 1.11 MN

Earthquake Record	Design Period (seconds) 8	10	12	Design Period (seconds) 8	10	12
EQ-1	51.4 (0.052g)	56.2 (0.057g)	45.2 (0.046g)	46.7 (0.048g)	34.2 (0.035g)	23.6 (0.024g)
EQ-2	49.8 (0.051g)	33.8 (0.034g)	24.3 (0.025g)	48.4 (0.49g)	28.9 (0.029g)	18.3 (0.019g)
EQ-3	53.2 (0.054g)	27.5 (0.028g)	18.6 (0.019g)	50.1 (0.051g)	27.0 (0.028g)	15.9 (0.016g)
EQ-4	49.9 (0.051g)	27.7 (0.028g)	21.8 (0.022g)	47.0 (0.048g)	27.5 (0.028g)	17.0 (0.017g)

Building Acceleration at Top (cm/sec^2) — Building Base Shear/Building Mass (cm/sec^2)

EQ-1	1.77	1.32	.90	46.6 (0.047g)	38.9 (0.040g)	27.8 (0.028g)
EQ-2	1.85	1.10	.73	48.4 (0.049g)	30.8 (0.031g)	18.5 (0.019g)
EQ-3	1.91	1.02	.60	49.9 (0.051g)	26.9 (0.027g)	16.1 (0.016g)
EQ-4	1.76	1.03	.64	47.6 (0.048g)	27.7 (0.028g)	18.7 (0.019g)

Building Displacement (cm) — Pile Shear/Total Mass (cm/sec^2)

EQ-1	75.2	71.5	61.3	624	725	729
EQ-2	77.4	77.1	59.8	649	686	701
EQ-3	78.7	67.1	57.1	689	637	625
EQ-4	74.0	67.6	56.7	699	704	738

Pile Displacement (cm) — Pile Stress (MPa)

Table 1.3 Time-History Analysis Results; Case G,
β_p = 0.5%, f_Y = 689 MPa, $\dot{P}$ = 2.22 MN

	pile period (secs)	pile design value	pile response (average)	pile response high
pile displacement (cm)	8	82.3	76.3	78.8
	10	71.6	69.9	77.1
	12	61.0	58.7	61.3
pile acceleration (cm/sec^2)	8	50.9	48.2	51.1
	10	28.4	31.2	38.9
	12	16.8	20.4	27.8
pile stress (MPa) (for all periods)	Table 1.2	414	413	450
	Table 1.3	689	684	738

Table 1.4 Comparison of Time-History Results With Design Values

	Acceleration at Building Top (cm/sec^2) First Mode	Acceleration at Building Top (cm/sec^2) All Modes	Building Base Shear/Building Mass (cm/sec^2) First Mode	Building Base Shear/Building Mass (cm/sec^2) All Modes
EQ-1	53.6 (0.055g)	70.5 (0.072g)	33.6 (0.034g)	35.7 (0.036g)
EQ-2	34.7 (0.035g)	41.0 (0.042g)	28.7 (0.029g)	29.5 (0.030g)
EQ-3	27.5 (0.028g)	30.1 (0.031g)	27.7 (0.028g)	27.2 (0.028g)
EQ-4	28.8 (0.049g)	37.2 (0.051g)	27.0 (0.027g)	27.2 (0.028g)

	Building displacement (cm) First Mode	Building displacement (cm) All Modes	Pile Shear/Total Mass (cm/sec^2) First Mode	Pile Shear/Total Mass (cm/sec^2) All Modes
EQ-1	1.29	1.24	37.1 (0.038g)	37.3 (0.038g)
EQ-2	1.10	1.05	29.2 (0.030g)	29.1 (0.030g)
EQ-3	1.01	.95	27.0 (0.027g)	27.0 (0.027g)
EQ-4	1.02	.94	27.1 (0.028g)	27.0 (0.027g)

	Pile Displacement (cm) First Mode	Pile Displacement (cm) All Modes	Pile Stress (MPa) First Mode	Pile Stress (MPa) All Modes
EQ-1	71.7	71.7	723	717
EQ-2	70.0	70.0	686	686
EQ-3	67.7	67.7	640	640
EQ-4	67.2	67.2	699	698

Table 2.1 (Ref. 4) Peak Time-History Responses;

Case G: f_y = 689 MPa, P = 2.22 MN, t = 2.54 cm;
β_p = 0.5%; T_{px} = 10 sec.

	Single Pile	Pile Group	Single Pile	Pile Group
EQ-1	70.5 (0.072g)	65.8 (0.067g)	35.7 (0.036g)	31.9 (0.032g)
EQ-2	41.0 (0.042g)	61.4 (0.063g)	29.5 (0.030g)	30.0 (0.031g)
EQ-3	30.1 (0.031g)	42.3 (0.043g)	27.2 (0.028g)	26.9 (0.027g)
EQ-4	37.2 (0.038g)	48.5 (0.049g)	27.2 (0.028g)	27.3 (0.028g)
	Acceleration at Building Top (cm/sec^2)		Building Base Shear/Building Mass (cm/sec^2)	

EQ-1	1.24	1.28	37.3 (0.038g)	32.9 (0.034g)
EQ-2	1.05	1.13	29.1 (0.030g)	29.4 (0.030g)
EQ-3	.95	.96	27.0 (0.027g)	27.0 (0.027g)
EQ-4	.94	.95	27.0 (0.027g)	26.9 (0.027g)
	Building Displacement (cm)		Pile Shear/Total Mass (cm/sec^2)	

EQ-1	71.7	71.7	717	701
EQ-2	70.0	69.9	686	687
EQ-3	67.7	67.8	640	640
EQ-4	67.2	67.1	698	697
	Pile Displacement (cm)		Pile Stress (MPa)	

Table 2.2 (Ref. 4) Peak Time-History Responses: Pile Group Effect;

Case G: f_y = 689 MPa, P = 2.22 MN, t = 2.54 cm;

β_p = 0.5%; T_{px} = 10 sec.

All Building Modes

	Without Building Overturning	With Building Overturning
EQ-1	617	664
EQ-2	630	670
EQ-3	719	752
EQ-4	709	728

T_{px} = 8 sec.

EQ-1	717	744
EQ-2	686	710
EQ-3	640	676
EQ-4	698	715

T_{px} = 10 sec.

EQ-1	699	717
EQ-2	669	683
EQ-3	594	618
EQ-4	711	720

T_{px} = 12 sec.

Table 2.3 (Ref. 4) Peak Time-History Response - Pile Stress (MPa): Building Overturning Effect; 10-Story Building Case G: f_y = 689 MPa, P = 2.22 MN, t = 2.54 cm; β_p = 0.5%.

	Viscous-Damped System β_p(%)				
	6	8	10	15	20
Acceleration at Building Top (cm/sec^2)	16.5	16.1	15.6	14.5	13.6
Building Base Shear/ Building Mass (cm/sec^2)	7.6	7.1	7.0	6.7	6.4
Building Displacement (cm)	.26	.26	.26	.24	.25
Pile Shear/Total Mass (cm/sec^2)	8.1	7.8	7.4	7.0	6.7
Pile Displacement (cm)	23.0	21.7	20.6	18.2	16.2
Pile Stress (MPa)	413	396	381	347	320

	Hysteretic-Damped System Fraction of Plastic Force (β_p = 10%)				
	0.6	0.8	1.0	1.5	2.0
Acceleration at Building Top (cm/sec^2)	19.1	18.0	16.4	14.1	12.6
Building Base Shear/ Building Mass (cm/sec^2)	8.6	8.7	8.7	8.3	8.2
Building Displacement (cm)	.34	.34	.31	.28	.26
Pile Shear/Total Mass (cm/sec^2)	9.7	9.8	9.3	8.6	8.4
Pile Displacement (cm)	26.0	25.7	24.6	20.9	19.3
Pile Stress (MPa)	463	460	447	399	378

Table 2.4 - Peak Time-History Responses: Viscous vs. Hysteretic Damping; Case G: f_y = 689 MPa, P = 2.22 MN, t = 2.54 cm; T_{px} = 12 sec. EQ-4

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m(in) β = 10%
10	10	147	10	18.0

<u>Pile Design</u>: Thickness = 1.0 in.

With Damper
L = 30.6 ft
D = 9.6 in

Without Damper
L = 43.0 ft
D = 12.4 in

<u>Damping Mechanism</u>

n	F_p(kips)	r(in)	ℓ (in)	a(in)
5.5	3.85	1.25	53.1	13.6
10.5	7.36	1.50	48.0	18.0

Table 3.1(a) - Sample Design, 10-Story Building

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m(in) β = 10%
10	10	147	12	16.8

<u>Pile Design</u>: Thickness = 1.0 in.

With Damper
L = 28.3 ft.
D = 8.8 in.

Without Damper
L = 36.9 ft.
D = 10.8 in.

<u>Damping Mechanism</u>

n	F_p(kips)	r(in)	ℓ (in)	a(in)
5.5	2.50	1.25	81.9	8.2
10.5	4.77	1.50	74.1	10.9

Table 3.1(b) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) β = 10%
20	10	515	10	18.0

Pile Design: Thickness = 1.0 in

With Damper
L = 43.9 ft
D = 19.3 in

Without Damper
L = 61.4 ft
D = 25.0 in

Design Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
3	7.36	1.50	48.0	18.0
5.5	13.49	2.00	62.1	18.6

Table 1.3(c) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) β = 10%
20	10	515	12	16.8

Pile Design: Thickness = 1.0 in

With Damper
L = 40.7 ft
D = 17.6 in

Without Damper
L = 52.5 ft
D = 21.3 in

Design Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
3	4.77	1.50	74.1	10.9
5.5	8.74	1.75	64.2	14.7

Table 1.3(d) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) β = 10%
33.3	10	1214	10	18.0

Pile Design: Thickness = 1.0 in

With Damper
L = 57.7 ft
D = 31.9 in

Without Damper
L = 79.6 ft
D = 41.5 in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
2	11.56	1.75	48.5	20.8
3.5	20.23	2.25	58.9	20.0

Table 3.1(e) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) β = 10%
33.3	10	1214	12	16.8

Pile Design: Thickness = 1.0 in

With Damper
L = 53.6 ft
D = 28.9 in

Without Damper
L = 68.6 ft
D = 35.0 in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
2	7.49	1.50	47.2	17.1
3.5	13.11	2.00	63.9	16.1

Table 3.1(f) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) β = 10%
10	20	251	10	18.0

Pile Design: Thickness - 1.0 in

With Damper
$L = 35.3$ ft
$D = 12.8$ in

Without Damper
$L = 49.7$ ft
$D = 16.6$ in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
5.5	6.57	1.50	53.8	16.1
10.5	12.6	2.00	66.7	17.3

Table 3.2(a) - Sample Design, 20-Story Building

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) β = 10%
10	20	251	12	16.8

Pile Design: Thickness = 1.0 in

With Damper
$L = 32.5$ ft
$D = 11.7$ in

Without Damper
$L = 42.7$ ft
$D = 14.3$ in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ (in)	a(in)
5.5	4.26	1.25	48.0	14.0
10.5	8.13	1.75	69.0	13.7

Table 3.2(b) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) $\beta = 10\%$
20	20	879	10	18.0

Pile Design: Thickness = 1.0 in

With Damper

L = 51.9 ft

D = 26.2 in

Without Damper

L = 72.4 ft

D = 34.4 in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
3	12.55	2.00	66.7	17.3
5.5	23.01	2.25	51.8	25.0

Table 3.2(c) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m(in) $\beta = 10\%$
20	20	879	12	16.8

Pile Design: Thickness = 1.0 in

With Damper

L = 48.1 ft

D = 23.9 in

Without Damper

L = 61.9 ft

D = 29.0 in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ (in)	a(in)
3	8.13	1.75	69.0	13.7
5.5	14.91	2.00	56.2	19.2

Table 3.2(d) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) $\beta = 10\%$
33.3	20	2071	10	18.0

Pile Design: Thickness = 1.0 in

With Damper
$L = 68.9$ ft
$D = 44.5$ in

Without Damper
$L = 93.0$ ft
$D = 57.6$ in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
2	19.72	2.25	60.5	21.5
3.5	34.51	2.50	47.4	30.4

Table 3.2(e) - Sample Design

Pile Spacing (ft)	Number of Stories	Pile Load (kips)	Design Period (sec)	y_m (in) $\beta = 10\%$
33.3	20	2071	12	16.8

Pile Design: Thickness = 1.0 in

With Damper
$L = 64.2$ ft
$D = 40.0$ in

Without Damper
$L = 81.2$ ft
$D = 48.5$ in

Damping Mechanism

n	F_p(kips)	r(in)	ℓ(in)	a(in)
2	12.75	2.00	65.5	16.4
3.5	22.36	2.25	53.3	22.7

Table 3.2(f) - Sample Design

FIGURES

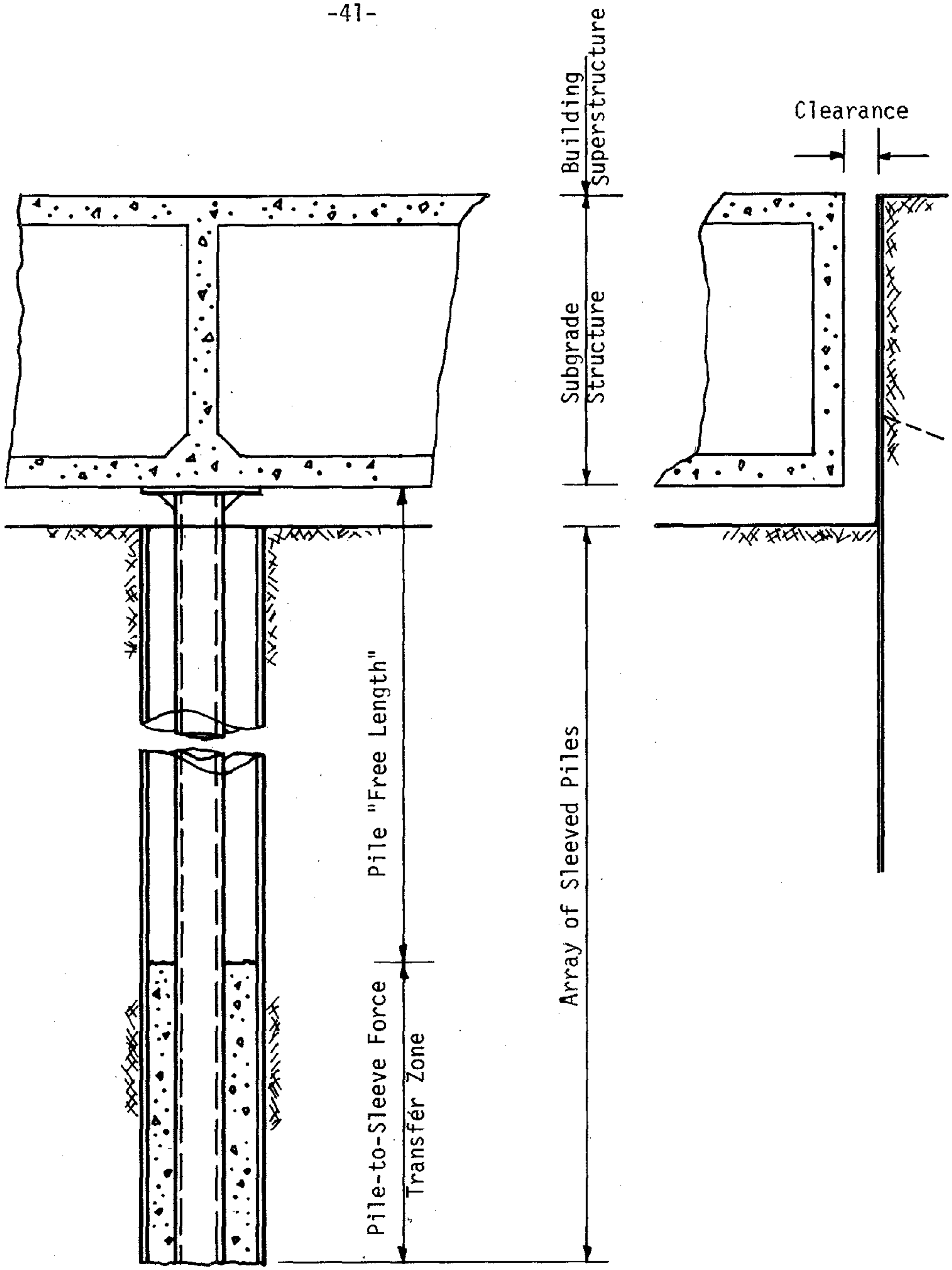


Fig. 1.1 Concept Sketch of Aseismic Building Foundation, Using Sleeved Piles

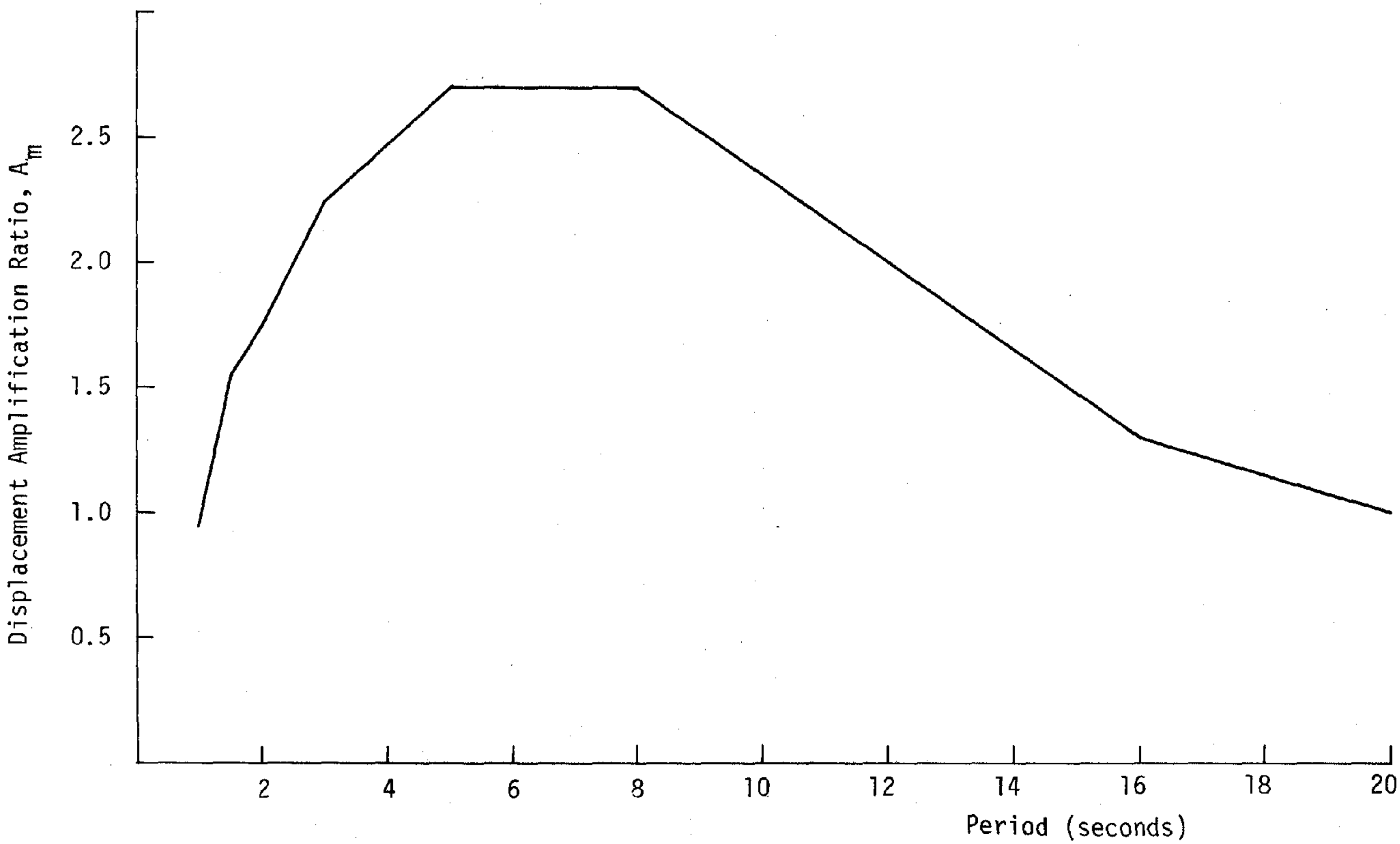


Fig. 1.2 Horizontal Displacement Design Spectrum
0.5% Damping

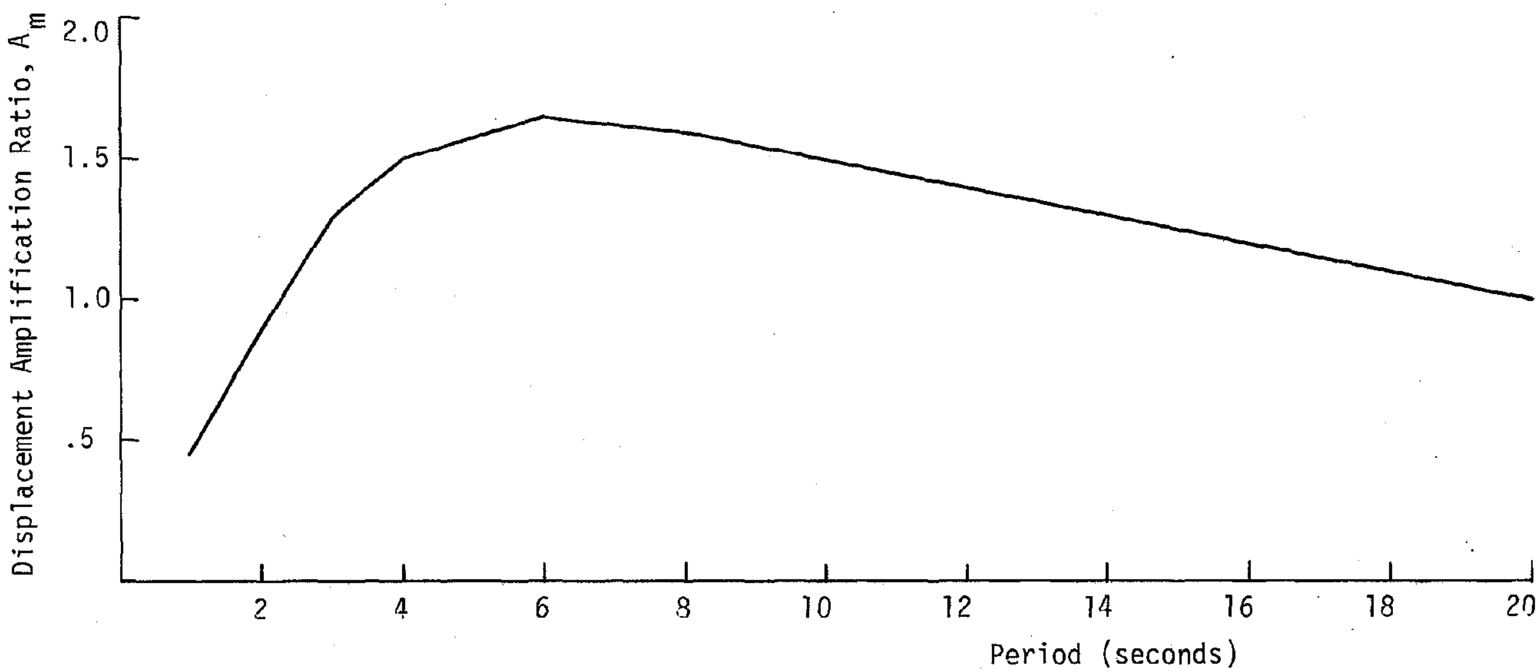


Fig.1.3 Horizontal Displacement Design Spectrum
10% Damping

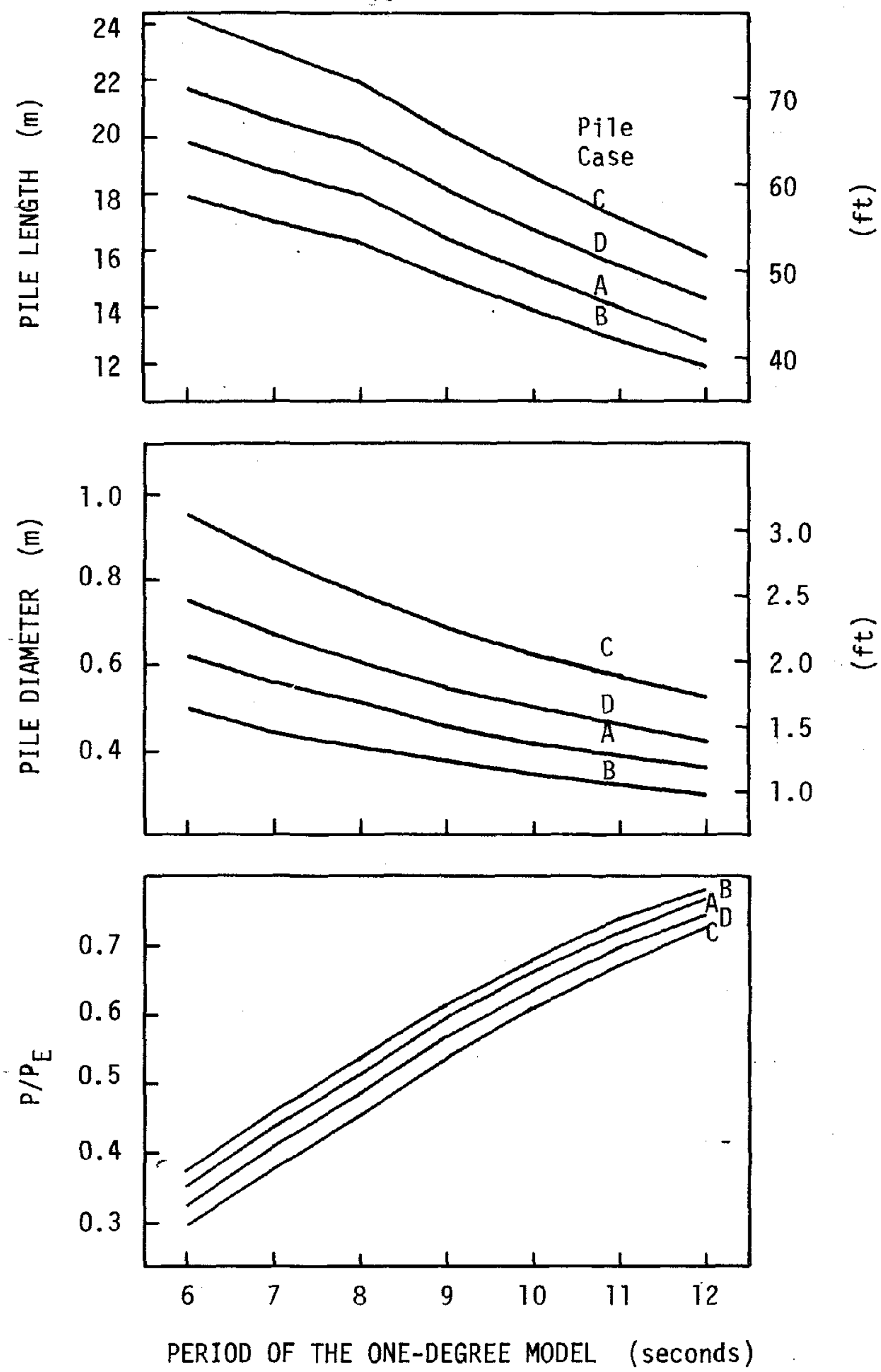


Fig. 1.4 Pile Free Length, Pile Diameter, and Ratio P/P_E for $\beta_p = 0.5\%$, f_y = 414 MPa (60 ksi)

See Table 1.1 for Pile Case Identification

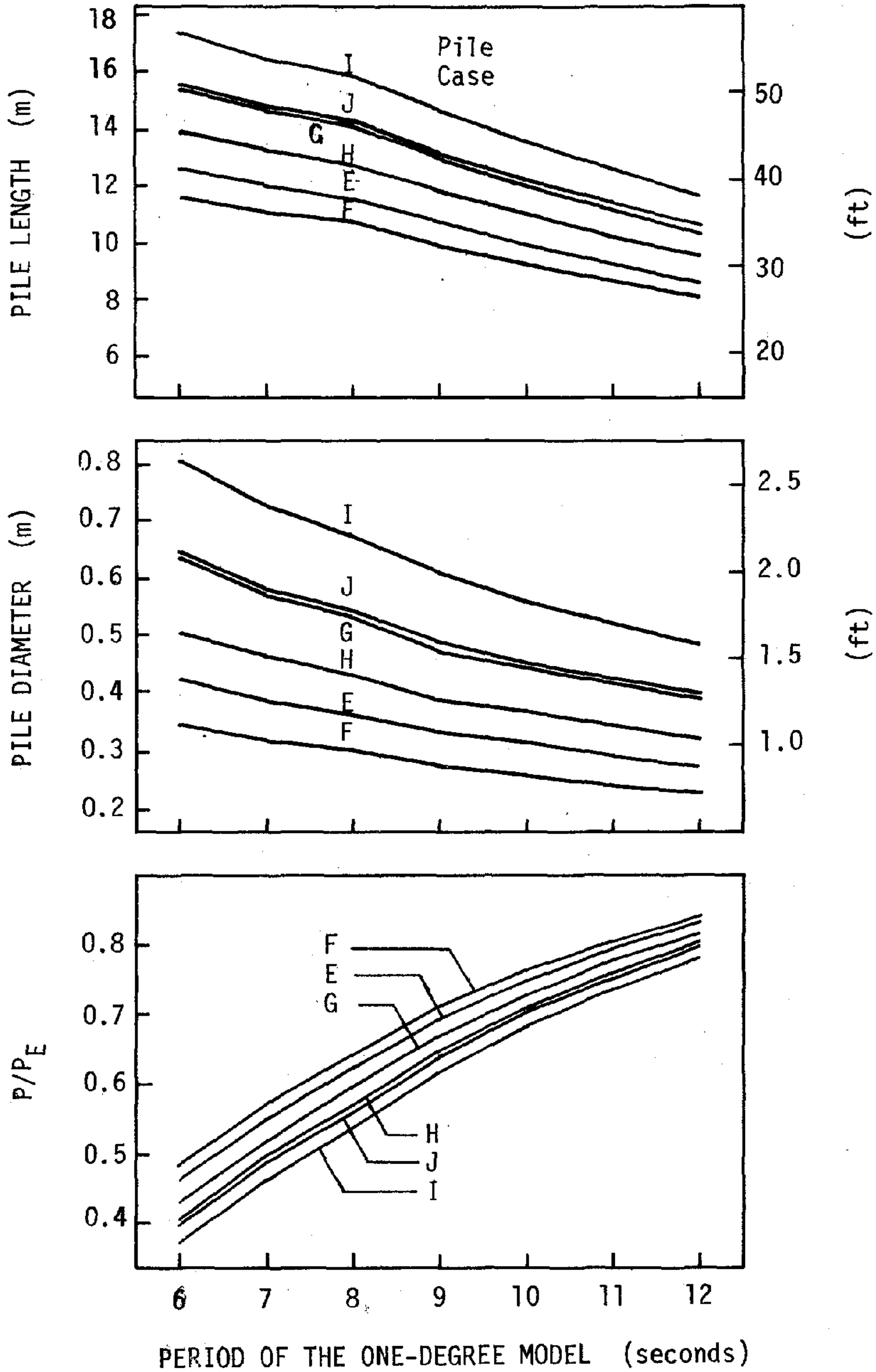


Fig. 1.5 Pile Free Length, Pile Diameter, and Ratio P/P_E for β_p = 0.5%, f_y = 689 Pa (100 ksi)

See Table 1.1 for Pile Case Identification

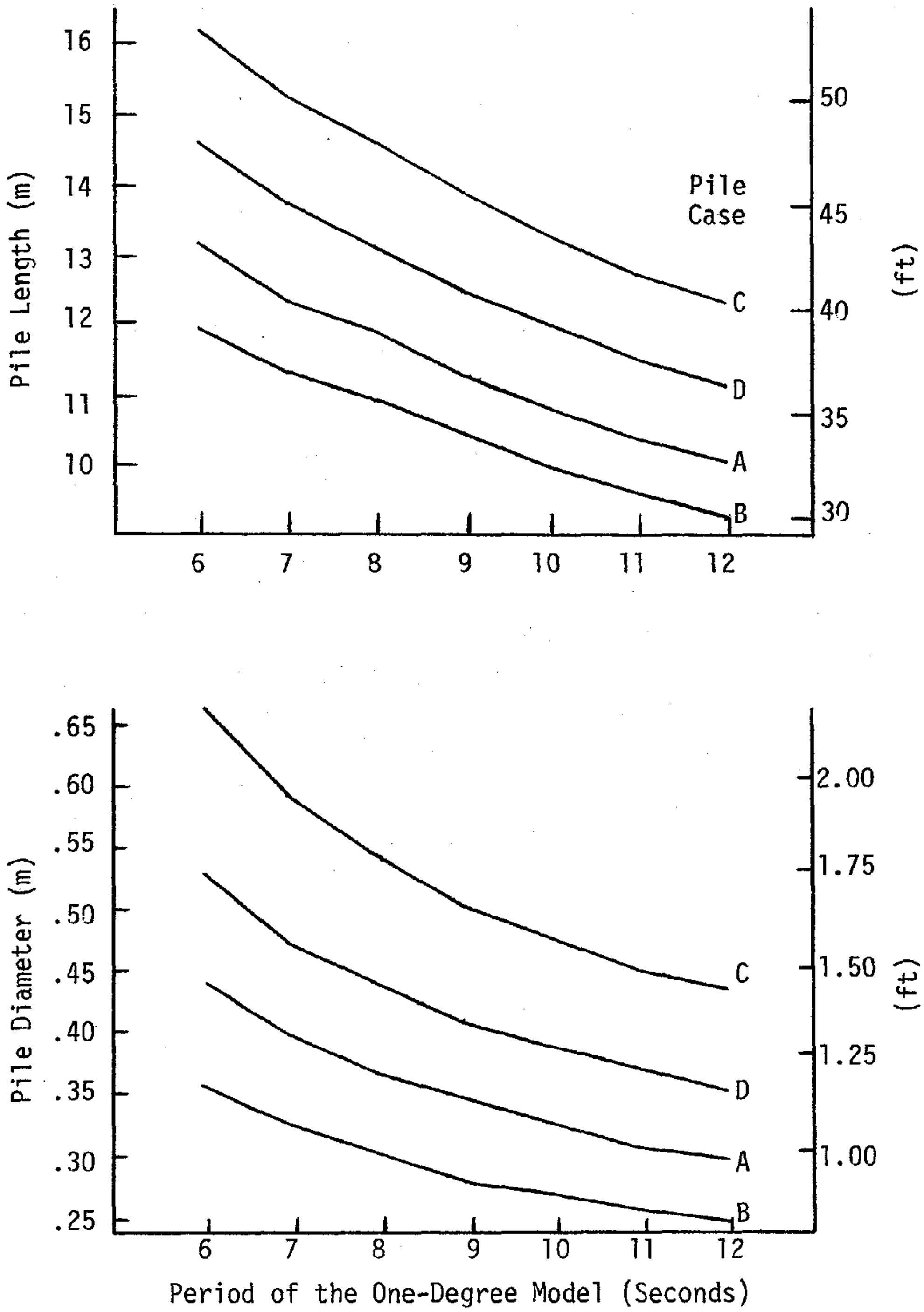


Fig. 1.6 Pile Free Length and Diameter
For β = 10%, f_y = 60 ksi

See Table 1.1 for Pile Case Identification

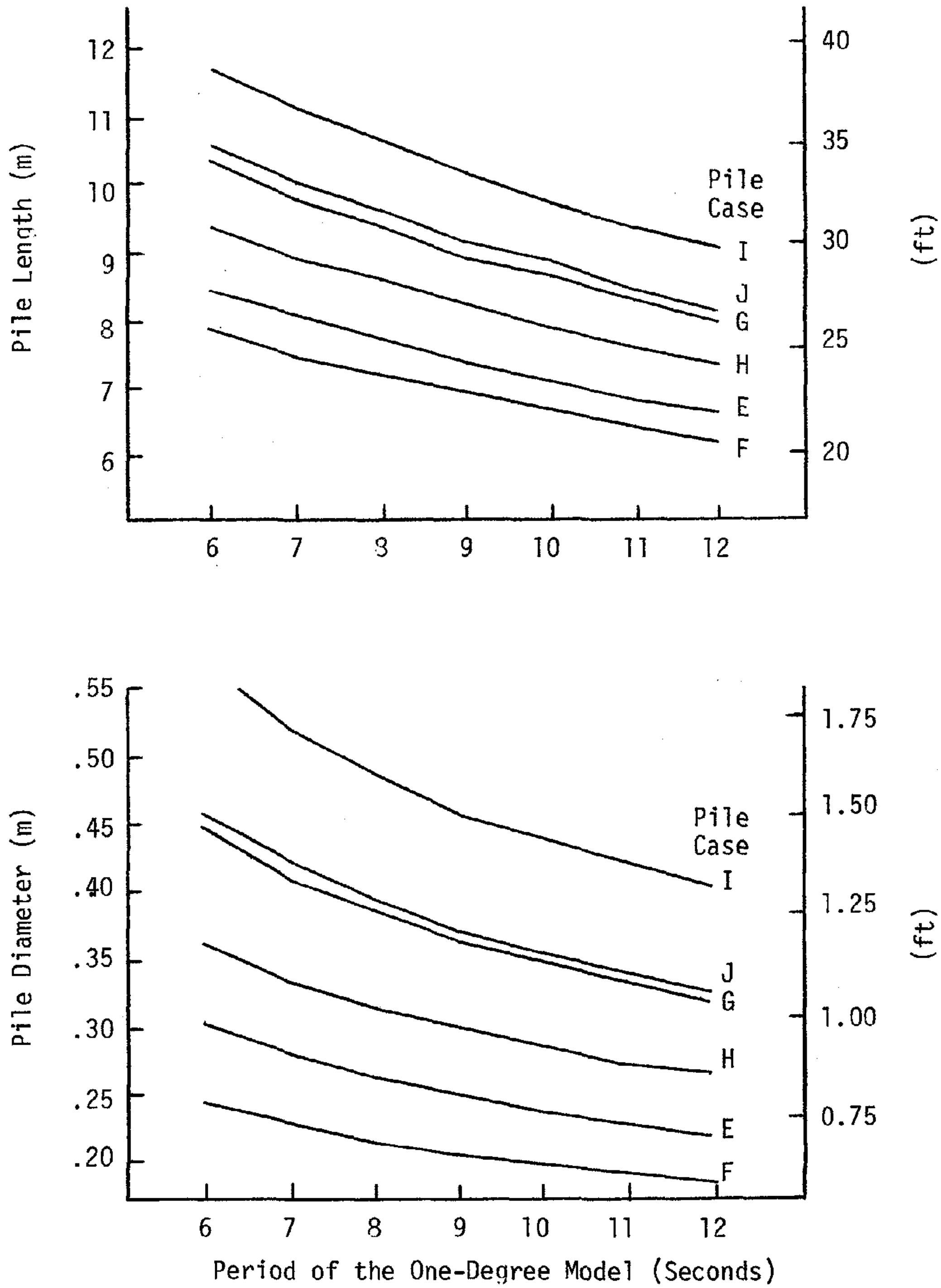


Fig. 1.7 Pile Free Length and Diameter For β = 10%, f_y = 100 ksi

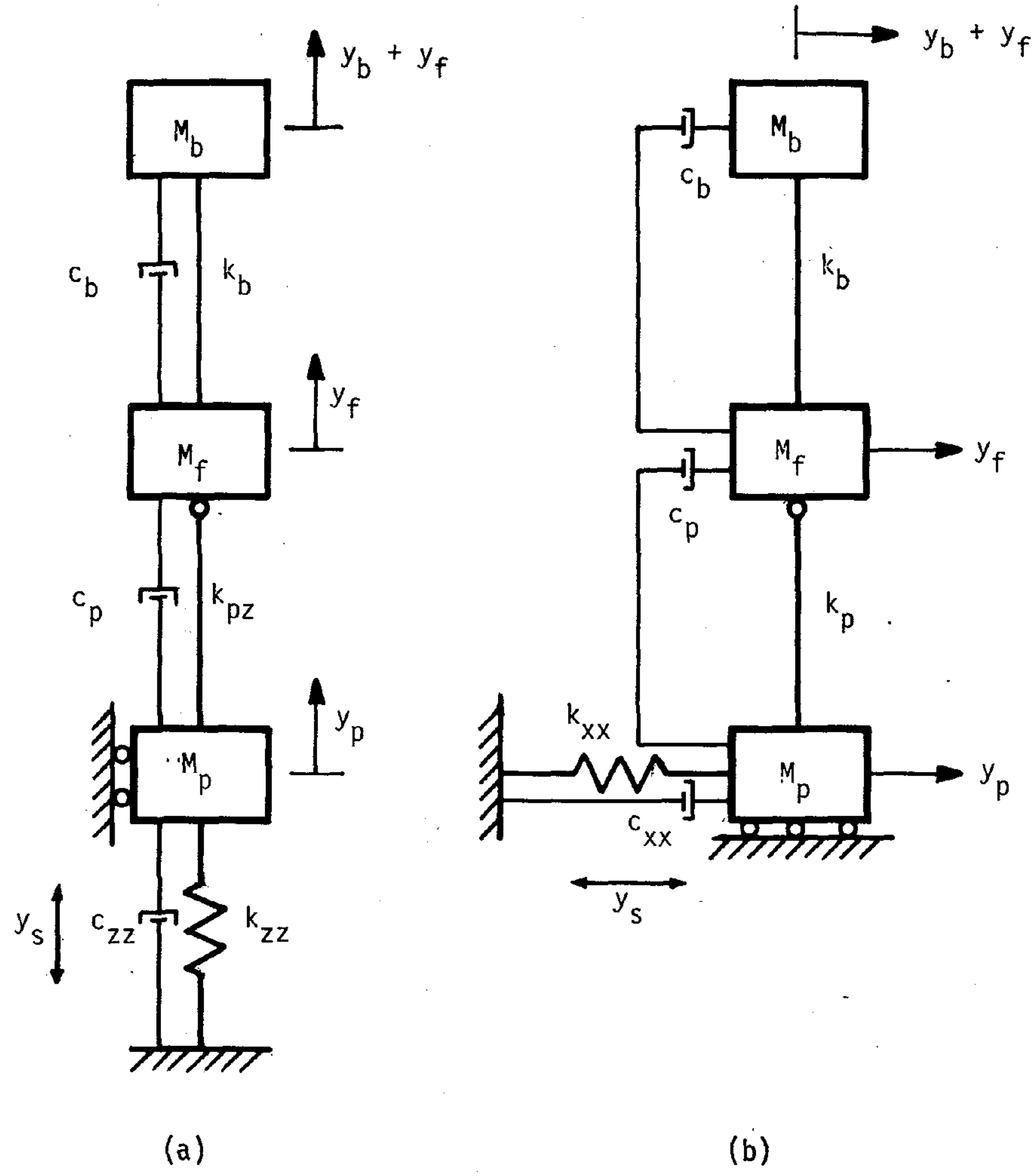


Fig. 1.8 Three Degree-of-Freedom Analytical Model for (a) Vertical and (b) Horizontal Response Used in the Preliminary Studies

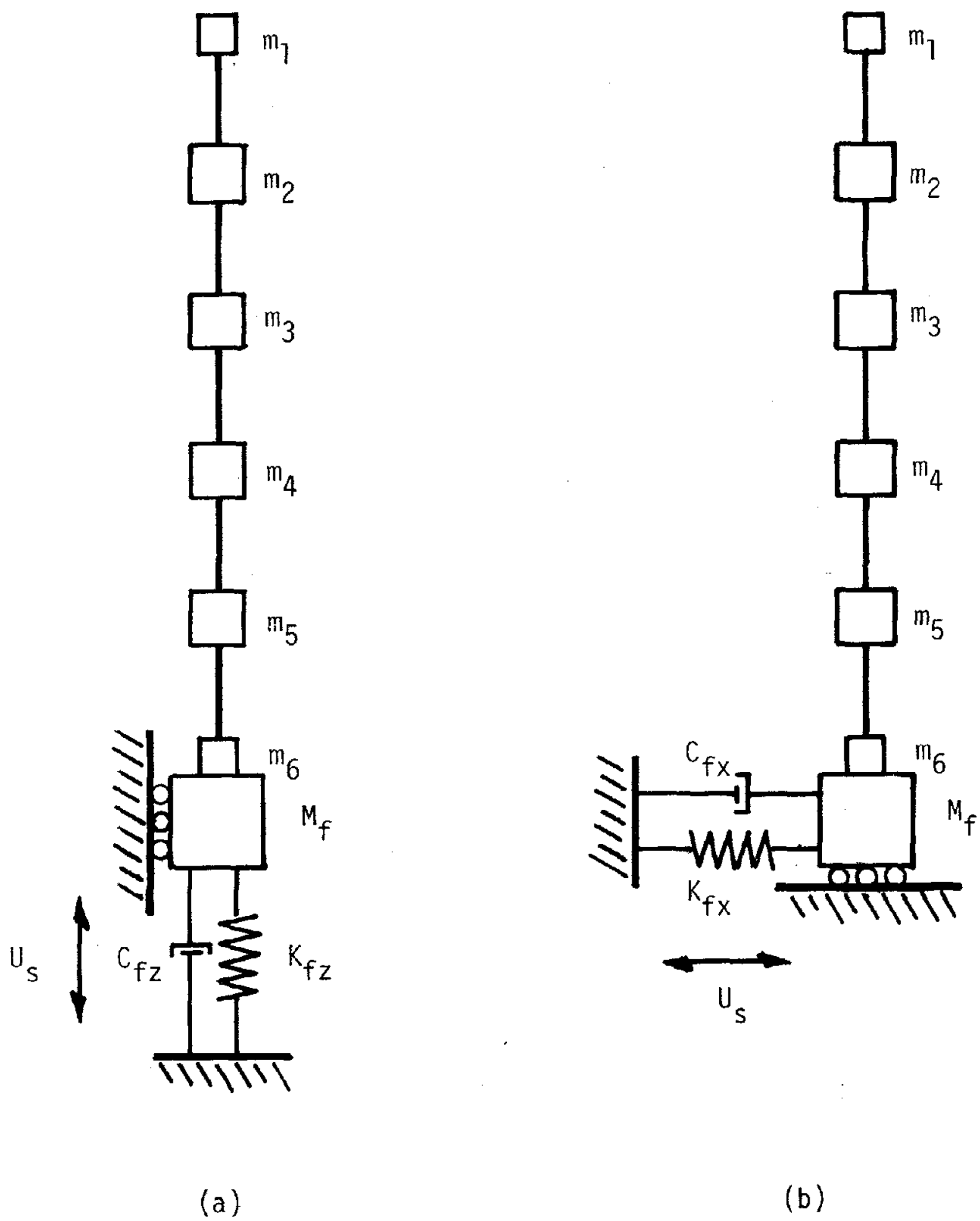


Fig. 2.1 Six Degree-of-Freedom Analytical Model for (a) Vertical and (b) Horizontal Motions

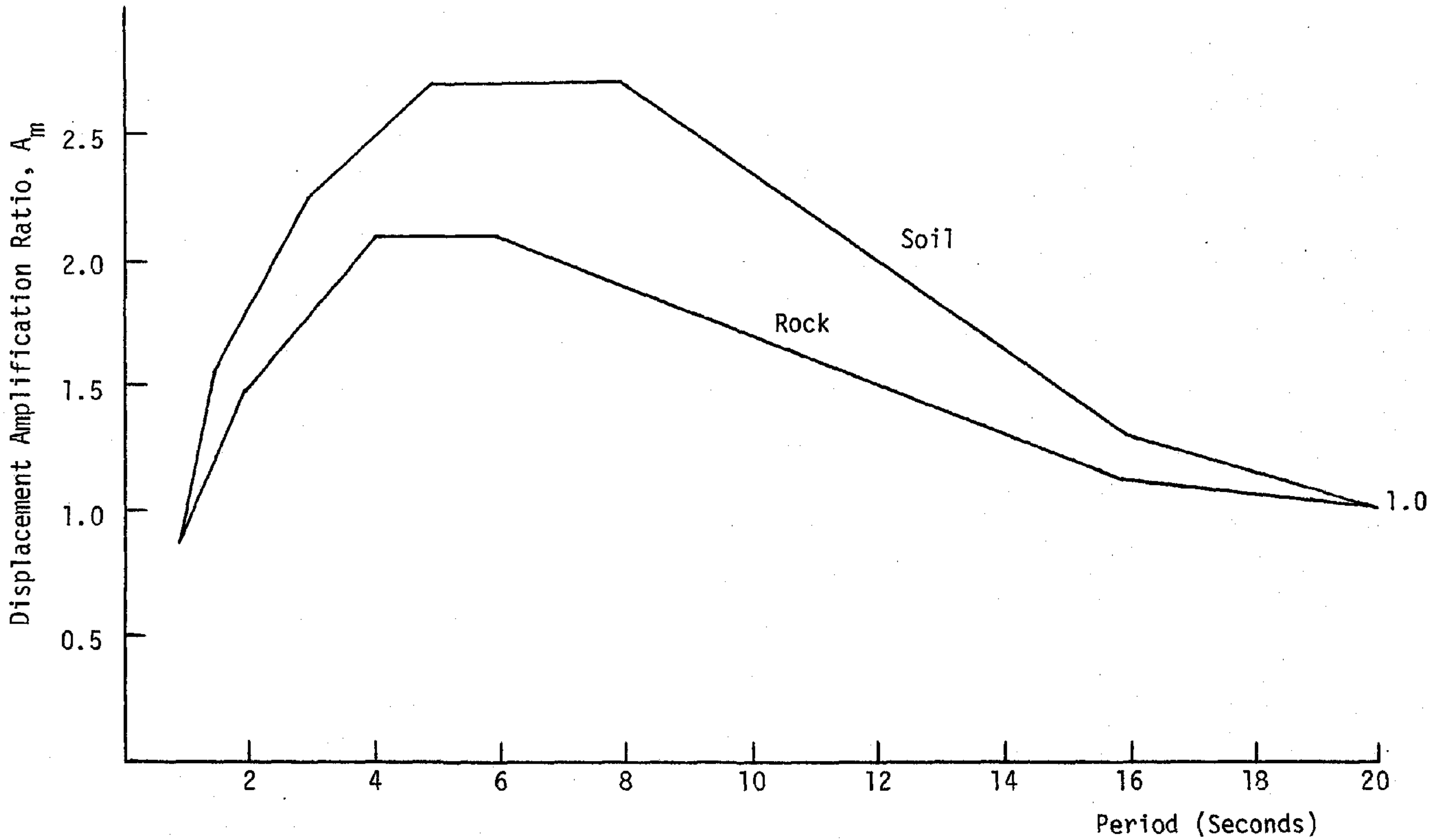


Fig. 2.2. Horizontal Displacement Design Spectrum
0.5% Damping

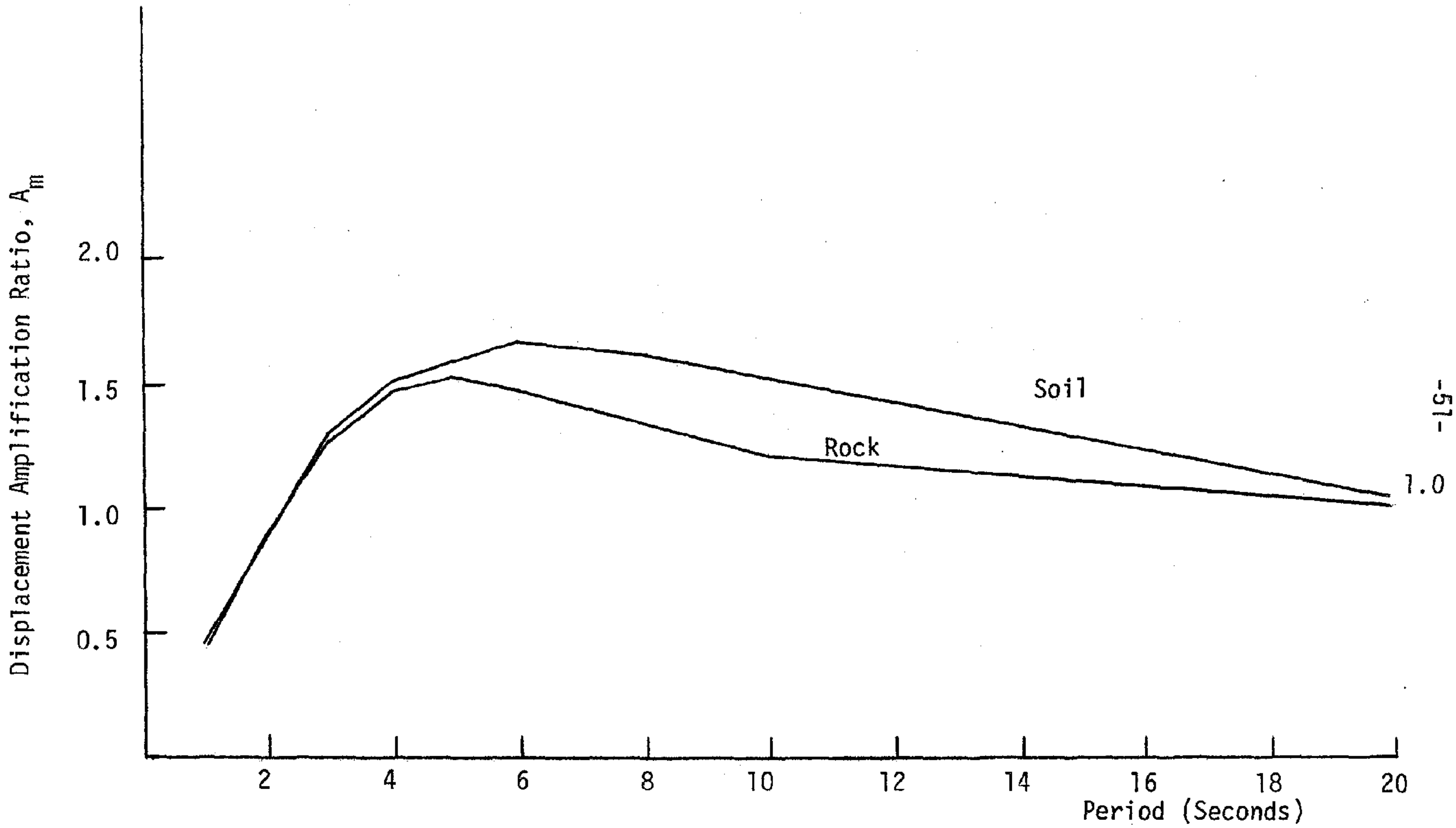
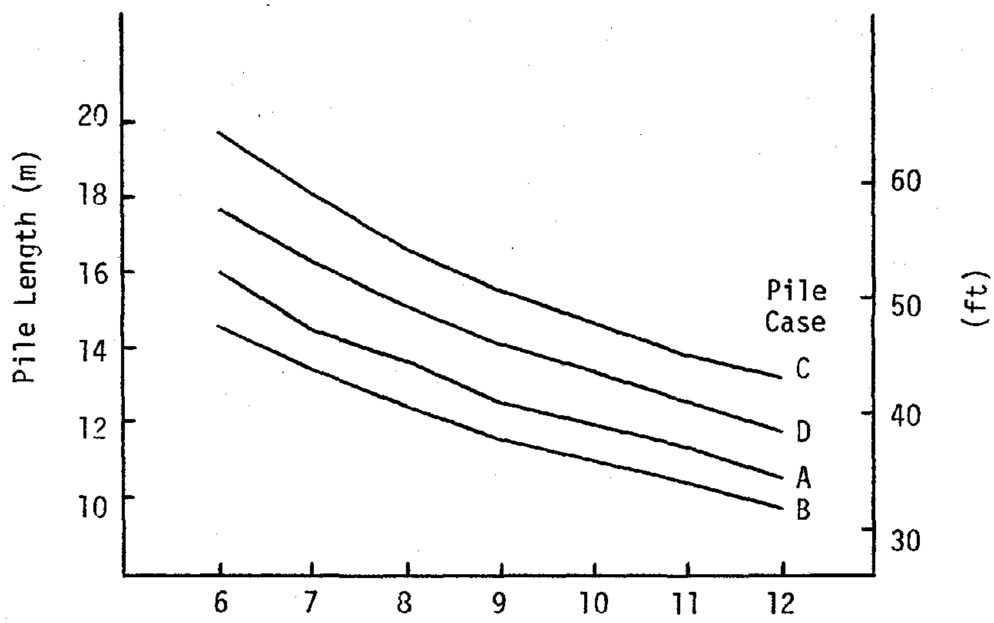


Fig. 2.3 Horizontal Displacement Design Spectrum
10% Damping



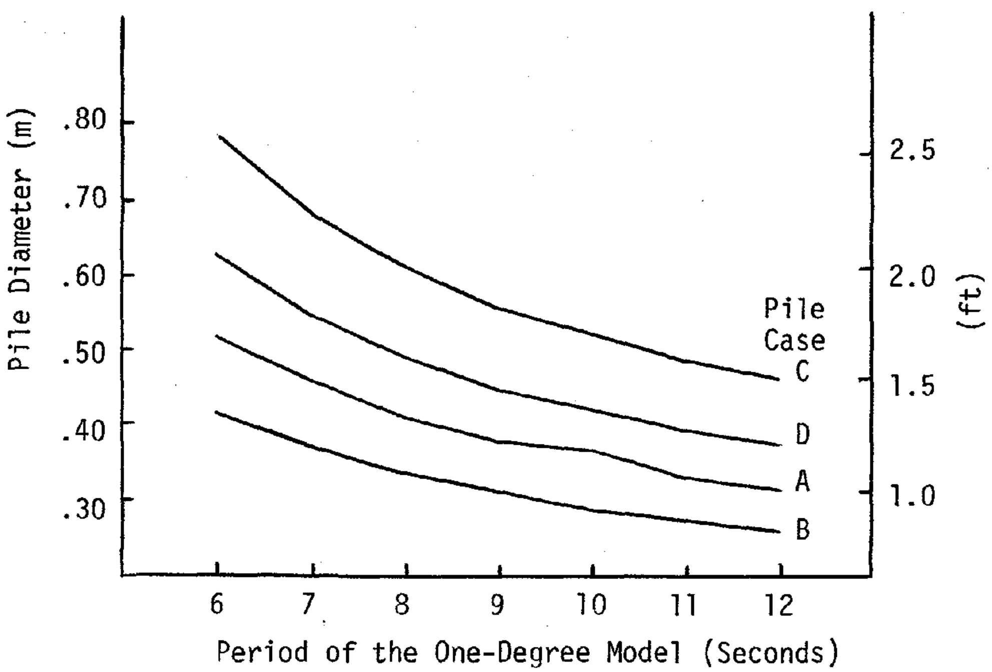


Fig. 2.4 Pile Free Length and Diameter
For $\beta = 0.5\%$, $f_y = 60$ ksi
Rock Base Geologic Site Condition

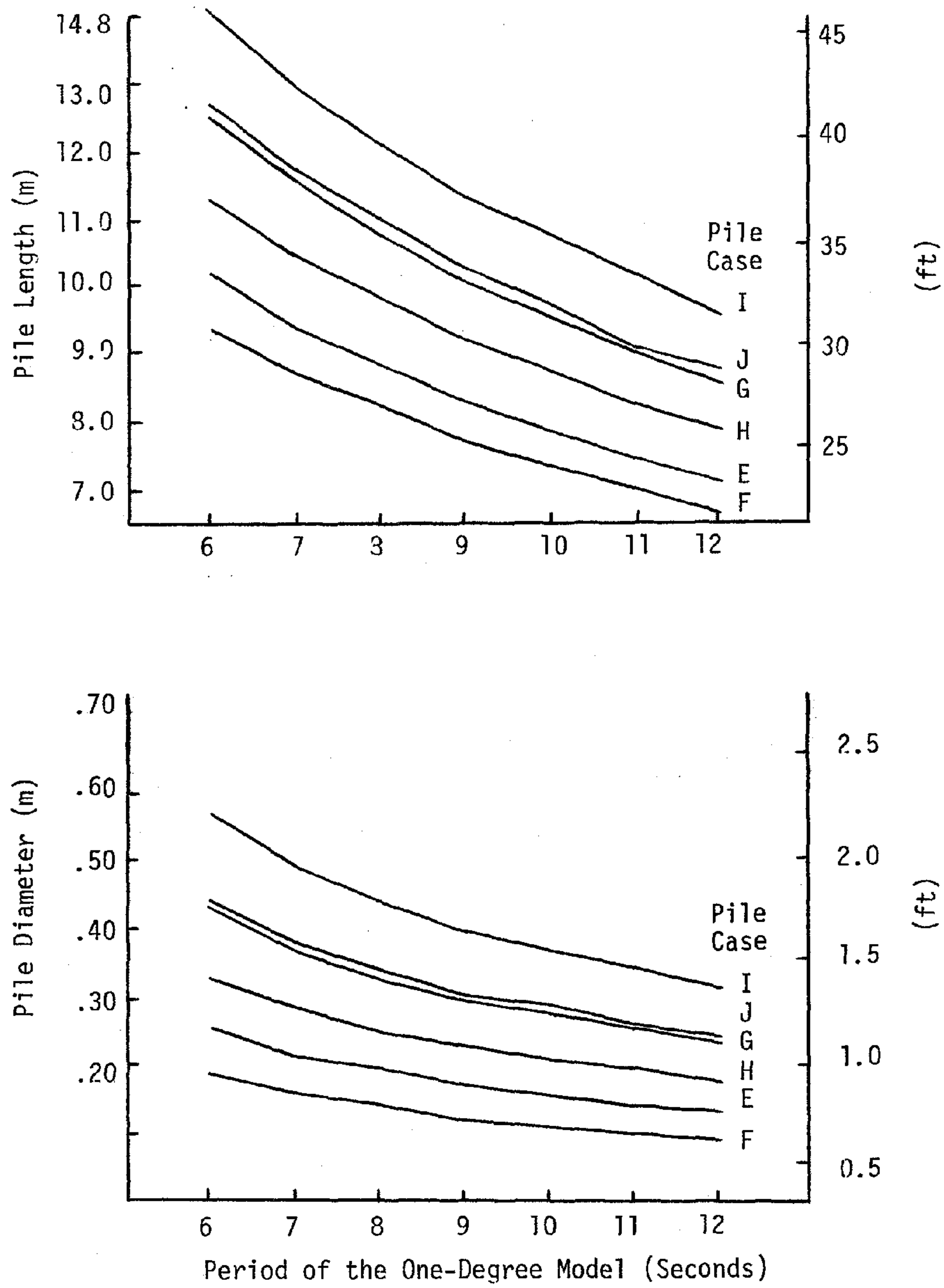


Fig. 2.5 Pile Free Length and Diameter

For β = 0.5 %, f_y = 100 ksi

Rock Base Geologic Site Condition

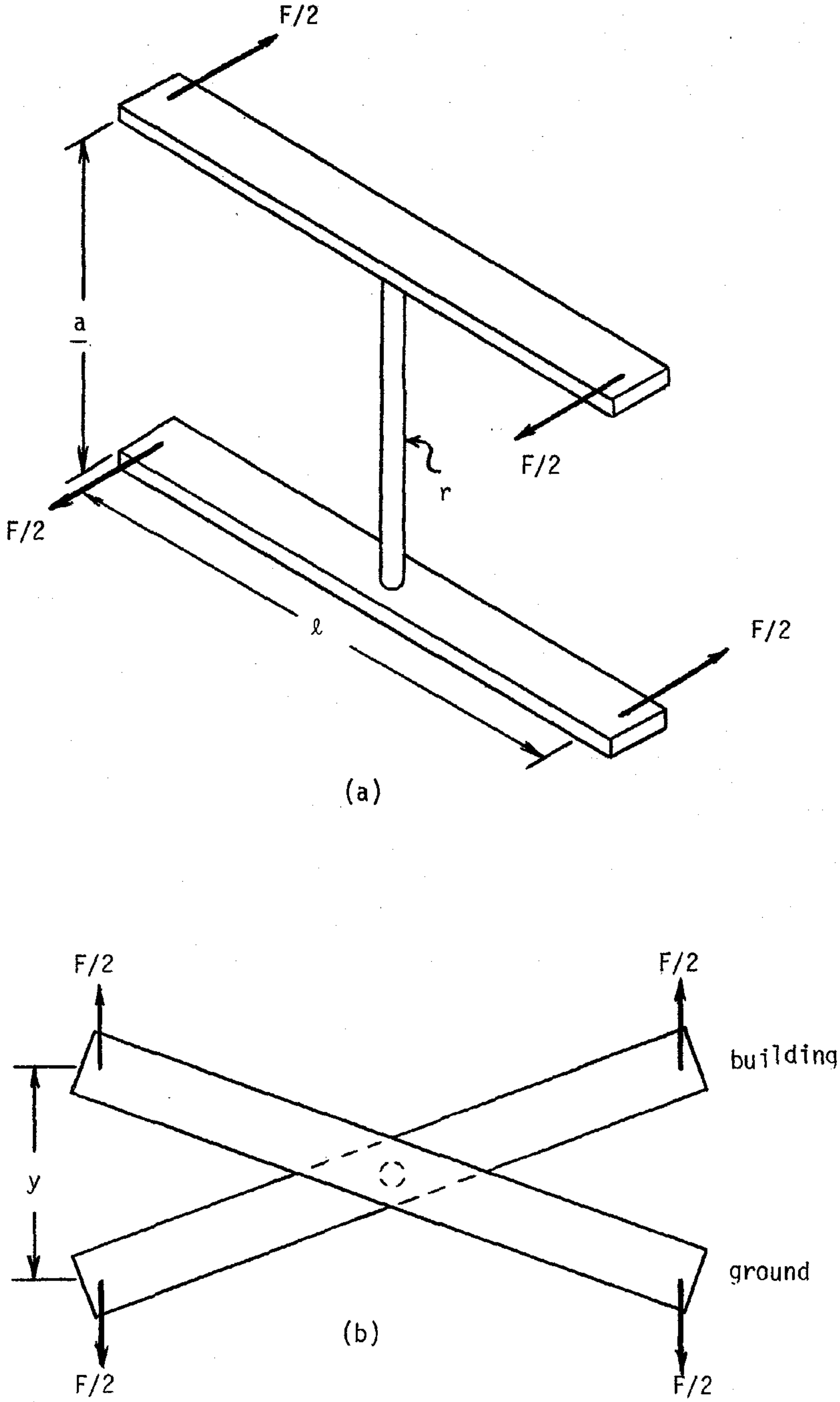


Fig. 2.6 (a) - Isometric View of Damping Mechanism in Undeformed State
(b) - Top View of Damping Mechanism in Deformed State

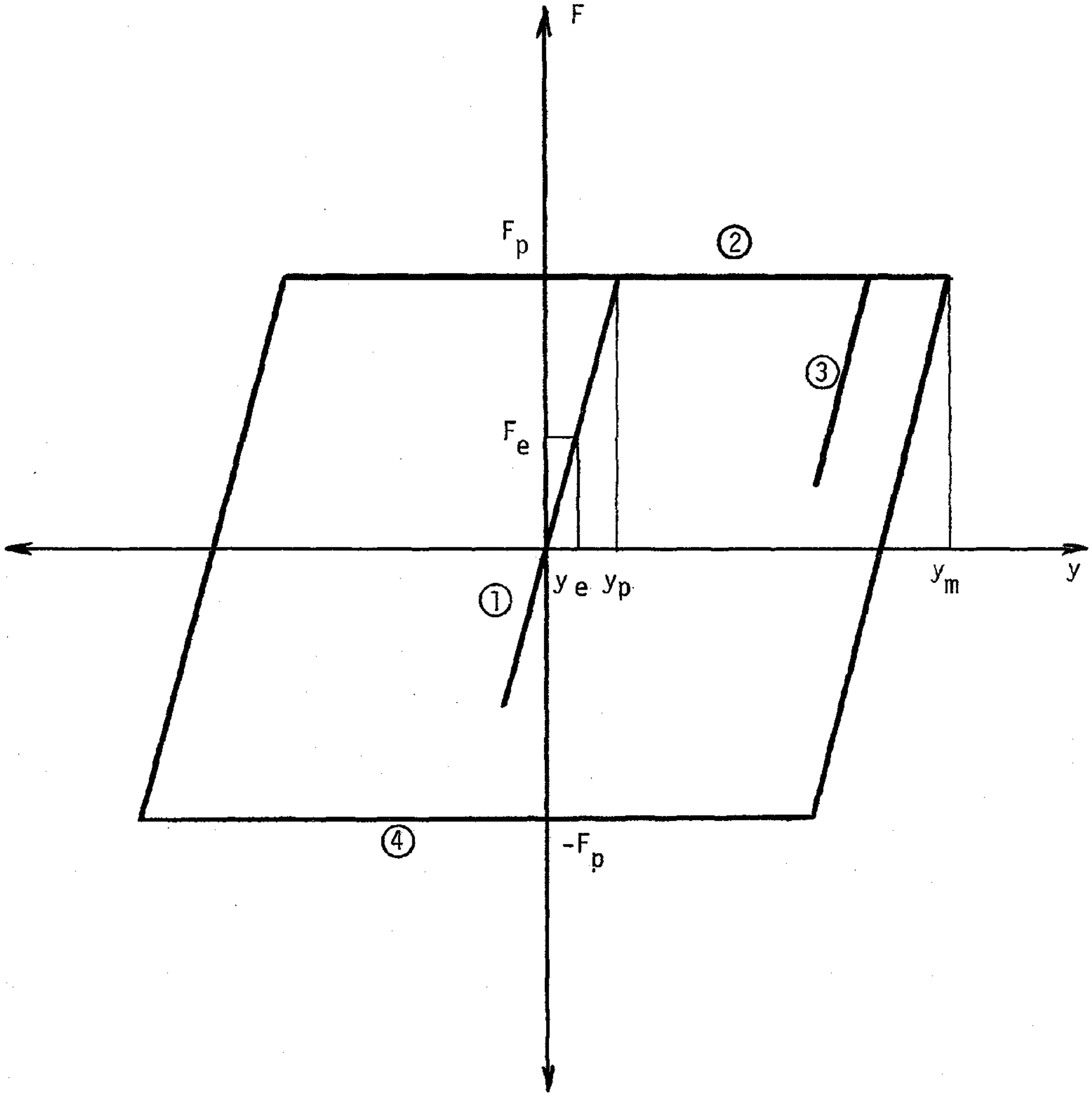


Fig. 2.7 Idealized Version of the Hysteretic Loop Produced by an Actual Device

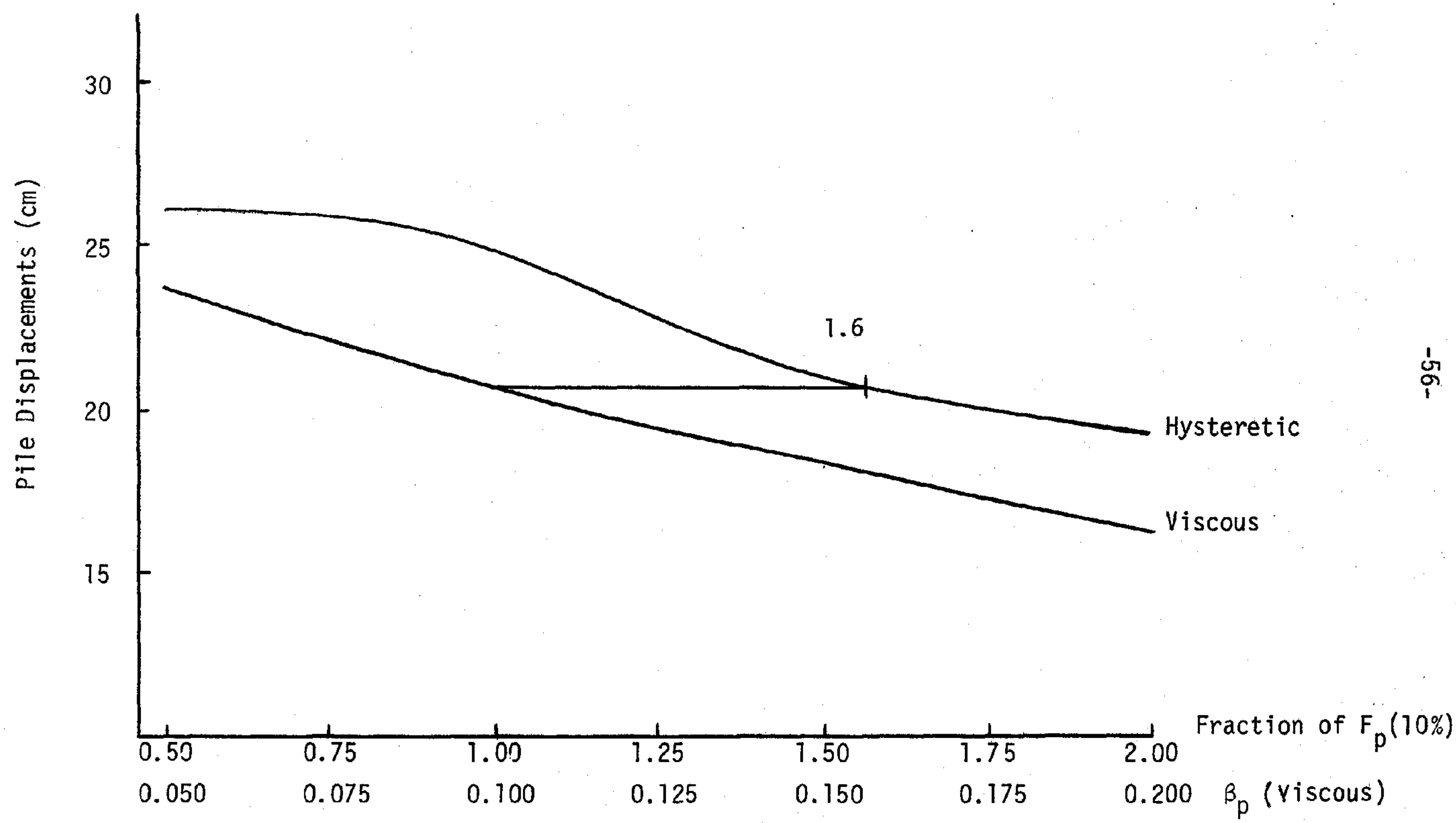


Fig. 2.8 Pile Displacement - Viscous vs. Hysteretic Damping

EQ-4 T_{px} = 12 seconds

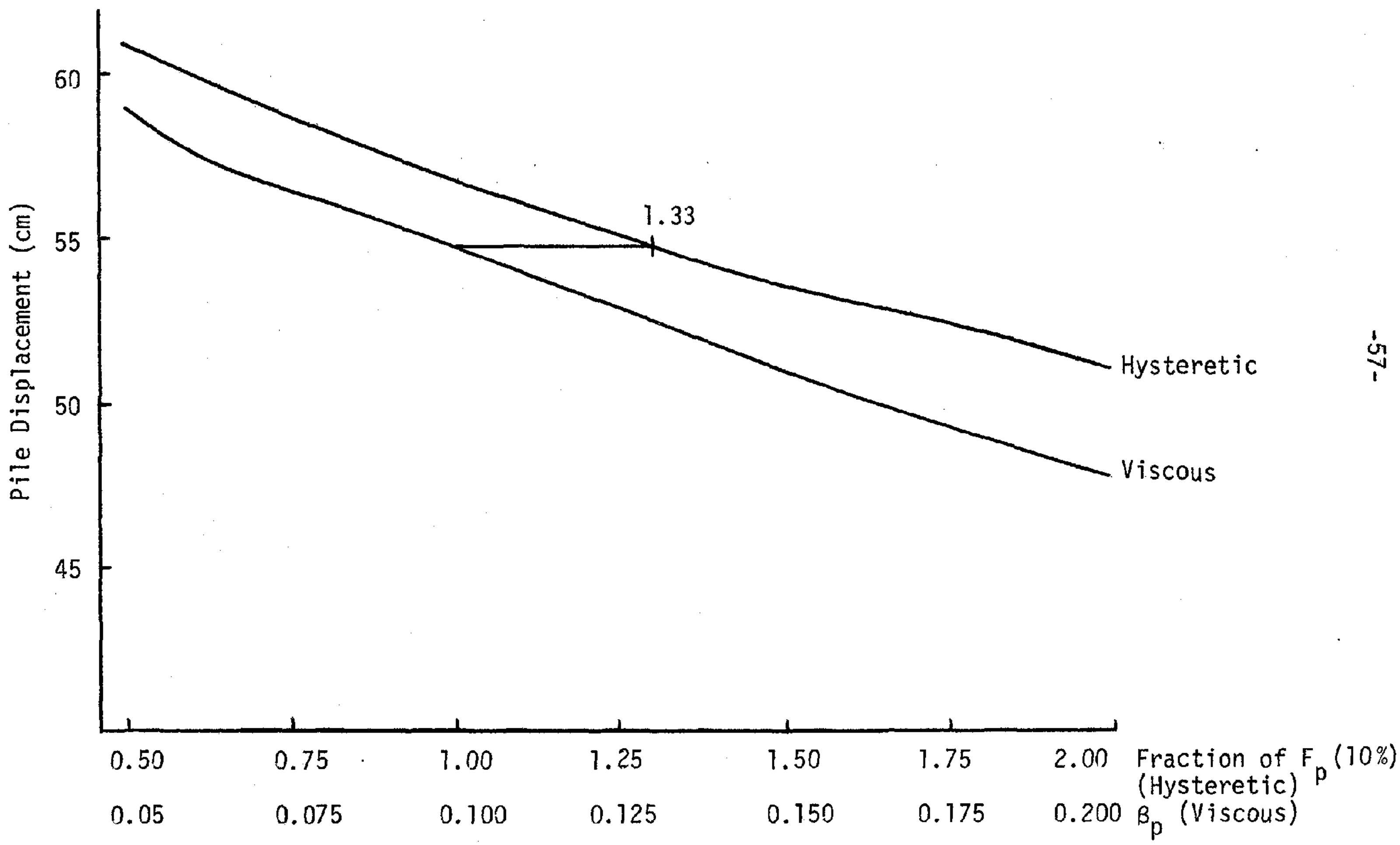


Fig. 2.9 Pile Displacement - Viscous vs. Hysteretic Damping

EQ-2 T_{px} = 10 seconds

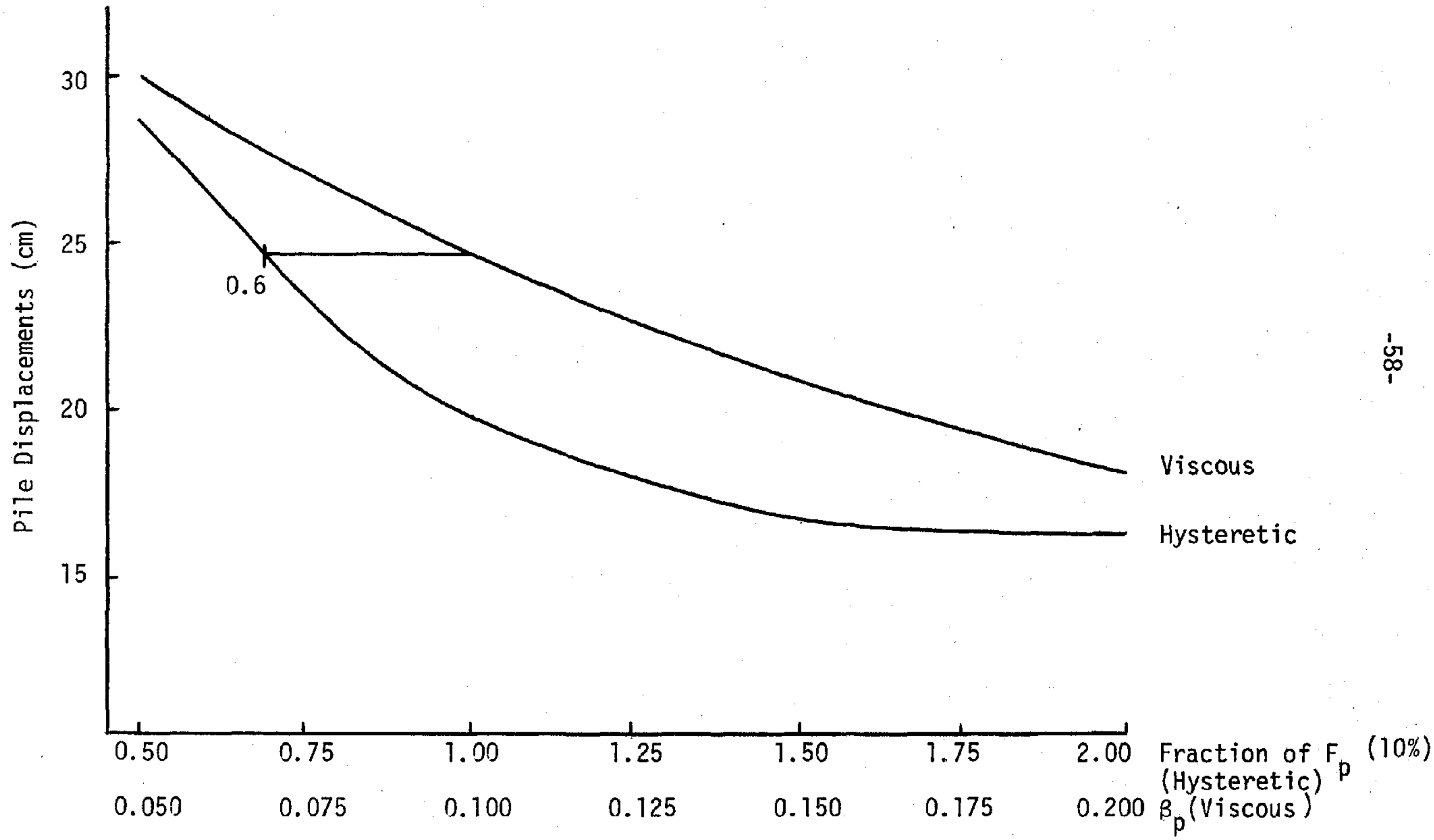


Fig. 2.10 Pile Displacement - Viscous vs. Hysteretic Damping

EQ-4 T_{px} = 10 seconds

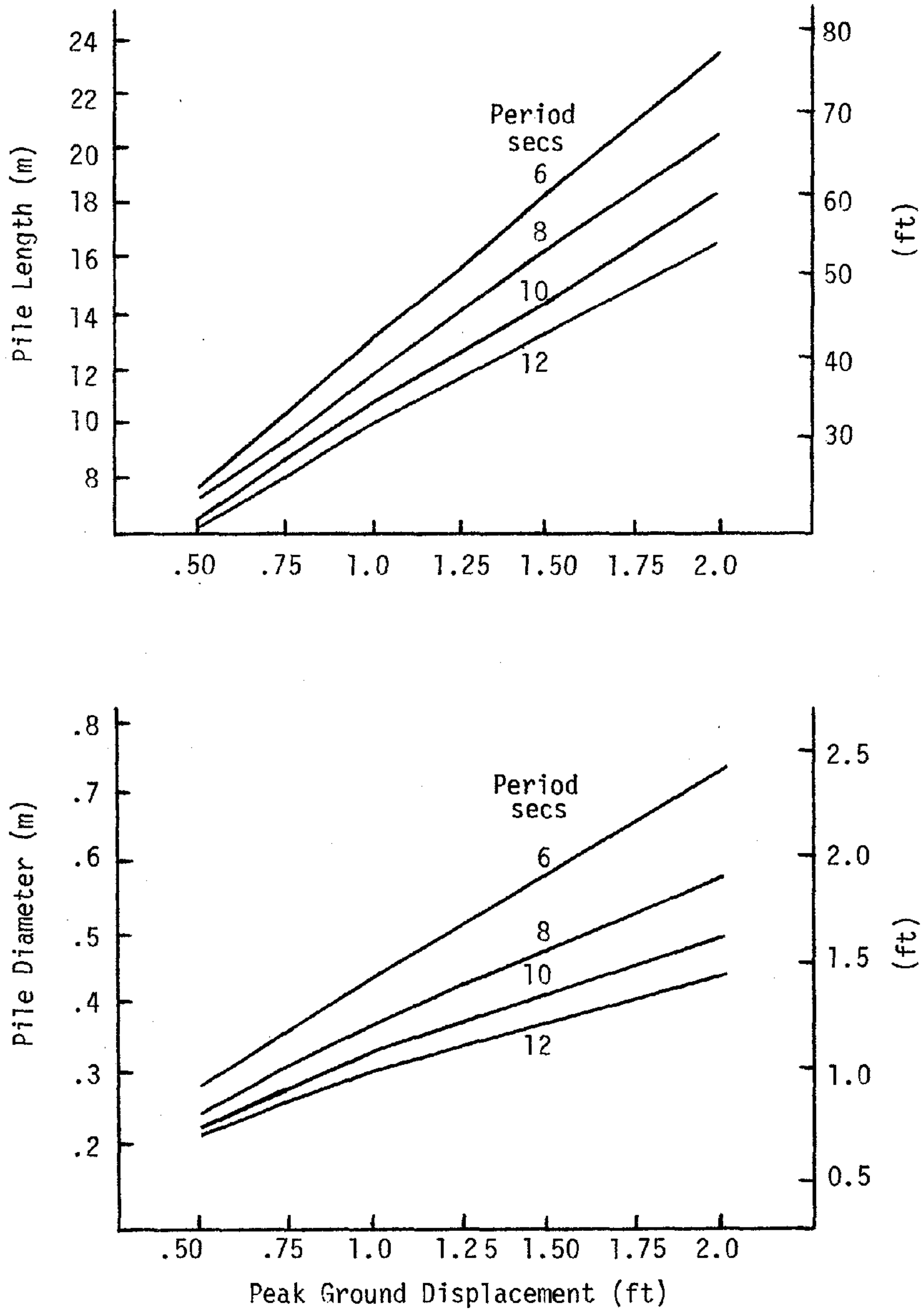


Fig. 2.11 Effects of Peak Ground Displacement on Length and Diameter
For Case A, $\beta = 10\%$
Soft Soil Geologic Site Condition

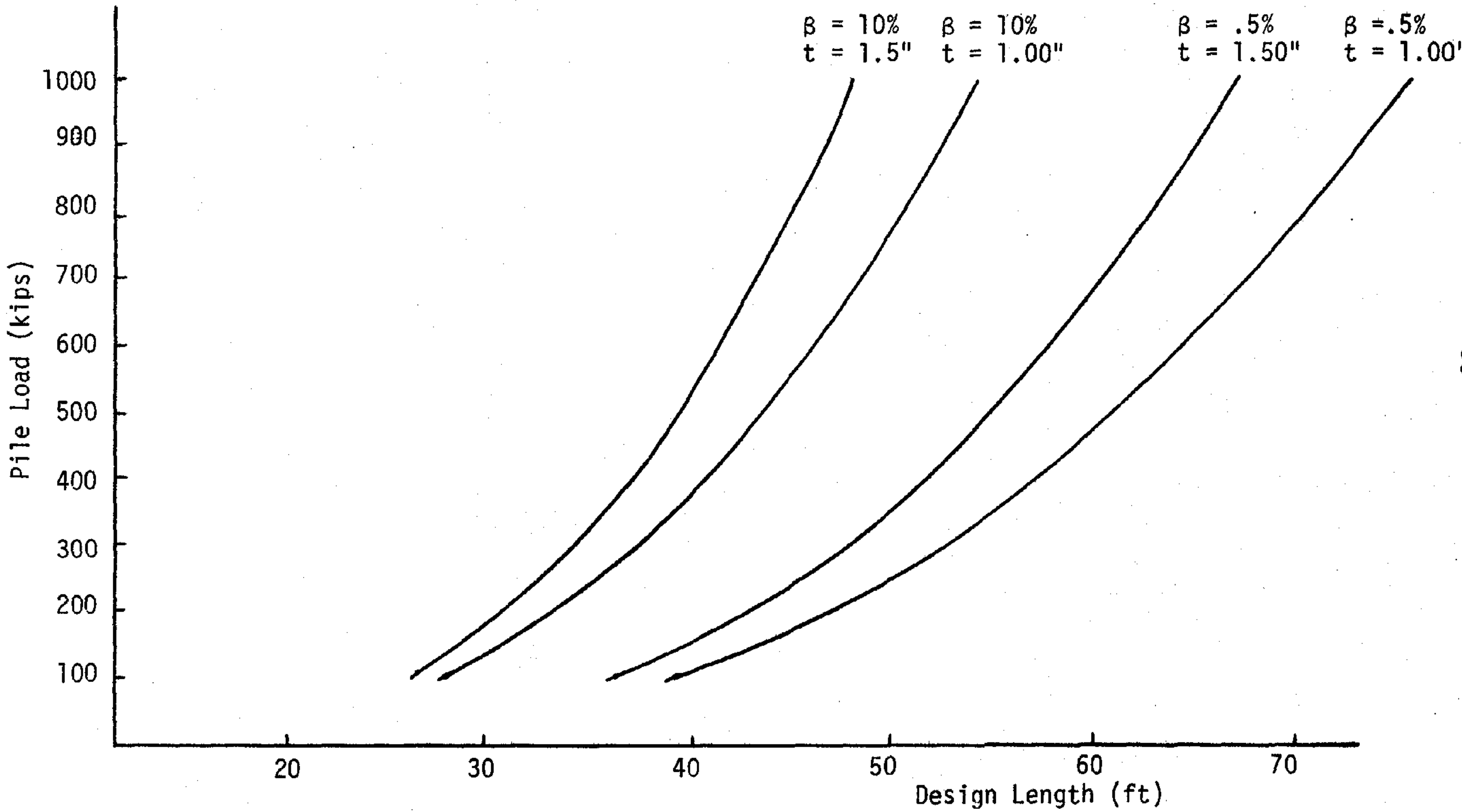


Fig. 3.1(a) - Pile Load vs. Design Length

Soil Site T_p = 10 s

F_y = 60 ksi

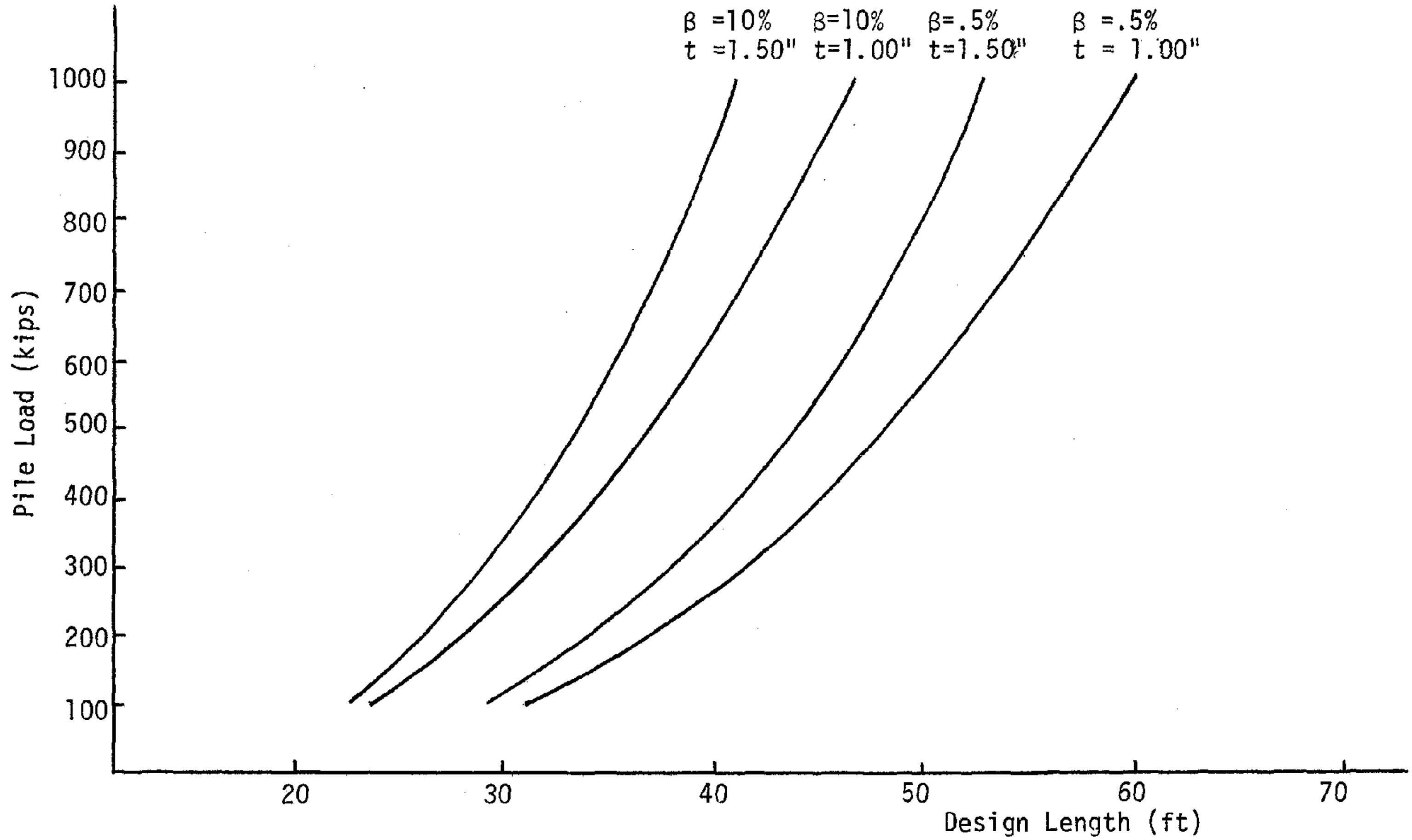


Fig. 3.1(b) - Pile Load vs. Design Length
Rock Site T_p = 10 seconds
F_y = 60 ksi

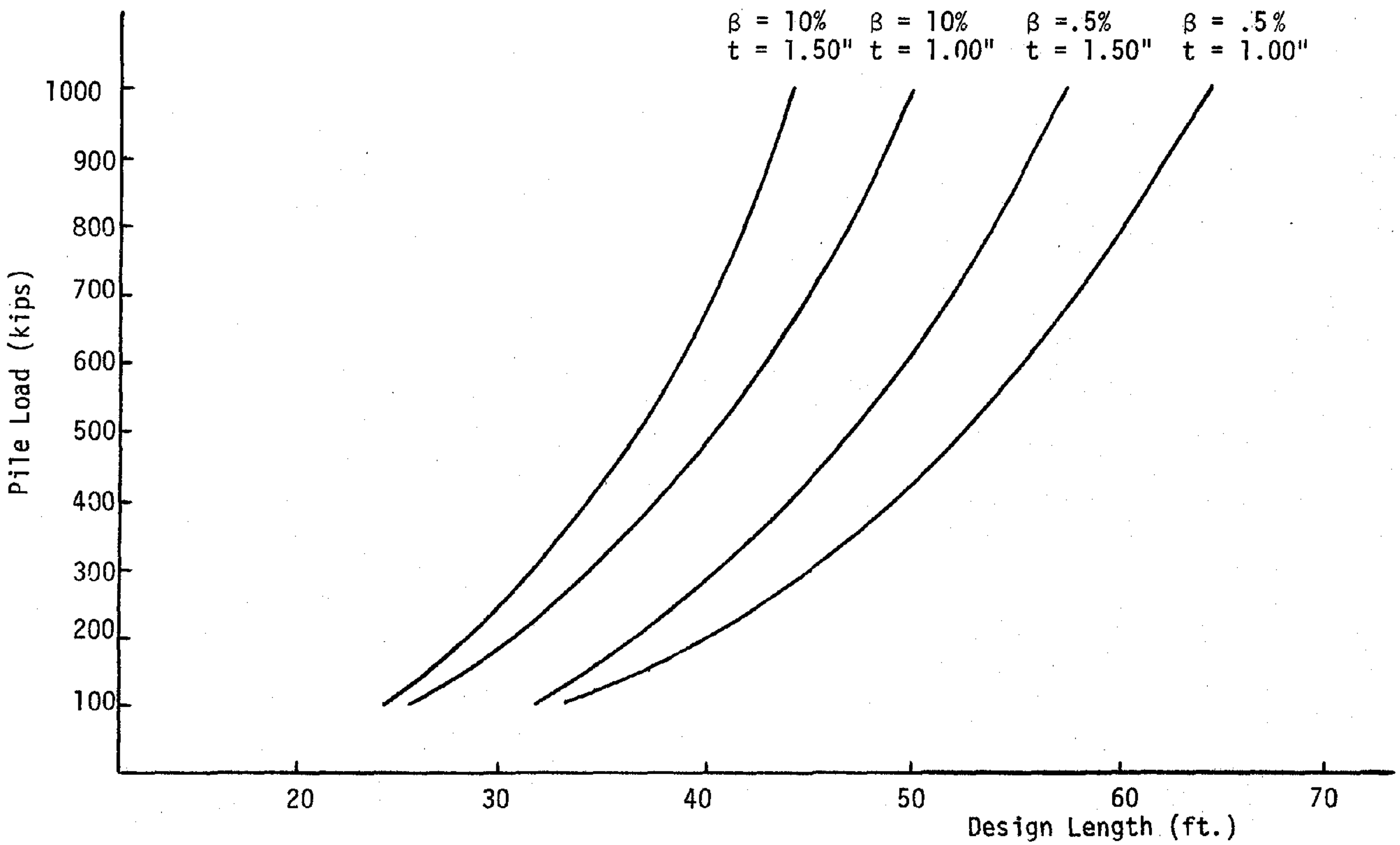


Fig. 3.1(c) - Pile Load vs. Design Length

Soil Site T_p = 12 seconds

F_y = 60 ksi

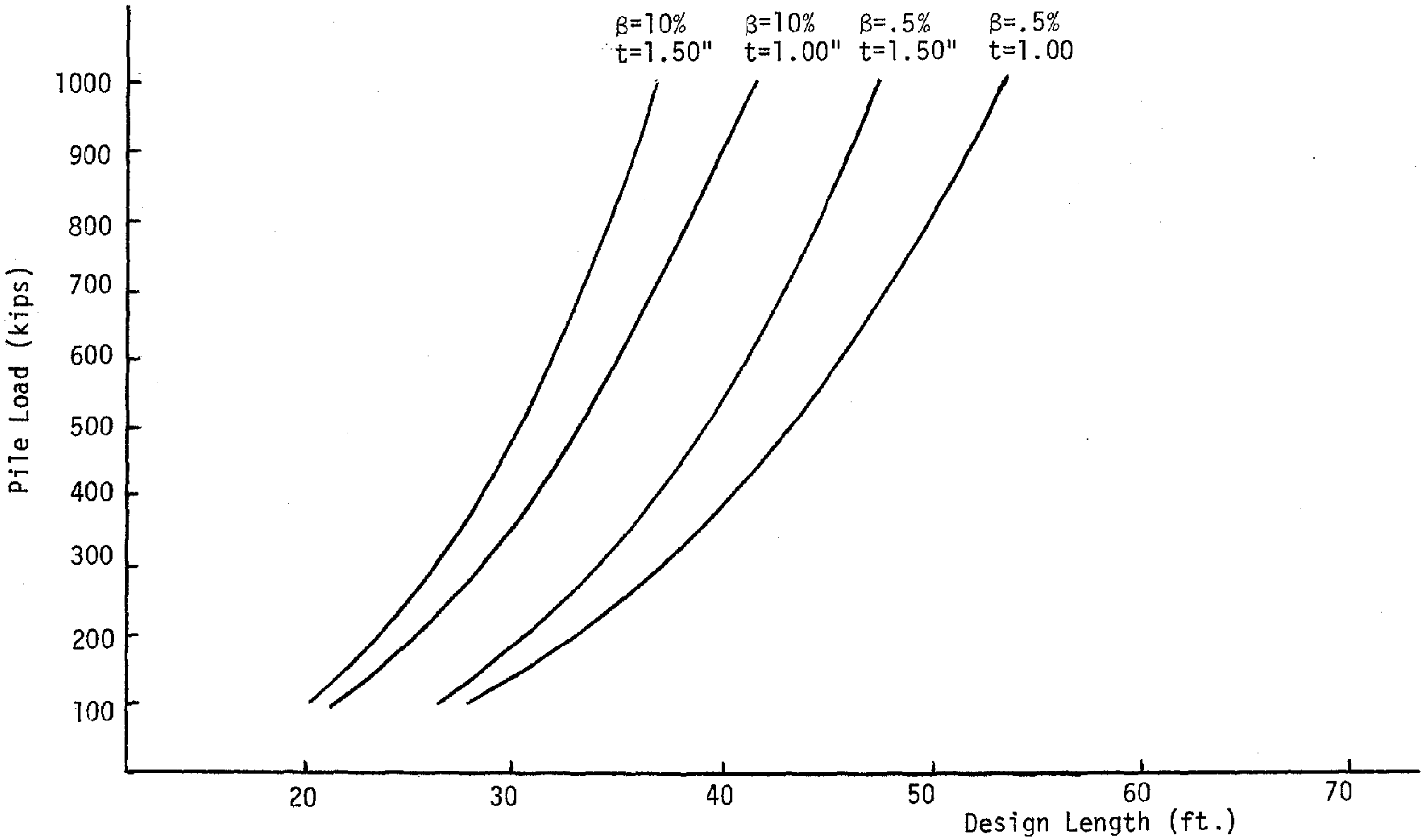


Fig. 3.1(d) - Pile Load vs. Design Length

Rock Site T_p = 12 seconds

F_y = 60 ksi

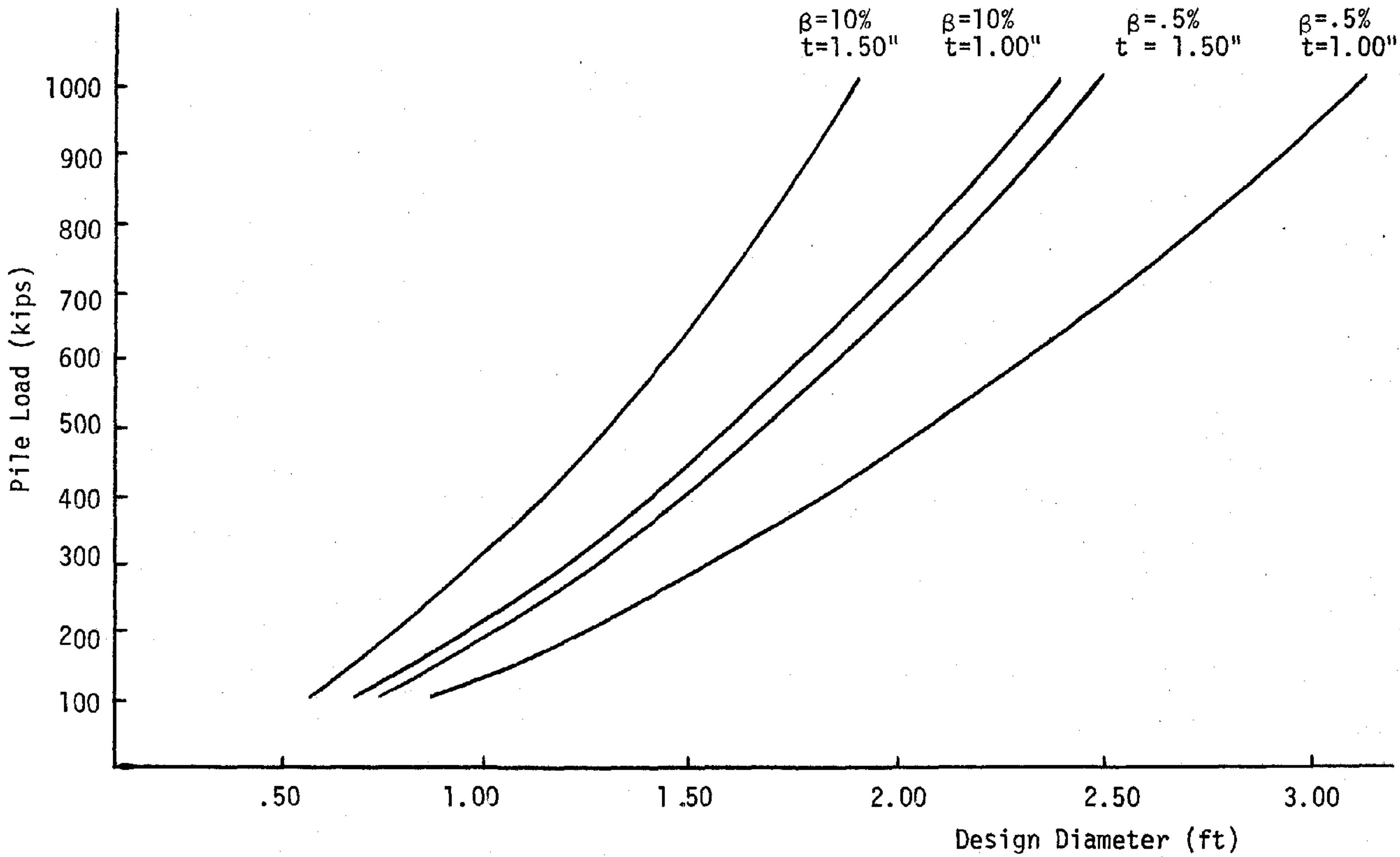


Fig. 3.2(a) - Pile Load vs. Design Diameter

Soil Site T_p = 10 seconds

F_y = 60 ksi

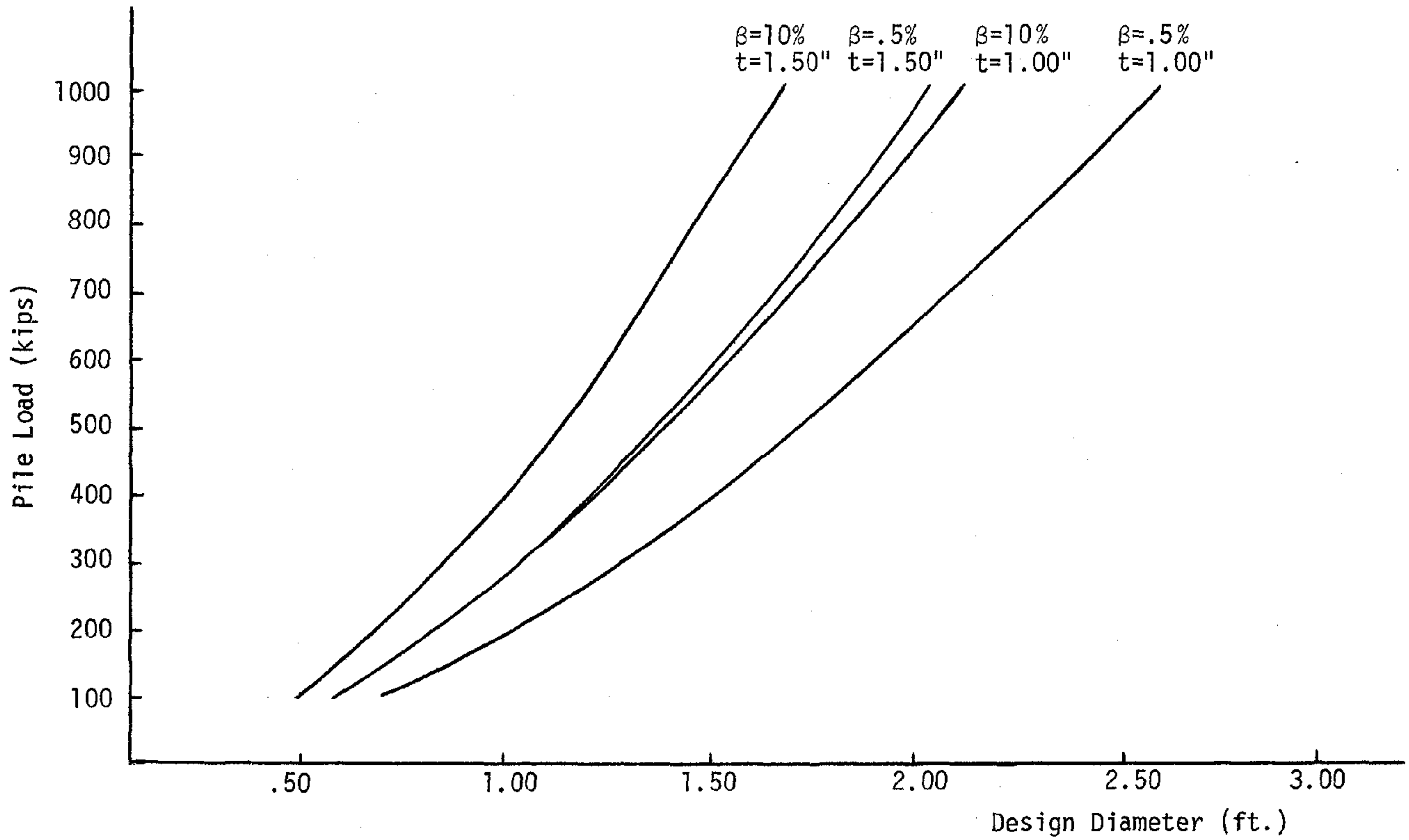


Fig. 3.2(b) - Pile Load vs. Design Diameter
Rock Site T = 10 seconds
F_y = 60 ksi

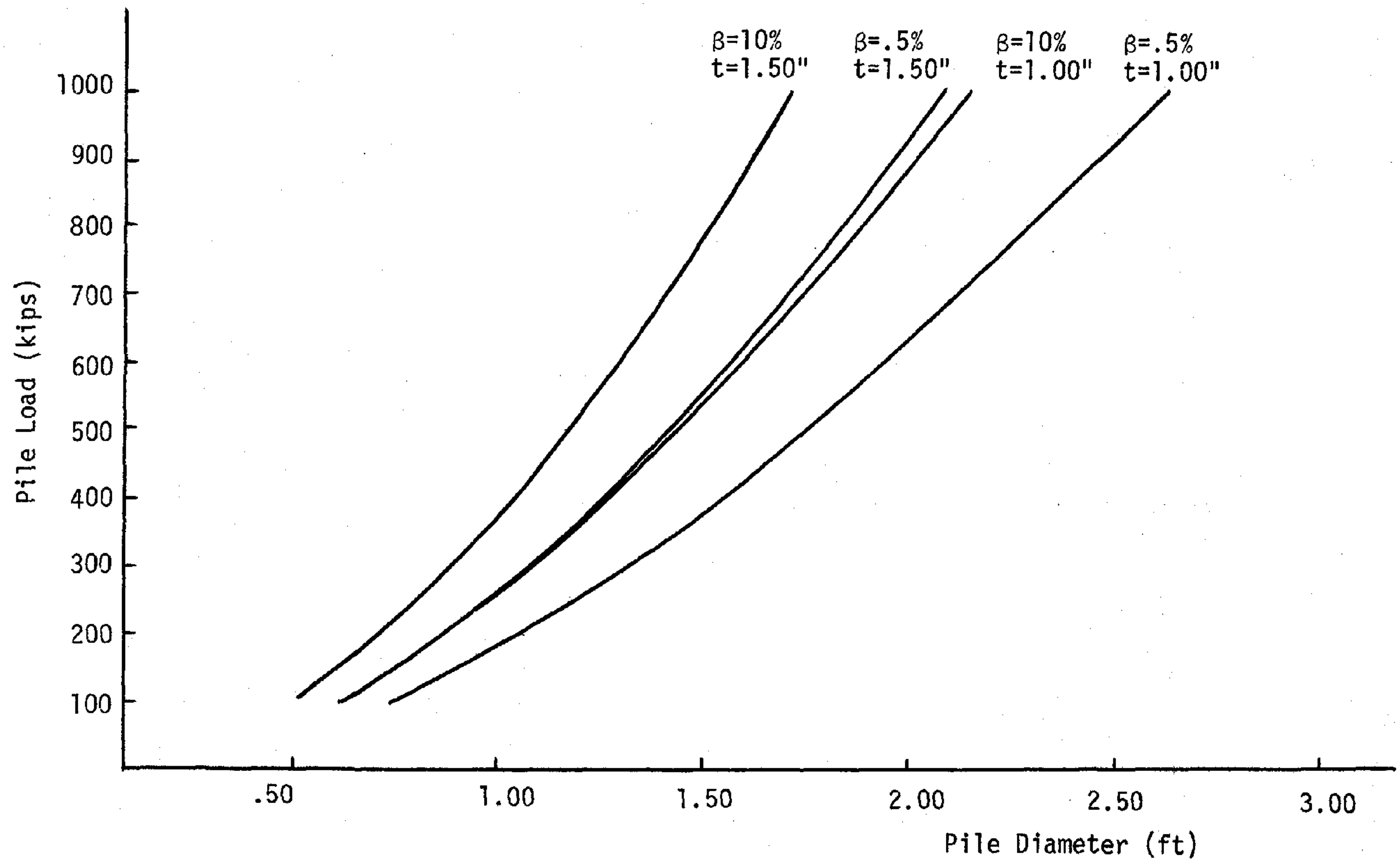


Fig. 3.2(c) - Pile Load vs. Design Diameter

Soil Site T_p = 12 seconds

F_y = 60 ksi

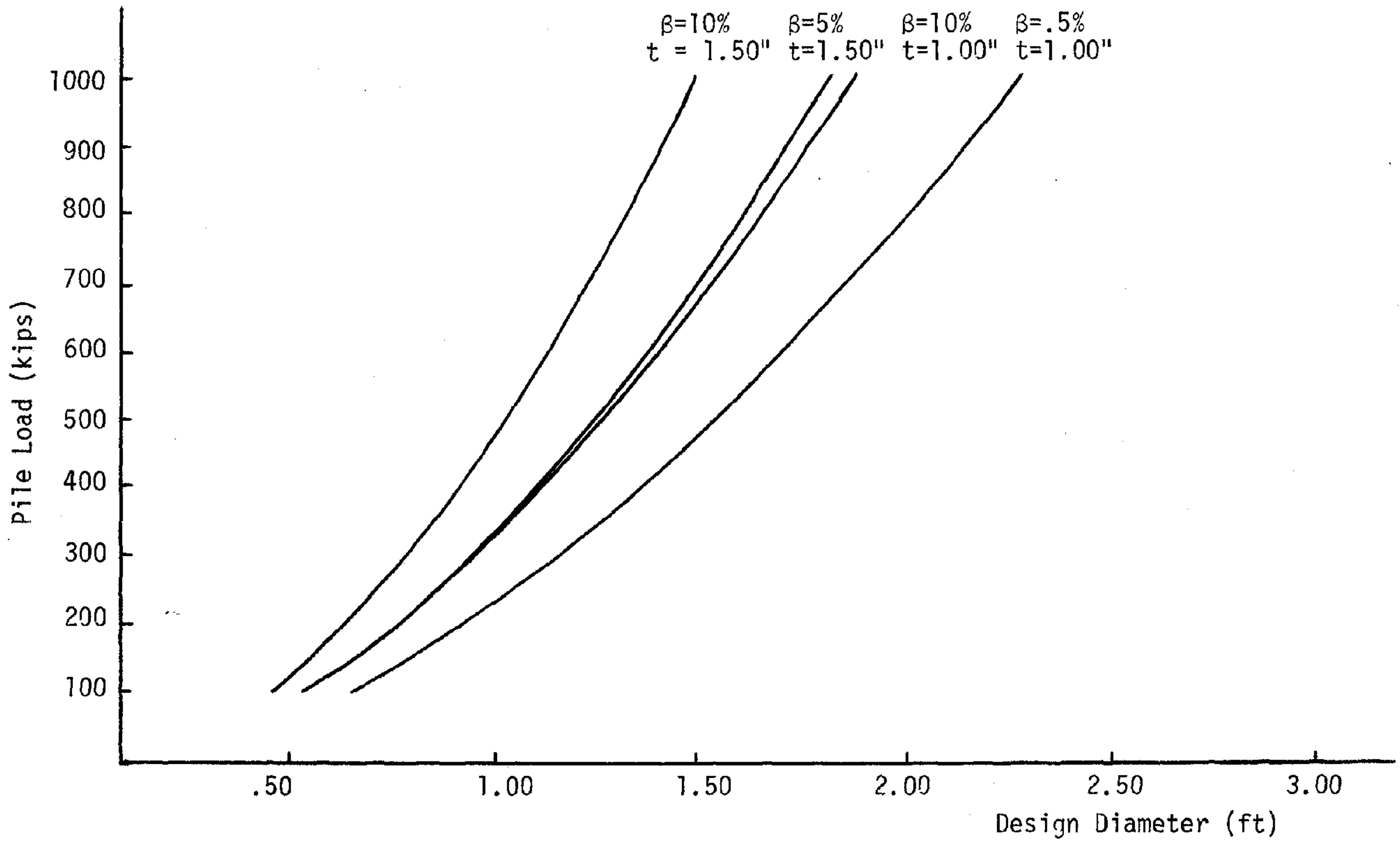


Fig. 3.2(d) - Pile Load vs. Design Diameter

Rock Site T_p = 12 seconds

F_y = 60 ksi