# SEISMIC RESPONSE OF HIGHWAY OVERCROSSINGS

by

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## SYNOPSIS

Linear and nonlinear analytical procedures for determining the seismic response of curved highway overcrossings are presented. Example solutions for one major overcrossing which collapsed during the San Fernando, California, earthquake of February 9, 1971, are described and the numerical results obtained are compared with apparent prototype behavior. Finally, present day design procedures for overcrossings are examined and recommendations are made for improving these procedures.

## INTRODUCTION

The damages caused to highway overcrossings during the San Fernando earthquake of February 9, 1971, pointed out the urgent need for both theoretical and experimental research related directly to seismic effects on bridge structures. As a direct result, a three-year research investigation entitled "An Investigation of the Effectiveness of Existing Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances" was initiated in 1971 within the Earthquake Engineering Research Center, University of California, Berkeley, under the sponsorship of the U.S. Department of Transportation, Federal Highway Administration. This investigation consists of the following phases: (1) A review of the world literature on earthquake resistant design of bridges with specific attention to those publications documenting the experiences of and investigations by bridge engineers in Japan where the incidence of damage to bridge structures due to earthquakes has been considerable. (2) A review of the damages to highway structures during the San Fernando earthquake and based on this review, the establishment of priorities for subsequent investigations of specific structural types. (3) An analytical investigation of specific highway structures requiring special consideration as identified in (2) above. (4) Detailed model experiments to determine local as well as gross structural behavior of selected overcrossings to simulated earthquake motions using the new University of California, Berkeley, shaking table. (5) Correlation of dynamic response obtained from analyses, model experiments, and field observations for the specific purpose of developing improved design criteria and specifications.

A report covering Phases 1 and 2 has recently been published [1]. This paper presents a part of Phase 3. The remaining portions of the overall investigation will be completed and reported at a later date.

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## DAMAGES TO A PROTOTYPE STRUCTURE

Many bridge structures located in a rather localized area of San Fernando suffered light to heavy damage during the earthquake of February 9, 1971. One major overcrossing which collapsed was located at the Golden State-Antelope Valley interchange. The photograph in Fig. 1 shows an aerial view of this interchange taken prior to the earthquake looking in a southwesterly direction. The photograph in Fig. 2 shows a corresponding view as observed after the earthquake. The collapsed south connector overcrossing is easily identified in Fig. 2.

A plan and elevation view of this overcrossing is given in Fig. 3. The collapsed portion of the structure consists of the tall central column 4 and the long section of prestressed concrete deck supported directly above. The initial cause of collapse of this structure appears to have been the large relative deck displacements produced in the west expansion joint near column 5, which allowed the box girder to fall off its bearing supports. The falling cantilevered portion of the deck span pulled the prestressed concrete deck system to the west permitting the east end of the span near column 3 to fall off its supports in a similar manner. Both cantilevered portions of the deck broke off at the top of column 4 allowing them to fall almost directly down from their original positions. The central supporting column fell in a westerly direction landing nearly on top of the west portion of the collapsed deck span.

The principle causes of collapse of this overcrossing were (1) the large vibratory motions induced in the super-structure by the high intensity vertical and horizontal ground accelerations, and (2) the relative ground displacements which occurred between supporting columns. It is the opinion of the authors that the former was the major cause of collapse in this particular case. Both of these effects can, of course, combine to produce relative separation of the deck at expansion joints. In this case, it appears quite certain that these relative displacements were sufficiently large to cause the prestressed concrete deck span to drop off its support at the west expansion joint; thus, initiating complete collapse of the structure.

## MATHEMATICAL MODELLING

To determine the seismic response of high curved (or straight) overcrossings, mathematical models of discrete form are used. The mass of the complete, structure from abutment to abutment is lumped at discrete nodal points as shown in Fig. 4. All masses are permitted six degrees of freedom except those masses located adjacent to expansion joints and at the end abutments.

In determining internal structure forces, the columns and the bridge deck are divided into finite elements. Figure 5a shows a typical box girder element having 12 generalized deformation coordinates as indicated in Fig. 5b. Longitudinal and transverse views of the idealized foundation model are shown in Fig. 5c for a single column. As indicated schematically, foundation flexibility and damping are modelled using discrete springs and dashpots, respectively, in the six coordinate directions.

Because of its influence on dynamic response, it is essential that expansion joint behavior be modelled properly. Figure 6a shows an expansion joint in the bridge deck which is provided with a transverse key. This key prevents a relative displacement along the joint in the $\eta$ direction. While a relative separation across the joint in the $\xi$ direction is permitted, longitudinal ties through the end diaphrams of the deck may be provided to resist this type of motion, Fig. 6b. Time dependent Coulomb friction forces may also be included to resist this separation. Ties may be used to resist separation along the joint in the vertical direction when the compressive reaction forces are eliminated or when they change sign due to large vertical motions in the deck. Relative rotations of the two ends of the bridge deck are freely permitted about the $\eta$ axis but may be partially or totally restrained about the $\xi$ and $\zeta$ axes by the vertical and horizontal ties, respectively. When any separation reduces to zero with a non zero relative velocity, impact will occur. This phenomenon is modelled by placing very short, stiff, compression springs between the impacting surfaces.

Both linear and nonlinear mathematical models have been used in this investigation. For the linear models, it is assumed that all deck, column, and foundation elements are linear, that no vertical separation is permitted in the expansion joints, that no horizontal ties are present in the expansion joints, that no impact is present in the expansion joints, and that the Coulomb coefficient of friction in the expansion joints is either zero or infinity. Mode shapes and frequencies for the first four modes of vibration are given in Figs. 7 and 8, respectively, for the zero and infinite coefficient of friction cases. The assumption of an infinite coefficient of friction implies that no expansion joints are actually present.

For the nonlinear models, all nonlinear features of the expansion joints may be included, i.e., elasto-plastic tie bars under elongation but bars which are incapable of carrying compressive loads, separations with associated impacts when they reduce to zero, and Coulomb friction. Plastic flexural yielding is also permitted at the ends of the columns when their combined loadings reach certain specified levels. Because of space limitations, it will be impossible however to describe here the basic yield criteria used for these elements under combined time dependent loadings. Elasto-plastic foundation springs may be used also, if desired.

Linear viscous damping in the overall structural system is provided by including a full damping matrix in the mathematical formulation of the dynamic equations of motion.

## COMPUTER PROGRAMS

The computer program used for linear seismic analysis is capable of handling bridge structural systems modelled by a combination of the following linear elements: (1) three dimensional linear truss elements,

(2) three dimensional linear straight beam elements, (3) three dimensional curved beam elements, (4) three dimensional linear boundary elements, i.e. spring elements, and (5) linearized expansion joint elements. The program provides for the following analysis options: (1) static response analysis, (2) determination of mode shapes and frequencies, (3) determination of individual modal responses and their combined root-mean-square response to a prescribed earthquake acceleration spectrum, and (4) determination of response time histories to prescribed time histories of dynamic loads, rigid ground excitations, or multiple ground excitations at the bases of columns and abutments. This computer program is a modified version of SAP, recently developed by E.L. Wilson and his co-workers(2).

It was necessary to develop a new computer program for the nonlinear seismic analysis. While this program follows closely the analytical procedures described above for the linear case, it differs in that it calculates new linearized structural parameters for the nonlinear elements after each time interval of integration. Initially, this program assembles all linear elements corresponding to the initial state of the structure. Following the initiation of seismic disturbances, linearized stiffnesses within the overall system are successively modified after each time interval according to the prescribed nonlinear characteristics of corresponding elements as follows: (1) three dimensional elasto-plastic straight beam elements are modified to linear straight beam elements, (2) three dimensional bilinear boundary elements are modified to linear elements and (3) the nonlinear expansion joint elements are modified to linear elements. In the computer program, a simple step-by-step integration algorithm with an equilibrium correction for each time step is used. Either constant or linear acceleration can be specified over each time interval of the analysis.

## NONLINEAR RESPONSE CHARACTERISTICS

Nonlinear analyses were carried out for the south connector overcrossing of the Golden State-Antelope Valley freeway interchange using vertical and horizontal ground motion intensities ranging from moderate to severe. Structural parameters varied in this investigation were (1) yield strengths, $F_y$, of horizontal tie bars (Type A, 3 - 6/4" Ø bars, $F_y$=70$^k$ ea.; Type B, 3 - 9/4" Ø bars, $F_y$=480$^k$ ea.; Type C, $F_y$=0 ) and (2) Coulomb coefficient of friction (0.2 and 0.4). Reinforcing details of columns were assumed adequate to fully develop and maintain flexural yield capacities during periods of excitation. Tie bars were assumed capable of experiencing unlimited elongations without fracturing.

Accepting numerical results of these nonlinear analyses, the following observations were apparent: (1) Separations of expansion joints 1 and 2 (see Fig. 4), when Type A or Type C tie bars were used, greatly exceeded their seat dimensions (15") under severe ground shaking and were of similar magnitudes for moderate shaking; thus, collapse of the structure between these two joints was definitely indicated for severe shaking and appeared likely to occur even for moderate shaking. (2) Separations of expansion joints 1 and 2, when Type B tie bars were used, remained below their seat dimensions even for severe ground shaking; thus collapse of the structure was not indicated in this case. (3) Tie bars across expansion joints were very effective in preventing structural collapse provdided they possessed yield strengths comparable to Type B bars and elongation capacities approaching the seat dimension of 15". (4) Assuming seat dimensions sufficiently large to prevent spans falling from their supports, flexural yield deformations in the columns under severe shaking conditions reached levels considerably above those levels which could actually be provided; however, under moderate shaking they remained within tolerable levels. (5) Under moderate to severe shaking, the overall structural response was considerably affected by vertical ground accelerations which caused large vertical oscillations to develop in the deck; thus, removing the vertical compressive reaction forces at expansion joints and as a direct result removing the horizontal Coulomb friction forces, as well.

## GENERAL RECOMMENDATIONS

Based on the previously described nonlinear response characteristics general recommendations are deduced as follows: (1) Major overcrossings of the type considered in this investigation should be designed on the basis of dynamic analyses using appropriate nonlinear mathematical models representing the structural systems and using time histories of ground motions suitable to their respective sites. (2) Because of their effectiveness in reducing dynamic response and in preventing structural collapse, horizontal tie bars should be provided which have sufficiently high yield strengths and yield capacities. (3) Vertical tie bars should be provided at expansion joints wherever possible. (4) Inelastic flexural yielding in columns should be avoided under moderate earthquake conditions but should be permitted to a limited degree under severe conditions; therefore, reinforcing details should be adequate to develop and maintain full yield capacities throughout their specified ranges of yielding. (5) Static lateral seismic loads used for preliminary design (elastic design philosophy) should considerably exceed those values specified by the "Bridge Planning and Design Manual", California State Division of Highways, Vol. 1 - Design Specifications, March, 1968, pg. 2 - 24, which was in effect prior to the San Fernando earthquake of February 9, 1971.

It should be pointed out that the Bridge Department, California State Division of Highways, has adopted new Interim Criteria For Earthquake Design which are consistent with the above recommendations.

## BIBLIOGRAPHY

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(2) "SAP - A General Structural Analysis Program" by E.L. Wilson, Report to Walla Walla District, U.S. Engineers Office, Structural Engineering Report No. SESM 70-20, University of California, Berkeley, Sept. 1970.

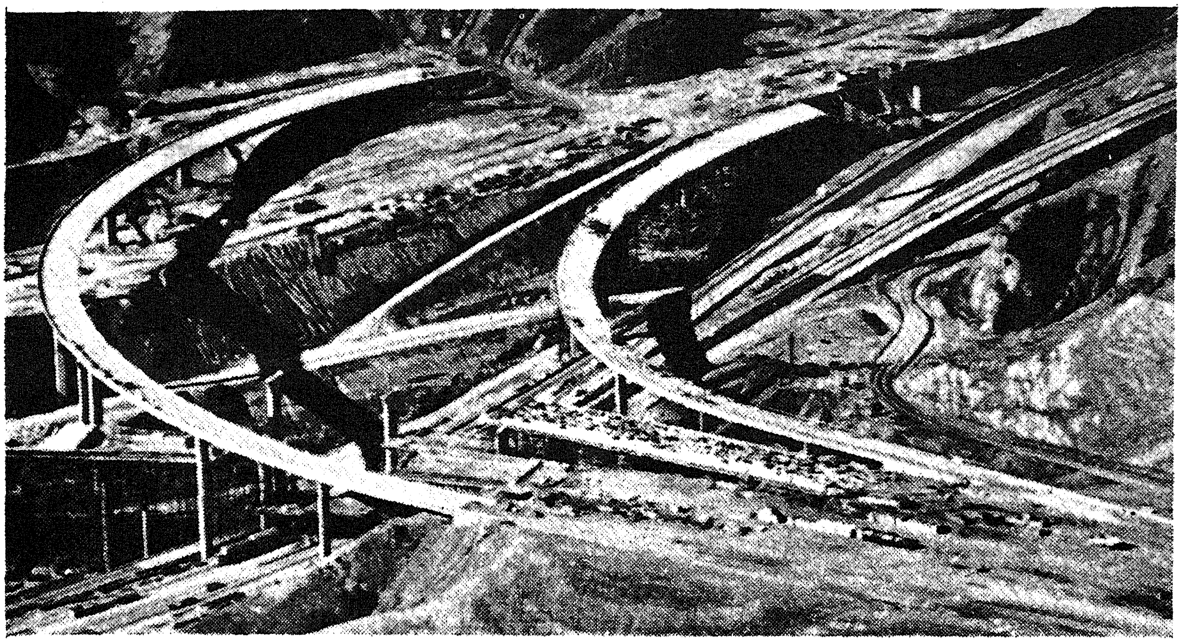

Figure 1 Pre-earthquake view of the Golden State-Antelope Valley freeway interchange, looking west. Photographed by Ralph Samuels

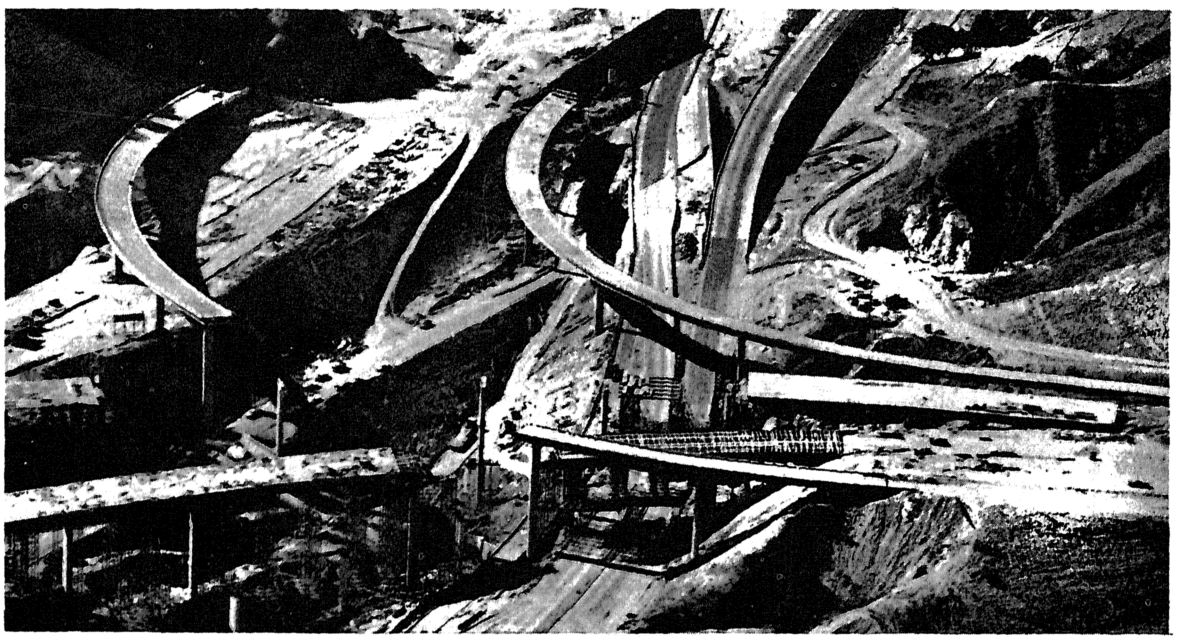

Figure 2 Post-earthquake view, looking west, of the Golden State-Antelope Valley freeway interchange. The center two spans of the highest overpass have collapsed. Photographed by Ralph Samuels

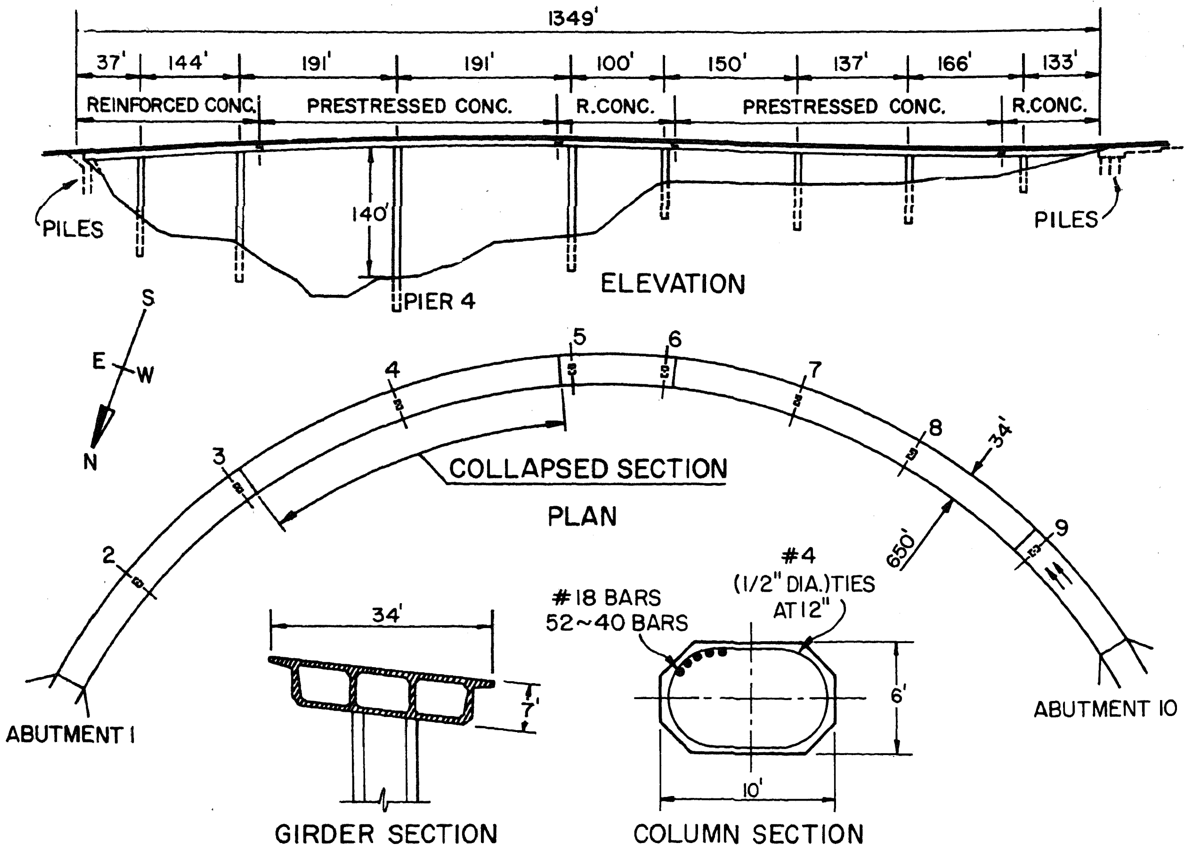


FIGURE 3 SOUTH CONNECTOR OVERCROSSING

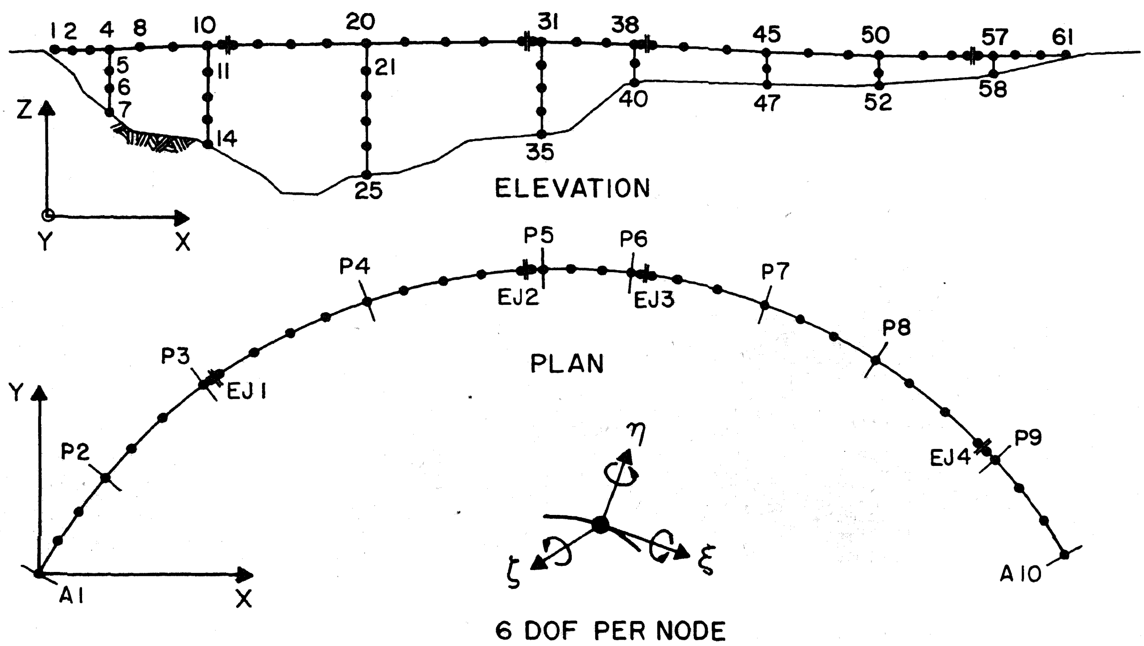


FIGURE 4 IDEALIZED LUMPED PARAMETER SYSTEM

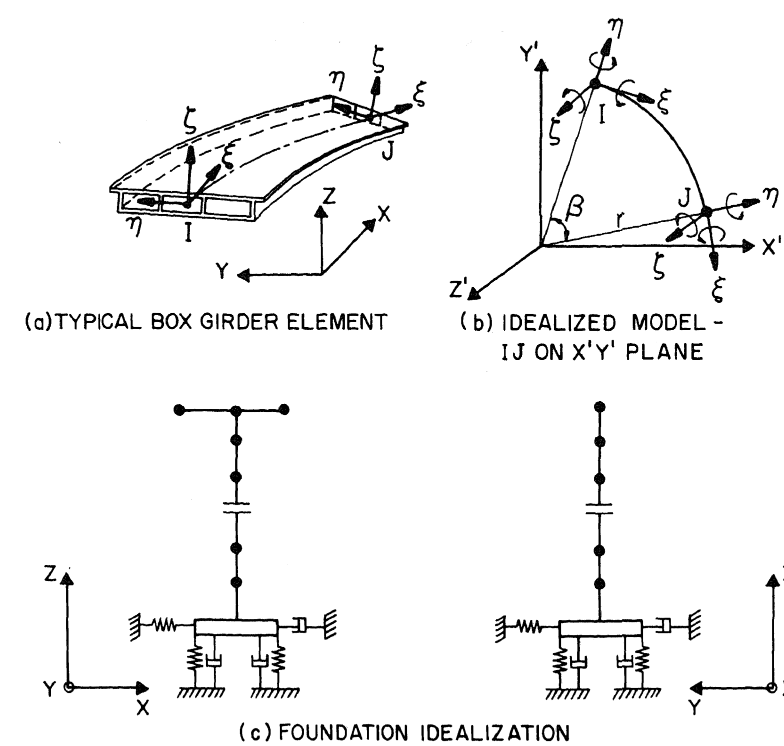


FIGURE 5 MATHEMATICAL MODELING OF BRIDGE STRUCTURAL SYSTEM

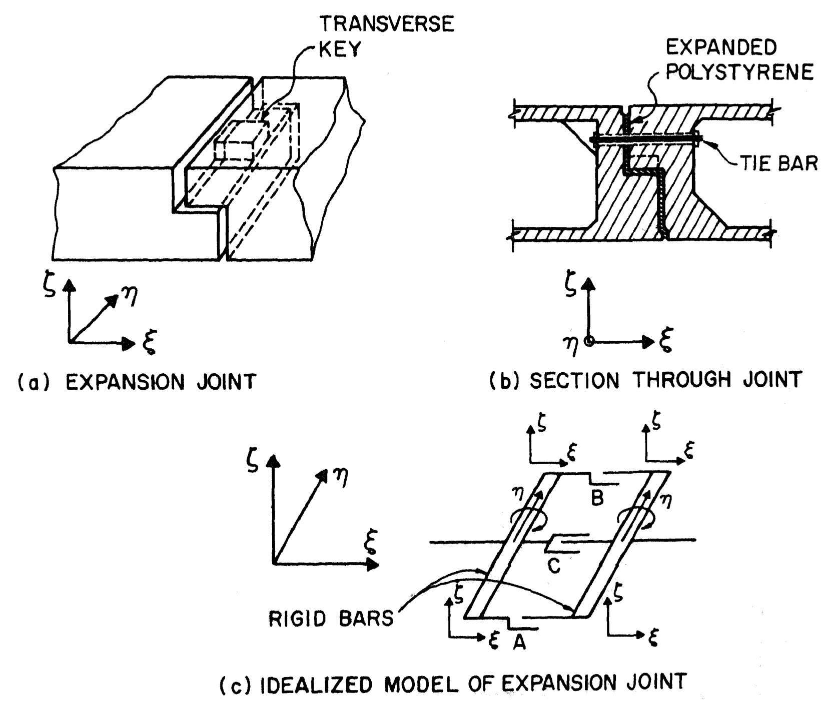


FIGURE 6 IDEALIZATION OF BRIDGE EXPANSION JOINT

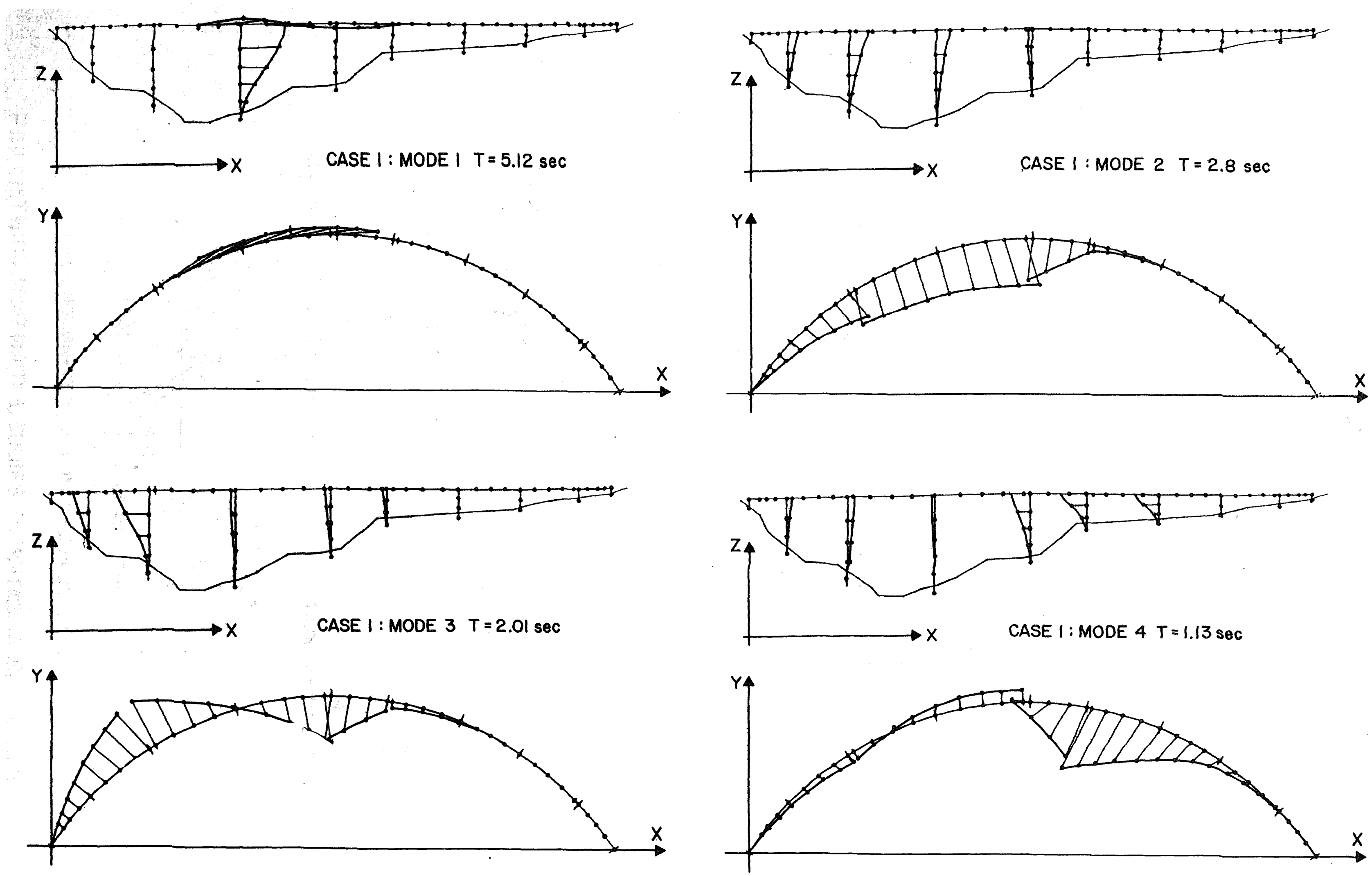


FIG. 7 MODE SHAPES OF CASE I: ZERO FRICTION AT EXPANSION JOINTS

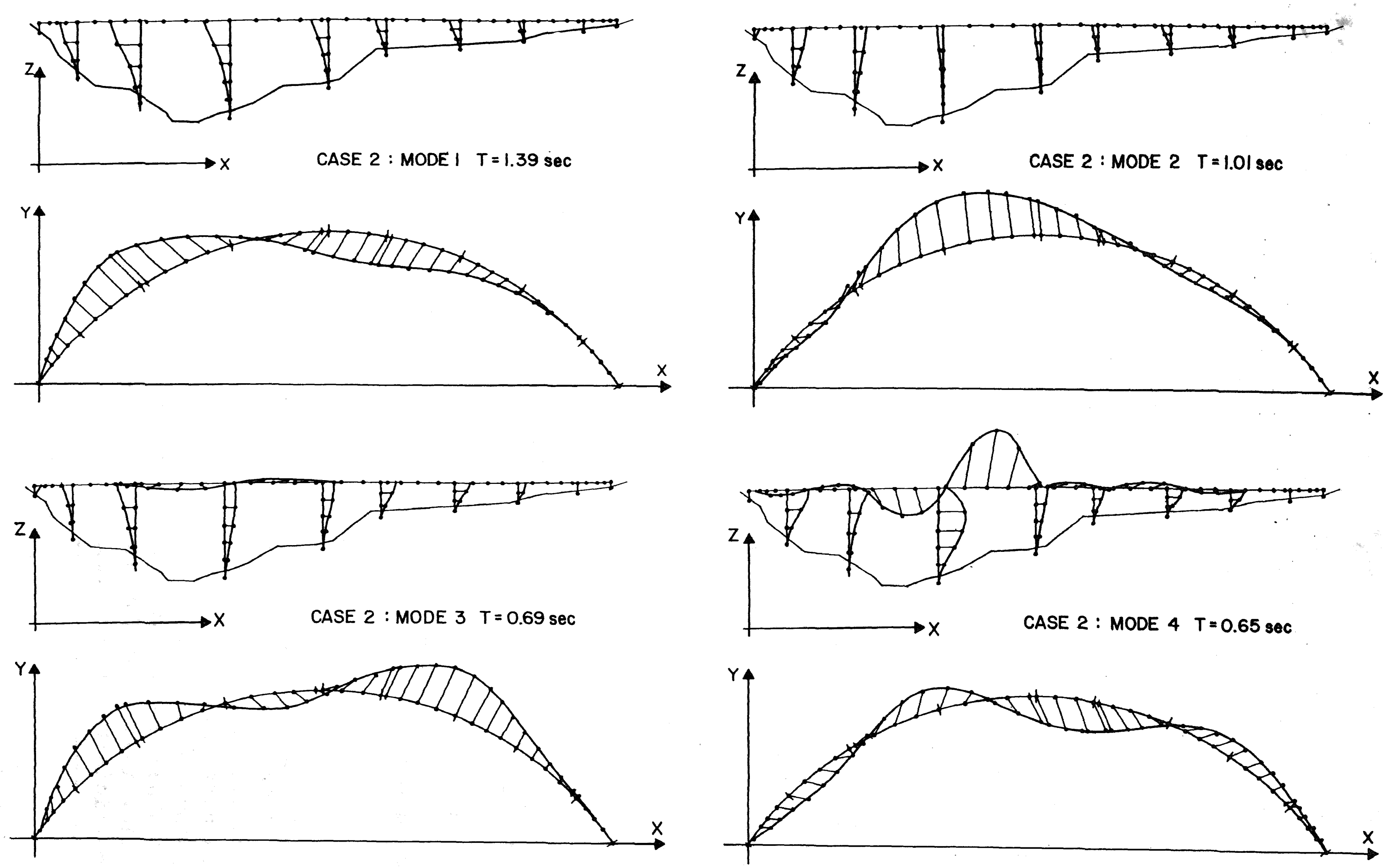


FIG.8 MODE SHAPES OF CASE 2 : INFINITE FRICTION AT EXPANSION JOINTS