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INFLUENCE OF FOUNDATION MODEL ON THE UPLIFTING OF STRUCTURES

by

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Ewing/Kariotis/Englekirk & Hart **EKEH**

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PREFACE

This report presents the results of Category 2.0, Task 2.3 of the U.S. Coordinated Program for Masonry Building Research. The program constitutes the United States part of the United States-Japan coordinated masonry research program conducted under the auspices of The Panel on Wind and Seismic Effects of the U.S.-Japan Natural Resources Development Program (UJNR).

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INFLUENCE OF FOUNDATION MODEL ON UPLIFTING OF STRUCTURES

By John C. Kariotis[1], Ahmad M. El-Mustapha[2], and Robert D. Ewing[3]

ABSTRACT: The influence of three different foundation models on the dynamic response of uplifting structures is investigated. These three models are an elastic spring with high viscous damping, an elasto-plastic spring with low viscous damping, and an elasto-plastic spring with low viscous damping and impact damping. An algorithm to generate the nonlinear equations of motion within an existing general purpose program is presented. An impact damping mechanism is developed and implemented as part of the numerical scheme presented. A specific foundation model is selected and used in the analysis of two buildings designed according to the NEHRP document. The response of these two buildings with fixed base and uplifting support conditions is also presented.

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INTRODUCTION

It is well known that the seismic design of buildings is accomplished using building code formulas that are based on a generalized nonlinear, fixed base model. These formulas define the required strength of the lateral load resisting system and the required stability to overturning. These formulas are also based on probable ground motions that can be exceeded during the life of the building. The generalized model is appropriate for many buildings. However, foundation flexibility and uplift (i.e. separation between the building foundation and the soil) are important for some types of buildings, specially if the probable ground motions are exceeded. Thus, allowing uplift and soil flexibility could change the fundamental period of the system and lead to significantly different response under seismic forces. This interaction phenomenon between the structure and the foundation is a nonlinear one, due to the uplift of the structure, and requires a numerical solution. This paper presents a model and algorithm that accounts for foundation flexibility and uplift on buildings.

Uplift of some structures has been observed in the Alaska earthquake of 1964 (5), and the Imperial Valley earthquake of 1979 (11). The dynamic response of structures free to uplift on their base has been recently investigated by many researchers. Housner (6) was the first to notice the good behavior of unstable looking inverted pendulum type structures in comparison to more stable looking concrete structures during the Chilean earthquake of 1960. He attributed the good behavior to the effects of impact damping during uplift and recontact. He investigated the rocking response of rigid blocks on rigid foundations and derived an expression for the amplitude-dependent response period. His model also included the energy dissipation associated with the change of the pole of rotation during impact. The problem of rocking rigid blocks on rigid foundations has been investigated experimentally and analytically by several people (1,12). However, these investigations were motivated by the stability of sensitive equipment during earthquakes and do not relate to flexible structures on flexible foundations.

The problem of flexible superstructures was first investigated by Meek (8,9) in his study of tipping core buildings. He concluded that uplift is beneficial, since it reduces the structural deformations and the base overturning moments. Shaking table experiments by Clough and Huckelbridge (2,7) on building frames that were allowed to uplift show a significant reduction in the ductility demand on the frame, as compared to fixed base frames. They also concluded that allowing uplift enhances the chances of survival under a severe ground motion. However, the flexibility of the soil was not included in the experiment or in the analysis. The finite element method was used by Wolf (13,14) to investigate the uplift of nuclear reactor structures on flexible foundations. In this study, reductions in overturning moments, structural deformations, and strength requirements were observed due to the uplift when compared to the case where the superstructure is fully bonded to the flexible foundation. Conclusions were made in favor of allowing uplift instead of preventing it.

Psycharis (11) was the first to investigate the problem using a closed form analytical solution. The problem as a whole is a nonlinear one; however, it was broken down into a series of linear problems with coupled equations of motions. He investigated the rocking of rigid blocks on rigid foundations and compared his results to Housner's (6). He also derived the equations of motion for a rigid structure on a flexible foundation and for a flexible structure on a flexible foundation. Two foundation models were used, a two-spring system and a Winkler system. An equivalence between the two-spring foundation and the Winkler foundation was developed for linear elastic superstructure and foundation elements. In this work, he derived the equations of motion for a multi-degree of freedom flexible shear-type superstructure that was used in the analysis. In his study, three models were suggested to represent the energy dissipation mechanism in the soil foundation; namely, elastic spring with viscous damping, elasto-plastic spring element, and an impact damper which can be associated with either one of the above mentioned elements. However, the elasto-plastic spring element was not investigated due to the complexity of obtaining an analytical solution for the problem. He concluded that an elastic spring with viscous damping is suitable for the analysis. However, an equivalence between the two systems was not presented. Psycharis concluded that it is not appropriate to make general statements about the beneficial effects of uplift, since these effects depend on the parameters of the problem.

Yim and Chopra (15,16) extended Psycharis's work and investigated the flexible superstructure, both on the two-spring foundation and on the Winkler foundation. Elastic spring elements with high viscous damping were used in the analysis. They presented a base shear response spectrum for the El Centro 1940 ground motion and concluded that uplift is beneficial and should not be prevented. However, in a discussion by Meek (10), he pointed out that the superstructure aspect ratio was kept constant in those response spectrum. The aspect ratio is an important parameter that can alter the response significantly when soil flexibility and uplift are included in the analysis. Later on in their work, they developed a simplified analysis procedure for multi-story superstructures allowed to uplift.

Several factors of importance have not been considered in the previous work of other researchers. Some of these factors are the nonlinearity of the superstructure, the nonlinearity of the foundation, impact damping effects, and the characteristics of the ground motion. The uplift phenomenon has the tendency to elongate the fundamental response period. Thus, the frequency content of the ground motion could play an important role on the dynamic response of the system. When uplift of the superstructure is allowed and the soil is recontacted by the superstructure, energy is dissipated in the soil by material damping, radiation damping, and impact damping. To investigate the influence of different foundation models on the dynamic response, three systems are considered herein: elastic spring with high viscous damping, elasto-plastic spring with low viscous damping, and elasto-plastic spring with low viscous damping and impact damping. The elasto-plastic spring is used to model the soil material behavior. The impact damping is used to model energy dissipation due to inelasticity associated with the change of the pole of rotation. The low viscous damping is used to insure minimum energy dissipation at low strains in the case where yielding in the spring element does not occur. It is also a representation of radiation damping in the large volume of soil remaining elastic. The model presented in this paper is implemented as part of a general purpose program (3) where a change of element material behavior can be easily represented.

ANALYSIS PROCEDURE

Consider a superstructure idealized as a single degree of freedom resting on foundation springs, as shown in Fig. 1(a). The structure can displace horizontally, vertically, and it can rotate about the center point of contact with the foundation. In this model, the equations of motion are generated using a compatibility approach. A compatibility matrix relating internal spring deformations and external absolute degrees of freedom is used to generate those equations. As an example of how these equations are generated, consider the case of a single degree of freedom superstructure resting on a two-spring foundation as represented in Fig. 1(a). This system has three unknown absolute degrees of freedom and one known degree of freedom and they are shown graphically in Fig. 1(a). A local stiffness matrix is diagonal and takes the following form:

$[S] = [k_i]$; $i = 1,2,3$..(1)

Where k_1, k_2, and k_3 are the stiffness values of spring elements 1, 2, and 3 as shown in Fig. 1(a). The external degrees of freedom vector is defined as follows:

$\{U\} = \{X_g, X, \theta, V\}$..(2)

Where $X_g(t)$ is the displacement of the ground, $X(t)$ is the absolute displacement of the superstructure, $\theta(t)$ is the rotation angle, and $V(t)$ is the vertical displacement at the center point of contact between the superstructure and foundation. The compatibility matrix, [A], relating the external absolute degrees of freedom and the compatibility matrix, U, to internal deformation of springs 1, 2, and 3 is expressed as $\{e\} = [A]^T\{U\}$, takes the following form:

$$[A] = \begin{bmatrix} 0 & 0 & +1 \\ 0 & 0 & -1 \\ -b & +b & h \\ -1 & -1 & 0 \end{bmatrix} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Where h is the height of the structure, b is the distance between the vertical centerline and the edge foundation spring as shown in Fig. 1(a). Internal spring deformations $\{e\}$ are related to external degree of freedom through the relation $\{e\} = [A]^T \{U\}$. Since absolute degrees of freedom are used, with respect to a fixed reference axis, mass coupling does not occur and the mass matrix can be defined as follows:

$$[M] = \begin{bmatrix} 0 & & & \\ & m & & \\ & & I_o & \\ & & & m \end{bmatrix} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Where m is the mass of the superstructure and I_o is the mass moment of inertia about point 0. Thus, the equations of motion for the system considered with undamped elements can be written as follows:

$$[M]\ \{\ddot{U}(t)\} + [K]\ \{U(t)\} = \{0\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Where [M] is the diagonal mass matrix, and [K] is a global stiffness matrix computed as $[A]\ [S]\ [A]^T$. The above system of equations contains an equation for the known degree of freedom X_g that is eliminated from the computation process. Elimination of the first equation for X_g and calculation of the expression for the global stiffness matrix leads to the following three equations for the degrees of freedom X, θ and V, respectively

$$m\,\ddot{X}(t) - k_3\,X_g(t) + k_3\,X(t) - h\,k_3\,\theta(t) = 0 \qquad (6a)$$

$$I_o\,\ddot{\theta}(t) - h\,k_3\,(X(t) - X_g(t) - h\theta(t)) + b^2(k_1 + k_2)\,\theta(t) + b\,(k_1 - k_2)\,V(t) = 0 \qquad (6b)$$

$$m\,\ddot{V}(t) + b(k_1 - k_2)\,\theta(t) + (k_1 + k_2)\,V(t) = 0 \qquad (6c)$$

The above equations are for the case where the foundation has a negligible mass in comparison to the superstructure mass. The applied excitations are implicit in those equations, since $X_g(t)$ is a known displacement function. The weight does not appear in the equation for V(t) since its reference axis is the static equilibrium position. Internal deformation in the superstructure can be calculated using the following equation:

$$u(t) = X(t) - X_g(t) - h\theta(t) \qquad (7)$$

Where u(t) is the internal deformation in the superstructure relative to the rotated position. Computing $\ddot{X}(t)$, X(t) from Eq. 7 and substituting into Eq. 6 yield the following equations:

$$m\ddot{u}(t) + m(h\ddot{\theta}(t)) + k_3 u(t) = m\ddot{X}_g(t) \quad \text{(8a)}$$

$$I_o\ddot{\theta}(t) + M_f - k_3 h\ u(t) = 0 \quad \text{(8b)}$$

$$m\ddot{V}(t) - F_f = 0 \quad \text{(8c)}$$

where —

$$M_f = b^2(k_1 + k_2)\theta(t) + b(k_1 - k_2)\ V(t) \quad \text{(8d)}$$

$$F_f = b(k_1 - k_2)\ \theta(t) + (k_1 + k_2)\ V(t) \quad \text{(8e)}$$

The above equations are the same as those derived by Yim and Chopra (15) and by Psycharis (11). The problem considered illustrates how the equations of motion can be generated. We will consider the case when the superstructure rests on a Winkler Foundation. Since it will be used later in the analysis, a schematic description is shown in Fig. 1(b), and the compatibility matrix takes the following form:

$$[A] = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & +1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1 & 0 \\ -b_1 & -b_2 & -b_3 & -b_4 & -b_5 & -b_6 & b_7 & b_8 & b_9 & b_{10} & b_{11} & b_{12} & h & +1 \\ -1 & -1 & -1 & -1 & -1 & -1 & -1 & -1 & -1 & -1 & -1 & -1 & 0 & 0 \end{bmatrix} . \quad \text{(9)}$$

Where b_i is the distance between element i and center point 0. The generation of the equations of motion are identical to the two-spring case considered earlier.

In this procedure, element nonlinearities can be included and the equations of motion are solved using a step-by-step integration.

An explicit conditionally stable scheme using the Runge Kutta and a predictor-corrector method is used. Loading and unloading are also incorporated into the elasto-plastic foundation element as will be shown later in the analysis.

IMPACT DAMPING

Energy is dissipated due to the inelastic impact between rigid bodies. Housner (6) solved the problem of rigid blocks rocking on rigid foundation. In his solution, damping was shown to be present due to the inelasticity associated with the change of the pole of rotation of the block. This damping is the result of deformation occurring only in the contact region between the two bodies. Psycharis (11) extended Housner's work and developed an impact damping mechanism for a rigid block rocking supported on a flexible foundation. An impact dashpot is used in series with the foundation spring and dashpot and is locked or unlocked depending on the solution stage. Psycharis also found a significant influence of impact damping on the response of flexible superstructures for low coefficients of restitution. An impact damper is presented in this paper that is based on Psycharis's work, and it is extended to be useful in a numerical algorithm. This damper is also tested in free vibration and transient vibration problems; and it is used in the next section in conjunction with the elasto-plastic foundation elements.

Consider the case where a mass m is dropped from a height h_o on the system composed of three elements in parallel, an elastic spring with stiffness k, a viscous damper c, and an impact damper c^*, as shown in Fig. 2(a). The impact phenomenon is associated with a small time duration where only dashpot c^* is active while spring k and damper c are not. However, the problem can be formulated where all three elements are active while the contribution to the solution by spring k and dashpot c during the impact period is negligible. Assuming that the mass is lumped to the system, the equation of motion during the impact period can be shown to take the following form:

$$m\ddot{y}(t) + c^*\dot{y}(t) = 0 \quad \text{(10a)}$$

$$y(0) = 0 \quad \text{(10b)}$$

$$\dot{y}(0) = v_o \quad \text{(10c)}$$

Where y(t) is the displacement function during impact, v_o is the impact velocity with which mass m contacts the system considered. Using Eq. 10, the displacement increment Δy during which impact is occurring can be estimated as follows:

$$\Delta y = (v_o m/c^*) \; [1 - e^{-c^*\Delta t/m}] \quad \text{(11)}$$

Where Δt is the duration of impact. Consider the case where all three elements are activated, the equation of motion during the impact period takes the following form:

$$m\ddot{y}(t) + (c + c^{*})\,\dot{y}(t) + ky(t) = 0 \quad \text{(12a)}$$

$$y(o) = 0 \quad \text{(12b)}$$

$$\dot{y}(o) = v_o \quad \text{(12c)}$$

The solution of the above equation can be easily obtained. Neglecting c in comparison to c^{*}, it can be shown that the solution approaches that of the previous case when $c^{*} >> c_{cr}$, c_{cr} being the critical damping coefficient. Therefore, the two systems yield similar results if c^{*} is chosen appropriately. During the impact duration, the velocity is expected to be reduced by the following equation:

$$\dot{y}(\Delta t) = \epsilon\, v_o \quad \text{(13)}$$

Where ϵ is the coefficient of restitution. Using the solution of Eqs. 10 and 13, the coefficient of restitution can be defined as follows:

$$\epsilon = e^{-c^{*}\Delta t/m} \quad \text{(14)}$$

Solving the above equation for the impact duration and substituting in Eq. 11 yields the displacement range where dashpot c^{*} is shut off. This displacement increment can be shown to be as follows:

$$\Delta y = (v_o m/c^{*})\,(1 - \epsilon) \quad \text{(15)}$$

Where c^* is a multiple of critical damping. However, the case where a block is contacting the ground at a point of contact 0 is a slightly different problem because of the rotational inertia induced on the spring due to block rotation. Psycharis derived the equation of motion for this case. The equation of motion associated with the element lumped at point 0 of Fig. 2(b), can be shown to be as follows:

$$m\ddot{z}(t) + (c^* I_o / I_M)\,\dot{z}(t) = 0 \quad \text{.................................}(16)$$

Where I_o is the mass moment of inertia of the block about the point where the spring is lumped, I_M is the mass moment of inertia of the block about the central point M of the foundation spring and z(t) is the total displacement in the foundation spring element. The value of I_o/I_M approaches unity for more slender structures. Thus, the impact displacement increment takes the following form:

$$\Delta z = (v_o m I_M / c^* I_o)(1 - \epsilon) \quad \text{.................................}(17)$$

The impact damper discussed is implemented into a general purpose computer program (3) so that it can be used in association with other types of elements. This damper is activated only upon recontact from an uplift situation for a displacement increment that is controlled by the impact velocity, the coefficient of restitution and other factors that are used in Eq. 17. The amount of energy dissipated by impact depends also on the position of the spring being recontacted with respect to the center point. As a test of this element, consider a shear wall 24 feet high by 12 feet wide, with a lumped weight of 200 kips at the top and resting on a Winkler foundation of the type shown in Fig. (1b). The superstructure stiffness is 500 kips/in. and the foundation stiffness is 6000 kips/in. in compression only. The properties of the superstructure and foundation springs are graphically shown in Fig. 3. Consider two cases, a free vibration case and a transient vibration case. For the free vibration case, the superstructure is undamped and the foundation has impact damping only. The displacements in the left spring and at the top of the wall are presented in Fig. (4) for $\epsilon = 1.0$ and $\epsilon = 0.70$. For the transient vibration case, the El Centro 1940 ground motion is used, the superstructure has five percent viscous damping. The results are presented in Fig.(5). It is apparent from these figures that impact damping is significant and can influence the solution dramatically, particularly for lower value of ϵ. The case considered deals with foundation elements with no other form of damping besides impact damping. This is chosen so that the influence of the impact mechanism is recognized.

NUMERICAL RESULTS

The model presented in previous sections is used to investigate the response of two classes of structures on three different cases of foundation models. The first class of structures corresponds to those of the short-period type and the second class corresponds to those of the long-period type. In each of these two classes, three foundation cases are investigated. The first case, Case A, consists of an elastic foundation spring with high viscous damping, 40 percent of critical is selected. The second case, Case B, consists of an elasto-plastic spring with low viscous damping, 10 percent of critical is selected. This low viscous damping is used to model radiation damping in the medium that is remaining elastic. The plasticity in the spring element is associated with a few feet of soil beneath the footing base and is described in Fig. 6. The third case, Case C, is the same as Case B in addition to impact damping with a coefficient of restitution of 0.7. The impact damping is associated with the inelasticity due to the change of the pole of rotation as investigated by Housner (6). This inelasticity occurs in a micro-layer in the contact zone between the footing and the soil. In all three cases described above, uplift of the superstructure is allowed. In the elastic spring case, uplift occurs when tensile displacement exceeds static displacement. In the elasto-plastic spring case, uplift occurs when displacement exceeds the static displacement offset by the accumulated plastic deformation. The two structures considered are described in the next sections.

Short-Period Structures.— A reinforced masonry building was designed in accordance with the NEHRP document (4). The base width of each of the shear walls that provide both the vertical support and the lateral resistance of the building is 20 feet. The height to the rigid diaphragm that is at the top of the shear wall is 28 feet. The vertical load supported by each shear wall is 202 kips. The weight of the shear wall that is coupled with rotational translation is 70 kips. The foundation area is 100 square feet and the soil character is assumed to be equal to granular material with a bearing capacity in excess of twice the design loads.

The superstructure's fixed base period is 0.15 seconds and the vertical period for the fully bonded case is .06 seconds. The rotation mass moment of inertia about point o is estimated to be 7694 kips.in.s^2. The elasto-plastic spring was assumed to yield at three times the static displacement. The free vibration response was investigated and the left spring displacement, superstructure deformation for Cases A, B, and C and for $\dot{X}(0) = 20$ are shown in Fig. 7. The response of the system due to the N-S component of the El Centro 1940 ground motion for Cases A, B, and C is presented in Fig. 8. The results show that Case A has a faster decay than Cases B and C but yields larger maximum deformations in the superstructure. The influence of impact damping is also shown by comparing Cases B and C. Significant reduction in superstructure maximum deformation are shown as a result of using the elasto-plastic model.

Long-Period Structures.-- A second building was designed in accordance with the NEHRP document (4) and used to investigate the response of long-period structures. This building is identical to the short-period structure except for the following characteristics. The vertical support and lateral resistance of the roof weight is provided by steel framing and vertical X-braced trusses. The weight of the braced frame and cladding that is coupled with torsional translation is 16 kips. The vertical load on the foundation and the diaphragm weight that is horizontally coupled with the bracing truss is 202 kips. The foundation area and the character of the bearing soils are identical to that used for the short period building.

The period of the superstructure was estimated to be .59 seconds and that of the vertical fully bonded structure to be .06 seconds. The rotational mass moment of inertia about point 0 is estimated to be 1740 kips.in.s^2. The superstructure aspect ratio is the same as that of the previous building. The free vibration response for $\dot{X}(0) = 20$ and the response due to the El Centro 1940 ground motion are presented in Figs. 9 and 10, respectively. These results show that the response decay is faster in Case A than Cases B and C. However, the maximum deformations reached are lower in the superstructure and higher in the foundation. The influence of impact damping is also significant in reducing the plastic deformation in the soil.

To investigate the influence of uplift on the response of structures, the two buildings previously analyzed are used with the foundation model of Case C where material damping, radiation damping and impact damping are included. The two buildings are analyzed for two support conditions: fixed base condition, and uplift-allowed condition. The superstructure deformation for the two support conditions and the two buildings are presented in Figs. 11 and 12. Significant reduction in superstructure deformation are shown in these results.

CONCLUSIONS

The response of two structures was investigated using three different foundation models, and it was concluded that an elasto-plastic spring with low viscous damping and impact damping is the most appropriate model for foundation systems. This model includes material damping, radiation damping and impact damping. The elastic spring with high viscous damping was shown to overestimate the maximum superstructure deformation and the rate of decay of the response. Since foundation nonlinearity is included in the analysis, in addition to impact damping, it was concluded that a Winkler foundation model is necessary and cannot be replaced with an equivalent two-spring system. The influence of impact damping was also shown to reduce the plastic deformation in the soil. Moreover, the model presented can be used by practicing engineers since the generation of the equations of motion can be achieved in the program used. The influence of uplift on the response of structures is even more beneficial in reducing structural deformation if the model proposed is used. Other parameters that need to be investigated are the frequency content of the probable ground motion, the nonlinearity of the superstructure and the influence of a coupled mass that does not contribute to the stability criterion.

APPENDIX I.-- REFERENCES

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APPENDIX II.-- NOTATION

The following symbols are used in this paper:

A = compatibility matrix

b = half the superstructure width

c = damping coefficient

c^* = impact damping coefficient

h = superstructure height

I_M = mass moment of inertia about point M

I_o = mass moment of inertia about point o

K = stiffness matrix

k_i = spring stiffness

M = mass matrix

m = superstructure mass

U = absolute displacement vector

u = superstructure deformation

V = vertical displacement

v_o = initial recontact velocity

X = superstructure absolute displacement

X_g = ground absolute displacement

y = vertical displacement in drop-mass problem

z = vertical displacement in rocking block problems

Dt = impact duration

Dz = impact displacement increment

ϵ = coefficient of restitution

θ = rotation angle

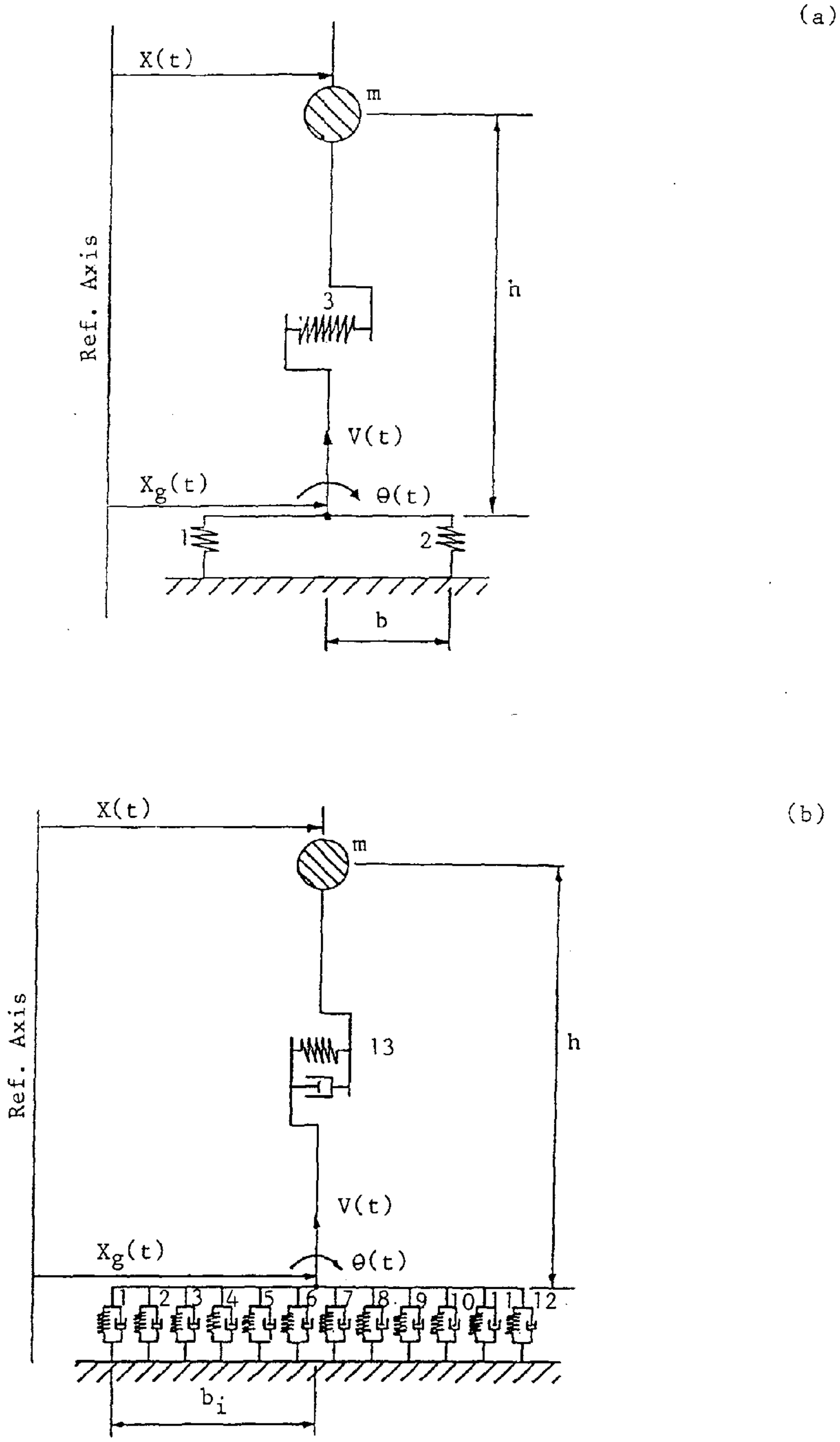


FIG. 1.—Single Degree of Freedom Superstructure. (a) Two-Spring Foundation; (b) Winkler Foundation

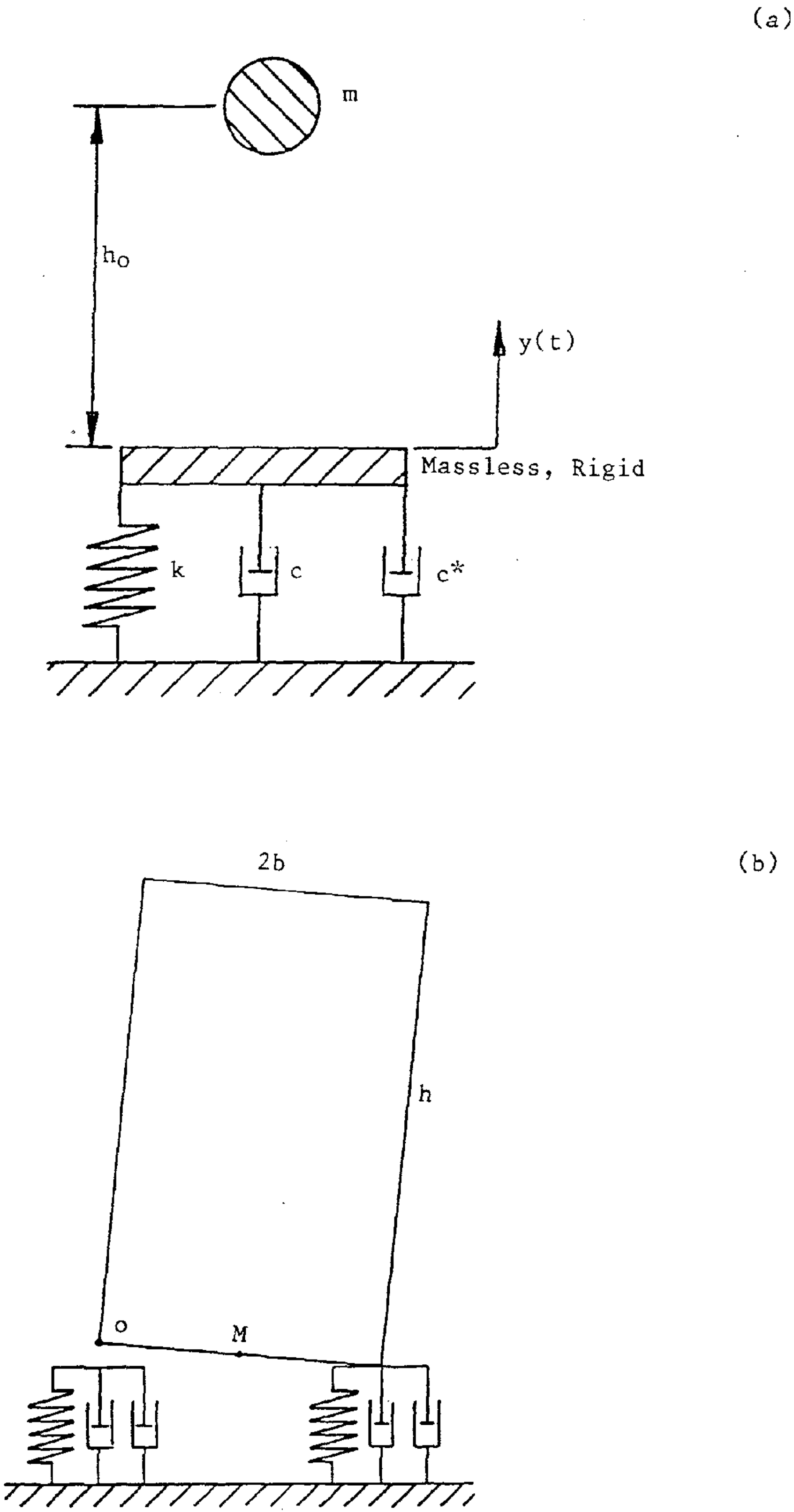


FIG. 2.—Impact Damping Mechanism. (a) A Drop-Mass Problem; (b) A Rocking Block Problem

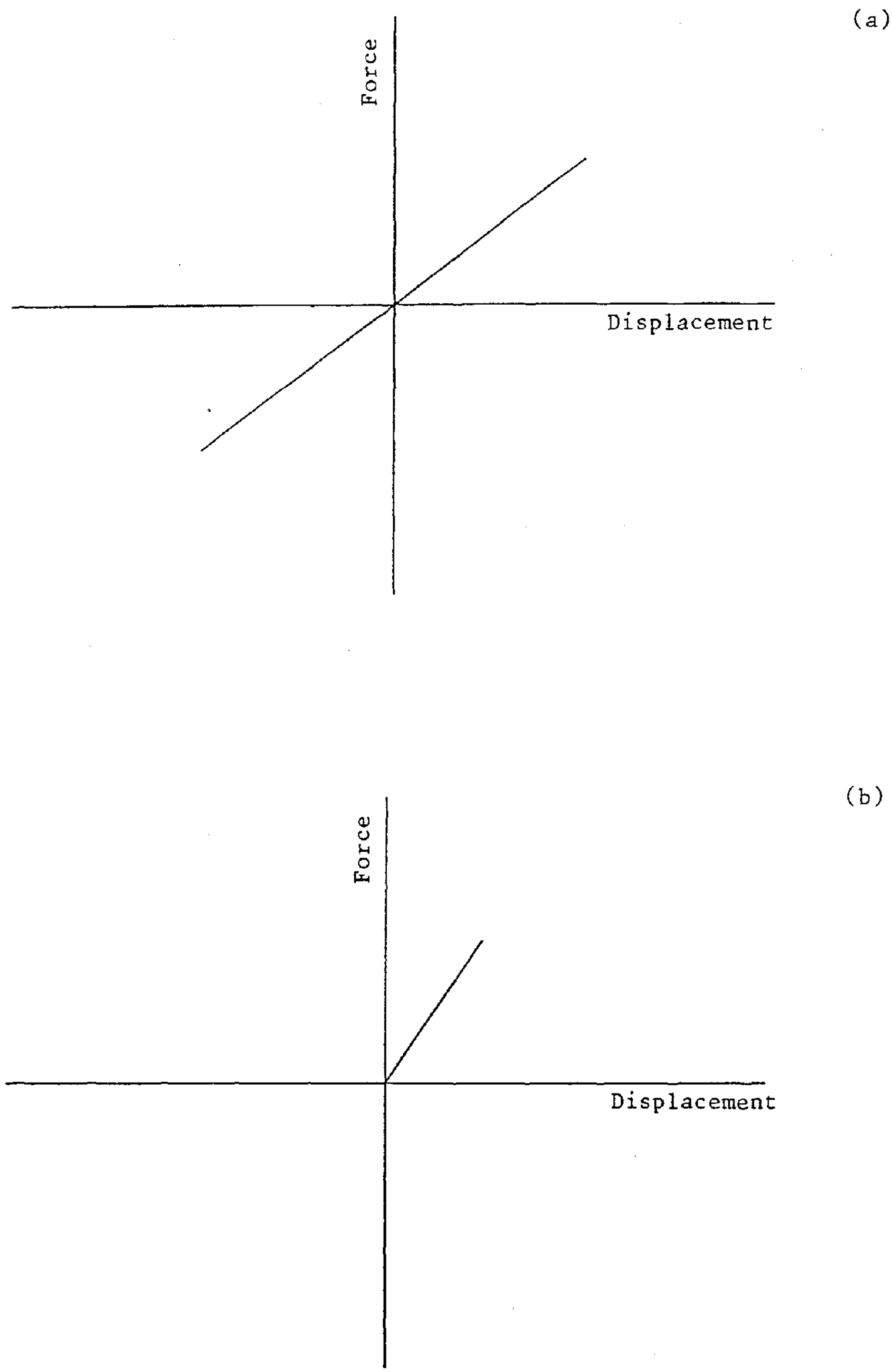


FIG. 3.—Force-Displacement Relations. (a) Superstructure; (b) Elastic Foundation Spring

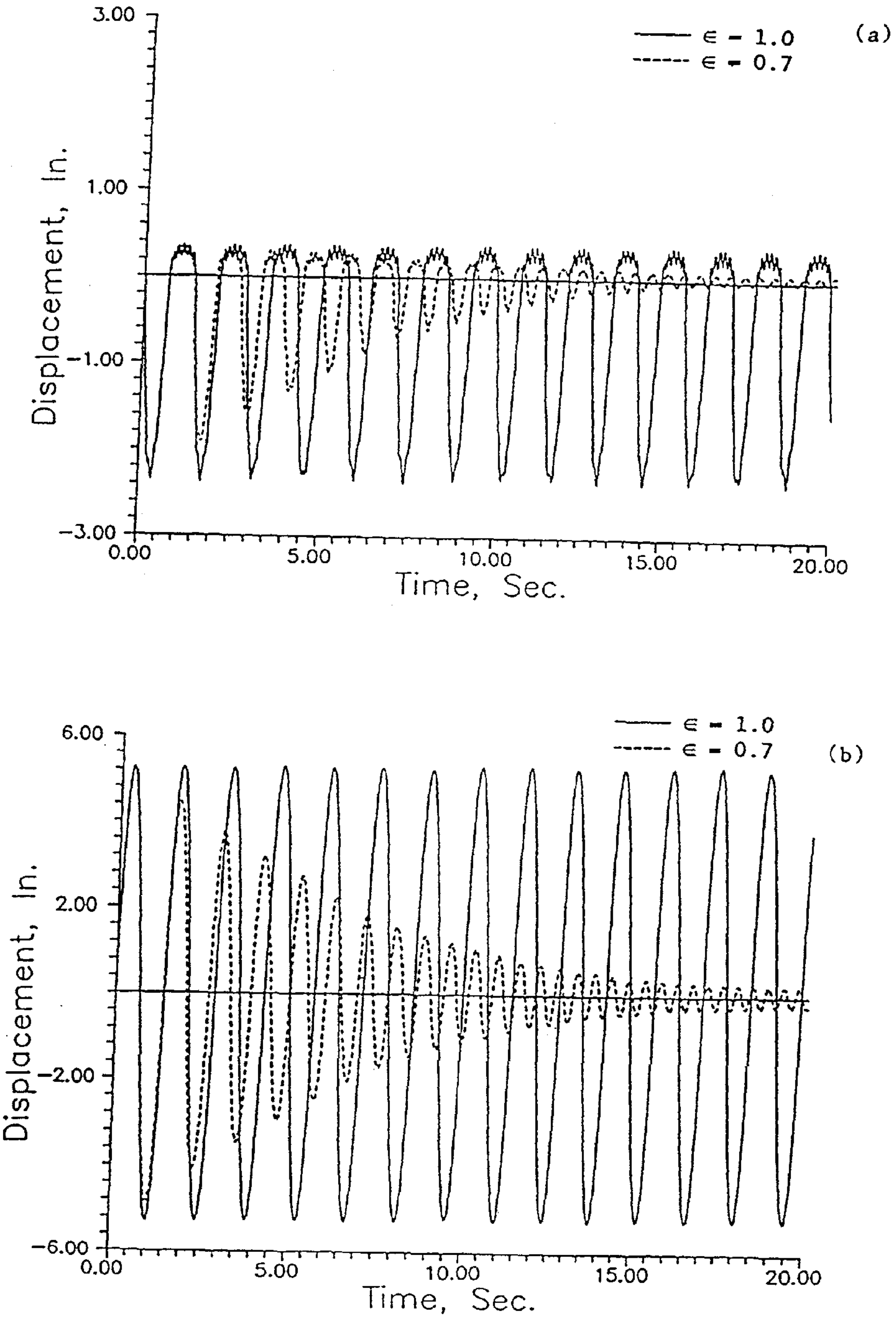


FIG. 4.—Free Vibration Response of SDOF Superstructure on Winkler Foundation for $\epsilon = 1.0$, $\epsilon = 0.7$ and $\dot{X}(0) = 20$. (a) Displacement in Left Foundation Spring; (b) Total Displacement of Superstructure

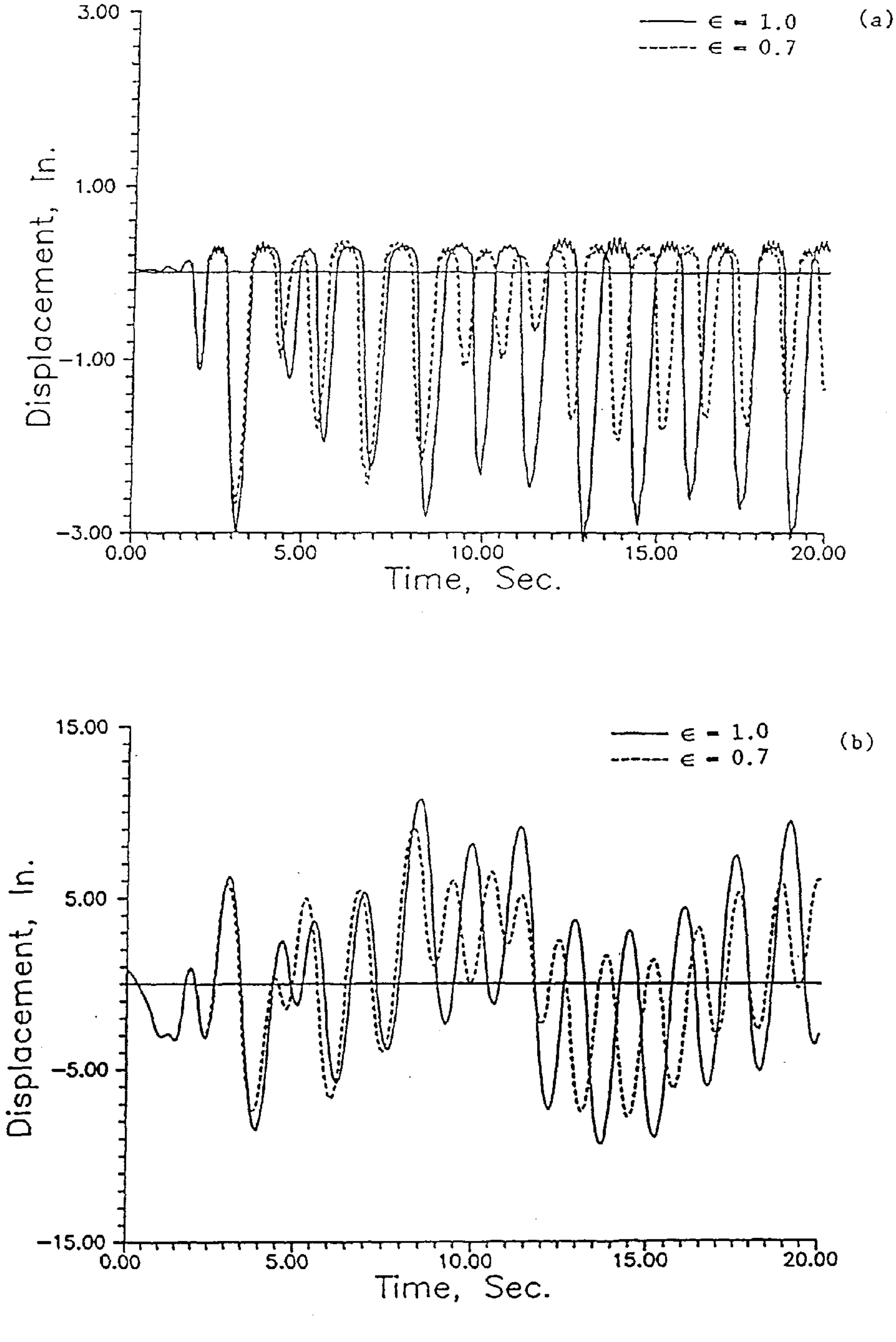


FIG. 5.—Response of SDOF Superstructure on Winkler Foundation to El Centro 1940 for $\epsilon = 1.0$, 0.70 and $\dot{X}(0) = 20$. (a) Displacement in Left Foundation Spring; (b) Total Displacement of Superstructure

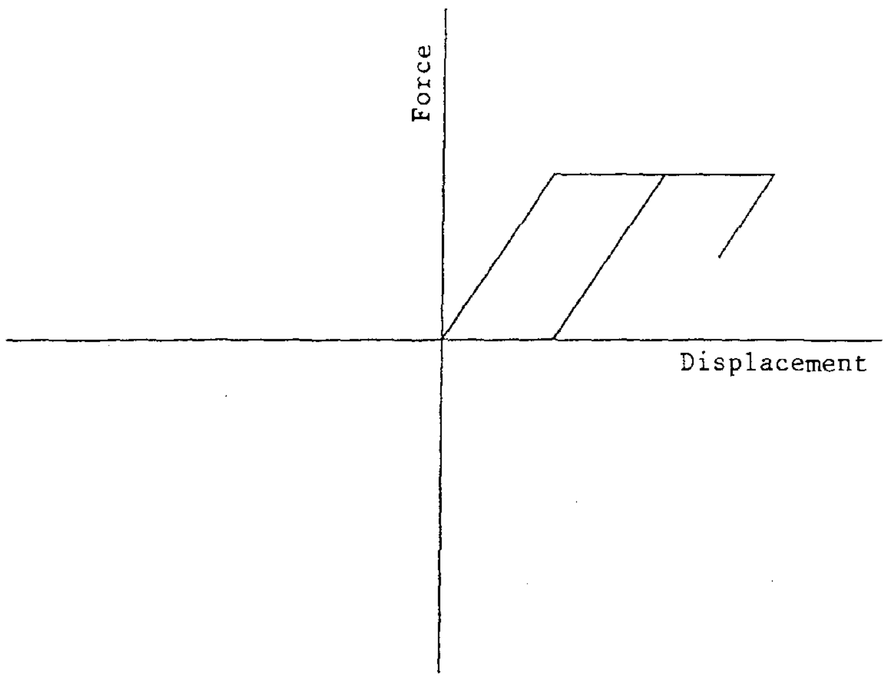


FIG. 6.—An Elasto-Plastic Foundation Spring

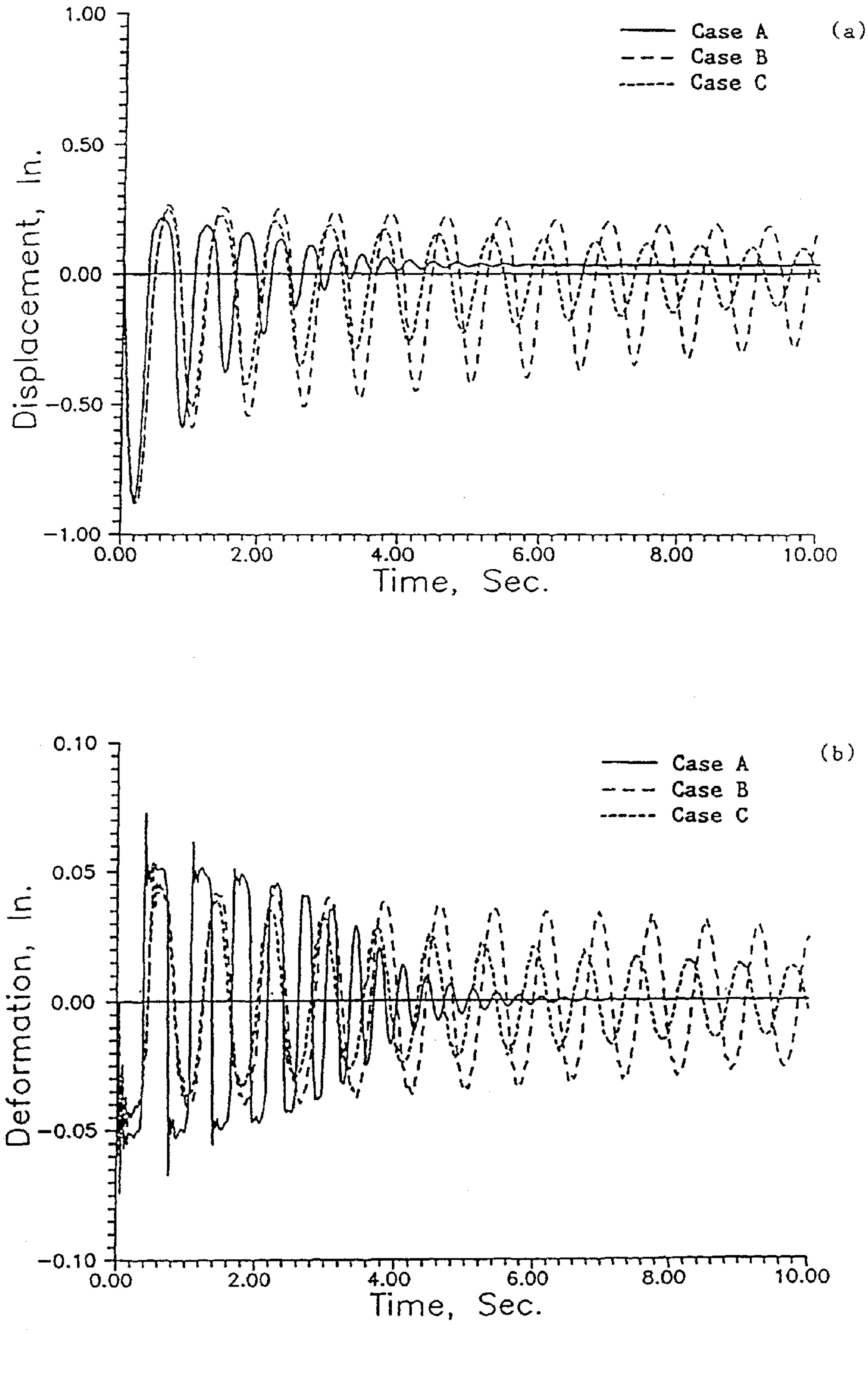
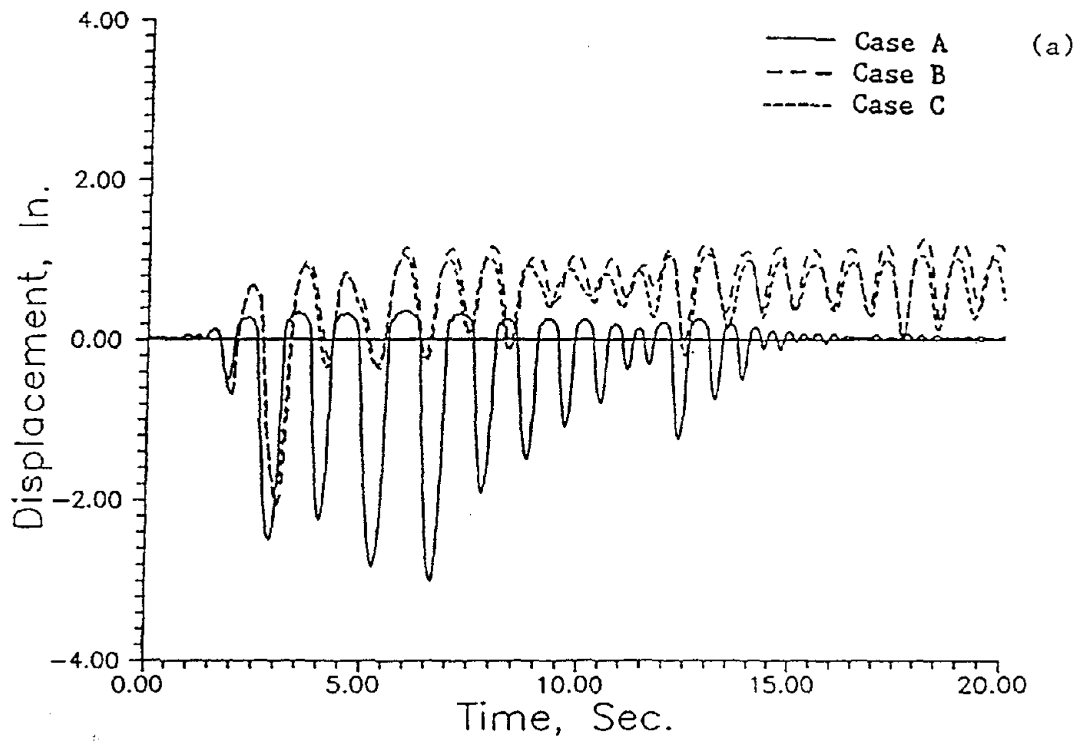


FIG. 7.—Free Vibration Response of a Short-Period Structure for Cases A, B, and C for $\dot{X}(0)$ = 20. (a) Foundation Left Spring Displacement; (b) Superstructure Deformation



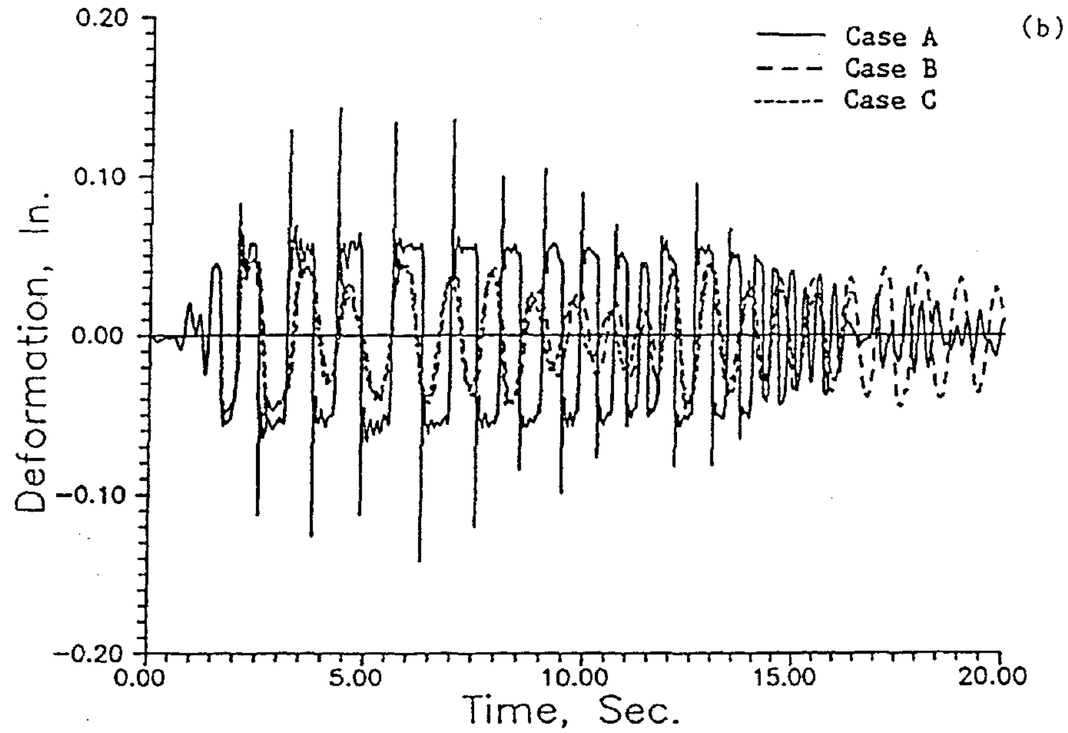


FIG. 8.—Response of a Short-Period Structure to El Centro 1940 for Cases A, B, and C. (a) Foundation Left Spring Displacement; (b) Superstructure Deformation

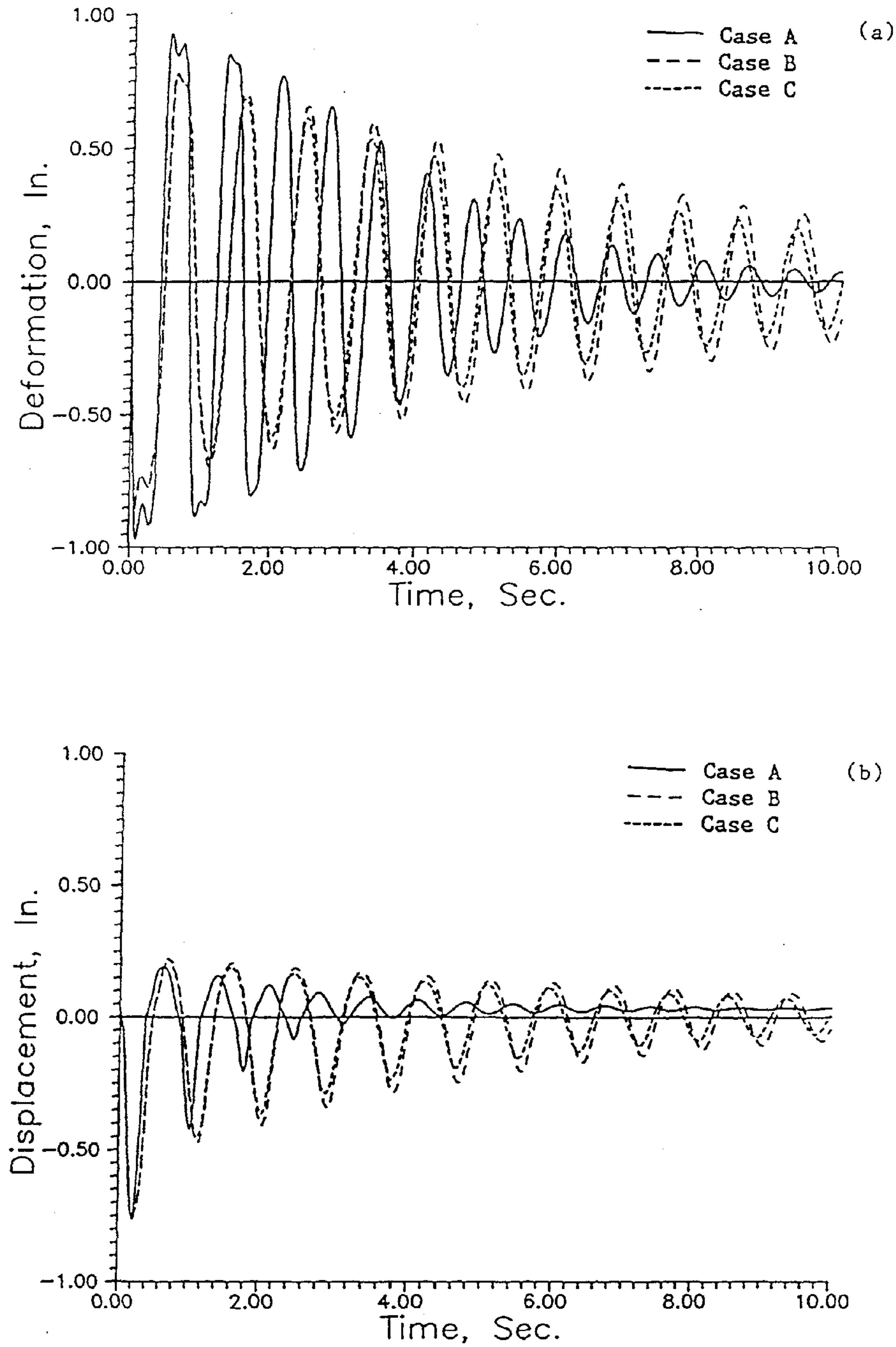


FIG. 9.—Free Vibration Response of a Long-Period Structure for Cases A, B, and C for $\dot{X}(0) = 20$. (a) Superstructure Deformation; (b) Left Spring Foundation Element Displacement

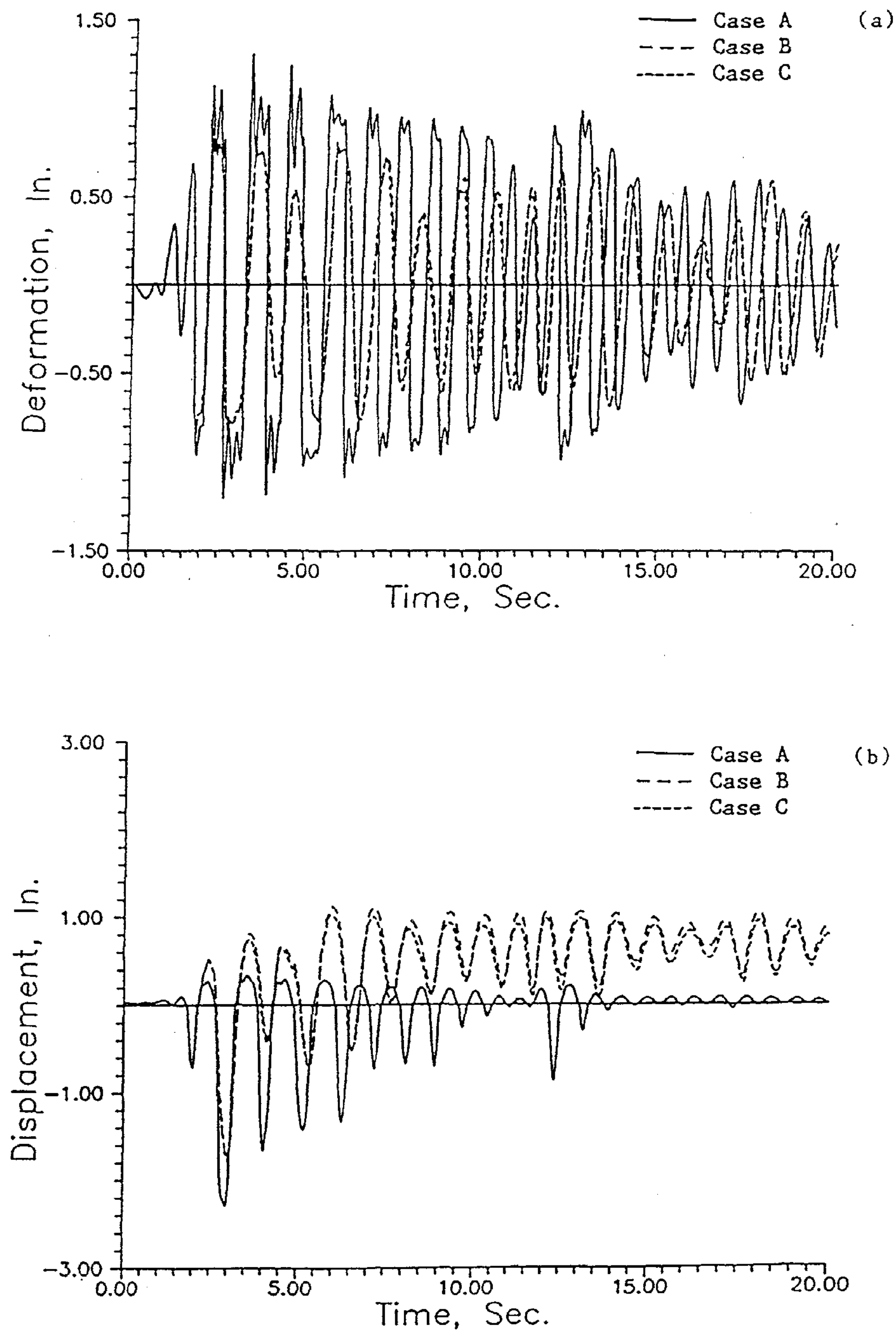


FIG. 10.—Response of a Long-Period Structure to El Centro 1940 for Cases A, B, and C. (a) Superstructure Deformation; (b) Left Spring Foundation Element Displacement

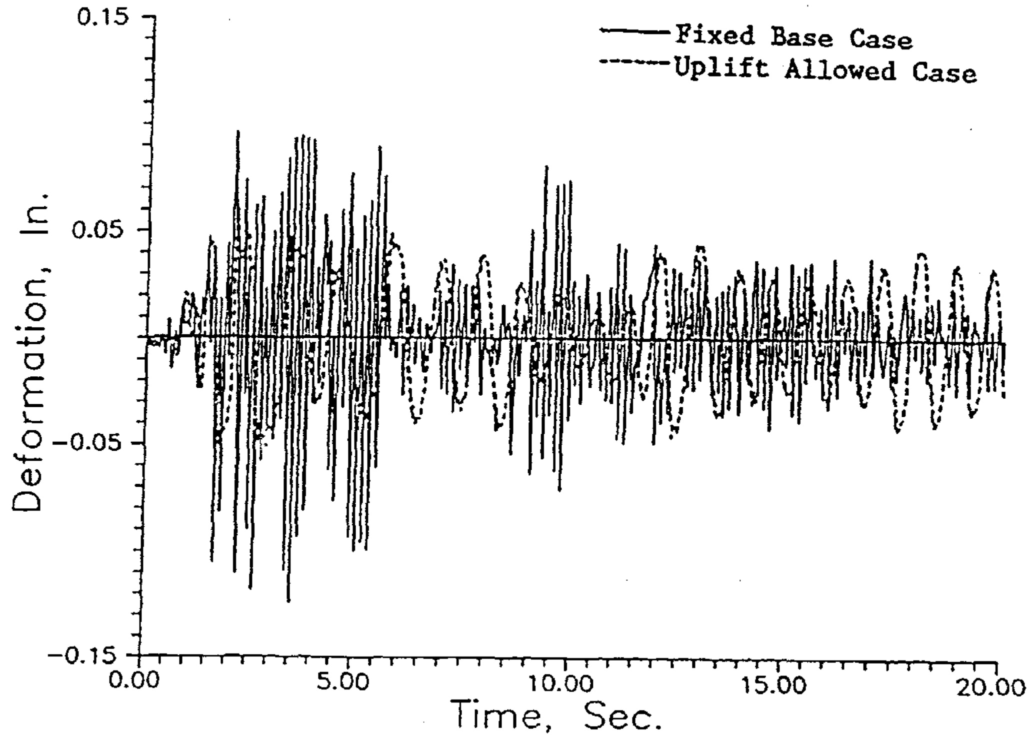


FIG. 11.—Short-Period Structure Deformation for Fixed Base and Uplift Allowed Cases Using El Centro 1940

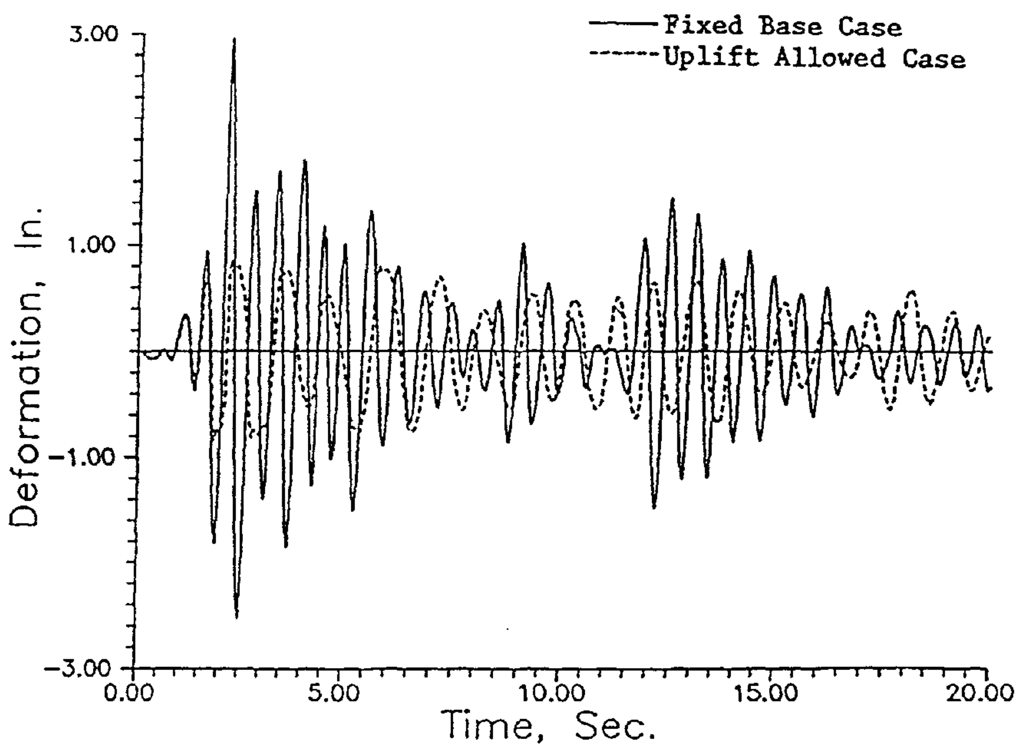


FIG. 12.—Long-Period Structure Deformation for Fixed Base and Uplift Allowed Cases Using El Centro 1940