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I am submitting herewith a dissertation written by James Harold Deatherage entitled "Investigation of Variables Affecting Beam-Slab Bridge Load Capacity." I have examined the final copy of this dissertation for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, with a major in Civil Engineering.

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INVESTIGATION OF VARIABLES AFFECTING
BEAM-SLAB BRIDGE LOAD CAPACITY

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ABSTRACT

The ability to predict the load-carrying capacity of beam-slab bridges is of vital interest to researchers, bridge design engineers, maintenance engineers, public servants, and the general public. Often laboratory tests or full scale bridge tests fail to verify or substantiate the load carrying capacity predicted by conventional engineering analysis.

The purpose of this research was to identify and investigate variables having a major influence on bridge load carrying capacity which would not normally be considered in conventional analysis techniques.

The ability to properly distribute applied loads to the primary structural members is of fundamental importance. Current design specifications and analysis techniques result in overly conservative estimates of load distribution to individual components. A Guyon-Massonet method has been developed to allow a practicing bridge engineer to more accurately distribute applied loads to the individual components of a beam-slab bridge system and thusly improve the quality of structural analysis to develop better correlation with test results.

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LIST OF SYMBOLS

NOTATION	DESCRIPTION
$2b$	width of equivalent bridge
b_1	spacing of longitudinal girders
B_G	flexural stiffness of each longitudinal girder = EI_G
B_D	flexural stiffness of each transverse diaphragm - EI_G
C_G	torsional stiffness of each longitudinal girder = GJ_G
C_D	torsional stiffness of each transverse diaphragm = GJ_D
e	eccentricity of line load relative to longitudinal centerline of bridge
E_G	modulus of elasticity of girder
E_D	modulus of elasticity of deck
f_c	compressive strength of concrete
f_y	yield strength of reinforcing steel
J_G	polar moment of inertia of girder
J_D	polar moment of inertia of decks

NOTATION

DESCRIPTION

K	distribution coefficient
kP	fraction of applied load porportioned to each girder
L	span of bridge
L ₁	spacing of diaphragms
P	intensity of load
t	thickness
w(x,y)	deflection at point (x,y)
W	midspan deflection
α	torsion parameter
	$= \left(\frac{Y_G + Y_D}{2 \sqrt{\rho_G \rho_D}} \right)$
θ	flexural stiffness parameter
	$= \frac{b}{L}^4 \sqrt{\rho_G / \rho_D}$
ρ _G	flexural stiffness per unit width of equivalent bridge = B _D /b ₁
ρ _D	fluexural stiffness per unit length of equivalent bridge = B _D /L ₁
Y _G	torsional stiffness per unit width of equivalent bridge = C _G /b ₁

NOTATION

DESCRIPTION

Y_D

torisional stiffness per unit length of
equivalent bridge = C_D/L_1

CHAPTER I

INTRODUCTION AND RESEARCH APPROACH

1.1 Background

Approximately seventeen years ago a program of periodic inspections of bridges in the United States was begun. This program has evolved through the years to include within its scope most of the bridges currently in service in this country. An integral part of the bridge inspection program, as it exists today, is an evaluation guide developed by the Federal Highway Administration (FHWA). Based on an inspection of a bridge, that bridge is rated according to FHWA guidelines and assigned some rating that identifies, within the limits of accuracy of the rating system, the condition of the bridge.

One very important factor in the evaluation process is the load carrying capacity of a bridge. An accurate assessment of this capacity is both critical and difficult--critical because most bridge engineers look at this quantity as the single most important consideration in determining whether a bridge should be left in service or not, and difficult because, frequently, information relating to some of the important variables which

affect the capacity of a bridge is not generally available to the people charged with making this assessment.

If the results of a typical inspection and evaluation process lead to the conclusion that a structure is deficient and that replacement of the structure may be appropriate, further study of the condition of the bridge and a more detailed consideration of actual load carrying capacity are generally justified. Replacement of any bridge is expensive, and if the bridge is on a major traffic artery, the expense is magnified by the inconvenience involved in the replacement. Further inspection and analysis--even expensively extensive inspection and analysis--are frequently justified.

A problem inherent in making the "further analysis" which might very well result in keeping a bridge in service is the lack of adequate information needed to define a realistic structural model to calculate load capacity. However, considerable testing of bridge components and some testing of full-scale bridges have been done. These tests have led to numerous published reports which describe bridge behavior in the elastic range and to a few reports which describe this behavior from zero load to failure. The objective of this research is to assemble, review, and summarize available data; to identify and analyze variables which are not normally considered in conventional analysis techniques; and to develop an improved methodology for calculating bridge load capacity based on a more

accurate lateral load distribution. To accomplish these objectives the research effort has been divided into tasks. The tasks are: (1) to assemble and review relevant test data and any other information available relating to load capacity for a number of different bridge types; (2) to identify those aspects of behavior which can produce significant variations in load capacity compared to that calculated by current methods; and (3) to develop a series of influence coefficients which will more accurately describe the actual distribution of loads than the current AASHTO provisions might dictate.

1.2 Definition of Bridge Load Capacity

The term "load capacity" does not have a universally accepted unique meaning. To the writer of this dissertation the term "bridge load capacity" means simply "the largest load a bridge can support." Another way to phrase this particular definition is to say that the "ultimate load carrying capacity" of a bridge is that load beyond which an increase in bridge deflection results in a decrease in load. Such a concept of capacity is consistent with current practice in the design of both steel and concrete buildings under American Institute of Steel Construction (AISC) or American Concrete Institute (ACI) specifications.

When this research was begun, the writer had in mind the definition of "bridge load capacity" just presented. Talks with DOT bridge engineers

from Tennessee, Texas, Florida, Pennsylvania, and Kentucky raised questions about whether such a definition was indeed the most appropriate one or whether more attention should be focused on some point on the load-deflection curve for a bridge which might represent the load at which the bridge begins to behave inelastically. Further talks between the writer, doctoral committee members, and various bridge engineers and researchers confirmed two things: (1) the writer should indeed give considerable attention to a definition of "bridge load capacity" that considers the limit of elastic behavior; and (2) apparently, there does not exist a single, universally accepted, unambiguous definition of the term "bridge load capacity."

For a well-designed bridge, the design load is well below the limit of elastic behavior and is not the subject of interest in this research effort. However, two points are of interest. The limit of elastic behavior could be taken as the limit of useful load-carrying capacity and, as such, will be considered the "load capacity" of interest in this research. Ultimate capacity, as a measure of the safety factor inherent in bridge design, is also of interest. Thus, this research effort has been concerned with both the limit of elastic behavior and the ultimate capacity, with particular emphasis on the former. In this dissertation the review of literature and the summary of information obtained were written with both of these load limits in mind.

CHAPTER II

HISTORICAL REVIEW

2.1 Introduction

One of the major tasks in this research project is to assemble and review pertinent literature relating to bridge load capacity. Special emphasis has been given to literature reporting a comparison between test data and an analytical approach. Certainly the most important phase of this research is to evaluate the assumptions used in analytical studies and to determine the significance of the contribution of a number of different variables in predicting actual bridge behavior.

2.2 Background

The literature survey indicates that limited bridge testing was being performed in England as early as the 1920's (44). This was done, primarily, to determine the ability of existing bridges (some dating to medieval times) to carry military traffic. Modern bridge testing, in this country, had its beginnings at the University of Illinois with the classic work of Newmark,

Seiss, et. al. and their "Studies of Slab and Beam Highway Bridges." (112) These works are reported in the late 1940's.

In 1949 Guyon (66) presented his classic analysis solving the differential equation describing plate behavior assuming that the torsional stiffness of the plate-girder system is negligible. In 1950 Massonnet (103) extended Guyon's work to develop a series of deflection coefficients which included the effects of torsional stiffness in a plate girder system.

Failure of the Point Pleasant Bridge in 1967 spawned a series of bridge tests in the United States which were conducted between 1967 and 1975. It is these tests which provide the major source for data for researchers to evaluate the behavior of a full scale bridge when subjected to service, overload, ultimate, and fatigue loading conditions.

The impoundment of the Tims Ford Reservoir in 1972 provided the University of Tennessee Department of Civil Engineering with the unique opportunity to test four existing highway bridges to failure (29). Each of these bridges was a different type of construction and thus offered the possibility of evaluating a broad spectrum of beam - slab bridge types. This test program, conducted at the University of Tennessee, may well be the most important series of full scale bridge tests ever conducted. The broad spectrum of beam-slab bridges, the quality of data obtained, and the ability to predict analytically the test results make this program, without question, the most consistently referenced tests in the literature.

Other notable researchers have made significant contributions in full scale bridge testing. Specifically the work of Bahkt and Csagoly at the Ontario Ministry of Transportation, Beal at the New York Department of Transportation, Buckle in New Zealand, Gosbell in Australia, and Sanders & Elleby at Iowa State University have contributed significantly to the state of the art in bridge testing and analysis of load capacity.

CHAPTER III

VARIABLES INFLUENCING BRIDGE BEHAVIOR

3.1 Introduction

Many administrators and engineers seem to be laboring under the misconception that most highway bridges have capacities far in excess of the capacities predicted by conventional analysis procedures. There is the thought that certain variables, which may have significant influence on load capacity, may be typically neglected or often times ignored. It is with this apparent misconception in mind that this research has undertaken to investigate and evaluate the parameters affecting bridge load capacity.

It is the commonly accepted procedure in the U.S. to evaluate existing bridges in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (2). While this document is an excellent reference for design, its use for evaluation purposes may result in load capacity predictions which are too conservative.

Recently the Ontario Bridge Design Code (113) was revised (1983). It is interesting to note that the 1983 Ontario Code contains a section which is specific to the evaluation of existing structures. It is also interesting to note

that methods for distributing the loads to longitudinal girders have been significantly modified from the distribution procedures identified in the 1983 AASHTO specification.

An in-depth review of literature reporting the results of many bridge tests reveals several variables which influence load capacity. The variables affecting bridge behavior, which are normally neglected in conventional analysis, are outlined below.

1. Unintended Composite Action
2. Contribution of Parapets and Railings
3. Differences Between Actual and Assumed Material Properties
- 4. Participation of Bracing and Secondary Members
5. Differing Support Characteristics and Unintended Continuity
- 6. Refined Estimates of Lateral Load Distribution to Longitudinal Members
- 7. Effects of Deterioration
8. Effects of Strain Hardening in Reinforced Concrete Structures.

It is these variables which will be evaluated in detail to determine their contribution to bridge behavior. The investigation will also include recommendations as to which of these variables can be routinely included in conventional analysis techniques to improve bridge load capacity estimates.

It is generally accepted that the simplest analytical model that would adequately predict the structural behavior provides the most desirable approach to structural analysis of a bridge system. Since most of the factors listed above add to the complexity of the structural analysis, it is not surprising that often times these factors are initially ignored in attempts to predict structural behavior. However, a comparison of predicted versus actual behavior often leads the analyst to consider additional factors which may affect stiffness, ultimate strength, or yield strength of the bridge system. The purpose of the following discussion is to evaluate each parameter, determine its effect on behavior, and determine and discuss the feasibility of considering the parameter for analysis purposes.

3.2 Unintended Composite Action

In beam-slab bridge systems, the behavior of the deck girder/interface often is directly related to the overall performance characteristics of the bridge. Test results indicate that composite action between the bridge deck and the girder system often takes place without having been accounted for in the design. However, in order to assure composite behavior, the bridge girders usually contain shear connectors (studs, channels, etc.) so that the deck and girder system behave as a single unit.

It has been noted in several tests (48) that the bond between the deck slab and the girder is sufficient to assure composite behavior for at least a

portion of the loading range. This "unintended composite action" results in a bridge system behavior that is much stiffer than that predicted for a noncomposite deck-girder system.

Bridge 4 in the Tennessee bridge tests (29) is an excellent example of unintended composite action. The bridge was originally designed as a noncomposite structure. Since the actual behavior was much stiffer than expected, the test results were reanalyzed assuming composite behavior. Unusually close correlation between the actual and calculated values resulted from the composite analytical mode.

It is important to note that at or near the load indicating first yield the "unintended composite" behavior was lost and the bridge system behaved as a noncomposite analytical model would predict. Several researchers (14, 29, 48, 15) have reported that a natural bond can be expected to exist between concrete deck and steel girders. It has also been shown that the shear strength of this bond can be between 400 psi and 500 psi. Certainly allowable shear stresses of this magnitude would induce composite behavior in any deck-girder system.

The reliability of a natural bond (i.e., without shear connectors) is questionable. It has been reported that mechanical connectors should be used if the bond stresses exceed 25 psi. It has also been shown that the bond strength is sensitive to repeated loads, thermal stresses, and impact. Some bridge evaluators indicate that they assume composite behavior for deck-

girder systems when analyzing for inventory loads (working stress analysis). However, at this time the evidence in favor of assuming composite behavior in bridges not designed for composite action is both random and erratic. There are not enough data to support this assumption at the working stress level and certainly not at an ultimate strength level. Unquestionably, unintended composite action is a factor in the behavior of some bridge systems. However, it should be treated as a random variable of unknown probability of occurrence. The benefits of its contribution to bridge capacity should be accepted as a bonus, and the lack of its contribution should not be a deterrent to the expected bridge behavior.

3.3 Parapets and Railings

Parapets and rails are normally neglected in conventional analysis techniques to determine moment capacities of bridge structures. Most analysts apparently have the impression that a parapet or handrail will add a small amount of moment capacity to the analysis; however, neglecting its contribution offers the two-fold advantages of: (1) not having to deal with the additional complexities of geometry; and (2) adding a small amount of conservatism to the predicted capacity. As to whether or not the neglect of a parapet or handrail is conservative is the subject of the following discussion.

3.3.1 Effects on Working Stress Analysis Techniques

The inclusion of parapets or handrails in an elastic analysis will significantly affect the stiffness of the component nearest the parapet or handrail. The increased stiffness will therefore attract more load and, in turn, have a tendency to reduce the load on other portions of the structure. Commonly accepted methods for distributing the load, such as American Association of State Highway and Transportation Officials (AASHTO) criteria, would tend to neglect the effects of the added stiffness; therefore, including its effect in the analysis would have little effect. Modeling the bridge system as a series of discrete elements (finite element or grid analysis) is the most effective way to account for the parapets and handrails in an elastic analysis. The benefit of this analysis is that a more accurate description of the actual load distribution on the bridge structure is obtained. At this point it is difficult to ascertain whether including the parapets and handrails is conservative or unconservative since it greatly depends on the location of the applied loads.

3.3.2 Effects on Load Factor Analysis Techniques

A detailed investigation on the effects of including parapets and handrails in an ultimate strength analysis of Tennessee Bridge #3 has been reported in (29). Interestingly, the analysis is extremely sensitive to failure criteria. If one assumes that failure is defined as some value of

strain in the concrete (AASHTO uses .003) and strain hardening of the reinforcing steel is considered, then certain geometric configurations which include parapets and handrails may actually predict lower failure loads than neglecting the parapets and handrails would predict. The lower failure loads are a result of the remote distance of the extreme fibers of the parapets or handrails from the neutral axis of the bridge cross section and the resulting lower strains in the reinforcing steel. Therefore, crushing of the concrete in the handrail would occur, but the capacity of the bridge would increase after that.

3.4 Differences Between Actual and Assumed Material Properties

Values of material properties that are assumed for design purposes are almost always conservative when used for predicting actual capacities of bridges. The commonly accepted practice of using 28 day predicted strengths of concrete may be erroneous and conservative as much as a factor of 2 to 2.5. Table I is a comparison of design values and actual test values for various material properties taken from several research projects. It is not surprising that the design values are significantly less than those normally encountered in the field since, statistically, design properties require that a high confidence level be achieved to assure that actual values exceed the design value.

TABLE I
DIFFERENCES BETWEEN ACTUAL AND
ASSUMED MATERIAL PROPERTIES

Bibliography Reference No.	Material	Specified Strength	Measured Strength
29	Concrete (f'c) (ksi)	6.0	6.87
29		2.5	6.5
50		6.0	7.02
55		5.4	6.5
59		2.5	7.2
60		5.0	7.0
82		3.0	6.89
92		3.5	5.75
109		5.75	6.90
150		3.0	4.97
164		2.5	7.8
29	Reinforcing Steel fy (ksi)	36	40
29		36	40
59		30	45
82		40	53
109		40	50
164		30	45
5	Prestressing Steel	fy = 246 ksi	fy = 277 ksi
55		fy = 160 ksi	fy = 210 ksi
109		fy = 270 ksi	fy = 284 ksi

In order to predict accurately the behavior of a specific structure, the analyst must know, as precisely as possible, the actual material properties associated with the structure. In order to predict analytically the response of a particular structure for a specific test condition, the mean or average value of the actual material properties provides reasonable results. In order to analyze the structure for a particular loading condition and to predict with some confidence that the structure would be safe to withstand the applied loads, the mean value less some multiple of the standard deviation for the actual properties should be used. It is not the intent of this research to establish factored resistance criteria; however, it is important in comparison of measured and computed response made as a part of this research that actual values of material properties be used to predict structural responses.

3.5 Participation of Bracing and Secondary Members

Bracing and secondary members create redundancies within a bridge structure that enable adjacent primary members to share in load carrying capability. This ability to share or redistribute load under certain conditions is essential for current bridge design practice to be satisfactory. The lack of adequate bracing and secondary members has sometimes resulted in catastrophic failure.

The full-scale bridge tests performed by Burdette and Goodpasture in 1970 used the total bridge cross section as an analytical model. It was assumed that the bridge acted ^{as} a "wide beam" and that every component was capable of carrying its full share of the applied load. This in fact proved to be a very realistic model for the University of Tennessee tests. The wide beam analytical model has been used to predict the results of both the Australian and New Zealand full-scale bridge tests with mixed results. One can then conclude that each of these bridges has either bracing or secondary members (the bridge deck can be considered a secondary member) with the inherent ability to transfer load from one girder to another.

Heins (68, 69) did some analytical research using computer models to simulate and predict the effects of various bracing conditions and their ability to transfer load between cracked girders. Heins' conclusion that bracing creates a redundancy which permits redistribution of loads appears valid and appears to have been proven with various actual tests (29, 60, 56, 163).

3.6 Differing Support Characteristics and Unintended Continuity

The accuracy of the bridge engineer's structural predictions depends on the quality of the analytical model chosen for a particular analysis. Stiffness characteristics, redundancies, and abilities to redistribute loads in

analytical models have been subjects of previous discussions. However, boundary conditions and restraints on the analytical model may be the primary source of error in accurately predicting deflections, load causing first permanent set, or final collapse load. It is well known that assuming a structure to be simply supported when in fact there is considerable resistance to rotation will result in predicting excessive deflections, a conservative prediction of the yield load, and a conservative prediction of the collapse load.

One of the primary justifications for full-scale bridge testing is that, although the in situ material properties can be determined with reasonable accuracy and modern analysis techniques allow for the inclusion of redundancies, the actual support or restraint characteristics of the structure cannot be accurately determined. Bakht and Csagoly's testing of the Perley Bridge (14, 15) was done because of the inability to accurately predict the resistance to rotation or the fixity of the bridge's connections. It was suspected, and later proven correct, that the bridge would behave much stiffer than the analysis predicted.

Several factors can effect a change in the original support characteristics of a bridge. Roller bearings may become frozen by rust with time. Bearing plates may become corroded and joints between adjacent members may become filled with dirt and debris. It is not all together uncommon for some joints between "simply supported" members to be

grouted during the construction process. Each of these factors causes the support to deviate from a "pure" simple support in that some resistance to rotation is inherent in the support or joint.

In the full-scale bridge tests at Tennessee, Bridge 3 is an excellent example of unintended continuity at the support. It is hypothesized that restraints at the supports resulting from friction of the beams on bearing plates, or thermal expansion of adjacent beams, or frozen roller bearings provided resistance to rotation not accounted for in the simply supported analytical model.

The fact that bolted connections provided resistance to rotation on the Perley Bridge and that differing support conditions provided restraint in the Tennessee bridge tests has been documented in the test programs. The question as to whether the analyst can include these anomalies in a strength evaluation is far more difficult to answer. It is difficult to evaluate the effects of slippage in mechanical connections or the amount of rotation due to the hole tolerances. Differing opinions exist concerning whether or not it is possible to account for connection stiffness; however, the apparent consensus is that the Perley Bridge analysis was sufficient and could not be significantly improved without validation through testing.

The uncertainties in the restraints or supports in the Tennessee tests could not be counted on in a typical evaluation analysis. It has been concluded that a great deal of the restraint resulted from the close

proximity of the beams as a result of thermal expansion. Had the test been performed in January rather than July, significant differences in the restraint characteristics of the beams might have occurred.

In conclusion, it can be expected that support characteristics may differ from time to time within a structure. There are times when a structure has unintended continuity which affects the behavior. However, the predictability of changes in restraints is uncertain and cannot be analytically determined without full-scale testing.

3.7 Load Distribution

Of the variables being considered in this investigation, lateral load distribution is probably the most important. The ability of a structure to distribute a particular load to its individual components is an important factor in the behavior of a bridge under large loads. It is this ability to distribute loads efficiently, particularly as the loads are increased above the elastic limit, that has probably contributed to the apparently widely held perception that large excess capacity exists in a typical bridge.

Bridges are routinely designed as components using loading conditions which may be unrealistic when compared to actual loads encountered in practice. These loading criteria used in design reflect previous monumental research efforts to achieve simplified approaches to permit a bridge designer to evaluate and properly size a particular

component. However, when a bridge is loaded with a particular set of loads, it acts as an entity, not as a series of individual components. Under a typical bridge test loading, not every component will be loaded with a portion of the total load consistent with the factors used in the design. Typically, there is some redistribution of load laterally as the test load is increased, particularly if the test load is increased beyond the elastic limit. Thus, any load capacity calculated on the basis of load distribution used in design will almost certainly be lower than that measured in a test.

Chapter 4 deals specifically with the ability of the researcher or bridge engineer to predict more accurately the performance of a structure by using refined load distribution techniques. Predicted behavior is compared with behavior observed in full-scale tests.

3.8 Effects of Deterioration

Deterioration is one of the most subjective criteria included in the FHWA biannual bridge evaluation. The long-term effects on the structure as a result of corrosion of structural steel members or deterioration of concrete bridge decks or corrosion of reinforcing steels have been widely studied with mixed results.

Some quantitative data concerning the amount of deterioration can be obtained by measuring reductions in web and flange thickness in structural steel components. However, deterioration in concrete structures remains

an extremely subjective area. Considerable research has been performed to study the effects of deterioration. Results of the research are varied; however, generally even a severely weathered or deteriorated structure has performed satisfactorily when subjected to overload conditions.

While deterioration is obviously not an area where enhancement to capacity can be expected, it is, however, an area that should be considered and approached with extreme caution. Since deterioration is such a subjective FHWA evaluation item, it is proposed that full-scale overload testing be a method of evaluating severe cases where deterioration is a major factor resulting in reducing the load rating of a bridge.

3.9 Strain Hardening

Commonly accepted analysis techniques to determine the ultimate moment capacity of a reinforced concrete cross section employ the following basic assumptions to develop the equations presented in both the AASHTO and ACI codes.

1. Strain compatibility: i.e., plane sections remain plane throughout the cross section's moment resisting range (or the ultimate strain in the concrete is directly proportional to the ultimate strain in the reinforcing steel).

2. Equilibrium of forces: i.e., for pure bending the tensile forces in the reinforcing steel are statically balanced by the compressive forces in the concrete.

The codes further simplify the basic assumptions of strain compatibility by assuming perfectly elasto-plastic behavior of the reinforcing steel. Actually, commonly used reinforcing steels such as ASTM A615 Grade 60 do not exhibit perfectly elasto-plastic behavior and in fact strain harden during the latter stages of their stress-strain relationship. "The increase in strength due to the effect of strain hardening of the reinforcement is neglected for strength computations" according to the ACI commentary, and the tensile force (T) developed by the reinforcement can be computed as:

$$T = A_s f_y$$

Where A_s : area of tensile reinforcement

f_y : yield strength of the reinforcing steel.

The prediction of the ultimate moment capacity of a concrete cross section reinforced by conventionally accepted practices will result in a conservative prediction which will be less than the actual moment capacity in the presence of strain hardening of the steel reinforcement. Since the compressive force in the concrete is equal to the force in the reinforcing steel, the couple forms the resisting moment (M_n) where:

$$M_n = T(jd)$$

jd is the moment arm of the internal couple and can be expressed as follows for rectangular beams:

$$jd = (d - a/2) \quad \text{where } a \text{ is the depth of the compressive concrete block}$$

Thus, the resisting moment (M_n) can be expressed as:

$$M_n = A_s f_y (d - a/2) \quad (\text{AASHTO code equation 8-16})$$

If the reinforcing steel exhibits strain hardening characteristics, the assumption that the tensile forces are equal to the area of steel times the yield stress is not valid. Examination of the stress-strain relationship for a strain hardening steel would indicate that a substantially higher stress level could be achieved in the reinforcing steel prior to reaching a compressive strain of 0.003 in the concrete stress block. It is worthwhile to examine the effects of strain hardening to determine its worthiness of consideration in predicting moment capacities using load factor designs.

In summary it is clear that, for most reinforced concrete girder bridges, strain hardening adds significantly to the load carrying capacity. Thus, a capacity calculated on the basis of steel yield will be lower than the actual ultimate capacity. However, strain hardening does not contribute additional resistance to that generally calculated for the elastic limit or the point beyond which some permanent deflection will occur. Also, to consider strain hardening requires that the actual stress-strain curve for the steel be

known. Thus, it must be concluded that consideration of strain hardening is justified only in unusual cases and that, in those cases, it only affects the calculation of ultimate load capacity.

CHAPTER IV

LATERAL LOAD DISTRIBUTION

4.1 Background

Lateral distribution of applied loads to the longitudinal supporting elements of bridge structures has commanded a major portion of researchers' attention for a considerable period of time. The work of Newmark and Seiss at the University of Illinois (110) represents some of the earliest theoretical work performed on lateral load distribution. It has been previously noted that in 1946 Guyon (66) presented his classic analysis solving the differential equation describing behavior of a stiffened plate assuming that the torsional stiffness of the plate-girder system is negligible. In 1950, Massonnet (103) extended Guyon's work to develop a series of deflection coefficients which included the effects of torsional stiffness in a deck-girder system. In 1961 Mattock and Kaar (104) extracted portions of the Guyon-Massonnet theory and described the behavior of a prestressed model bridge obtaining excellent correlation with test results. Again in 1970 Sanders and Elleby (126) referenced Guyon and Massonnet extensively in developing what is presently the AASHTO distribution criteria for load distribution of highway bridges.

It is the opinion of this writer, based on a study of available test data, that the current AASHTO criteria of load distribution yields conservative results when used to define the expected load on a beam-slab bridge component or when used to evaluate beam-slab bridges in the bi-annual FHWA bridge evaluation. In an attempt to provide a bridge engineer with a method to better evaluate load distributions for beam-slab bridges, the writer has extended the Guyon-Massonnet theory to develop a series of influence coefficients which will enable the analyst to more accurately evaluate the effects of a load applied laterally at any point on the cross-section of the bridge and longitudinally at the center line of the bridge span.

4.2 Development of Theory

Mattock and Kaar provide an excellent overview of the Guyon-Massonnet theory. The following portion of the theory is applicable to this work.

Consider the case of a bridge having m girders spaced at a distance b_1 apart with a span L and an equivalent width $2b$ as shown in Figure 1. The partial differential equation representing the deformed surface of the bridge can be expressed as:

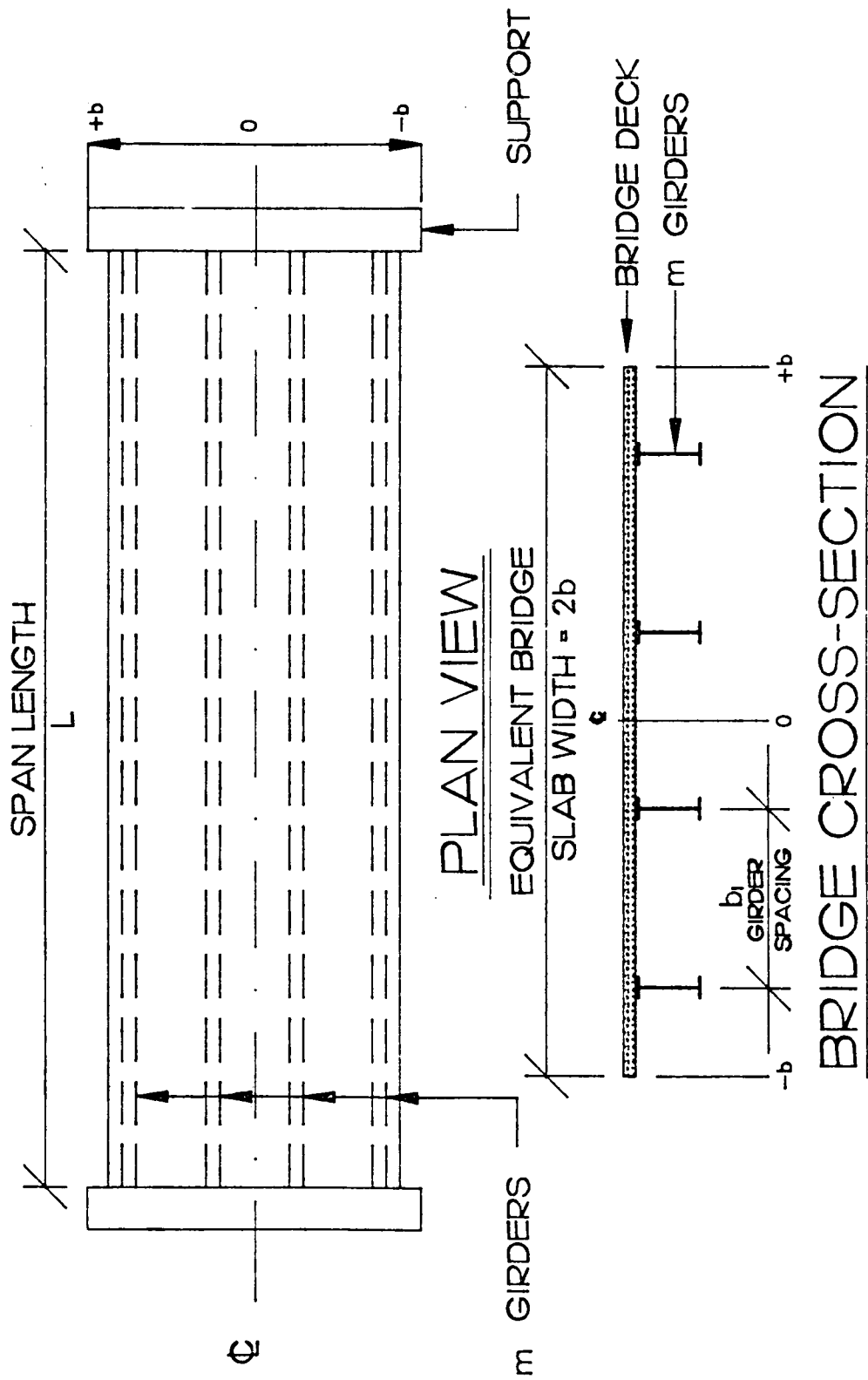


Figure 1. Typical Beam-Slab Bridge.

$$\rho_G \left(\frac{\partial^4 w}{\partial x^4} \right) + (\gamma_G + \gamma_D) \left(\frac{\partial^4 w}{\partial x^2 \partial y^2} \right) + \rho_D \left(\frac{\partial^4 w}{\partial y^4} \right) = p(x, y) \quad (1)$$

Massonnet (103) has shown that in practice the equation can be rewritten as:

$$\rho_G \left(\frac{\partial^4 w}{\partial x^4} \right) + 2\alpha \sqrt{\rho_G \rho_D} \left(\frac{\partial^4 w}{\partial x^2 \partial y^2} \right) + \rho_D \left(\frac{\partial^4 w}{\partial y^4} \right) = p(x, y) \quad (2)$$

where α is a torsional parameter varying between 0 and 1 and expressed by the equation:

$$\alpha = \frac{(\gamma_G + \gamma_D)}{2 \sqrt{\rho_G \rho_D}} \quad (3)$$

Massonnet has further shown that the behavior of a bridge can be completely defined by the torsion parameter α and the flexural stiffness parameter θ where

$$\theta = \frac{b}{L} \sqrt[4]{\frac{\rho_G}{\rho_D}} \quad (4)$$

If the structure is loaded with the sinusoidal line load shown in Figure 2 the deflected shape is of the form

$$w(x, y) = p_1 W(y) \sin \frac{\pi x}{L} \quad (5)$$

Now consider replacing the sinusoidal line load with a load uniformly distributed across the width of the bridge and sinusoidally distributed along the length. The deflected shape is then of the form:

$$wm(x, y) = p_1 Wm \sin \frac{\pi x}{L} \quad (6)$$

Where Wm is the midspan deflection due to the uniform load and Wy is the midspan deflection at y location due to the sinusoidal line load.

Dividing equation (5) by equation (6) yields

$$w(x, y) = wm(x, y) \frac{W(y)}{W(m)} \quad (7)$$

$$\text{Now let } \frac{W(y)}{W(m)} = K(y) \text{ where}$$

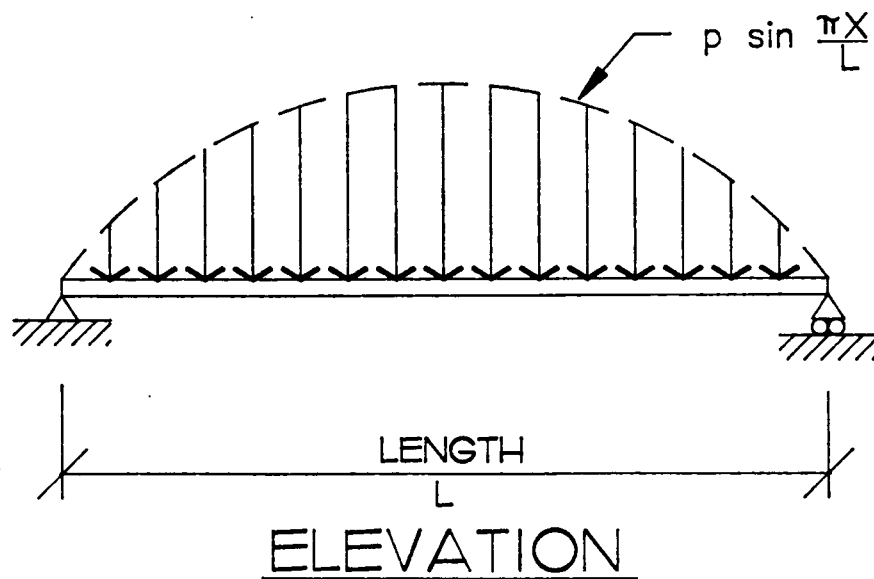
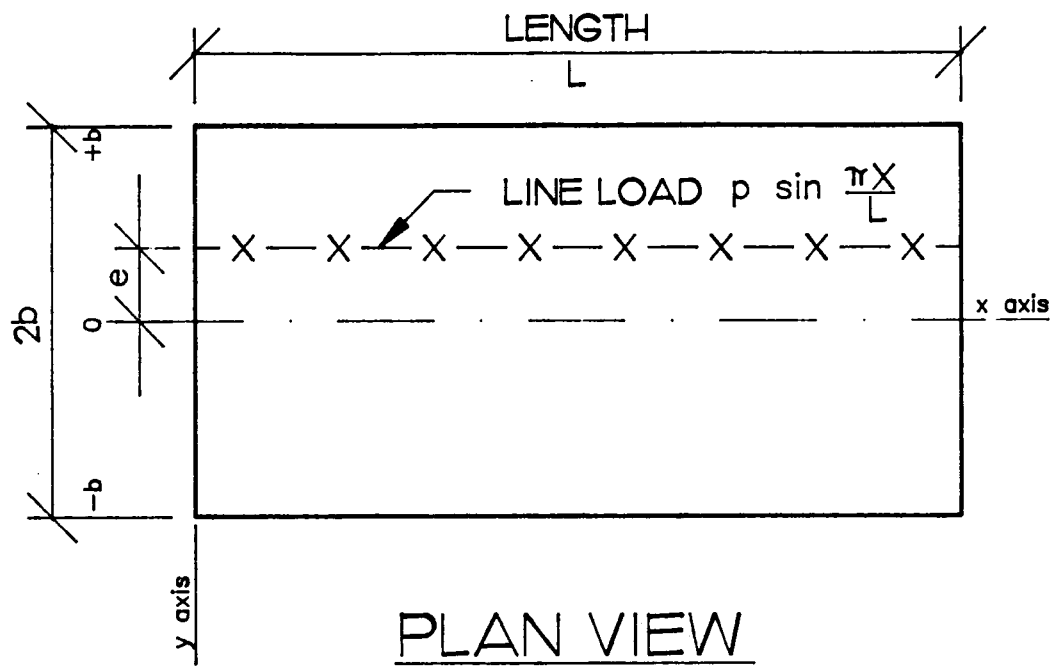


Figure 2. Loading System on the Equivalent Bridge.

$K(y)$ is the distribution coefficient representing the ratio of the vertical deflection of a point on the bridge under the effect of a line load to the deflection of the same point when the same load is distributed over the entire width of the bridge. The value of the distribution coefficient "K" depends on:

1. The value of the flexural stiffness parameter, θ .
2. The value of the torsional parameter, α .
3. The relative eccentricity, e/b , of the line load,
4. The relative ordinate, y/b , of the point on the bridge under consideration.

It has been stated earlier that Guyon (66) solved equation (1) for the case of α equal to zero. Thus he was able to obtain values of K for values of θ equal to 0.669, 1.057, 1.295, and 1.778. Massonnet extended Guyon's work by solving equation (1) for the case of α equal to one and for the same values of θ . Massonnet made additional studies to determine the variation of k if α alone varied and determined that K follows the following equation very closely:

$$K_{\alpha} = K_0 + (K_1 - K_0) \sqrt{\alpha} \quad (8)$$

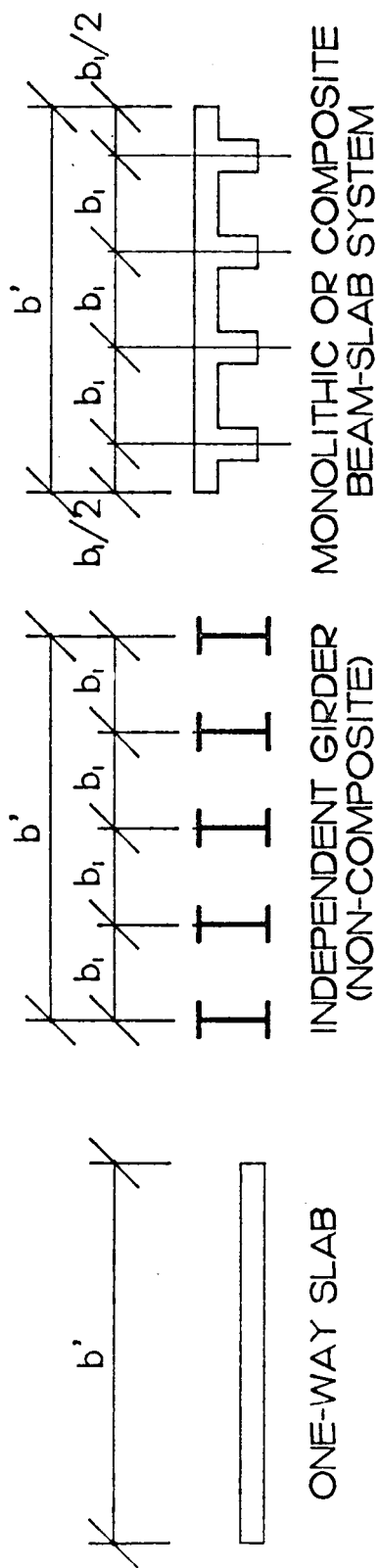
Thus it is possible to interpolate using equation (8) and obtain a distribution coefficient, K , for any given value of α .

4.2.1 Width of Equivalent Bridge

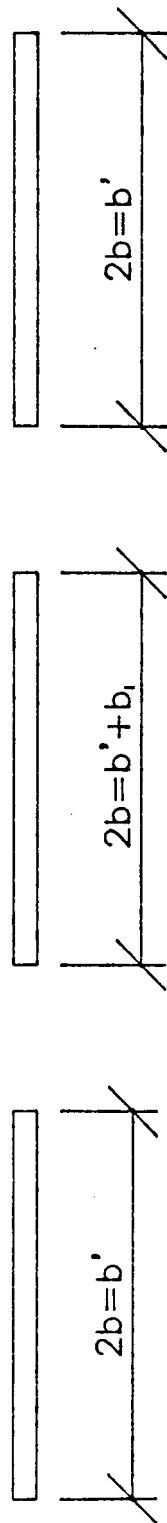
The effective width of the equivalent bridge to which the tabulated values of K refer depends on the type of bridge under consideration. For the case of a one way slab bridge the equivalent width is equal to the actual width. For the case of an independent girder bridge (i.e. the deck slab does not contribute significantly to the longitudinal girder stiffness) the equivalent width is the actual width plus the distance between two adjacent girders. If the bridge is composite or constructed monolithically the equivalent bridge is equal to the distance between the outside girders plus one half the girder spacing cantilevered to either side of the bridge. Figure 3 is a graphic representation of the various equivalent bridges.

4.2.2 Distribution Coefficients

The distribution coefficients developed by Guyon and Massonnet are shown in Tables II and III. It is these coefficients which later in this work are normalized in order to develop influence coefficients to aid the designer in evaluating a particular bridge system.



ACTUAL BRIDGES



EQUIVALENT BRIDGES

Figure 3. Effective Width of Various Bridge Types.

TABLE II
DISTRIBUTION COEFFICIENT K FOR $\alpha = 0.0$

$\theta = \frac{b}{4\sqrt{\rho_G/\rho_D}}$	Position of the Load	Position of the Girder									
		-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3 b/4	b	
0.669	0	+0.120	+0.595	+1.055	+1.442	+1.618	+1.442	+1.442	+1.055	+0.595	+0.120
	b/4	-0.294	+0.149	+0.603	+1.051	+1.442	+1.646	+1.646	+1.527	+1.226	+0.874
	b/2	-0.517	-0.175	+0.195	+0.603	+1.055	+1.527	+1.527	+1.909	+2.064	+2.113
	3b/4	-0.635	-0.427	-0.175	+0.149	+0.595	+1.226	+1.226	+2.064	+3.004	+3.871
	b	-0.692	-0.635	-0.517	-0.294	+0.120	+0.874	+2.113	+3.871	+5.984	
1.057	0	-0.733	-0.109	+0.993	+1.944	+2.403	+1.944	+1.944	+0.993	+0.109	-0.733
	b/4	-0.595	-0.156	+0.339	+1.045	+1.944	+2.492	+2.492	+1.894	+0.798	-0.383
	b/2	-0.301	-0.153	-0.003	+0.339	+0.993	+1.804	+1.804	+2.490	+1.996	+1.049
	3b/4	+0.002	-0.114	-0.153	-0.156	+0.109	+0.798	+0.798	+1.996	+3.427	+4.403
	b	+0.225	+0.002	-0.301	-0.595	-0.733	-0.383	-0.383	+1.049	+4.403	+9.452
1.495	0	-0.467	-0.119	+0.594	+2.073	+3.344	+2.073	+2.073	+0.594	-0.119	-0.467
	b/4	-0.114	-0.157	-0.058	+0.576	+2.073	+3.398	+3.398	+2.132	+0.455	-0.883
	b/2	+0.023	-0.073	-0.152	-0.119	+0.594	+2.132	+2.132	+3.464	+2.043	-0.217
	3b/4	+0.044	+0.001	-0.073	-0.157	-0.119	+0.455	+0.455	+2.043	+3.894	+3.887
	b	+0.051	+0.044	+0.023	-0.114	-0.467	-0.883	-0.883	-0.217	+3.887	+13.827
1.778	0	-0.213	-0.186	+0.293	+2.045	+3.953	+2.045	+2.045	+0.293	-0.186	-0.213
	b/4	+0.038	-0.109	-0.172	+0.281	+2.045	+3.985	+3.985	+2.102	+0.229	-0.804
	b/2	-0.012	-0.029	-0.108	-0.172	+0.293	+2.102	+2.102	+4.100	+2.039	-0.816
	3b/4	+0.032	+0.016	-0.029	-0.109	-0.186	+0.229	+0.229	+2.039	+4.319	+3.218
	b	+0.013	+0.032	-0.012	+0.038	-0.213	-0.804	-0.804	-0.816	+3.218	+15.801

TABLE III
DISTRIBUTION COEFFICIENT K FOR $\alpha = 1.0$

$\theta = \frac{b}{14\sqrt{\rho_G/\rho_D}}$	Position of the Girder	Position of the Load								
		-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0.669	0	+0.733	+0.849	+0.995	+1.157	+1.248	+1.157	+0.995	+0.849	+0.733
	b/4	+0.511	+0.617	+0.759	+0.947	+1.157	+1.300	+1.263	+1.158	+1.058
	b/2	+0.364	+0.455	+0.580	+0.759	+0.995	+1.263	+1.477	+1.525	+1.511
	3b/4	+0.269	+0.346	+0.455	+0.617	+0.849	+1.158	+1.525	+1.875	+2.105
	b	+0.202	+0.269	+0.364	+0.511	+0.733	+1.058	+1.511	+2.104	+2.811
1.057	0	+0.428	+0.612	+0.926	+1.383	+1.710	+1.383	+0.926	+0.612	+0.428
	b/4	+0.219	+0.330	+0.533	+0.887	+1.383	+1.753	+1.480	+1.091	+0.825
	b/2	+0.113	+0.178	+0.301	+0.533	+0.926	+1.480	+1.939	+1.796	+1.541
	3b/4	+0.063	+0.102	+0.179	+0.331	+0.613	+1.092	+1.796	+2.510	+2.733
	b	+0.037	+0.063	+0.113	+0.219	+0.482	+0.825	+1.541	+2.733	+4.429
1.495	0	+0.191	+0.362	+0.775	+1.593	+2.356	+1.589	+0.775	+0.362	+0.191
	b/4	+0.070	+0.140	+0.323	+0.758	+1.589	+2.375	+1.639	+0.879	+0.510
	b/2	+0.025	+0.053	+0.128	+0.323	+0.774	+1.639	+2.497	+1.903	+1.300
	3b/4	+0.010	+0.021	+0.053	+0.140	+0.362	+0.879	+1.903	+3.104	+3.072
	b	+0.005	+0.010	+0.026	+0.070	+0.191	+0.510	+1.301	+3.071	+6.263
1.778	0	+0.106	+0.243	+0.658	+1.660	+2.795	+1.660	+0.658	+0.243	+0.106
	b/4	+0.031	+0.076	+0.223	+0.658	+1.660	+2.804	+1.687	+0.726	+0.349
	b/2	+0.009	+0.023	+0.071	+0.223	+0.658	+1.687	+2.885	+1.895	+1.093
	3b/4	+0.003	+0.007	+0.023	+0.076	+0.243	+0.726	+1.895	+3.453	+3.122
	b	0	+0.003	+0.009	+0.031	+0.106	+0.349	+1.093	+3.122	+7.449

4.3 Influence Coefficients

Mattock and Kaar (104) and Morice and Little (106) give examples of specific attempts to employ the Guyon-Massonnet theory to predict analytically lateral load distributions and to correlate these analyses with laboratory test results. Until this work no attempt had been made to generalize Massonnet's coefficients and develop distribution coefficients which would be applicable to a wide range of actual bridge structures. This work uses the distribution coefficients developed by Massonnet, normalizes the coefficients, and determines the appropriate distribution to any girder for a bridge having three thru nine girders. These influence coefficients are plotted for values of α equal to 0.0 and 1.0 and values of θ equal to 0.669, 1.057, 1.495, and 1.770 for each bridge geometry. Appendix A contains the of these plots.

Figure 4 is a graphic representation of the development of the influence coefficients for a four girder bridge with an applied load at the outer edge of the bridge. The area under the curve is calculated using the trapezoidal rule for numerical integration. The area allocated to a particular girder is then calculated using similar numerical integration techniques so that for a particular load location the influence coefficient can be represented as:

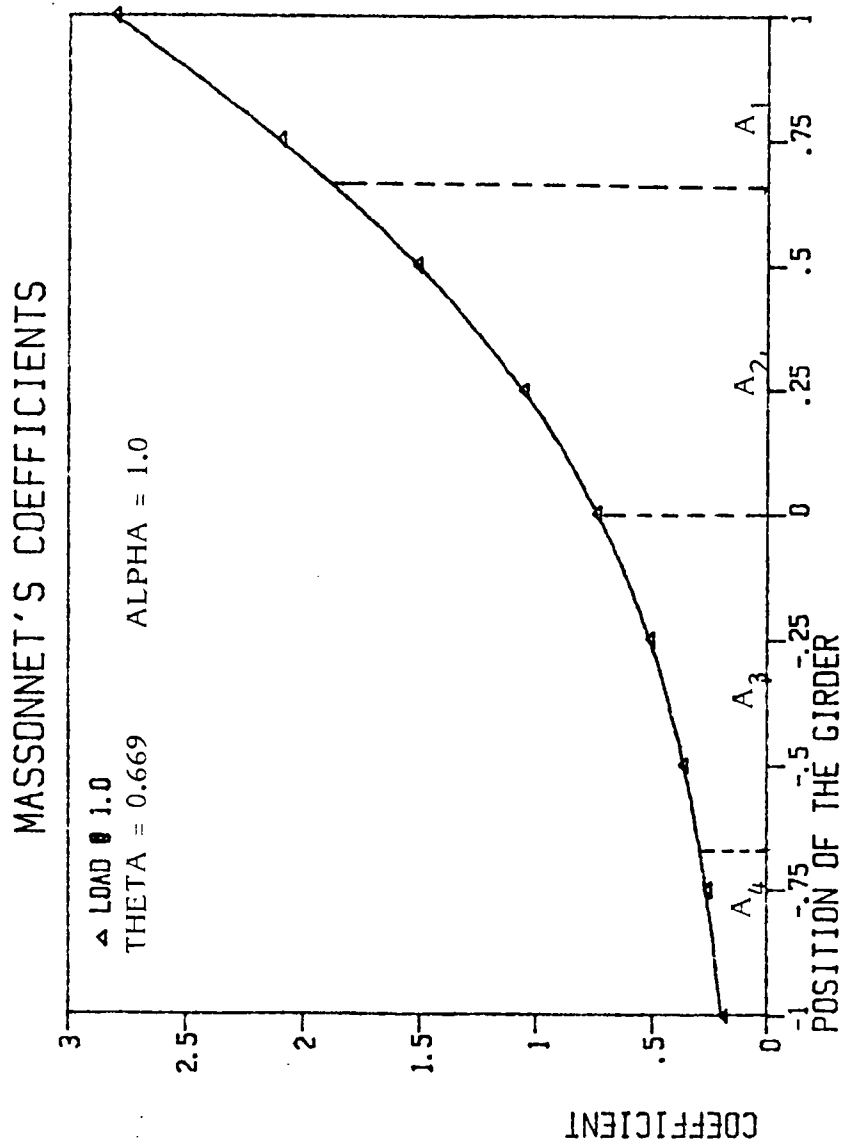


Figure 4. Development of Influence Coefficient for a Four Girder Bridge.

$$\frac{\text{Influence coefficient for particular load location}}{\text{particular load location}} = \frac{\text{Area associated with a particular girder}}{\text{Total area under curve}} \quad (9)$$

This process is then repeated for a different load location until the coefficients are developed for nine points of load location, $-b$, $-3b/4$, $-b/2$, $-b/4$, 0 , $b/4$, $b/2$, $3b/4$, b . The influence coefficients are then plotted for a given value of α and θ and repeated until each girder configuration can be evaluated.

4.4 Case Studies

Recently Ozbek (114) compared the results of various full scale bridge tests to the predicted results using AASHTO distribution criteria. Some liberties were taken in the interpretation of the AASHTO criteria, but basically the intent of the code remains intact. Ozbek's results are based on the theory that an applied load will be either distributed directly to the girder over which it is applied or will only be distributed by beam action to its two adjacent girders.

It is obvious from examination of the influence curves that under normal circumstances the influence of a particular load will be felt by more than one of the two adjacent girders. It can then be concluded that the AASHTO criteria is a conservative prediction of lateral load distribution.

Six bridges are evaluated in the case studies: Tennessee bridges 1, 3, and 4, and the Mangeteweka, Karaka, and Barker bridges in New Zealand.

4.4.1 Description of Bridges

Tennessee Bridge 1 was a continuous four span bridge with four W36x170 rolled steel girders. The thickness of the deck was seven inches and the distance between girders was 8.3 feet. The bridge was designed to act compositely in the positive moment regions, but the uplift of one of the end supports affected the location of positive and negative moments during loading to failure. Composite action was assumed to exist where shear connectors were provided according to the construction drawings.

Tennessee Bridge 3 was a simple span reinforced concrete T-Beam bridge monolithically constructed. The span was 50 feet and girder spacing was six feet and ten inches. The bridge was built in 1938 for H-15 loading and had a skew angle of 60 degrees.

Tennessee Bridge 4 was a 3-span continuous bridge with 27-in. deep steel rolled beams. The center span was 60 feet, and the thickness of the slab was 7-in. The bridge was designed to be non-composite, but acted compositely until yielding of the steel. The girder spacing was 7 feet, 4 inches center-to-center. The bridge was designed for an H-15 loading and built in 1956.

The Magateweka Stream bridge was a reinforced concrete T-Beam bridge with four girders. It was built in 1937 and was typical of many of the short span, reinforced concrete, T-Beam highway bridges built in New Zealand in the 1930's. The curb-to-curb distance was 24 feet and the clear span was 31 feet-6 inches. The deck was 8 inches thick, and four main girders were spaced on 6 feet 8 inches centers. Prior to testing, the bridge was structurally in sound condition.

The Karaka Creek bridge was a reinforced concrete T-beam, 2 lane bridge with a span of 30 feet. It was built in 1937 and had four girders. The curb-to-curb distance was 24 feet. The deck was 8" thick and the girders was spaced 6'-9" centers. The bridge was structurally in sound condition prior to testing, but some cracking was present in both slab and beams. The cracking was thought to be due to shrinkage in the concrete.

Barker's bridge was constructed in 1940. The bridge had 3 spans; the approach spans were formed by cantilevers at either end of the main span with concrete link slabs from the end of the cantilevers to the approach roadway. The cantilever spans were 12 feet long, and the main span was 42 feet. The width of the bridge was 24 feet curb-to-curb. The slab was 7 inches thick, and there were four main beams. Diaphragms were located above the piers and at midspan of the center span. Except for the piers, the bride was in good condition prior to testing.

4.4.2 Lateral Locations of the Applied Load

Figures 5, 6, 7, and 8 are graphic representations of the applied loads on the bridge cross-sections for each case.

Table IV details the bending and torsional parameters used to determine the distributions to the various girders.

Ozbek (114) has calculated the bending capacity at yield for each of the test cases based on the appropriate moment of inertia of the deck-girder component and the measured material properties. The live plus dead load bending moment diagram was then examined and the contribution of the live load, in this case the applied test loads, was assessed. In this manner the theoretical load could be determined which could be compared to the test load causing first yield.

4.5 Results of Analysis

Using the influence coefficients developed in Appendix A for four girder bridges, the applied load shown in Figures 5, 6, 7, and 8 were appropriately distributed to the bridge girders for values of α equal to 0.0 and 1.0. Each girder load was then modified for the appropriate value of α using Massonnet's equation for interpolation (Equation 8). The calculated distribution of the applied loads for each of the test cases is presented in Tables V, VI, VII, VIII, IX, and X.

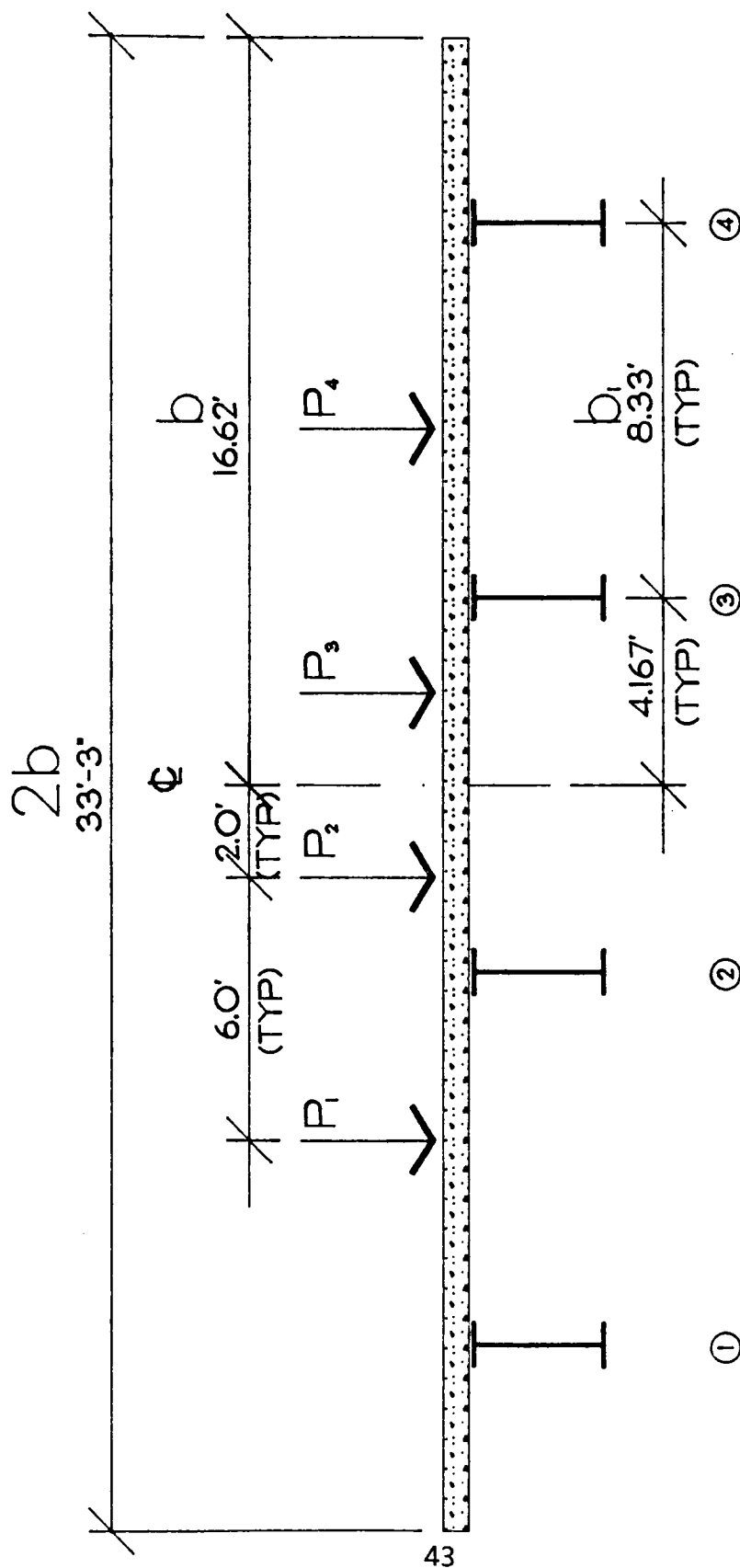


Figure 5. Loading Diagram - Tennessee Bridge 1.

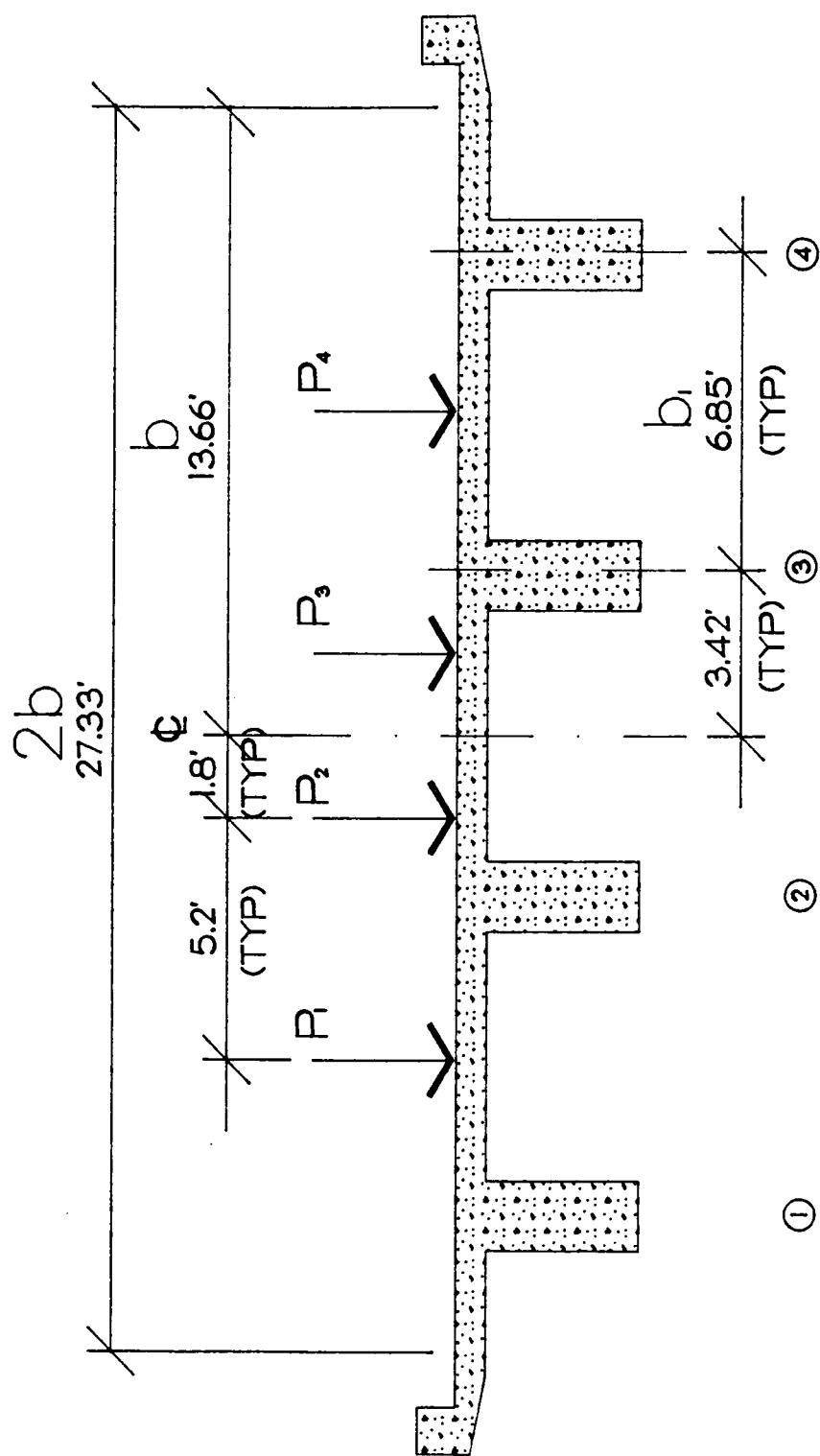


Figure 6. Loading Diagram - Tennessee Bridge 3.

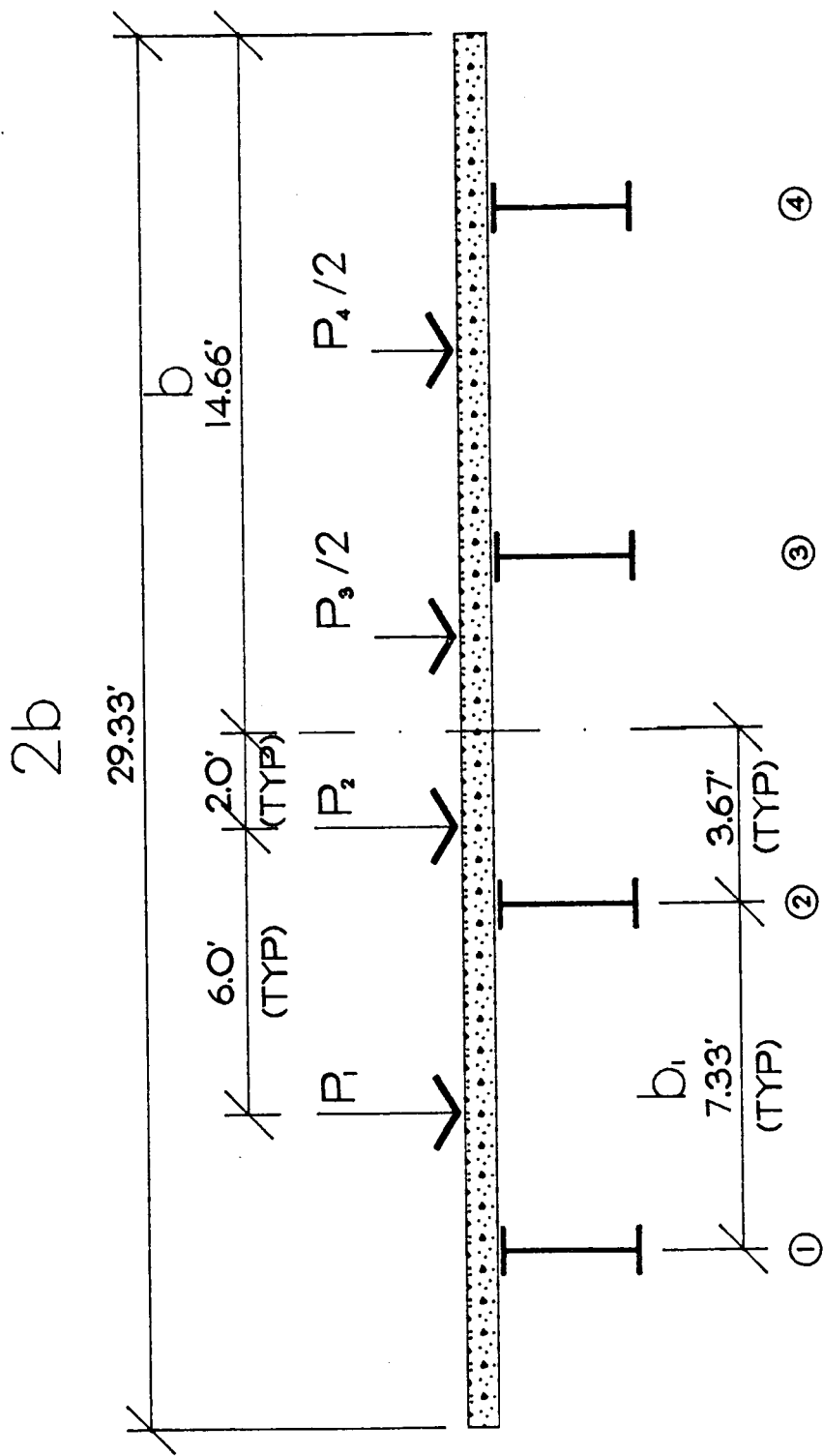


Figure 7. Loading Diagram - Tennessee Bridge 4.

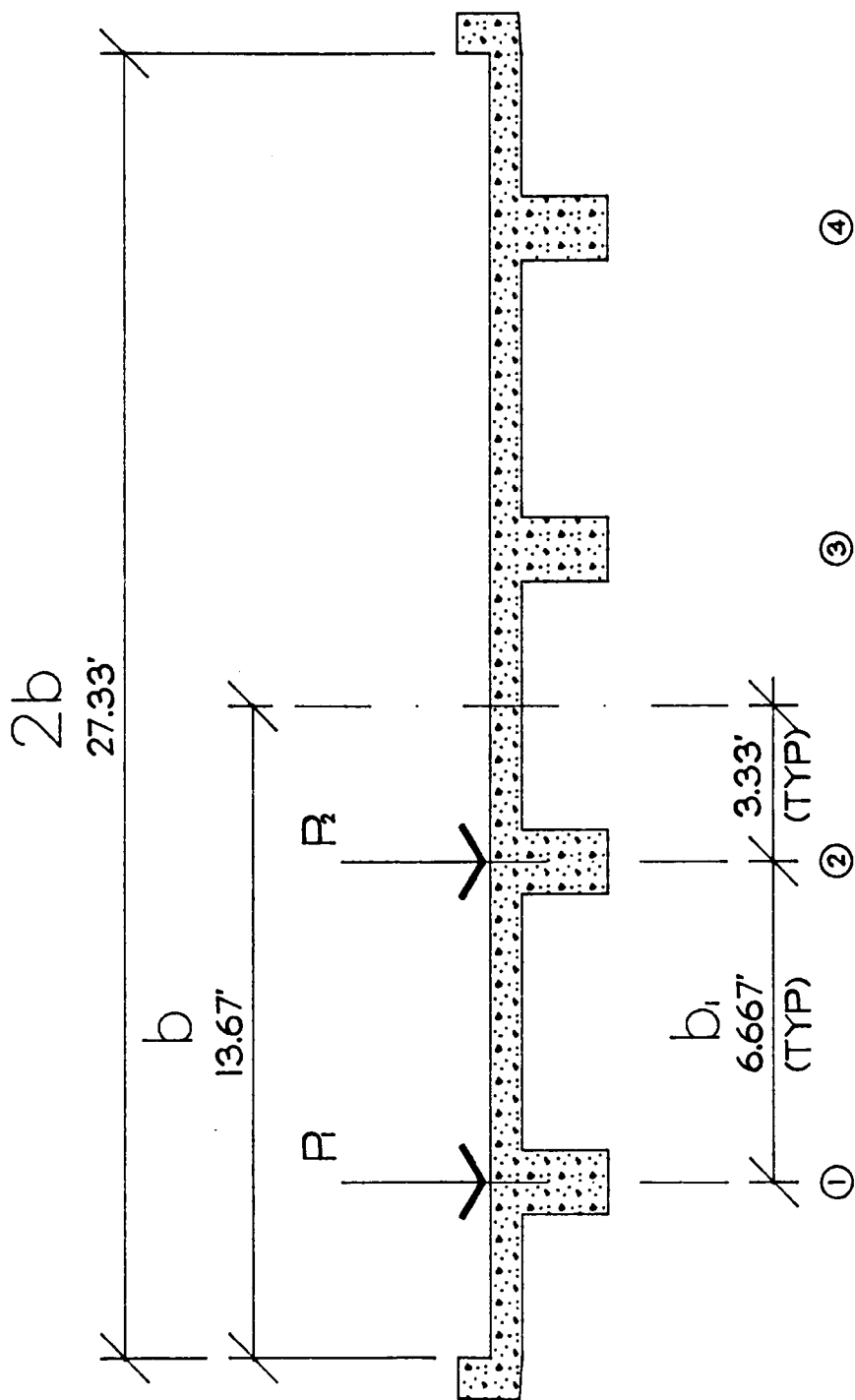


Figure 8. Typical Loading Diagram - New Zealand Bridges.

TABLE IV
BENDING AND TORSION PARAMETERS

Case	θ	α
Tenn #1	0.503	0.126
Tenn#3	0.705	0.232
Tenn #4	0.564	0.213
Mangatewka	0.725	0.279
Barker	0.690	0.392
Karaka	0.715	0.339

TABLE V
TENNESSEE BRIDGE 1-LOAD DISTRIBUTION TO VARIOUS
GIRDERS $\alpha = 0.126$

Girder	kP_1	kP_2	kP_3	kP_4	Σk
1	0.31	0.13	0.07	0.00	0.51
2	0.50	0.45	0.35	0.19	1.49
3	0.19	0.35	0.45	0.50	1.49
4	0.00	0.07	0.13	0.31	0.51
Σ	1.0	1.0	1.0	1.0	4.0

TABLE VI
TENNESSEE BRIDGE 3-LOAD DISTRIBUTION TO VARIOUS
GIRDERS $\alpha = 0.232$

Girder	kP_1	kP_2	kP_3	kP_4	Σk
1	0.32	0.13	0.08	0.02	0.55
2	0.48	0.45	0.34	0.18	1.45
3	0.18	0.34	0.45	0.48	1.45
4	0.02	0.08	0.13	0.32	0.55
Σ	1.0	1.0	1.0	1.0	4.0

TABLE VII
TENNESSEE BRIDGE 4-LOAD DISTRIBUTION TO VARIOUS
GIRDERS $\alpha = 0.213$

Girder	kP_1	kP_2	kP_3	kP_4	Σk
1	0.32	0.14	0.04	0.0	0.5
2	0.49	0.44	0.16	0.10	1.19
3	0.19	0.33	0.22	0.24	0.98
4	0.00	0.09	0.08	0.16	0.33
Σ	1.0	1.0	0.5	0.5	3.0

TABLE VIII
MAGNATEWEKA BRIDGE-LOAD DISTRIBUTION TO VARIOUS
GIRDERS $\alpha = 0.279$

Girder	kP_1	kP_2	Σk
1	0.43	0.18	0.61
2	0.48	0.49	0.97
3	0.11	0.28	0.39
4	0.02	0.05	0.03
Σ	1.0	1.0	2.0

TABLE IX
KARAKA CREEK BRIDGE-LOAD DISTRIBUTION TO VARIOUS
GIRDERS $\alpha = 0.339$

Girder	kP_1	kP_2	Σk
1	0.41	0.18	0.59
2	0.48	0.48	0.96
3	0.12	0.28	0.40
4	0.01	0.06	0.05
Σ	1.0	1.0	2.0

TABLE X
BARKER'S CREEK BRIDGE-LOAD DISTRIBUTION TO VARIOUS
GIRDERS $\alpha = 0.392$

Girder	kP_1	kP_2	Σk
1	0.40	0.18	0.58
2	0.47	0.48	0.95
3	0.13	0.28	0.41
4	0.0	0.06	0.06
Σ	1.0	1.0	2.0

CHAPTER V

CONCLUSIONS

This dissertation's primary objective has been to investigate behavioral aspects of beam-slab bridges which are not normally considered in conventional analysis procedures. It has been previously stated that some individuals concerned with rating bridges appear to be under the impression that bridge capacities far in excess of calculated capacities exist in actual practice. The literature does not support this theory. Analyses which have been performed to determine bridge load capacities in conjunction with test programs have generally yielded results which compared reasonably well with the actual test results.

In fairness to the allegation that conventional analysis procedures are too conservative, the following can be stated. The behavioral aspects included in this dissertation and again listed below should be evaluated in each case in order to predict load capacities consistent with those which actually exist.

1. Unintended composite action
2. Contribution of parapets and railings

3. Differences between actual and assumed material properties
4. Participation of bracing and secondary members
5. Differing support characteristics and unintended continuity
6. Refined estimates of lateral load distribution to longitudinal members
7. Effects of deterioration
8. Effects of strain hardening on reinforced concrete structures.

It should be noted that the neglect of these behavioral aspects individually may not result in a capacity prediction that is unduly conservative. However, the failure to consider any of the itemized aspects, particularly number 6, may lend credence to the idea that actual capacities are two or three times greater than predicted capacities.

In order to refine the analysis required to obtain the best correlation with test results as has been presented in Table XI, every aspect of behavior listed above was evaluated and included in the analysis of appropriate. Table XII evaluates the aspects of behavior included in the analysis to optimize the predicted results.

Inspection of Table XI indicates that contribution of parapets and railings, deterioration, and strain hardening were not factors in the test cases. This does not necessarily diminish their importance. For the test cases selected, the interior girders were the critical component in determining first yield; therefore, parapets and railings were of nominal

TABLE XI

COMPARISON OF YIELD CAPACITIES FOR THE TEST CASES

Bridge	Test Results (kips)	Component Analysis (kips)	Test Results (%)	Wide Beam (kips)	Distrib. Load Analysis	% Test Result (%)
UT Birdge #1 Continuous, Partially Composite Steel Beams	620	526	85%	714	544	88%
UT Bridge #3 Simple Span Composite T-Beams	660	535	81%	759	546	83%
UT Bridge #4 Continuous, Partially Composite Steel Beams	520	462	89%	601	505	97%
Mangateveka Simple Span Composite T-Beams	254	160	63%	409	164	65%
Karaka Creek Simple Span Composite T-Beams	336	279	83%	543	290	86%
Barker's Continuous Composite T-Beams	300	183	61%	379	193	64%

TABLE XII
BEHAVIORAL ASPECTS INCLUDED IN ANALYSIS

Bridge	Unintended Composite Action	Contribution of Parapets & Railings	Actual Material Properties	Participation of Bracing	Support Characteristics	Lateral Load Distribution	Deterioration	Strain Hardening
Tennessee Bridge 1			•	•		•		
Tennessee Bridge 3			•	•	•	•		
Tennessee Bridge 4	•		•	•		•		
New Zealand Mangatewaka			•	•		•		
New Zealand Karaka Creek			•	•	•	•		
New Zealand Barker's			•	•	•	•		

importance. Deterioration was not a factor on any of the bridges analyzed and strain hardening would only apply if analytical predictions were based on ultimate loading conditions .

Refining estimates of lateral load distribution to longitudinal members has been the subject of a massive amount of research. With the amount of work that has been done on this subject, no well defined methodology exists which allows the designer or bridge evaluator to appropriately proportion applied loads to longitudinal members. This work has developed a simplified approach, based on a previously developed theory, which accounts for bending stiffness and torsional stiffness in laterally distributing the loads. The method is not as sophisticated as some discrete element computer analysis might be. It does, however, provide the analyst with a method which results in less conservative lateral distributions than current AASHTO criteria would dictate and at the same time satisfies static equilibrium across the bridge. Table XII compares the test results, Ozbek's AASHTO and wide beam analysis (114), and the Massonnet method thusly demonstrating an improvement of yield load capacity prediction using Massonnet's method.

It can be stated that the distributed load method used in this dissertation provides a definite improvement over existing AASHTO methods for distributing loads to various girders. It should however, be

noted that the AASHTO method provided results that were much closer than expected.

Cast in place concrete beams appear to be more difficult to predict than bridges with steel girders. Variations in concrete beams, which are impossible to inspect or detect, such as the depth of reinforcement make significant differences in the ability to predict yield capacities.

It is important to note that the load distribution method developed appears to provide a lower bound solution thus giving conservative results when predicting yield capacities.

Finally a significant aspect of the analysis procedure is that it provides for the ability to analyze a complex structure without requiring the use of a computer.

CHAPTER VI

RECOMMENDATIONS

Massonnet's (103) original work was completed in 1950, long before the readily available computing tools of today. The original work was completed in closed form solution using a sinusoidal forcing function to provide the particular solution to equation (1). Today's numerical analysis techniques provide the ability to develop "improved" coefficients for a concentrated load forcing function modeled using several terms of a Fourier Series rather than the single term sinusoidal load used by Massonnet. The "improved" coefficients can then be used to develop influence coefficients which will again improve the accuracy of predicting lateral load distribution.

It is also suggested that special attention be paid to the FHWA load capacity evaluation. In particular, the evaluation should include provisions for evaluating aspects of behavior included in this dissertation. Prior to undergoing the expense of derating (posting for limited allowable loading) the evaluator should be required to evaluate and justify the omission of the subject aspects of behavior from his analysis. An example of such a justification would be that the "simplified" analysis indicates

sufficient strength for its intended use. The inclusion of the identified behavioral aspects in conventional evaluation procedures can potentially have enormous economic benefits to the users.

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APPENDICES

APPENDIX A

THREE GIRDER BRIDGE (GIRDER @ 0.0)

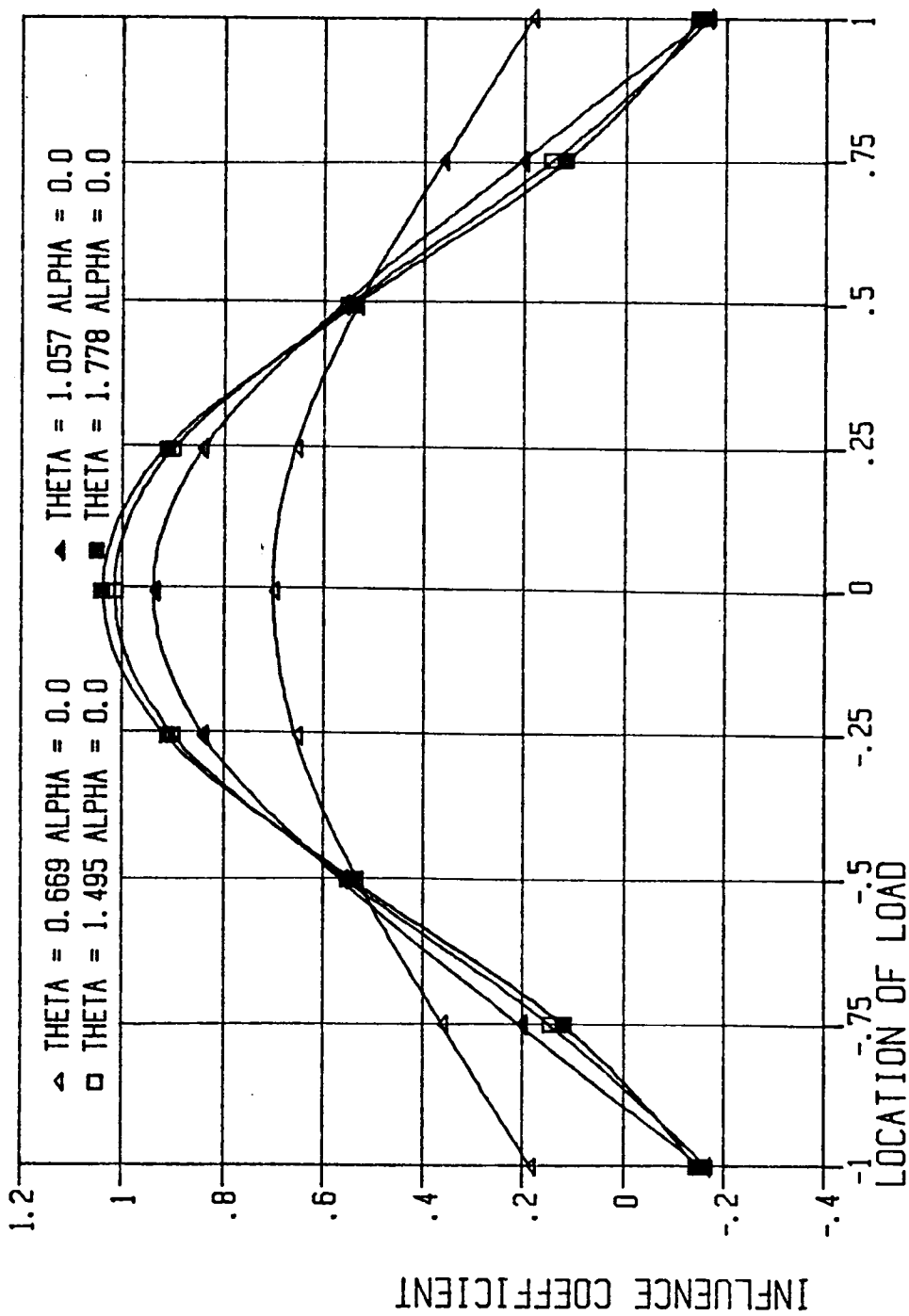


Figure A-1. Three Girder Bridge (Girder @ 0.0) - $\alpha = 0.0$.

THREE GIRDER BRIDGE (GIRDER @ 0.0)

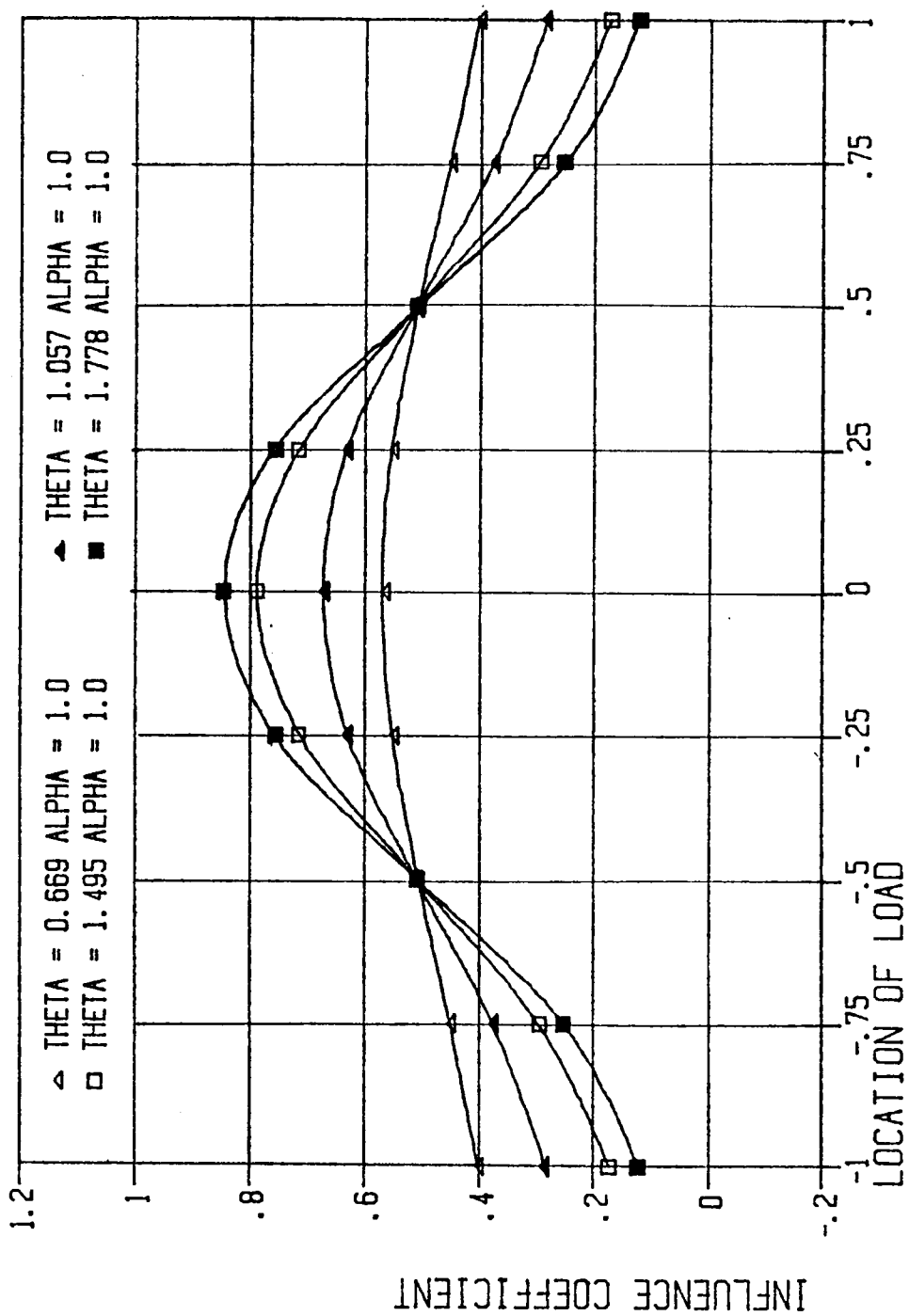


Figure A-2. Three Girder Bridge (Girder @ 0.0) - $\alpha = 1.0$.

THREE GIRDER BRIDGE (GIRDER @ 1.0)

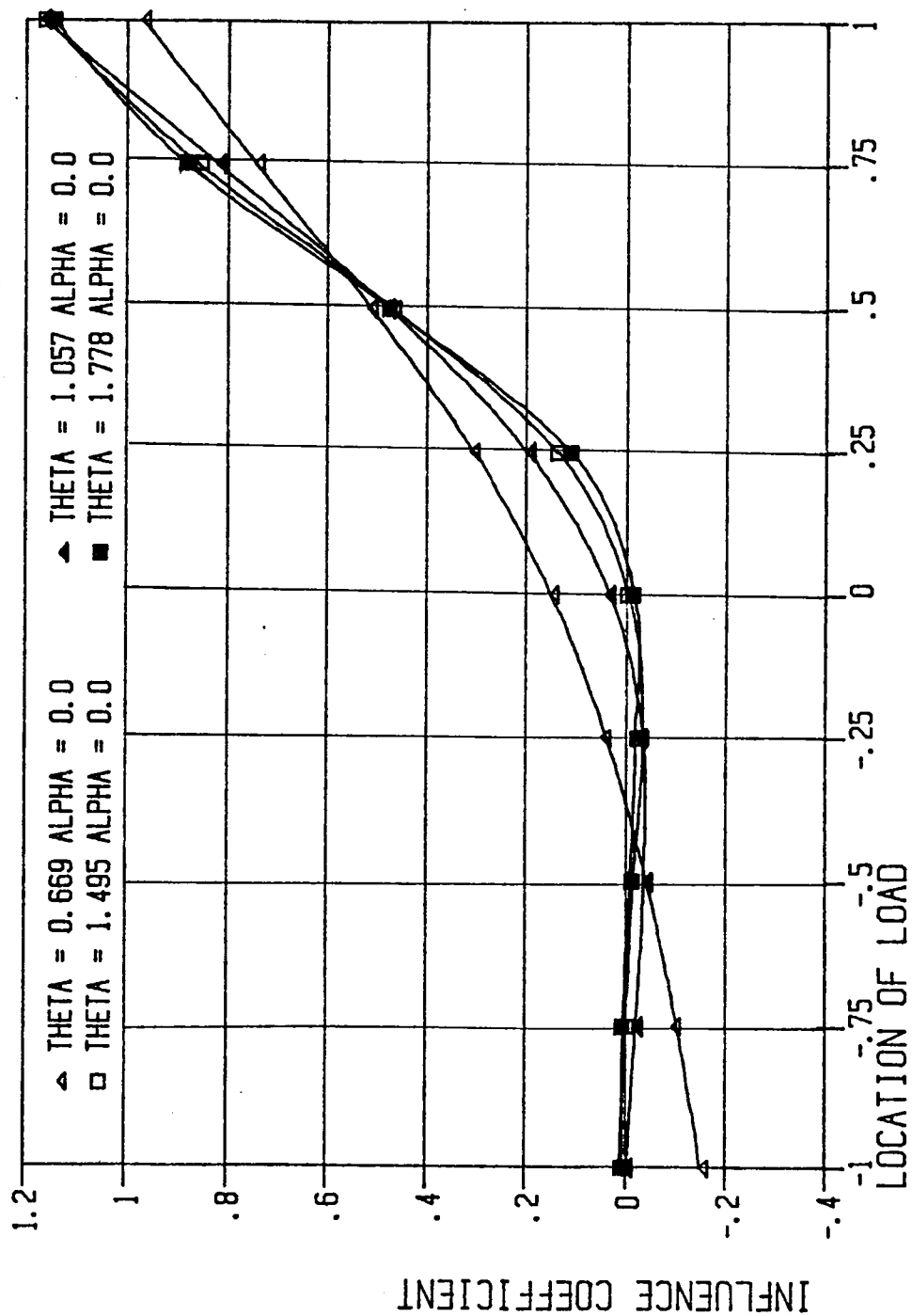


Figure A-3. Three Girder Bridge (Girder @ 1.0) - $\alpha = 0.0$.

THREE GIRDER BRIDGE (GIRDER @ 1.0)

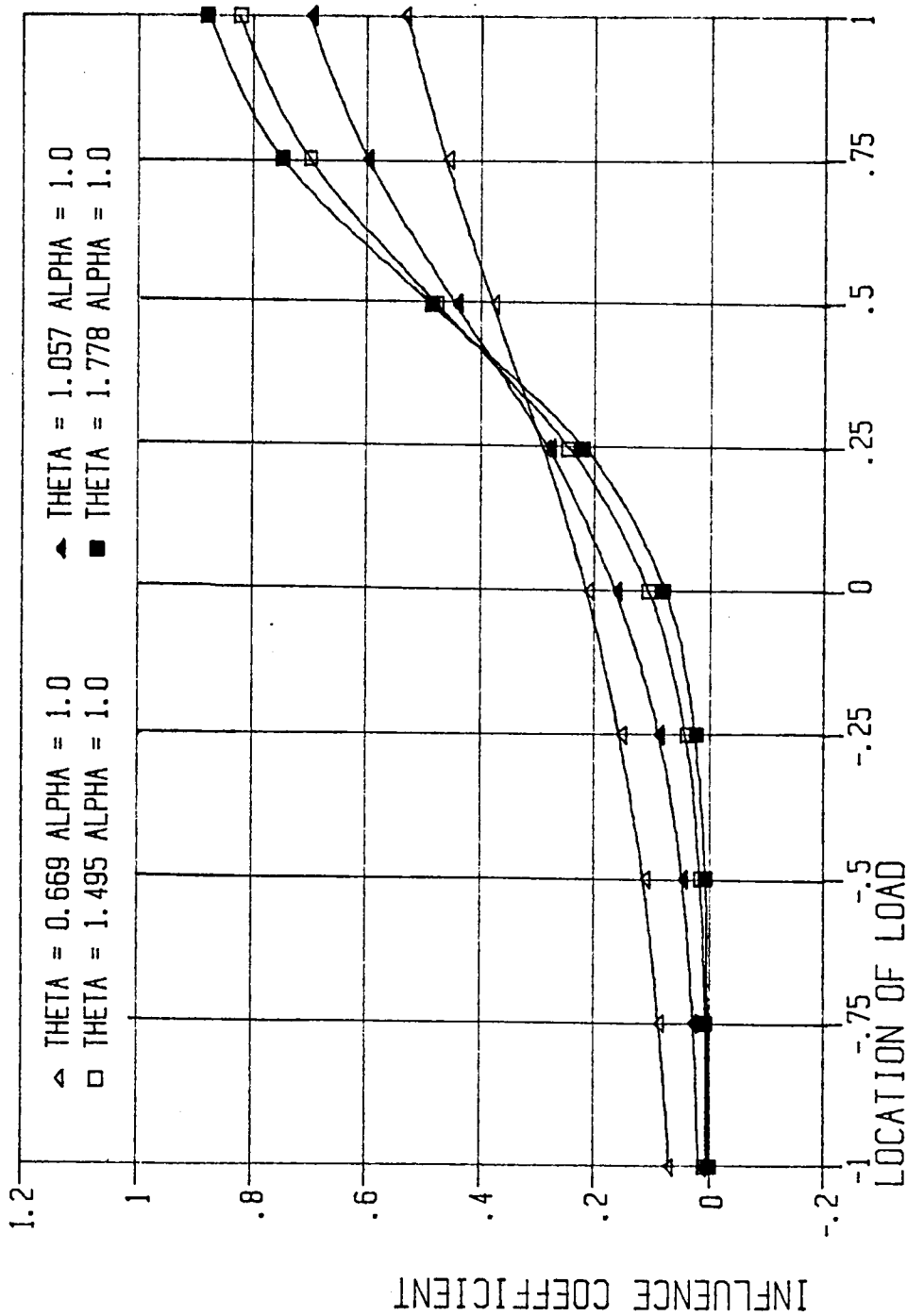


Figure A-4. Three Girder Bridge (Girder @ 1.0) - $\alpha = 1.0$.

FOUR GIRDER BRIDGE (GIRDER @ 0.33)

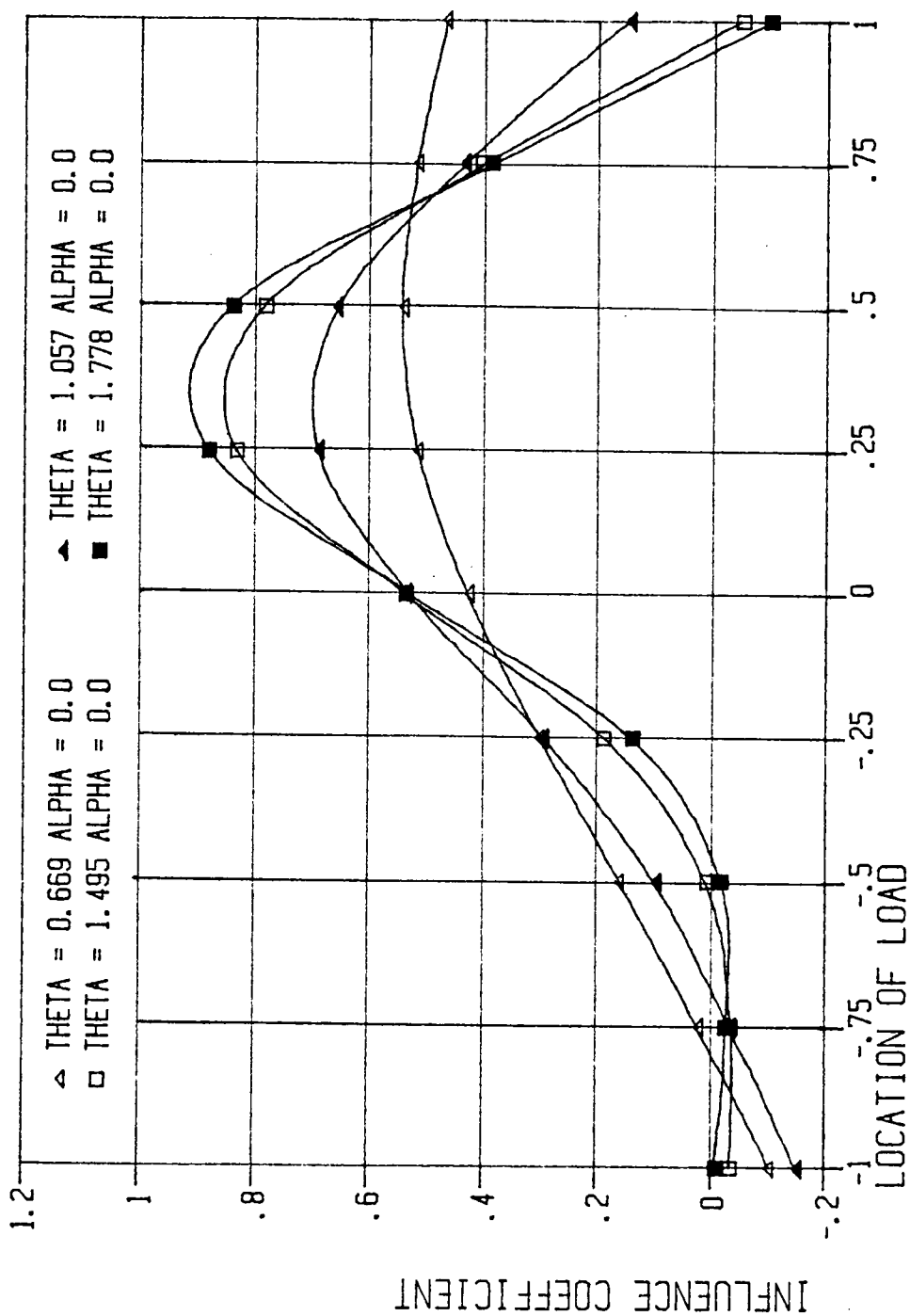


Figure A-5. Four Girder Bridge (Girder @ 0.33) - $\alpha = 0.0$.

FOUR GIRDER BRIDGE (GIRDER @ 0.33)

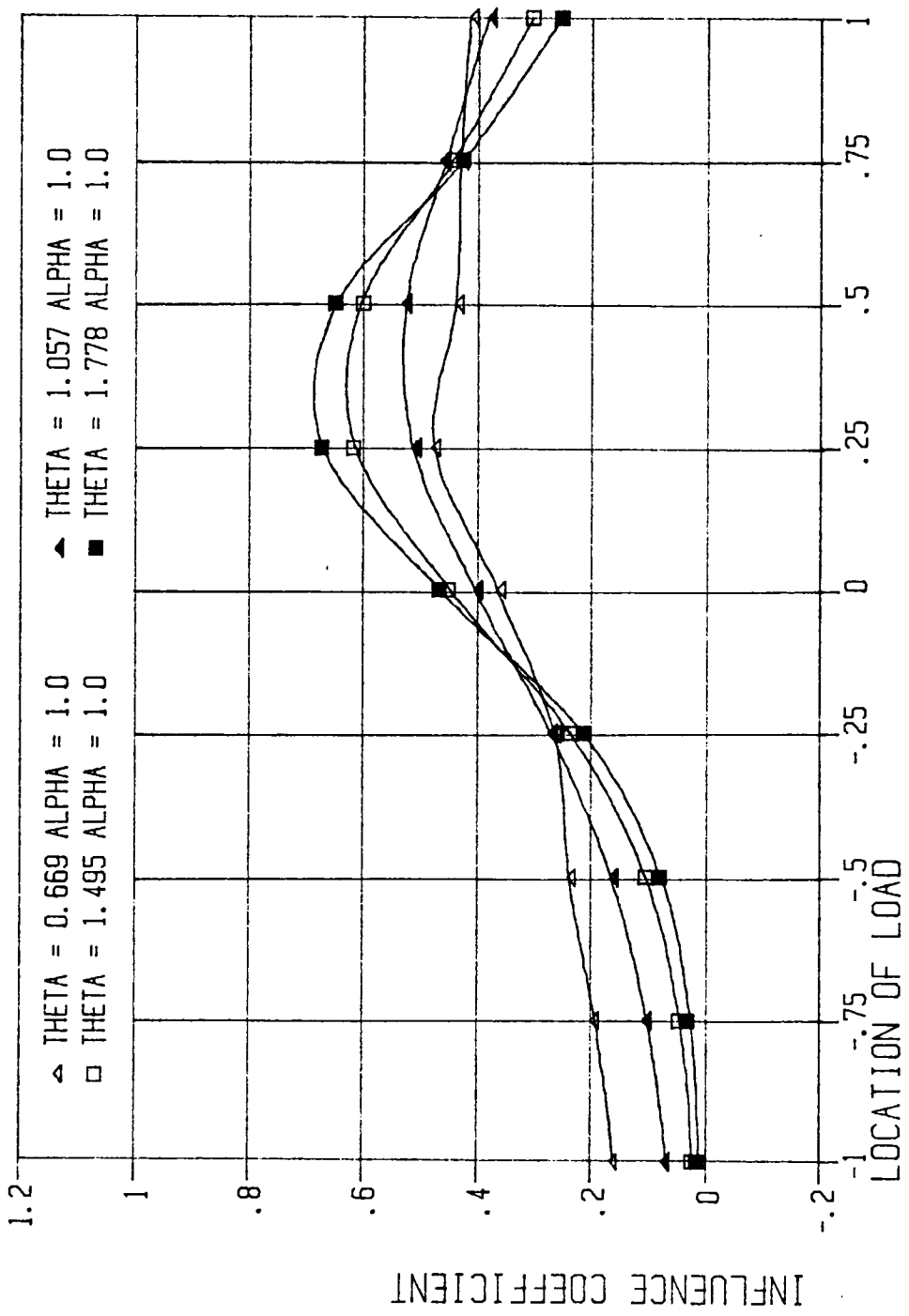


Figure A-6. Four Girder Bridge (Girder @ 0.33) - $\alpha = 1.0$.

FOUR GIRDER BRIDGE (GIRDER @ 1.0)

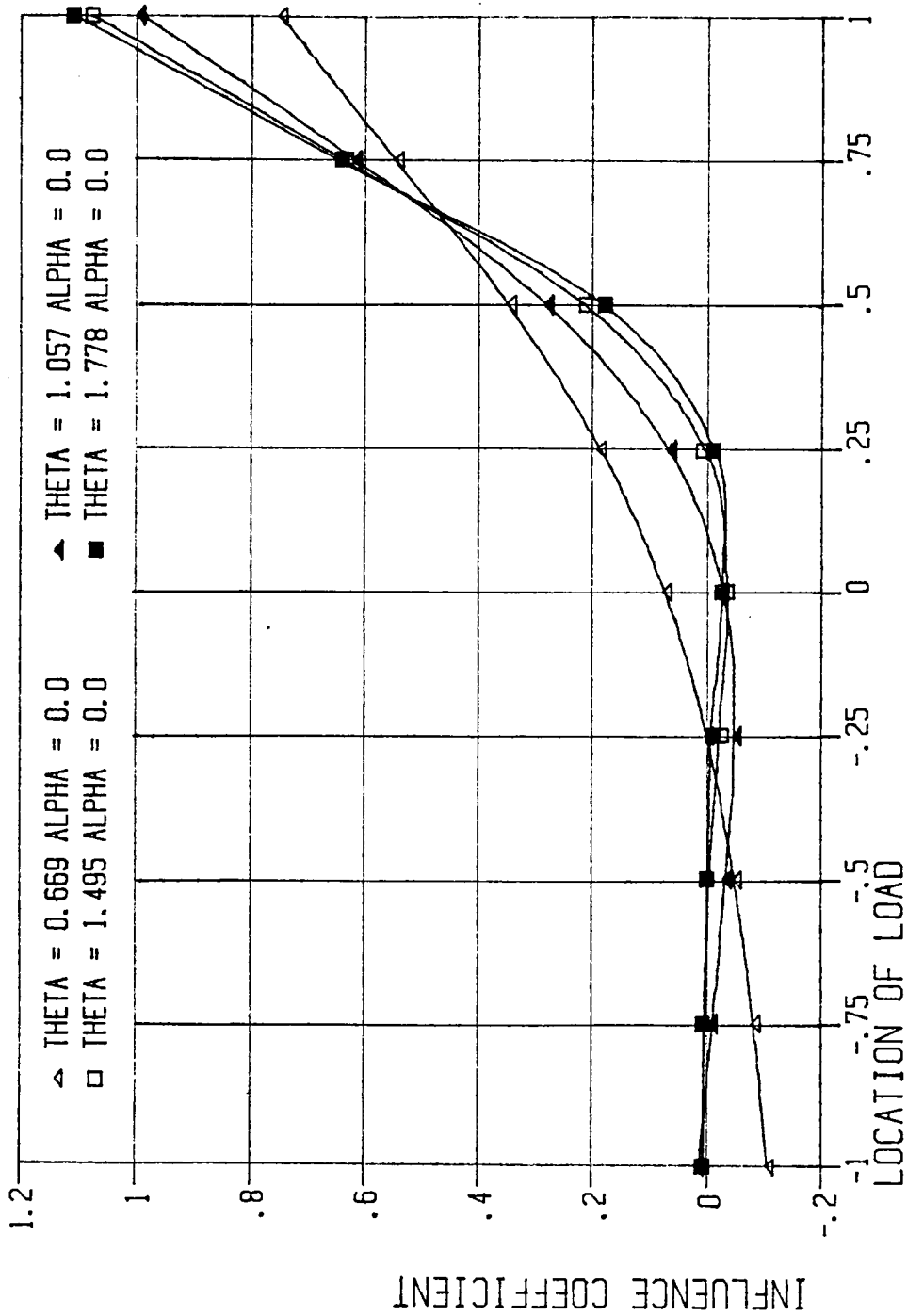


Figure A-7. Four Girder Bridge (Girder @ 1.0) - $\alpha = 0.0$.

FOUR GIRDER BRIDGE (GIRDER @ 1.0)

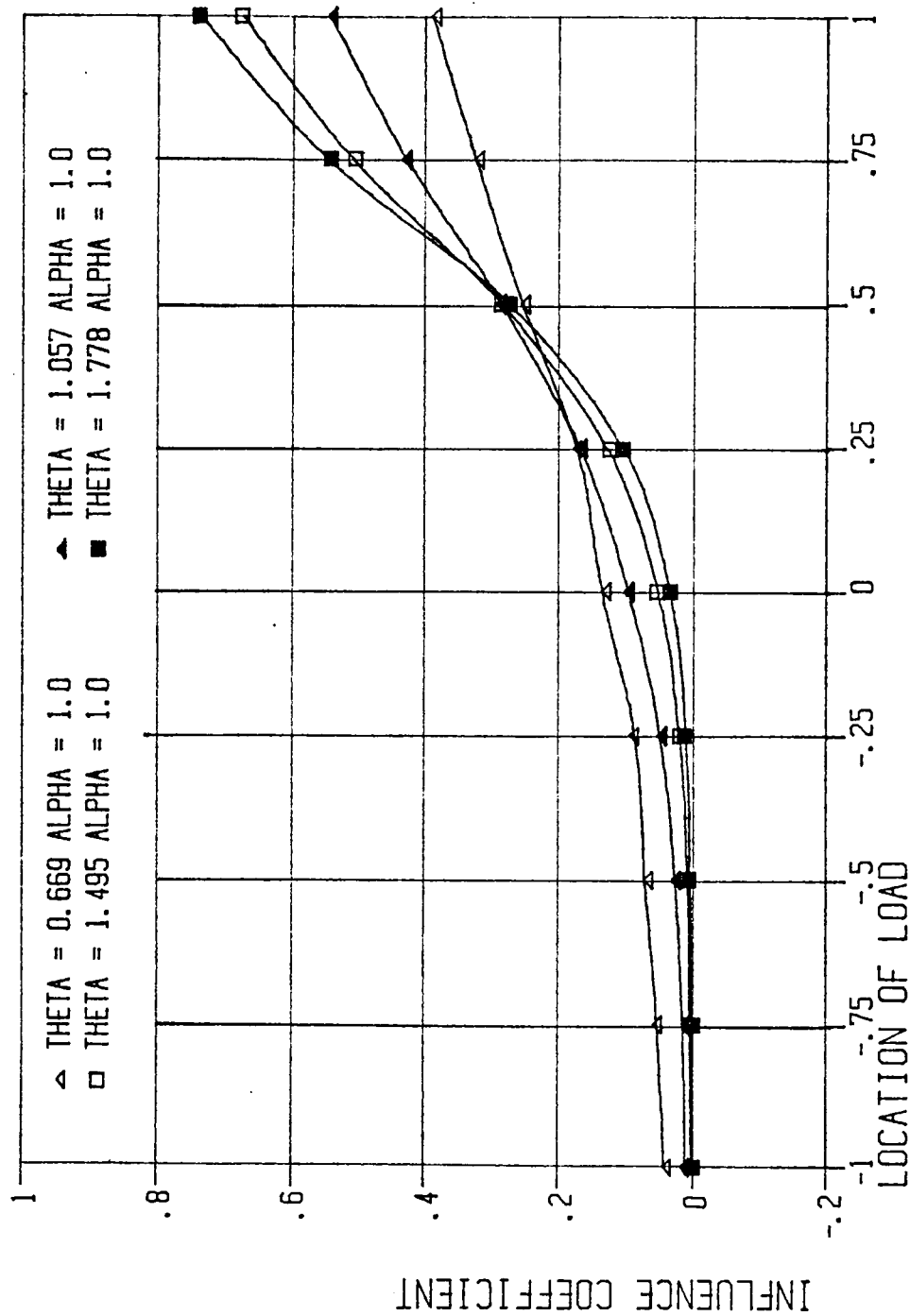


Figure A-8. Four Girder Bridge (Girder @ 1.0) - $\alpha = 1.0$.

FIVE GIRDER BRIDGE (GIRDER @ 0.0)

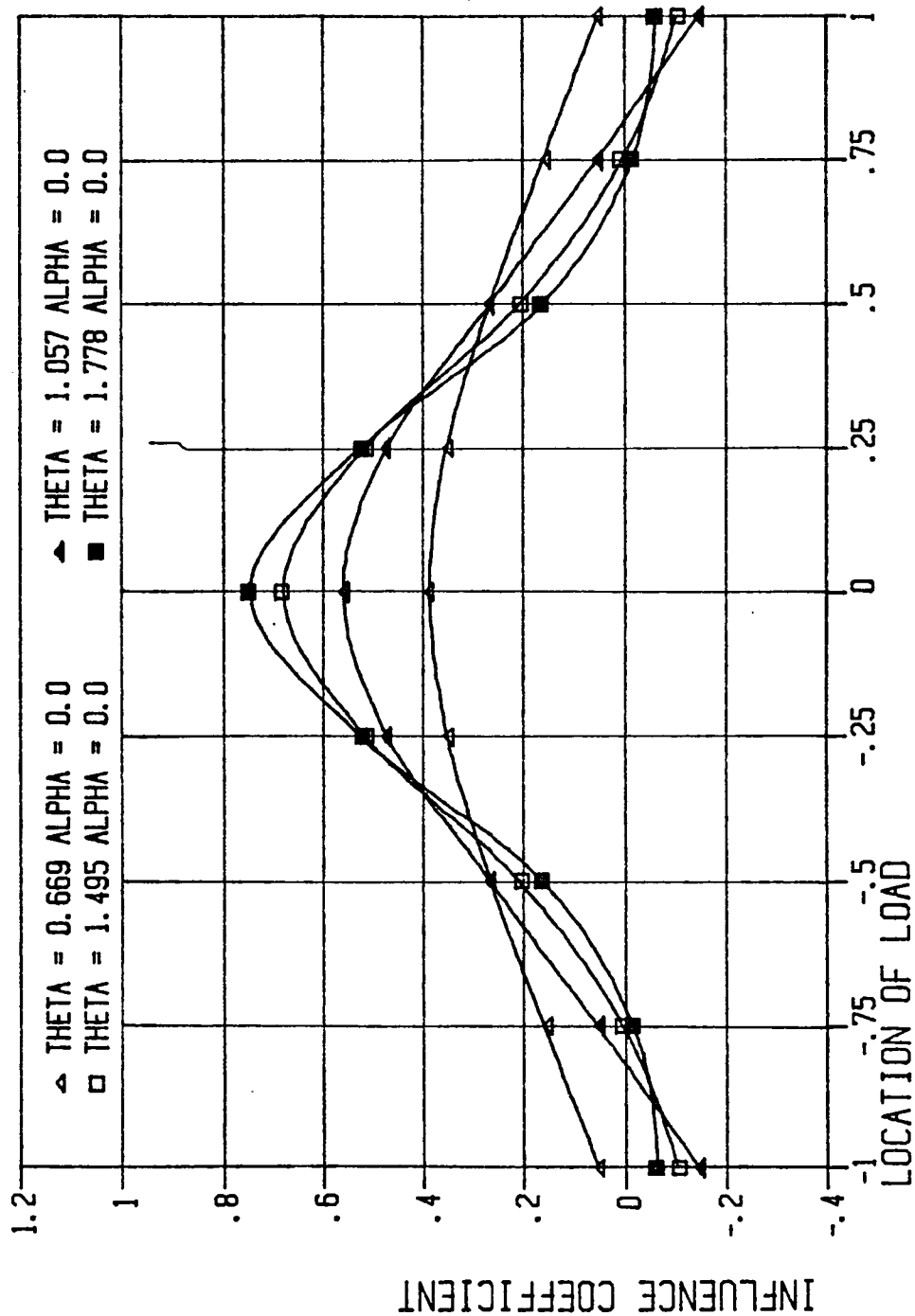


Figure A-9. Five Girder Bridge (Girder @ 0.0) - $\alpha = 0.0$.

FIVE GIRDER BRIDGE (GIRDER @ 0.0)

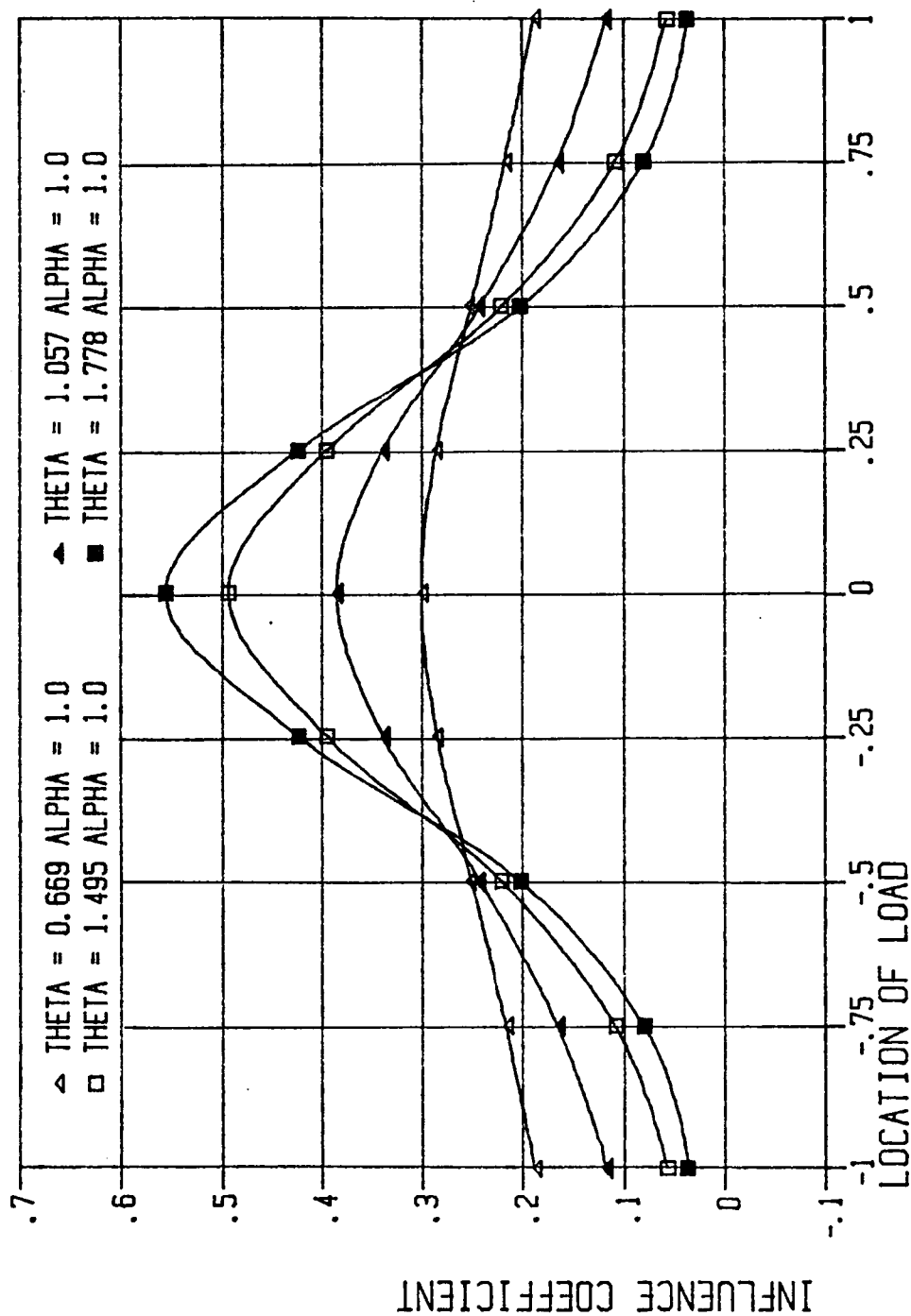


Figure A-10. Five Girder Bridge (Girder @ 0.0) - $\alpha = 1.0$.

FIVE GIRDER BRIDGE (GIRDER @ 0.5)

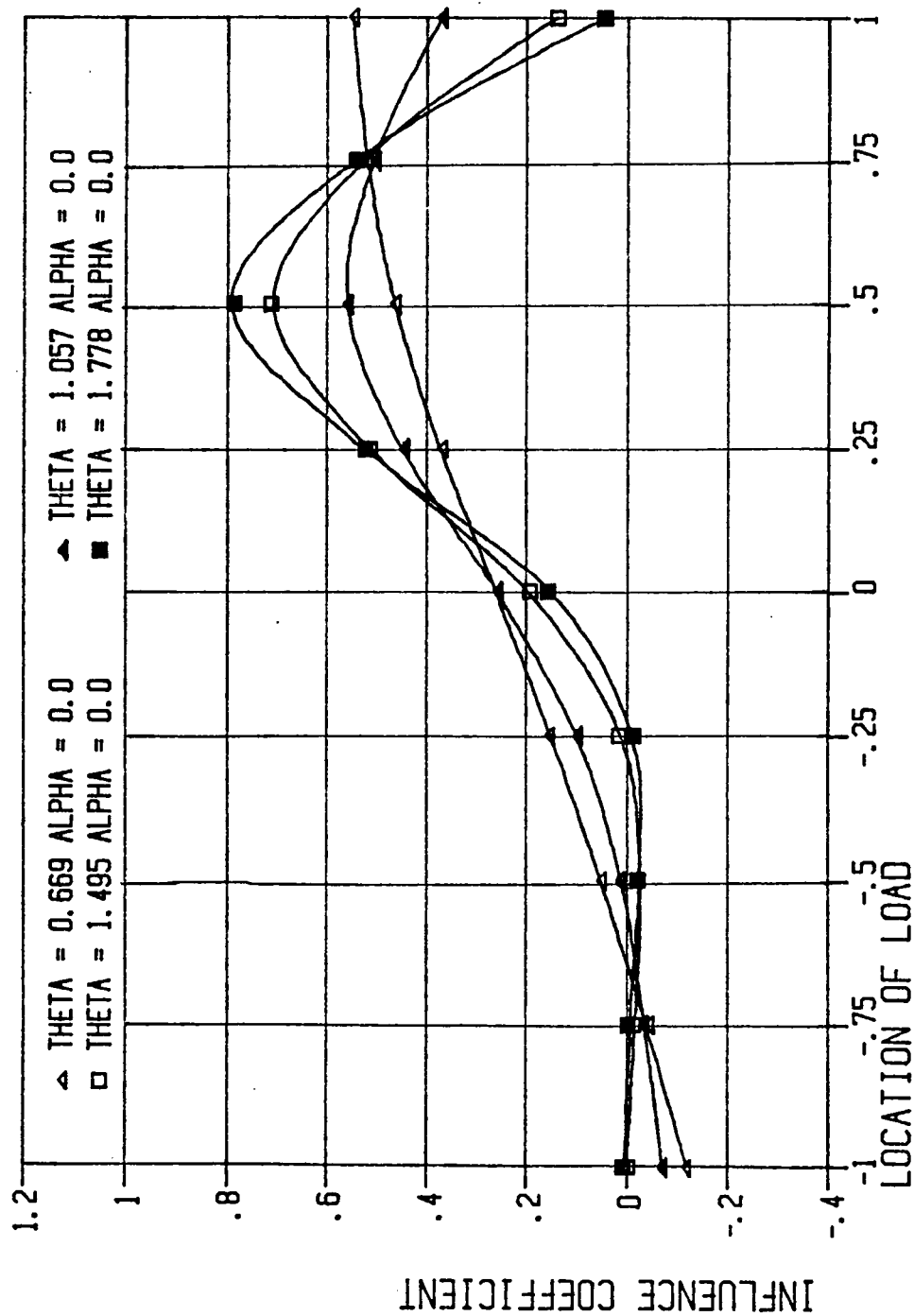


Figure A-11. Five Girder Bridge (Girder @ 0.5) - $\alpha = 0.0$.

FIVE GIRDER BRIDGE (GIRDER @ 0.5)

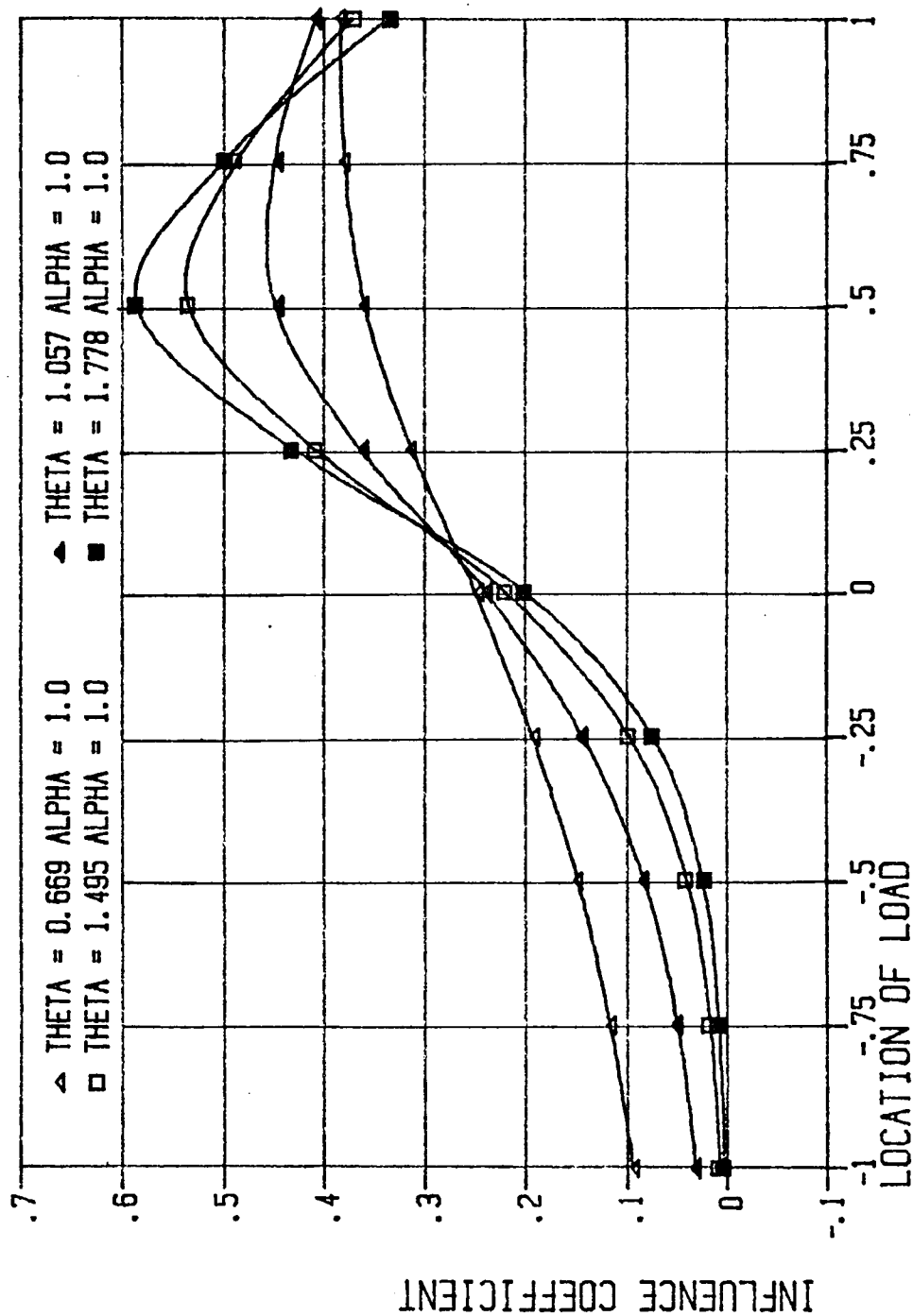


Figure A-12. Five Girder Bridge (Girder @ 0.5) - $\alpha = 1.0$.

FIVE GIRDER BRIDGE (GIRDER @1.0)

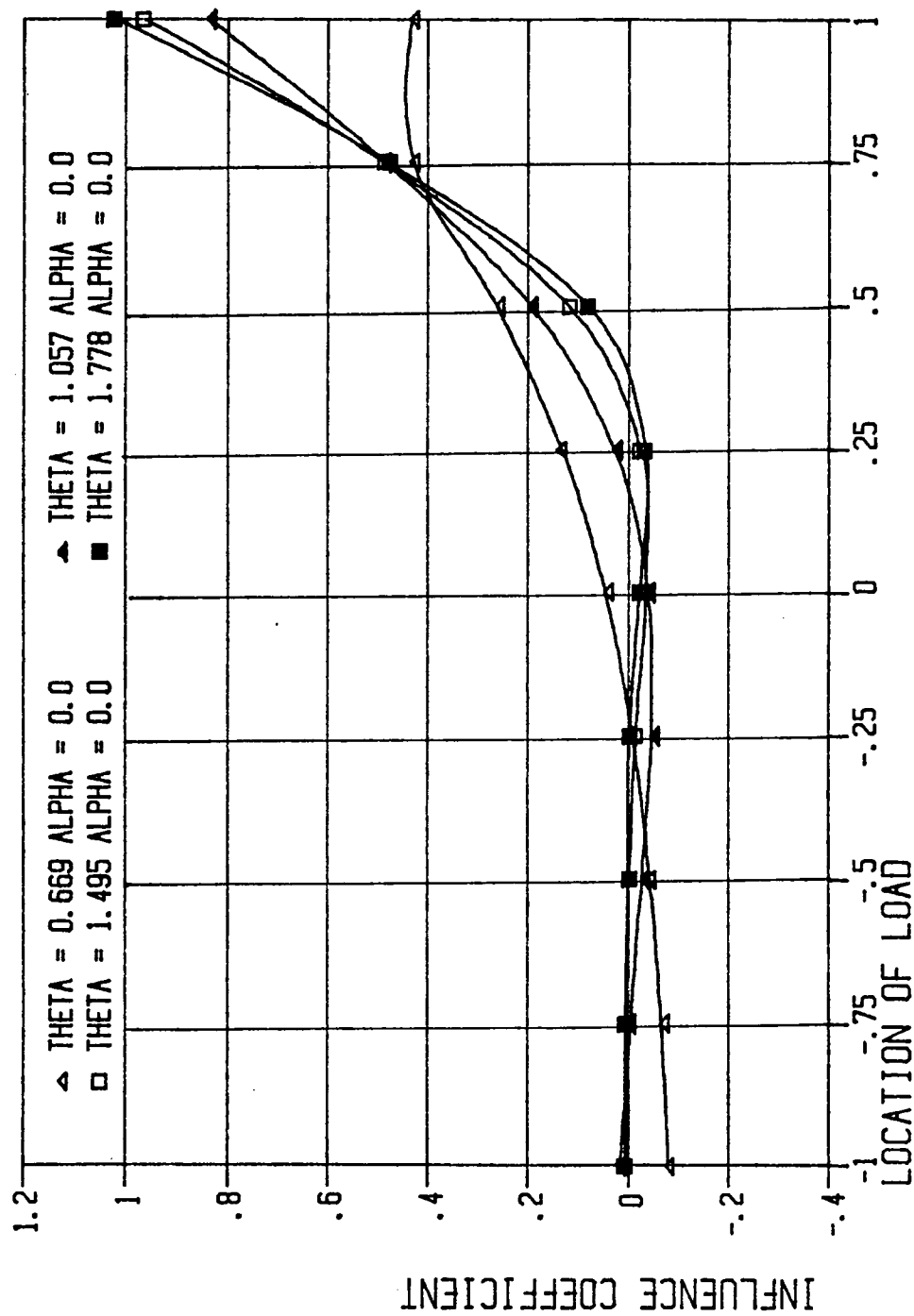


Figure A-13. Five Girder Bridge (Girder @ 1.0) - $\alpha = 0.0$.

FIVE GIRDER BRIDGE (GIRDER @ 1.0)

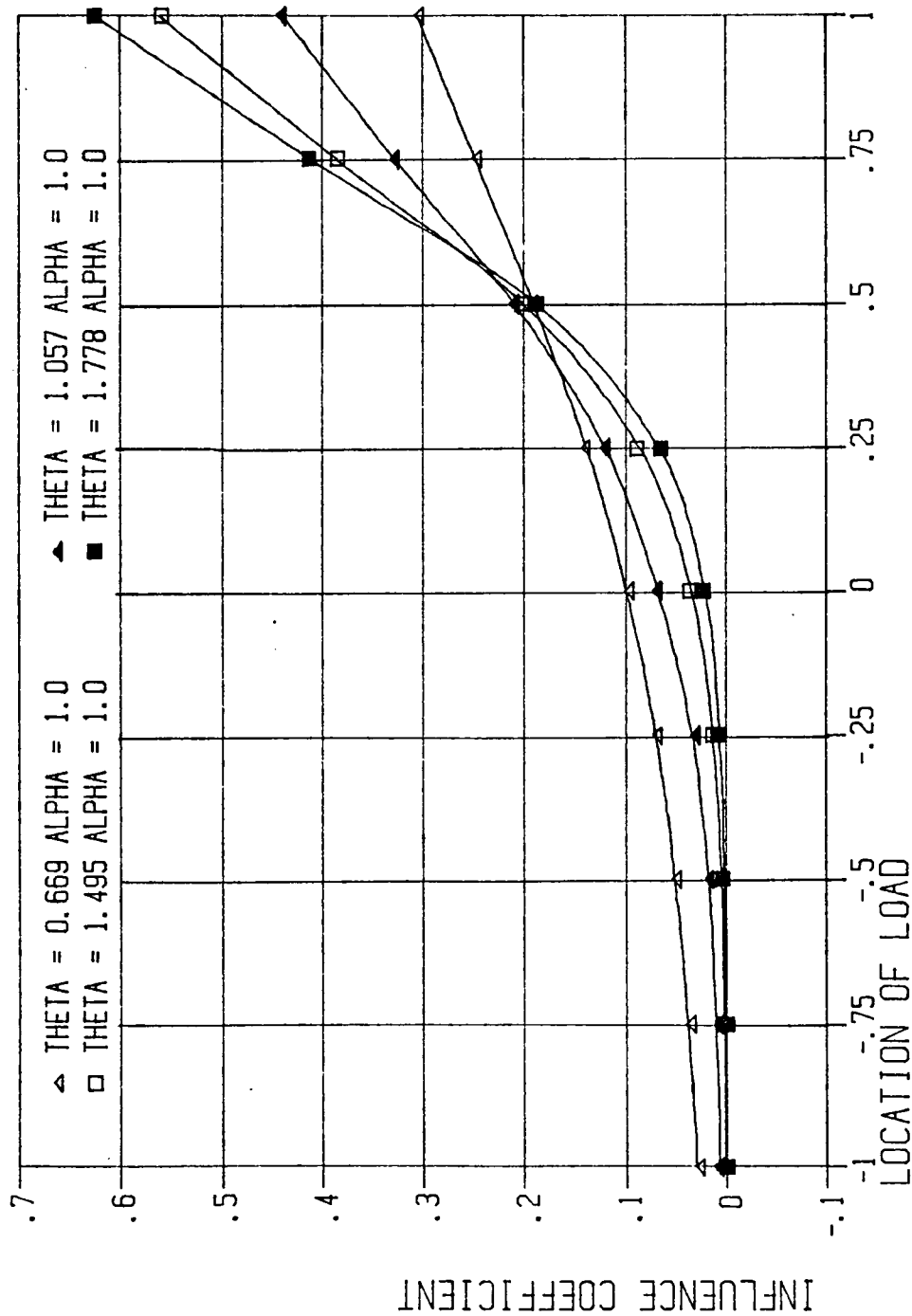


Figure A-14. Five Girder Bridge (Girder @ 1.0) - $\alpha = 1.0$.

SIX GIRDER BRIDGE (GIRDER @ 0.2)

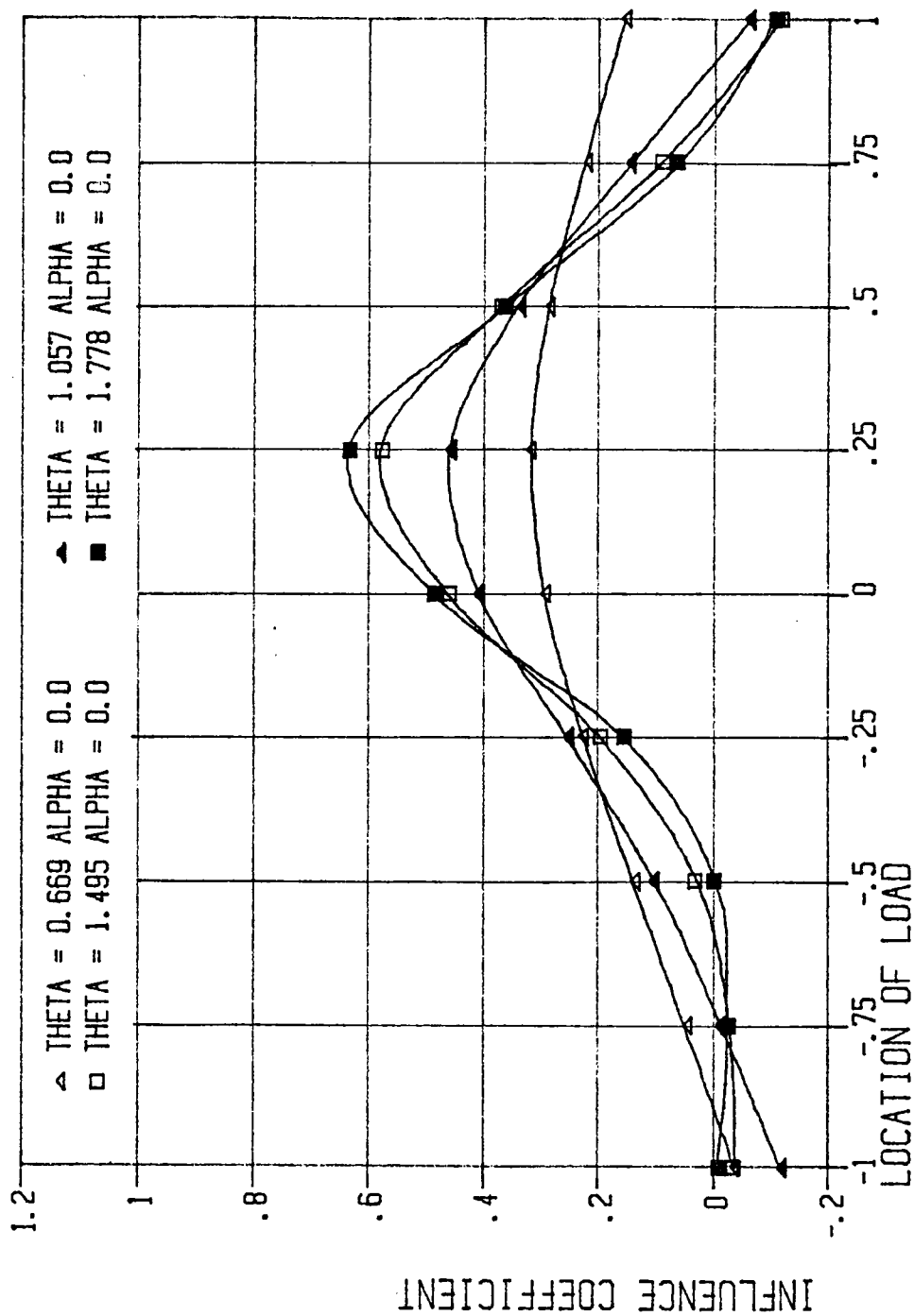


Figure A-15. Six Girder Bridge (Girder @ 0.2) - $\alpha = 0.0$.

SIX GIRDER BRIDGE (GIRDER @ 0.2)

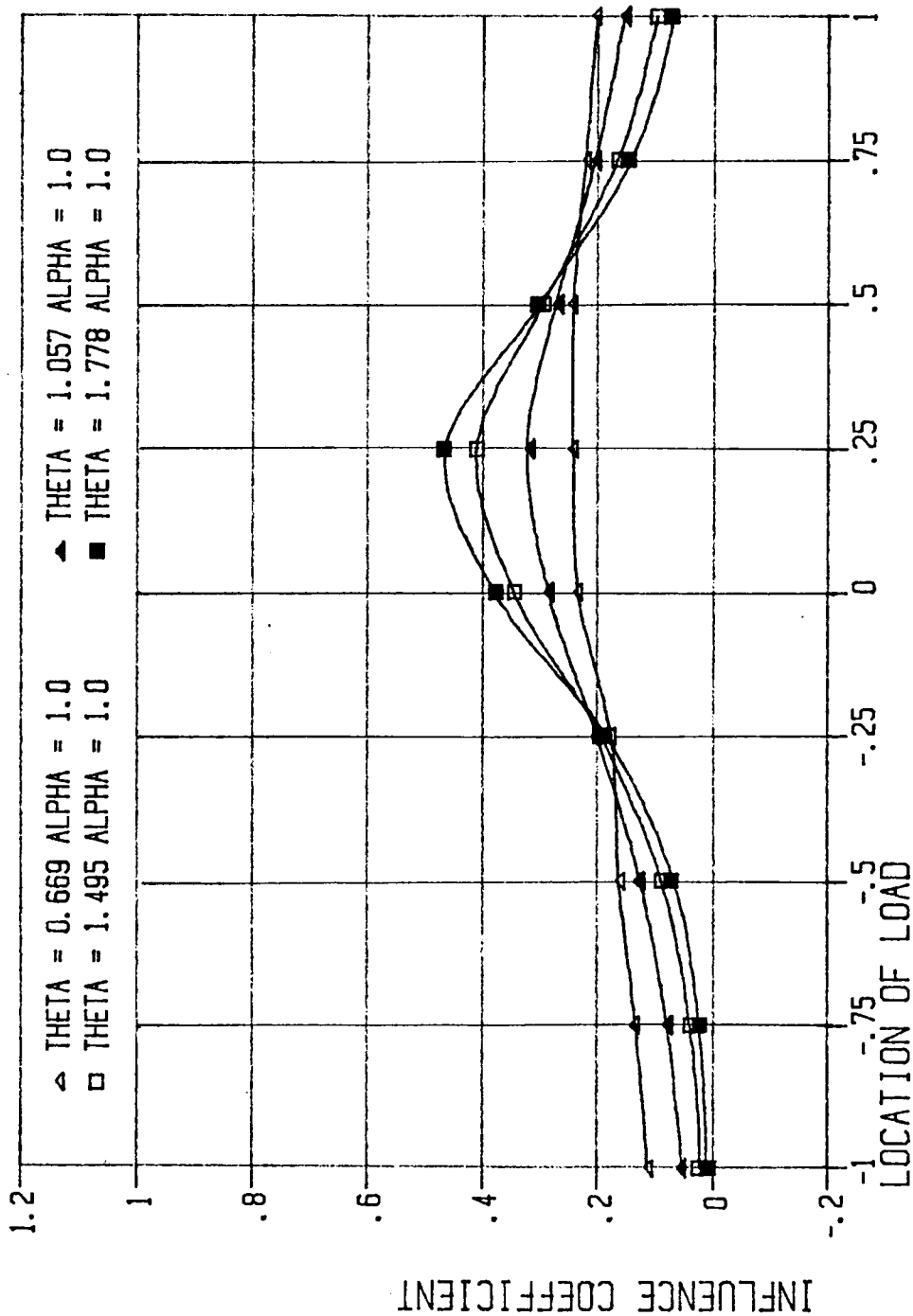


Figure A-16. Six Girder Bridge (Girder @ 0.2) - $\alpha = 1.0$.

SIX GIRDER BRIDGE (GIRDER @ 0.6)

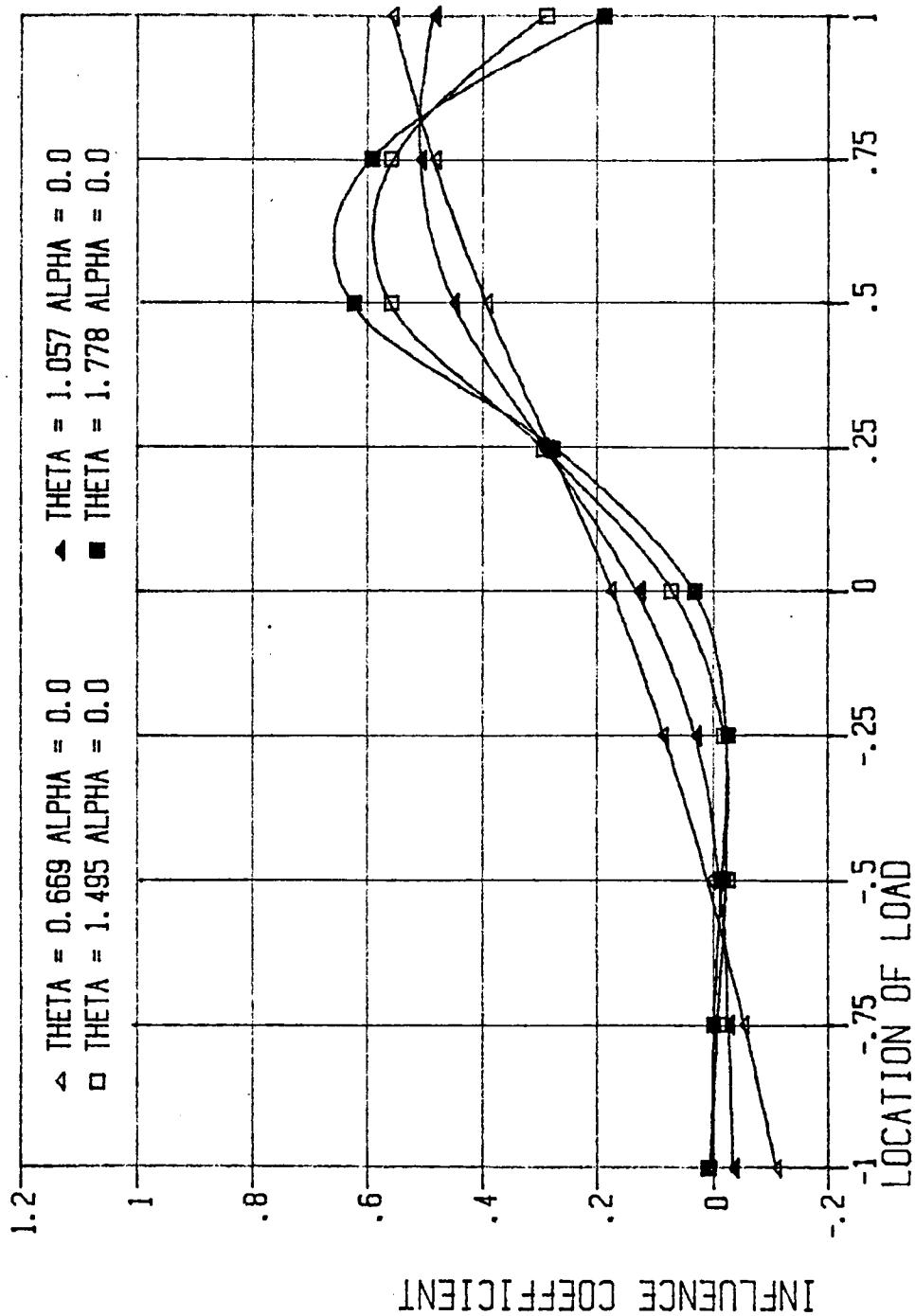


Figure A-17. Six Girder Bridge (Girder @ 0.6) - $\alpha = 0.0$.

SIX GIRDER BRIDGE (GIRDER @ 0.6)

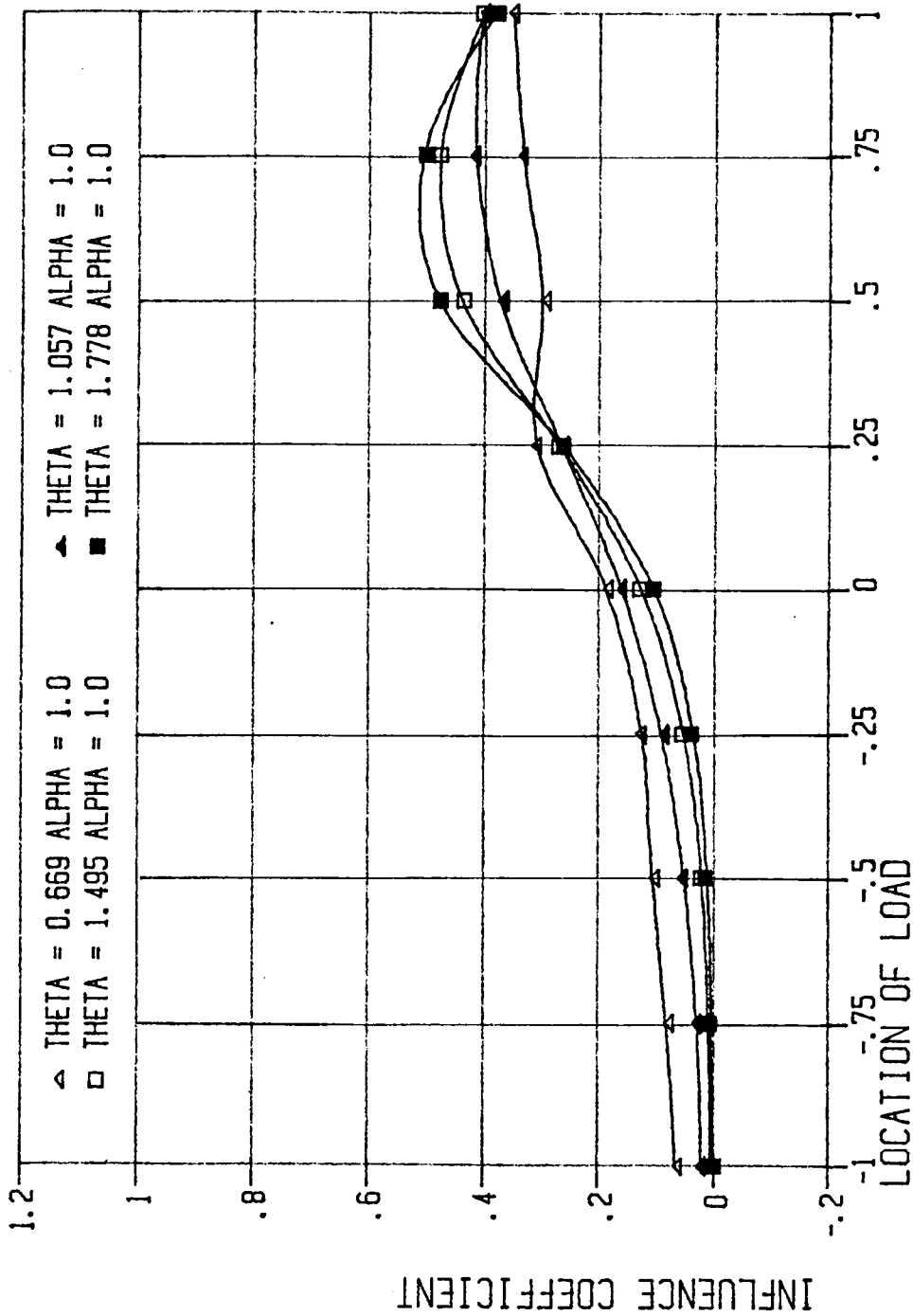


Figure A-18. Six Girder Bridge (Girder @ 0.6) - $\alpha = 1.0$.

SIX GIRDER BRIDGE (GIRDER @ 1.0)

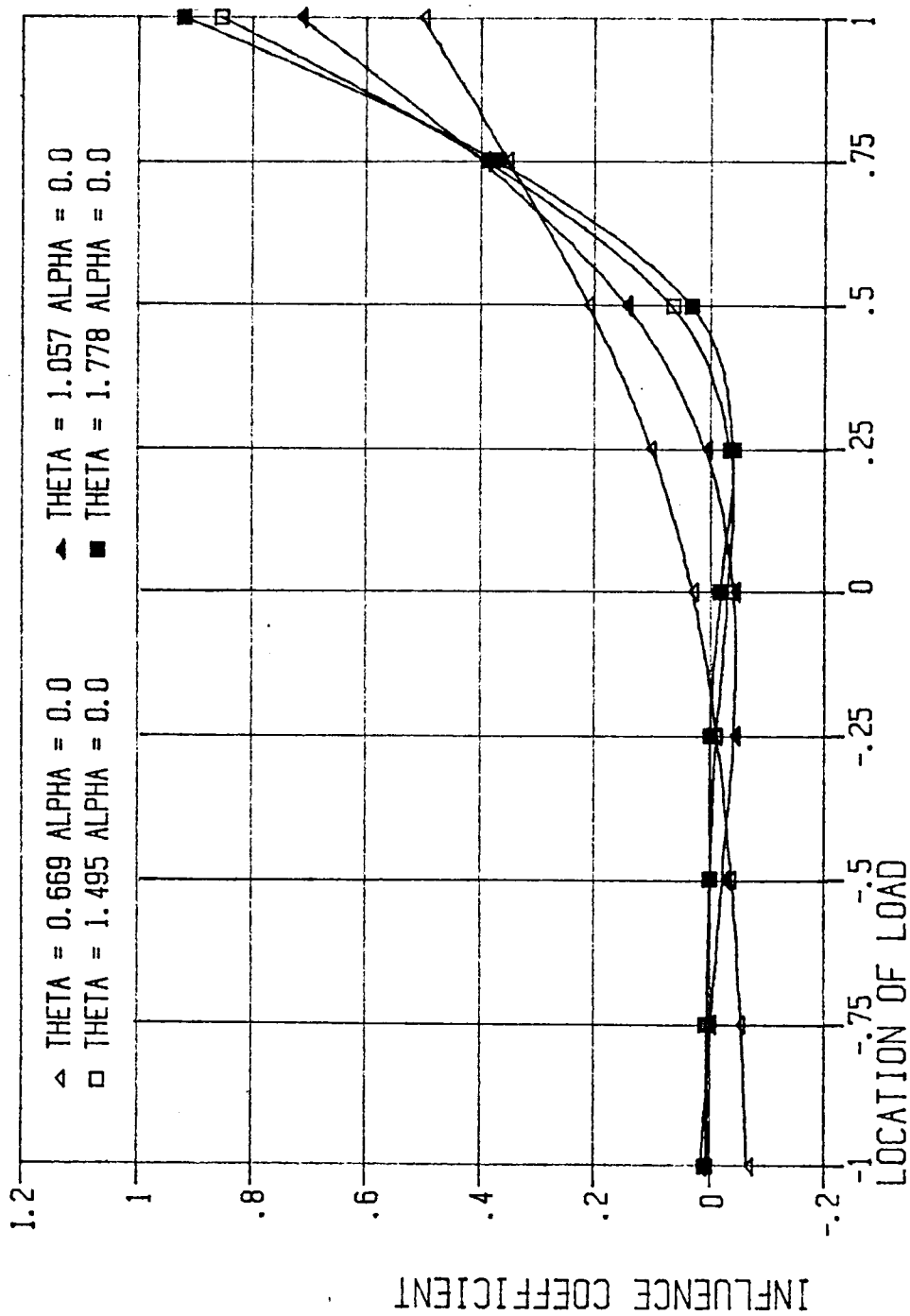


Figure A-19. Six Girder Bridge (Girder @ 1.0) - $\alpha = 0.0$.

SIX GIRDER BRIDGE (GIRDER @ 1.0)

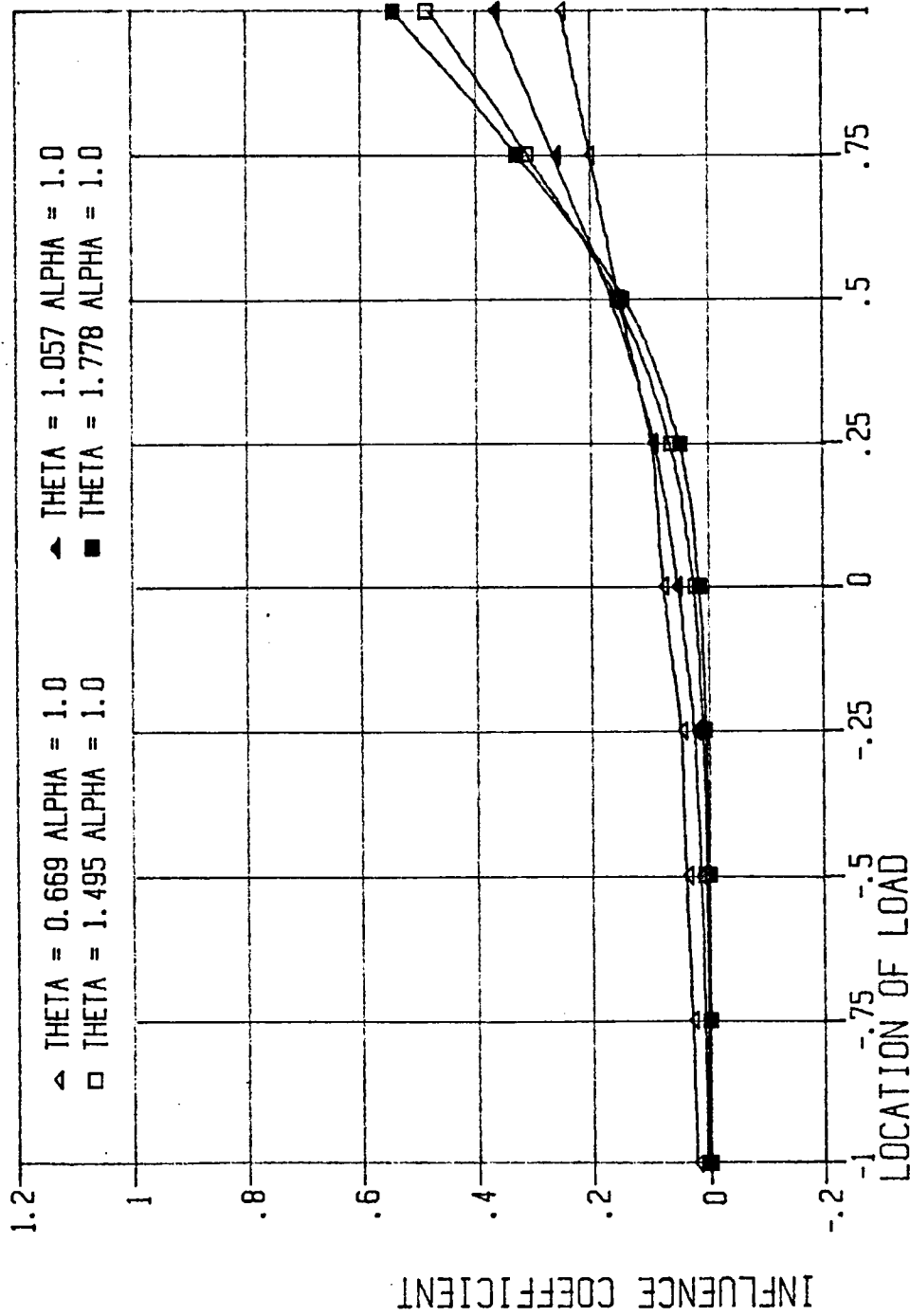


Figure A-20. Six Girder Bridge (Girder @ 1.0) - $\alpha = 1.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 0.0)

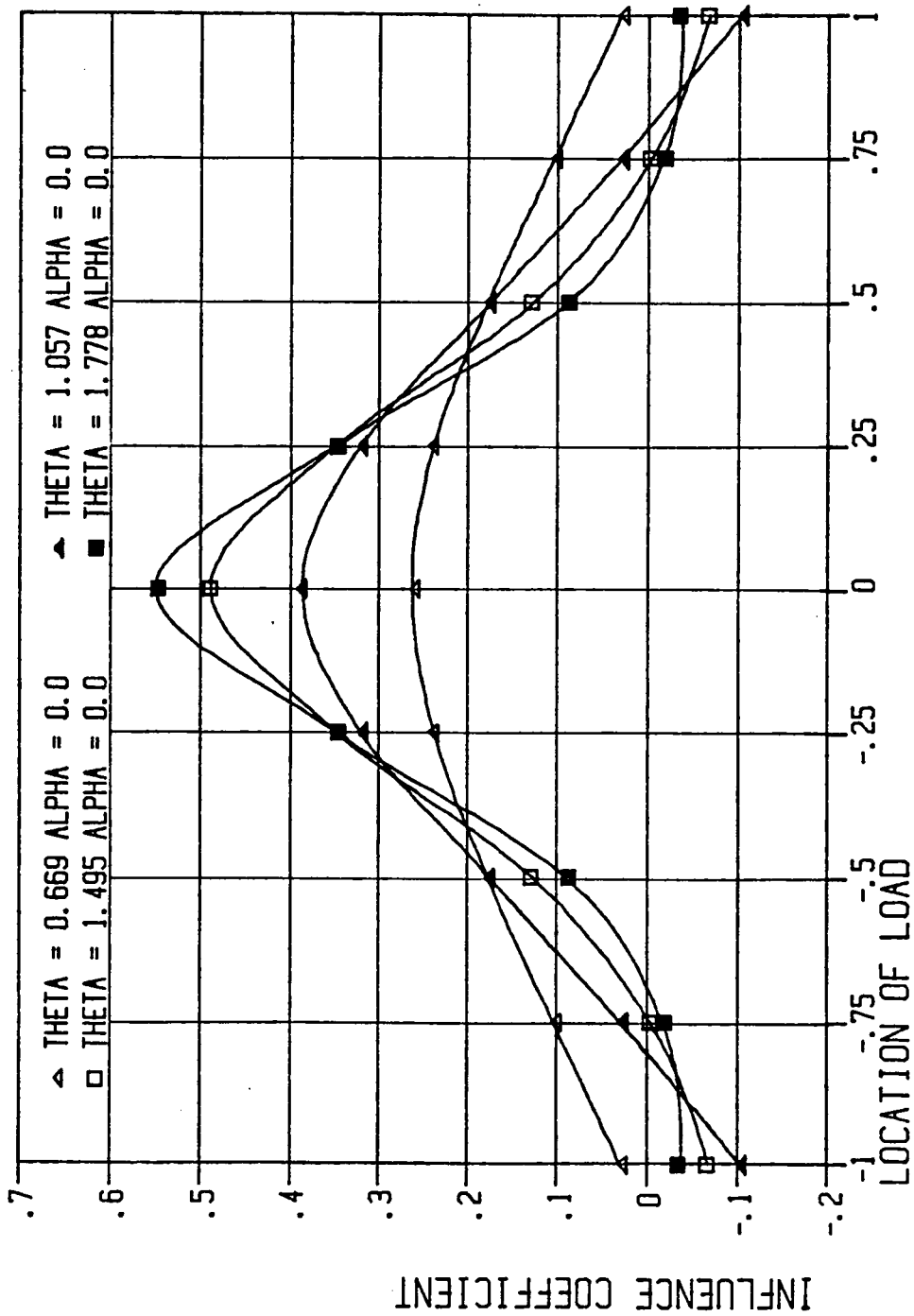


Figure A-21. Seven Girder Bridge (Girder @ 0.0) - $\alpha = 0.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 0.0)

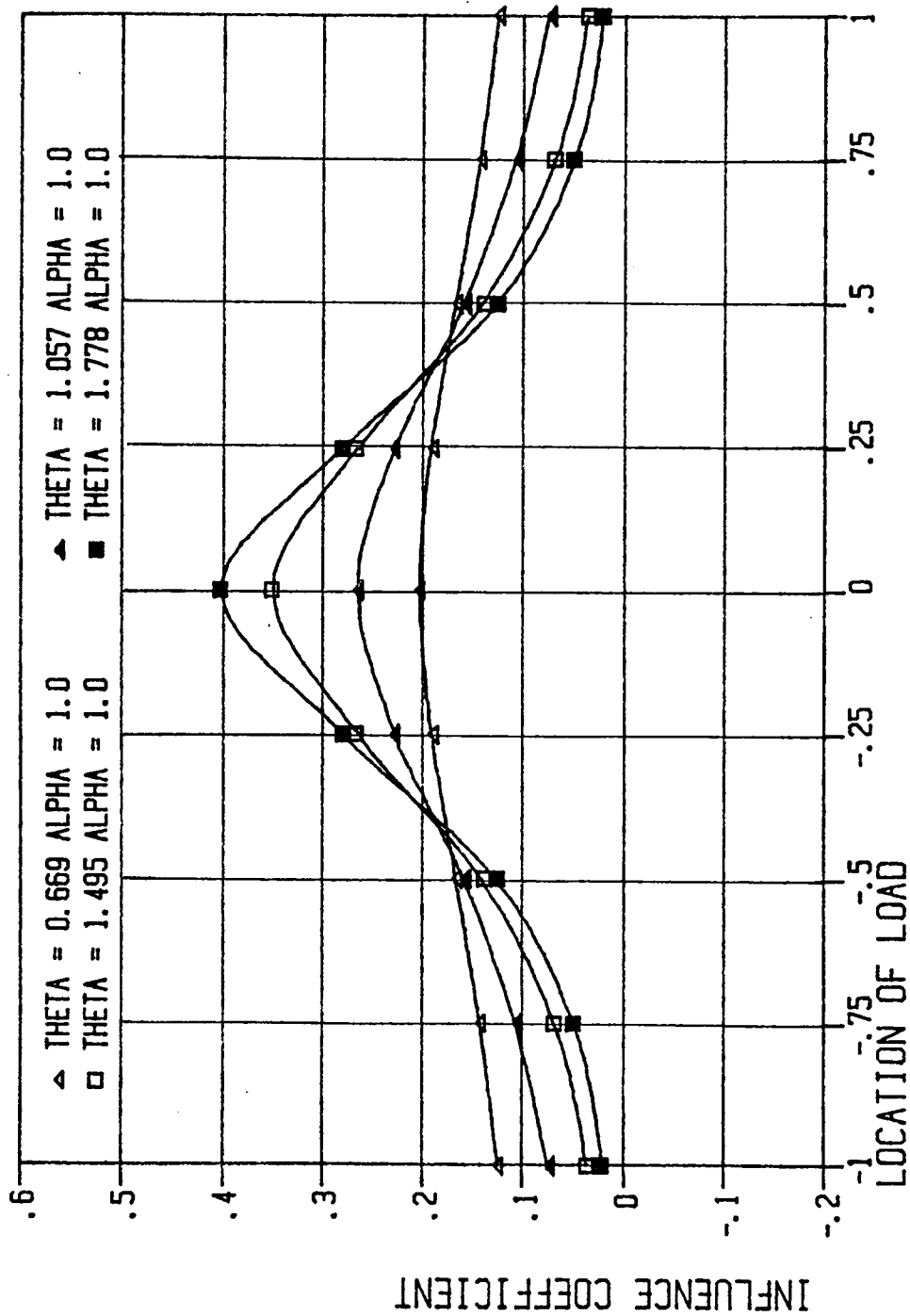


Figure A-22. Seven Girder Bridge (Girder @ 0.0) - $\alpha = 1.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 0.33)

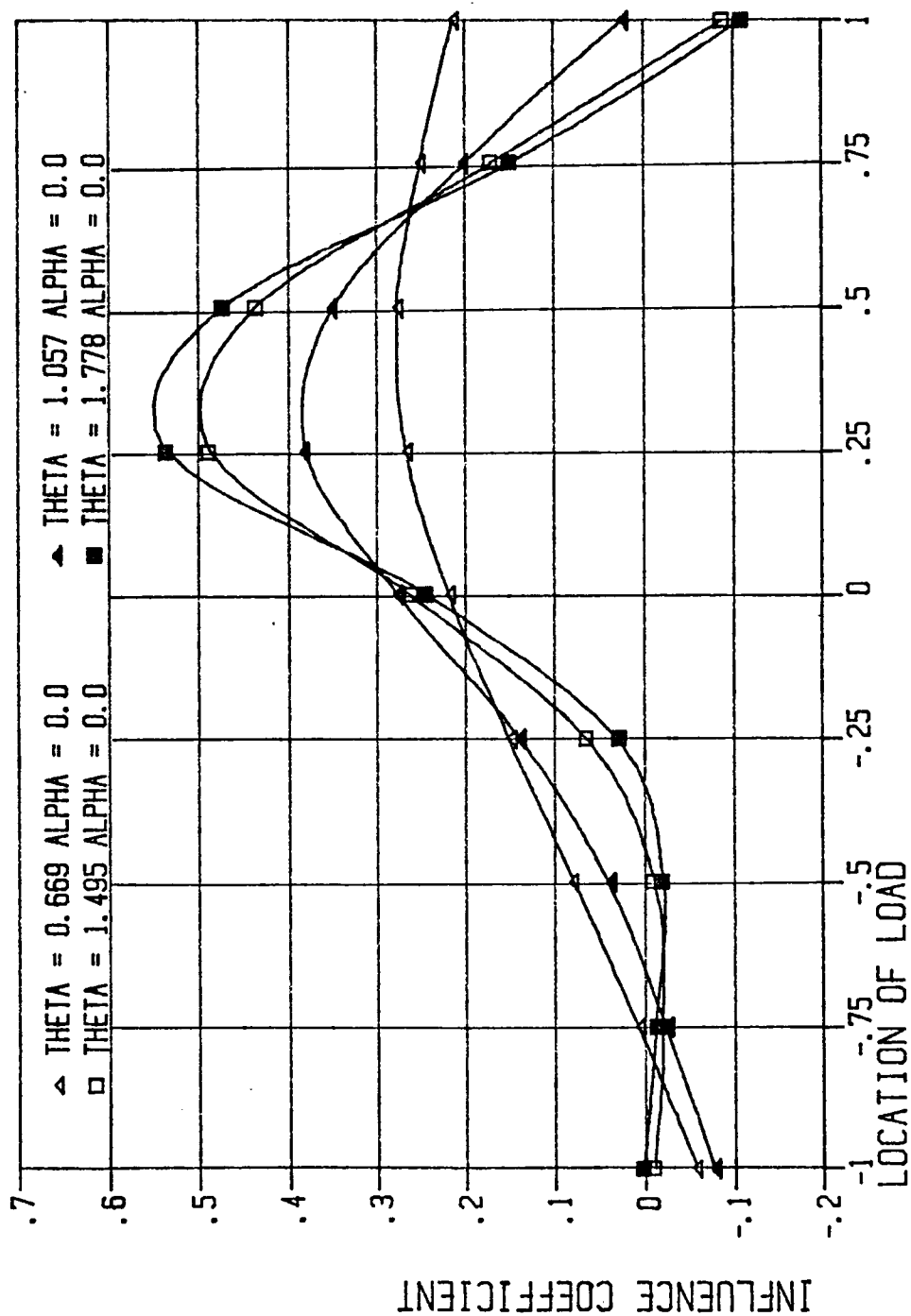


Figure A-23. Seven Girder Bridge (Girder @ 0.33) - $\alpha = 0.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 0.33)

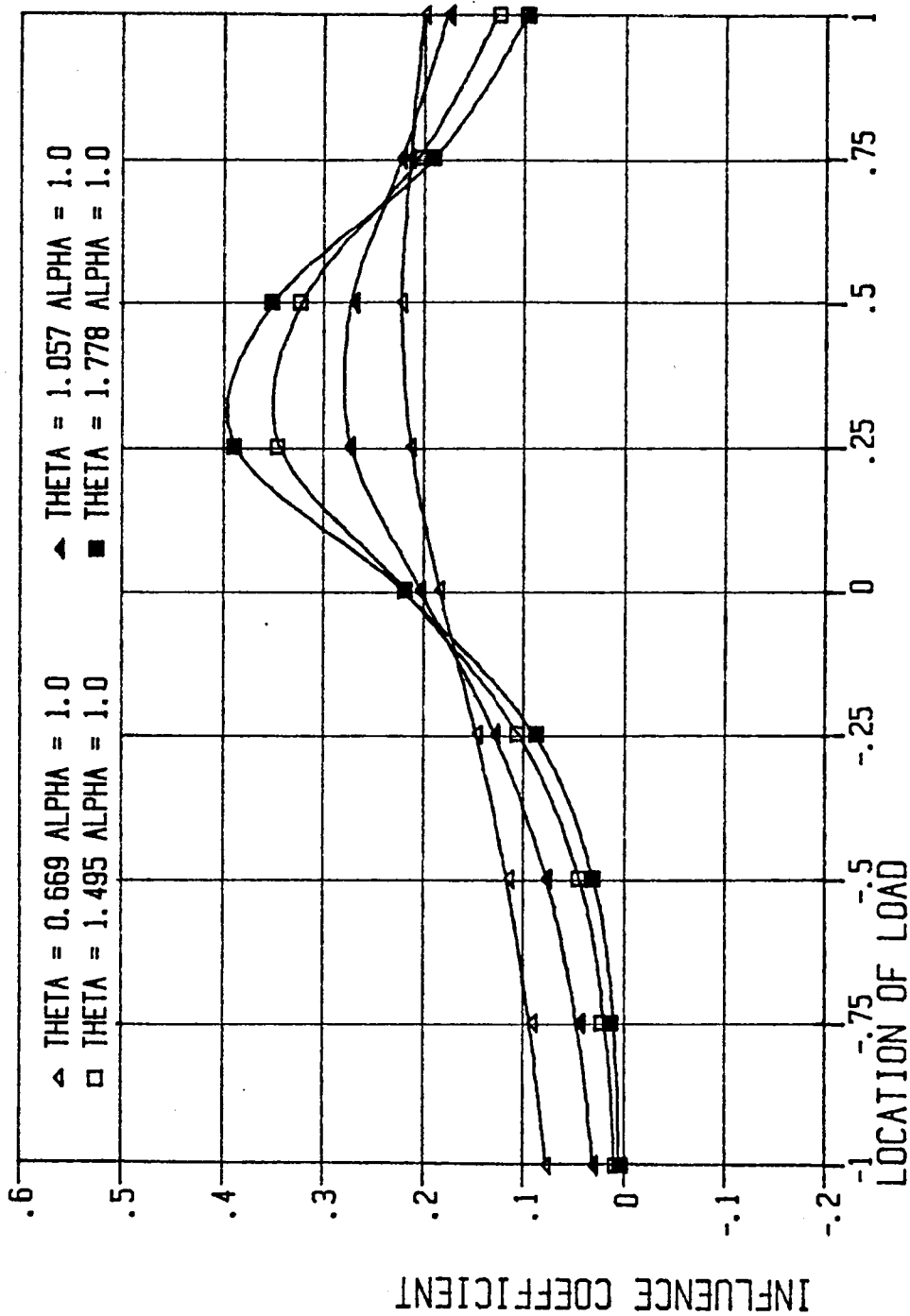


Figure A-24. Seven Girder Bridge (Girder @ 0.33) - $\alpha = 1.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 0.67)

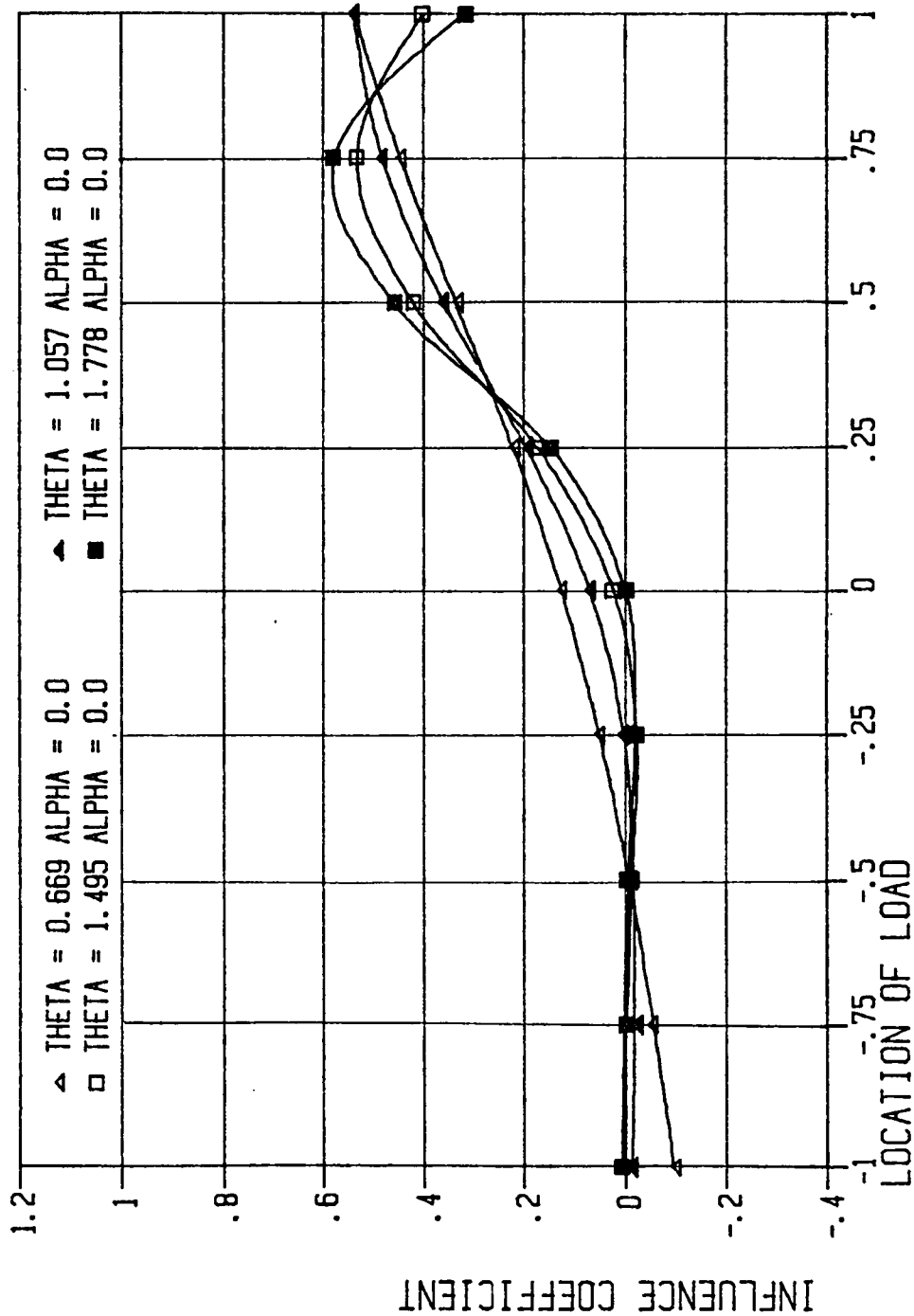


Figure A-25. Seven Girder Bridge (Girder @ 0.67) - $\alpha = 0.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 0.67)

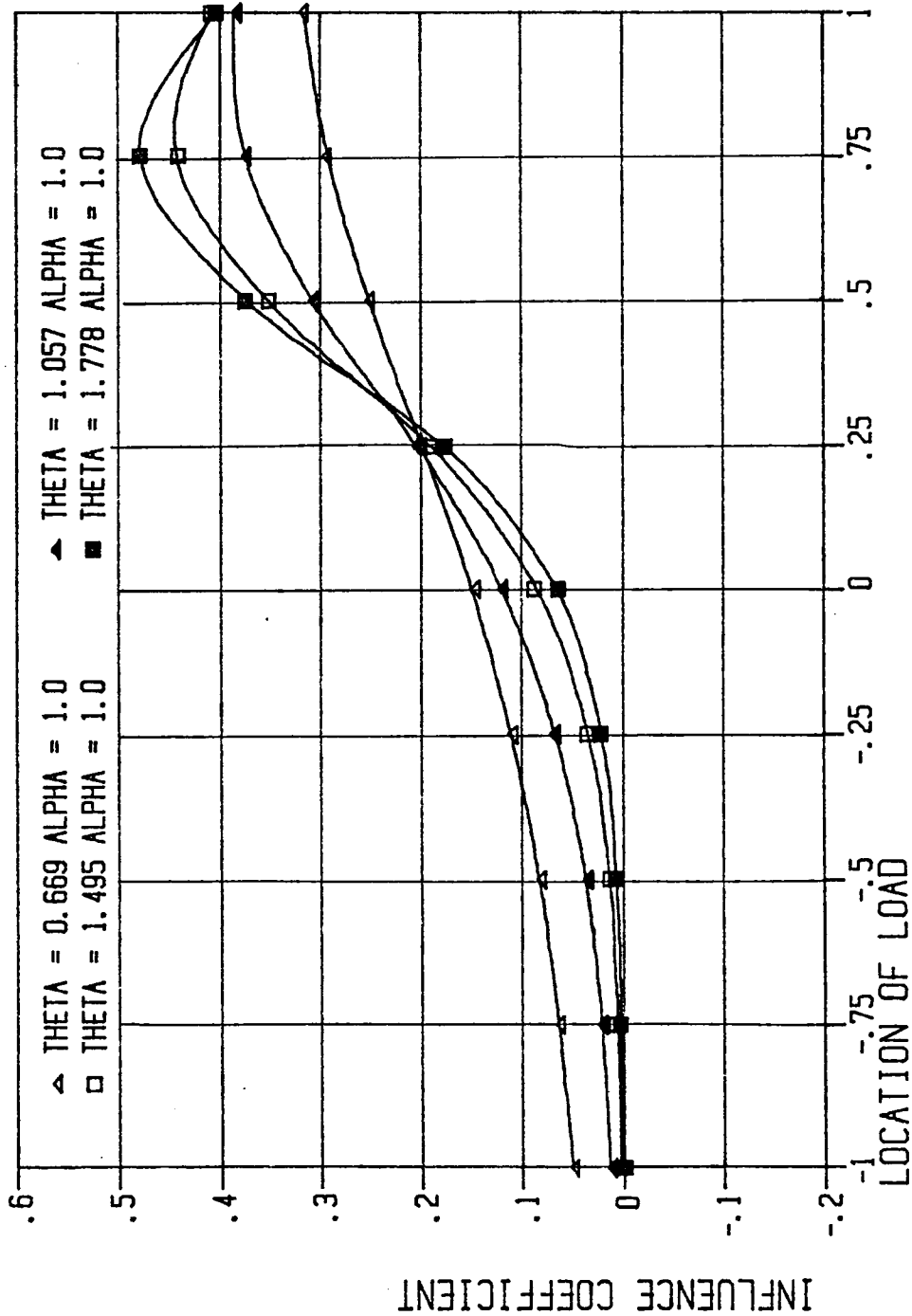


Figure A-26. Seven Girder Bridge (Girder @ 0.67) - $\alpha = 1.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 1.0)

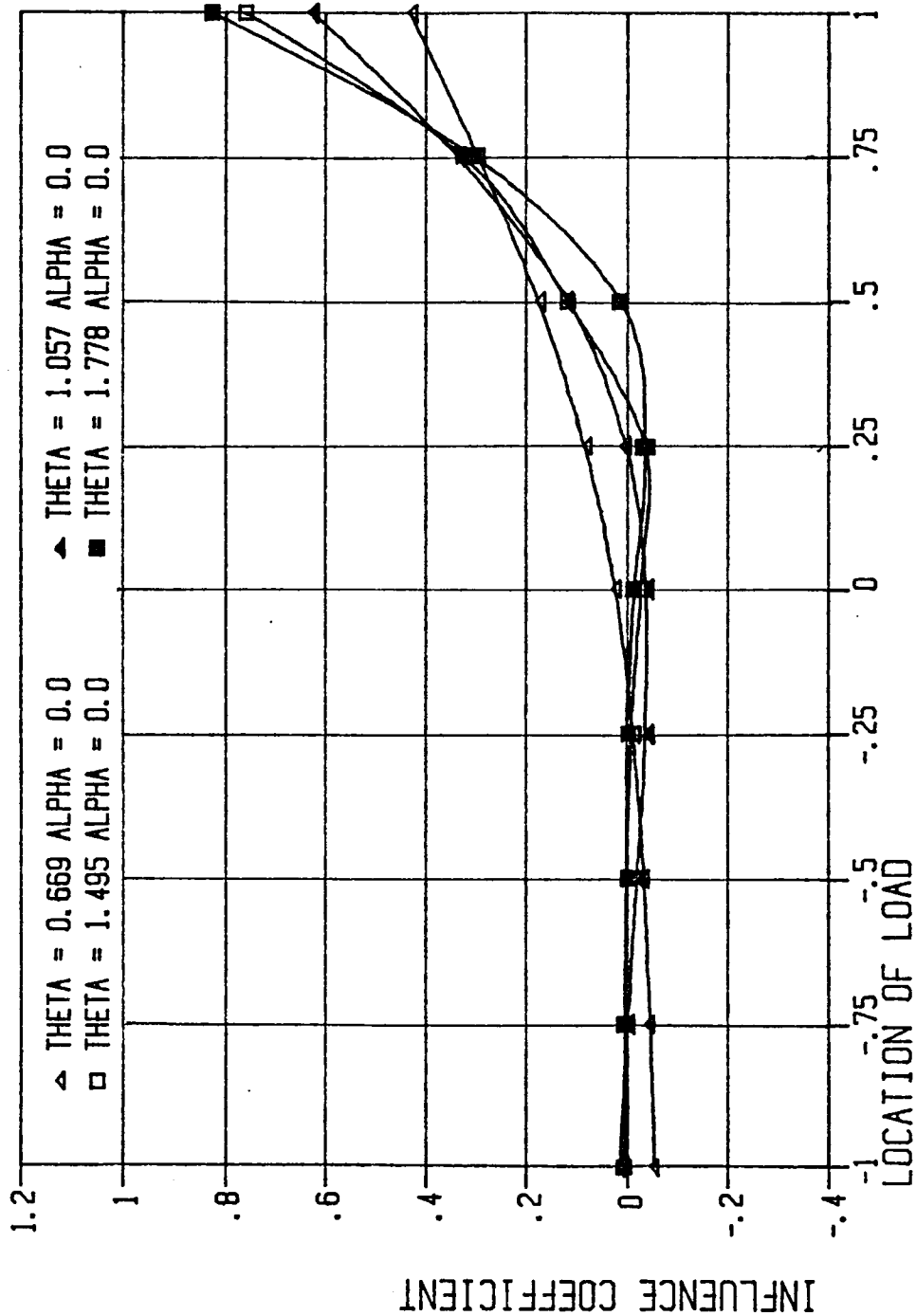


Figure A-27. Seven Girder Bridge (Girder @ 1.0) - $\alpha = 0.0$.

SEVEN GIRDER BRIDGE (GIRDER @ 1.0)

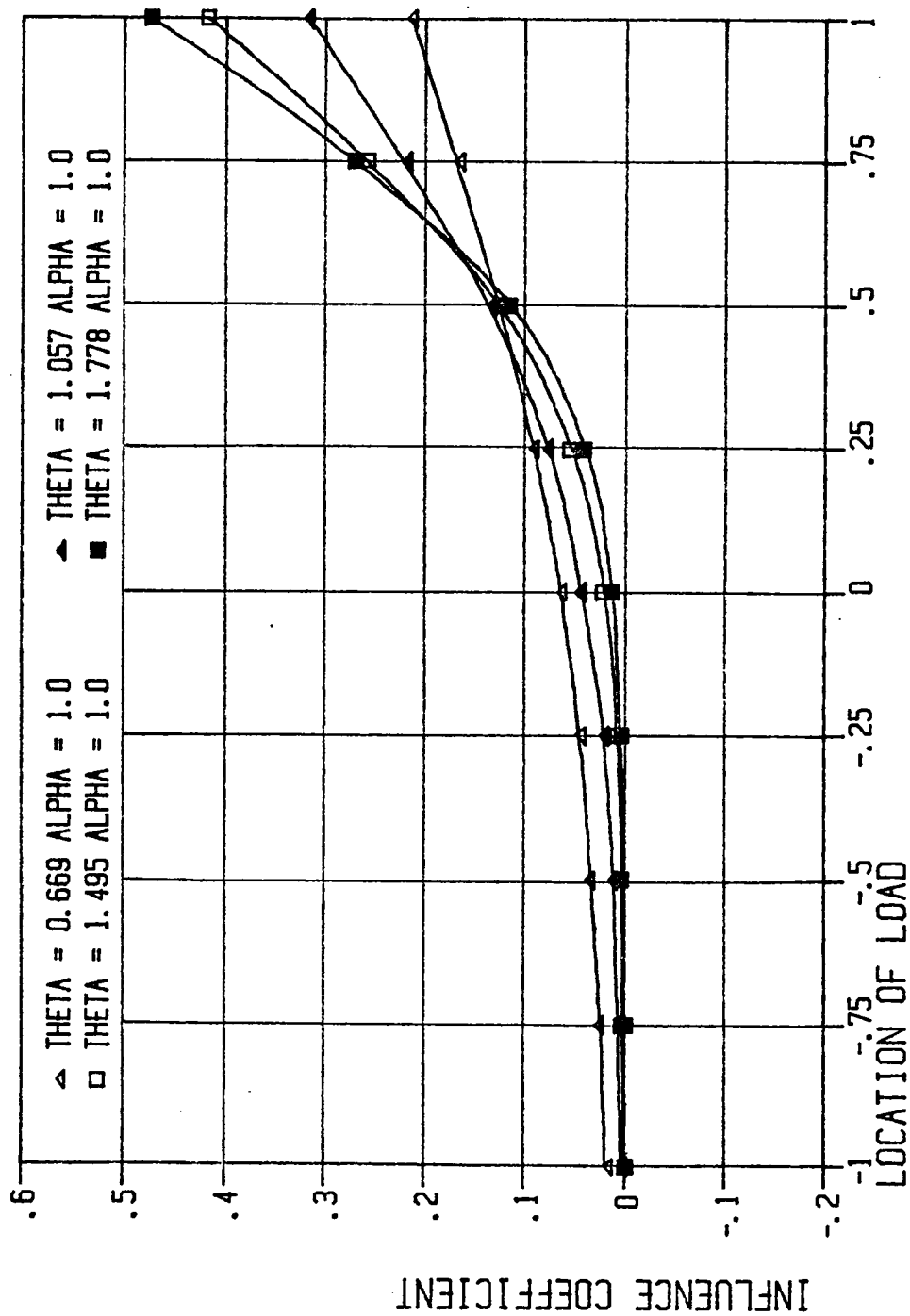


Figure A-28. Seven Girder Bridge (Girder @ 1.0) - $\alpha = 1.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 0.143)

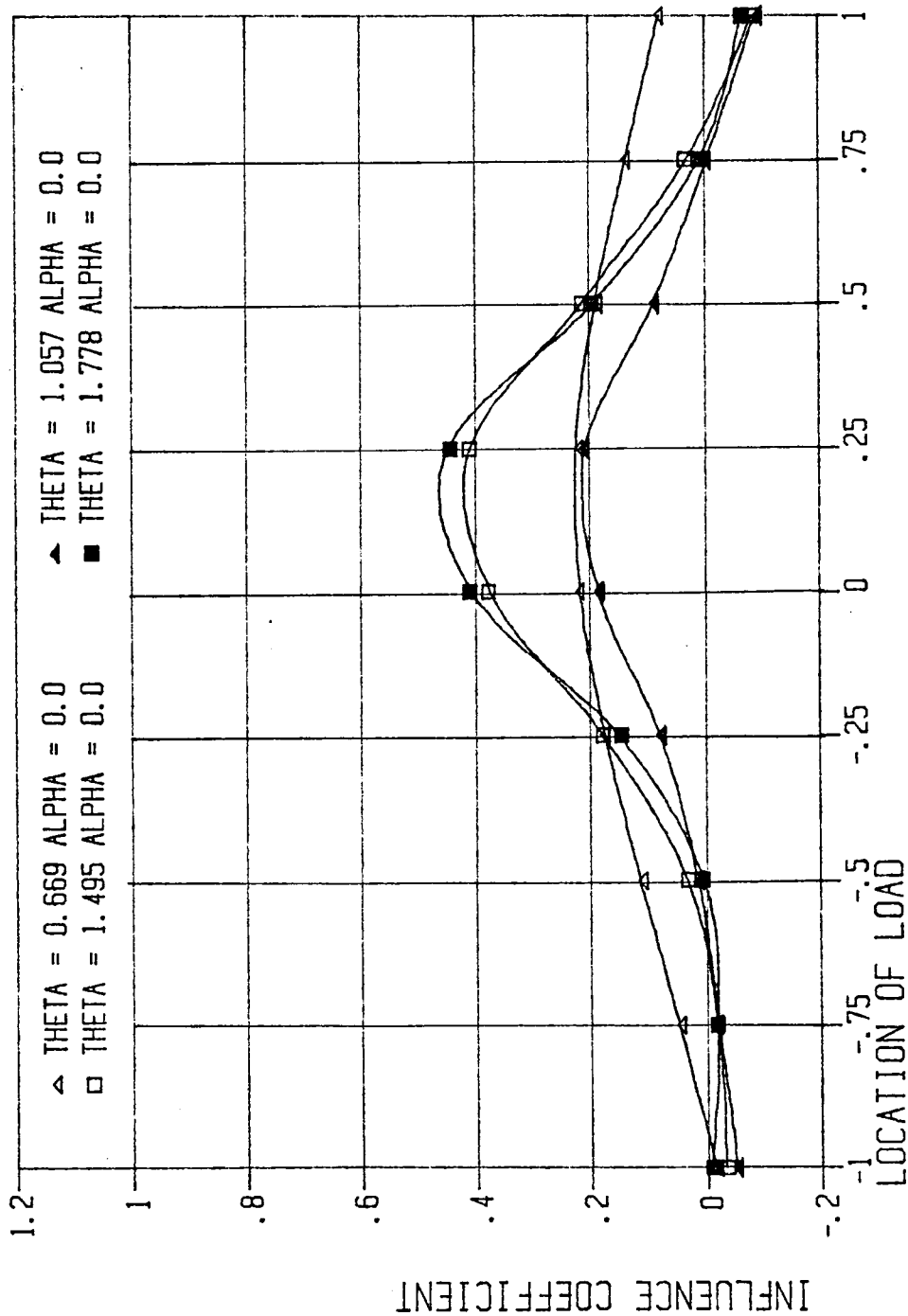


Figure A-29. Eight Girder Bridge (Girder @ 0.143) - $\alpha = 0.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 0.143)

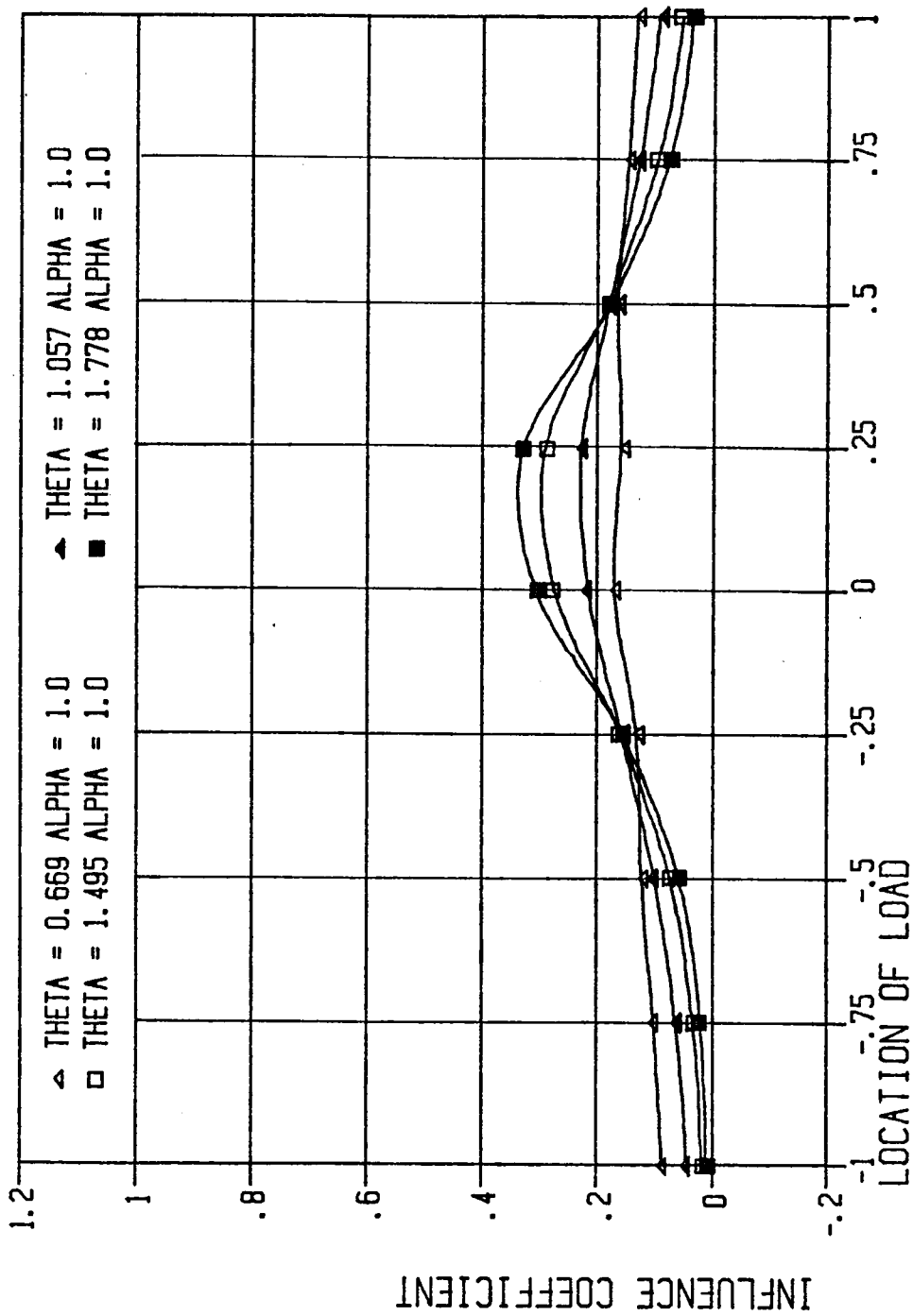


Figure A-30. Eight Girder Bridge (Girder @ 0.143) - $\alpha = 1.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 0.428)

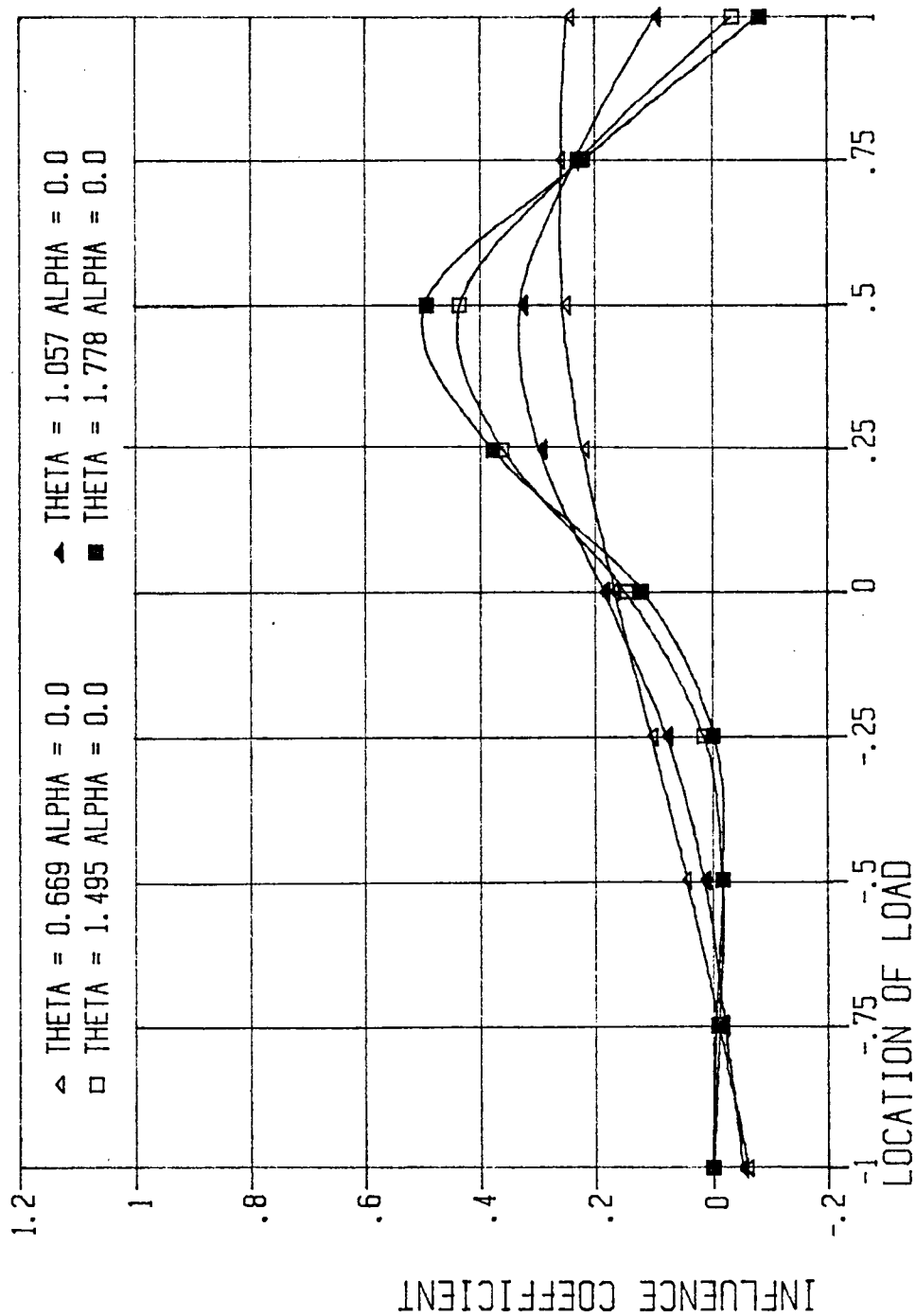


Figure A-31. Eight Girder Bridge (Girder @ 0.428) - $\alpha = 0.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 0.428)

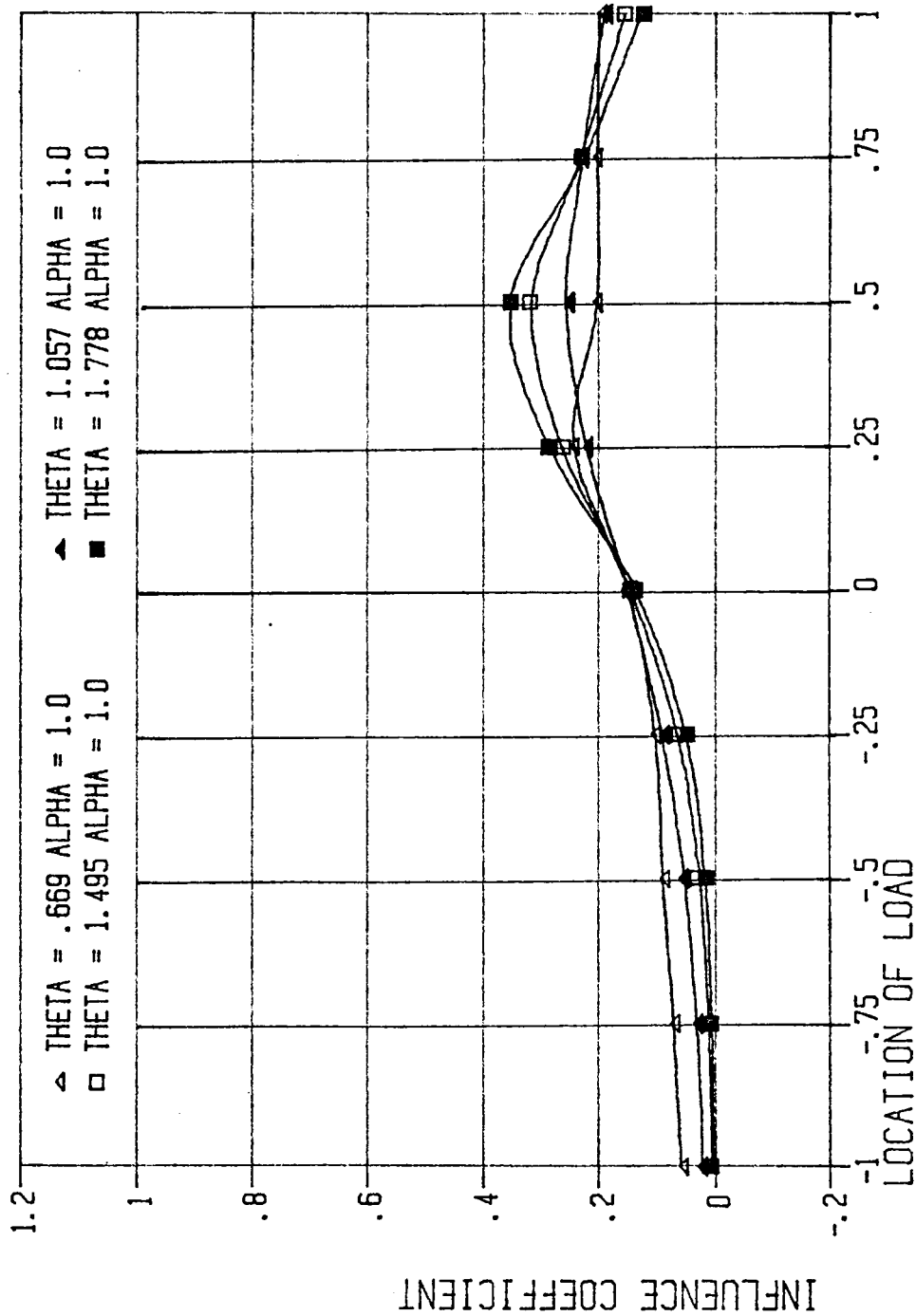


Figure A-32. Eight Girder Bridge (Girder @ 0.428) - $\alpha = 1.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 0.714)

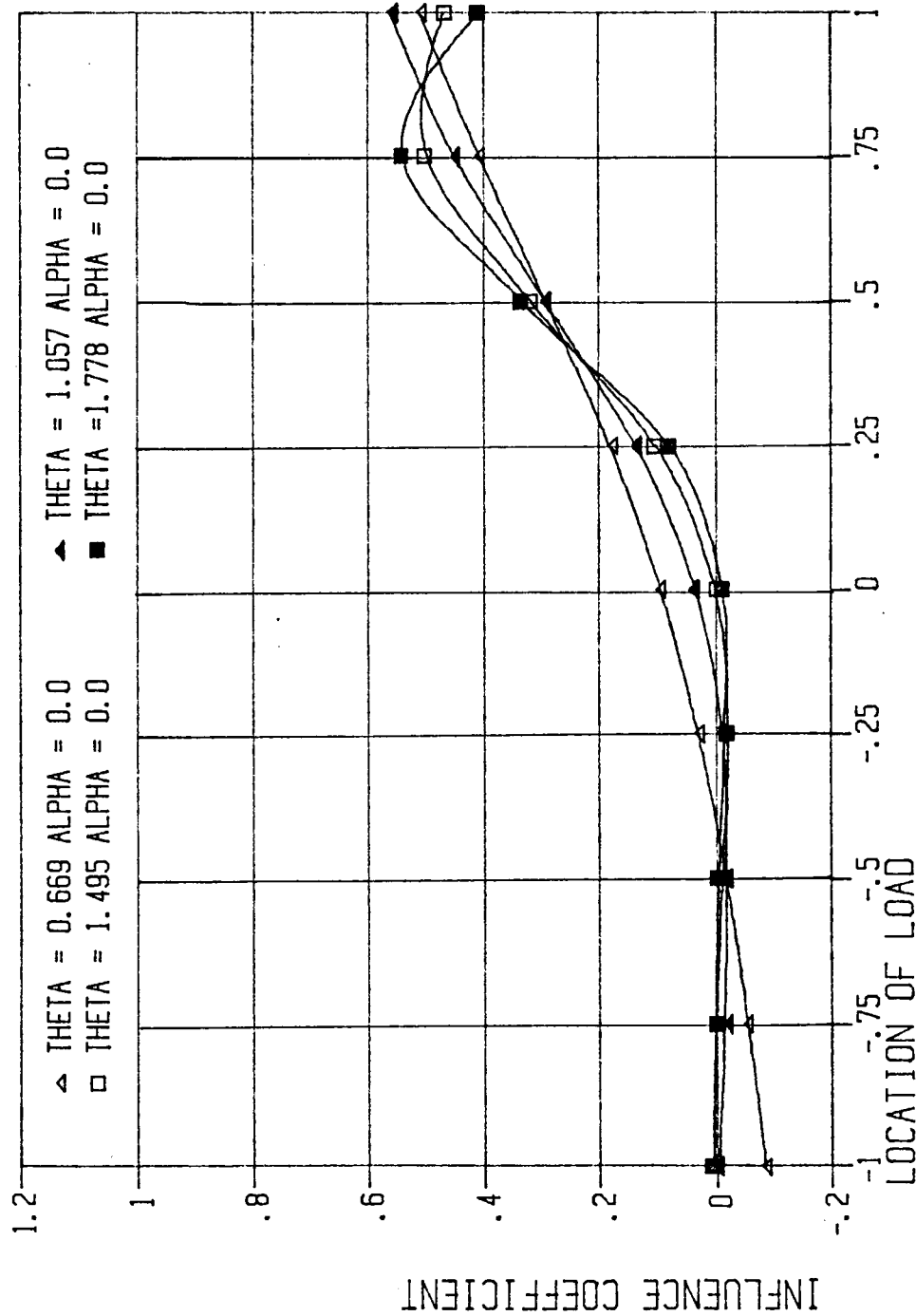


Figure A-33. Eight Girder Bridge (Girder @ 0.714) - $\alpha = 0.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 0.714)

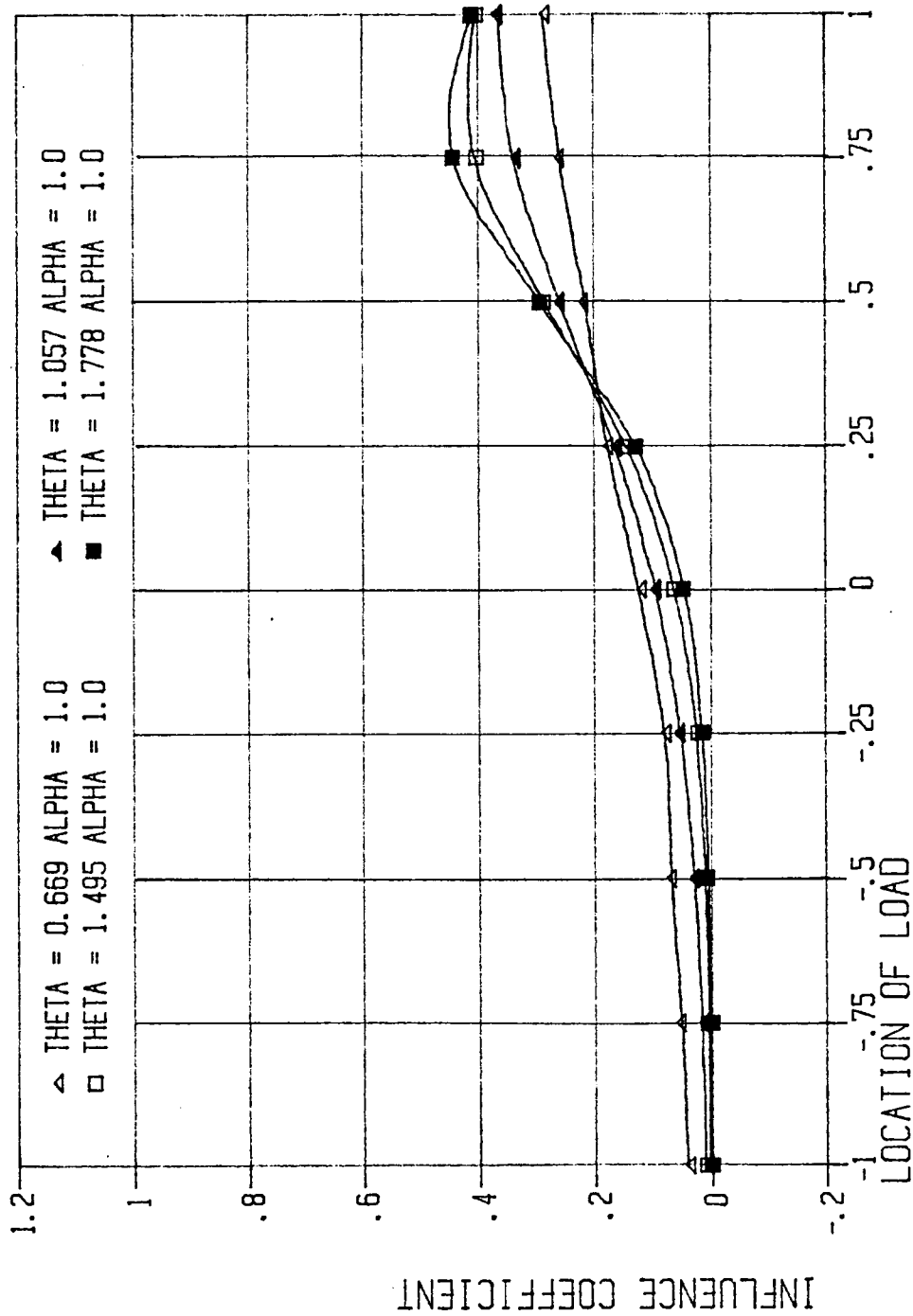


Figure A-34. Eight Girder Bridge (Girder @ 0.714) - $\alpha = 1.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 1.0)

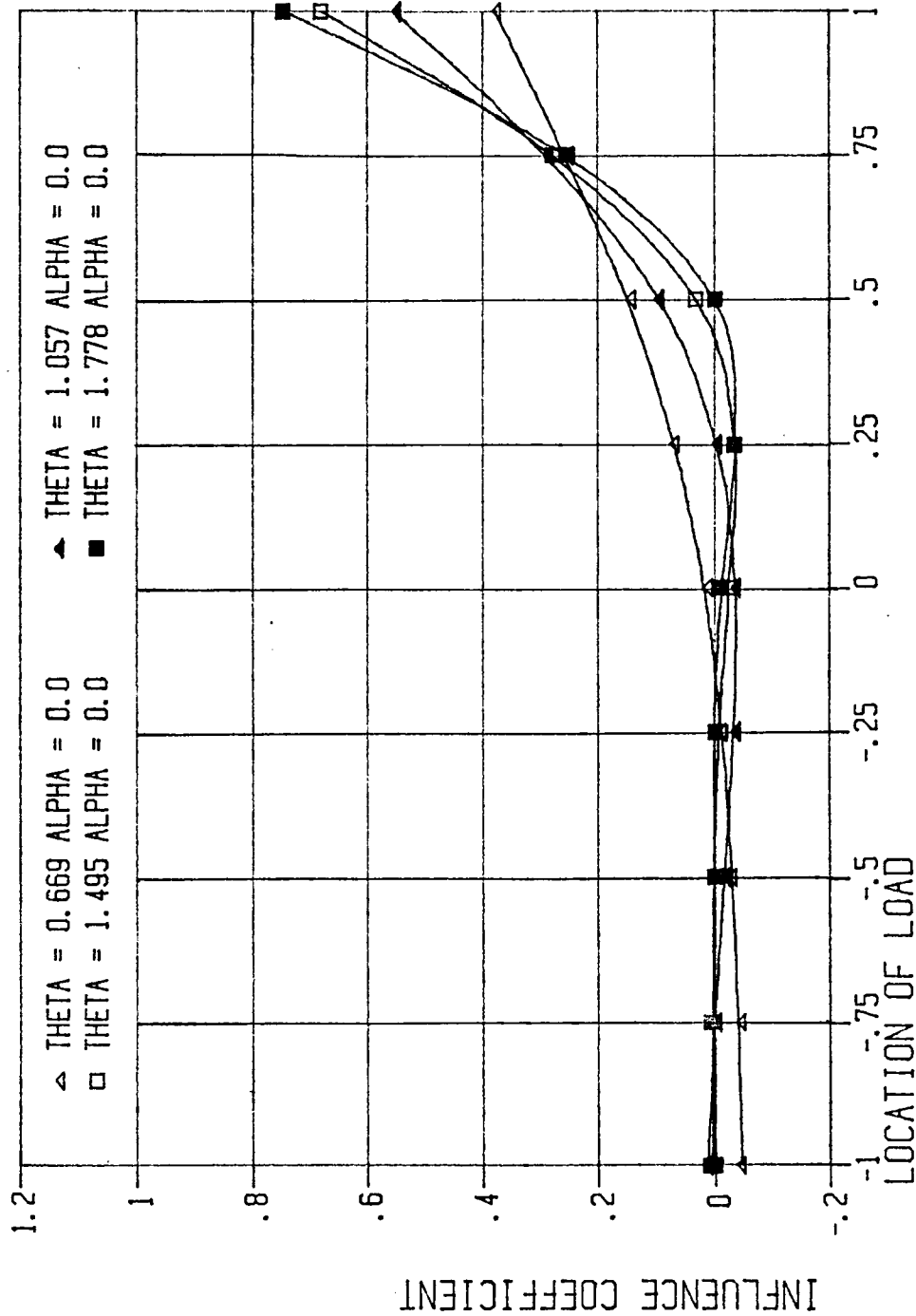


Figure A-35. Eight Girder Bridge (Girder @ 1.0) - $\alpha = 0.0$.

EIGHT GIRDER BRIDGE (GIRDER @ 1.0)

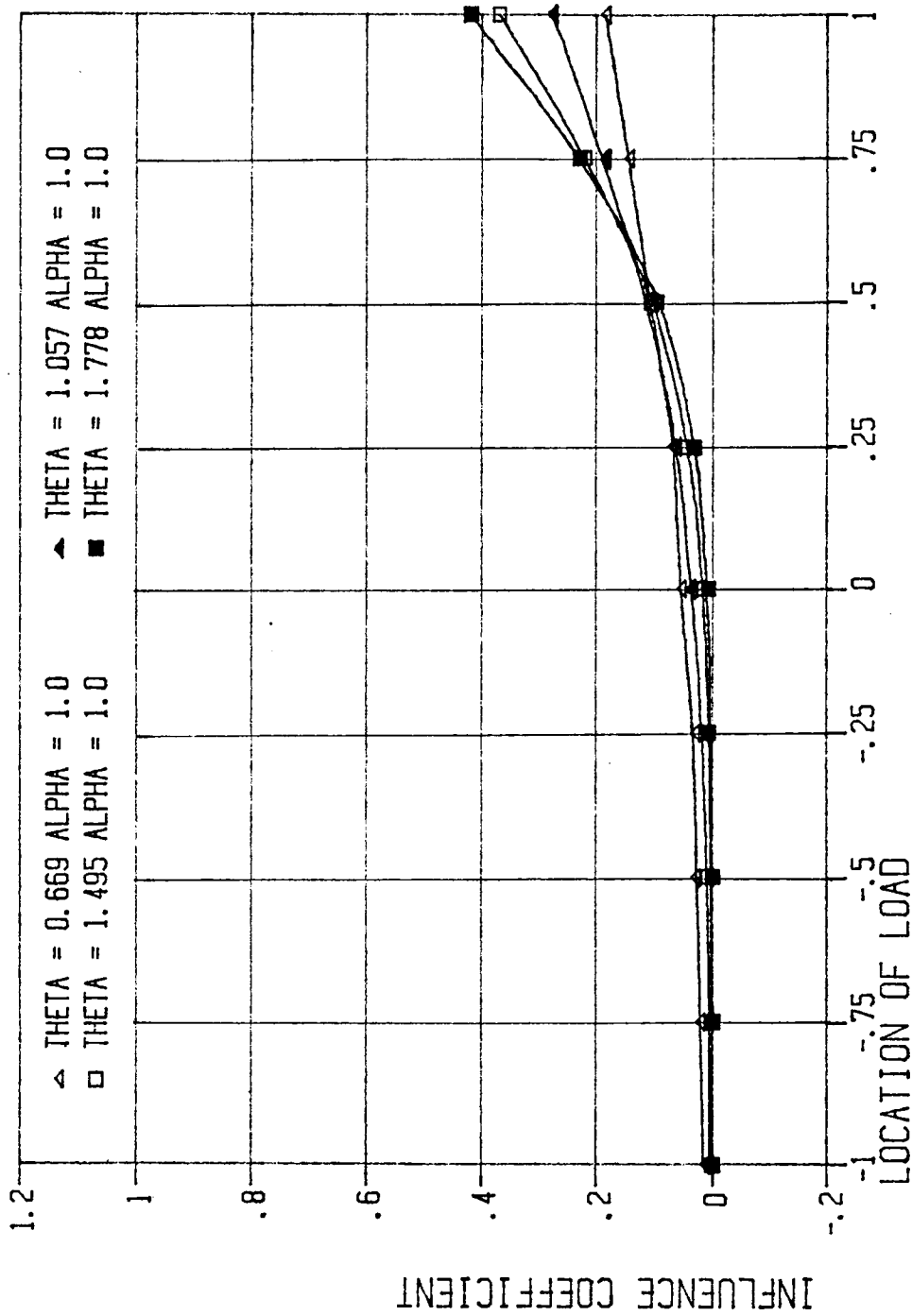


Figure A-36. Eight Girder Bridge (Girder @ 1.0) - $\alpha = 1.0$.

NINE GIRDER BRIDGE (GIRDER @ 0.0)

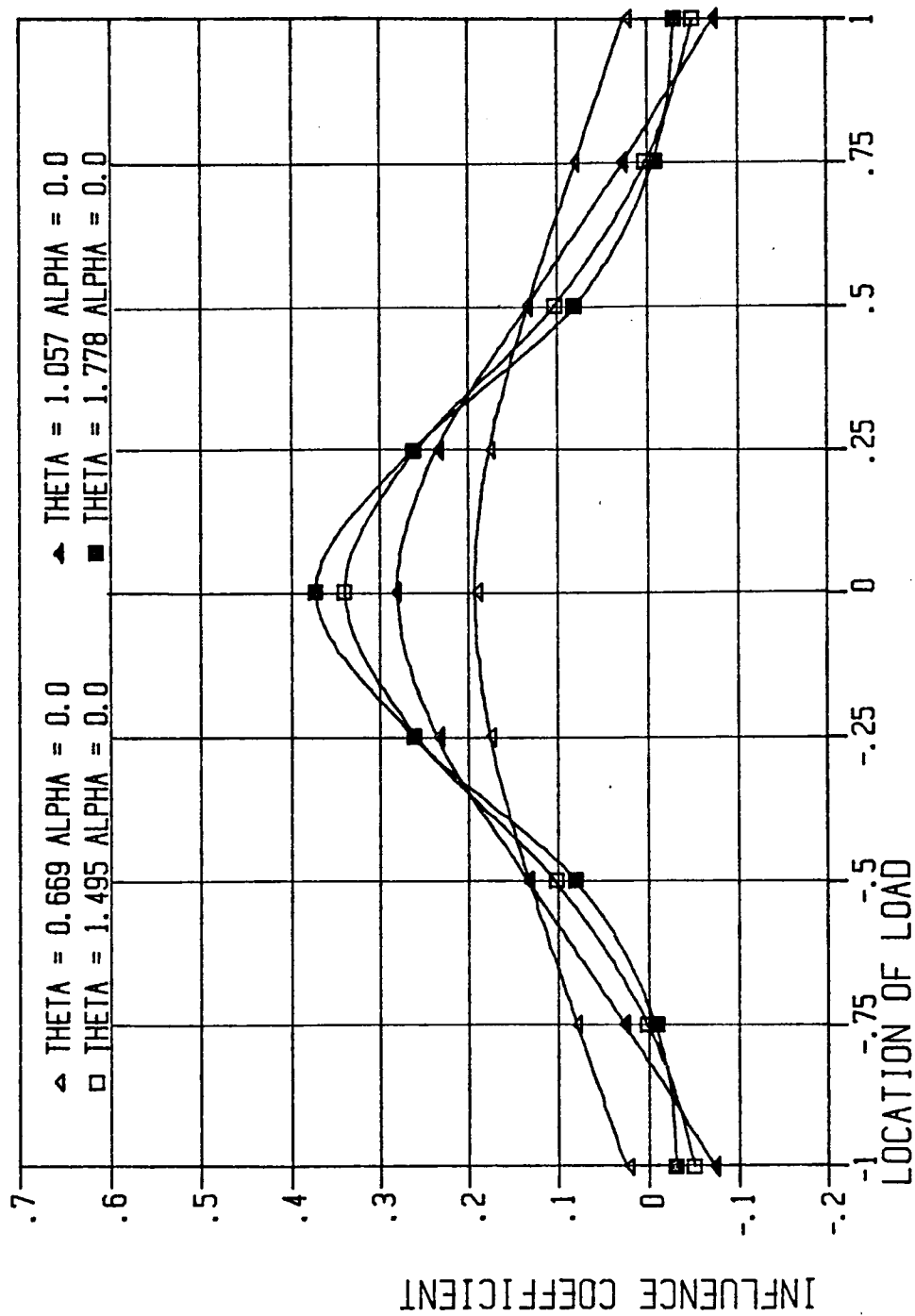


Figure A-37. Nine Girder Bridge (Girder @ 0.0) - $\alpha = 0.0$.

NINE GIRDER BRIDGE (GIRDER @ 0.0)

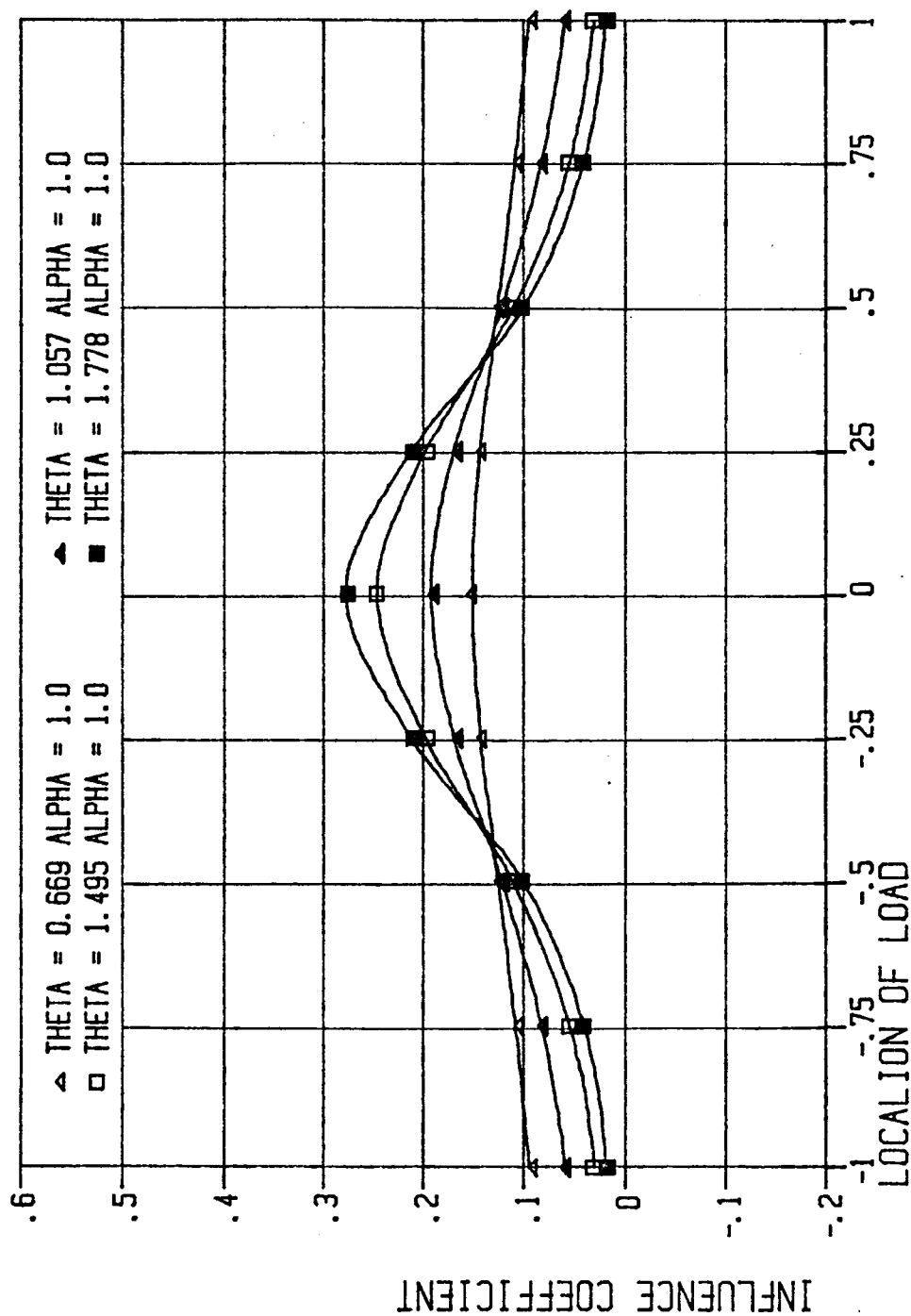


Figure A-38. Nine Girder Bridge (Girder @ 0.0) - $\alpha = 1.0$.

NINE GIRDER BRIDGE (GIRDER @ 0.25)

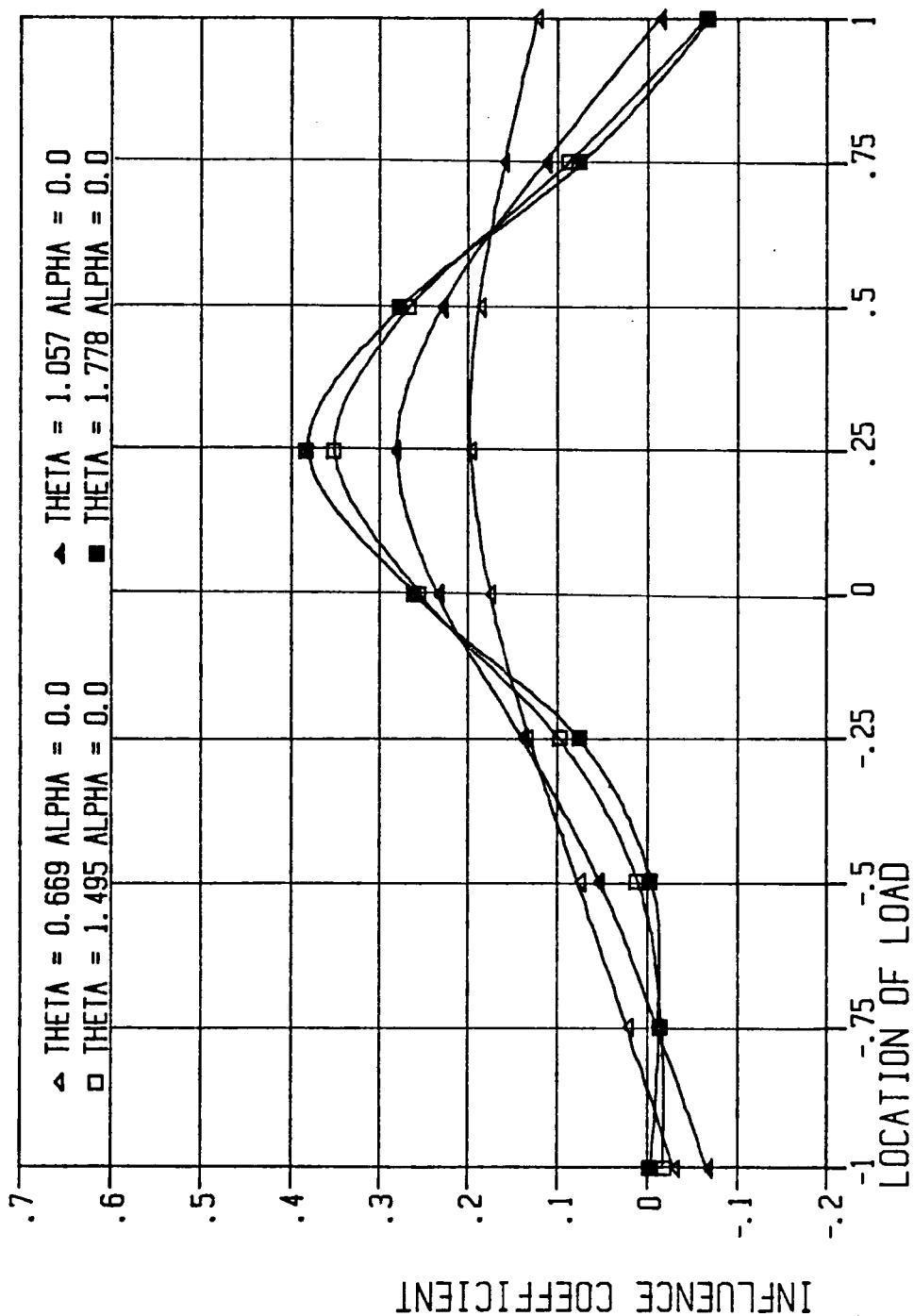


Figure A-39. Nine Girder Bridge (Girder @ 0.25) - $\alpha = 0.0$.

NINE GIRDER BRIDGE (GIRDER @ 0.25)

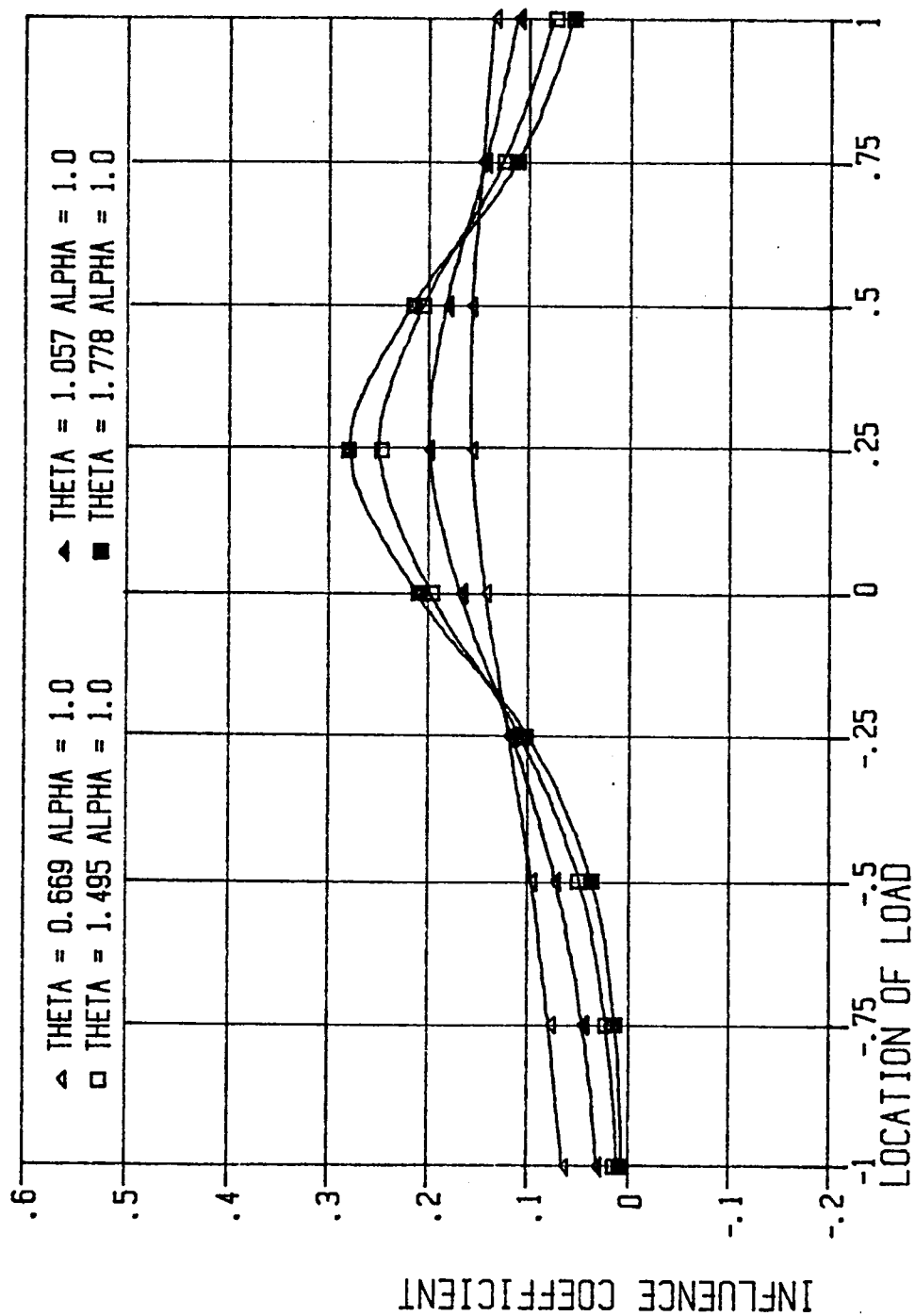


Figure A-40. Nine Girder Bridge (Girder @ 0.25) - $\alpha = 1.0$.

NINE GIRDER BRIDGE (GIRDER @ 0.5)

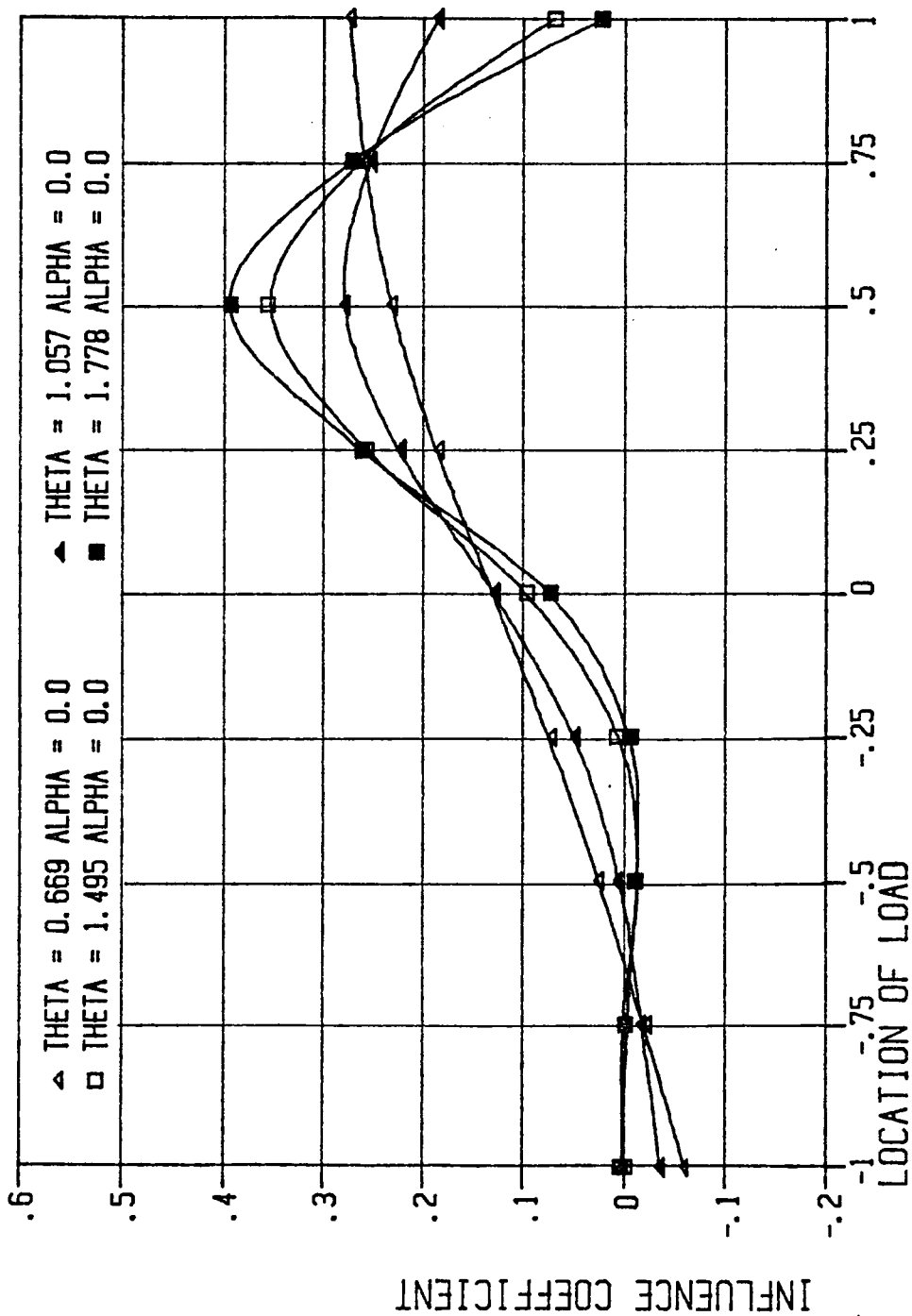


Figure A-41. Nine Girder Bridge (Girder @ 0.5) - $\alpha = 0.0$.

NINE GIRDER BRIDGE (GIRDER @ 0.5)

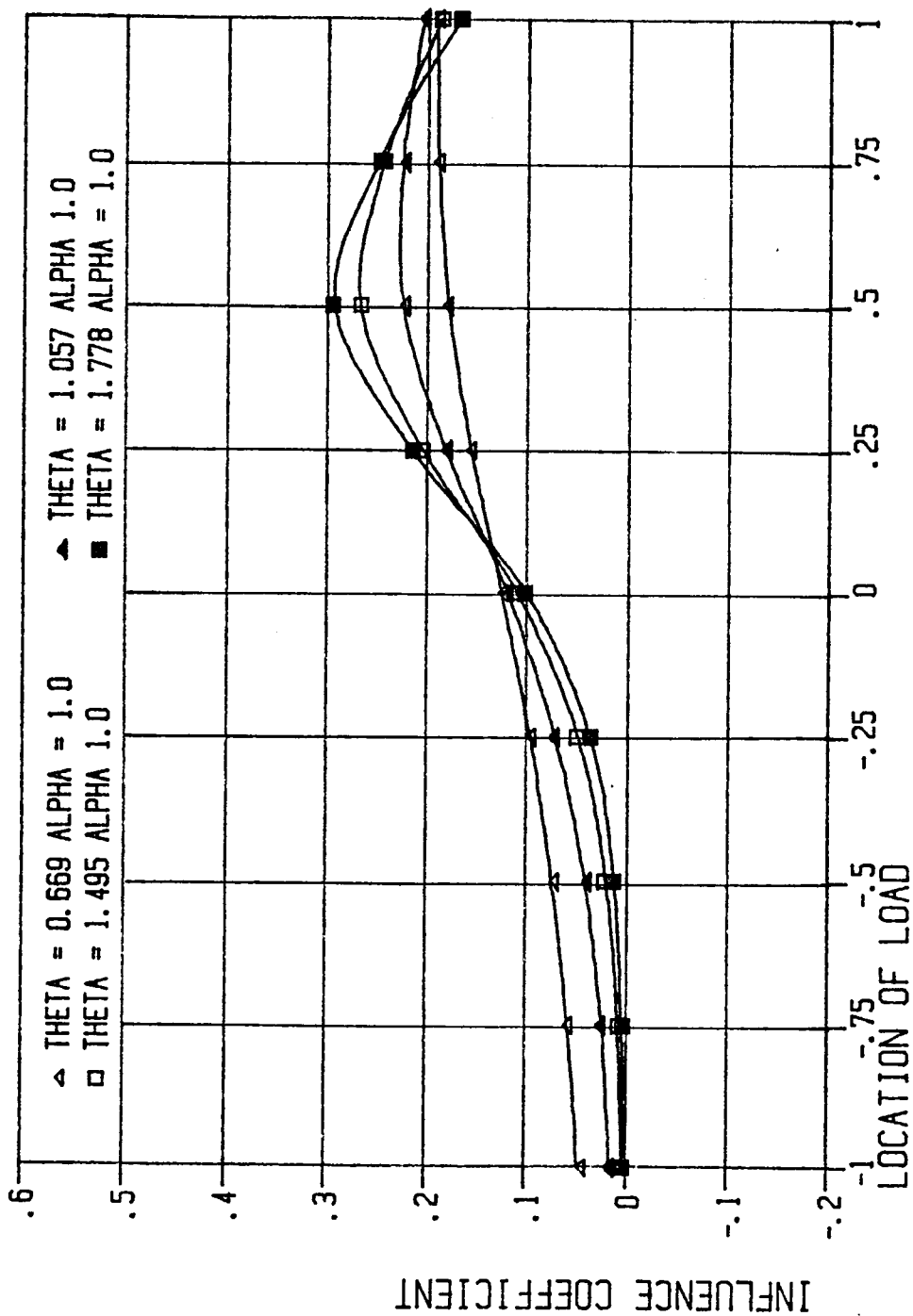


Figure A-42. Nine Girder Bridge (Girder @ 0.5) - $\alpha = 1.0$.

NINE GIRDER BRIDGE (GIRDER @ .75)

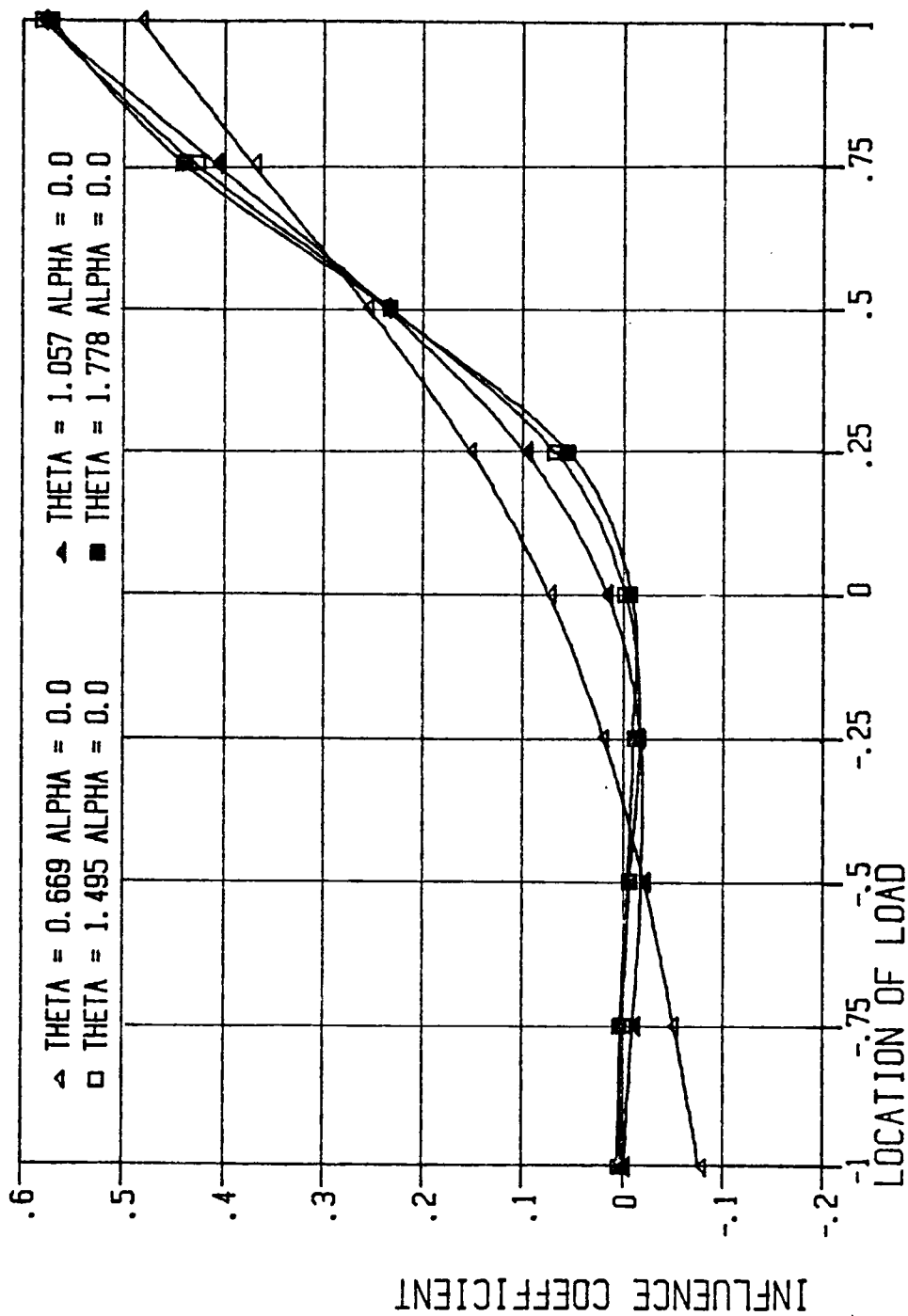


Figure A-43. Nine Girder Bridge (Girder @ 0.75) - $\alpha = 0.0$.

NINE GIRDER BRIDGE (GIRDER @ 0.75)

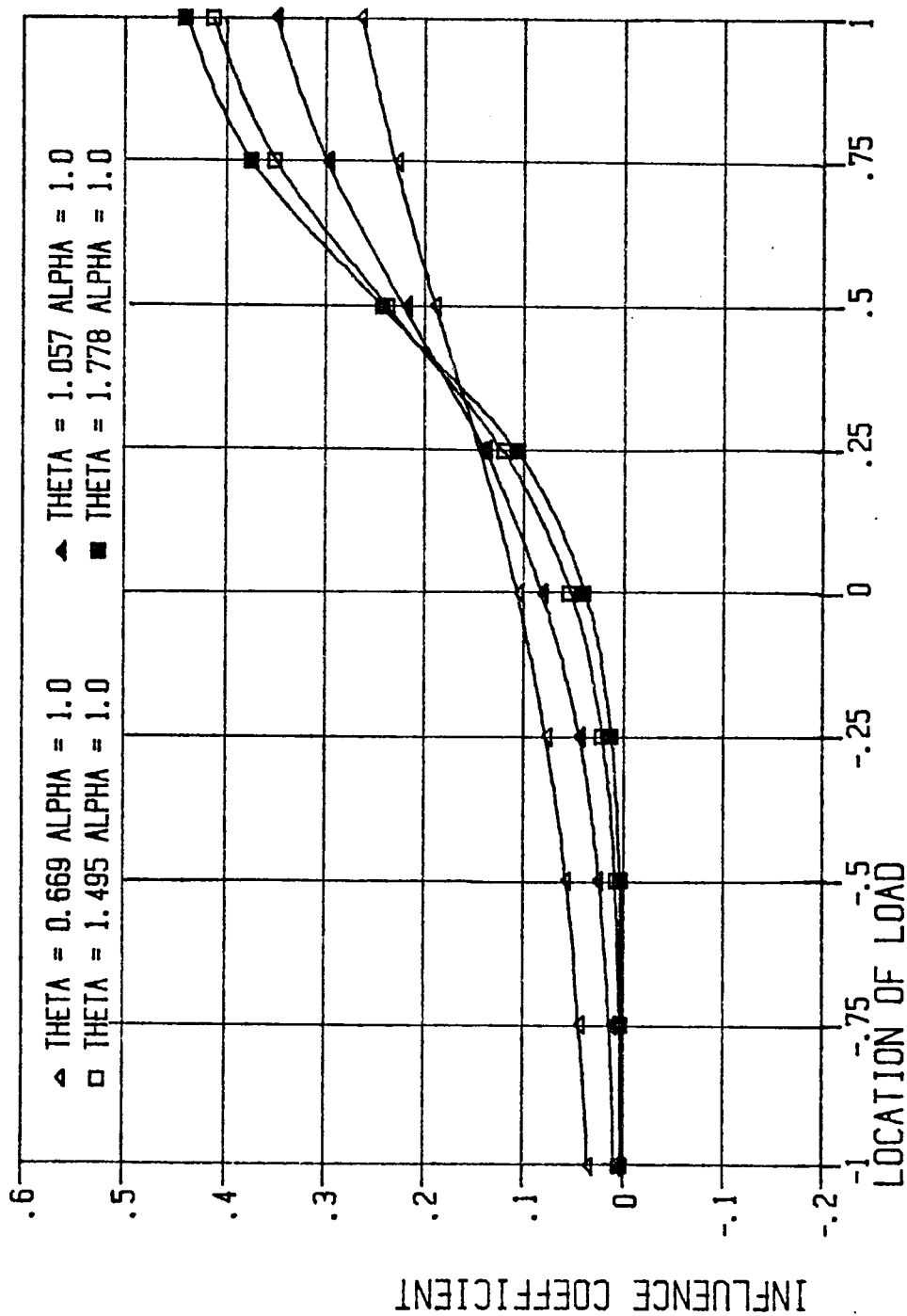


Figure A-44. Nine Girder Bridge (Girder @ 0.75) - $\alpha = 1.0$.

NINE GIRDER BRIDGE (GIRDER @ 1.0)

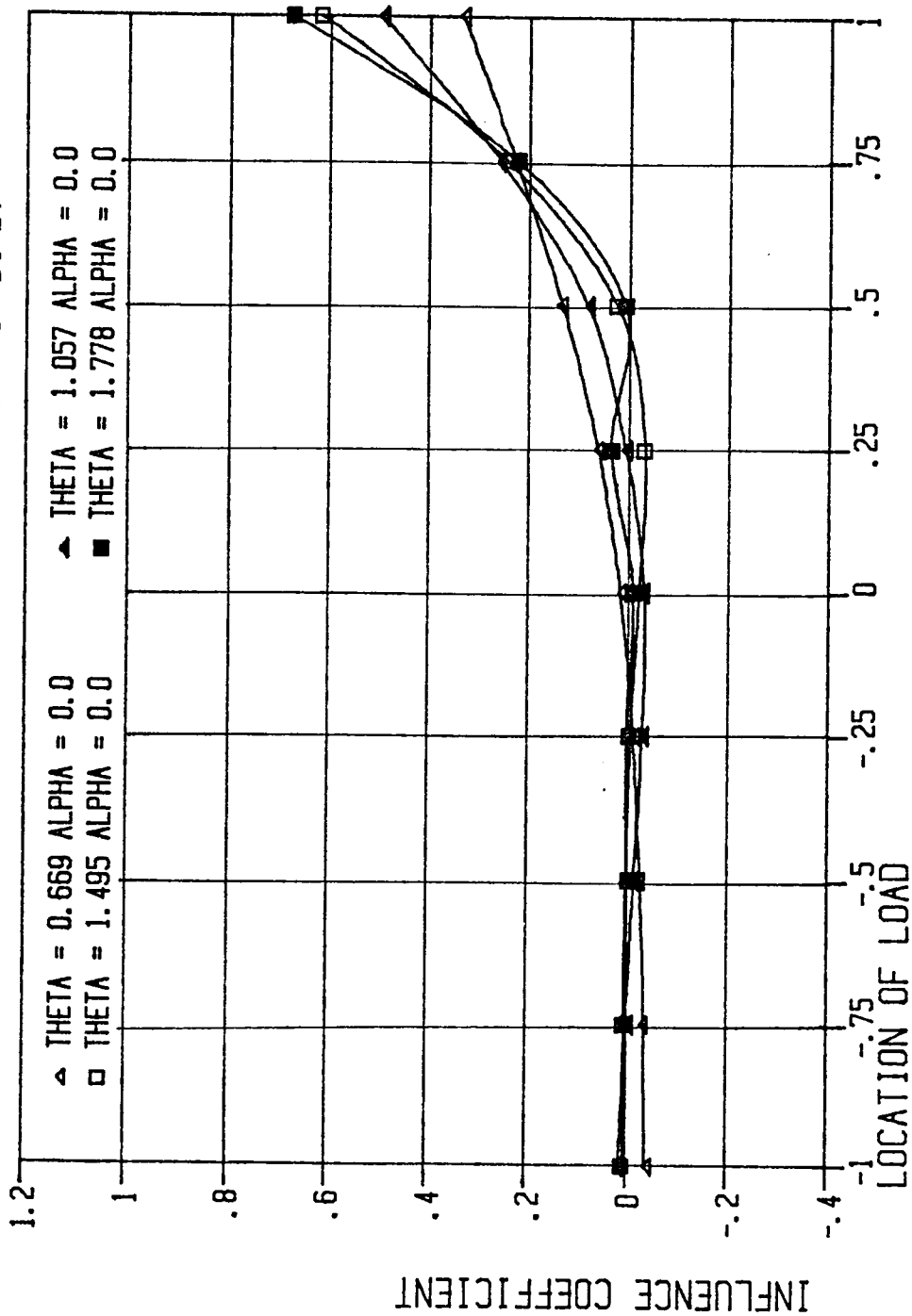


Figure A-45. Nine Girder Bridge (Girder @ 1.0) - $\alpha = 0.0$.

NINE GIRDER BRIDGE (GIRDER @ 1.0)

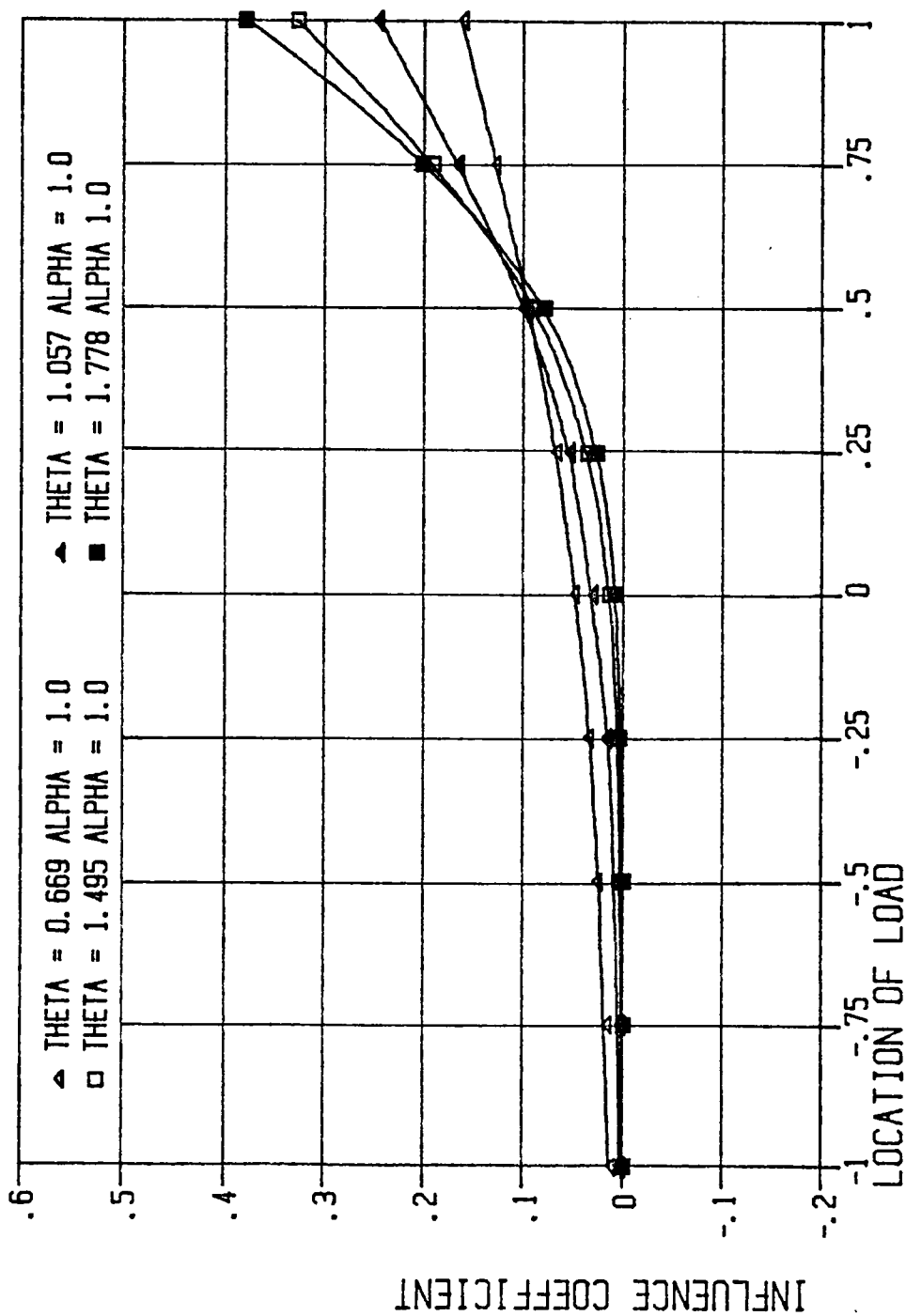


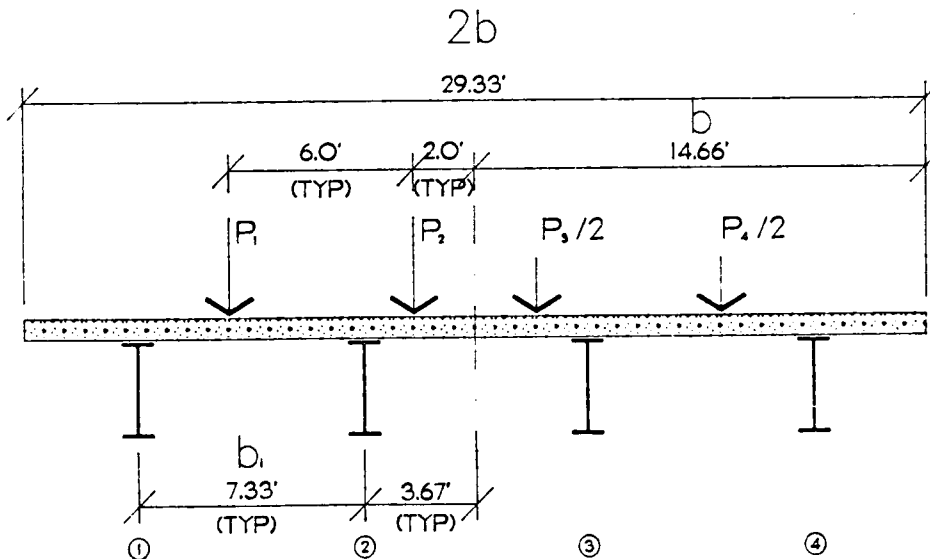
Figure A-46. Nine Girder Bridge (Girder @ 1.0) - $\alpha = 1.0$.

APPENDIX B

SAMPLE CALCULATIONS

B - 1 CALCULATE LOAD DISTRIBUTIONS FOR TENNESSEE BRIDGE 4.

The applied loads on Tn. Bridge 4 are shown below. Because of the way the loads were applied, $P_1 = P_2 = P_3 = P_4$.



CROSS SECTION - TN BRIDGE 4

GEOMETRIC PROPERTIES:

Length of span: $L = 60$ feet

Deck Thickness: $h = 7$ inches

Girder Size: W27 x 102 (interior girder)

W27 x 94 (exterior girder)

note: (1) girders behave compositely w/deck.

(2) interior and exterior girder stiffness assumed equal.

Moment of inertia of composite section:

$$I_G = 10537 \text{ in}^4 \text{ (Ref 114)}$$

Moment of inertia of unit width of deck: I_D

$$I_D = \frac{12 (7)^3}{12} = 343 \text{ in}^4$$

Girder spacing:

$$b_1 = 7.33' = 88''$$

Width of equivalent bridge: $2b$ (see figure 3)

$$b = \frac{29.33}{2} = 14.66'$$

Polar moment of inertia of composite girder: J_G

$$J_G = \Sigma J(\text{BEAM}) + J(\text{DECK} - \text{TRANSFORMED})$$

$$J(\text{BEAM}) = 5.27 \text{ in}^4 \quad (\text{REF AISC})$$

$$J(\text{TRANSFORMED DECK}) =$$

$$(b_t - 0.63h) \frac{h^3}{3}$$

$$b_t = \frac{b_1}{n} = \frac{88}{6.8} = 12.94$$

Note: n is defined below

$$J(\text{TRANSFORMED DECK}) =$$

$$\left[12.94 - (0.63)(7) \right] \left(\frac{7^3}{3} \right)$$

$$= 975 \text{ in}^4$$

$$J_G = 975.0 + 5.27 = 980.3 \text{ in}^4$$

PHYSICAL PROPERTIES:

Modulus of elasticity of beam: $E_s = 29000 \text{ ksi}$

Compressive strength of concrete: $f_c' = 5600 \text{ psi}$

Modulus of elasticity of concrete:

$$E_c = 57 (f_c')^{\frac{1}{2}} \text{ (ksi)}$$

$$= 57 (5600)^{\frac{1}{2}} = 4265 \text{ ksi}$$

$$n = \frac{E_S}{E_C} = \frac{29000}{4265} = 6.8$$

CALCULATE θ :

$$B_G = E_G I_G = 29(10^6) (10537) = 305636 (10^6)$$

$$B_D = E_D I_D = 4.265(10^6) (343) = 1463 (10^6)$$

$$\rho_G = \frac{B_G}{b_1} = = \frac{305636 (10^6)}{88} = 3473 (10^6)$$

$$\rho_D = \frac{B_D}{W} \text{ where } W = \text{unit width} = 12''$$

$$\rho_b = \frac{1463 (10^6)}{12} = 122 (10^6)$$

$$\theta = \frac{b}{L} \sqrt[4]{\rho_G \rho_D} = \frac{14.66}{60} \sqrt[4]{3473 (10^6) / 122 (10^6)}$$

$$\theta = 0.564$$

CALCULATE α :

$$C_G = J_G G_G$$

$$G_G = \frac{E}{2(1 + \nu)} = \frac{29 (10^6)}{2(1 + 0.3)} = 11.15 (10^6) \text{ psi}$$

$$C_G = (980.3) (11.15) (10^6) = 10930 (10^6)$$

$$Y_G = \frac{C_G}{b_1} = \frac{10930 (10^6)}{88} = 124.2 (10^6)$$

$$C_D = J_D G_D$$

$$G_D = \frac{E_C}{2} = \frac{4.265 (10^6)}{2} = 2.13 (10^6) \text{ psi}$$

$$D_D = 868 (2.13) (10^6) = 1849 (10^6)$$

$$Y_D = \frac{C_D}{W} = \frac{1849 (10^6)}{12} = 154 (10)^6$$

$$\alpha = \frac{Y_G + Y_D}{2 Y \rho_G \rho_D} = \frac{124.2 + 154 (10^6)}{2 \sqrt{3473 (10^6) 122 (10^6)}} = 0.213$$

DEVELOP LOCATION OF LOADS: d_n/b

Where: d_n is distance from center line of bridge to applied load.

Location of Load	
P_1	0.54
P_2	0.13
P_3	-0.13
P_4	-0.54

DETERMINE THE DISTRIBUTION OF EACH LOAD TO INDIVIDUAL GIRDERS:

From the influence curves for a four girder bridge

$\theta = 0.564$ Use data for $\theta = 0.669$

$\alpha = 1$ $\theta = 0.669$

Girder	kP_1	kP_2	kP_3	kP_4	Σk
1	0.26	0.15	0.06	0.04	0.51
2	0.43	0.43	0.15	0.12	1.13
3	0.24	0.30	0.21	0.21	0.96
4	0.07	0.12	0.08	0.13	0.40
Σ	1.0	1.0	0.5	0.5	3.0

$$\alpha = 0 \quad \theta = 0.669$$

Girder	kP ₁	kP ₂	kP ₃	kP ₄	Σk
1	0.37	0.13	0.03	-0.03	0.50
2	0.55	0.46	0.18	0.08	1.27
3	0.15	0.36	0.23	0.27	0.01
4	-0.07	0.05	0.06	0.18	0.22
Σ	1.0	1.0	0.5	0.5	3.0

USE INTERPOLATION FORMULA TO DETERMINE DISTRIBUTION
FOR $\alpha = 0.213$

$$K_{\alpha} = K_0 + (K_1 - K_0) \sqrt{\alpha}$$

Example calculation for girder 1, distribution kP₁

$$K_0 = 0.37$$

$$K_1 = 0.26$$

$$\alpha = 0.213$$

$$K_{0.213} = 0.37 + (0.26 - 0.37) \sqrt{0.213}$$

$$K_{0.213} = 0.32$$

Thus, final distribution as shown in table VII is as follows:

**TENNESSEE BRIDGE 4-LOAD DISTRIBUTION TO VARIOUS
GIRDERS $\alpha = 0.213$**

Girder	kP ₁	kP ₂	kP ₃	kP ₄	Σk
1	0.32	0.14	0.04	0.0	0.5
2	0.49	0.44	0.16	0.10	1.19
3	0.19	0.33	0.22	0.24	0.98
4	0.00	0.09	0.08	0.16	0.33
Σ	1.0	1.0	0.5	0.5	3.0

From Ozbek's (114) calculations based on a maximum distribution to girder 2 of 1.3 a yield load can be predicted as:

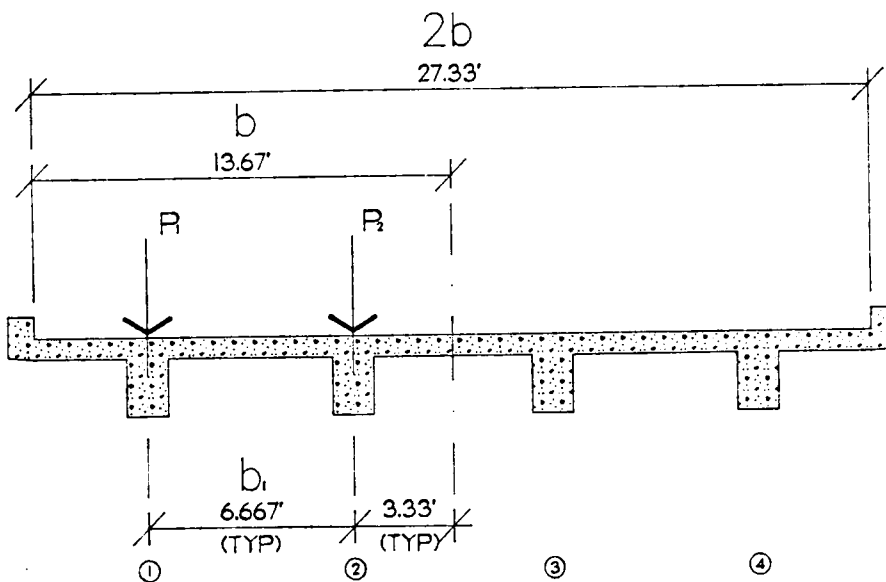
$$\frac{1.3}{1.19} (462) = 505 \text{ KIPS} \quad (\text{See Table XI for Comparisons})$$

SAMPLE CALCULATIONS

B-2 CALCULATE LOAD DISTRIBUTIONS FOR KARAKA CREEK BRIDGE (NEW ZEALAND)

The applied loads on Karaka Creek Bridge are shown below.

Note: $P_1 = P_2$



CROSS SECTION - KARAKA CREEK BRIDGE

GEOMETRIC PROPERTIES:

Length of span: $L = 30$ feet

Deck thickness: $h = 8$ inches

Moment of inertia (interior girder): 23020 in^4

Moment of inertia (exterior girder) = 20006 in^4

$$\text{Assume } I_G = \frac{23020 + 20006}{2} = 21513 \text{ in}^4$$

Moment of inertia of unit width of deck: I_D

$$I_D = \frac{12 (8)^3}{12} = 512 \text{ in}^4$$

Girder Spacing: $b_1 = 6.67$ feet or 80 inches width of equivalent bridge: $2b$

$$b = \frac{27.17}{2} = 13.58 \text{ feet}$$

Polar moment of inertia of girder: J_G

$$J_G = \Sigma (b - 0.63h) \frac{h^3}{3}$$

$$= [80 - (0.63)(8)] \frac{8^3}{3} + [20 - 0.63(15)] \frac{15^3}{3}$$

$$= 12793 + 11869 = 24661 \text{ in}^4$$

J_G (cracked section) $\approx 0.15 J_G$ (uncracked)

Ref: Park & Pauley "Reinforced Concrete Structures"

$$J_G = (0.15) (24661) = 3699 \text{ in}^4$$

PHYSICAL PROPERTIES:

$$E_C = 4.65 (10)^6$$

$$G_C = \frac{4.65}{2} (10^6) = 2.32 (10^6)$$

CALCULATE θ :

$$B_G = E_G I_G = 4.65 (10^6) 21513$$

$$\rho_G = \frac{B_G}{b_1} = \frac{4.65 (10^6) 21513}{80} = 1235 (10^6)$$

$$B_D = E_D I_D = 4.65 (10^6) (512)$$

$$\rho_D = \frac{B_D}{12} = \frac{4.65 (10^6) (512)}{12} = 198.4 (10^6)$$

$$\theta = \frac{b}{L} {}^4\sqrt{\rho_G / \rho_D}$$

$$= \frac{13.58}{30} {}^4\sqrt{1235 (10^6) / 198.4 (10^6)}$$

$$\theta = 0.715$$

CALCULATE α :

$$Y_G = \frac{GJ_G}{b_1} = \frac{\frac{4.65}{2} (10^6) (3699)}{80} = 107.5 (10^6)$$

$$Y_D = \frac{GJ_D}{12} = \frac{\frac{4.65}{2} (10^6) (1188)}{12} = 230 (10^6)$$

$$\alpha = \frac{Y_G + Y_D}{\sqrt{\rho_G \rho_D}} = \frac{107.5 (10^6) + 230 (10^6)}{2 \sqrt{1235 (198.4) (10)^{12}}}$$

$$\alpha = 0.34$$

DEVELOP LOCATION OF LOADS: dn/b

Where dn is distance from center line of bridge to applied load

Location of Load	
P_1	0.25
P_2	0.75

DETERMINE THE DISTRIBUTION OF EACH LOAD TO INDIVIDUAL GIRDERS.

From the influence curves for a four girder bridge:

$$\theta = 0.715 \quad \text{Use } \theta = 0.669$$

$$\alpha = 1 \quad \theta = 0.669$$

GIRDER	kP ₁	kP ₂	Σk
1	0.32	0.17	0.49
2	0.44	0.47	0.91
3	0.19	0.26	0.45
4	0.05	0.10	0.15
Σ	1.0	1.0	2.0

$$\alpha = 1 \quad \theta = 0.669$$

GIRDER	kP ₁	kP ₂	Σk
1	0.55	0.19	0.74
2	0.53	0.51	1.04
3	0.03	0.31	0.34
4	-0.11	-0.01	-0.12
Σ	1.0	1.0	2.0

USE INTERPOLATION FORMULA TO DETERMINE DISTRIBUTION
FOR $\alpha = 0.34$

$$K_{\alpha} = K_0 + (K_1 - K_0) \sqrt{\alpha}$$

Example calculation for girder 1, distribution for kP₁.

$$K_0 = 0.55$$

$$K_1 = 0.32$$

$$K_{0.34} = 0.55 + (0.32 - 0.55) \sqrt{0.34}$$

$$K_{0.34} = 0.41$$

Thus the final distribution as shown in Table IX as follows:

Girder	kP ₁	kP ₂	Σk
1	0.41	0.18	0.59
2	0.48	0.48	0.96
3	0.12	0.28	0.40
4	0.01	0.06	0.05
Σ	1.0	1.0	2.0

From Ozbek's (114) calculations based on a maximum distribution to girder 2 of 1.0 a yield load can be predicted as follows.

$$\frac{1.0}{0.96} (279) = 290 \text{ KIPS (See Table XI for Comparisons)}$$

Vita

James Harold Deatherage was born in Knoxville, Tennessee on November 13, 1941. He attended elementary schools in that city and was graduated from Holston High School in June 1959. The following September he enrolled at the University of Tennessee for a brief period of time. He later attended East Tennessee State University prior to enrolling at the University of Missouri at Rolla in January 1963. In January 1965 he received his B.S. degree in Geophysics. In January 1966 he received his M.S. degree with a major in Engineering Mechanics.

After graduation, Mr. Deatherage was employed by McDonnell Douglas Astronautics Company as a structural engineer until 1973. At this time he returned to Knoxville to pursue his career in private practice as a consulting engineer and general contractor. After twelve years as a practicing engineer Mr. Deatherage returned to the University of Tennessee to pursue his Ph.D. in Civil Engineering. He received this Doctor of Philosophy with a major in Civil Engineering in June 1987.

The author is a registered engineer in Tennessee, Kentucky, South Carolina, Virginia and Florida. He is a member of Chi Epsilon, Tau Beta Pi, and American Society of Civil Engineers. Mr. Deatherage will be

employed by the Department of Civil Engineering at the University of Tennessee after graduation.