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EFFECTS OF DIAPHRAGMS IN CONTINUOUS SLAB AND GIRDER HIGHWAY BRIDGES

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16. Abstract

The results of a study of the effects of diaphragms on load distribution of continuous, straight, right slab and girder highway bridges are presented and discussed. The parameters for the study are: the relative girder stiffness, H; the ratio of the girder spacing to span, b/a; the beam spacing, b; and the position and the stiffness of the diaphragms. Load distribution characteristics of bridges subjected to a single load, 4-wheel loadings aligned in the transverse direction and two truck loadings were studied.

In general, the criterion for comparison is the maximum moments, both positive and negative, in the beams due to 4-wheel loadings. The variations of the maximum moments with diaphragm stiffness were studied, and the conclusions of this investigation were based on the analyses of a three-span continuous bridge and various two-span continuous bridges.

Although diaphragms may improve the load distribution characteristics of some bridges which have a large b/a ratio, the usefulness of diaphragms is minimal and they are harmful in most cases. On the ground of cost effectiveness, it is recommended that diaphragms not be installed in highway bridges. The comparison of the design moment coefficients as obtained in this investigation and the AASHO recommended design values shows that the AASHO design coefficients are unconservative in many cases and unduly conservative in others.

17. Key Words	18. Distribution Statement
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INTRODUCTION

1.1 General

In Illinois, the most commonly built highway bridges are slab and girder bridges. The girders are either steel girders or prestressed concrete girders. The slabs are cast-in-place reinforced concrete slabs. Composite action between the slabs and girders is assured by the shear connectors on the top of the girders.

During the last two decades, new techniques and methods have been proposed and developed to study the load distribution characteristics in these bridges. With the aid of the electronic computer, more realistic approximations for the analysis of slab and girder bridges are now practical. Although many studies on the load distribution of slab and girder bridges have been published, at the present time, the 1969 AASHO specifications (1) for bridge design still do not reflect the state of the art of the increasingly advanced structural analysis. Especially, the guidelines for the installation of diaphragms in bridges as recommended in the AASHO specifications are quite arbitrary and the problem of where and what kind of diaphragm that should be installed in a bridge is mostly up to the discretion of the engineers. Furthermore, the effects of the diaphragms are not accounted for in the proportioning of the girders. Thus, the question whether, in all senses of practicality, diaphragms are needed in bridges needs further in-depth study.

1.2 Previous Studies

Various analytical methods have been used to analyze load distributions in highway bridges. Assumptions have been made to simplify the problem in order to get a more manageable solution. The slab and girder bridges can be looked upon as a plate structure stiffened by the girders. Along this line of thought, orthotropic plate theory is used to solve this class of problem. The work was initiated by Guyon (2) and subsequently improved and modified by Massonnet (3) to include all ranges of torsional stiffness of the girders. Both Guyon and Massonnet assumed that the Poisson's ratio for the equivalent plate material to be zero. Rowe (4) further improved the analytical technique to allow for any value of Poisson's ratio. The idealization of a slab and girder bridge as an orthotropic plate can only be justified if the girders are placed close together. For most of the highway bridges, the girder spacings are not close enough for the bridge to be satisfactorily considered as an orthotropic plate. In order to find a closed form solution for the slab-girder-diaphragm system, Dean (5) proposed an exact macro-discrete field approach to solve this class of problem, but his method is too complicated to use.

In the area of numerical approximations, Newmark's moment distribution procedure (6) is readily adaptable to slab and girder bridges. The whole slab of the bridge is considered to be an isotropic plate and is independent of the supporting beams, and the beams are treated as interior supports. Knowing the boundary conditions of the plate and the loading condition, the resultant stresses in the plate are solved for by the Levy solution. Then a slab strip is considered to be a continuous beam over flexible supports and a moment distribution is carried out for each harmonic to determine the moments in the beams. The total of moments in the slab and girders is the sum of the moments due to each harmonic. In this method, the in-plane forces in the slabs and the contribution of the slab to the stiffness of the T-beam girders are neglected, but torsional stiffness of the girders can be taken into account in this method. In the early forties, Newmark and Siess (7) analyzed slab and girder bridges using this method and the results of their analyses were one of the bases of the AASHO specifications on load distribution in highway bridges.

The finite element method (8) is a powerful tool for the analysis of slab and girder bridges. Analyses of slab and girder bridges with the finite element method have been carried out by Gustafson (9), using plate elements which include both flexural and in-plane forces. The only disadvantage of this method is that a large number of simultaneous equations have to be solved in order to get an accurate answer. For a long bridge, this method may become uneconomical.

Lazarides (10) idealized the slab and girder bridge as an equivalent grid system and solved the grid work problem by determining the deflection compatibility equations at each beam intersection. This method also leads to a large system of simultaneous equations and furthermore, the effects of the twisting moments in the slab cannot be incorporated in an equivalent grid system.

By combining the Goldberg and Leve folded plate theory and the two-dimensional theory of elasticity, Van Horn and Daryoush (11) included the effects of in-plane forces and T-beam action in the analysis of slab and girder bridges. Sithichaikasem (12) went one step further by including the torsional and warping stiffness of the girders in the analysis. The model of Van Horn, Daryoush, and Sithichaikasem is much more realistic but they needed a large, fast electronic computer to make their solution process feasible.

The effects of diaphragms on load distribution of slab and girder bridges were studied by B.C.F. Wei (13), Siess and Veletsos (14), and Sithichaikasem (12). The studies of Wei, Siess and Veletsos neglected the effects of torsional stiffness of the girders and they all used Newmark's moment distribution procedure. The investigation by Sithichaikasem included the effects of the torsional sitffness and warping stiffness of the girders and the effects of in-plane forces in the slab.

The analyses as mentioned above were all performed on simply supported bridges. Much of the present design criteria on load distribution are based on the results of the analyses of simply supported bridges and provision for the design of the negative moment regions is inferred from the behavior of the positive moments. Since most highway bridges are continuous bridges, analysis on the effects of diaphragms on continuous bridges will undoubtedly provide new data and supplement the data on the design of slab and girder bridges.

1.3 Object and Scope of Investigation

The object of this investigation is to study the effects of diaphragms on load distribution characteristics on continuous slab and girder highway bridges. The variation of maximum positive and negative moments as a function of diaphragm stiffness and location will be studied. The effects of continuity will be investigated. The range of bridge parameters under investigation will be such that they will adequately cover the range of the highway bridges built. The results of the analyses will be compared to that of the recommended design procedure as stated in 1969 AASHO specifications. Finally, recommendations on the use of diaphragms will be made, based on the results of this investigation. The scope of the studies can be stated as follows:

1. To study the effect of number of harmonics used as to the accuracy of the solution.
2. To find the best locations of diaphragms that will improve load distribution for a point load.
3. To study the effects of diaphragm stiffness on load distribution of point load.
4. For various bridge parameters (i.e., H, b/a, b) and diaphragm stiffness, k, the effects of diaphragm stiffness on load distribution of 4 wheel loads at various positions along the bridges will be studied.
5. To study the variation of the influence lines due to 4 wheel load as a function of the diaphragm stiffness.
6. To compare the load distribution characteristics of simply supported bridges and continuous bridges.
7. To compare the present AASHO recommended design moment coefficients with the moment coefficients as calculated in this investigation.
8. To study the effects of diaphragms on moments from truck loadings in continuous slab and girder bridges.
9. Recommendations on the use of diaphragms on highway bridges will be made based on the results of the analyses.

It was recommended in Ref. 12 and will be recommended in this report that interior diaphragms be eliminated from most prestressed I-beam bridges unless they are required for erection purposes. One of the practical arguments that has been raised whenever this change has been proposed in the past is the feeling that diaphragms help limit damage to an overpass structure which is struck transversely from below by an oversized load.

There appears to be conflicting evidence as to whether the diaphragms are damage limiting or damage spreading members, and the only comment the authors would make at this time is that the diaphragms currently being used in bridges are probably the wrong shape and size, and are usually in the wrong locations, if one of their valid functions is the reduction of damage to the structure due to horizontal impact on the side of the bridge. The analyses reported here are not relevant to this particular question.

1.4 Notation

The following notation is used throughout this study:

a	length of one span of the bridge
b	beam spacing
b/a	aspect ratio
d'	distance between mid-depths of top and bottom flanges (see Fig. 2.2)
h	thickness of the slab

$k = \frac{E_d I_d}{E_g I_g}$	ratio of the stiffness of the diaphragm to that of the girder
k_1, k_2, k_3	constants for calculating J
t_b, t_t, t_w	thickness of the bottom flange, top flange and web of the idealized cross section of the girder, respectively
v	Poisson's ratio
$C = \frac{(d')^2 I_t I_b}{I_t + I_b}$	warping constant of the girder
C_m	moment coefficient = $\frac{\text{Moment}}{P_a}$
$D = \frac{E_s I_s}{1-v^2}$	stiffness of an element of the slab
E_d	elastic modulus of the material of the diaphragm
E_g	elastic modulus of the material of the girder
E_s	elastic modulus of the material of the slab
F_B, F_B^*	relative flexibility and absolute flexibility matrices for the bridges
F_S, F_G, F_D, F_X	flexibility matrices for the plate, beam, diaphragm and an element X
G	shear modulus of the material of the girder
$H = \frac{E_g I_g}{aD}$	ratio of the stiffness of the girder to that of the slab
I_g	moment of inertia of the girder with a composite slab
$I_s = \frac{h^3}{12}$	moment of inertia per unit width of the cross section of the slab

I_t, I_b — moments of inertia of top and bottom flanges, respectively

J — torsional constant of the modified cross section of girder, analogous to the polar moment of inertia of a circular cross section

P — single wheel load or a single concentrated load

$Q = \frac{\pi^2 E_g C}{a^2 GJ}$ — ratio of warping stiffness to torsional stiffness of the girder

R — vector of internal forces at the junction of one diaphragm and girder, or the interior supports reactions

S_{ij} — vector of internal forces at section i of the primary structure due to loads at locations j

S_{in} — vector of internal forces at section i of the primary structure due to unit loads at sections n

$T = \frac{GJ}{E_g J_g}$ m — ratio of torsional stiffness of the girder to the flexural stiffness of the girder

Δ — vector of relative displacements at the diaphragm locations of absolute displacements at the location of interior supports

$\delta_k = 0.05$ — $\frac{C_{mk=0.05} - C_{mk=0.0}}{C_{mk=0.0}} \times 100$, percentage change in moment coefficients due to addition of diaphragms with k = 0.05

Chapter 2

STUDY OF PARAMETERS AND IDEALIZATION OF THE BRIDGE

2.1 Idealization of the Bridge and Its Components

In the analysis, the bridge is idealized as shown in Fig. 2.1. The prestressed girders are idealized as being made up of rectangular sections, as shown in Fig. 2.2. The dimensions, which are the same as the real girder, are: the width of the top flange, the thickness of the slab, the depth of the girder, the moment inertia of the girder, and the centroid of the section. The sections used are based on the set of standard prestressed concrete girders developed by the Bureau of Public Roads. The properties of these girders are described in a Portland Cement Association Bulletin, "Concrete Information" (15).

2.2 Study of Parameters

The parameters that may affect the load distribution of a bridge can be divided into five main categories:

1. Material properties of the slabs and girders;
2. Relative dimensions of the girders and slabs;
3. Geometry of the bridge;
4. Type of loading on the bridge;
5. Location and stiffness of diaphragms and the location of supports.

2.2.1 Material Properties

In the design of highway bridges, the elastic modulus of the slab concrete is usually taken as somewhat less than that of the girder concrete, since the slab concrete design strength is usually less than that of the precast girders. The test bridge at Tuscola, Illinois (16), had shown that the elastic modulus of the deck concrete was approximately the same as that of the girder concrete. Because of the uncertainty in concrete properties, the elastic modulus of both the slab and girders is taken as 4,000,000 $lb/in.^2$ in the analysis. Poisson's ratio is also assumed to be 0.15 for both the slab and girders.

2.2.2 Geometry of the Bridge

In this analysis, only straight, right, continuous bridges are considered. The symbols are defined when they are first used and in Section 1.4. The parameter b/a is defined as the girder spacing, b, divided by the length of one span, a, as shown in Fig. 2.1. Unless explicitly specified otherwise, the bridge is a five beam, two span continuous bridge with equal spans. The range of b/a varies from 0.05 to 0.2, and this will adequately cover the range of highway bridges that are usually built. A small b/a ratio means a relatively long bridge and a large b/a ratio means a relatively short one. The range of beam spacing, b, varies from 5 to 9 ft. This also is within the practical range of highway bridge design. The range of the parameters is shown in Table 2.1.

2.2.3 Properties of the Girders and Slabs

The parameters that are under investigation are H, T, and Q. They are defined as follows:

a	length of one span of the bridge
d'	distance between mid-depths of top and bottom flanges (see Fig. 2.2)
h	thickness of the slab
k_1, k_2, k_3	constants for calculating J
t_b, t_t, t_w	thickness of the bottom flange, top flange and web of the idealized cross section of the girder, respectively
v	Poisson's ratio
$C = \dfrac{(d')^2 I_t I_b}{I_t + I_b}$	warping constant of the girder
$D = \dfrac{E_s I_s}{1-v^2}$	stiffness of an element of the slab
E_g	elastic modulus of the material of the girder
E_s	elastic modulus of the material of the slab
G	Shear modulus of the material of the girder
$H = \dfrac{E_g I_g}{aD}$	ratio of the stiffness of the girder to that of the slab
I_t, I_b	moments of inertia of top and bottom flanges, respectively
I_g	moment of inertia of the girder with a composite slab

$I_s = \frac{h^3}{12}$ moment of inertia per unit width of the cross section of the slab

J torsional constant of the modified cross section of girder, analogous to the polar moment of inertia of a circular cross section

$Q = \frac{\pi^2 E_g C}{a^2 GJ}$ ratio of warping stiffness to torsional stiffness of the girder

$T = \frac{GJ}{E_g J_g}$ ratio of torsional stiffness of the girder to the flexural stiffness of the girder

The effects of T and Q on load distribution of a simply supported bridge was studied by Sithichaikasem (12) and Van Horn (11), and the results of the studies showed that the parameter Q has no influence on the load distribution characteristics of a bridge. The study also showed that for typical values of T for prestressed concrete I-section girders, the effect is small unless the value of T approaches that of a box girder section. Thus, in this investigation, Q is taken to be zero and T is the value calculated for the idealized section of the girder. The idealized section is shown on Fig. 2.2

H is a measure of relative stiffness of the slab and girder; the larger the value of H, the greater is the girder stiffness relative to that of the slab. The range of H is varied from 5 to 20. The details of the range of parameters are shown in Table 2.1.

2.2.4 Types of Loading on the Bridge

A wheel loading on the bridge is idealized as a point load. In the analysis, the point loading is represented by a series, and because of the behavior of the Fourier series, the "point load" will not be exactly a point load, but rather is a concentrated load slightly spread out about the point of application. This will in a way compensate for the idealization of a wheel load as a point load. The effect of multiple loading can be analyzed by the principle of superposition since only the elastic behavior of the structure is considered. The analysis is conducted by first analyzing to find the effects of a point load, and then the effects of four wheel loads aligned across the bridge in the transverse direction, with the spacing as shown in Fig. 2.3a, are investigated. The effects of two trucks running in the same direction, representing AASHO HS loadings, are also studied. The details of the loading and the wheel spacing of the loads are shown in Fig. 2.3b.

2.2.5 Location and Stiffness of Diaphragms and Location of Supports

The location, stiffness, and number of diaphragms that may improve the load distribution within a bridge will be studied. Although it has been shown that a relatively flexible diaphragm at midspan of a simply supported bridge may improve the load distribution to some extent (12), the effects of diaphragms in continuous bridges have not previously been studied. In this report, the effects of diaphragms at the 5/10 point, 4/10 point, and two diaphragms at the 1/3 point of the spans, are considered. The relative stiffnesses of the diaphragm to girder considered will be zero, 0.05, 0.10, 0.20, 0.40, and 1000.0. From the study of the effects of the location of the diaphragms, an optimum location is selected. The effects of this optimally located diaphragm on load distribution for various bridges are studied in greater detail.

In order to verify that the results obtained by analyzing a two span bridge can also be applied to a multispan bridge, a three span continuous bridge with $H = 20$, $b/a = 0.1$, and $b = 7$ ft will be analyzed. A comparison of the results of the analyses of both simply supported and continuous bridges will be made.

Chapter 3

METHOD OF ANALYSIS OF BRIDGE

3.1 Introduction

A slab-girder bridge can be idealized as an assemblage of plate and girder elements interconnected at longitudinal joints. Various methods have been developed to analyze this kind of system, as mentioned in Chapter 1. Of all the methods of analysis, Fourier harmonic analysis is the best for this kind of structure in terms of precision and computational effort. By representing the loading and the internal forces in the structure by series in one direction, it has the advantage of transforming a two-dimensional problem into a one-dimensional one, which greatly reduces the size of the problem. The only limitation of this method is that the structure must be straight and the ends must be simply supported. Fortunately, the above conditions are satisfied by most highway bridges.

3.2 Basic Assumptions

The basic assumptions of the analysis are those of the ordinary theory of plate flexure and two-dimensional theory of elasticity and the following:

1. Diaphragms are installed at all supports. The support diaphragms are perfectly rigid in their own planes but are free to rotate in the direction normal to its plane.
2. Adequate shear connectors are provided to insure full composite action of the girder and slab.

3. Spacing of the girders are the same for all girders in a bridge.
4. Shear deformations of the diaphragms and girders are neglected.
5. Internal supports are nonyielding.

3.3 Method of Analysis

The method of analysis is essentially a force method of analysis based on the Goldberg and Leve (17) prismatic folded plate theory and harmonic analysis. The external load, displacements and internal forces are all expressed in Fourier series components and the structure is solved for the internal forces due to the applied external load. Once the analysis of a loading has been developed for a particular harmonic, the total effects of the load can be obtained by summing up all the harmonics considered. The number of harmonics needed to obtain an acceptable solution will be discussed later.

The problem of load distribution of a multispan bridge subjected to multiwheel loadings can be solved in four steps.

3.3.1 Analysis of the Primary Structure

The flexibility matrix of the plate and the beam elements are derived from the theory of elasticity and classical plate flexure theory. The detailed equations for the plate and beam flexibility matrix were reported in Refs. 11 and 12. By considering the compatibility at each joint, we can solve for the internal forces acting at each joint as follows:

F_s is the flexibility of plate element

F_G is the flexibility of the beam element

$L_{x\ell}, L_{xr}$ are the displacement vectors at the left and right edges, respectively, of element x due to applied external load on that element

$P_{x\ell}, P_{xr}$ are the vectors of internal forces at the left and right edges of element x

The flexibility matrix can be partitioned such that for an element x

$$F_x = \left[\begin{array}{c|c} F_{x\ell\ell} & F_{x\ell r} \\ \hline F_{xr\ell} & F_{xrr} \end{array}\right]$$

At joint N, the right edge of the plate element is connected to the left edge of the beam element, the displacement of the right edge (i.e., r edge) of plate element j is

$$F_{Sr\ell}\, P_{j\ell} + F_{Srr}\, P_{jr} + L_{jr}$$

and the displacement of the left edge of beam element i is

$$F_{G\ell\ell}\, P_{i\ell} + F_{G\ell r}\, P_{ir} + L_{i\ell}$$

For compatibility of joint N, the displacement at the right edge of element i must equal the displacement at the left edge of element j. Observing

$$P_{jr} = P_{i\ell} = p_N$$

we can write

$$F_{Sr\ell}\, P_{j\ell} + F_{Srr}\, p_N + L_{jr} = F_{G\ell\ell}\, p_N + F_{G\ell r}\, R_{ir} + L_{i\ell}$$

and that

$$\left[F_{G\ell\ell} - F_{Srr}\right] p_N + F_{G\ell r} P_{ir} - F_{Sr\ell} P_{j\ell} = L_{jr} - L_{i\ell}$$

where p_N is the internal force vector at joint N.

By applying the above condition for all the joints we have

$$F^* p^* = L^*$$

where

F^* is the assembled flexibility matrix of the structure

p^* is the force vector at all joints

L^* is the applied load flexibility vector

Solving the above matrix equations, the internal forces at the joints are found. Internal forces in the slabs and girders can be obtained by substituting the joint forces into the equilibrium equations of the individual element. In this analysis, there is no way to calculate the effective width of the I-section girder. In order to get the composite moment of the girder, the condition that there is no axial force in a composite beam under pure bending is used (i.e., $\int \sigma dA = 0$). The moment in the girder which satisfies the above condition is taken as the composite moment of the girder.

3.3.2 Analysis of the Effects of Diaphragms and Interior Supports

The effects of diaphragms and interior supports are such that vertical reactive force and twisting moment are developed at the junctions of the girders and diaphragms. The diaphragms are idealized as being cross beams simply supported by the exterior girders. The flexibility of the diaphragm is obtained by the conjugate beam method. Unit loads are placed at B, C, D; unit couples are applied at A, B, C, D, and E (see Fig. 3.1); and the flexibility coefficients are the displacements at points A, B, C, D, and E due to the unit loads. Two equations of equilibrium are also needed, i.e.,

$$\sum F_z = 0$$

$$\sum M_x = 0$$

to account for the missing unit load at A and E. The flexibility matrix of the primary structure at the location of the diaphragms or the interior supports is obtained from solutions of unit loads applied on the primary structure. To calculate the relative flexibility of the structure at the location of the diaphragms and the absolute flexibility at the interior support, unit loads and couples are placed at A, B, C, D, and E consecutively at each diaphragm and interior support location. The relative displacements due to unit loads and couples are calculated with respect to the line joining the deflected points of A and E (see Fig. 3.2). When the section considered is an interior support, the absolute displacements of the structure at the interior support section are used to generate the flexibility coefficients.

The compatibility equations for the diaphragms and supports are as follows:

At the sections where the diaphragms join the girders:

$$F_D R = F_B R + \Delta$$

At the interior supports:

$$F_B^* R + \Delta = 0$$

where

F_D is the flexibility matrix of the diaphragm

F_B is the relative flexibility of the structure at the diaphragm locations

F_B^* is the absolute flexibility of the structure at the interior support locations

Δ is the relative displacement vector at the sections of diaphragms or the absolute displacement vector at the interior support section due to applied external loads

R is the reactive force vector at the junction of one diaphragm or interior support and the girders

The solution to the above equations will give the reactive forces at the diaphragm and support locations.

If S_{ij} is the internal force vector at section i of the primary structure due to loads at location j, and S_{in} is the internal force vector of section i of the primary structure dut to unit loads at sections n, then the internal forces at section i of the continuous bridge with diaphragms will be S_{ij}^*, where

$$S_{ij}^* = S_{ij} + \sum_{n=1}^{K} S_{in} R_n$$

and

K = sum of the number of sections with diaphragms and interior supports

For a two span continuous bridge with five girders, fifteen unit load loadings and fifteen unit couple loadings are needed to generate the flexibility matrix of the structure if there is one diaphragm in each span. If symmetry is considered, only six unit load loadings and six unit couple loadings are needed.

3.3.3 Determination of the Influence Lines for Four Wheel Loadings

Concentrated loads are placed at various locations on the beams of the bridges as shown in Fig. 3.3, and an influence surface could be generated. Instead of using the influence surfaces to find the critical moments in the beams due to a system of applied loads, influence lines for the moments in the beams due to a four wheel loading moving along the bridge are used. The choice of the four wheel loading is to simulate the loading of single axles of two trucks going in the same direction along the bridge. A previous study (12) has shown that two lane loadings always produce maximum moments in girders, if the reduction factors for multilane loadings given in the AASHO specifications are used. The axles are aligned in the transverse direction. The spacing of the wheels is in accord with the AASHO recommendations (see Fig. 2.3).

Then the problem is to locate the position of the wheels in the transverse direction which will give a maximum moment for a beam at a section. The maximum moment in each beam at section i due to a four wheel loading (4W loading) at section j can be found by searching for the maximum moment induced in a beam as the 4W loading moves from one edge to the other edge of the bridge. Moments at section i in each beam due to unit loads applied at A, B, C, D, and E at section j are first calculated and a curve is fitted through the five data points, giving the influence lines for the moment in a beam at section i due to a unit load moving transversely at section j. The effect of a 4W loading is the sum of the coordinates of the influence line at the positions of the loads multiplied by a scalar constant (i.e., the magnitude of the load).

After some trial and error, it was found that the best fit curve for the influence line for beam A is either a fifth degree polynomial or three piecewise smooth parabolas connecting A, B, C; B, C, D; and C, D, E. The three piecewise smooth parabolas were used with the average value of the two curves being used between B, C, and C, D. For beam B, the best fit curve is three piecewise smooth parabolas as in beam A. For beam C, a seventh degree polynomial is used. The results of the curve fitting are shown on Fig. 3.4. The points at the beams, A through E, are the given data points, and the intermediate points are those calculated in the curve fitting process. The curves show good agreement with the value when the load is applied on the slab. Note that there is a kink in the curve for beam C between A B and D E when k = 0.0. This kink may give a small error for the total moment due to a 4W loading, but the error is small because of the large influence coefficient at C and the wheels will be in the regions between B and D at maximum moment because of symmetry.

3.3.4 Determination of Effects of Truck Loadings

To get the absolute maximum moment in the beams, theoretically, the trucks have to wiggle along the bridge. However, it has been assumed that the trucks will move along straight lines parallel to the girders in the bridge. The result of preliminary analyses showed that the absolute maximum moment in a beam due to a 4W loading will be when the 4W loading is at or near the 4/10 point of one span from the simply supported end. For the purpose of finding the effect of truck loadings, the transverse position of the wheel is assumed to be that which will induce a maximum positive moment at the 4/10 point. With the distance of the wheel from the edge beam thus fixed, we can proceed to find the maximum moment in the beams due to the two truck loading. Again, having the influence coordinates for a beam at a particular section, a curve is fitted to pass through all the points. The Lagrangian interpolation function (18) is used to fit the data points and the moment due to a two truck loading is the sum of the influence coordinates at the position of the wheel multiplied by the respective fractions of weight in each line of wheel. For example, for a HS20 loading the front axles will have a scalar factor of 0.25 and the real axles will have a scalar factor of 1.0.

3.4 Computer Program Information

3.4.1 General Programming Considerations

Four computer programs were developed to analyze the bridge. The first program which handles the analysis of the primary structure was a modification of the program developed by Sithichaikasem (12). Extensive use of secondary storage is needed to store the data generated during the analysis of the primary structure. The second program is used to add in the effect of supports and diaphragms and to do the back substitution. The third program takes care of the curve fitting process and finds the locations of the wheels to give the maximum moments for 4W loadings and two truck loadings. The input of the third program is the output of the second program and no rearrangement of data is needed for the input of the third program. The fourth program plots the influence lines for sections 2 through 9 (designated in Fig. 3.3) and punches the influence coefficients on data cards.

3.4.2 Convergence Behavior of the Method of Computation

In order to find the effect of the number of harmonics on the accuracy of the solutions, a simply supported bridge with H = 20, b/a = 0.05, and b = 8 ft which was subjected to three loading cases was studied. The loadings were

1. A 10 kip concentrated load at midspan of beam A
2. A 10 kip concentrated load at midspan of beam B
3. A 10 kip concentrated load at midspan of beam C

The results of the analysis are shown in Figs. 3.5 to 3.7. The total composite moment of all beams is compared with the total static moment as calculated from the elementary beam theory. It is noted that by using just one harmonic the total moment at midspan is already 82 percent of the static moment, and by using 5 harmonics 93 percent of the total static moment is obtained. The rate of convergence is much smaller after using 5 harmonics, and at 35 harmonics 97 percent of the total static moment is distributed among all the beams. The figures show that regardless of the loading condition, the unloaded beams always converge to a constant moment after a few harmonics but the moments in the loaded beam continue to increase. Of the 3 percent of the total static moment that is unaccounted for at 35 harmonics, 1 to 2 percent of the total moment may be the slab moment (7) and the other 1 percent may probably belong to the moment of the loaded beam.

Chapter 4

PRESENTATION AND DISCUSSION OF RESULTS

4.1 General Discussion

The main objective of this investigation is to study the effects of diaphragms on the load distribution characteristics of continuous slab-girder bridges. The effects of diaphragms on load distribution of simply supported bridges were investigated by Veletsos and Siess (14), Wei (13), and Sithichaikasem (12), and others (19). It is difficult to make a direct comparison between the results of an analysis of a continuous bridge and a simply supported one because of the change of the "effective" span length due to the negative moment at the interior support. The studies of Veletsos and Siess, Wei, and Sithichaikasem paved the way and brought out the significant parameters that influence the load distribution characteristics of the bridges. Most of the attention will be focused on the moments in the beams since the magnitudes of the beam moments are the governing parameter which control the design of the girders. In addition to the effects of the diaphragms, the influences of the bridge parameters, H, b/a and b on moment distributions were also studied. The results are presented as follows:

1. Effects of diaphragms stiffness on distribution of a point load in continuous bridges,
2. Effects of location of diaphragms on load distribution,
3. Effects of diaphragms stiffness on load distribution of a 4W (4-wheel) loading,

4. Effects of bridge parameters on moment distribution,
5. Comparison of the effects of diaphragms for a simply supported bridge and a two span continuous bridge,
6. Effects of diaphragms on moments from truck loadings, and
7. Effects of diaphragms on three-span bridges.

Based on the results of the previous investigation of simply supported bridges, Ref. 12, the effects of warping were found to be negligible and the effects of torsional stiffness of the beam are important only when the torsional stiffness of the beam approaches that of a box section. In this analysis the warping coefficient is taken to be zero and the torsional stiffness is calculated based on the transformed sections of the prestressed girders.

4.2 Effects of Diaphragm Stiffness and Location on Distribution of a Point Load in Continuous Bridges

Moment envelopes of beams A, B, C are shown in Fig. 4.1 to 4.3, respectively for single loads moving along the beams. The influence lines of moment at the interior supports of beams A, B, C are shown on Fig. 4.4 to Fig. 4.6. The figures show that the diaphragms reduce the moment at the vicinity of the locations of diaphragm and that a diaphragm is more effective in moment reduction for the interior beams than for the edge beam. In all cases, the absolute maximum positive moment is reduced and the location of the maximum moment is displaced. Considering the relative diaphragm stiffness of $k = 0.4$, it seems that for a point loading, the greatest moment reduction occurs when the diaphragm is at 4/10 point of the span from the simply supported end. The diaphragms cause less drastic changes in the negative moments at the interior supports. Fig. 4.4 to 4.6 show that the reduction in negative moment is approximately the same for all beams, and it seems that there is no single optimum location of diaphragm, when the maximum negative moment at the support is concerned.

The effects of location of diaphragms on load distributions in simply supported bridges were studies by Sithichaikasem (12). The results of his study showed that the diaphragms have the greatest effect on the load distribution at the immediate vicinity of the diaphragms and the best location of the diaphragm is where the maximum moment would occur. For a simply supported bridge, the optimum location is at the midspan The effects of adding more than one diaphragm at other locations for a simply supported bridge were also studied. The results of the analyses showed that the effects of having one diaphragm at mid-span or two diaphragms at 5/12 point or two diaphragms at 1/4 points and one at mid-span are practically the same.

For a continuous bridge, the maximum positive moment will no longer occur at the midspan but rather the maximum will be somewhere away from the midspan towards the simply supported end. Three locations of diaphragms were studied: a) one diaphragm at midspan of each span, b) one diaphragm at the 4/10 point of each span, and c) one diaphragm at each 1/3 point of each span.

The positive moment envelopes and the influence lines for negative moments at the interior supports are shown in Fig. 4.1 to Fig. 4.6. The moment coefficients, C_m, due to 4W loadings are shown in Fig. 4.7.

For the maximum positive moment in the beams, the effects of one diaphragm at midspan of each span or two diaphragms at the 1/3 points of each span are practically the same.

The bridge with one diaphragm at 4/10 point is slightly different than the other two in three ways:

1. At k = 0.05, the moments in beams A and B are approximately the same, whereas for the other two cases, the beam B moment is greater than the beam A moment,
2. The maximum moment in the exterior girder increases much more rapidly and becomes the controlling girder once the relative diaphragm stiffness is greater than 0.05, and
3. At k = 0.4 the difference in moment among the girders are the greatest and the magnitude of the positive moment in the exterior girder is the greatest.

The variation of maximum negative moment at the interior support bears no resemblance to that of the positive moments in the girders. The best load distribution occurs when the diaphragm in at the 4/10 point with the k value ranging from 0.1 to 0.4, although no combination of diaphragms results in a reduction in negative moment in excess of 7.5 percent. For the bridge with diaphragms at midspans, the optimum k value is approximately 0.1. With 1/3 point diaphragms, the optimum k is slightly smaller.

It can be concluded that the optimum diaphragm is one at the 4/10 point, with a k value between 0.05 to 0.075. Such a flexible diaphragm is rarely practical or realistic because it would have to be either way shallow or very thin. Most of the diaphragms cast in prestressed concrete girder highway bridges in Illinois are 8-in. thick and about 0.8 the depth of the girders, and have k values of approximately 0.3 to 0.4. In view of this fact, the best choice in lieu of a flexible diaphragm at 4/10 point is a midspan diaphragm, since stiff diaphragm at this point has the least detrimental influence on load distribution characteristics of the bridge.

For this particular bridge, no arrangement of diaphragms is capable of causing a significant reduction in maximum moments, and from a cost-effectiveness point of view, no diaphragm is undoubedly the optimum since a flexible diaphragm will cost about the same as a stiff one.

4.3 Effects of Diaphragm Stiffness on Load Distribution of 4W Loading

Fig. 4.8 to Fig. 4.16 show the effects of diaphragm stiffness for a 4W loading, where the four wheels are spaced as shown in Fig. 2.3. As discussed in Ref. 14, the influence of diaphragm on load distribution will be smaller for a multiple load loading than for a point load. Thus, the change in moment coefficients of a 4W loading with various diaphragm stiffness will be small.

For bridges with H = 5, b/a = 0.05 and 0.10, beam A is always the controlling girder regardless of beam spacing, for both positive and negative moments. For a small beam spacing of 5 ft, the C_m values ($C_m = \frac{M}{Pa}$) for the maximum positive moments of beam B and C are fairly close together, but this is not the case for the negative moment at the support. When b = 5 ft and b/a = 0.1, the C_m values at the supports for beams A and B are the same (Fig. 4.9), in contrast to the distinct differences in negative moments of beams A, B and C when b/a = 0.05 (Fig. 4.8). With the larger b/a ratio of 0.20, the load distribution characteristics are markedly different from that of the previous cases, as shown in Fig. 4.10. When b = 5 ft, the center beam, beam C, is the controlling girder while the moment in beam A and B are fairly close together. With the larger beam spacings of 7 ft and 9 ft, the controlling beam is beam B, but the moment in girder C is only slightly less than girder B.

When k = 0.0, beam A has the smallest C_m, but as k increases, the moments in beam A quickly catch up with the moments in beam C and are greater than in beam C at k = 0.4. In any case, for H = 5, b/a = 0.2, and b = 5, 7 or 9 ft, the controlling girder is never the edge girder and the increase in diaphragm stiffness helps to bring the moments in the beams closer to the same values. For the bridges with H = 5, b/a = 0.05 and 0.10, and b = 5, 7 or 9 ft the controlling girder is always the edge girder, beam A, and the increase in diaphragm stiffness only does more harm than good.

Tables 4.1 and 4.2 show the moment coefficients as k changes from zero to 0.4. It is obvious that C_m for beam C always decreases (Ref. 14) and has the largest percentage change. It seems that for a same H value and b/a ratio, the largest changes in C_m for positive moment, with various k values, occur in bridges with b = 7 ft. For the case of negative moment coefficients, the larger the beam spacing, the greater the change in C_m as k varies.

As the relative stiffness of the girders increases to H = 10 and 20, the behavior of the bridges with b/a = 0.05 is approximately the same as that for H = 5 (Fig. 4.11 and Fig. 4.14). For the bridges with H = 10 and 20 and b/a = 0.10, the controlling girders, when there are no diaphragm, are still the same as in the case of H = 5 - beam B or beam C. But as the diaphragm stiffness increases, the moments in beam A quickly become the controlling moments. The higher the relative girder stiffness, H, the greater is the value of k needed for beam A to attain the largest C_m. The optimum diaphragm stiffness for H = 10 is from 0.03 to 0.05, and for H = 20 is from 0.05 to 0.08. The behavior of the negative moment at the support is approximately the same as that of the maximum positive moment. As k varies from 0.00 to 0.4, the change of C_m is greater than that for H = 10 and 20 than for H = 5, but the trend is the same with beam C having the greatest change in C_m.

When b/a = 0.2, the bridges with H = 5, 10, and 20 are similar in that the interior beams are the controlling beams. As H increases, the difference in moments between the interior and edge girders increases, as can be seen in the moment values given in Tables 4.1 and 4.2. Even when k reaches a value of 0.4, girder A still has the smallest value of C_m and this is also true for the negative moments at the support. The optimum diaphragm stiffness for these bridges is k = 0.4 or higher.

So far we are concerned with the maximum moment in the loaded span or at the support. To complete the picture of the effects of diaphragm stiffness on a 4W loading, the effects of the load distribution characteristics in the unloaded span are shown in Fig. 4.17. Comparing Fig. 4.17 and Fig. 4.8 for the bridge with H = 5, b/a = 0.05 and b = 7 ft, we see that the effects of diaphragms on this bridge, which already has a good load distribution, are practically nonexistant. However, it is noted that the maximum negative moment at the mid-span of the unloaded span is only 42 to 45 percent of the maximum moment at the support instead of the 50 percent value found for a beam.

Comparing Fig. 4.12 and Fig. 4.16 to Fig. 4.18, there are big differences between the behavior of the negative moment at the midspan of the unloaded span and the maximum moment at the support. For the bridge with H = 10, b/a = 0.1, and b = 7 ft, the moment distribution at midspan of the unloaded span is unique in itself. It resembles neither the behavior of the maximum positive moment nor the maximum negative moment. The midspan moments in beams A, B and C of the unloaded span are 54.5, 42.0, adn 34.0 percent of the respective maximum negative moments at the support when k = 0.0 and the moments stay nearly constant as k increases.

For the bridge with H = 20, b/a = 0.2 and b = 7 ft, the moments in the beams of the unloaded span do not show the large difference in moments in the beams as exhibited in the beam moments at the support. The maximum negative moment at midspan of the unloaded span of beams A, B and C are 56.6, 45.0, and 46.0 percent of the respective maximum negative moments at the support. As k increases, the percentage of moments increases for beam A and decreases for beams B and C.

It can be concluded that the moments in the unloaded span of the two span continuous bridge are more evenly distributed than either the support negative moments or the positive moments in the loaded span.

The influence lines at various sections of a number of representative bridges due to a 4W loading are shown on Fig. 4.18. The positive moment envelope and the influence lines at various section of a two span continuous beam is shown on Fig. 4.19, for purposes of comparison. For a beam, the influence lines are always concave upwards while the influence lines for beam A of the bridges exhibit a concave-convex-convex-concave sequence, indicating that at some points the slabs are helping to support the beams. The influence lines for the interior beams are always concave upwards and they can be interpreted as the beams always support the slabs.

The addition of the diaphragms may not drastically change the maximum moments in the beams but the location of the maximum moment is changed. The higher the diaphragm stiffness, the greater is the shift of the location of the maximum positive moment. As the diaphragm's stiffness increases, the location of the maximum positive moment of beam A shifts from 0.42 of the span from the simply supported end towards mid-span, but the location of the maximum positive moment of beam B and C shift from 0.42 from the simply supported end towards the 0.3 point. It seems that the amount of shift is not significantly affected by the ratio of the girder's stiffness to slab stiffness, H. It seems that the shift is more dependent of the beam spacing, b, and the b/a ratio. In general, with the exception of beam C, the smaller the beam spacing, the lesser is the shift and the larger the b/a ration, the larger is the shift. There is no apparent relationship between the shifting of the maximum moment location of beam C and either b or b/a. The maximum shifting of maximum moment location is in beam C and the higher the diaphragm's stiffness, the greater is the shift. The following are typical values of shifting when the value of k varies from 0.0 to 0.4.

The point of maximum location is expressed as the fraction of a span length from the simply supported end; with the following values being typical:

For beam A the change is approximately from 0.44 to 0.5

For beam B the change is approximately from 0.42 to 0.38

For beam C the change is approximately from 0.42 to 0.36

The most severely affected bridge is the one with H = 20, b/a = 0.05 and b = 7 ft. The changes are:

Beam A from 0.44 to 0.5

Beam B from 0.42 to 0.38

Beam C from 0.42 to 0.3

The change in maximum moment location for a 4W loading will also change the location of the maximum moment from the truck loading. In order to accomodate this shifting, the profile of the prestressing steel in the prestressed concrete girder may have to be modified.

4.4 Effects of Bridge Parameters on Moment Distribution

Fig. 4.20 to Fig. 4.25 shows the effects of bridge parameters on the moment distributions in the beams. The moment coefficients shown are the maximum moment coefficients due to a 4W (four wheels) loading on a two span continuous bridge without intermediate diaphragms. In these figures, the maximum positive moment coefficients are compared to those of a simply supported bridges. The solid lines are for continuous bridges and the dotted lines are for simply supported bridges. The data points for the simply supported bridges are taken from Ref. 12, Fig. 5.27.

For a beam spacing of 5 ft (Fig. 4.20), it can be seen that the moment coefficients of a continuous bridge for beam A and beam B are not much affected by the relative girder stiffness, H. For beam A, they vary from 0.19 to 0.20 and for beam B, they vary from 0.185 to 0.192. The same trend occurs in the simple supported bridge for beam A. The moment coefficients of the center beam, beam C, increase as H increases and the magnitude of the moment coefficient is a function of both the relative girder stiffness, H, and b/a ratio. It seems that the larger the b/a ratio, the larger the increase in C_m values as H increases. The simply supported bridges also exhibit this monotonic increase in C_m with H and the curves for the simply supported bridge are approximately parallel to those for the continuous ones. If we make a comparison of the moment coefficients between a simply supported bridge and a continuous bridge, we will find that a much larger decrease in moment occurs in beam A than in the interior beams. Table 4.3 shows the percentage decrease in moment coefficients in beams for a continuous bridge relative to a simply supported bridge. The effects of continuity decreases the moment in beam A about 17 percent or more, while for the interior beams the decrease in moment coefficient usually varies from 9 to 15 percent.

Fig. 4.21 shows the effects of H on C_m values for beams A, B, and C when the beam spacing is 7 ft. For a continuous bridge, except when b/a = 0.05, the moment coefficients for beam A decreases as H increases. The moment coefficients increase as H increases for b/a = 0.05. For beams B and C, the moment coefficients always increase as H increases and as b/a decreases. The change in C_m for beam A with H is much smaller as compared to beams B and C. Only in the case when b/a = 0.05 is beam A the controlling beam. When b/a equals to 0.10 and 0.20, the controlling girder is beam B. If we compare the C_m of beam A for continuous bridges and simply supported bridges, we will find that for b/a = 0.05, the C_m of the simply supported bridge stays at a level of 0.3 and is unaffected by H, whereas for a continuous bridge, C_m increases as H increases. When b/a = 0.10, C_m first decreases and then increases with the increase of H. For beam B, the curves for the simply supported bridges are fairly similar to those of the continuous bridge. Table 4.3 shows the percentage decrease in C_m of beam A, varying from 16.6 to 25 percent and for beam B, varying from 10 to 13.6 percent, comparing positive moments in continuous and simply supported spans.

Fig. 4.22 shows the effects of H on the C_m for beams A, B, and C when b equals to 9 ft. For a continuous bridge, when b/a = 0.05, C_m for beam A increases with H and for b/a = 0.1 and 0.2, C_m decreases with H. For a simply supported bridge, C_m for beam A increases with H for both b/a = 0.05 and 0.10. For the interior beams, regardless of the type of bridges, C_m always increases as H increase. It seems that the increase in C_m with H of a simply supported bridge is larger than for the continuous bridge for beam C, otherwise, the trend is quite similar to the case when b equals to 7 ft.

It can be concluded that the edge girder A is the controlling girder only for the cases when b/a = 0.05 for various beam spacing and H value. For other b/a ratios, the interior beams will be the controlling griders.

The decrease in positive moment coefficients because of continuity can be separated into two categories; one for exterior girders and one for the interior girders. The edge girders always experience a larger decrease in positive moment coefficient ranging from 15 to 25 percent with an average of 18.6 percent. For the interior beams the decrease in C_m is smaller, ranging from 9 to 16 percent with an average of 12.7 percent.

The variations of maximum negative moment coefficients at the supports with respect to b, b/a and H are similar to those of the maximum positive moment coefficients. In general, the C_m values for the interior beams, beam B and C, increase with H and b/a ratio (Fig. 4.23 to Fig. 4.25) but the rate of increase is smaller than for the maximum positive moment coefficients. For the edge girder, girder A, the maximum negative moment coefficients are fairly insensitive of the variation of H. The maximum negative moment coefficient occurs when b/a = 0.05, and C_m decreases as b/a increases. When b/a = 0.05, C_m increases as H increases and when b/a = 0.10 and 0.20, C_m either stays constant or decreases slightly.

Recalling that for a two span continuous prismatic beam subjected to a moving point load, the ratio of the maximum positive moment to the maximum negative moment is about 2.17; it can be seen from Table 4.4 that the relationships between the maximum positive and maximum negative moments in a two span bridge are about the same as for the two span prismatic beam in spite of the fact that the loading is different.

4.5 Comparison of the Effects of Diaphragms on Simply Supported and Continuous Bridges

Table 4.5 shows the changes with increasing diaphragm stiffness, in the moment coefficients of the girders due to a 4 wheel loading moving on simply supported and two span continuous bridges with H = 20, b/a = 0.10, and b = 7 ft. The data of the simply supported bridge are taken from Ref. 12, Fig. 4.35. The changes in moment coefficients in the girders are expressed as percentage change relative to the maximum moments in the girders when k = 0.00 (i.e., no diaphragms).

Regardless of the type of bridge, the change in moment coefficients, whether it is an increase or decrease of positive or negative moment at k = 0.05 is about 40 to 60 percent of that when k = 0.4. That is to say that the change in maximum moments in the beams are not very sensitive to the diaphragm's stiffness once the diaphragms are installed. Depending on the beam spacing of the bridge, the effects of diaphragms are more pronounced in a simply supported bridge than in a continuous bridge. As k increases from zero to 0.4, the percentage change in C_m of the edge beam for both the simply supported and continuous bridges are approximately the same. For the interior girders, as k increases the change in C_m of the simply supported bridge is greater than in the continuous bridge. When k reaches the value of 0.4, the percentage change in C_m of the simply supported bridge are approximately twice that of the continuous bridge, especially for the bridges with large beam spacing. The largest change in moment coefficients for both types of bridges occur in the interior beams. At a k value of 0.4, the change in C_m ranges from 20 to 30 percent or more for girder C of a simply supported bridge, and it may be considerably less for girder B and in continuous bridges.

It was shown in Ref. 12 that the effects of the diaphragms decreases at the sections far away from the location of the diaphragm. But it is not necessarily so for the negative moments at the supports. Table 4.5 shows the percentage change in maximum negative moment at the supports. Table 4.5 shows the percentage change in maximum negative moment coefficients as a function of the diaphragm's stiffness. When k = 0.05, the percentage changes in maximum negative moment coefficients are approximately the same as for the maximum positive moment, and are slightly greater than the changes in maximum positive moment when k = 0.4.

4.6 Effects of Diaphragms on Moments from Truck Loadings

Having the influence lines for 4W loadings, the effects of two trucks with AASHO HS load distributions, running in the same direction along the bridge, can be analysed by the method of superposition. The spacings of the axles and the distributions of loads are shown in Fig. 2.3b. When considering the moments due to truck loading, the direction in which the trucks are going will affect the maximum moments in the beams. The absolute maximum moments in the beams due to two trucks running in either direction are designated as the maximum moments. As discussed in Ref. 14, the influence of the diaphragms will decrease as the number of load increases; so only the bridges which are significantly affected by the diaphragms will be discussed. Fig. 4.26 shows the maximum moment coefficients in beams A, B, and C due to two HS truck loadings on the bridges with H = 20, b/a = 0.2, and b = 5, 7, and 9 ft.

For the bridge with b = 5 ft, the length of one span is only 25 ft long and the full length of the truck cannot be parked in one span. Therefore, when considering the maximum positive moment for this case when b = 5 ft, only the two heavy axles of the trucks were considered. The maximum positive moments due to two axiles are slightly greater than those from one axle placed at the point of maximum moment in the moment envelope. There are virtually no changes at all in C_m for girders A and B and only a slight decrease in moment for beam C as k varies from 0.0 to 0.4. The diaphragms just cannot redistribute the loads in beam C to beams A and B.

The maximum positive moment induced in the beams due to 2 trucks running along the bridge are affected by the direction in which the trucks are going. Two solutions, one for the trucks running towards the interior support and one for the truck running away from the interior support were obtained and the maximum positive moments that would be induced in the beams are taken as the maximum moments. When the length of one span is small, the maximum positive moment may be due to the loading of the two rear heavy axles of the trucks alone and the lighter axles are somewhere beyond the pier.

For the bridge with b = 7 ft, the edge beam moment never has the controlling moment when k varies from 0.0 to 0.1. At higher diaphragm stiffnesses, loading due to two heavy axles induce a slightly larger moments in the beams. They are shown as dotted lines in Fig. 4.26. For the bridge with b = 9 ft, the span length is long enough that loading due to the two heavy axles along can no longer develop the maximum positive moment. Two trucks running away from the interior support will induce the absolute maximum positive moments in the beams. In all cases, the diaphragms tend to reduce the moments in the interior beams and increase the moments in the exterior girders. A relative diaphragm stiffness of 0.4 for b = 7 ft and 0.3 for b = 9 ft will help to improve the load distribution of the positive moment of the bridge, reducing the maximum positive live load moments by 10 to 15 percent.

The maximum negative moment at the supports are found by assuming that the two heavy rear axles of the trucks are right on the point of maximum moment in the influence line for moment at the interior supports. By doing so, we have the freedom of varying the spacing of the rear axles as long as they are kept within the limit of 14 to 30 ft. If the spacing of the axles thus found, is more than 30 ft, it is assumed that the two rear axles will have a spacing of 30 ft and they are placed symmetrically with respect to the interior support.

The 30 ft axle spacing limit controlled for the case of girder A when b = 7 ft and for all girders when b = 9 ft.

The graph of moment coefficients of the negative moment at the interior support against k are plotted on Fig. 4.26. For b = 5 ft, the maximum negative moments occur at beam C and for b = 7 and 9 ft, the maximum moments occur at the interior beams. Diaphragms help to redistribute the loads, but there is a limit to their usefulness and their effects on load distribution for negative moment are never as prominent as for positive moment.

4.7 Effects of Diaphragms on Three Span Bridge

In order to get an insight on the effects of diaphragm on multi-span bridges, a three span continuous bridge was analysed. By comparing the results of the analyses of the three span continuous bridge and the two span continuous bridge, a more general conclusion on the effects of diaphragms could be made. A three span continuous bridge with H = 20, b/a = 0.10 and b = 7 ft was analyzed and the maximum positive moment coefficients in each span and the maximum negative moment coefficients at the interior supports due to a 4W loading moving along the bridge were tabulated in Table 4.6.

The maximum positive moment coefficients for all beams of the end span of the three span bridge are slightly lower than in the two span bridge, and the reverse is true for the negative moment at the support. The results suggest that the three span bridge is a little bit stiffer than the two span bridge. The variation of the maximum positive C_m values of the end span of three span bridge with k is the same as for the two span bridge, and there are slightly higher variations in C_m for the maximum negative moment coefficients at interior supports and the maximum positive moment coefficient at interior span.

The trend of the maximum positive moment coefficients for the interior span is a little different from the end span. Fig. 4.27 shows the relation of C_m against k for the maximum positive moments and maximum negative moments. It can be seen that the change in C_m of the center span when k varies from zero to 0.40 are slightly greater than that in the end span. Physically, the results indicate that the end span of the three span continuous bridge is less sensitive to the effects of the diaphragms than is the interior span. This may be due to the increase in the effective b/a ratio and the concurrent increase in the H value because of the effects of the negative moments at the interior supports.

The influence lines for the moment at the supports and at the center of each span are shown in Fig. 4.28 and Fig. 4.29 and their trends are comparable to the two span continuous bridge. Thus it can be concluded that most probably, the effects of diaphragms on a multispan bridge are not much different than that of the two span continuous bridge.

Chapter 5

COMPARISON OF AASHO DESIGN MOMENT COEFFICIENTS WITH THEORETICAL DESIGN MOMENT COEFFICIENTS

Comparison of the beam design moment coefficients for slab and girder bridges from the 1969 AASHO specifications, Section 1.3.1, and the coefficients as obtained from this investigation are shown in Figures 5.1 to 5.4. The loading applied on the bridge is the 4 wheel loading rather than a two-truck loading. For the edge beams, the data points for b/a = 0.05 give the upper bound for the maximum positive moment coefficients for all beam spacings studied; and b/a = 0.2 gives the lower bound for the maximum positive moment coefficients. The AASHO is conservative for the beam spacing of 5 ft, but is unconservative for beam spacing of 7 and 9 ft. The calculated C_m values are approximately 9.6 percent and 5.1 percent higher than AASHO predicted C_m values for b = 7 and 9 ft respectively. The percentage values are based on the AASHO recommended C_m values.

The interior beams positive moment coefficients show a much larger scattering than the edge beam moments, when plotted verses the beam spacing, b. For b = 5 ft, the data points for beam C have a much larger scattering than beam B while for b = 7 and 9 ft, the scattering for beam C is only slightly greater than for beam B. The large scattering of data points indicates that the C_m values cannot be solely predicted by one parameter, the beam spacing, alone. The C_m values as calculated from AASHO are quite unconservative for some bridges. As beam spacing decreases, the differences between the maximum positive moment coefficients and the AASHO C_m values increase. At b = 5 ft, the maximum difference is about 31.9 percent greater than the AASHO value and at b = 7 and 9 ft the maximum differences are approximately 13.6 percent and 9.5 percent higher, respectively. It can be seen that most of the points that fall above the AASHO line are those with b/a = 0.1 and 0.2.

The behavior of the maximum negative moment coefficients are approximately the same as the maximum positive moment coefficients except that less scattering is observed in the interior beams moment values. The AASHO recommended C_m values for the edge beams are conservative for the beam spacing of 5 ft only and they are generally unconservative for beam spacing of 7 and 9 ft. The upper bound for the C_m values are those of b/a = 0.05 and the lower bound are those for b/a = 0.2. The maximum differences of the calculated C_m values to AASHO C_m values are approximately 14.2 and 11.7 percent higher for beam spacing of 7 and 9 ft. For the interior beams the AASHO recommended C_m values are unconservative for many bridges. The trend is similar to that of the positive moment coefficients with the maximum deviation from the AASHO line of 36.7, 16.5 and 13 percent for beam spacing of 5, 7 and 9 ft.

The comparison cited above is only confined to a 4 wheel loading and the results may not necessarily reflect the load distribution of truck loadings.

While it appears that some changes should be made in the AASHO load distribution values, this study has not been comprehensive enough to develop design recommendations.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

From the analyses of continuous bridges with various diaphragm stiffness and various bridge properties, the conclusions can be summarized as follows:

The effects of continuity:

1. The variation in the maximum positive moments of simply supported bridges due to the effects of the diaphragms is larger than in continuous bridges. The load distribution (of a 4-wheel loading) in continuous bridges without diaphragm is similar to that in simply supported bridges without diaphragms. The effects of continuity tend to cause a greater reduction in maximum positive moment in the edge girder, beam A; with an average reduction of 19 percent, than in the interior girders, with an average reduction of 13 percent, as compared to simply supported bridges of the same span, beam spacing, and H values.
2. The effects of continuity greatly stiffen the bridges.
3. The average ratio of the maximum positive moment, due to 4-wheel loads, to maximum negative moment at the support is approximately 2.17 (range: 2.00 to 2.38). This ratio is about the same as in a continuous beam.

The effects of diaphragms on moments due to 4W loadings:

1. Diaphragms are always helpful in reducing the maximum moments in the loaded girders if the load is a single point load.

2. For bridges with b/a = 0.20, the maximum moment always occurs in beam C, regardless of the beam spacing and loading condition. Stiff midspan diaphragms, with a relative diaphragm stiffness of 0.40 or above, are always helpful in improving the load distribution of the bridge especially with large, beam spacings. The reductions of maximum moment range from 5 to 24 percent, with an average of 12 percent.
3. For bridges with b/a = 0.10, a flexible midspan diaphragm in each span may be helpful in improving the load distribution. For H = 10 the optimum relative diaphragm stiffness is approximately from 0.03 to 0.05, giving an average reduction of maximum moment of 2 percent, and for H = 20, the optimum diaphragm stiffness is from 0.05 to 0.08, with an average reduction of maximum moment of 6 percent.
   As the diaphragm stiffness increases beyond the optimum stiffness, the maximum moment in the exterior beams will increase to values greater than the absolute maximum moments in the beams of the bridge without diaphragms.
4. Bridges with b/a equal to or less than 0.05 do not need any diaphragm. Diaphragms will do more harm than good in these bridges.
5. By adding a diaphragm at midspan of each span of a two span continuous bridge, the location of the maximum positive moment will be displaced. For the edge girders, the locations of maximum positive moments will tend to go towards midspan and for the interior girders, the points of maximum moments

will tend to go away from the midspan towards the 0.3 point, from the simply supported end.

6. Midspan moments in the unloaded span of two span continuous bridges are much more evenly distributed than in the loaded span. For b/a smaller than 0.1, the moments in the girders are nearly independent of diaphragm stiffness. The midspan moments in the interior beams are always less than 50 percent of the maximum negative moments at the interior supports, while the midspan moments in the edge girders may be less than or equal to 50 percent of the interior support moments, depending on the diaphragm stiffness. For the bridge with fairly bad load distribution characteristics (i.e., H = 20, b/a = 0.2), the moment distribution at the midspan of the unloaded span is much more uniform than that at the support. The increase in diaphragm stiffness tends to decrease the moments in the interior girders and raise the moments in the edge girders. The moments in the interior girders of these bridges are always less than half of the moment at the interior support and the moments in the edge girders are always greater than half of the moment at the interior support.

The results of the analysis of a three-span continuous bridge show that the three span bridge is a stiffer structure than the two span continuous bridge. The maximum positive moments in the end span of the three span bridge are slightly less than that of the two span bridge. The maximum negative moments at the interior supports are slightly higher for the three span bridge than the two span bridge. The effects of diaphragms on the three span bridge are primarily the same as those in the two span bridge, although the change in the maximum positive moments in the center span as a function of the diaphragm's stiffness is slightly higher than that of the end span. The optimum relative diaphragm stiffness of the three span continuous bridge with H = 20, b/a = 0.10, and b = 7 ft is approximately 0.07, which is slightly higher than for a similar two span bridge. In general, the behavior of the three span bridge is similar to the two span continuous bridge.

The comparison of the AASHO recommended C_m values for 4 wheel loading with the calculated C_m values for the positive and negative moments are only conservative for the edge beams with 5 ft beam spacing. The AASHO C_m values are generally unconservative for other cases.

The large scattering of data points for C_m values indicates that the distribution coefficient cannot be based on the beam spacing alone. Besides b, the beam spacing, both H and b/a are important parameters but it seems that a distribution factor as a function of b and b/a will give a more realistic value.

According to AASHO recommendation, diaphragms should be installed in bridges with spans more than 40 ft. Although flexible diaphragms, with relative stiffness of 0.05, may slightly improve the load distribution characteristics in some of the bridges with small b/a ratio, such diaphragms are seldom practical in terms of cost effectiveness, and it would be more economical to build these bridges without diaphragm. The results of this investigation show that only bridges with large b/a ratio could benefit from the addition of diaphragms. For these bridges, it generally would be more economical to increase the number of prestressing strands in the prestressed concrete girders to resist the load than to use diaphragms to distribute the loads to other beams. Therefore, unless necessary for temporary erection purpose, it is recommended that diaphragms should not be installed in straight highway bridges, whether they are simply supported or continuous.

Chapter 7

SUMMARY

The effects of diaphragms and continuity on load distribution of five beam, two and three span continuous bridges were studied. The method of analysis was based on Fourier harmonic analysis. For the complete analysis, four computer programs running on IBM 360-75 were used. Given the geometric and member properties of the slabs and girders, the first two computer programs will calculate all the internal forces in the girders of the bridge under point loads. The third program combines the point loads by superposition to simulate the desired loading condition and finds the maximum moments in the girders due to the specified loading. The fourth program then plots out the influence lines.

The diaphragms are assumed to provide torsional and shearing restraint at the junction of the girders and the diaphragms, and the girder supports are assumed to be nonyielding. The major concern in this investigation is the effects of diaphragms on load distribution in the slab and girder highway bridges. Based on the investigation of the effects of diaphragms on simply supported bridges, the selected parameters for this study are H, the relative stiffness of the girders to the slabs; b/a, the aspect ratio; b, the beam spacing; and k, the relative stiffness of the diaphragms. The range of these parameters are: H from 5 to 20, b/a from 0.05 to 0.2, b from 5 ft to 9 ft, and k from 0.0 to 1000.0. The criterion of comparison is, in general, the maximum moments in the girders, both positive and negative, as produced by the 4-wheel concentrated loading, and the truck loading. The effects of the location and stiffness of diaphragms on the load distribution of a point load was studied. The results of this preliminary investigation indicated that the best location of the diaphragm would be a midspan diaphragm in each span and for the sake of studying the effects of diaphragm, the variation of k from 0.0 to 0.4 was adequate.

With each bridge geometry, diaphragm stiffness, and girder property, the effects of diaphragms under a loading consisting of four point loads spaced according to the AASHO recommended wheel spacing were studied. The variation of the maximum moments, both positive and negative in the girders with various diaphragms stiffnesses were studied. A comparison of the simply supported bridges as reported in Ref. 12 with the two span continuous bridges was also made. The analyses also gave the influence lines for moments at various sections due to the 4-wheel loading. With this information, the effects of diaphragms on the maximum moments in the girders due to truck loadings were also investigated. Reference was made to the AASHO recommended design C_m values for both positive and negative moment for girders due to a 4-wheel loading, and the AASHO recommended C_m values was compared to the calculated C_m values as obtained from this investigation.

A three span continuous bridge was analyzed to give an insight to the load distribution in the multi-span bridge. In the discussion, only the moments in the girders are considered, but the internal forces in the girders are also available from the computer output. It is concluded that except for temporary erection purposes, diaphragms are not required in straight highway bridges.

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Table 2.1

COMBINATIONS OF PARAMETERS FOR THE STUDIES OF EFFECTS OF DIAPHRAGMS IN TWO SPAN CONTINUOUS BRIDGES

H	b/a	b (ft)	a (ft)	Slab Thickness (in.)
5	0.05	5	100	7.25
		7	140	6.00
		9	180	7.75
	0.10	5	50	7.00
		7	70	7.25
		9	90	8.00
	0.20	5	25	9.50
		7	35	9.50
		9	45	7.50
10	0.05	5	100	6.00
		7	140	6.25
		9	180	6.50
	0.10	5	50	7.25
		7	70	6.00
		9	90	7.75
	0.20	5	25	7.00
		7	35	7.25
		9	45	8.00
20	0.05	5	100	6.50
		7	140	7.75
		9	180	6.25
	0.10	5	50	6.00
		7	70	6.25
		9	90	6.50
	0.20	5	25	7.25
		7	35	6.00
		9	45	7.75

Table 4.1

MAXIMUM POSITIVE MOMENT COEFFICIENTS
DUE TO 4W LOADING

H	b/a	b (ft)	Beam A k = 0.0	Beam A k = 0.4	Beam B k = 0.0	Beam B k = 0.4	Beam C k = 0.0	Beam C k = 0.4
5	0.05	5	0.198	0.201	0.183	0.182	0.178	0.177
		7	0.241	0.248	0.223	0.221	0.194	0.189
		9	0.277	0.282	0.247	0.246	0.210	0.204
	0.10	5	0.196	0.202	0.186	0.186	0.192	0.187
		7	0.243	0.257	0.243	0.236	0.225	0.207
		9	0.285	0.297	0.276	0.268	0.259	0.234
	0.20	5	0.190	0.193	0.185	0.190	0.214	0.204
		7	0.228	0.243	0.270	0.253	0.265	0.241
		9	0.269	0.286	0.316	0.292	0.315	0.281
10	0.05	5	0.200	0.206	0.186	0.184	0.183	0.181
		7	0.248	0.261	0.233	0.229	0.206	0.195
		9	0.287	0.297	0.261	0.257	0.227	0.216
	0.10	5	0.196	0.207	0.189	0.190	0.203	0.194
		7	0.241	0.264	0.257	0.243	0.248	0.215
		9	0.285	0.305	0.293	0.278	0.288	0.246
	0.20	5	0.191	0.194	0.189	0.193	0.233	0.215
		7	0.225	0.268	0.287	0.250	0.283	0.221
		9	0.265	0.292	0.347	0.306	0.346	0.298
20	0.05	5	0.200	0.210	0.188	0.186	0.189	0.184
		7	0.252	0.272	0.242	0.235	0.219	0.201
		9	0.289	0.306	0.272	0.265	0.248	0.226
	0.10	5	0.193	0.208	0.191	0.192	0.214	0.200
		7	0.236	0.269	0.272	0.248	0.270	0.221
		9	0.281	0.311	0.315	0.287	0.320	0.257
	0.20	5	0.192	0.195	0.193	0.195	0.251	0.222
		7	0.220	0.246	0.302	0.270	0.298	0.259
		9	0.259	0.294	0.371	0.313	0.368	0.307

Table 4.2

MAXIMUM NEGATIVE MOMENT COEFFICIENTS
DUE TO 4W LOADING

H	b/a	b (ft)	Beam A		Beam B		Beam C	
			k = 0.0	k = 0.4	k = 0.0	k = 0.4	k = 0.0	k = 0.4
5	0.05	5	-0.093	-0.095	-0.087	-0.087	-0.076	-0.075
		7	-0.119	-0.121	-0.102	-0.101	-0.087	-0.084
		9	-0.138	-0.138	-0.110	-0.110	-0.092	-0.089
	0.10	5	-0.088	-0.092	-0.089	-0.088	-0.082	-0.080
		7	-0.113	-0.120	-0.112	-0.109	-0.107	-0.096
		9	-0.135	-0.139	-0.126	-0.122	-0.123	-0.107
	0.20	5	-0.085	-0.085	-0.085	-0.089	-0.097	-0.092
		7	-0.102	-0.108	-0.127	-0.119	-0.125	-0.115
		9	-0.121	-0.129	-0.150	-0.136	-0.149	-0.134
10	0.05	5	-0.093	-0.096	-0.089	-0.087	-0.077	-0.076
		7	-0.120	-0.124	-0.107	-0.105	-0.095	-0.087
		9	-0.141	-0.142	-0.115	-0.114	-0.104	-0.095
	0.10	5	-0.088	-0.093	-0.090	-0.089	-0.088	-0.084
		7	-0.110	-0.121	-0.119	-0.112	-0.118	-0.099
		9	-0.133	-0.141	-0.136	-0.126	-0.137	-0.112
	0.20	5	-0.086	-0.086	-0.087	-0.090	-0.107	-0.097
		7	-0.101	-0.119	-0.134	-0.116	-0.132	-0.100
		9	-0.118	-0.130	-0.164	-0.142	-0.161	-0.140
20	0.05	5	-0.092	-0.097	-0.090	-0.088	-0.080	-0.077
		7	-0.120	-0.127	-0.111	-0.107	-0.103	-0.089
		9	-0.140	-0.144	-0.121	-0.118	-0.116	-0.100
	0.10	5	-0.087	-0.094	-0.088	-0.089	-0.095	-0.088
		7	-0.107	-0.122	-0.126	-0.114	-0.127	-0.101
		9	-0.129	-0.142	-0.148	-0.130	-0.150	-0.116
	0.20	5	-0.087	-0.086	-0.088	-0.090	-0.117	-0.100
		7	-0.099	-0.109	-0.140	-0.125	-0.137	-0.122
		9	-0.116	-0.130	-0.173	-0.145	-0.169	-0.143

Table 4.3

EFFECTS OF CONTINUITY ON THE REDUCTION OF MAXIMUM POSITIVE MOMENT COEFFICIENTS IN PERCENT IN BEAMS RELATIVE TO MOMENTS IN SIMPLY SUPPORTED BRIDGES

b	H	b/a = 0.05	b/a = 0.10	b/a = 0.20	Maximum Moment Occurs in Beam
5	20	16.6,* 14.0	17.0,* 17.5	8.7	
	10	16.6,* 14.8	17.0,* 13.1	8.7	A and C
	5	23.5*	14.7	12.6	
7	20	16.6,* 10.7	25.3,* 13.3	13.8	
	10	16.6,* 10.0	19.3,* 11.6	13.6	A and B
	5	19.8*	11.2	13.4	
9	20	16.6,* 12.0	21.6,* 12.5	13.7	
	10	14.6,* 12.4	19.6,* 12.2	10.9	A and B
	5	16.1	18.6	12.5	

* Reduction in controlling moment in Beam A.

All other values are reductions in controlling interior beam moments.

Table 4.4

RATIO OF MAXIMUM POSITIVE MOMENT COEFFICIENTS TO MAXIMUM NEGATIVE MOMENT COEFFICIENTS, 4W LOADING

H	b/a	b (ft)	Beam A	Beam B	Beam C
5	0.05	5	2.13	2.10	2.34
		7	2.03	2.19	2.22
		9	2.00	2.24	2.28
	0.10	5	2.22	2.09	2.34
		7	2.15	2.17	2.10
		9	2.11	2.19	2.11
	0.20	5	2.23	2.17	2.20
		7	2.24	2.13	2.12
		9	2.22	2.11	2.11
10	0.05	5	2.15	2.08	2.38
		7	2.06	2.18	2.17
		9	2.04	2.27	2.18
	0.10	5	2.23	2.10	2.30
		7	2.19	2.16	2.10
		9	2.14	2.15	2.10
	0.20	5	2.22	2.17	2.18
		7	2.23	2.14	2.14
		9	2.25	2.12	2.15
20	0.05	5	2.17	2.09	2.36
		7	2.10	2.18	2.13
		9	2.06	2.24	2.14
	0.10	5	2.22	2.17	2.25
		7	2.20	2.16	2.13
		9	2.18	2.12	2.13
	0.20	5	2.20	2.19	2.15
		7	2.20	2.16	2.11
		9	2.23	2.14	2.18

Table 4.5

COMPARISON OF THE EFFECTS OF DIAPHRAGMS ON SIMPLY SUPPORTED BRIDGE AND CONTINUOUS BRIDGES SUBJECTED TO 4W LOADING

	Beam	b (ft)	$C_{mk=0}$	$\delta_k = 0.05$	$\delta_k = 0.4$
Simply Supported Bridge H = 20, b/a = 0.1 Positive Moments	A	5	0.232	+ 3.4	+ 8.6
	B	5	0.257	- 9.7	- 21.0
	C	5	0.257	- 9.7	- 21.0
	A	7	0.290	+ 5.9	+ 13.8
	B	7	0.300	- 5.7	- 12.6
	C	7	0.319	- 12.2	- 29.5
	A	9	0.345	+ 5.5	+ 12.5
	B	9	0.359	- 9.2	- 16.2
	C	9	0.361	- 14.4	- 36.2
Two Span Continuous H = 20, b/a = 0.1 Positive Moments	A	5	0.193	+ 2.1	+ 7.7
	B	5	0.191	+ 0.5	+ 0.5
	C	5	0.214	- 3.8	- 6.5
	A	7	0.236	+ 5.5	+ 14.1
	B	7	0.272	- 4.4	- 8.8
	C	7	0.270	- 7.8	- 18.1
	A	9	0.281	+ 5.0	+ 10.7
	B	9	0.315	- 5.4	- 8.9
	C	9	0.320	- 9.1	- 19.7
Two Span Continuous H = 20, b/a = 0.1 Negative Moment at Support	A	5	0.087	+ 1.1	+ 8.0
	B	5	0.088	+ 2.3	+ 1.1
	C	5	0.095	- 3.2	- 7.4
	A	7	0.107	+ 4.7	+ 14.0
	B	7	0.126	- 4.0	- 9.5
	C	7	0.127	- 7.1	- 20.5
	A	9	0.129	+ 3.9	+ 10.0
	B	9	0.148	- 6.8	- 12.2
	C	9	0.150	- 8.0	- 22.7

$$\delta_{k=0.05} = \frac{C_{mk=0.05} - C_{mk=0.0}}{C_{mk=0.0}} \times 100$$

$$\delta_{k=0.40} = \frac{C_{mk=0.4} - C_{mk=0.0}}{C_{mk=0.0}} \times 100$$

Table 4.6

THREE SPAN CONTINUOUS BRIDGE H = 20, b/a = 0.1, b = 7 FT
MAXIMUM MOMENTS COEFFICIENTS DUE TO 4W LOADING

	Beam	k = 0.0	$\delta_{k=0.05}$	$\delta_{k=0.40}$
Maximum Positive Moment in End Span	A	0.230	+ 5.2	+ 13.5
	B	0.265	- 4.5	- 7.9
	C	0.263	- 6.5	- 17.5
Maximum Positive Moment in Interior	A	0.193	+ 6.7	+ 16.6
	B	0.229	- 6.9	- 11.4
	C	0.228	- 8.8	- 21.5
Maximum Negative Moment at Support	A	0.112	+ 6.3	15.2
	B	0.130	- 4.6	9.2
	C	0.131	- 7.6	21.4

$$\delta_{k=0.05} = \frac{C_{mk=0.05} - C_{mk=0.0}}{C_{mk=0.0}} \times 100$$

$$\delta_{k=0.40} = \frac{C_{mk=0.40} - C_{mk=0.0}}{C_{mk=0.0}} \times 100$$

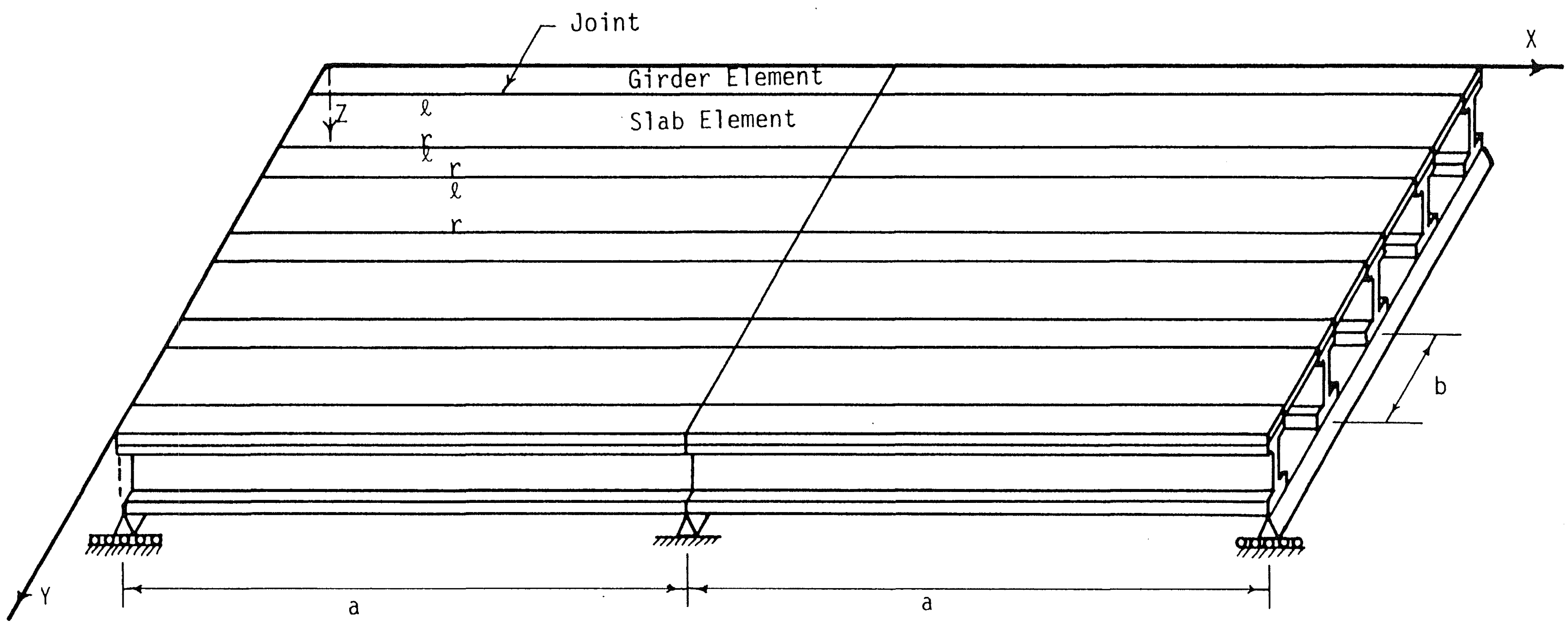


FIG. 2.1 LAYOUT OF THE BRIDGE

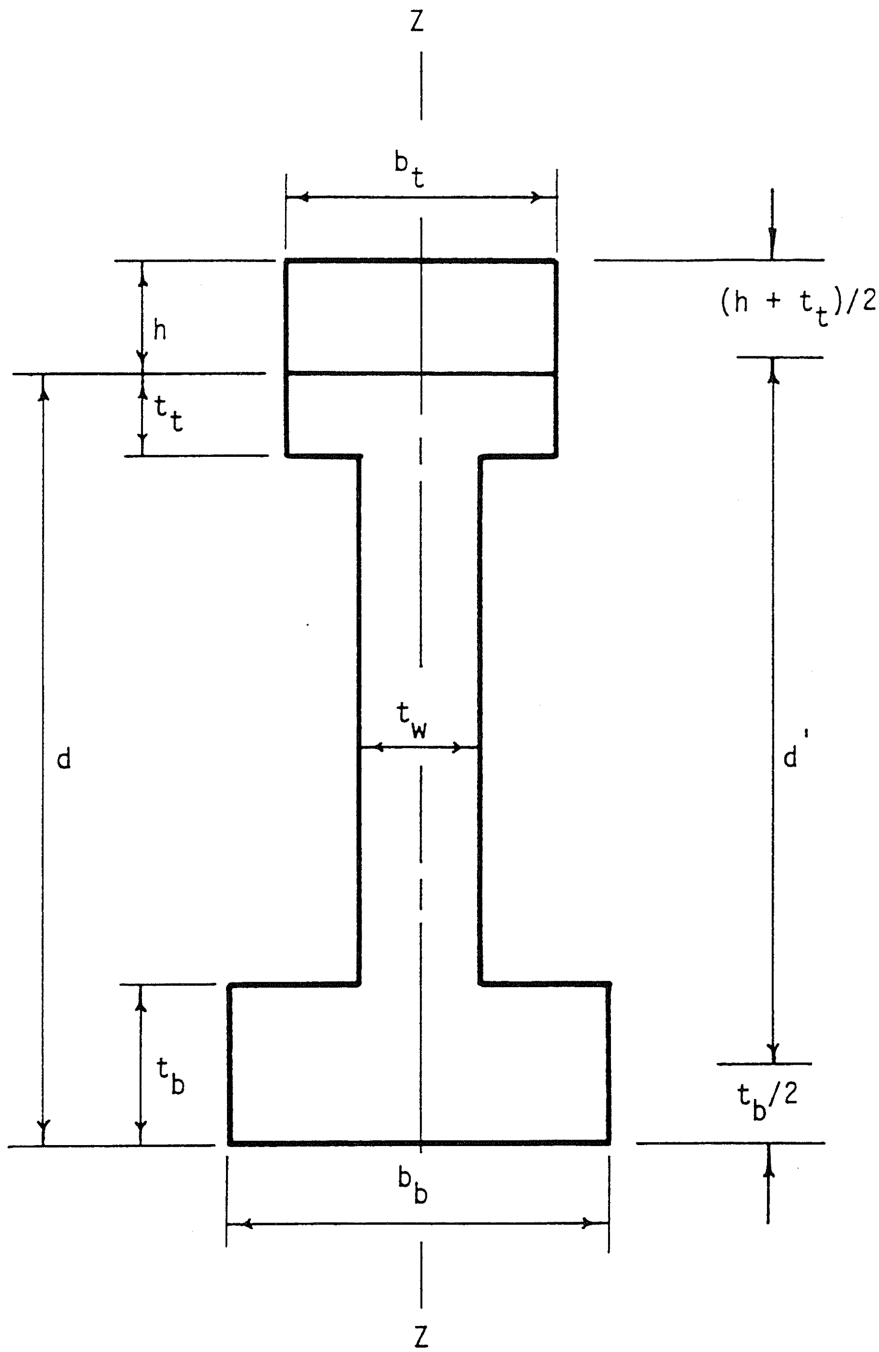
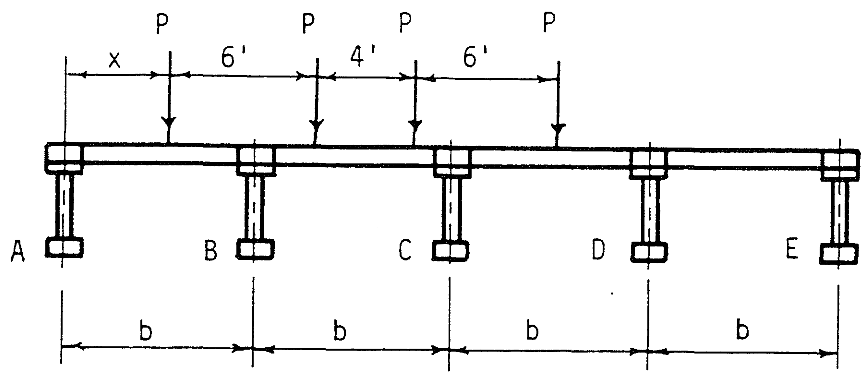
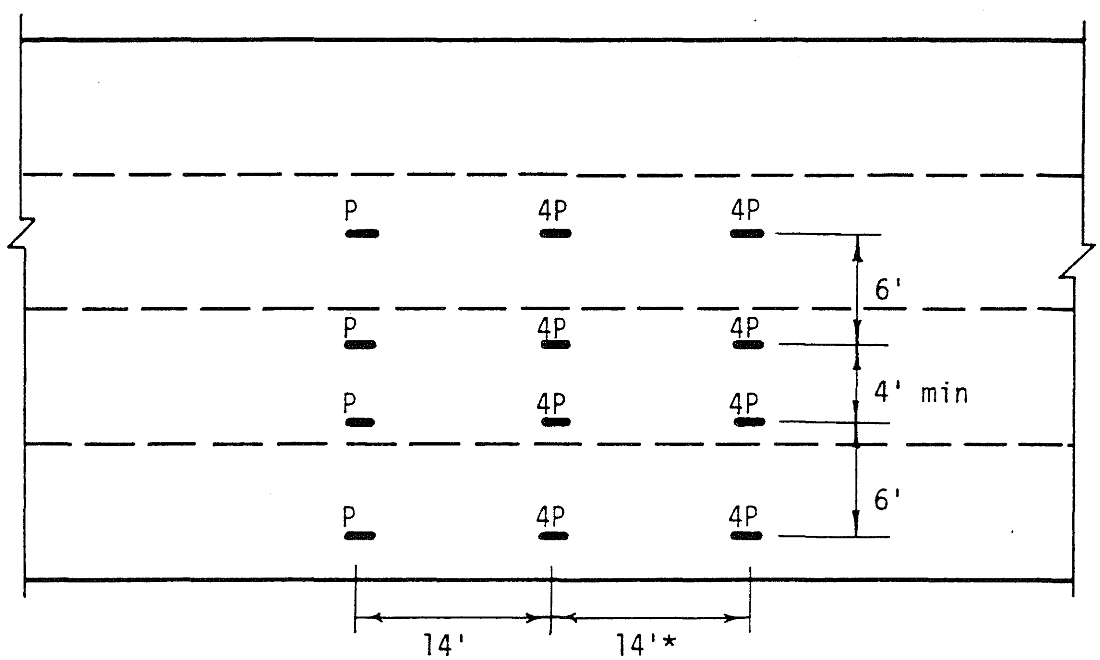


FIG. 2.2 IDEALIZATION OF THE GIRDER CROSS SECTION



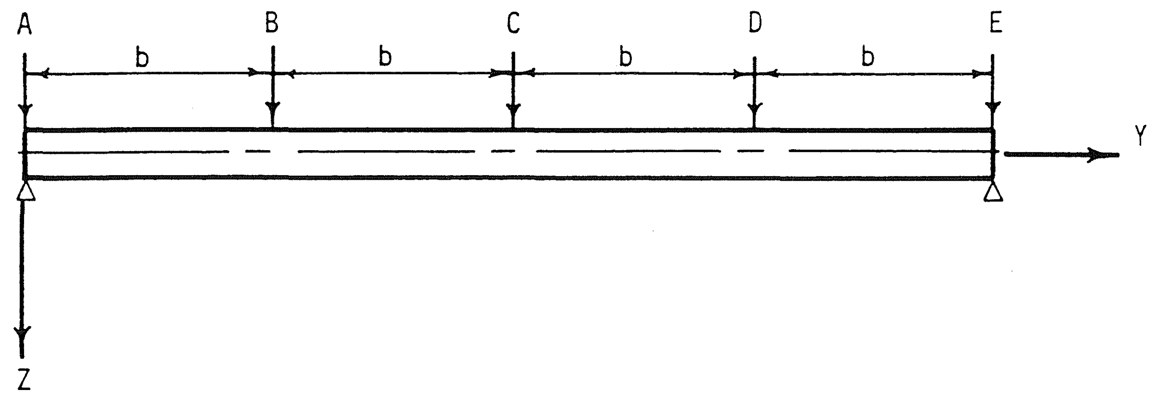
a. Position of 4 Wheel Loads at Distance x from Edge Beam



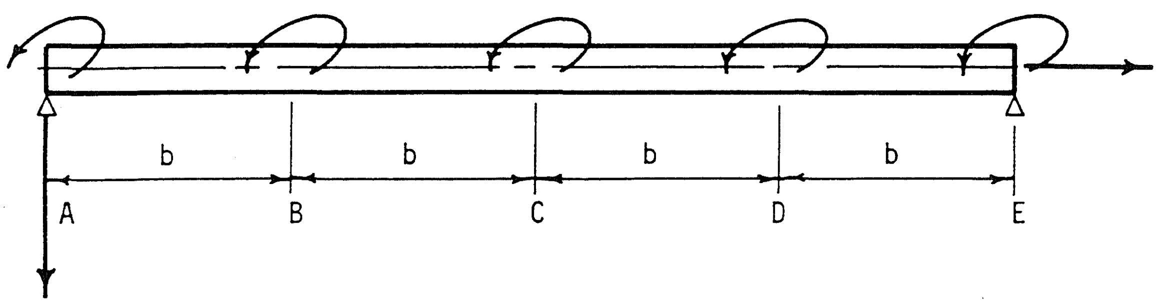
* Axle spacing variable, 14 to 40 ft.

b. Wheel Spacing of Two HS Truck Loadings

FIG. 2.3 POSITION OF LOADS REPRESENTING TRUCKS

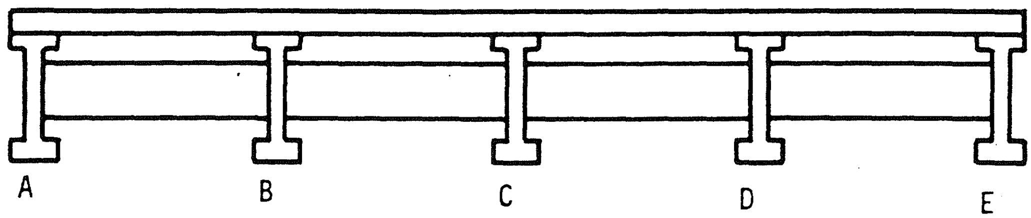


a. Vertical Forces Acting on Diaphragm

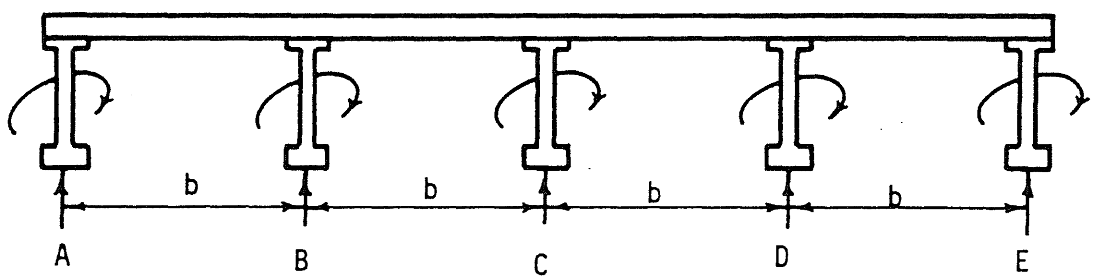


b. Couples Acting on Diaphragm

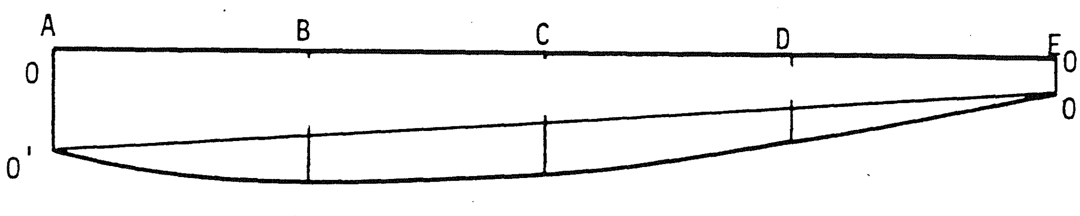
FIG. 3.1 FORCES AND COUPLES ACTING ON DIAPHRAGM



a. Cross Section of Bridge with Diaphragm

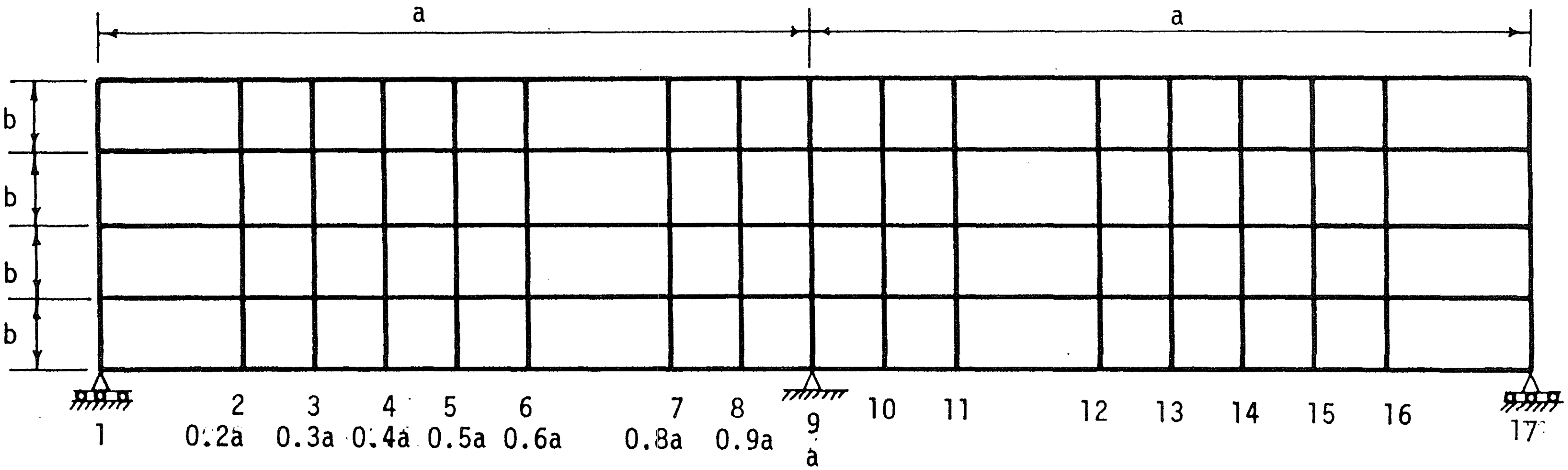


b. Forces and Couples Replacing Diaphragm or Support

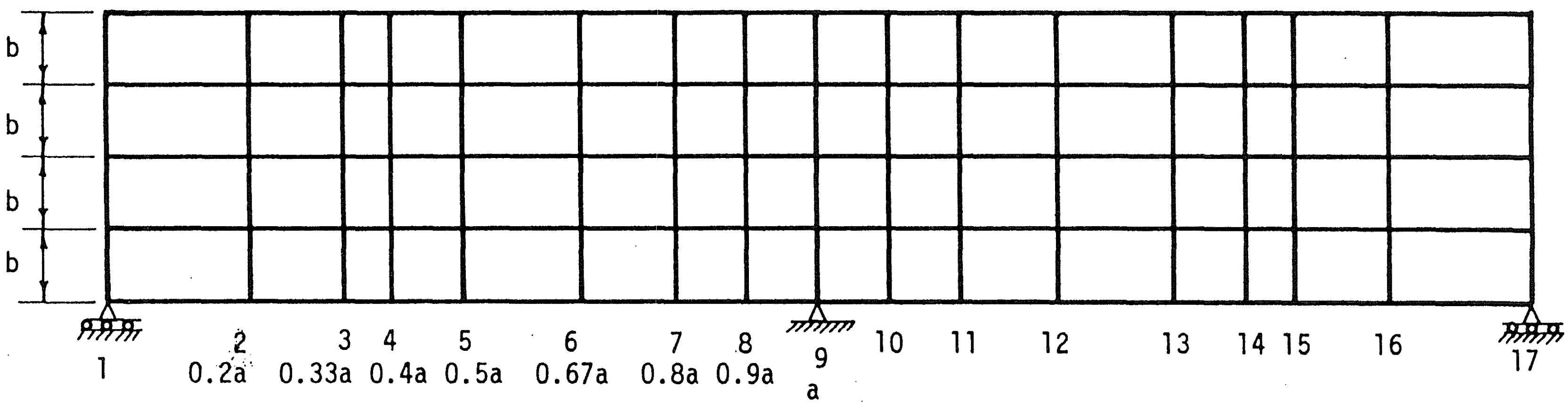


c. Displacement of Cross Section of Bridge

FIG. 3.2 FORCES AND DISPLACEMENTS AT CROSS SECTION CONTAINING DIAPHRAGM

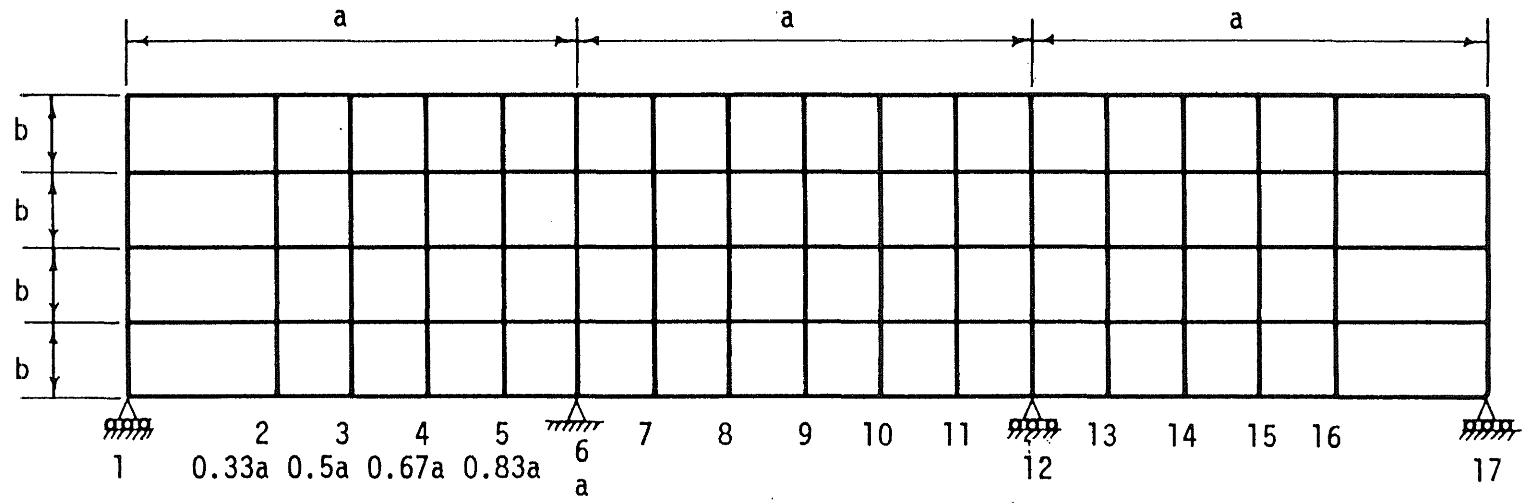


a. Sections Along the Bridge Where Internal Forces Are Computed for One Diaphragm at 4/10 or 5/10 Point of Spans



b. Sections Along the Bridge Where Internal Forces Are Computed for Two Diaphragms at One-Third Points of Spans

FIG. 3.3 LOCATIONS OF SECTIONS CONSIDERED FOR INFLUENCE COEFFICIENTS



c. Sections Along the Bridge Where Internal Forces Are Computed, Three Span Continuous Bridge

FIG. 3.3 (CONTINUED)

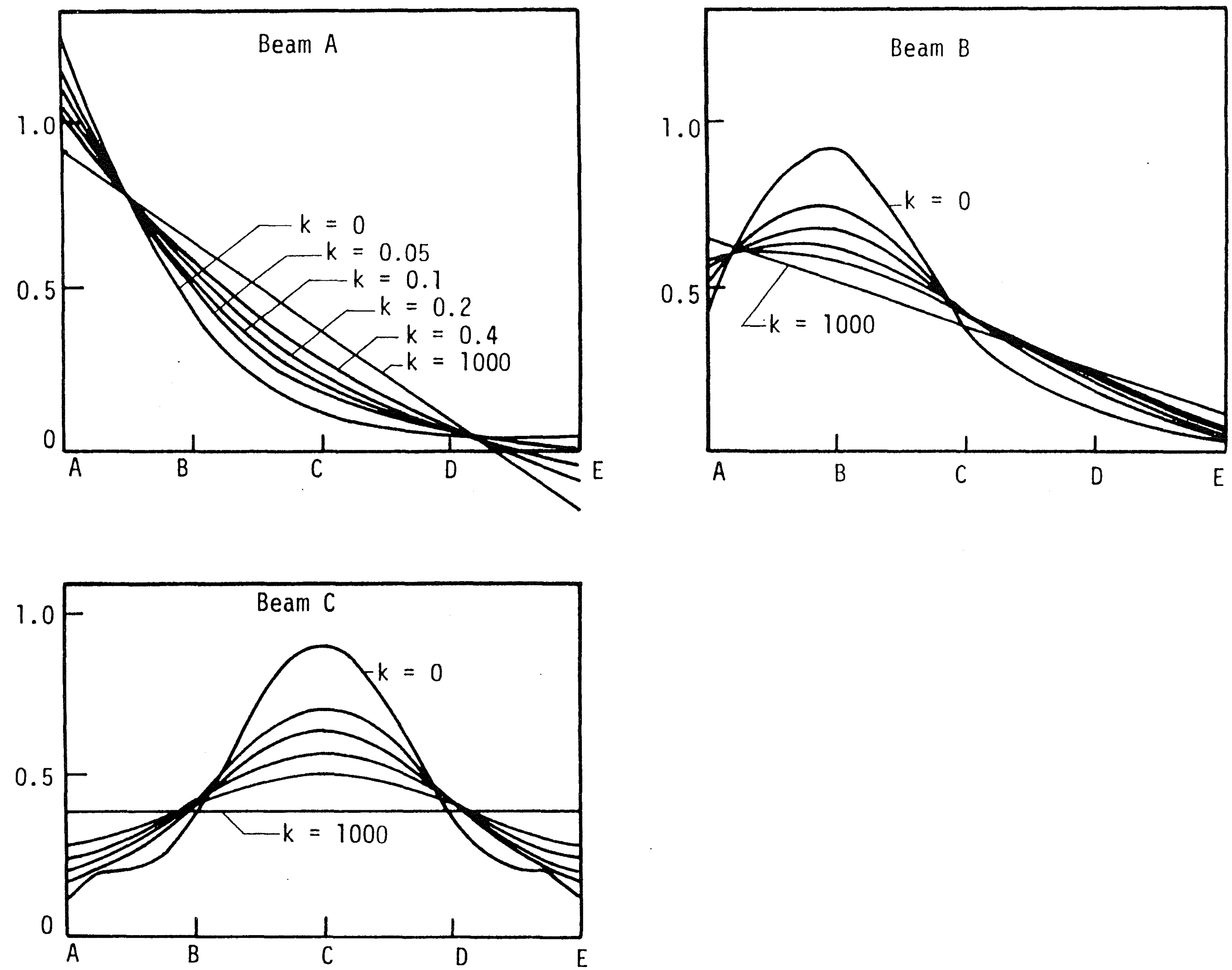
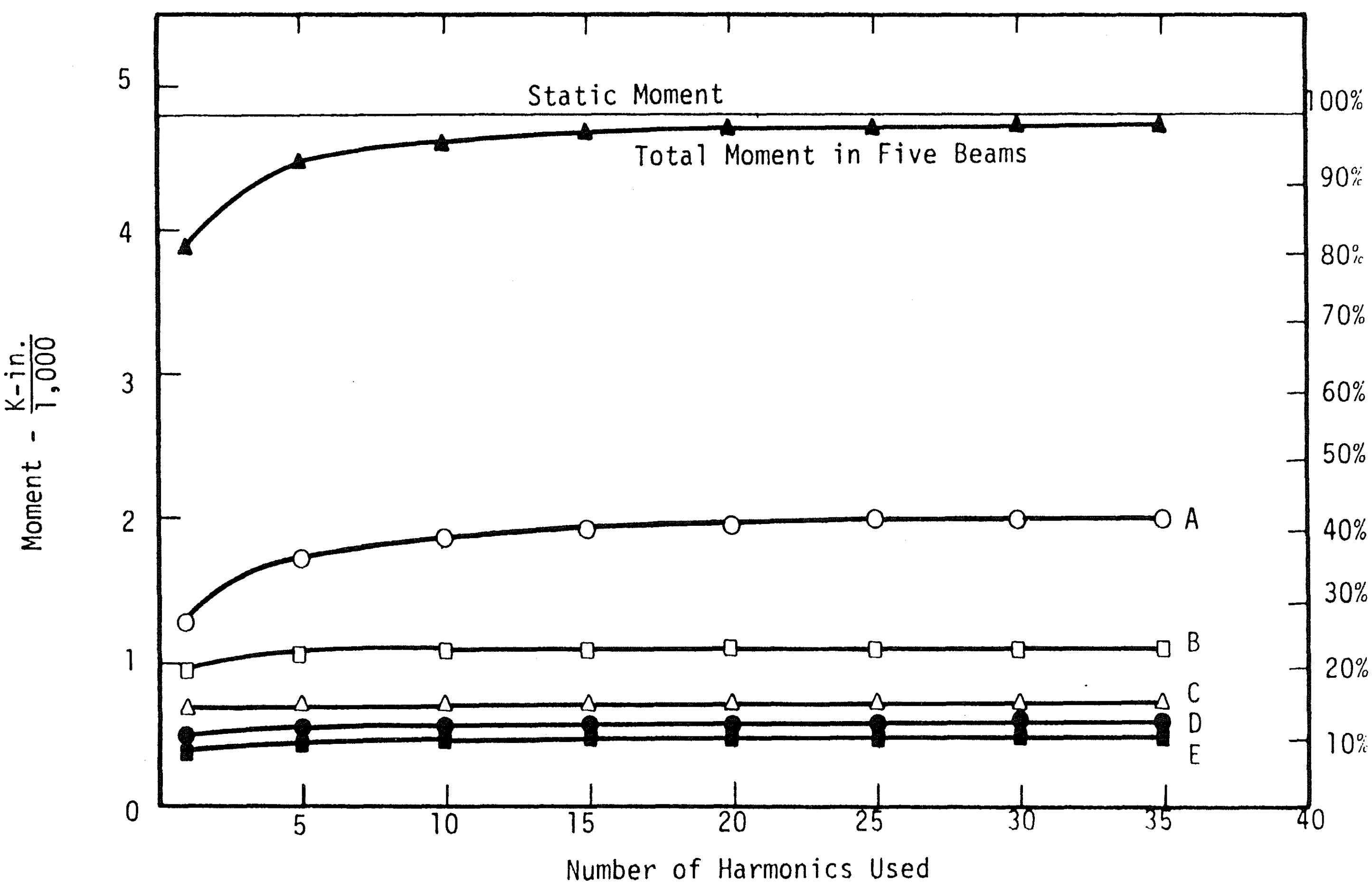
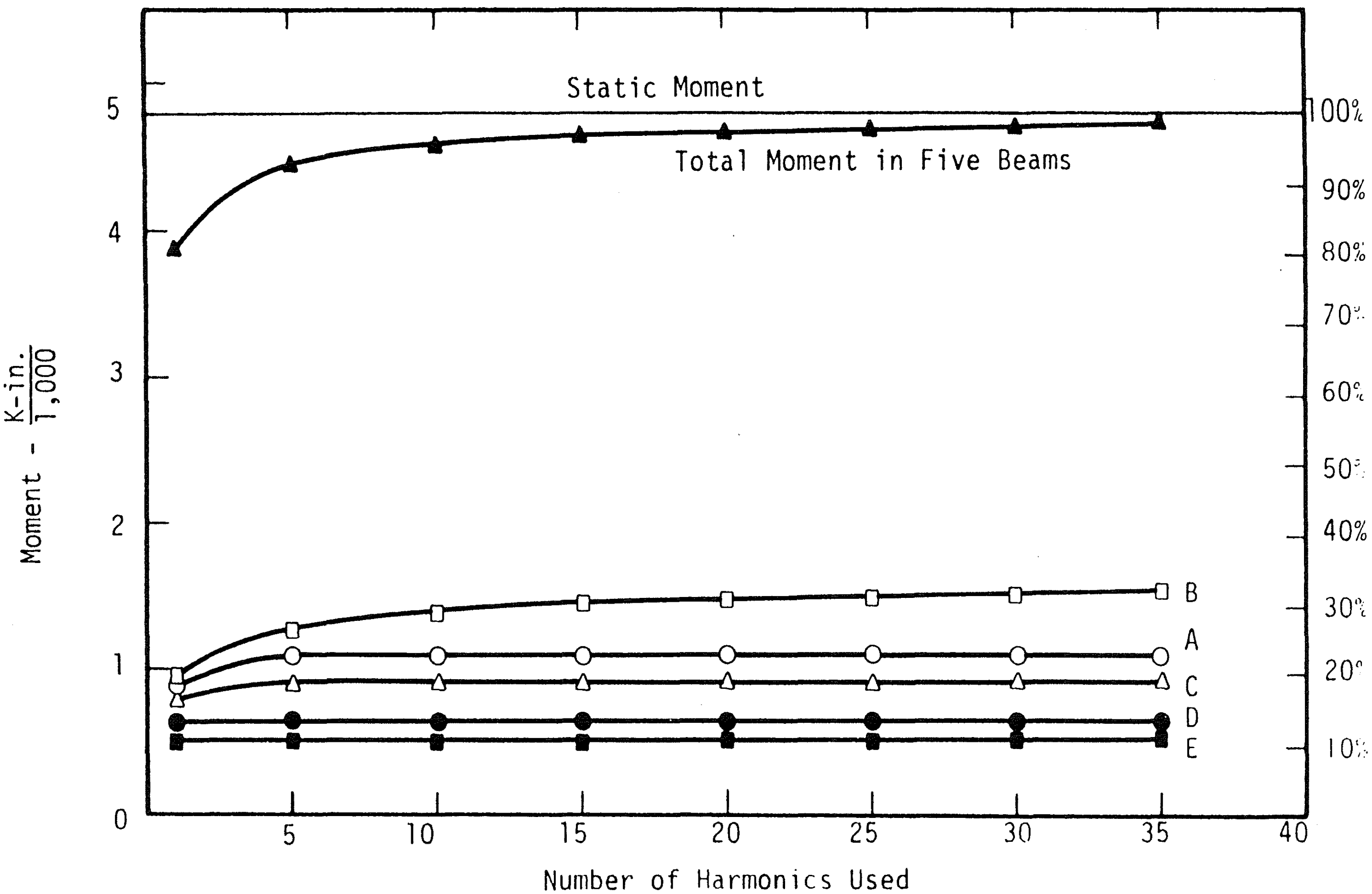


FIG. 3.4 GRAPHS TO SHOW THE CURVE FITTING OF INFLUENCE COORDINATES AT A, B, C, D, E



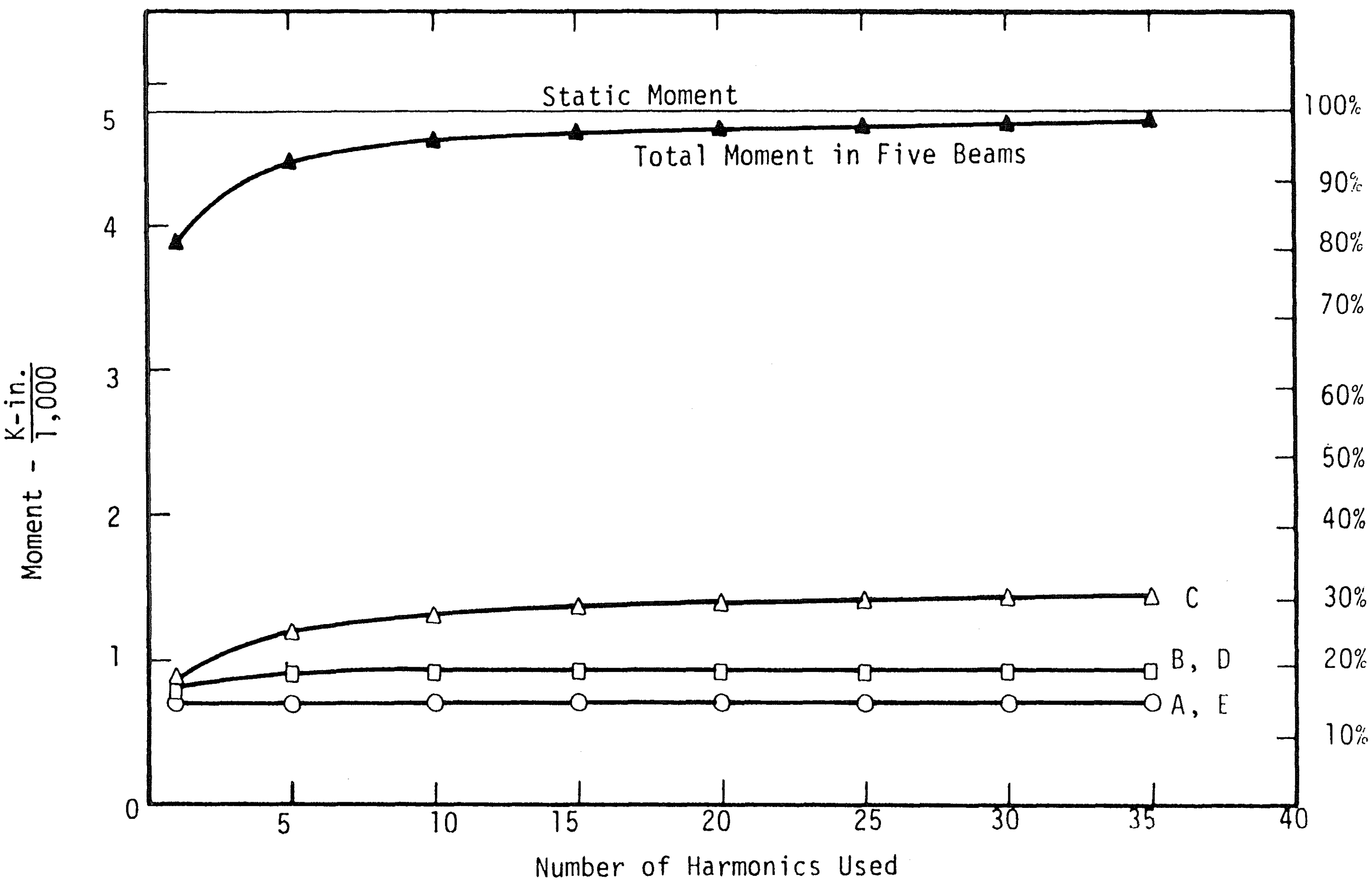
10 kip Concentrated Load at Midspan of Beam A, Five Girders, Simply Supported, H = 20, b/a = 0.05, b = 8 ft.

FIG. 3.5 EFFECTS OF NUMBER OF HARMONICS ON THE ACCURACY OF THE SOLUTION, BEAM A LOADED



10 kip Concentrated Load at Midspan of Beam B, Five Girders, Simply Supported, H = 20, b/a = 0.05, b = 8 ft.

FIG. 3.6 EFFECTS OF NUMBER OF HARMONICS ON THE ACCURACY OF THE SOLUTION, BEAM B LOADED



10 kip Concentrated Load at Midspan of Beam C, Five Girders, Simply Supported, H = 20, b/a = 0.05, b = 8 ft.

FIG. 3.7 EFFECTS OF NUMBER OF HARMONICS ON THE ACCURACY OF THE SOLUTION, BEAM C LOADED

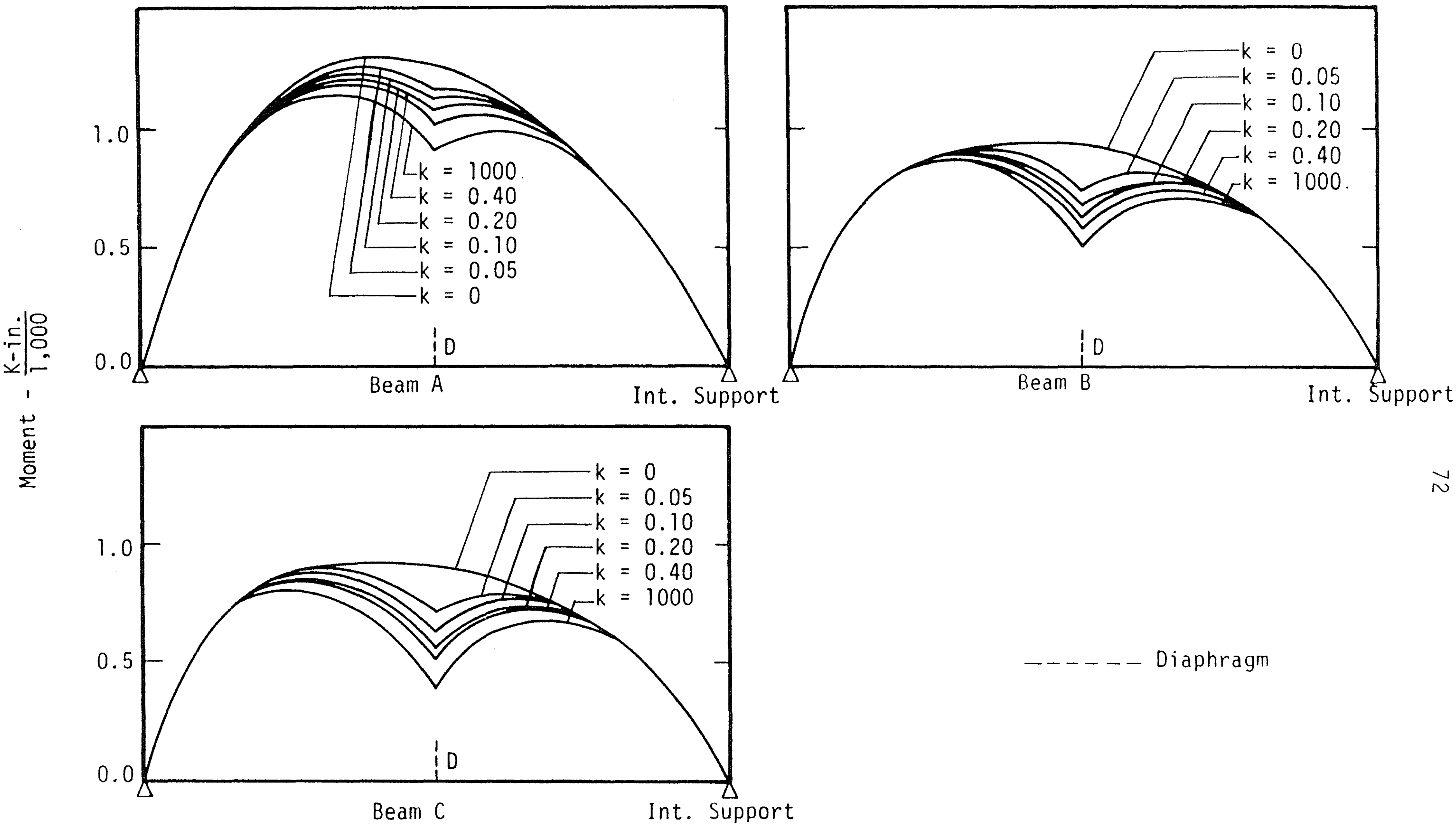


FIG. 4.1 POSITIVE MOMENT ENVELOPES OF GIRDERS DUE TO A 10 KIP CONCENTRATED LOAD MOVING ALONG THE BRIDGE, ONE DIAPHRAGM AT MIDSPAN OF EACH SPAN; H = 20, b/a = 0.1, b = 8 FT

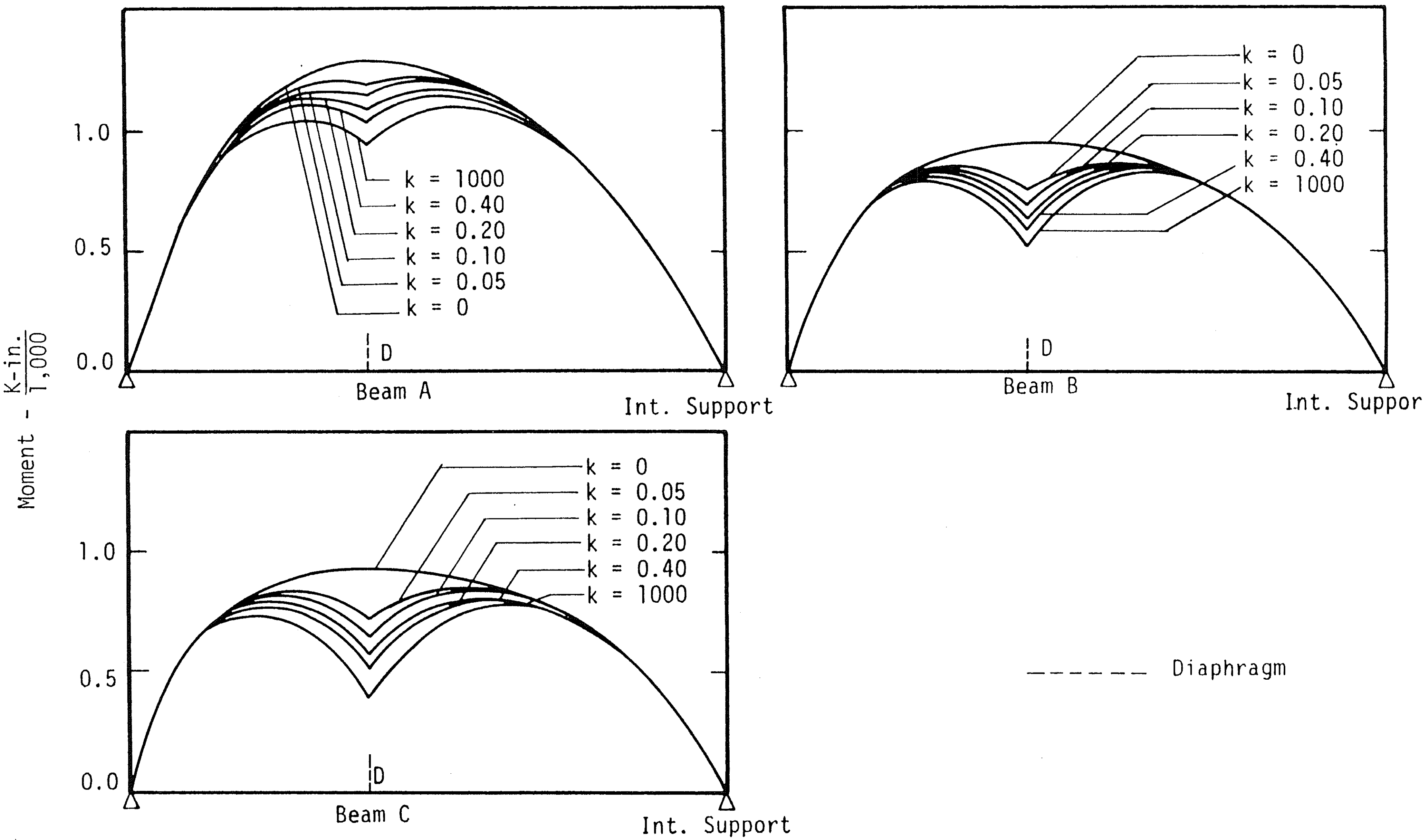


FIG. 4.2 POSITIVE MOMENT ENVELOPES OF GIRDERS DUE TO A 10 KIP CONCENTRATED LOAD MOVING ALONG THE BRIDGE, ONE DIAPHRAGM AT 4/10 POINT OF EACH SPAN; H = 20, b/a = 0.1, b = 8 FT

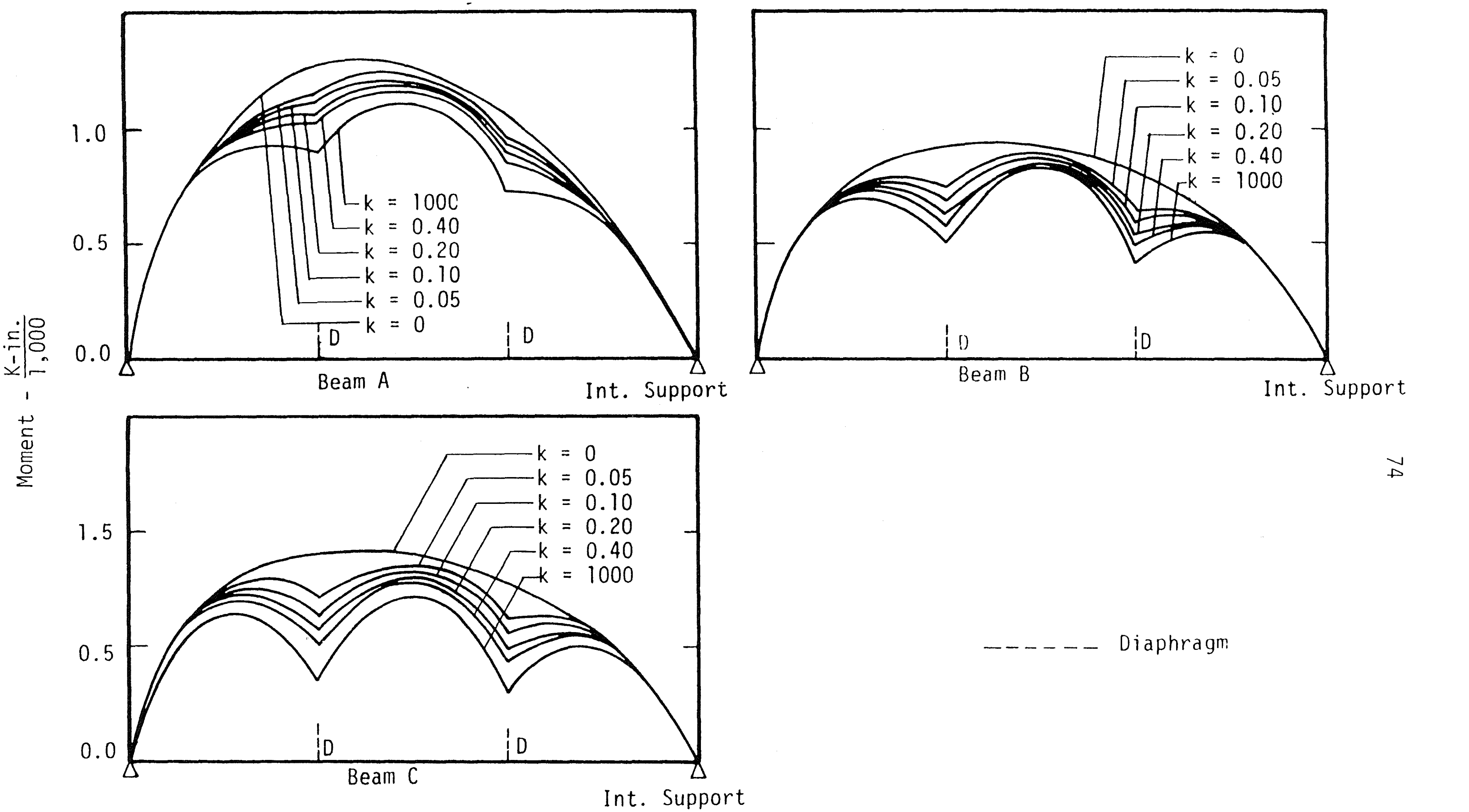


FIG. 4.3 POSITIVE MOMENT ENVELOPES OF GIRDERS DUE TO A 10 KIP CONCENTRATED LOAD MOVING ALONG THE BRIDGE, TWO DIAPHRAGMS AT 1/3 POINT OF EACH SPAN OF THE BRIDGE; H = 20, b/a = 0.1, b = 8 FT

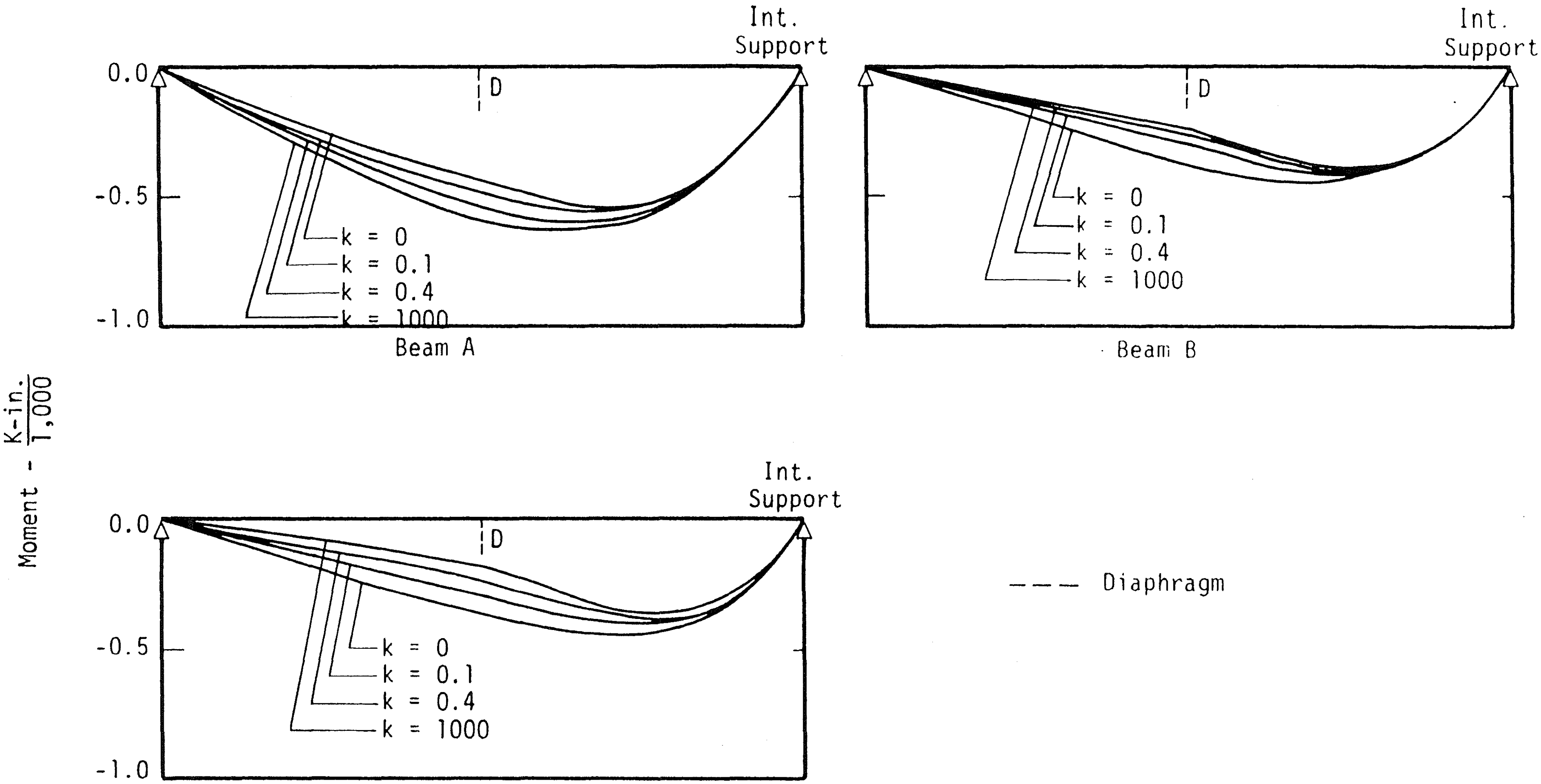


FIG. 4.4 INFLUENCE LINES FOR NEGATIVE MOMENT AT SUPPORT OF GIRDERS DUE TO A 10 KIP CONCENTRATED LOAD MOVING ALONG THE BRIDGE, ONE DIAPHRAGM AT MIDSPAN OF EACH SPAN; H = 20, b/a = 0.1, b = 8 FT

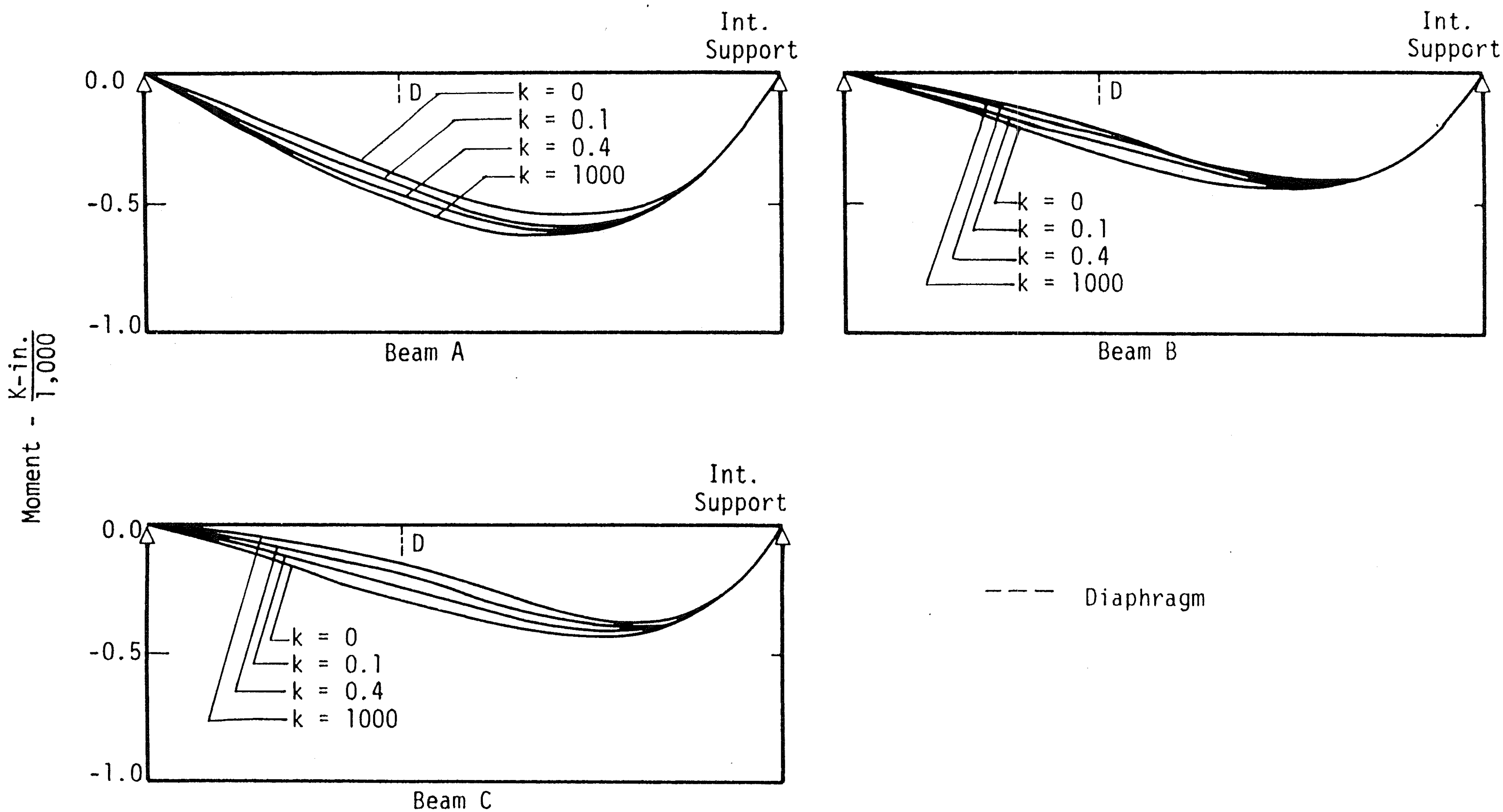


FIG. 4.5 INFLUENCE LINES FOR NEGATIVE MOMENT AT SUPPORT OF GIRDERS DUE TO A 10 KIP CONCENTRATED LOAD MOVING ALONG THE BRIDGE, ONE DIAPHRAGM AT 4/10 POINT OF EACH SPAN; H = 20, b/a = 0.1, b = 8 FT

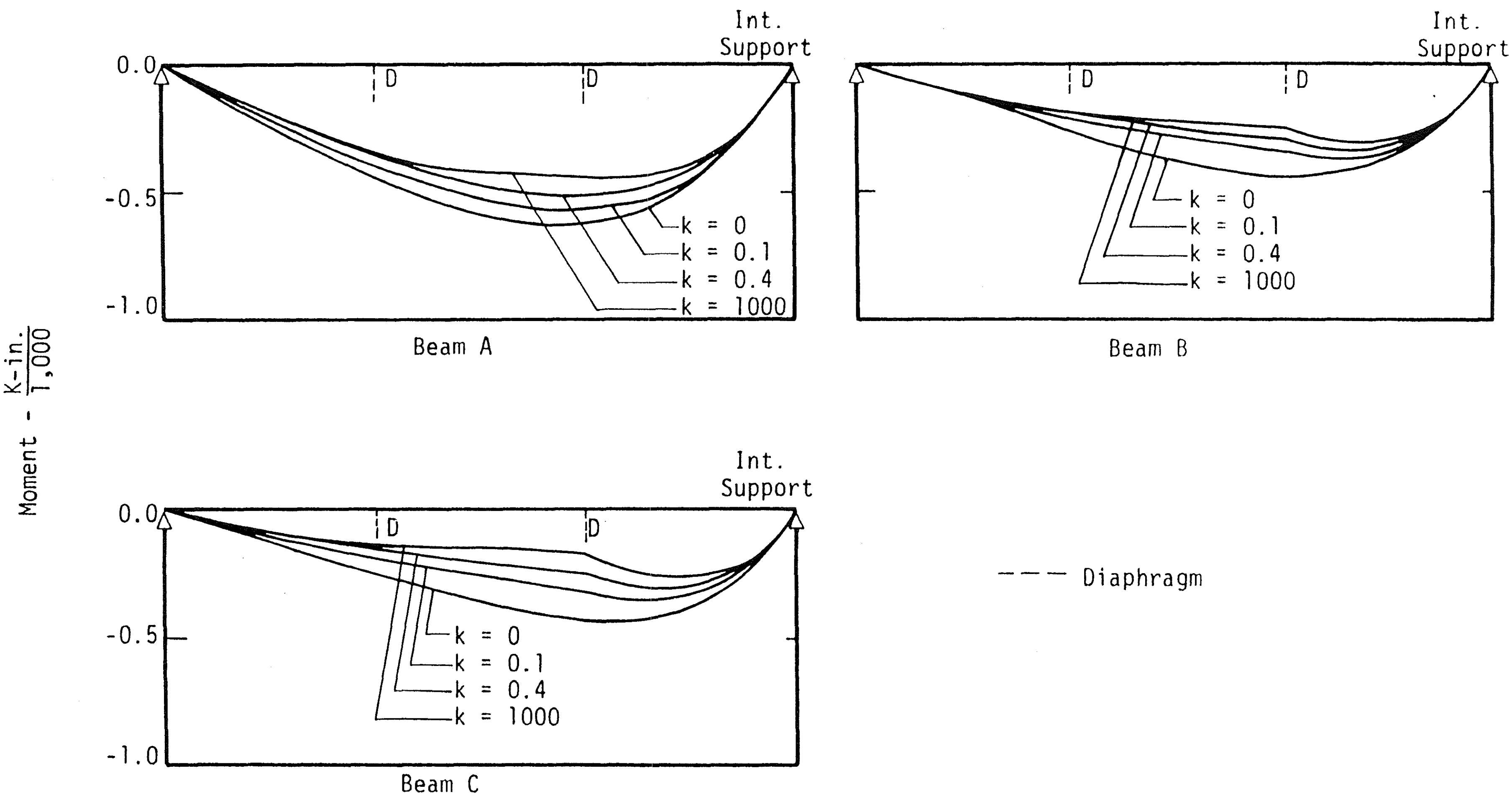


FIG. 4.6 INFLUENCE LINES FOR NEGATIVE MOMENT AT SUPPORT OF GIRDERS DUE TO A 10 KIP CONCENTRATED LOAD MOVING ALONG THE BRIDGE, TWO DIAPHRAGMS AT 1/3 POINT OF EACH SPAN; H = 20, b/a = 0.1, b = 8 FT

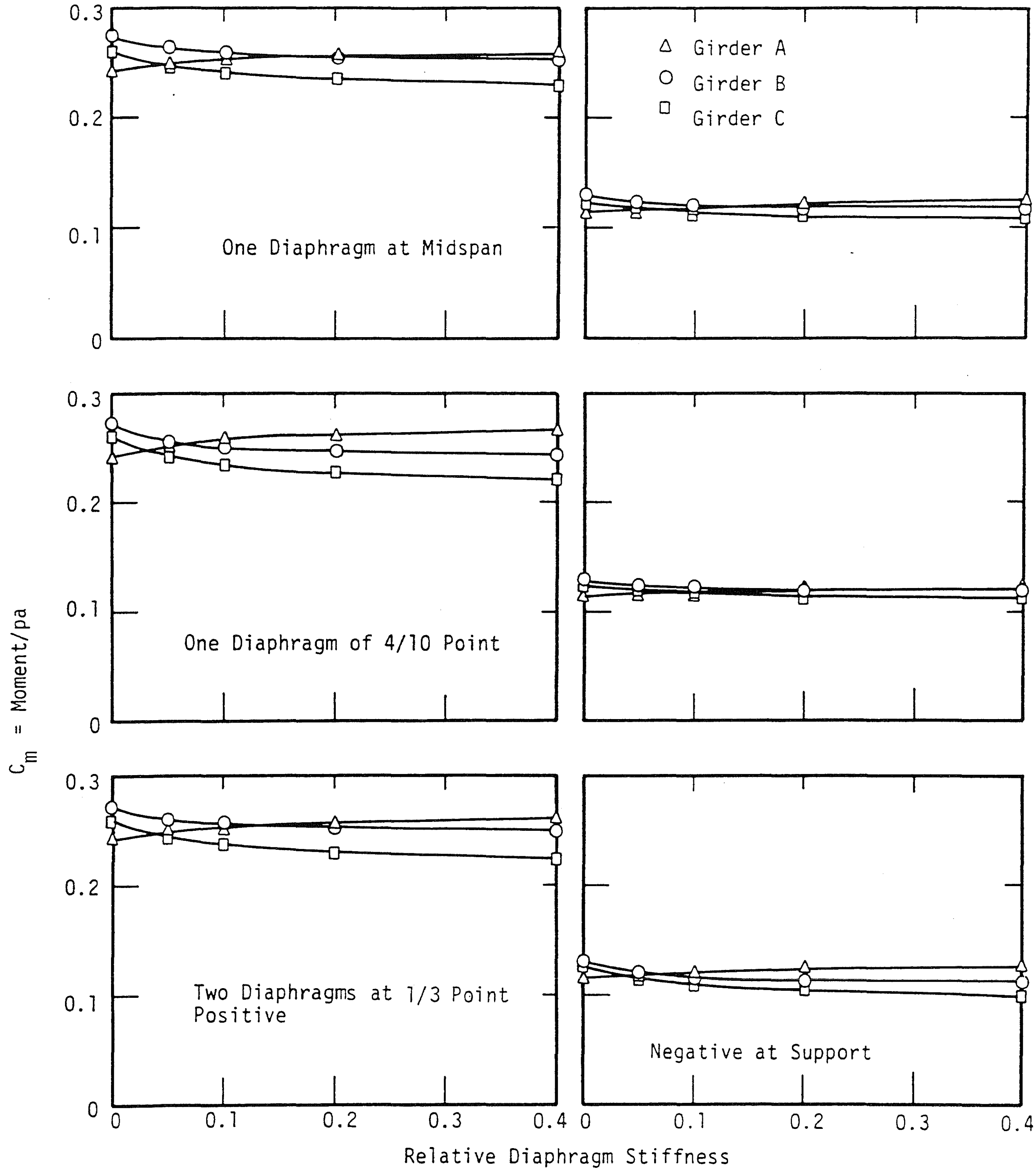


FIG. 4.7 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 20, b/a = 0.1, b = 8 FT

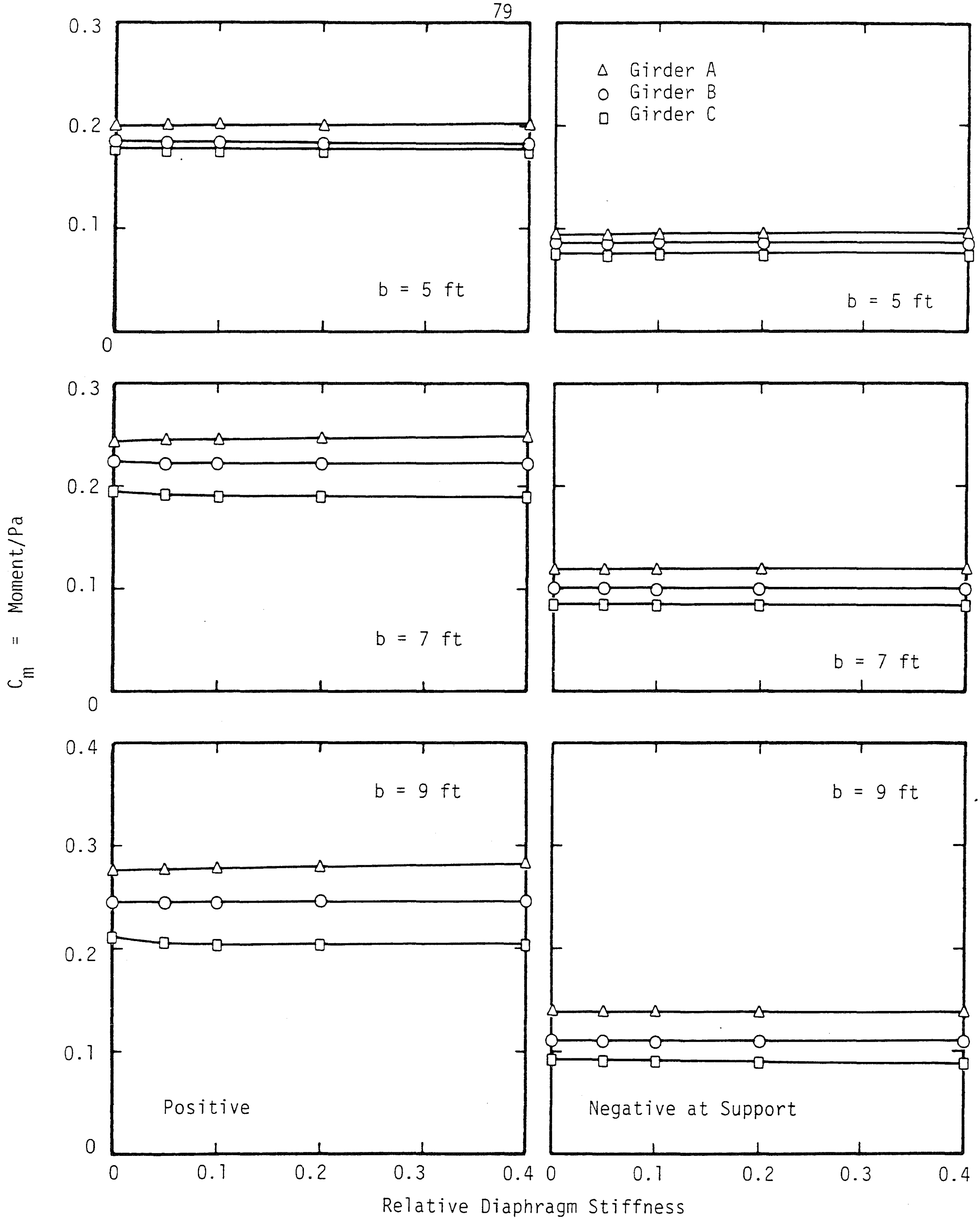


FIG. 4.8 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 5, b/a = 0.05

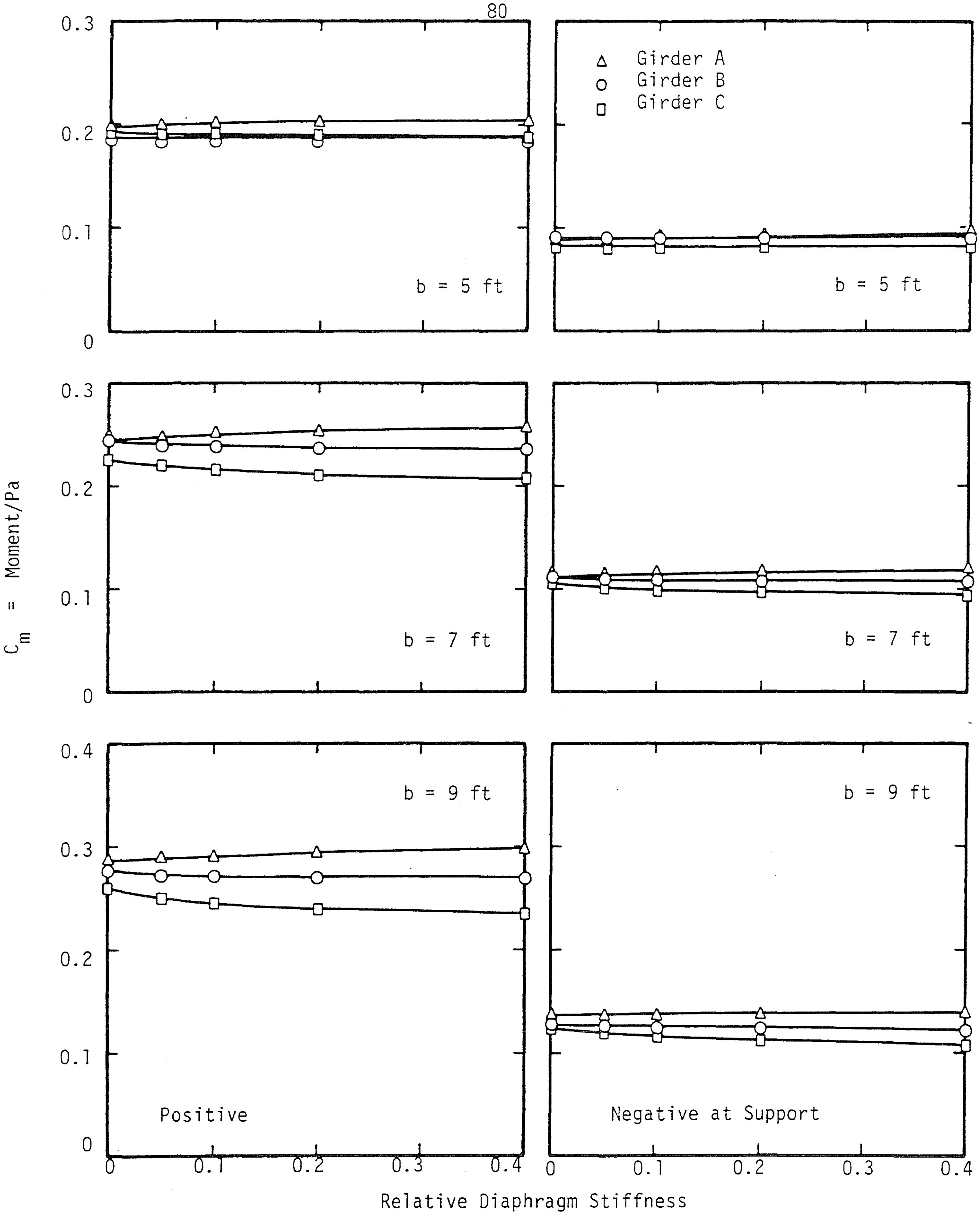


FIG. 4.9 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 5, b/a = 0.1

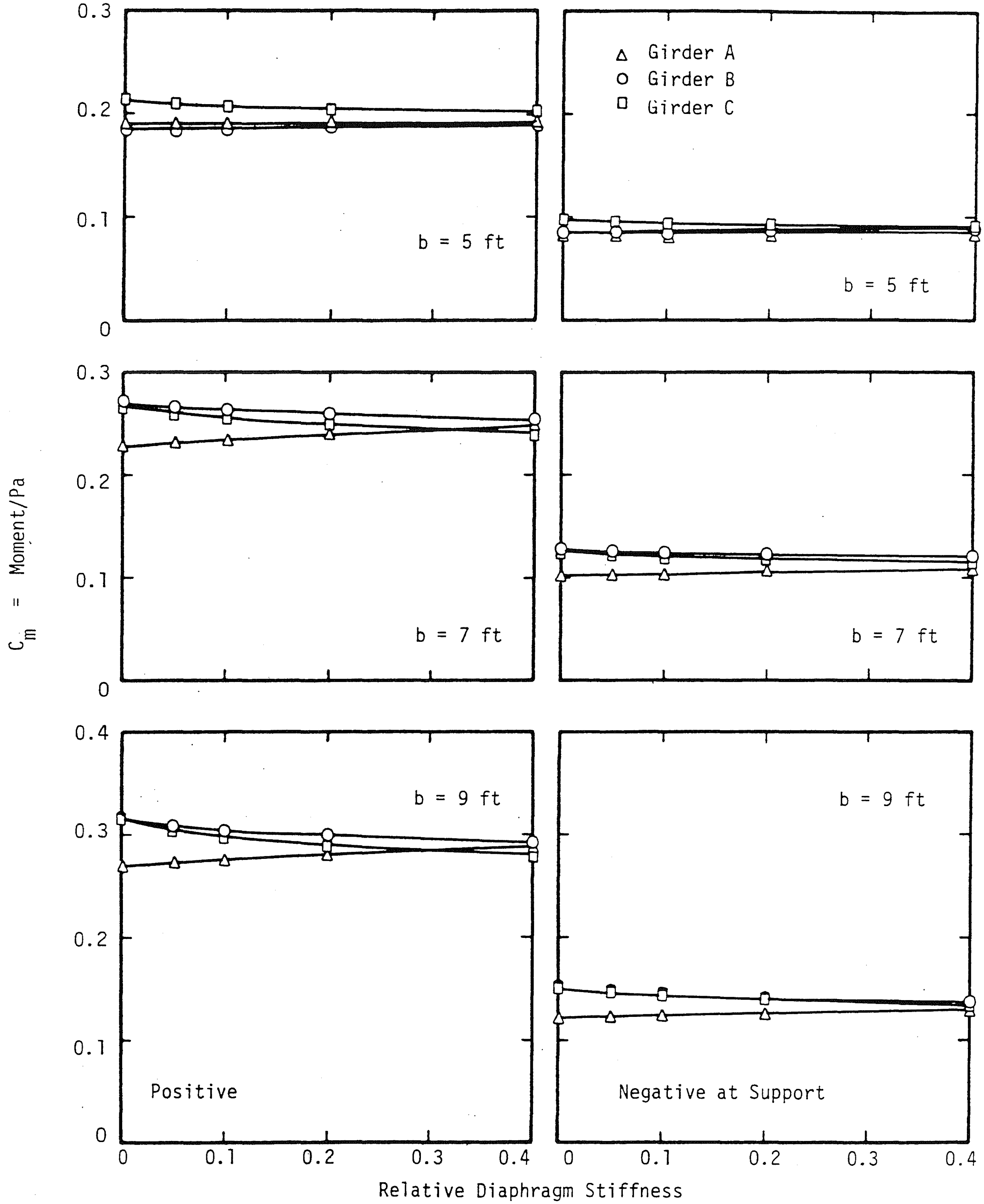


FIG. 4.10 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 5, b/a = 0.2

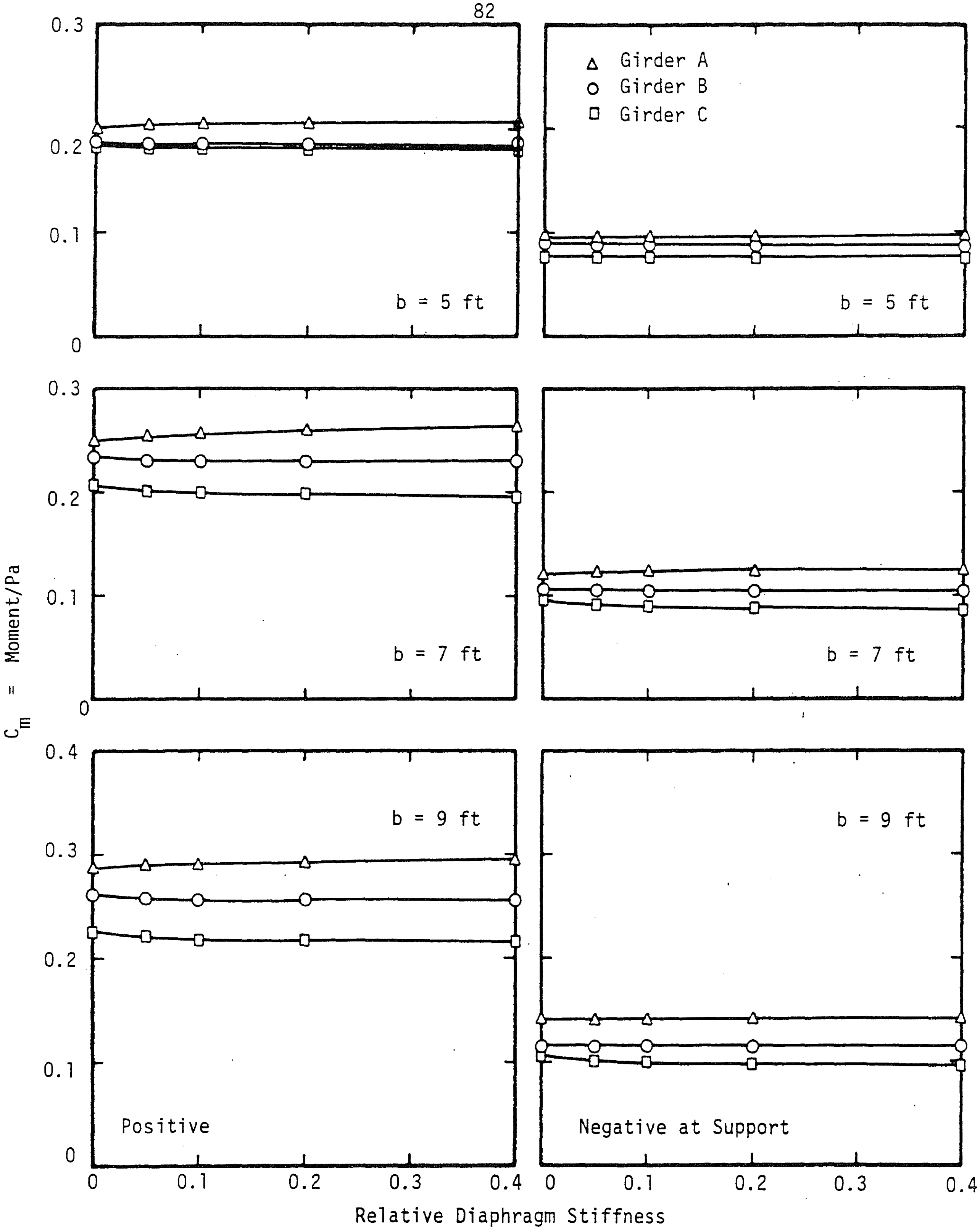


FIG. 4.11 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 10, b/a = 0.05

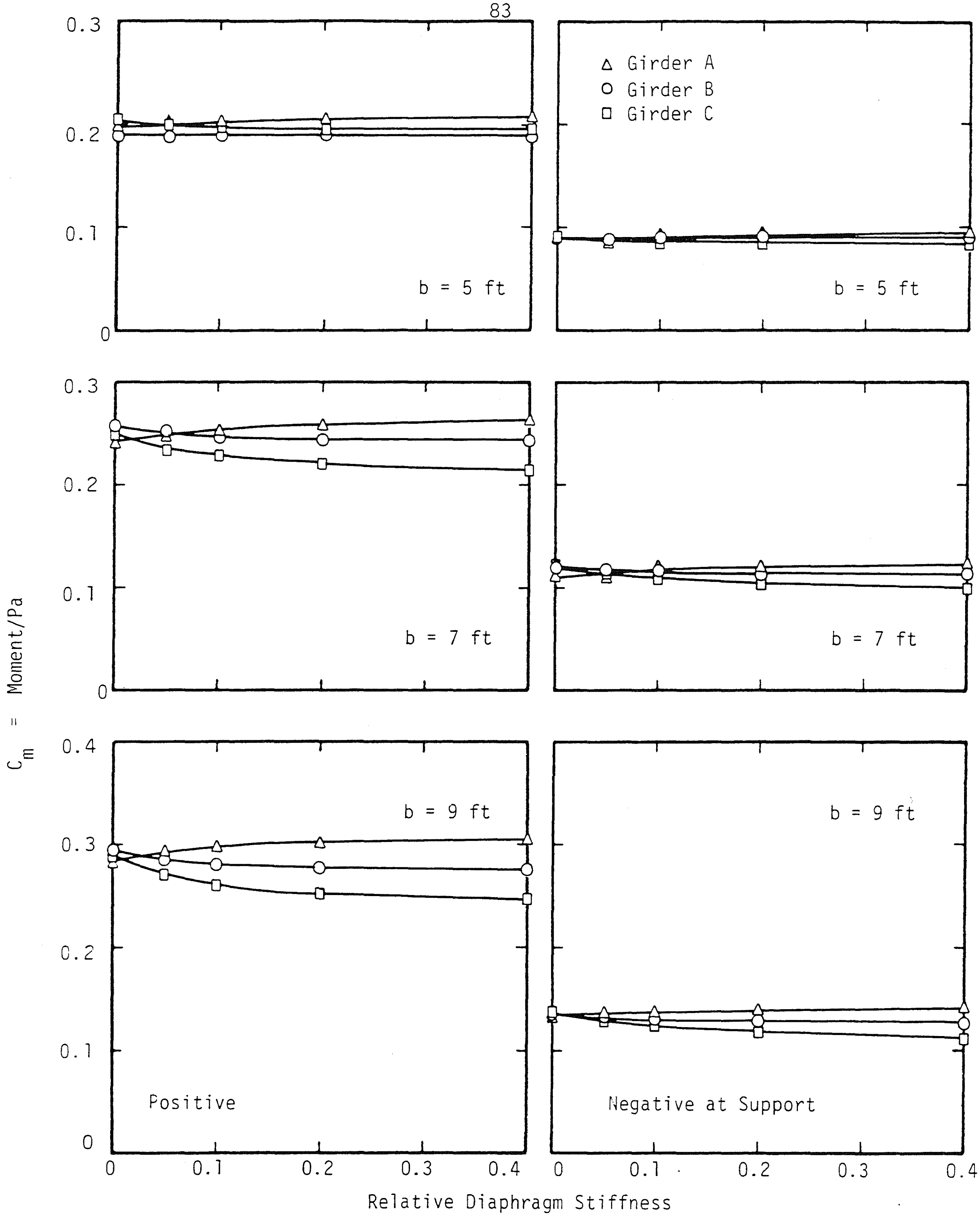


FIG. 4.12 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 10, b/a = 0.1

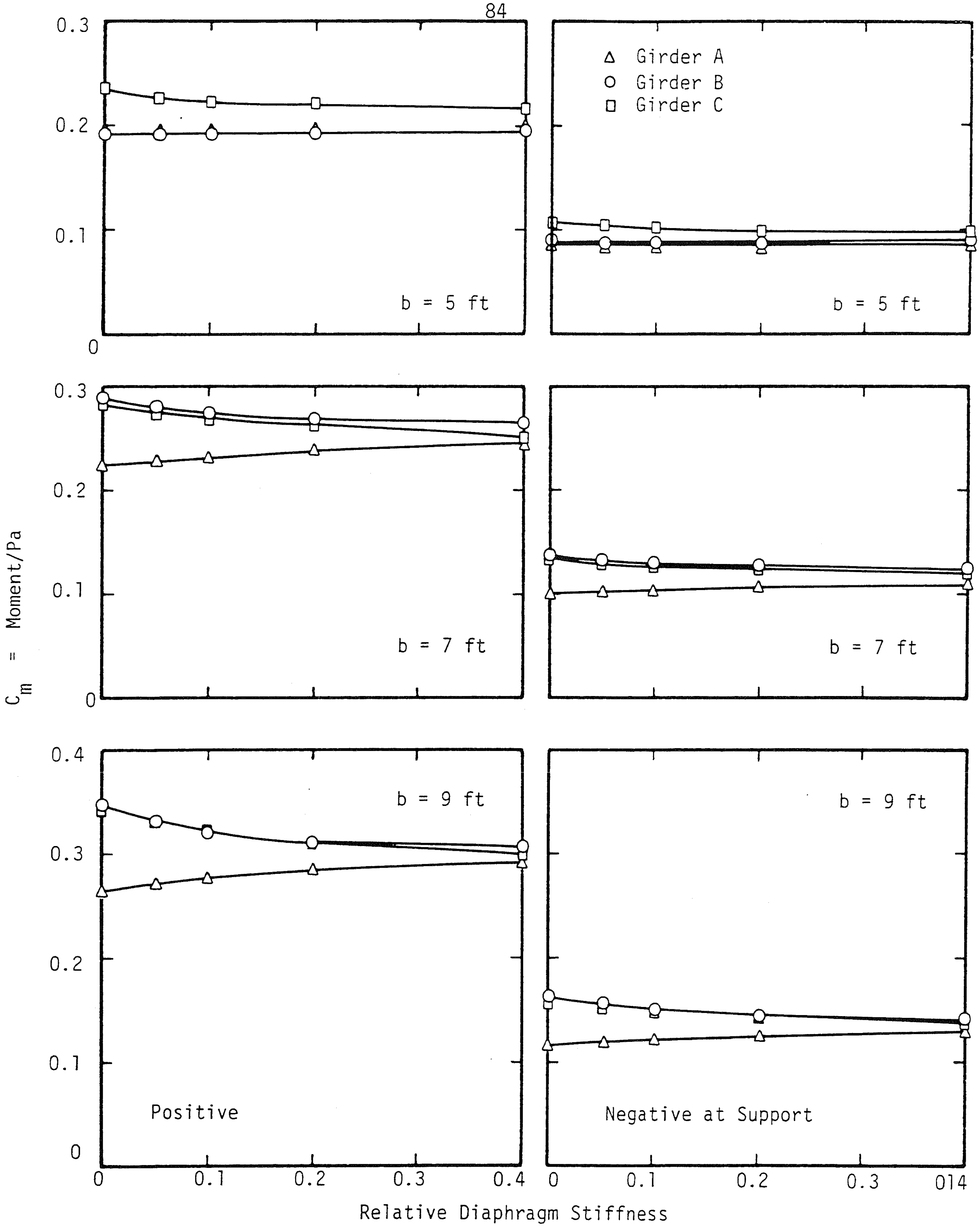


FIG. 4.13 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 10, b/a = 0.2

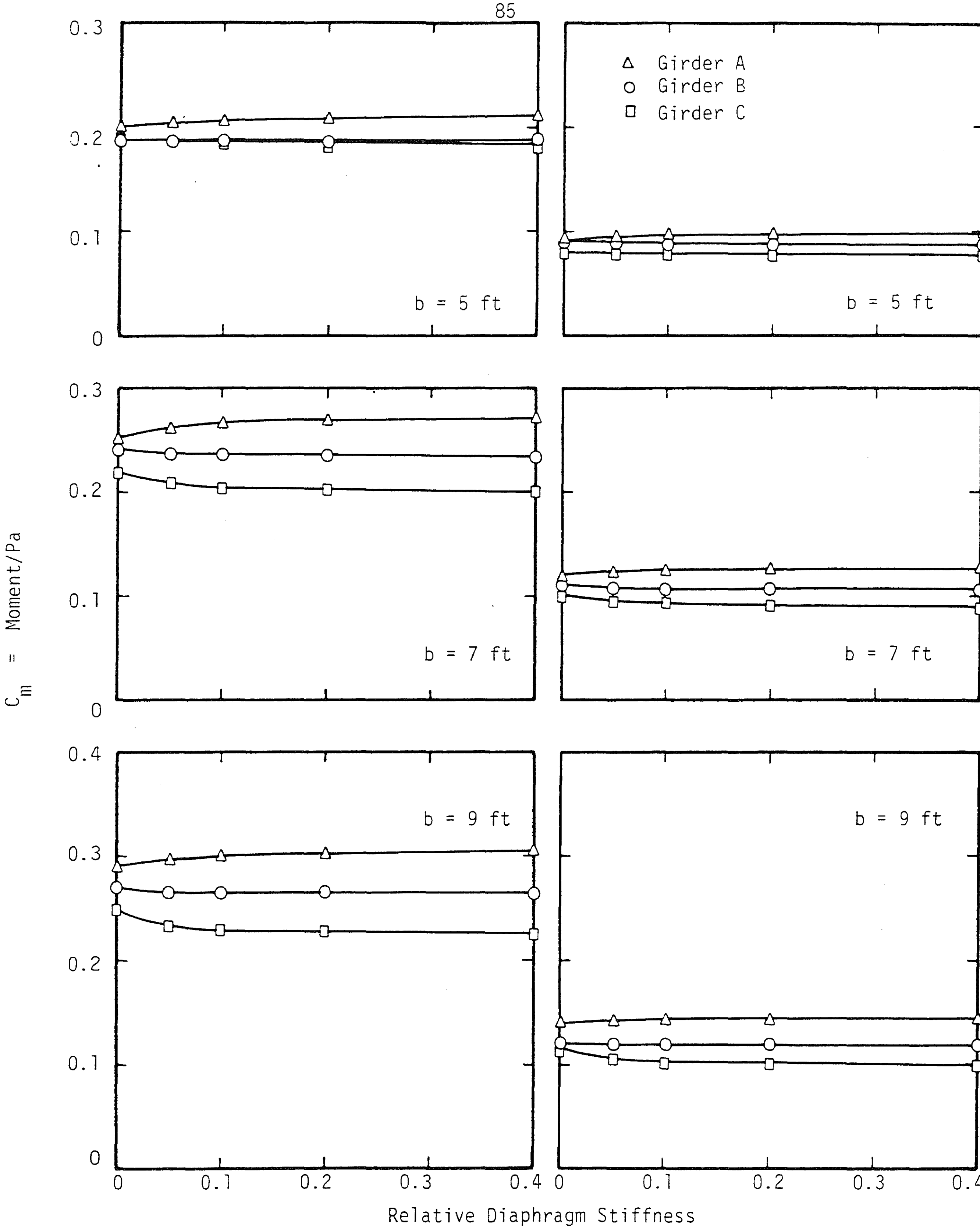


FIG. 4.14 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 20, b/a = 0.05

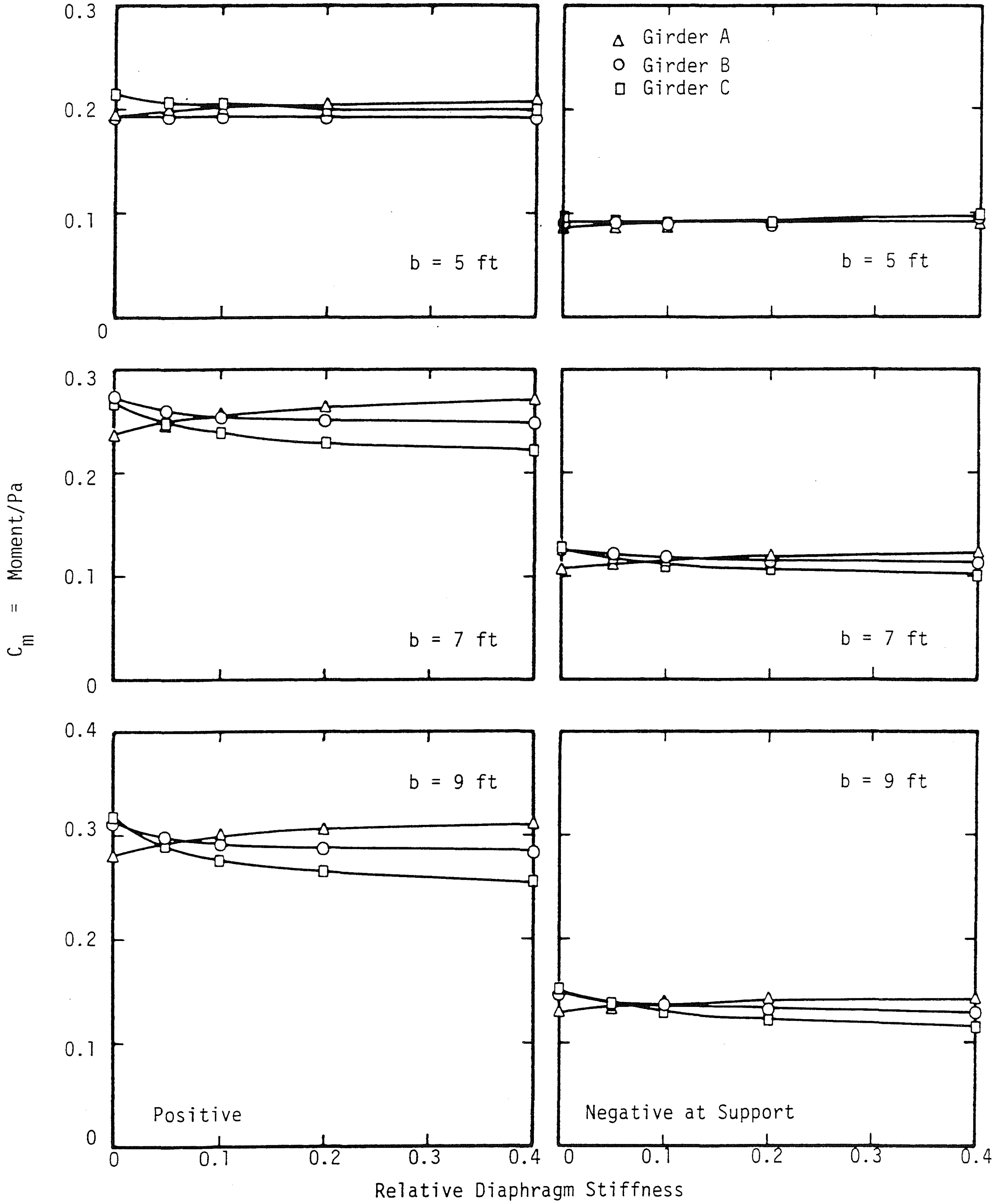


FIG. 4.15 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 20, b/a = 0.1

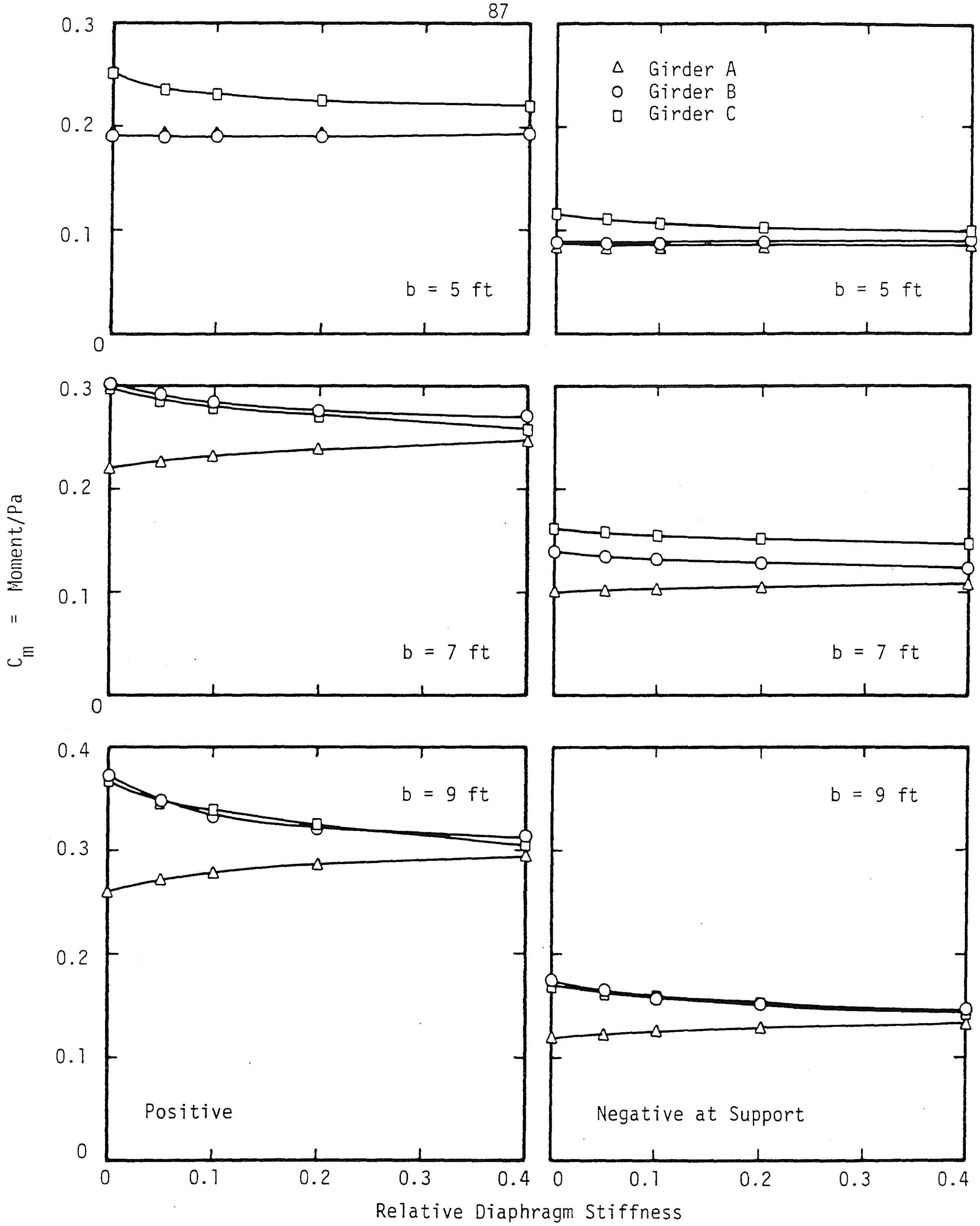


FIG. 4.16 MAXIMUM MOMENT COEFFICIENTS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; H = 20, b/a = 0.2

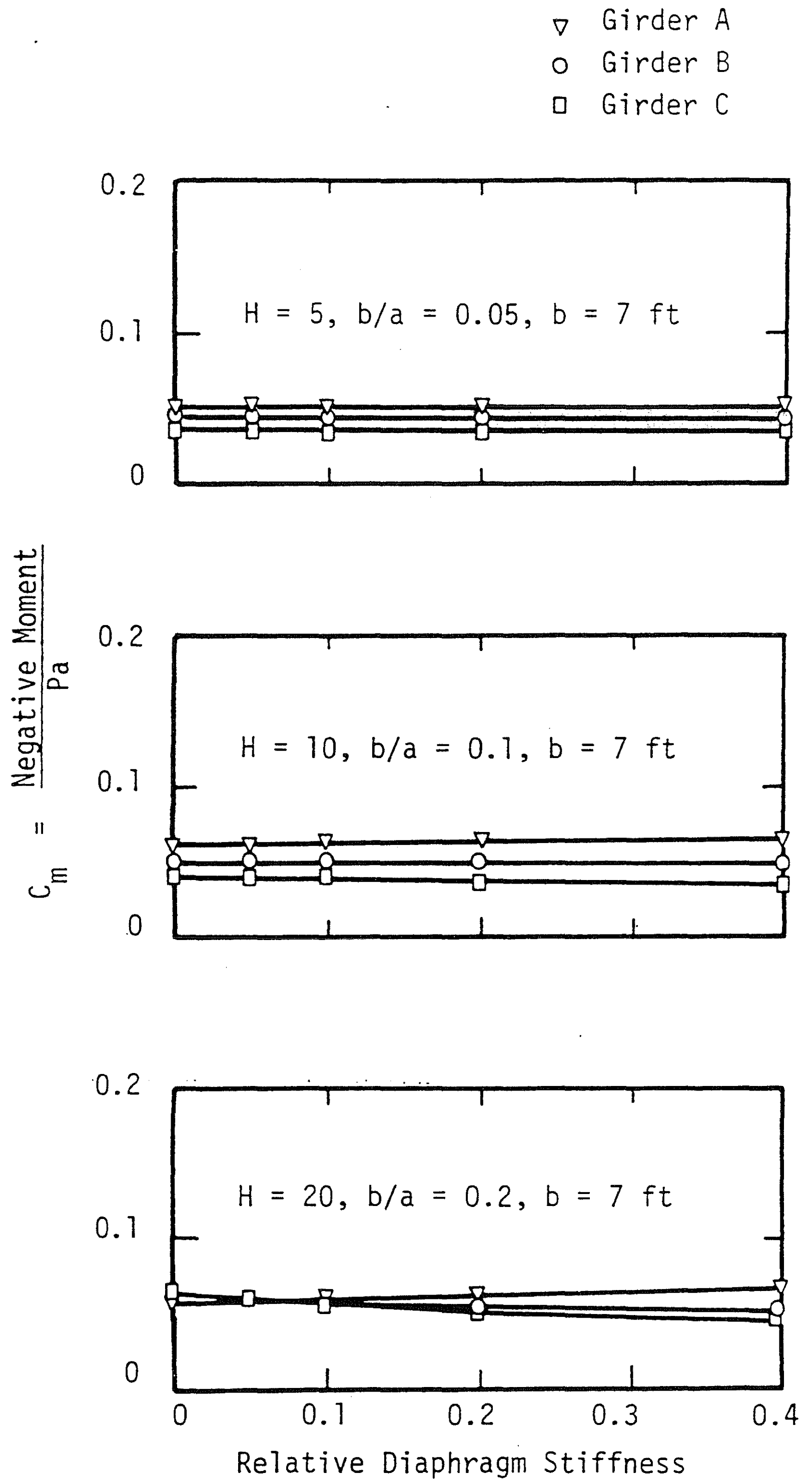


FIG. 4.17 MAXIMUM NEGATIVE MOMENT COEFFICIENTS AT MIDSPAN OF UNLOADED SPAN DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS

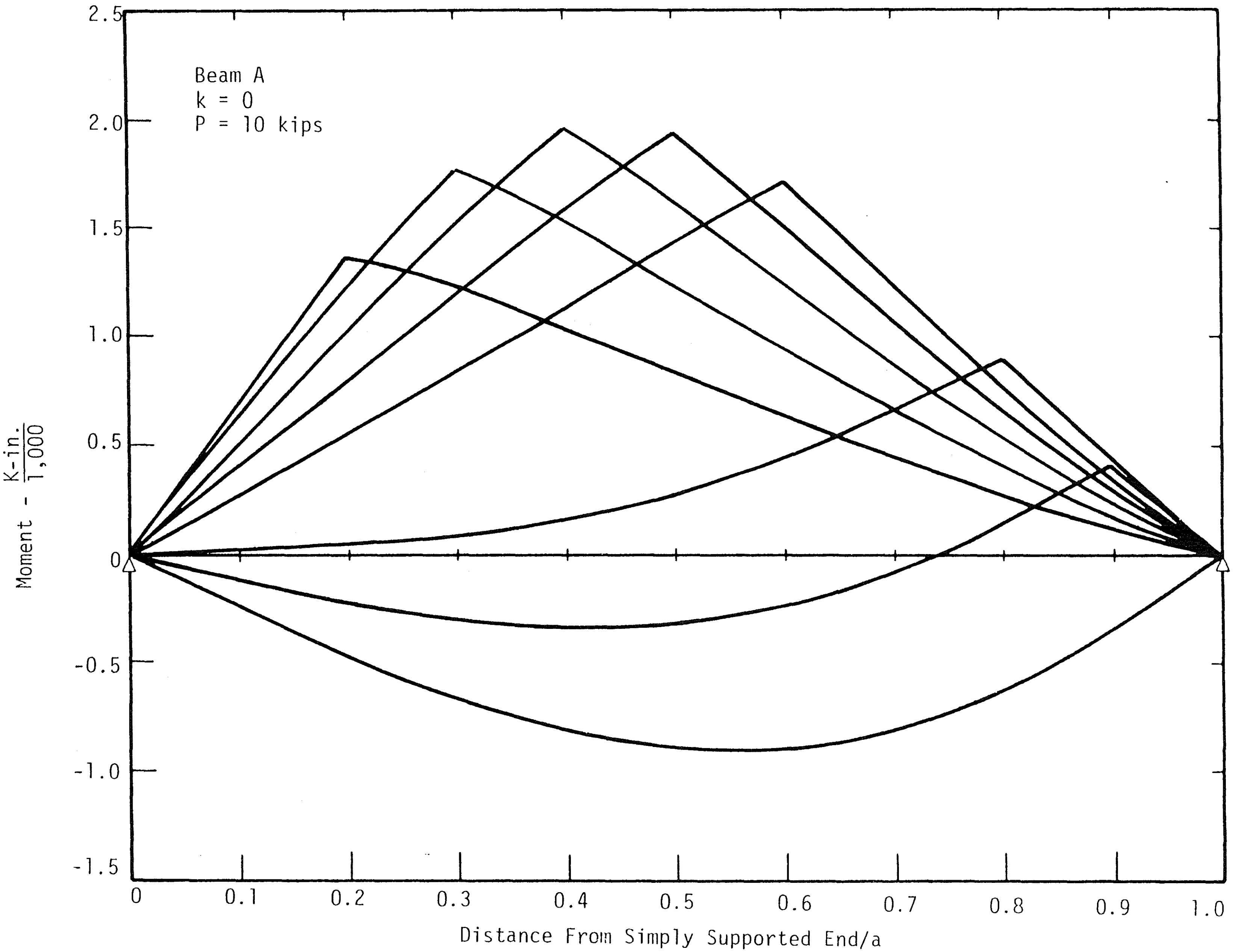


FIG. 4.18 INFLUENCE LINES FOR MOMENTS AT VARIOUS SECTIONS ALONG TWO SPAN CONTINUOUS BRIDGE DUE TO 4W LOADING; H = 20, b/a = 0.1, b = 7 FT

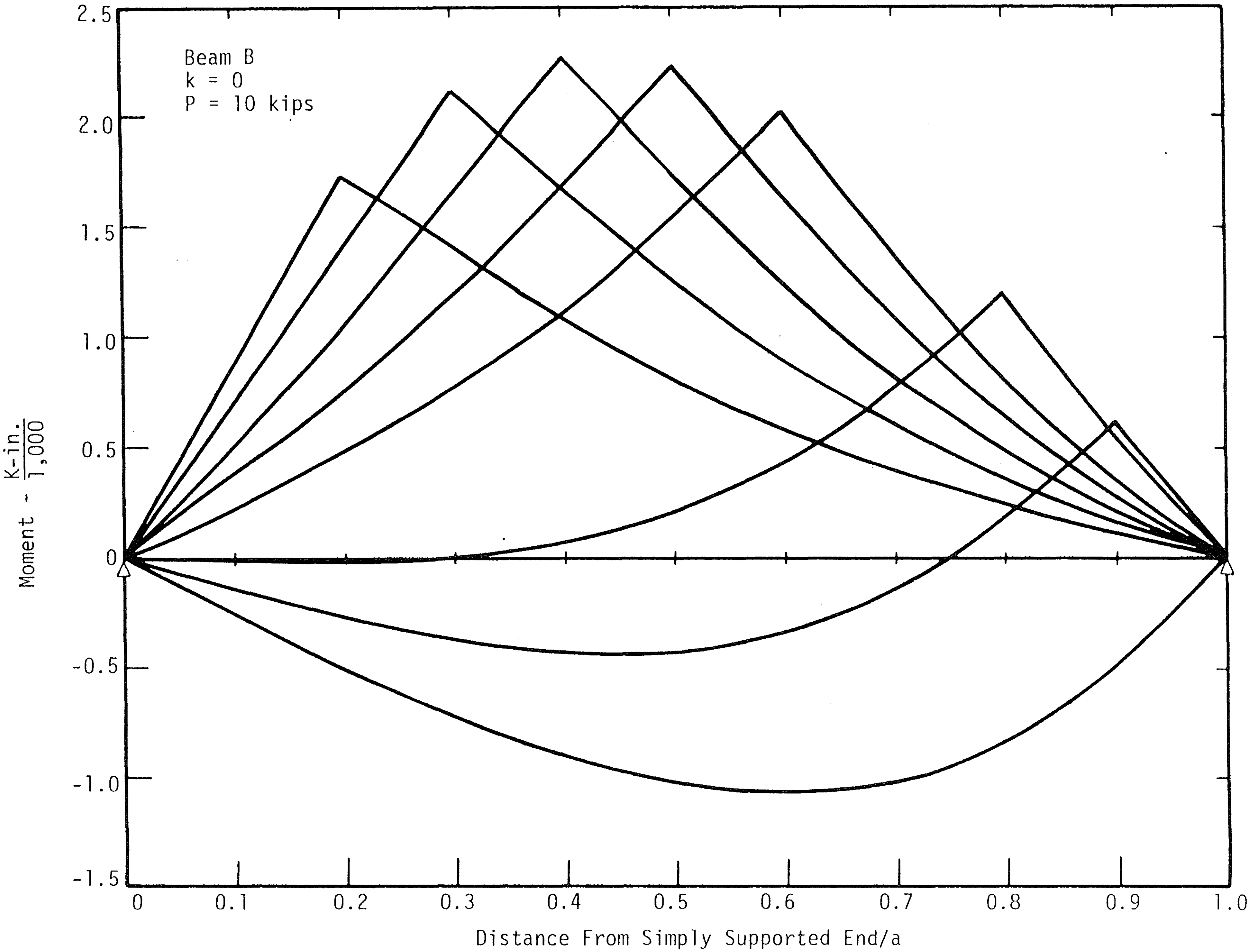


FIG. 4.18 (CONTINUED)

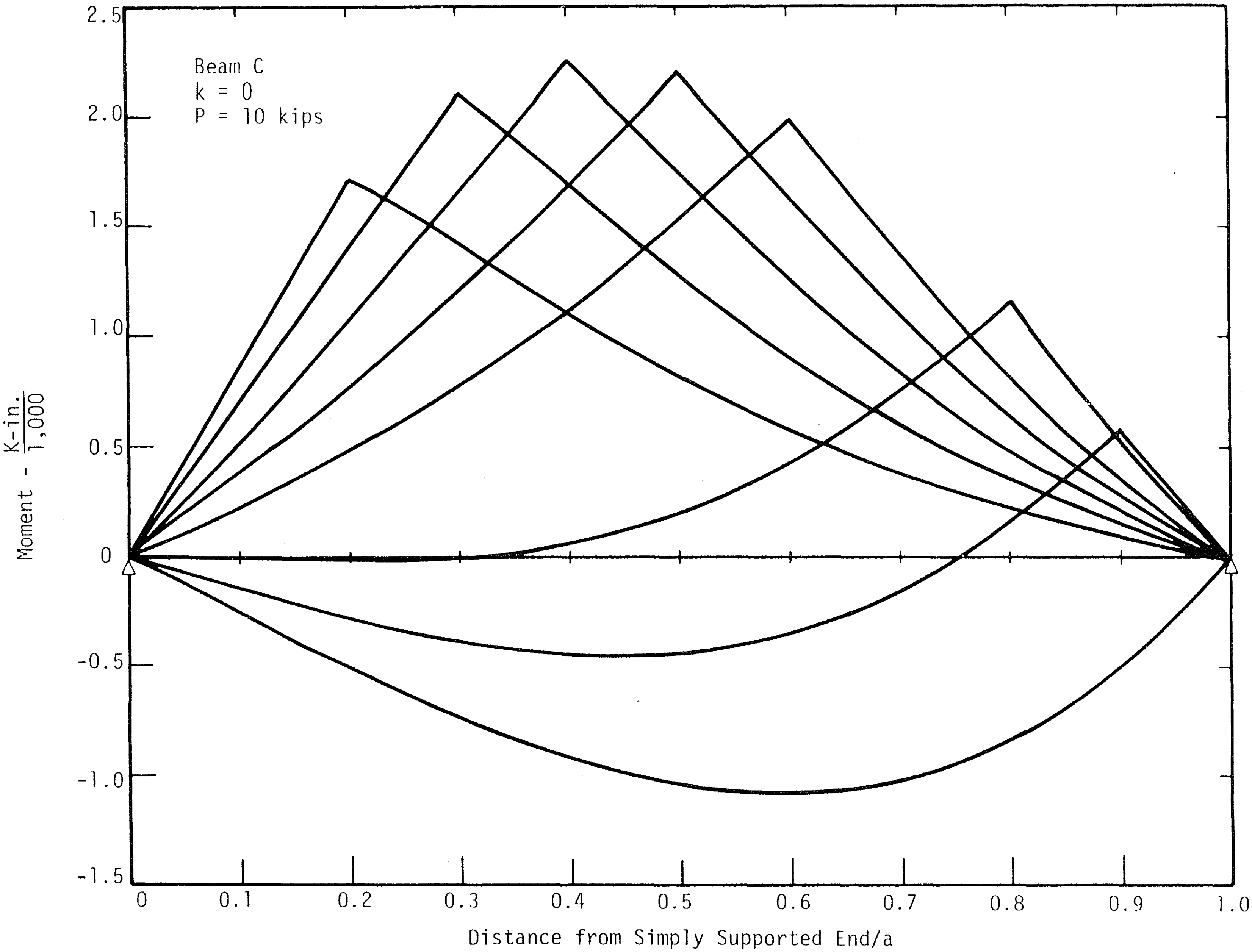


FIG. 4.18 (CONTINUED)

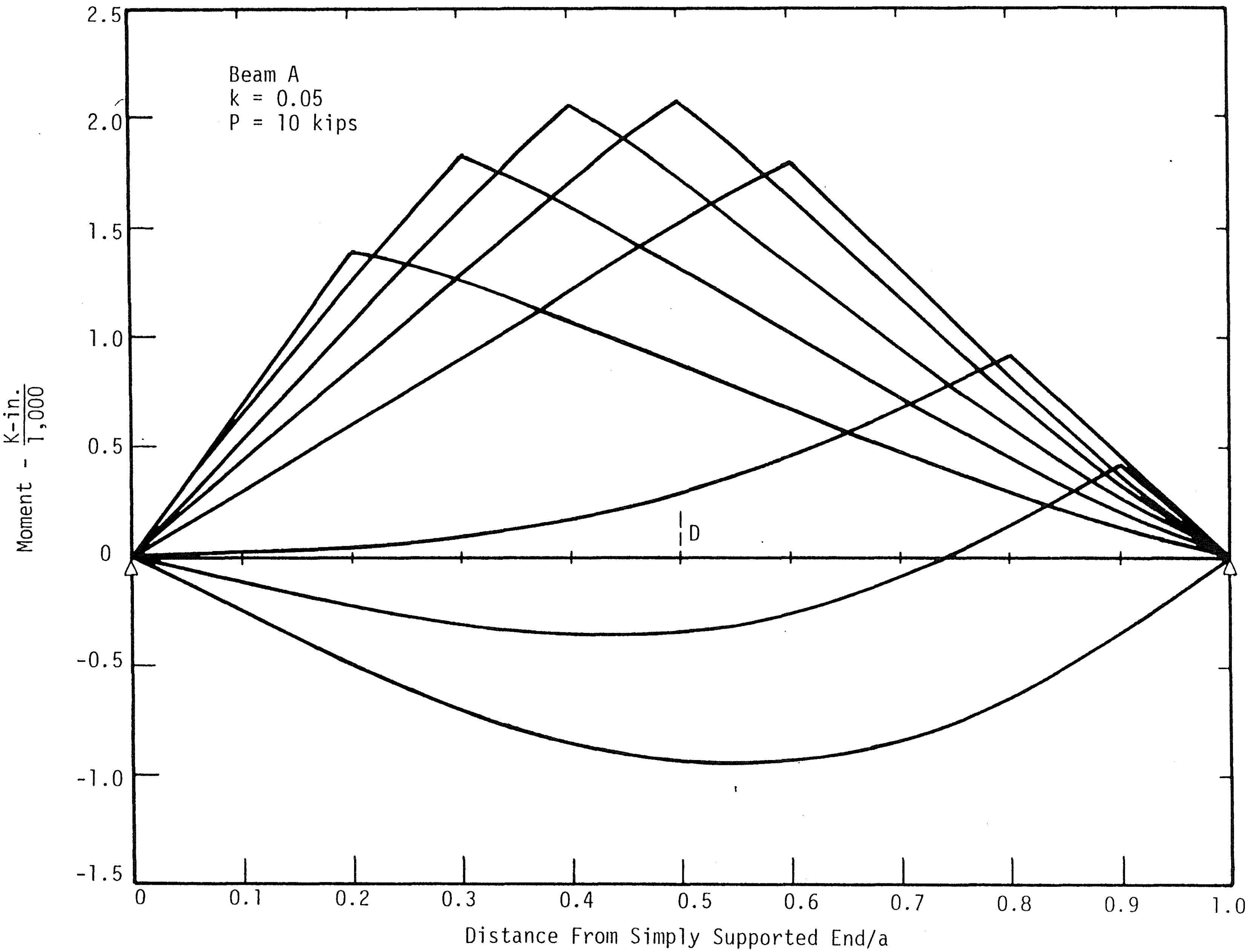


FIG. 4.18 (CONTINUED)

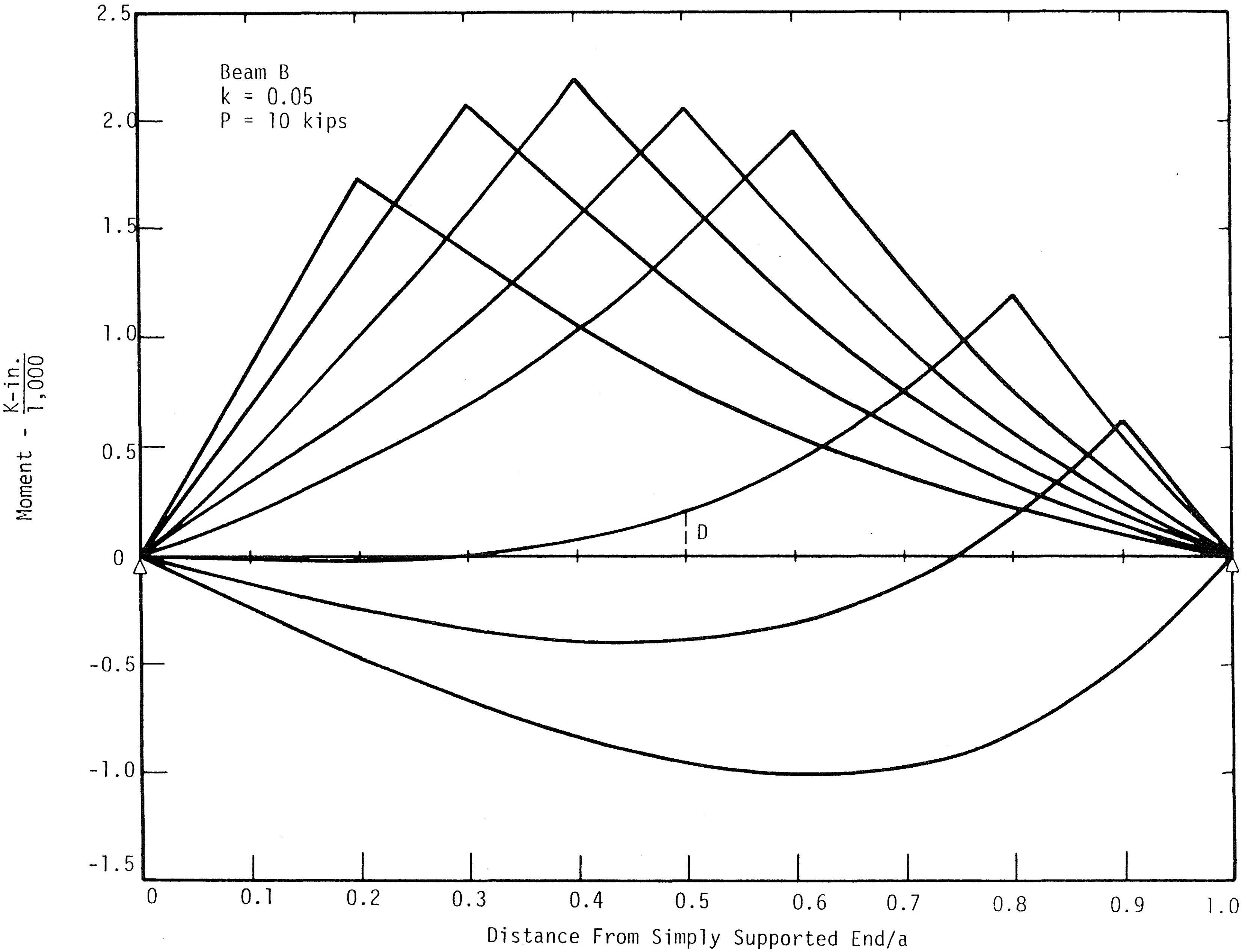


FIG. 4.18 (CONTINUED)

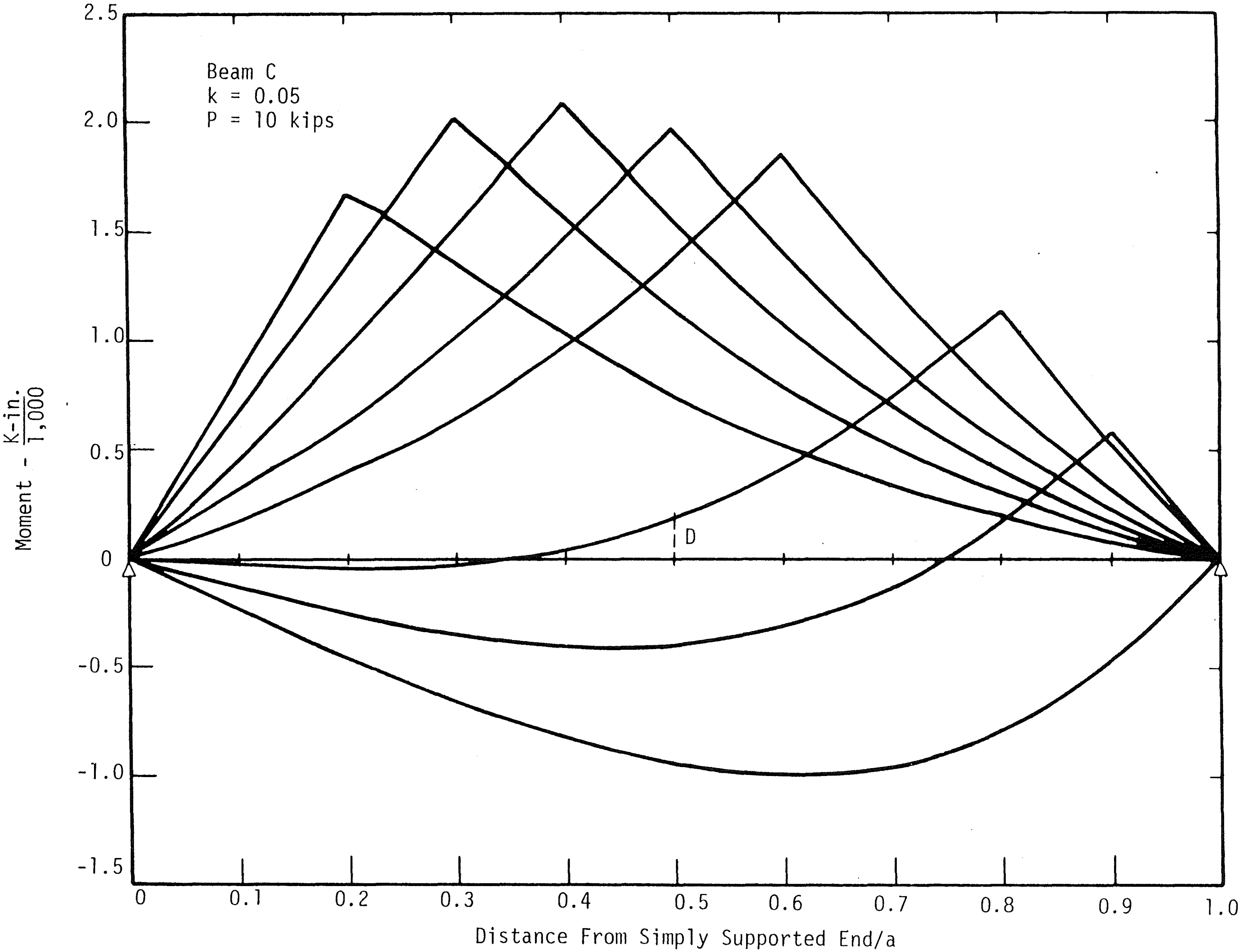


FIG. 4.18 (CONTINUED)

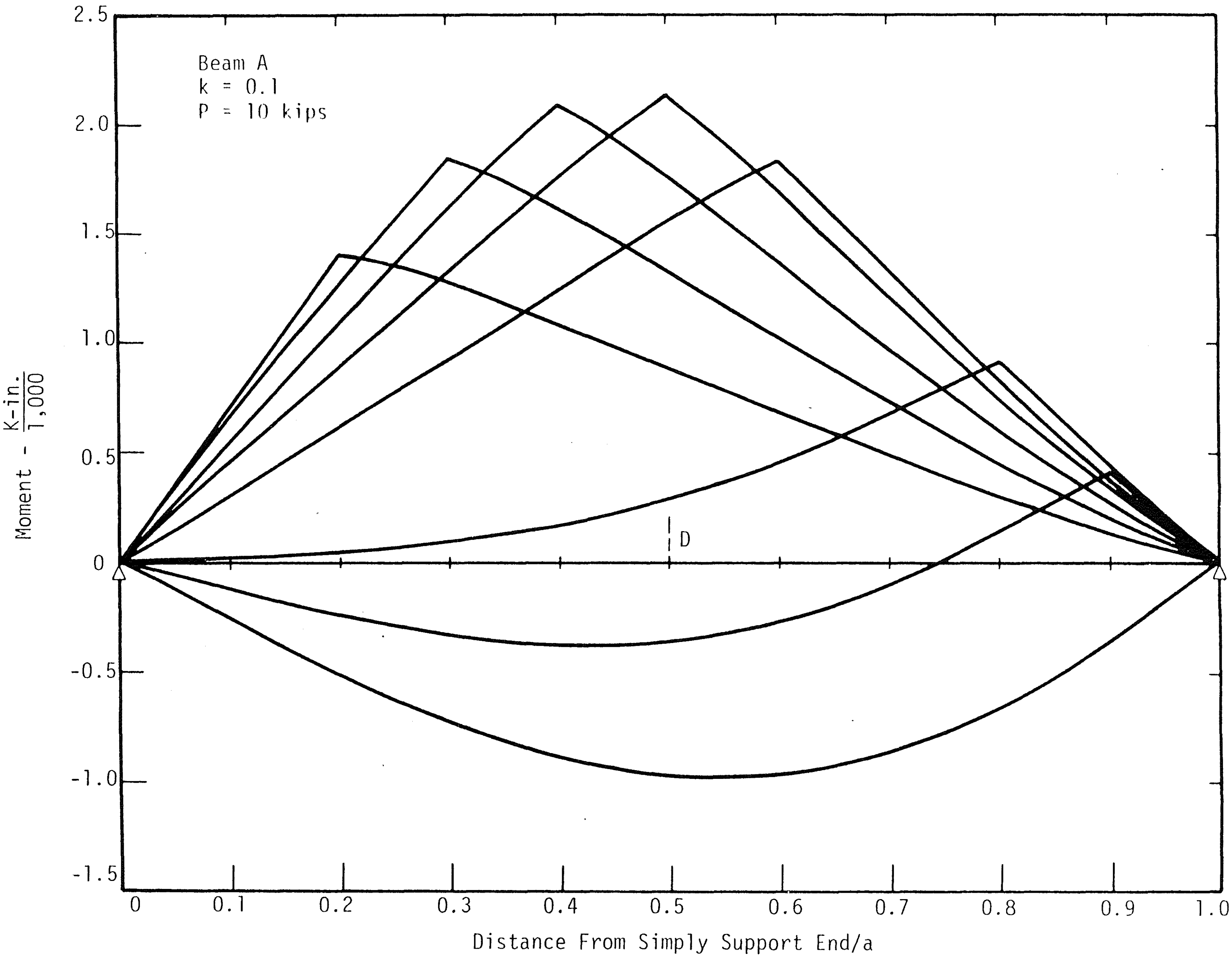


FIG. 4.18 (CONTINUED)

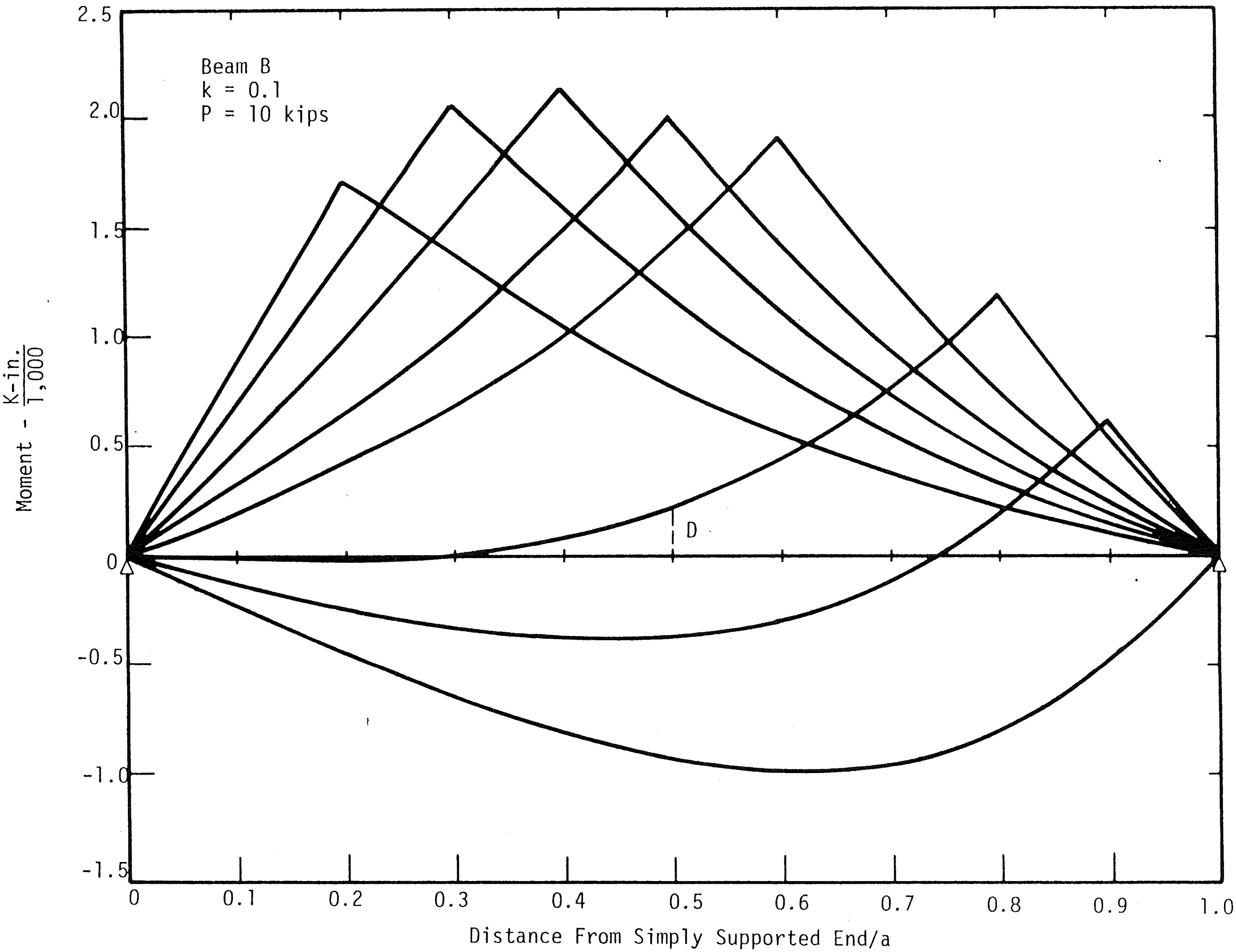


FIG. 4.18 (CONTINUED)

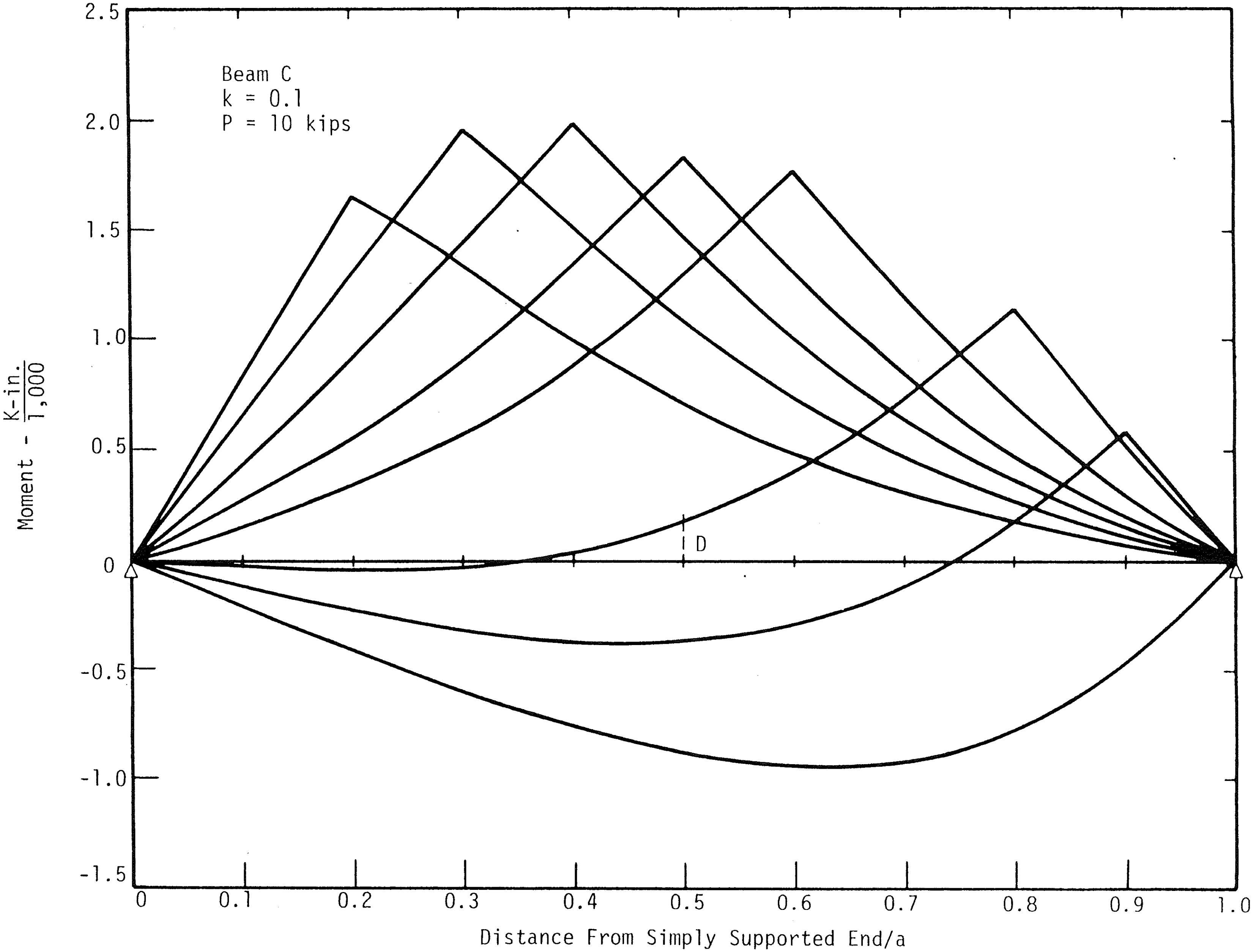


FIG. 4.18 (CONTINUED)

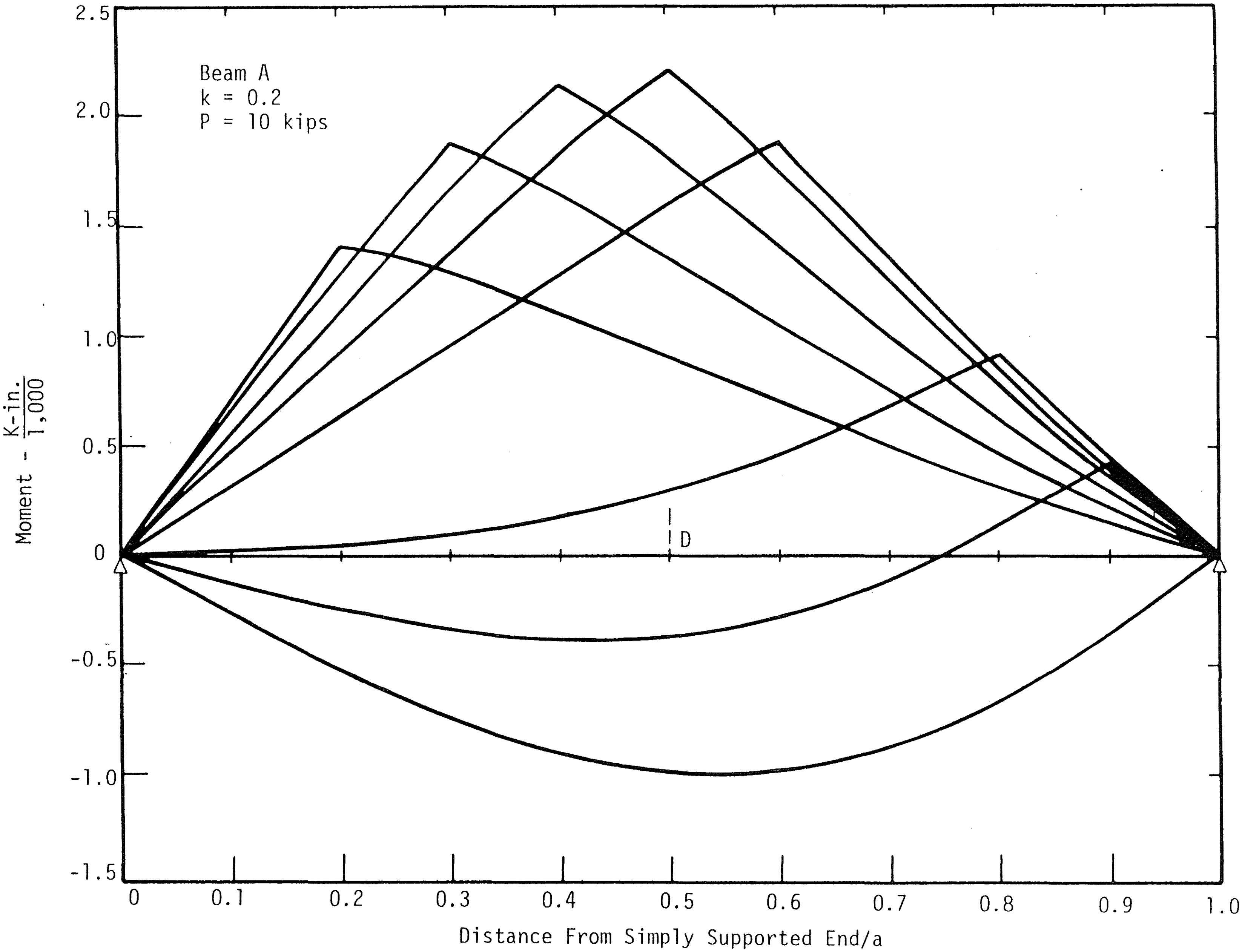


FIG. 4.18 (CONTINUED)

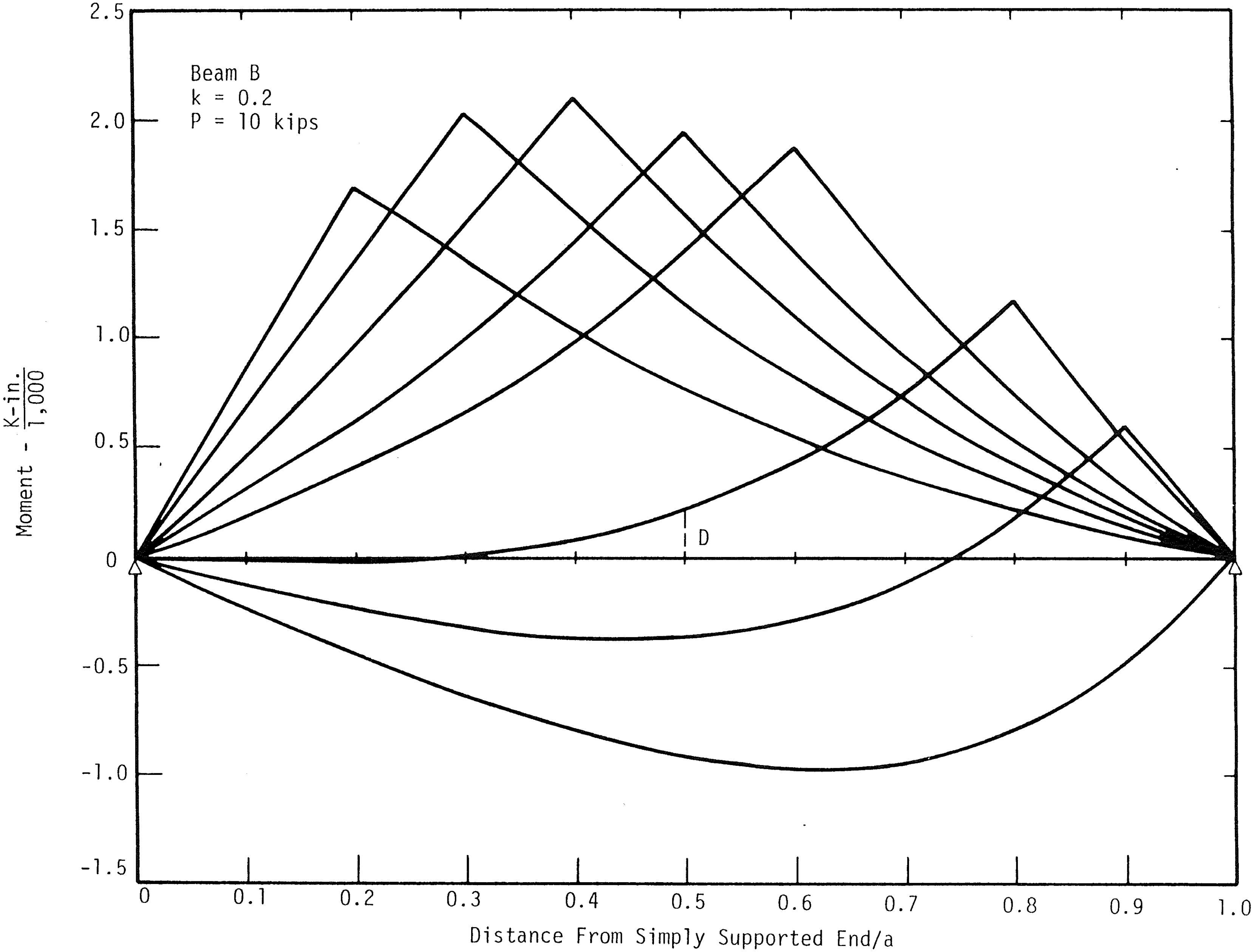


FIG. 4.18 (CONTINUED)

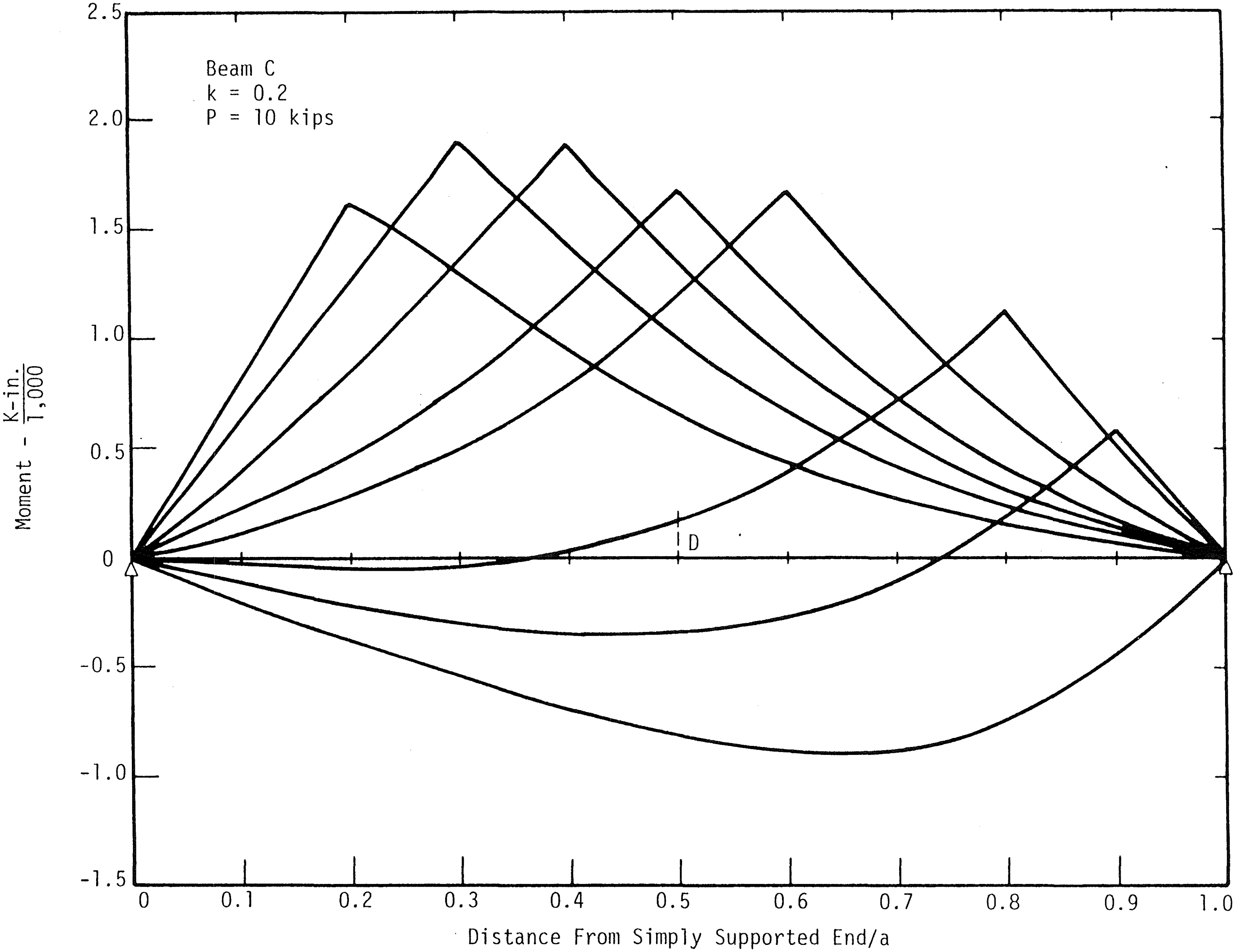


FIG. 4.18 (CONTINUED)

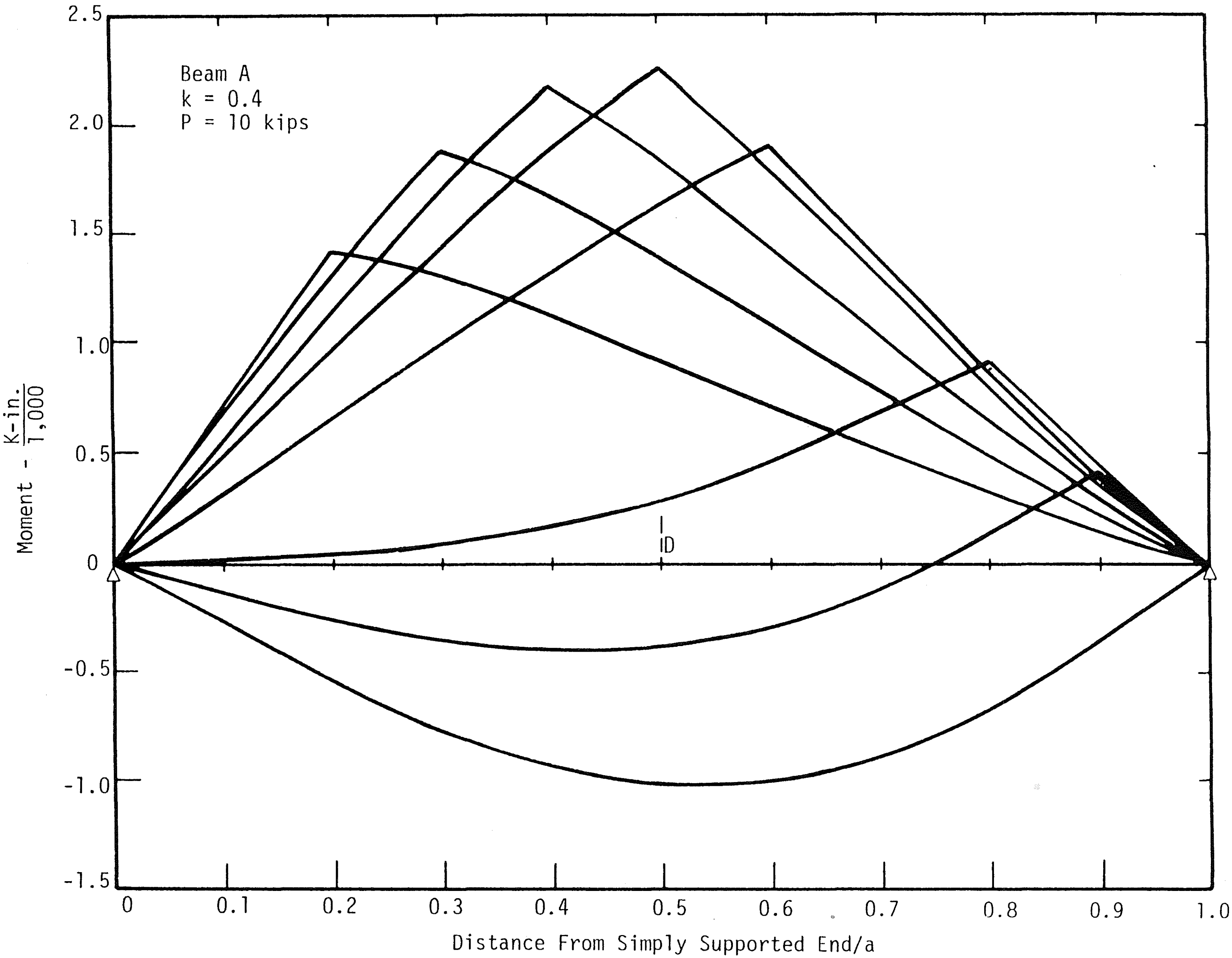


FIG. 4.18 (CONTINUED)

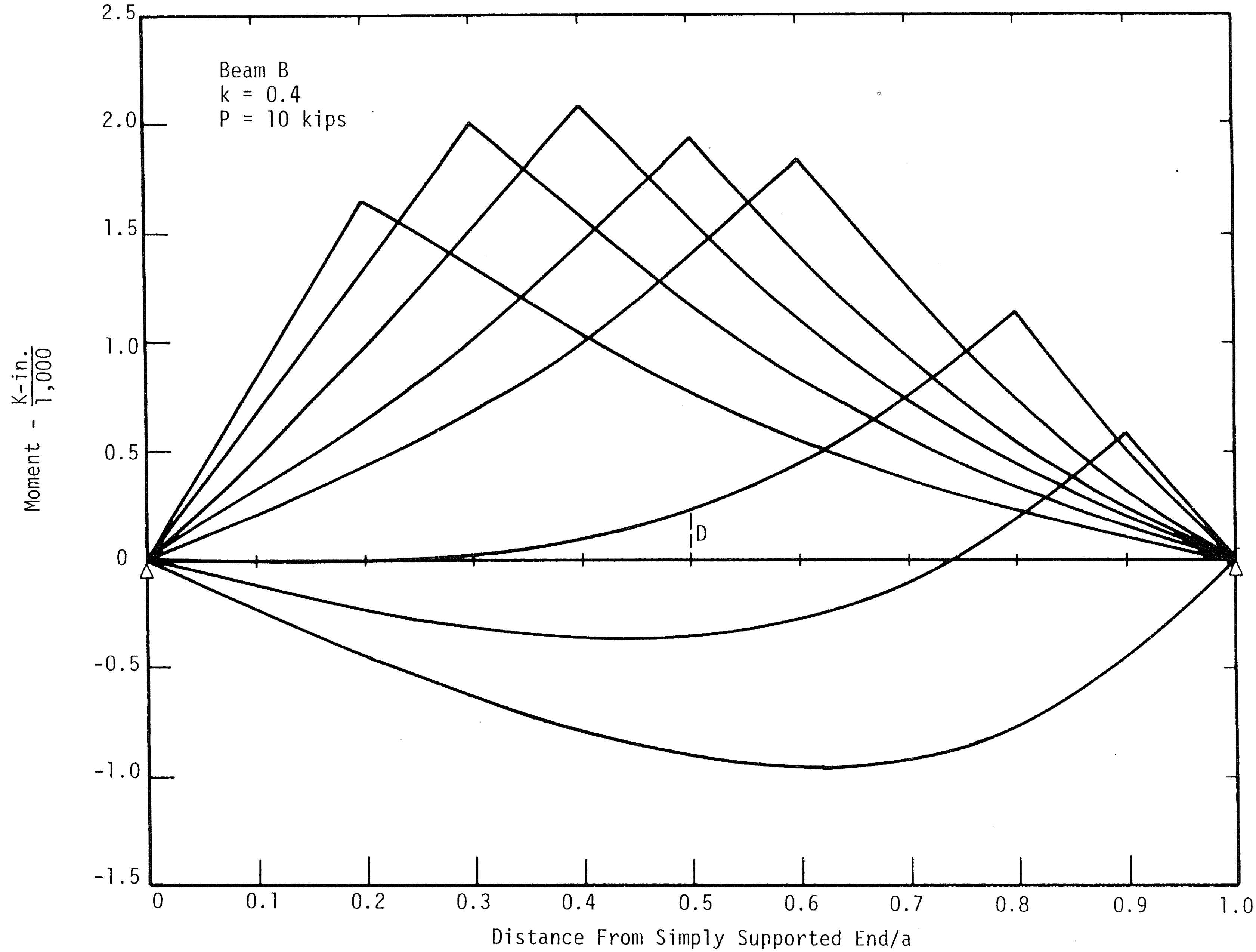


FIG. 4.18 (CONTINUED)

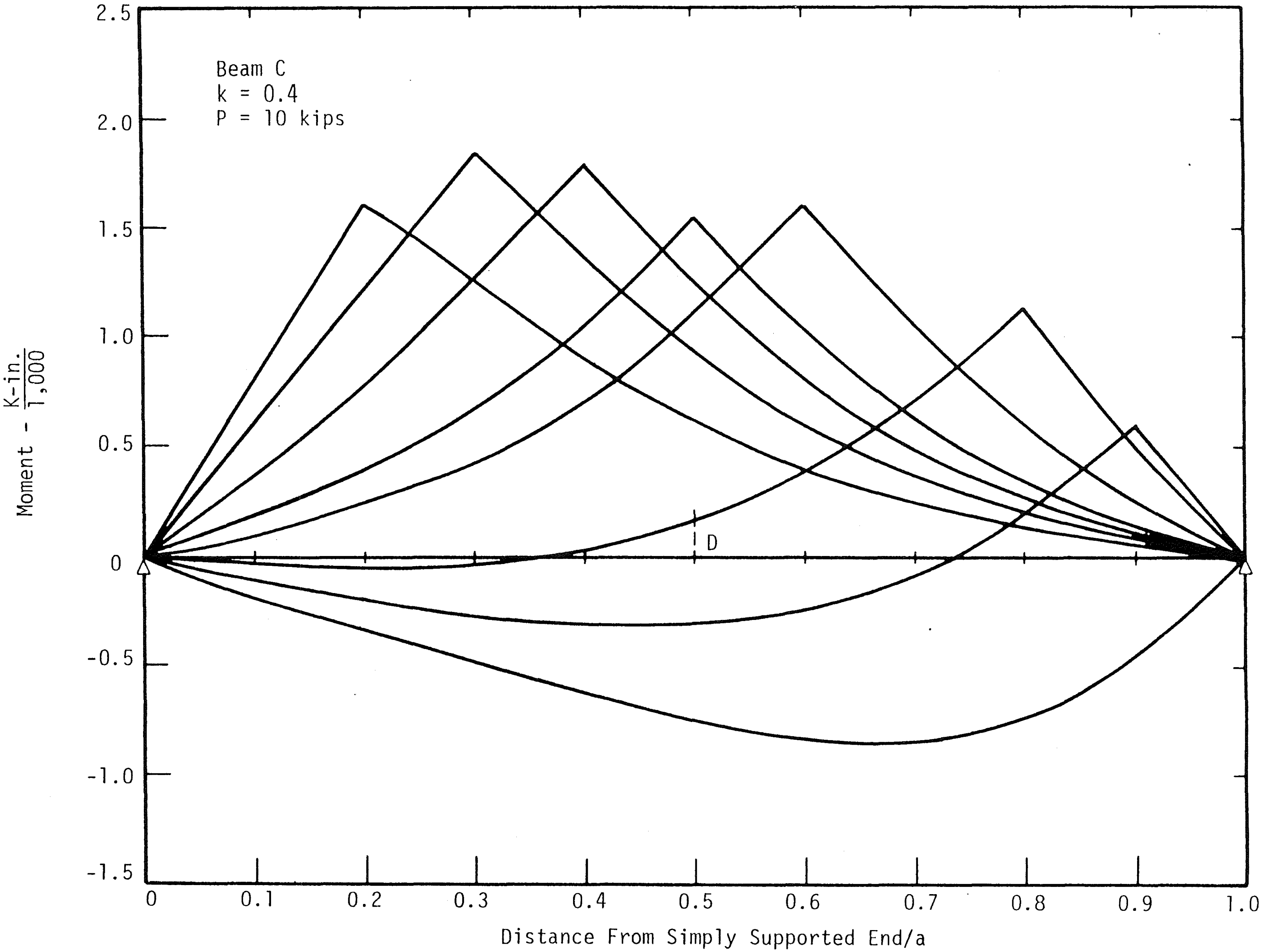


FIG. 4.18 (CONTINUED)

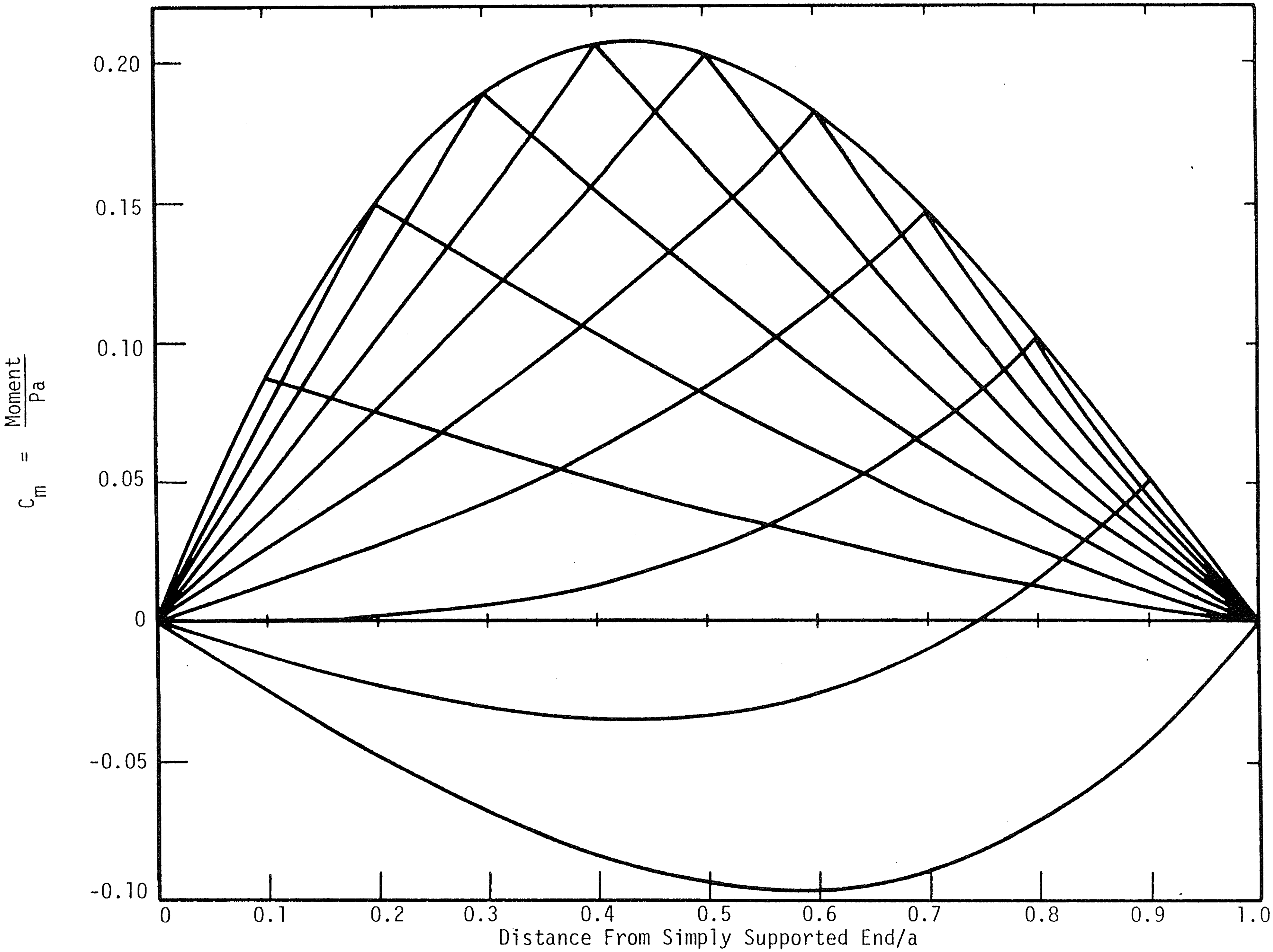


FIG. 4.19 POSITIVE MOMENT ENVELOPE AND INFLUENCE LINES FOR A TWO SPAN CONTINUOUS BEAM DUE TO A CONCENTRATED LOAD

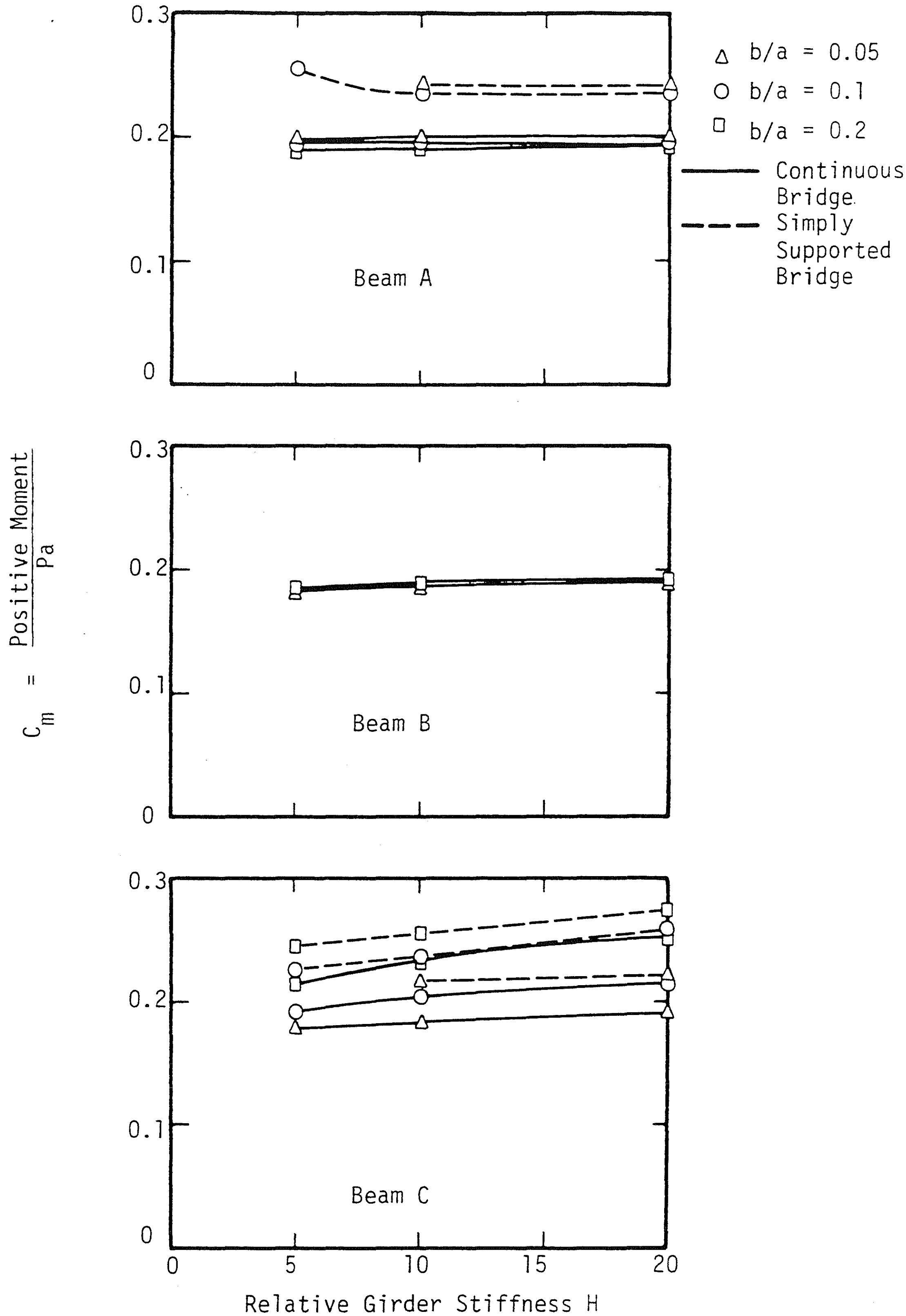


FIG. 4.20 RELATIONSHIPS BETWEEN MAXIMUM POSITIVE MOMENTS IN GIRDERS AND RELATIVE GIRDER STIFFNESS, H; NO DIAPHRAGM, 4W LOADING, b = 5 FT

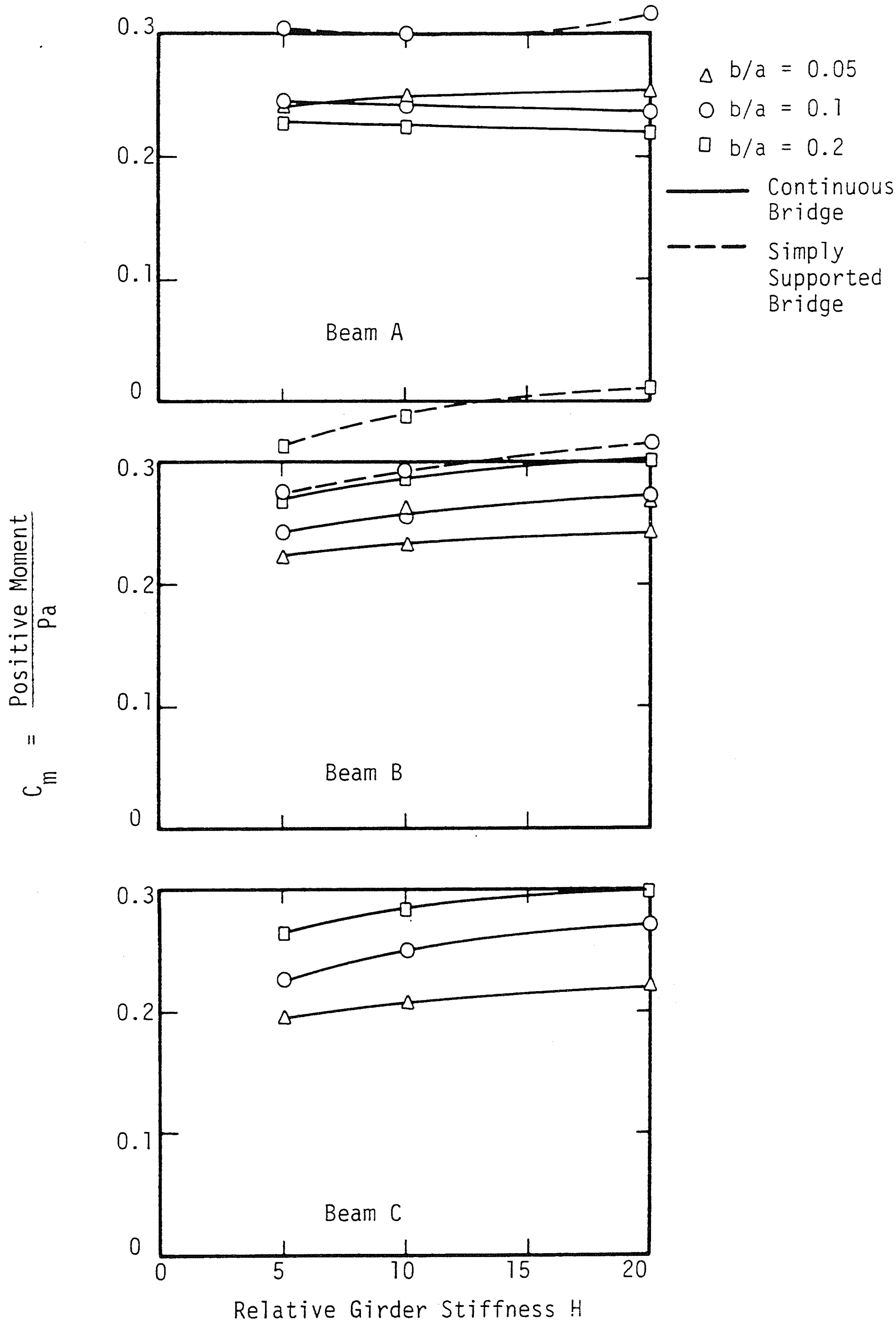


FIG. 4.21 RELATIONSHIPS BETWEEN MAXIMUM POSITIVE MOMENTS IN GIRDERS AND RELATIVE GIRDER STIFFNESS, H; NO DIAPHRAGM, 4W LOADING, b = 7 FT

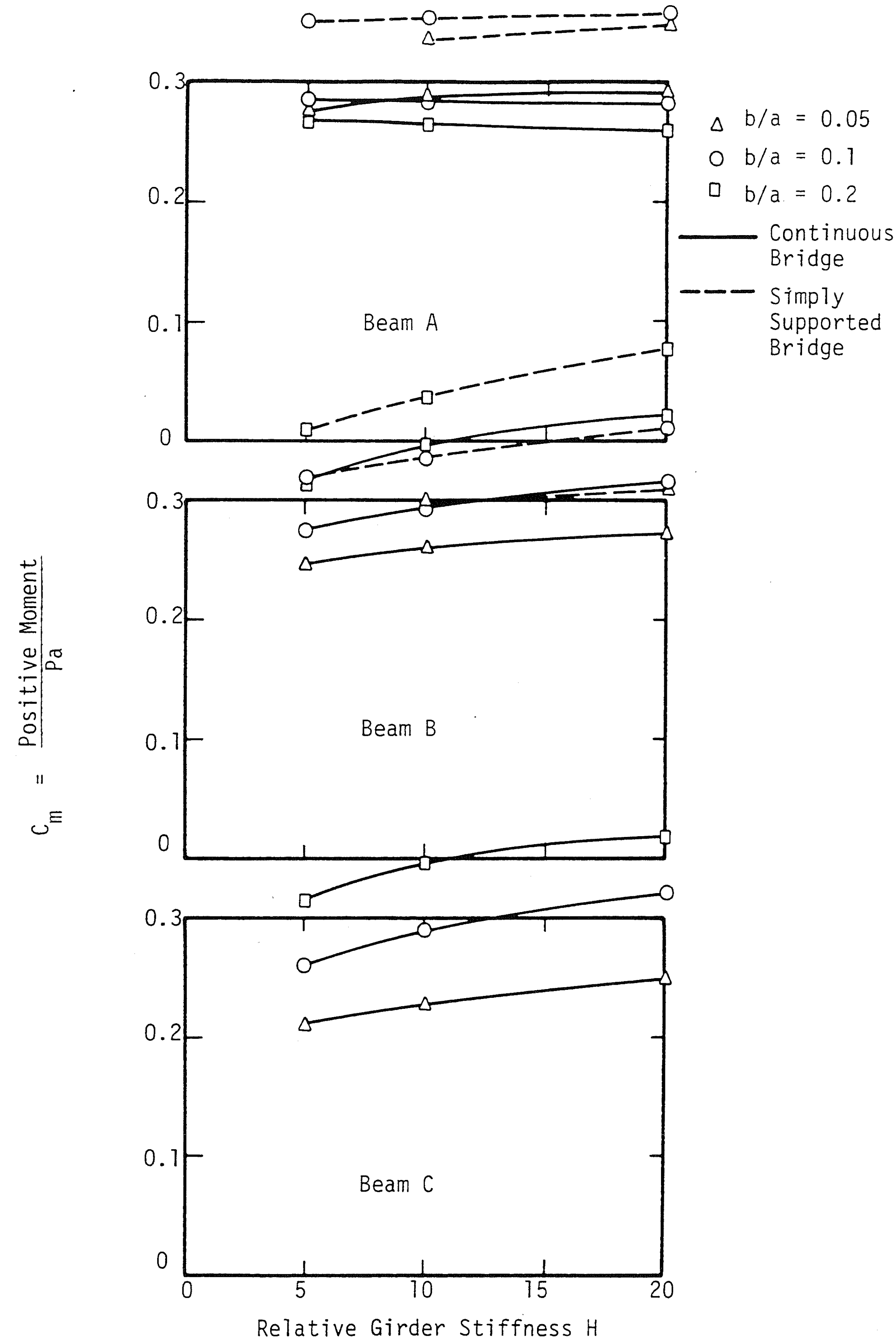


FIG. 4.22 RELATIONSHIPS BETWEEN MAXIMUM POSITIVE MOMENTS IN GIRDERS AND RELATIVE GIRDER STIFFNESS, H; NO DIAPHRAGM, 4W LOADING, b = 9 FT

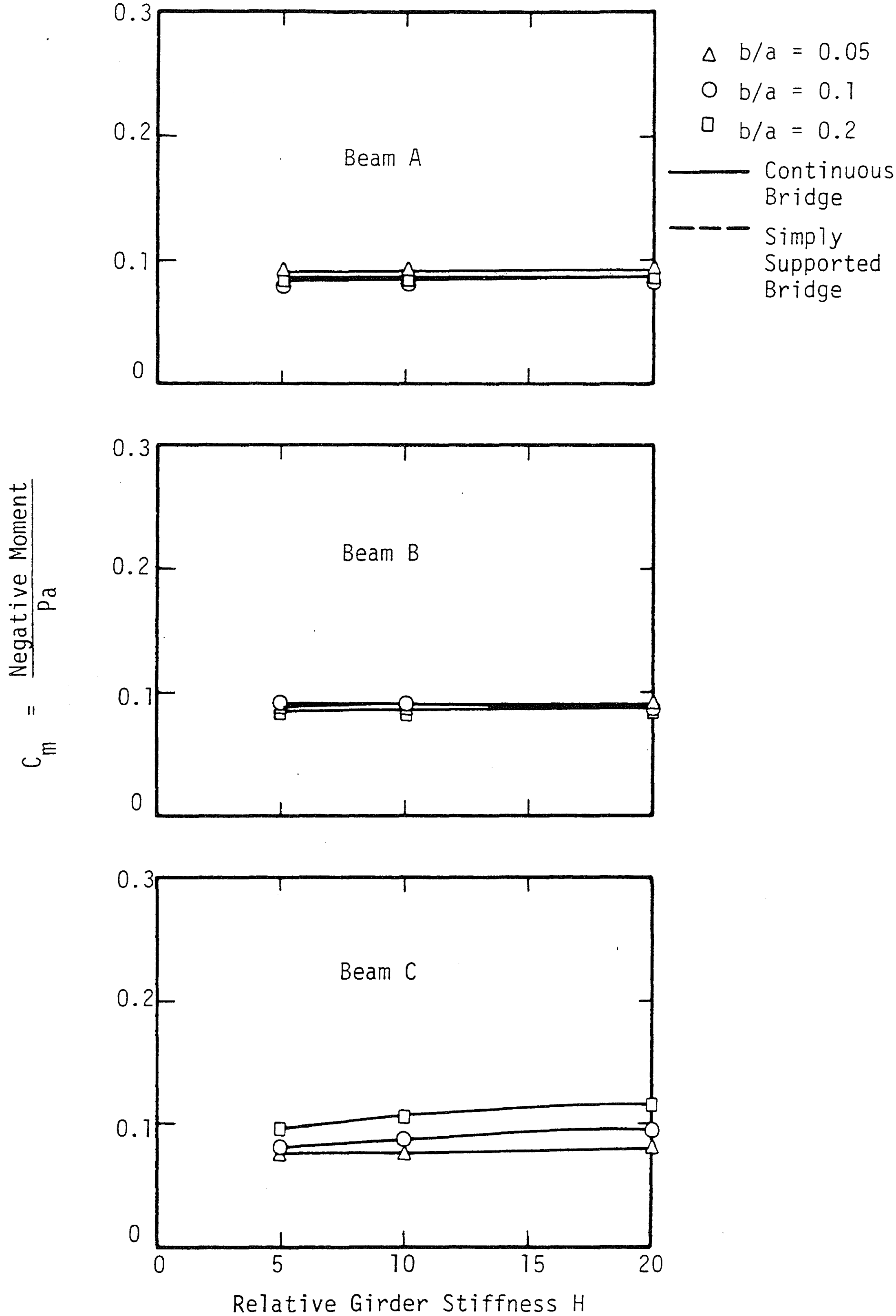


FIG. 4.23 RELATIONSHIPS BETWEEN MAXIMUM NEGATIVE MOMENTS IN GIRDERS AND RELATIVE GIRDER STIFFNESS, H; NO DIAPHRAGM, 4W LOADING, b = 5 FT

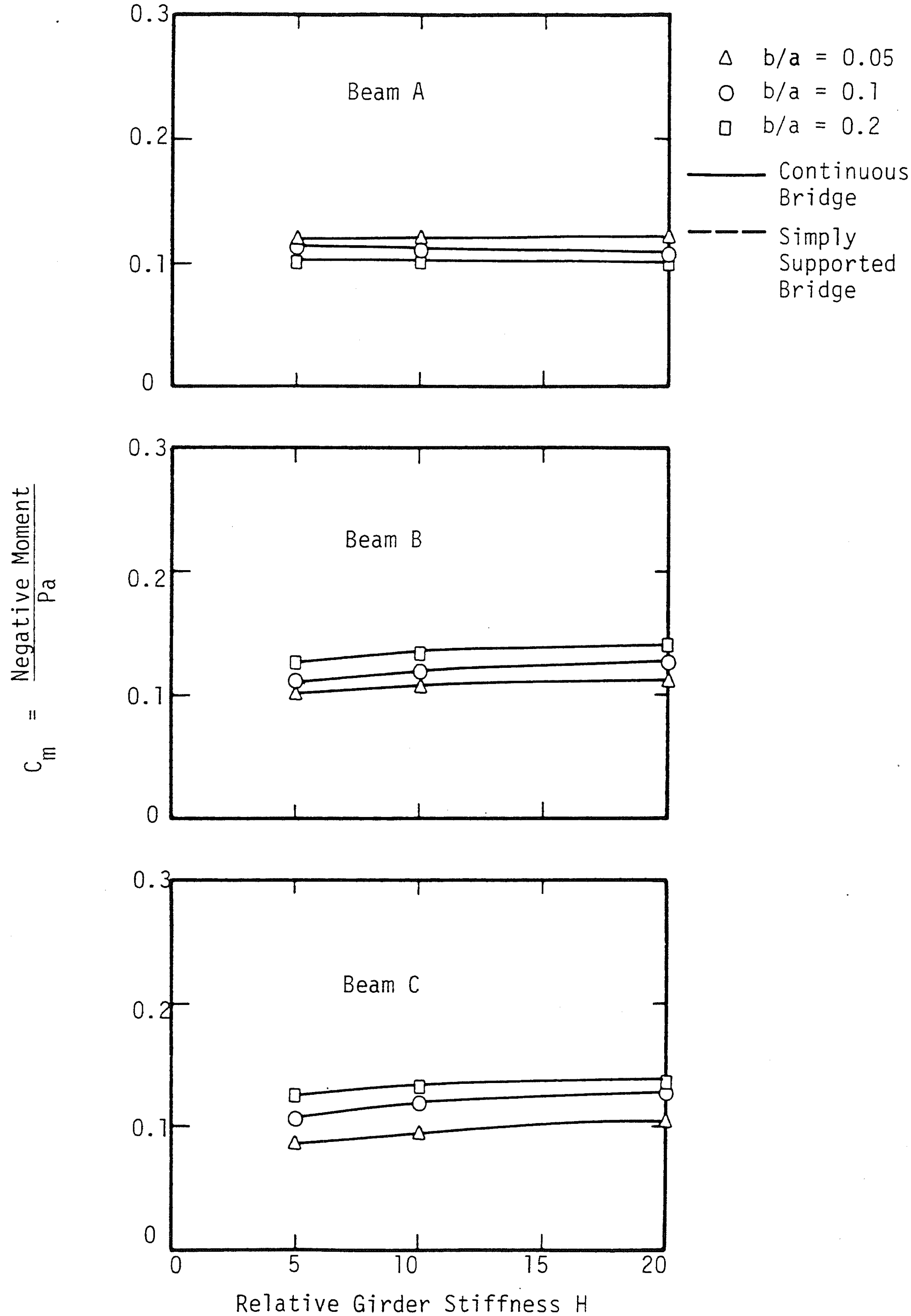


FIG. 4.24 RELATIONSHIPS BETWEEN MAXIMUM NEGATIVE MOMENTS IN GIRDERS AND RELATIVE GIRDER STIFFNESS, H; NO DIAPHRAGM, 4W LOADING, b = 7 FT

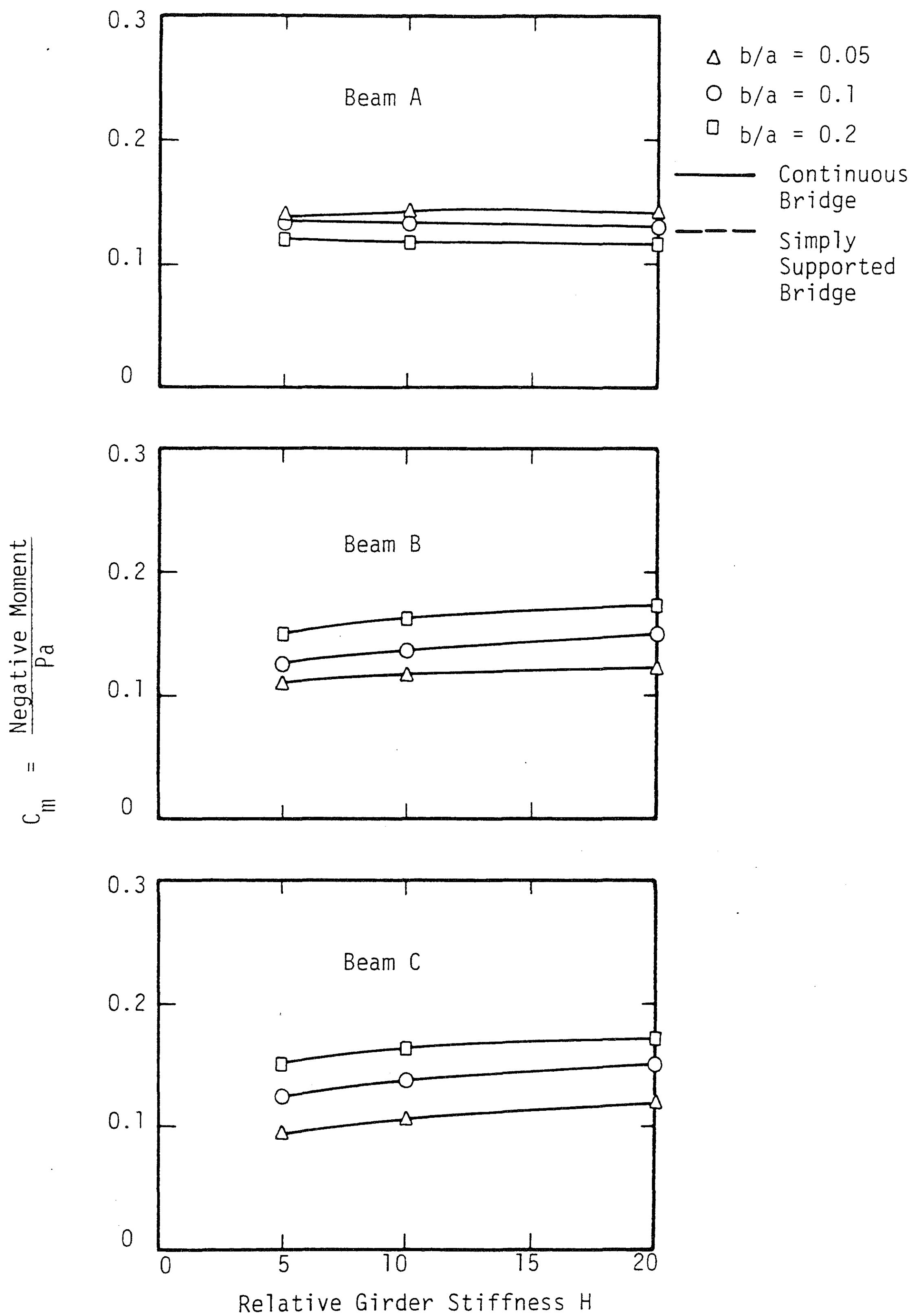


FIG. 4.25 RELATIONSHIPS BETWEEN MAXIMUM NEGATIVE MOMENTS IN GIRDERS AND RELATIVE GIRDER STIFFNESS, H; NO DIAPHRAGM, 4W LOADING, b = 9 FT

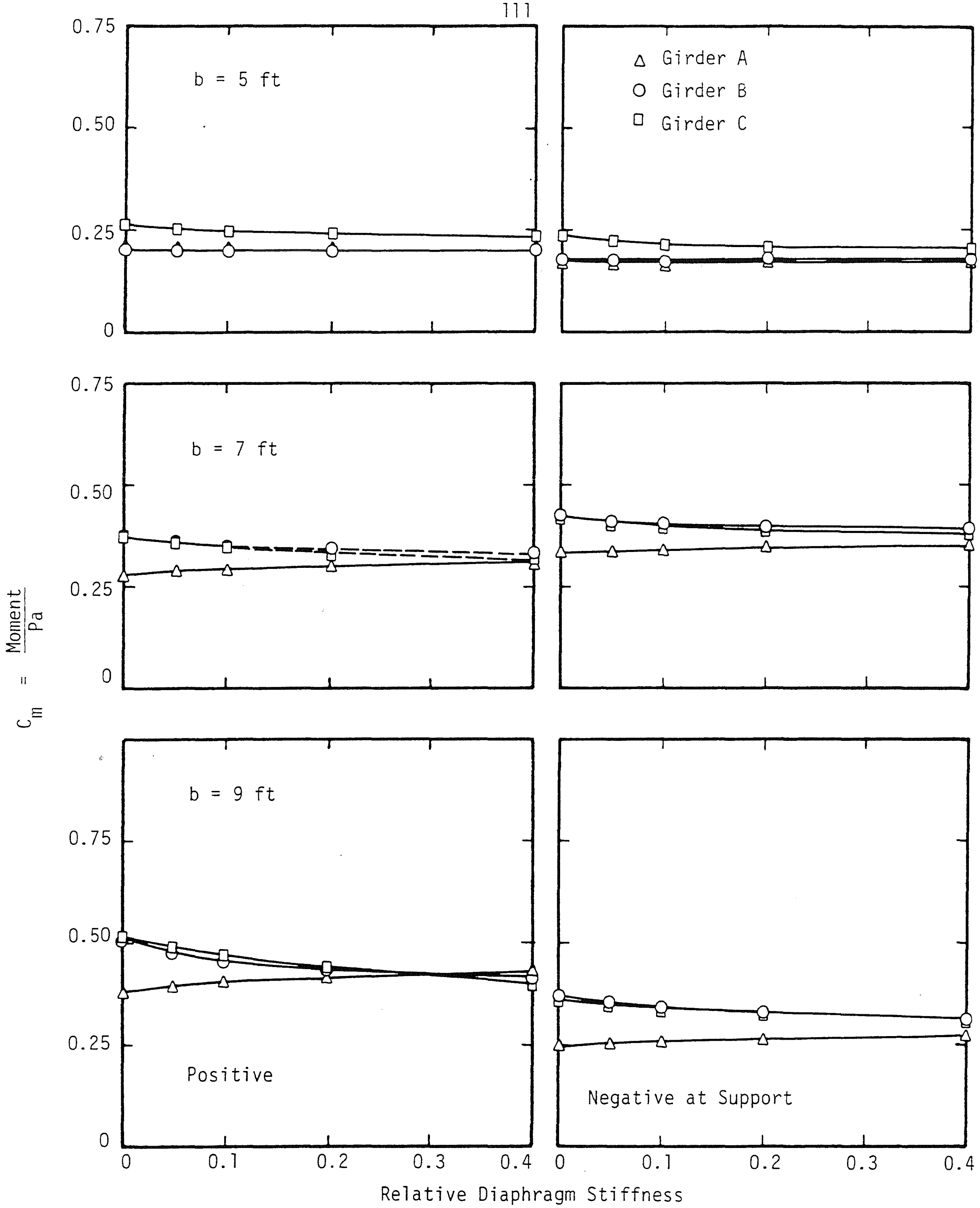


FIG. 4.26 MAXIMUM MOMENTS DUE TO TWO-TRUCK LOADING VERSUS DIAPHRAGM STIFFNESS; H = 20, b/a = 0.2

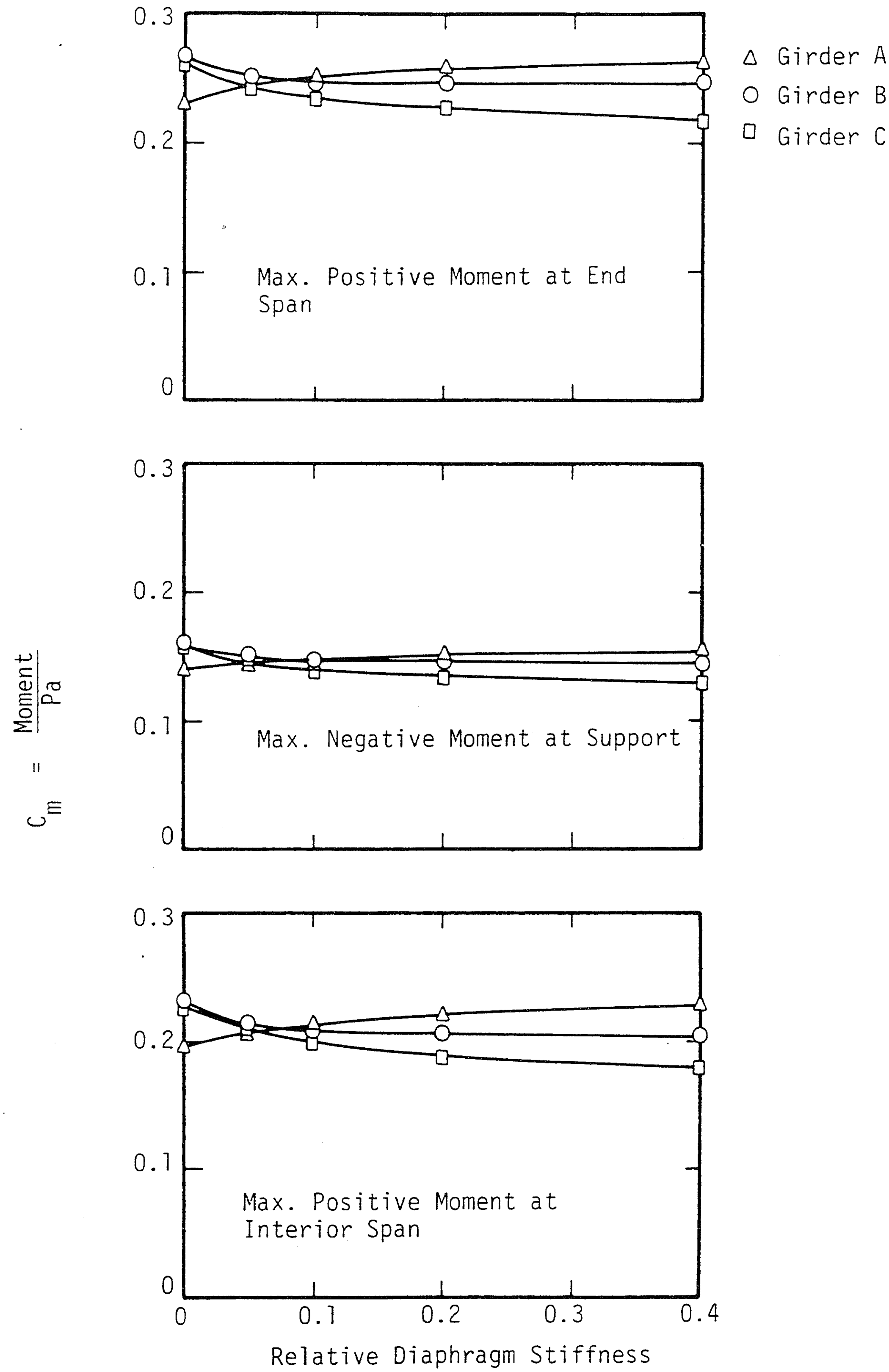


FIG. 4.27 MAXIMUM MOMENTS IN GIRDERS DUE TO 4W LOADING VERSUS DIAPHRAGM STIFFNESS; THREE SPAN CONTINUOUS BRIDGE, H = 20, b/a = 0.1, b = 7 FT

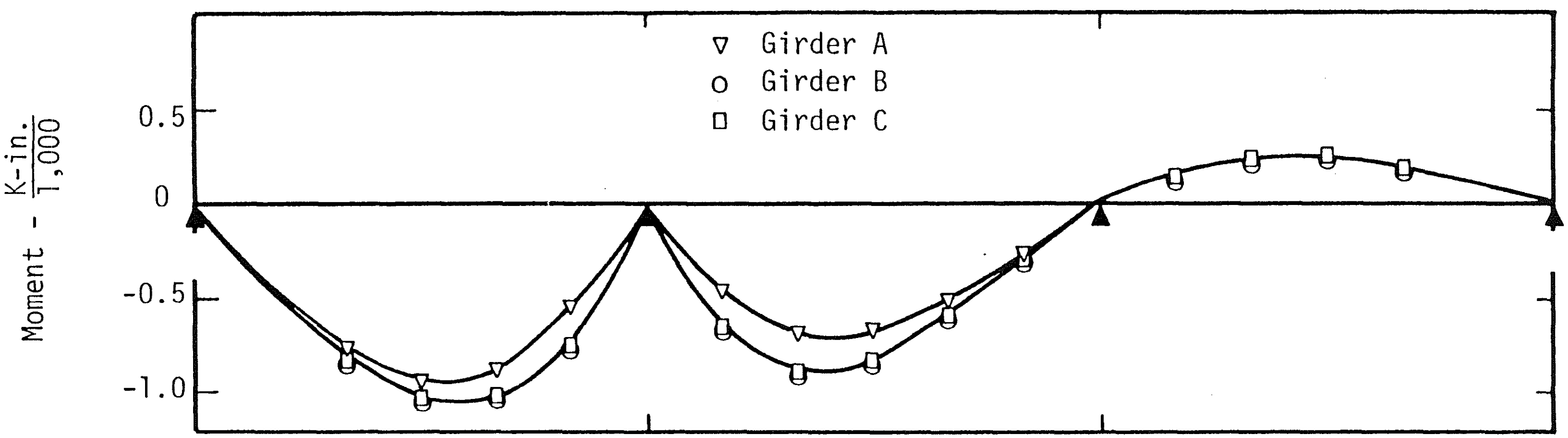


FIG. 4.28 INFLUENCE LINES FOR NEGATIVE MOMENTS AT INTERIOR SUPPORT OF THREE SPAN CONTINUOUS BRIDGE DUE TO 4W LOADING; H = 20, b/a = 0.1, b = 7 FT

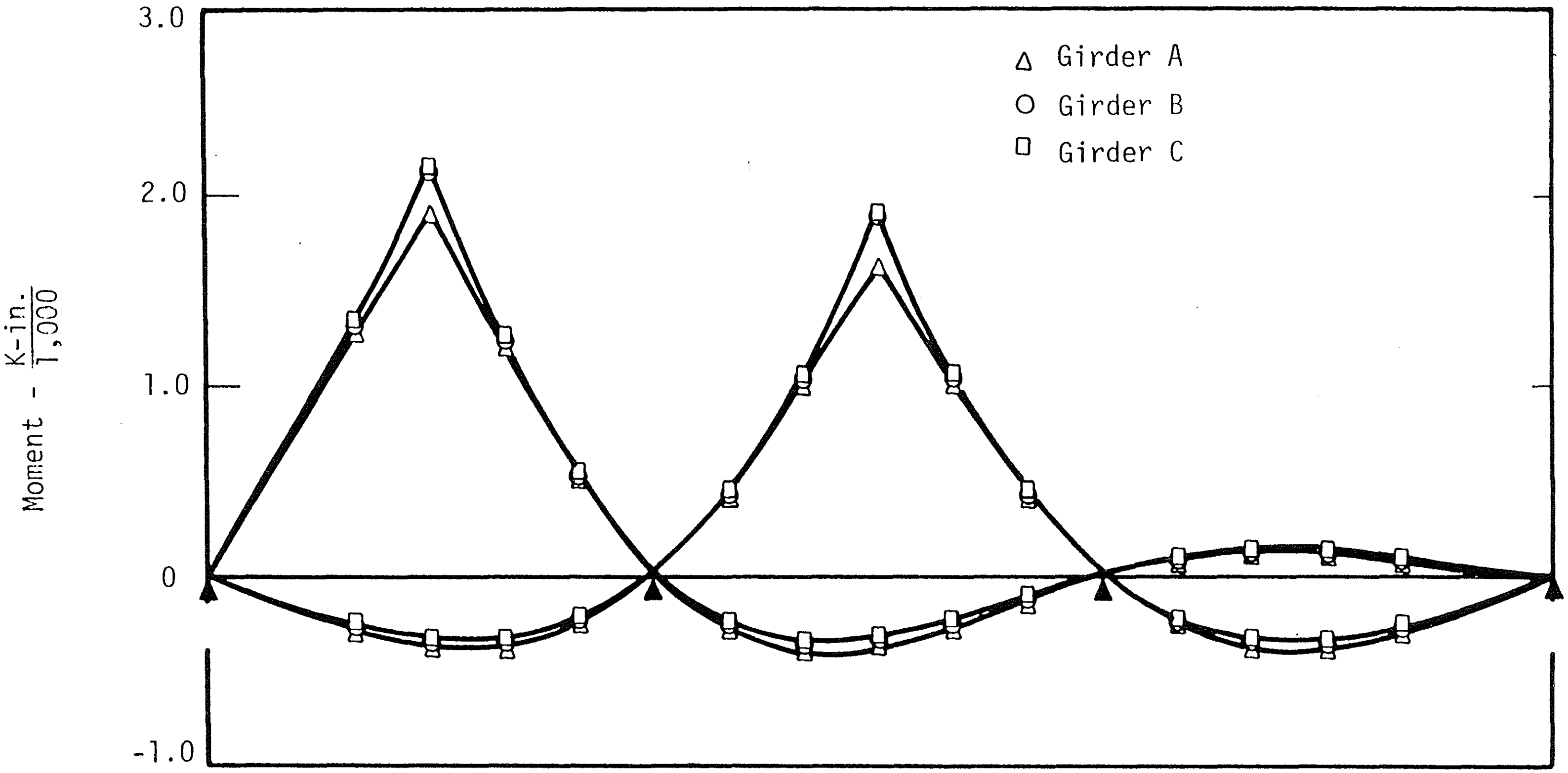
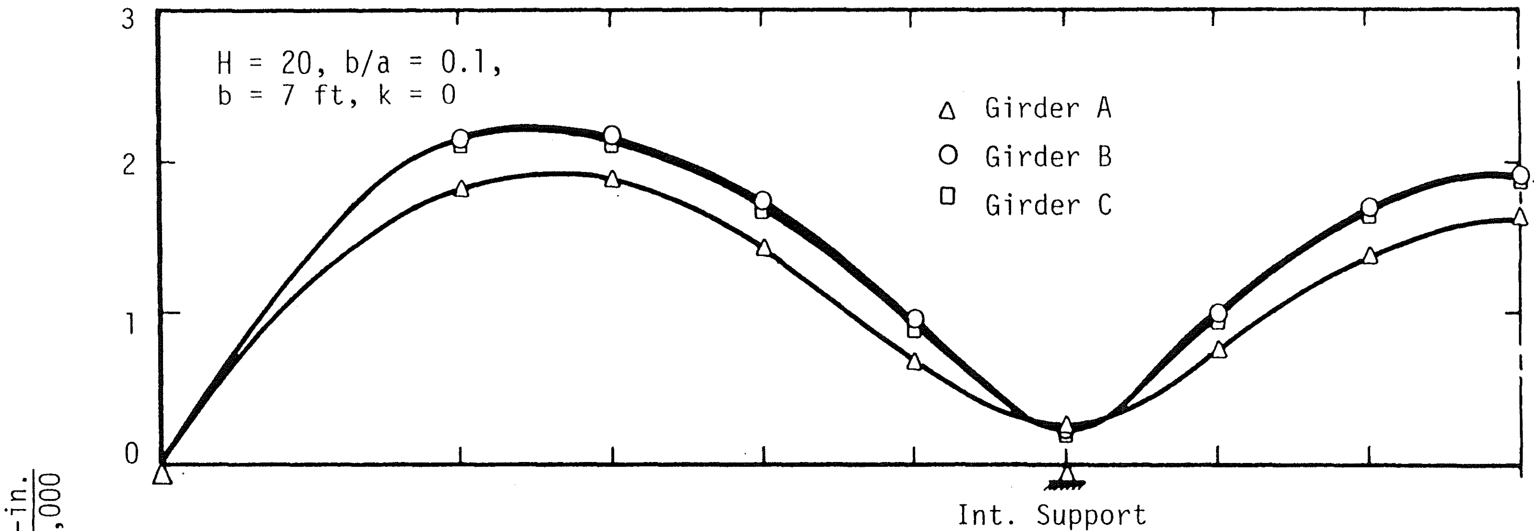
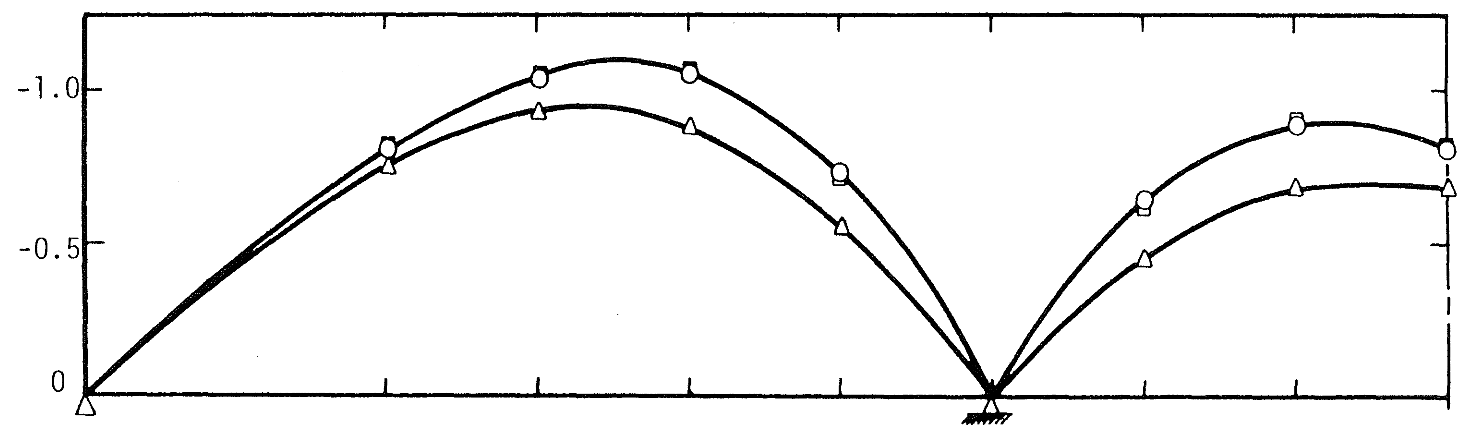


FIG. 4.29 INFLUENCE LINES FOR MIDSPAN POSITIVE MOMENTS IN GIRDERS OF THREE SPAN CONTINUOUS BRIDGE DUE TO 4W LOADING; H = 20, b/a = 0.1, b = 7 FT

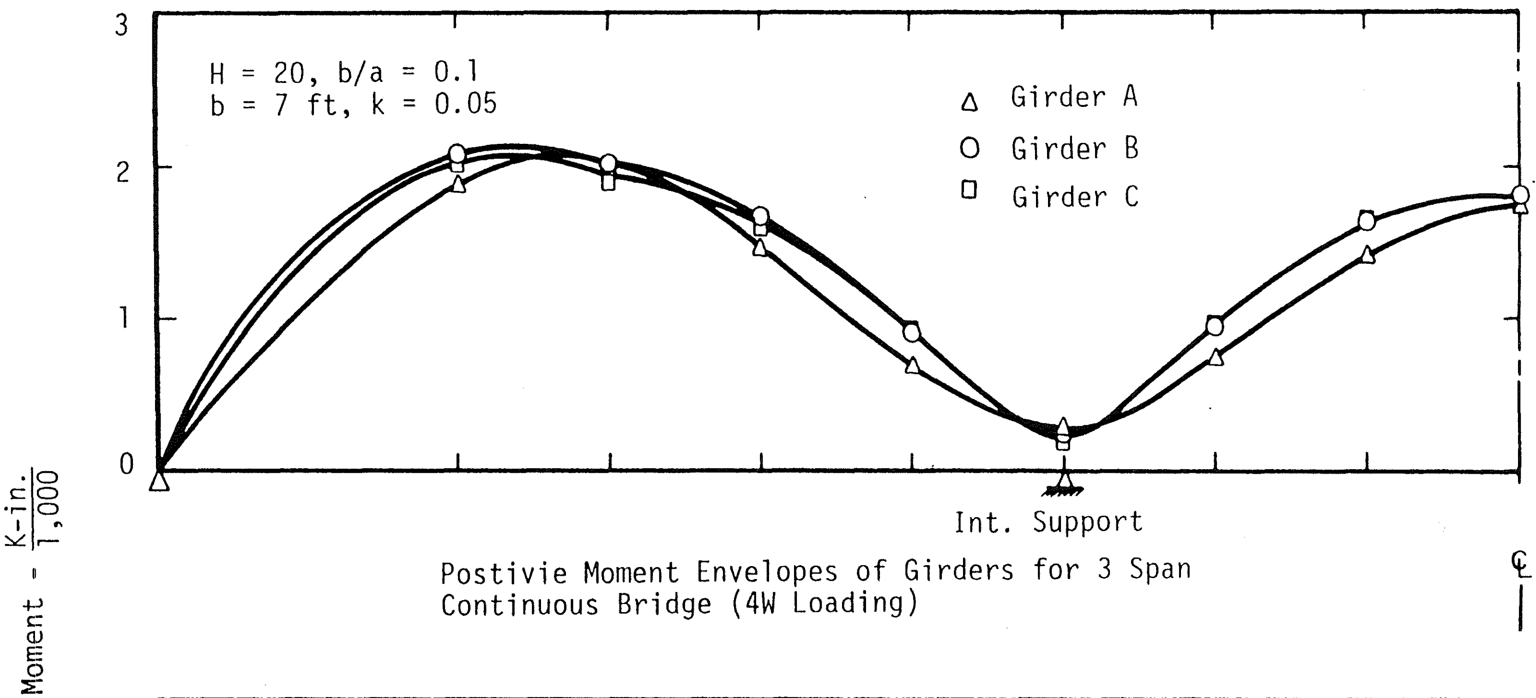


Positive Moment Envelopes of Girders for 4 Span Continuous Bridge (4W Loading)

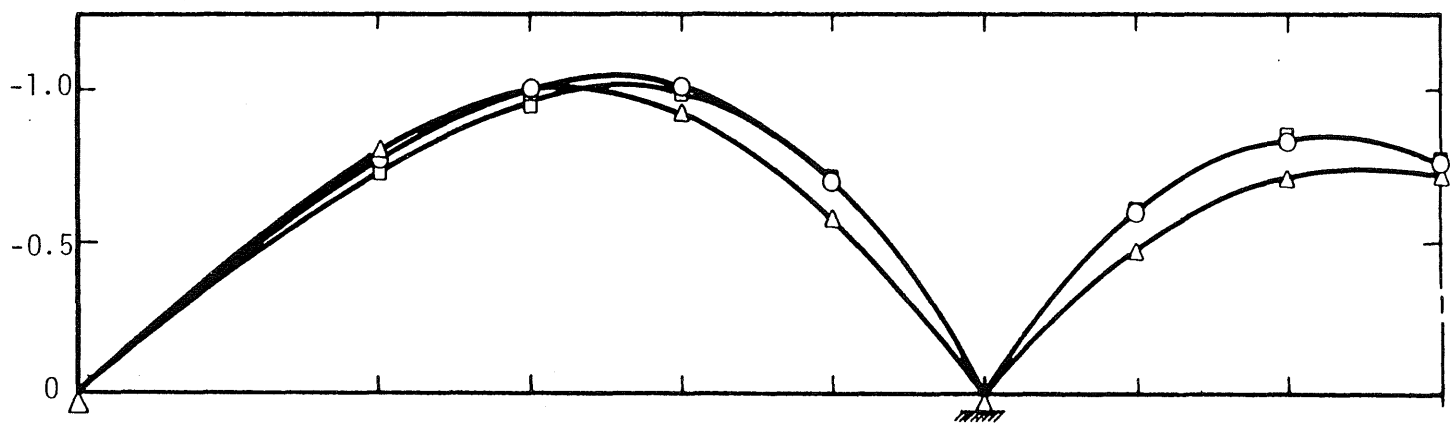


Influence Lines for Maximum Negative Moments in Girders at Interior Support - 4W Loading

FIG. 4.30 POSITIVE MOMENT ENVELOPES AND NEGATIVE MOMENT INFLUENCE LINES FOR THREE SPAN CONTINUOUS BRIDGES WITH VARIOUS DIAPHRAGM STIFFNESS

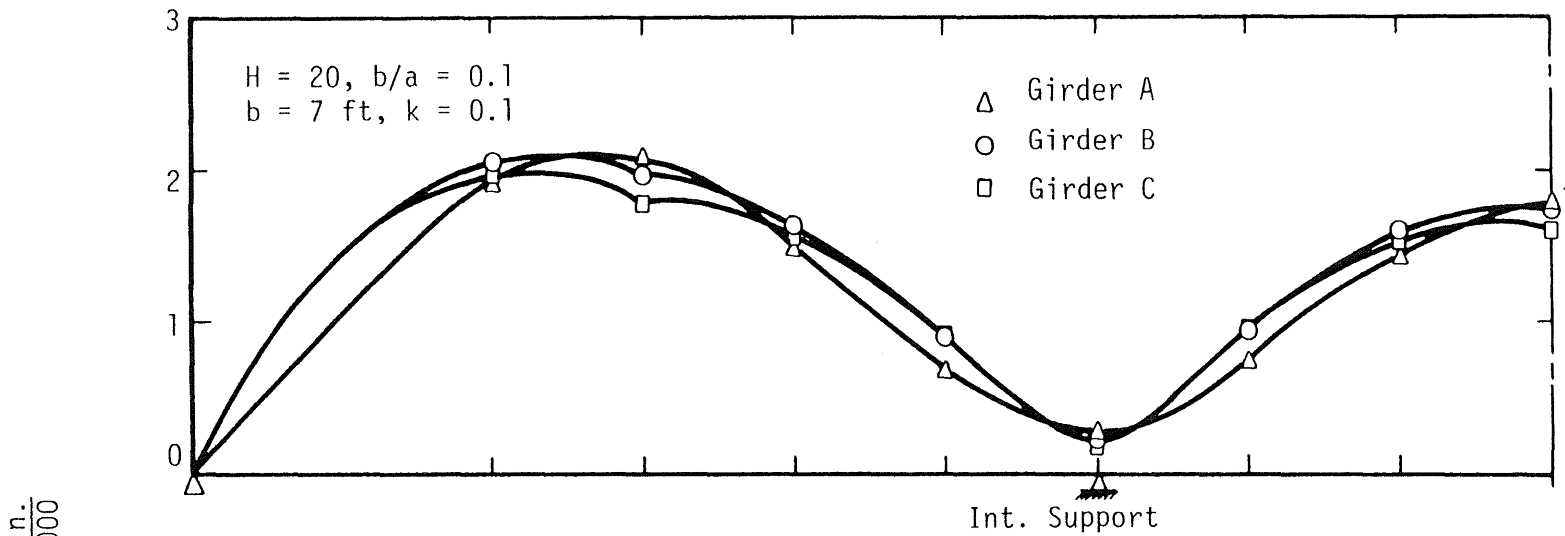


Postivie Moment Envelopes of Girders for 3 Span Continuous Bridge (4W Loading)



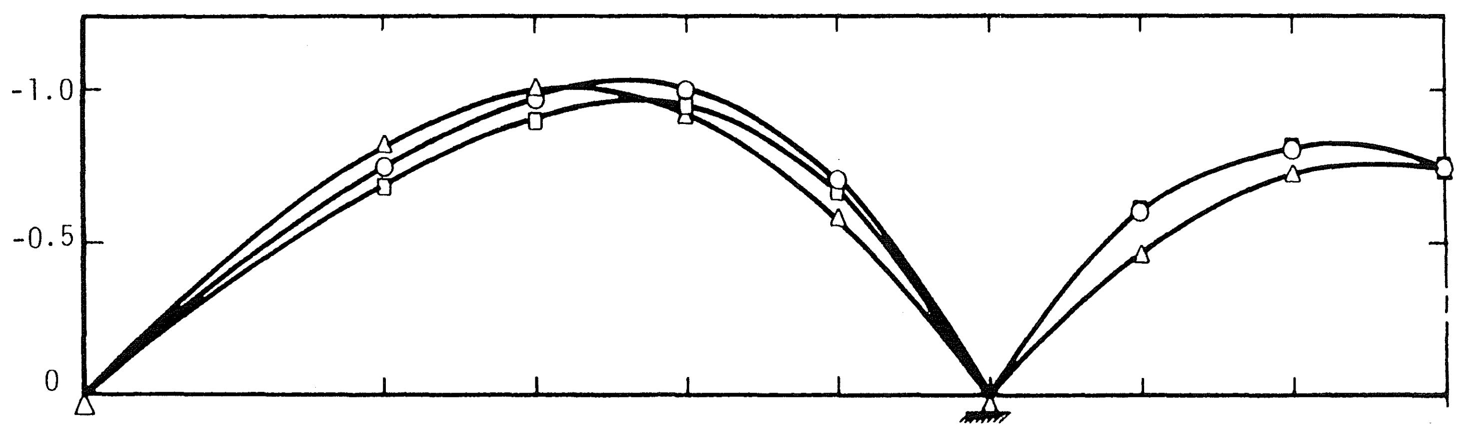
Influence Lines for Maximum Negative Moments in Girders at Interior Support - 4W Loading

FIG. 4.30 (CONTINUED)



Positive Moment Envelopes of Girders for 3 Span Continuous Bridge (4W Loading)

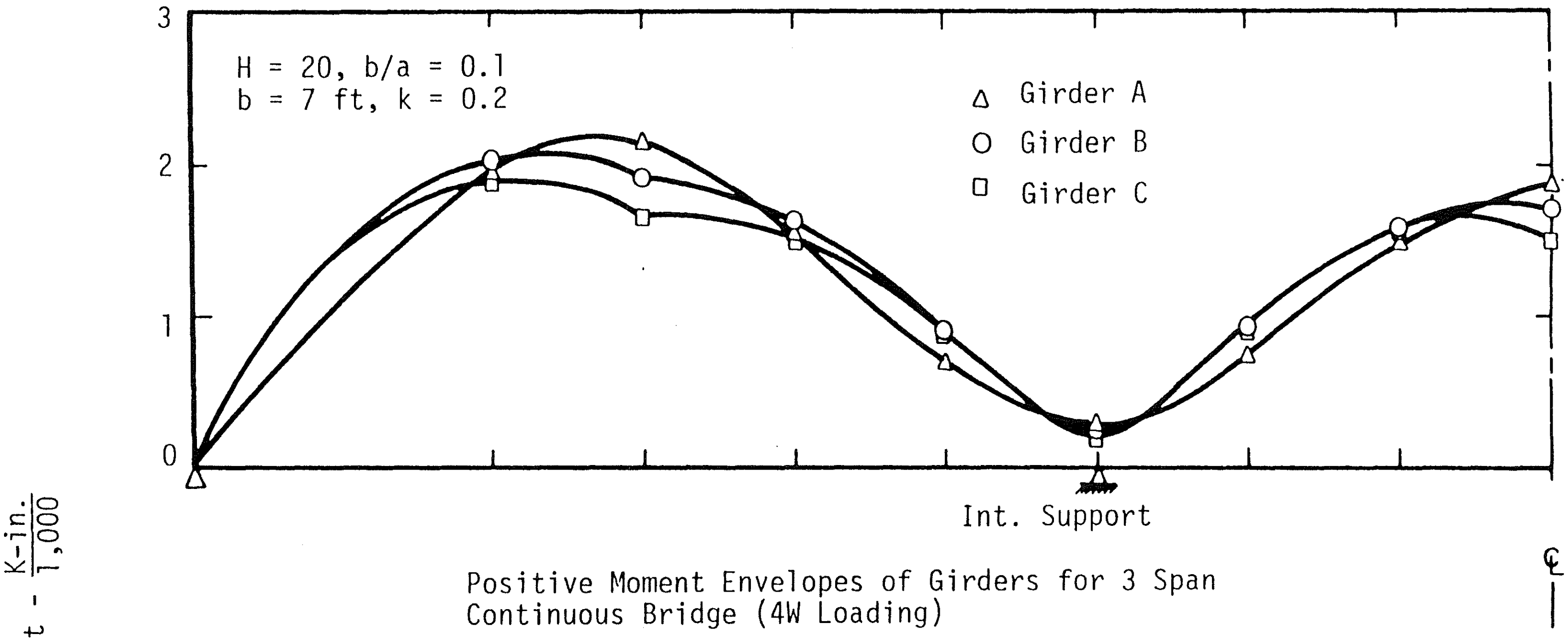
℄



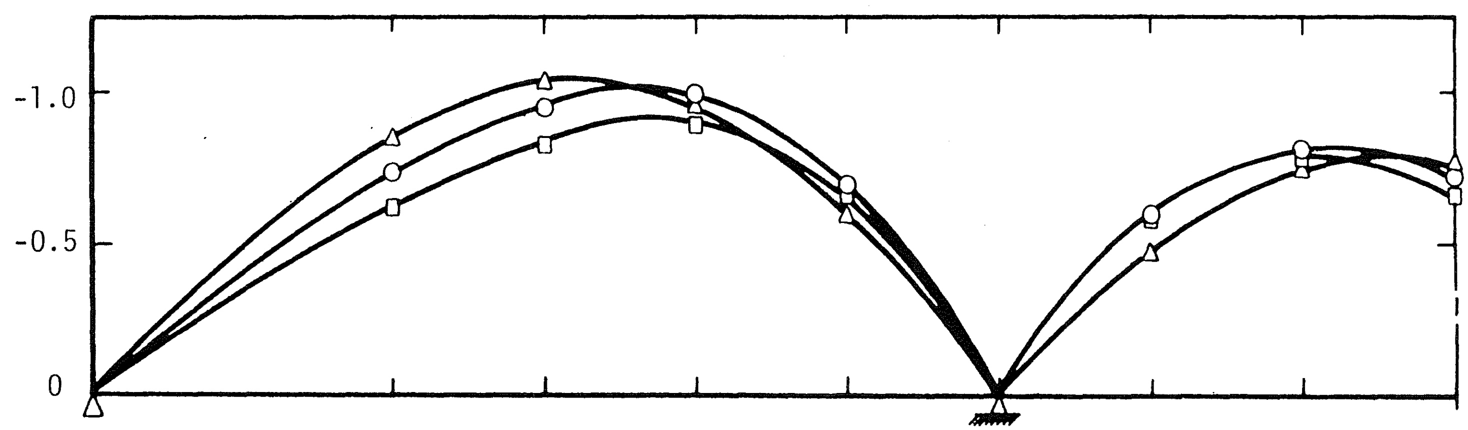
Influence Lines for Maximum Negative Moments in Girders at Interior Support - 4W Loading

℄

FIG. 4.30 (CONTINUED)

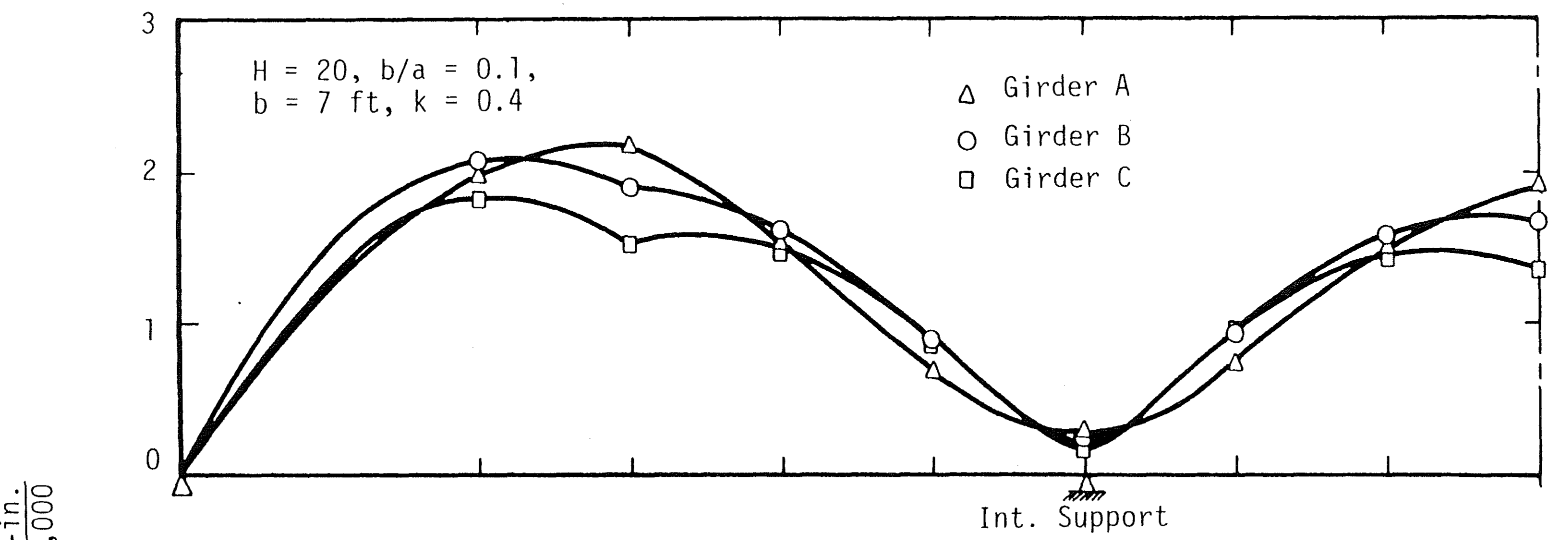


Positive Moment Envelopes of Girders for 3 Span Continuous Bridge (4W Loading)

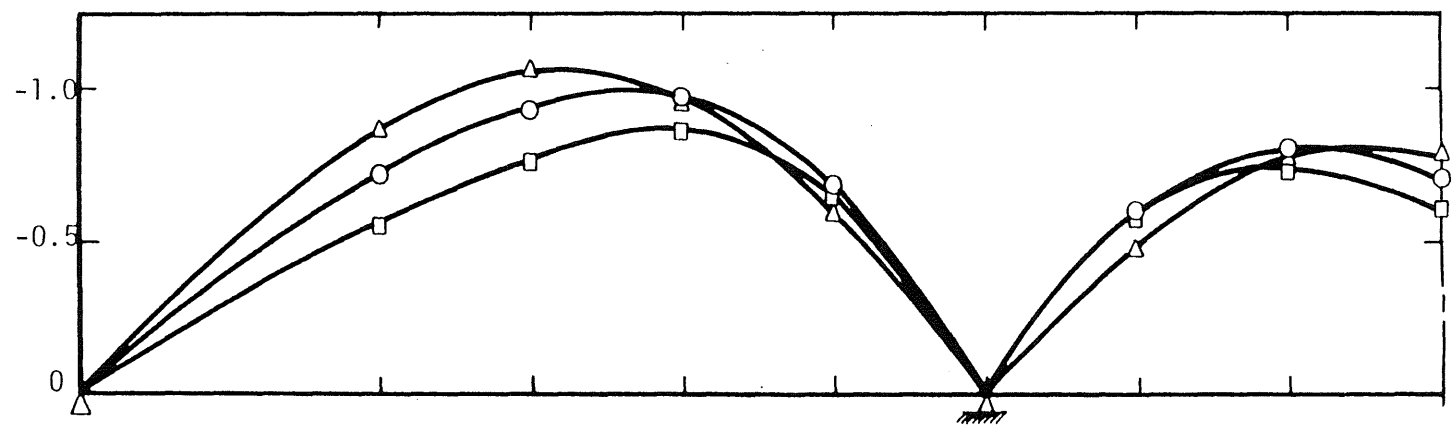


Influence Lines for Maximum Negative Moments in Girders at Interior Support - 4W Loading

FIG. 4.30 (CONTINUED)



Positive Moment Envelopes of Girders for 3 Span Continuous Bridge (4W Loading)



Influence Lines for Maximum Negative Moments in Girders at Interior Support - 4W Loading

FIG. 4.30 (CONTINUED)

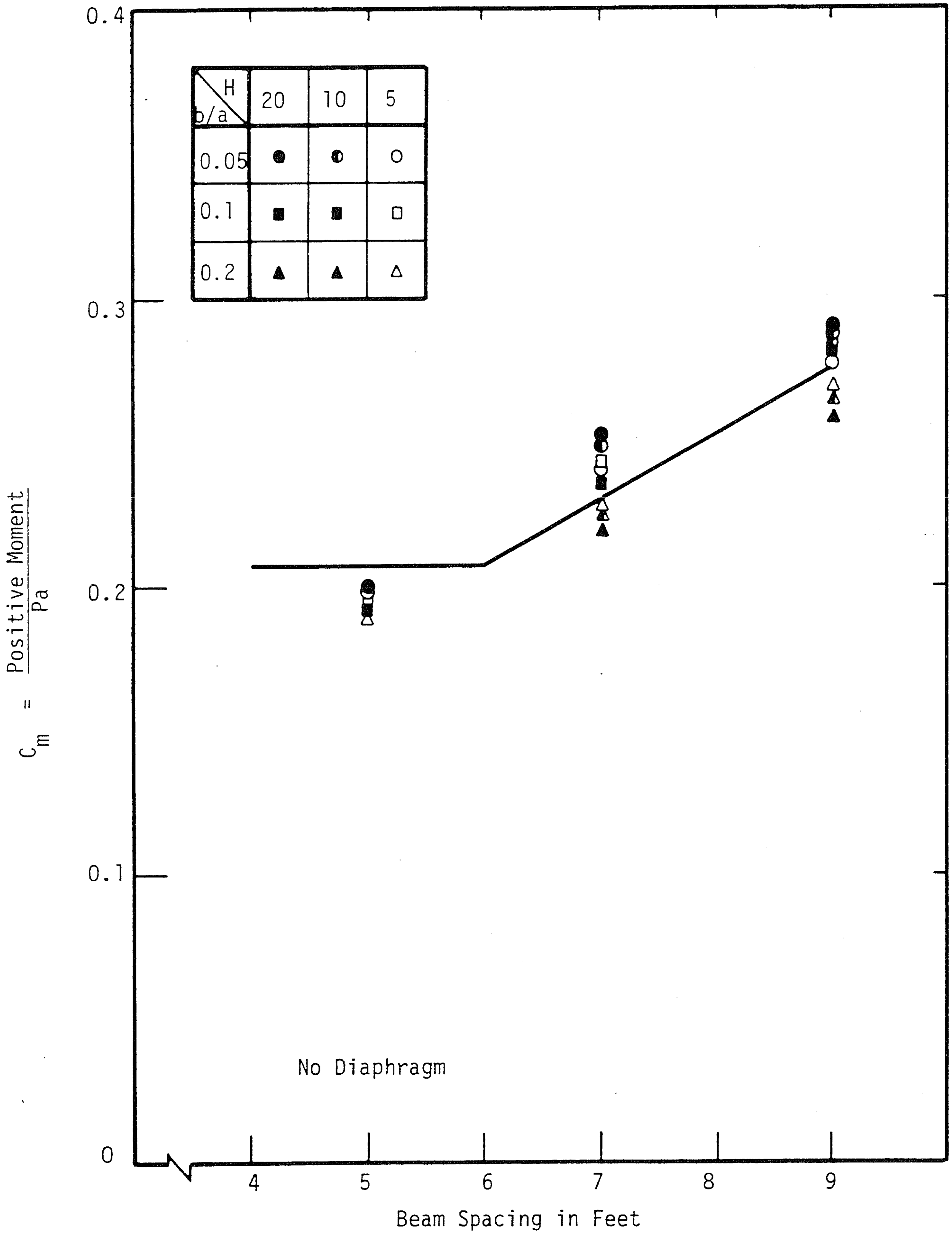


FIG. 5.1 MAXIMUM POSITIVE MOMENTS DUE TO 4W LOADING COMPARED WITH AASHO DESIGN VALUES; BEAM A

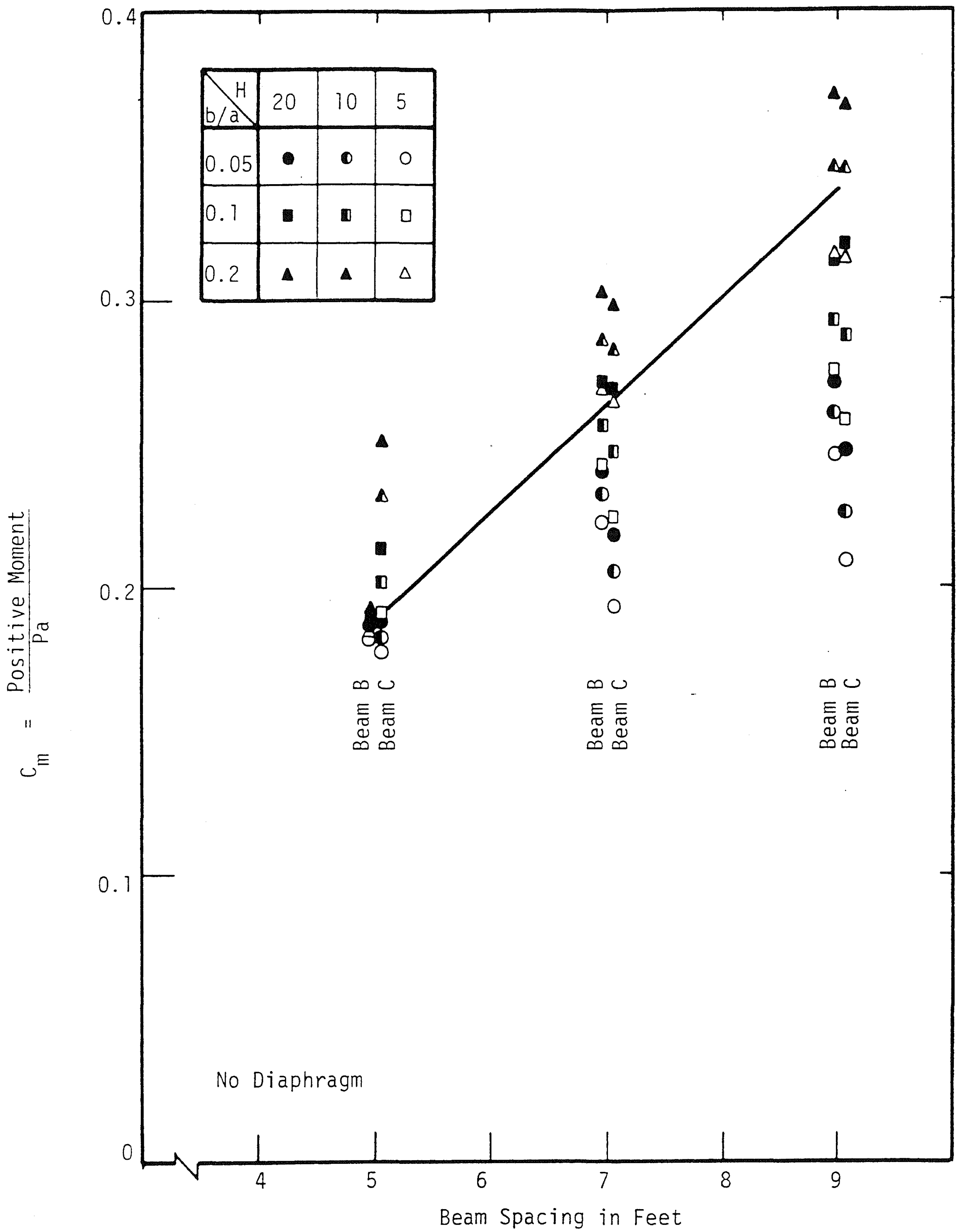


FIG. 5.2 MAXIMUM POSITIVE MOMENTS DUE TO 4W LOADING COMPARED WITH AASHO DESIGN VALUES; INTERIOR BEAMS

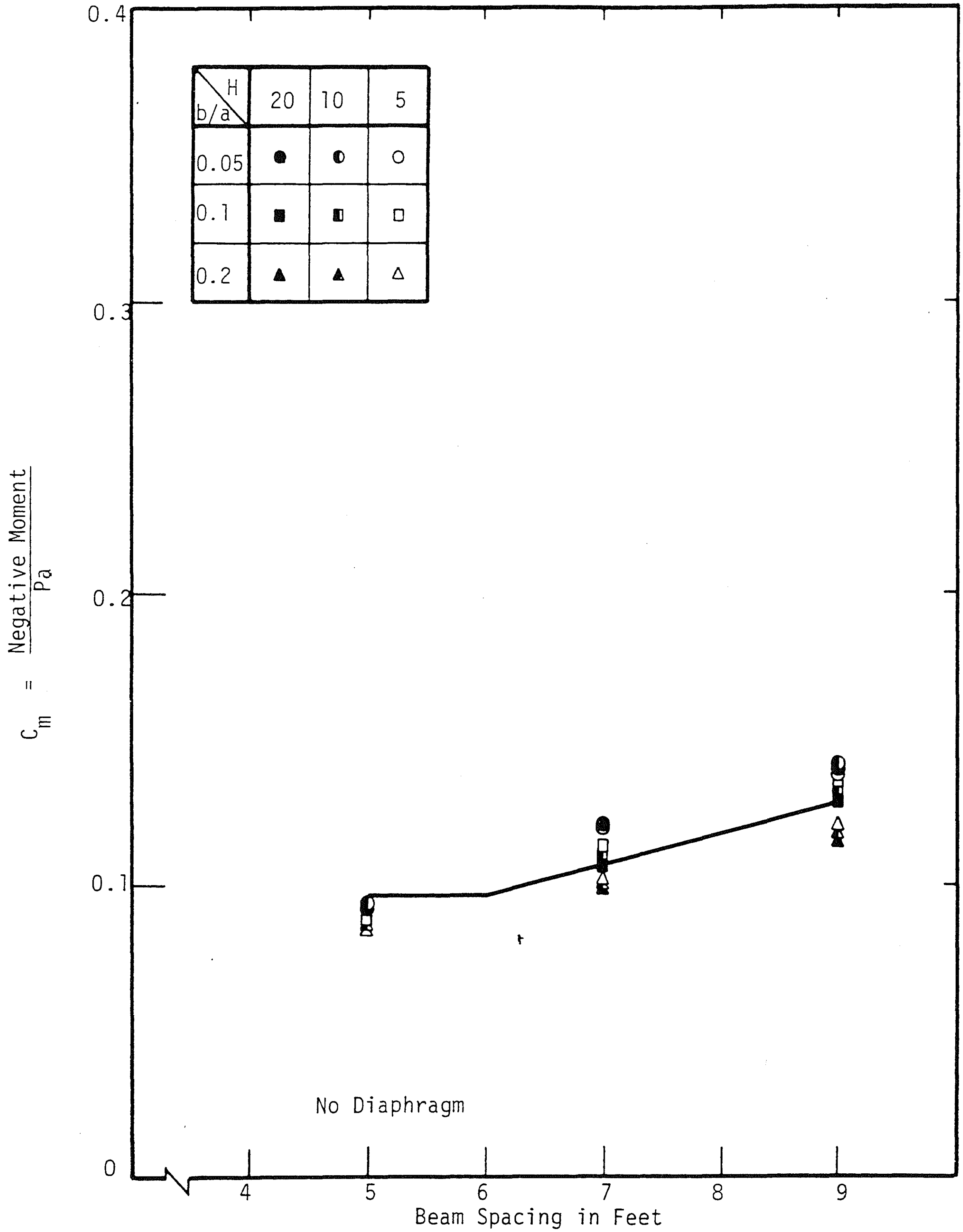


FIG. 5.3 MAXIMUM NEGATIVE MOMENTS DUE TO 4W LOADING COMPARED WITH AASHO DESIGN VALUES; BEAM A

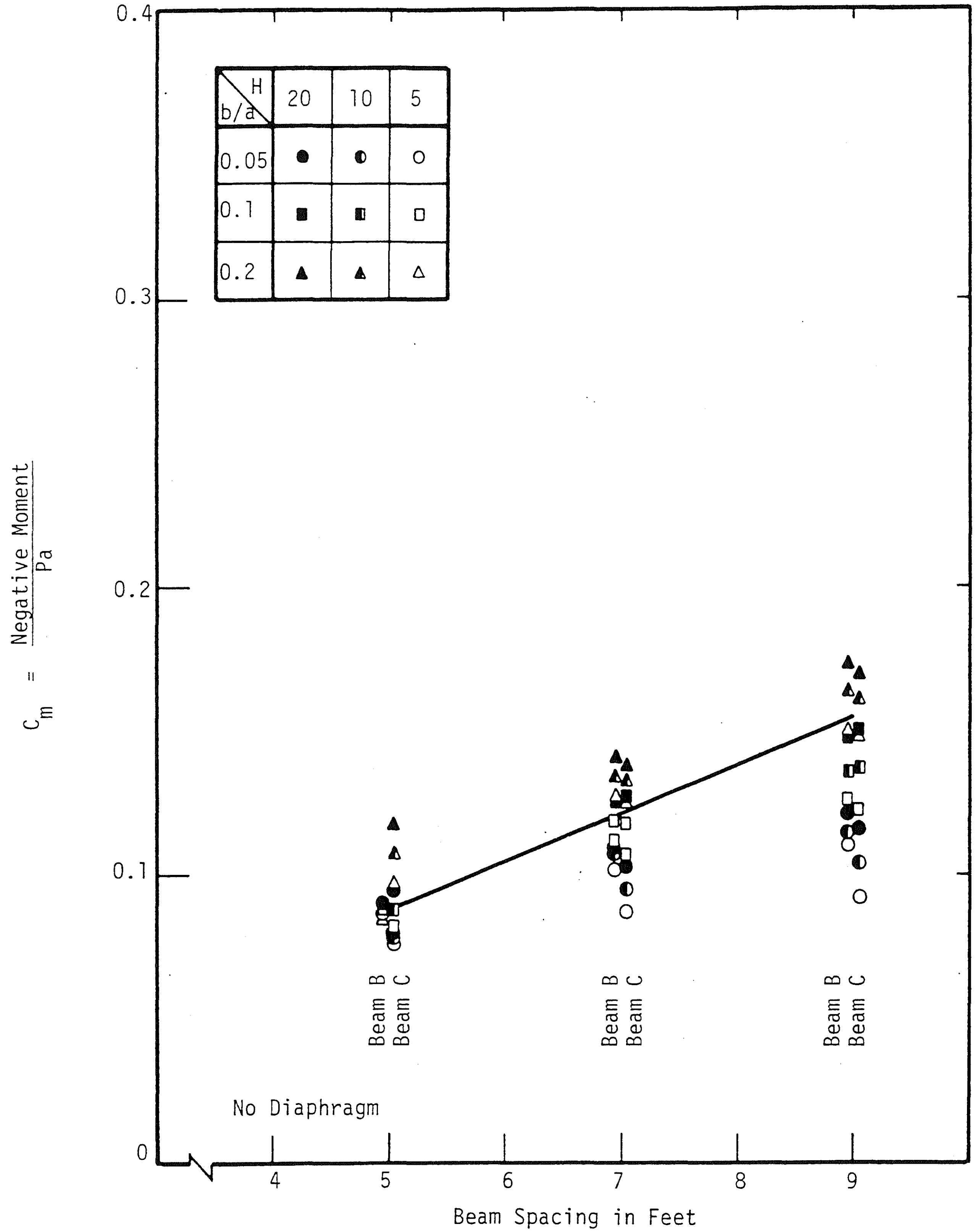


FIG. 5.4 MAXIMUM NEGATIVE MOMENTS DUE TO 4W LOADING COMPARED WITH AASHO DESIGN VALUES; INTERIOR BEAMS