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GUIDELINES FOR STRONG-MOTION INSTRUMENTATION OF HIGHWAY BRIDGES

December 1981

Final Report

Earthquake Protection

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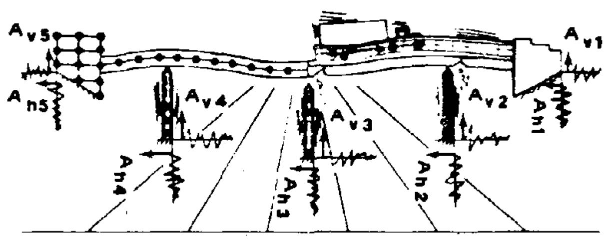

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Federal Highway Administration

Offices of Research & Development
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FOREWORD

This report is the result of research conducted at the Seismic Engineering Branch of the U. S. Geological Survey for the Federal Highway Administration (FHWA), Office of Research, under FHWA Purchase Order P.O. 5-3-0195. The report will be of interest to structural researchers, designers, and planners concerned with designing an instrumentation scheme to record bridge structural response to strong ground motion.

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Charles F. Scheffey

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Director, Office of Research
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NOTICE

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16. Abstract

This report suggests guidelines for the strong-motion instrumentation of highway grade separation bridges. It has been written for the civil or structural engineer who is not familiar with the objectives of strong-motion instrumentation programs, the instrumentation utilized in such programs, and where and how to install that instrumentation. It is designed to supplement a previously published companion report entitled "Use and interpretation of strong-motion records from highway bridges" (Raggett and Rojahn, 1978), Report No. FHWA/RD-78/158.

The report is divided into ten principal sections. The first section is a general discussion on strong-motion instrumentation and records. In the second and third sections, instrumentation program objectives and criteria for selecting a bridge for strong-motion instrumentation are introduced. The fourth section is a discussion on the linear dynamic behavior of bridges that is designed to familiarize the reader with the theoretical aspects of bridge response. In the fifth section, the failure of highway bridges during the 1971 San Fernando earthquake is discussed and in the sixth section, recommended guidelines for instrument placement on and adjacent to bridge structures are presented. The seventh and eighth sections describe recommended instrumentation type, installation techniques, and maintenance requirements, and the ninth contains a complete description of an actual strong-motion instrumentation scheme installed on a continuous two-span bridge near El Centro, California. The last (tenth) section contains concluding remarks, followed by a list of references and an appendix containing a discussion on the positive and negative aspects of recording various quantities of motion.

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INTRODUCTION

The failure of new highway grade separation bridges in the February 9, 1971 San Fernando, California earthquake demonstrated how little was known about the dynamic behavior of such bridges during earthquakes. As a consequence, state and federal government agencies have initiated strong-motion instrumentation programs in hope of obtaining information about dynamic bridge behavior in future earthquakes. This report is designed to assist in the development of those programs. It contains guidelines for selecting bridges to be instrumented as well as guidelines for designing strong-motion instrumentation schemes for those bridges. The suggested strong-motion instrumentation schemes have been designed specifically for typical highway grade separation bridges. The principles utilized in the design of these schemes, however, would be generally applicable in the design of instrumentation schemes for other types of bridges. The suggested guidelines have been written for the civil or structural engineer who is not familiar with strong-motion instrumentation; they are consistent with the analysis procedures suggested in a companion report entitled "Use and Interpretation of Strong-Motion Records from Highway Bridges" (Raggett and Rojahn, 1978).

The selection of a strong-motion instrumentation scheme for a highway grade separation bridge, that is, the selection of the type of instrumentation to be installed and where to place it, is based upon a number of factors. The most important factor is the objective of the instrumentation program. Specifically, what information is the instrumentation intended to yield? The second principal factor is expected bridge behavior and the third principal factor is the quantities that are to be recorded. The first two of these factors are covered in the main body of this report; the third is covered in the Appendix, a discussion on the positive and negative aspects of recording various quantities of motion. In general, absolute accelerations are the most convenient and desirable quantities of motion to record. For this reason, the sections of this report describing recommended instrumentation type and guidelines for instrument placement are concerned entirely with acceleration recording instrumentation. Similarly, the sections describing current installation techniques and maintenance requirements and an example instrumentation scheme already installed on an actual bridge (Meloland Overcrossing near El Centro, California) deal exclusively with acceleration recording instrumentation. It may, of course, also be desirable to install other types of instrumentation, such as strain gauges or displacement meters; a thorough treatment of such instrumentation, however, is beyond the scope of this report.

I. GENERAL DISCUSSION ON STRONG-MOTION INSTRUMENTATION AND RECORDS

A strong-motion record is defined here as the record of ground or structural motion caused by a relatively large earthquake. This strong-motion may be recorded in a number of ways. It may be a light trace on film, a line on a strip of paper, or an analog or digital signal on magnetic tape. In all cases the motion is recorded as a function of time.

The motion recorded is that for a point moving along a single axis and may have dimensions of displacement, velocity, or acceleration. Most often acceleration is recorded (see Appendix). The motion is recorded as an electrically or mechanically generated signal which is proportional to the motion within some acceptable error.

The adjective "strong" is included to differentiate these recordings from recordings taken with other common seismographs used to determine earthquake magnitudes, epicentral locations, and other earthquake parameters. Such seismographs are capable of detecting earthquakes almost anywhere in the world, and are necessarily very sensitive. Strong motions would cause these seismograph recordings to be off scale. In general, strong motions could cause damage to structures, and are recorded near the location where an earthquake has occurred. The actual level of ground motion above which motions are "strong" is not well defined. Typically, ground motions with a peak acceleration greater than 0.05 g (g = acceleration of gravity) are of interest to the structural engineer, and those greater than 0.20 g are currently considered to be potentially damaging.

In the United States, strong-motion records are normally recorded on accelerographs. The accelerographs are of two general types: analog or digital. In either case, acceleration is sensed by accelerometers and recorded as a continuous function of time. In the analog instrument, it is recorded on light-sensitive film or paper, or on magnetic tape, and in the digital instrument, it is recorded in digital form on magnetic tape. Both instrument types are powered by batteries, are triggered into operation by the strong-motion itself (i.e. motion having an acceleration equal to or greater than a pre-set triggering level, commonly 0.01 g in the vertical direction) and are available either as self-contained triaxial instruments or remote-accelerometer central-recording instruments. In the self-contained variety, all major components--accelerometers, recorder, and trigger--are housed in one container, whereas in the remote-recording type (the type recommended for installation on bridges), these elements can be physically separated but are interconnected by low-voltage data cable.

A typical analog film record from a three-component self-contained accelerograph is shown in figure 1 (records from remote-accelerometer

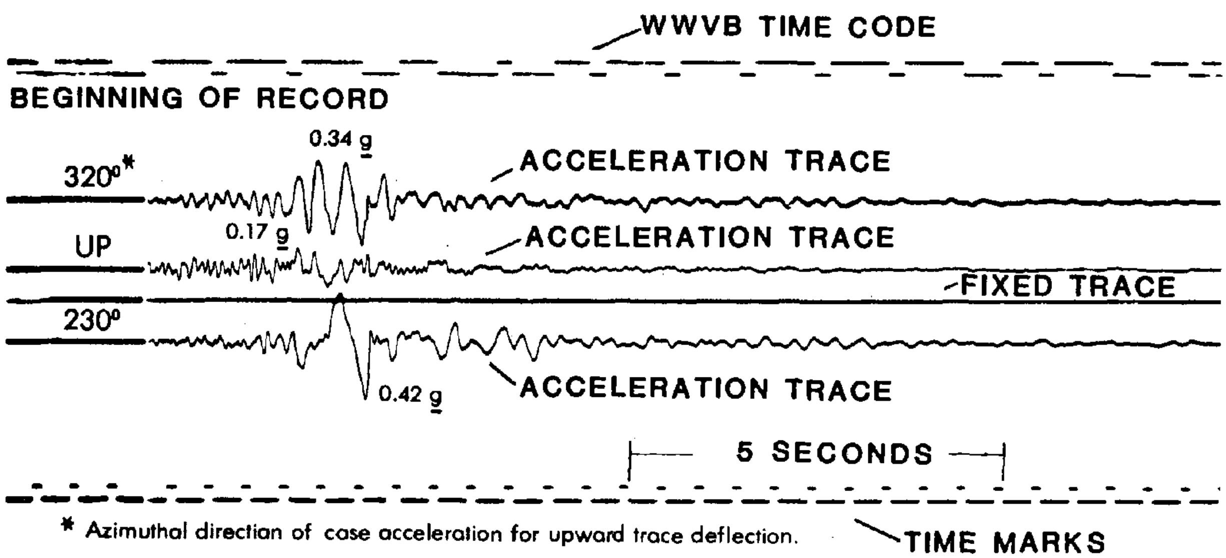


Figure 1.- Copy of a typical three-component strong-motion accelerograph record.

central-recording systems are similar although they normally contain additional traces). The record contains time marks, two fixed traces, three acceleration data traces and a WWVB time code. The time marks denote film speed and normally appear every half second; each fixed trace is a reference trace that is used to extract long-period errors from the data due to film distortion or the film drive mechanism; and the WWVB time code is used to identify when the event occurred. The most important elements of the record, the acceleration data traces, have values of amplitude that are approximately proportional to real values of recorded acceleration. For most instruments in present use, the sensitivity of the acceleration traces is on the order of 1.90 cm/g.

Records from a typical digital instrument are normally recorded in digital form on one- or four-track cassette tape and are not immediately available in analog form. Analog traces of a digital record can be obtained by processing the digital data with a digital-to-analog converter and plotter. Hardware of this type has been developed by the firms that manufacture the digital instruments and is normally available either through these firms or the agency or organization that maintains the instruments.

In order to convert both the analog film and digital cassette records into a digital form that can be processed on high-speed digital computers, additional processing is required. Analog records are converted into digitized form on systems known as digitizers. In such a system, a handheld cursor or automatic scanning device is used to follow the trace and assign numerical coordinate values (normally Cartesian) to all points of interest. Such points are normally selected at equal intervals or at all peaks, valleys, and points of inflection. The numerical values are recorded on computer-compatible cards or on magnetic tape. Data from digital strong-motion instruments, on the other hand, require processing for conversion from digitized form on cassette tapes into a computer-compatible format. This processing is handled by systems designed by the instrument manufacturer. The availability of such systems should be known by the strong-motion instrument manufacturer or the agency or organization maintaining the instrument.

After conversion into a high-speed computer-compatible format, the digitized data can be corrected to extract errors introduced during the recording and digitization process and to account for the dynamic characteristics of the recording instrument. Procedures for correcting the data are well established (Trifunac and Lee, 1973; Brady and Perez, 1976; Miklofsky and Mancini, 1977; Basili and Brady, 1978) and are presently performed routinely by the U.S. Geological Survey (e.g., Brady and Perez, 1976), the California Division of Mines and Geology (Porter, Ragsdale, and McJunkin, 1979), and the University of Southern California (Trifunac and Lee, 1979). When the processing is completed, the digitized acceleration data are then ready for analysis. Procedures for standardized analysis of three-component ground motion records are also well established (Brady and Perez, 1976; Trifunac and Lee, 1979) and are routinely performed. A methodology for the analysis of a set of earthquake records from a particular bridge, which is not yet a routine procedure, is presented in the companion report entitled "Use and interpretation of strong-motion records from highway bridges" (Raggett and Rojahn, 1978).

II. INSTRUMENTATION PROGRAM OBJECTIVES

In a general sense, all bridge strong-motion instrumentation programs should have the same purpose: to obtain data that will improve the state of knowledge of bridge behavior during earthquakes. The amount of strong-motion instrumentation that should be installed on a bridge is dependent, of course, on the intended use of the data. Based on the policies of existing structural strong-motion instrumentation programs such as those operated by the California Division of Mines and Geology and the U.S. Geological Survey, it is anticipated that most bridge strong-motion instrumentation programs will have one of the following primary goals: the acquisition of data to improve engineering design and construction practice; the acquisition of data to evaluate the potential earthquake hazard of existing similar structures; or the acquisition of data to evaluate the safety of particular structures after strong earthquake-induced ground shaking has occurred. Structures instrumented under programs having safety evaluation as their primary goal generally require substantially less instrumentation than those instrumented under programs having one of the other two above-mentioned goals.

This report is concerned primarily with strong-motion instrumentation schemes for programs devoted to the acquisition of data for (a) improving engineering design and construction practice or (b) evaluating the earthquake hazard of existing bridges. The principles suggested in this report, however, are also generally applicable for programs having safety evaluation as their primary goal. On the basis of the data analysis procedures described in the companion report (Raggett and Rojahn, 1978), we recommend that strong-motion instrumentation schemes (for the types of programs of concern here) be designed in accordance with one of the following instrumentation program objectives: (1) force level determination; (2) mathematical model identification; or (3) mathematical model verification.

Strong-motion instrumentation schemes designed in accordance with the first objective, force level determination, provide data that can be used to calculate maximum internal stresses and strains at any desired location and to study or predict possible failure modes. Force levels are computed from an extension of the well-known expression

$$(\text{Force}) = (\text{Mass}) \times (\text{Acceleration}).$$

Masses can be estimated from drawings, and accelerations can be measured directly at specific points.

Strong-motion instrumentation schemes designed in accordance with the second or third objective, the identification or verification of a mathematical model of the bridge, provide data that can be used to evaluate the assumptions made in the formulation of any mathematical model used in the design stage; this allows for the possibility of improving state-of-the-art techniques for modeling bridge behavior during earthquake excitation. In addition, mathematical models formulated through the analysis of strong-motion records can be used to study possible failure modes as well as predict significant bridge distortions of similar bridges during future earthquakes.

The identification or verification of a mathematical model is normally of greater interest than the determination of force levels for a particular bridge during a particular earthquake primarily because the results of a mathematical model study may have wider application. The potential for wider application, however, should not detract from the importance and necessity of performing more simplistic force determination studies, particularly in the case of damaged bridges.

III. CRITERIA FOR SELECTING A BRIDGE FOR STRONG-MOTION INSTRUMENTATION

The purchase and installation of strong-motion instrumentation on a highway bridge is an expensive venture ($20,000 to $30,000 per bridge in 1981 dollars), and therefore, the criteria for selecting a bridge for the instrumentation are critical. The two most important elements of these criteria are bridge location and bridge type. If an inappropriate location is chosen, e.g., one that probably will not experience earthquake induced strong ground shaking within the service life of the instrumentation or bridge, significant strong-motion data may never be obtained. Similarly, if an inappropriate type of bridge is chosen, e.g. one that is exceedingly complex or one that is not of a typical design, interpretation of any resulting strong-motion data may be exceedingly difficult (if not impossible) and may not be useful in terms of improving engineering design practice or evaluating the potential hazard of other highway bridges.

Criteria for selecting locations for instrumenting <u>buildings</u> have been under development for several years (Rojahn, 1976; Hart and Rojahn, 1978); these criteria would be appropriate as well for selecting locations for instrumenting bridges. In general, these criteria are based on the premise that the most desirable strong-motion records would be those obtained from structures damaged by an earthquake. This requires that the instrumented structures be located at sites where damaging levels of strong ground shaking can be expected to occur (with a high degree of probability) within the service life of the structure or the service life of the instrumentation (currently estimated to be 20 to 40 years), whichever is shorter. The frequency of occurrence of earthquake activity is a particularly important element of this criteria. It is also critical that selected sites be located within close proximity to the potential fault rupture zones or potential sources of energy release. In this regard it is of importance to note that most of the 62 highway overpass bridges damaged by the magnitude 6.4 1971 San Fernando, California earthquake, and all of the collapsed bridges (for that event), were located within 3 miles (5 km) of the surface fault rupture zone (Elliott and Nagai, 1973); the remaining damaged bridges were within 10 miles (16 km). For this earthquake, then, it would have been appropriate, using hindsight, to specify that all instrumented bridges be located within 10 miles of the fault rupture zone, and preferably within 3 miles. For other earthquakes in other regions, however, the distance specification would of course be different due to differences in earthquake magnitude, depth of energy release, ground motion attenuation characteristics, bridge design criteria, and other related factors. In general, for shallow earthquakes like those occurring in California, the closer the site is to the source of energy release, the greater the potential for earthquake damage. For earthquakes in other regions where the amplitude of ground shaking may attenuate less quickly with distance than in California (e.g., in the eastern U.S.) or in regions where the earthquakes may be deeper (e.g., in Romania), the areas that experience damaging levels of ground shaking may be substantially larger or substantially farther from the earthquake source region than would be the case for similar-sized earthquakes in California.

In regard to the type of bridge structures that should be instrumented, it is generally important that the structures be simple and typical in concept and design. Simplicity is required, at least in the early state of data acquisition and knowledge development, in order to minimize assumptions in the data interpretation process. Typicality is required in order to transfer lessons learned from the interpretation of data from one bridge to the design improvement or hazard evaluation of other similar bridges.

In regard to the simplicity recommendation, it is worth noting that in the California Strong-Motion Instrumentation Program, which is operated by the California Division of Mines and Geology, particular emphasis has been placed on the need to instrument simple bridges, primarily because of the rather minimal state of knowledge on basic dynamic bridge response. It is our belief that this restriction should not apply to bridges instrumented later in that program and other programs (although it is necessary now) because the need to instrument simple structures will decline as our knowledge of dynamic bridge behavior increases. Moreover, the need to instrument not-so-simple structures that are highly susceptible to earthquake damage is obvious.

IV. THEORETICAL LINEAR DYNAMIC BEHAVIOR OF BRIDGES

In order to obtain the maximum information from the minimum number of strong-motion instruments, an understanding of the expected linear dynamic bridge behavior is required. Following, therefore, is a discussion of the qualitative aspects of theoretical linear dynamic bridge behavior.

Throughout this discussion on linear dynamic behavior of bridges, it is generally assumed that the ground motion is identical and in phase beneath all supports of the bridge, i.e., there is no relative motion between supports. This assumption is required in order to simplify the discussion as well as to be compatible with the analysis techniques presented in the companion report (Raggett and Rojahn, 1978). In reality, this assumption does not always hold as it is highly dependent on several factors: bridge length, foundation conditions, and the nature of the seismic wave forms. Based on existing data, this assumption is likely to hold true for very short highway grade separation bridges on firm homogeneous foundation material; it can be expected to be erroneous, however, if the foundation material varies significantly from one support of the bridge to the next, or if the bridge spans are long (see section VI-A).

It is also assumed that all bridge connections (fixed, pinned, and roller-type) remain intact, i.e., they do not fail during the earthquake excitation. This is one of the underlying assumptions of linear behavior. In reality, however, these connections may and, in fact, do fail. What is important, then, is the behavior of the bridge until such failures occur as well as the behavior subsequent to those failures. Even though the response is nonlinear in this case, it may be possible to consider the total nonlinear response as a series of linear responses with different dynamic characteristics for each time segment. Because of their obvious importance, bridge failures are described in detail in the next section; they have also been a primary consideration in the development of suggested strong-motion instrumentation schemes presented later.

According to linear dynamic bridge theory, the motion of a bridge is composed of two principal parts: the motion of the ground plus (or minus) distortions relative to the ground. If there were no distortions of the bridge then all bridge motions would simply equal the ground motion. In this case, knowing the mass distribution of the bridge, forces throughout the bridge could be computed simply as a product of the mass and the recorded ground acceleration. In most cases, however, forces produce distortions which change the acceleration of the mass in question, thus changing the force. Therefore, since ground motions can be recorded with relative ease, it is the recording of the bridge distortions relative to the ground which is of greatest interest. These distortions are of particular concern because they may be far greater than the ground motions themselves. In other words, dynamic amplification may be significant. Just as likely, however, the bridge distortions might be negligible compared to the ground motions, so the motion of the bridge relative to its base might be far less than the ground motion itself. Because the actual bridge distortions may be between these two extremes, they are of great concern and are well worth being recorded. The emphasis of this discussion on bridge behavior, therefore, is on describing qualitatively distortions of bridges relative to the ground.

Without any knowledge of expected bridge behavior, one could record bridge distortions with many transducers uniformly spaced over the entire bridge, and get good results. Such an approach, however, is inefficient and expensive.

One could reduce the required instrumentation to a minimum without fear of losing valuable information by anticipating all significant bridge distortions under seismic loading. The first step in anticipating bridge behavior, then, is to define the minimum number of degrees of freedom required to describe all significant motions relative to the ground. For each degree of freedom there exists a unique distorted shape which can exist independently from all other possible distorted shapes. Possible types of distortion (degrees of freedom) for a bridge deck are shown in figure 2; these include: rigid body horizontal translations and rotations about a vertical axis; vertical and flexural distortions of the bridge deck; and torsional or twisting distortions of the deck about an axis parallel to the centerline of the roadway. For many bridge decks, many of the distortions shown in figure 2 might not be free to occur, i.e., they are <u>not</u> possible degrees of freedom. For example, if a bridge deck is pinned laterally and vertically at both of its ends to rigid rock abutments, and pinned at least at one end longitudinally, then all rigid body translations and rotations of the deck relative to the ground are not free to occur. The second step in anticipating bridge behavior, therefore, is to eliminate from all possible degrees of freedom, those that are not free to occur. The third and final step is to eliminate those that are free to occur but are not expected to be excited to any significant extent by seismic input.

Following is a detailed discussion on the linear dynamic behavior of several common bridge types. Throughout each discussion the prime consideration is the identification of response degrees of freedom, including those that are free to occur but should be eliminated. Other topics such as damping ratios and frequencies of vibration are of great concern, but have little or no bearing on the instrumentation scheme employed. When all

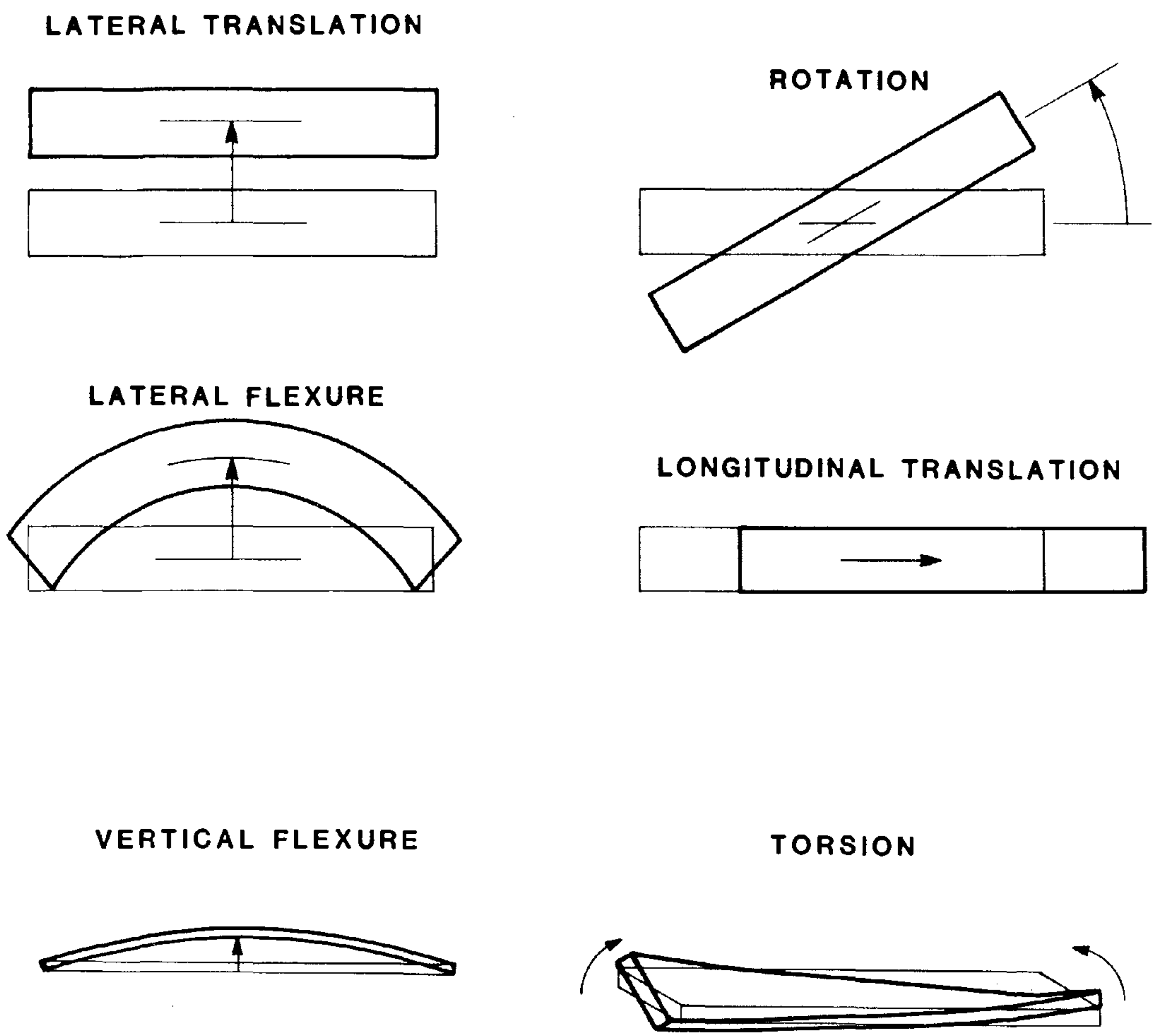


Figure 2.- Possible types of distortion of a bridge deck relative to the ground.

significant distortions of the bridge have been identified, other significant topics in the analysis of strong-motion records can be addressed.

A. Triple Simple Span Bridge

Consider the expected distortions of the triple simple span bridge shown in figure 3. Each of three spans is simply supported at both ends, pinned at one end, and on rollers at the other. Each span is cast integrally as a separate unit, and expansion joints separate all three spans longitudinally. The north and south piers and all 3 spans are assumed to be effectively rigid in the east-west or lateral direction.

First we shall consider the case in which the foundation material is assumed to be rigid.

Because the piers are considered to be rigid in the transvere direction, the only significant distortions of the north and south piers are flexural

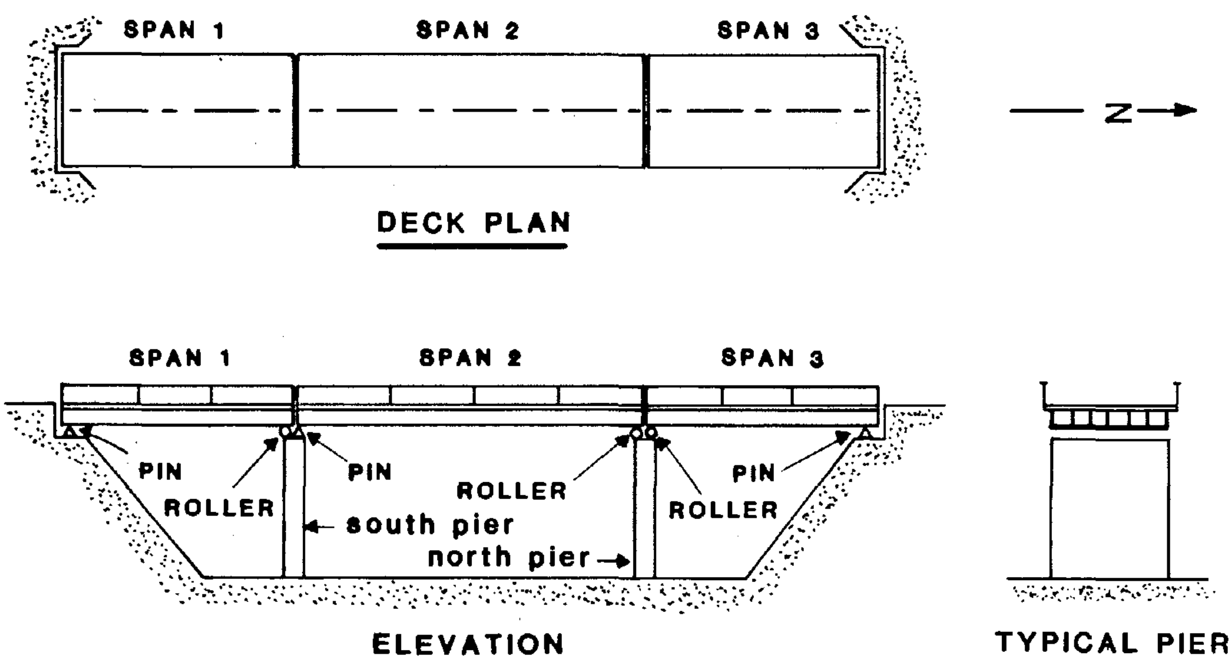


Figure 3.- Triple simple span bridge.

distortions in the longitudinal (north-south) direction. These distortions can be described by a single degree of freedom, namely the longitudinal translation of the pier top relative to the ground (fig. 4).

The significant distortions of spans 1 and 3, relative to the ground, are vertical flexural distortions, whereas those for span 2 are vertical flexural distortions plus longitudinal (north-south) rigid body translations. Vertical flexural distortions can be expected to be significant, even though bridge decks are designed primarily for vertical flexure and have factors of safety in design that usually can accommodate seismically induced vertical flexural distortions without overstress. Theoretically, the vertical deflection of each point along the span (and there are an infinite number of such points) is an independent degree of freedom. If it is assumed, however, that the vertical motions of the piers and abutments are identical, and that the fundamental

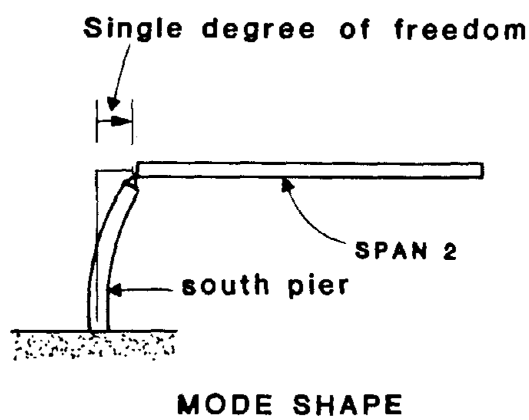


Figure 4.- Triple simple span bridge. Longitudinal translational mode of vibration of span 2 (including longitudinal flexural distortions of south pier; assuming rigid body conditions).

flexural mode of vibration (a thorough discussion on natural modes of vibration is presented in Raggett and Rojahn, (1978)) can be approximated as a half sine wave, then all vertical flexural distortions can be adequately described by a single degree of freedom, namely the vertical translation at or near mid-span (fig. 5). Similarly, the complete longitudinal translation of span 2 in the longitudinal bridge direction can be described by a single degree of freedom, namely the longitudinal translation of the south pier top relative to the ground (fig. 4) assuming that axial distortions can be neglected.

If the piers and bridge deck were not effectively rigid in their own plane, additional possible distortions would include lateral flexural distortions of the deck, lateral flexural distortions of the pier, longitudinal axial extensions of the deck, and torsional distortions of the

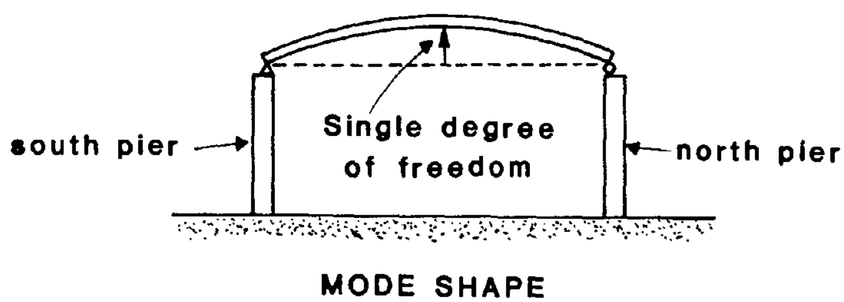


Figure 5.- Triple simple span bridge. Vertical flexural mode of vibration of span 2 (assuming distorted shape of span 2 can be approximated as a half sine wave).

deck about the roadway centerline.

For the case in which the foundation is flexible relative to the bridge supports, then, in addition to the distortions for the rigid foundation condition, longitudinal and lateral rocking of both piers and abutments may be expected. The longitudinal flexural and longitudinal rocking distortions of each pier can be measured by the motions of a single point (the pier top displacement in the longitudinal bridge direction), but one additional degree of freedom (a rotation about the horizontal axis at the base of each pier) must be introduced if these two types of distortions are to be differentiated. Lateral rocking distortions of each pier can be described by a single degree of freedom for each pier: the lateral translation of the pier top. The rocking distortions of the abutments in both directions (lateral and longitudinal) similarly can be described by a single degree of freedom for each distortion: the lateral and longitudinal translations of each abutment.

Lateral rocking of both the north and south piers allows for the possibility of lateral rigid body translation of the deck, rigid body deck rotation about a vertical axis, and warping of the deck about a centerline axis. Because the piers are assumed to rock laterally as rigid bodies, then all of these additional deck distortions can be described in terms of the two lateral pier top translations (fig. 6).

In summary, if abutments and pier bases are considered rigid, then all significant bridge distortions can be described by four degrees of freedom: one vertical translation near mid-span in each span and a longitudinal translation of the top of the south pier. If pier and abutment rocking is possible, then all significant distortions relative to the pier bases can be described by ten degrees of freedom: the above four plus the lateral translations of the top of each pier and the lateral and longitudinal translations of each abutment.

B. Continuous Two Span Bridge

Consider as a second example the expected distortions of the continuous two span bridge shown in figure 7. The bridge deck is a continuous, reinforced concrete box girder supported at midspan by a single column cast integrally with the deck. At each end, the bridge is effectively on rollers. Although this type of bridge is rarely built with actual rollers, the bridge deck is often simply resting on the foundation, which can be reasonably modeled as if on rollers. Keys and the dead load are assumed to effectively restrain the bridge deck and therefore do not allow lateral or vertical translations relative to the abutment. The central column is fixed to the bridge deck on top, and fixed to the foundation material at the bottom.

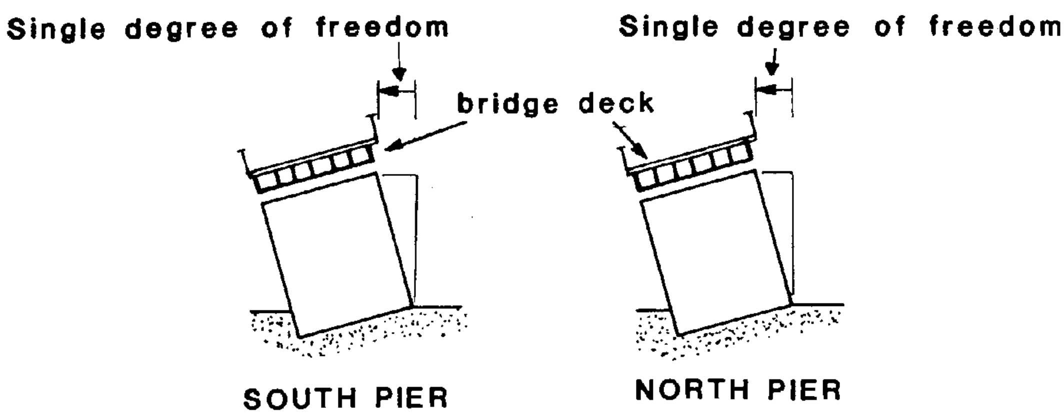


Figure 6.- Triple simple span bridge. Lateral rocking distortions of north and south piers (assuming foundation material is flexible relative to piers).

Again we consider first the case in which the foundation material is assumed to be rigid. In this case, because the ends of the bridge deck are assumed to be fixed laterally and vertically, only longitudinal rigid body motion of the deck, relative to the ground, is expected. This longitudinal rigid body motion will produce large moments at the top of the column which in turn will excite antisymmetric vertical bridge deck distortions. The longitudinal motion will occur in combination with the antisymmetric vertical flexure, i.e., the motion will be coupled. Assuming the distorted shape of the vertical motions can be approximated as a sine wave, two degrees of freedom will be required to describe the coupled longitudinal and vertical motion: a longitudinal translation at the center column and a vertical translation at mid-span of either span (fig. 8).

Because the bridge is long relative to its horizontal width, the bridge

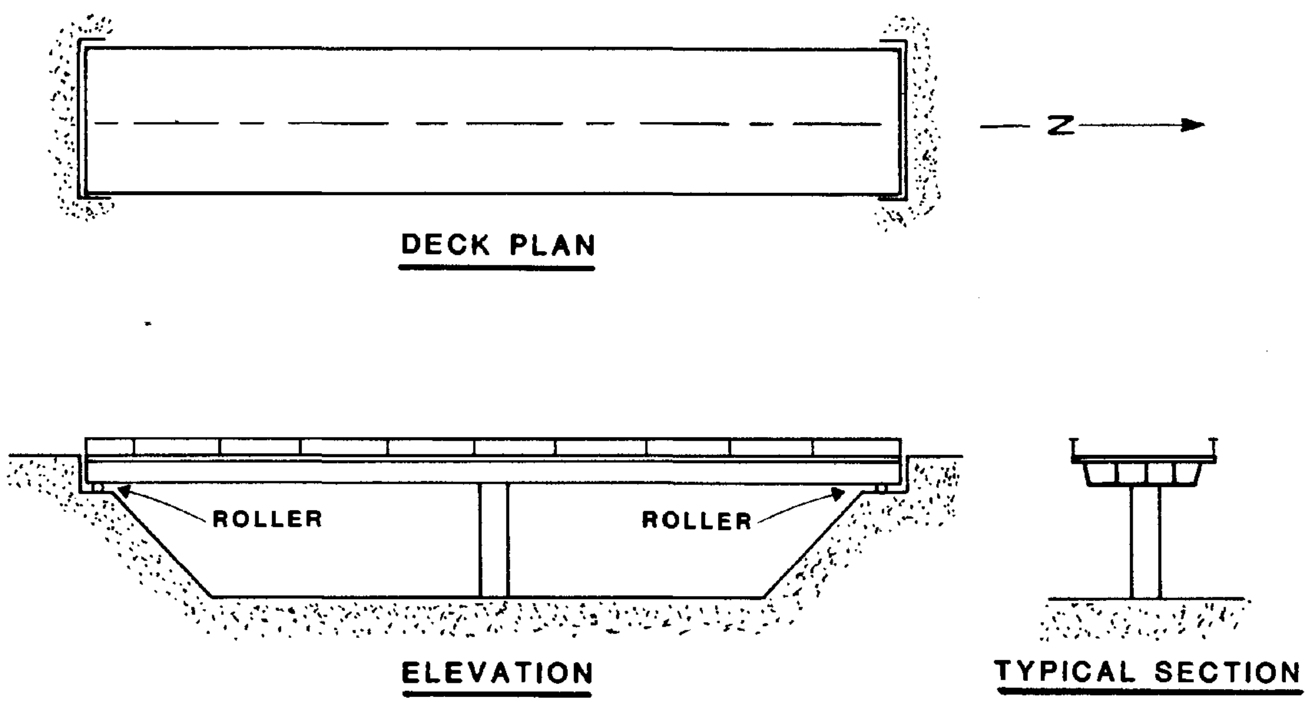


Figure 7.- Continuous two-span bridge.

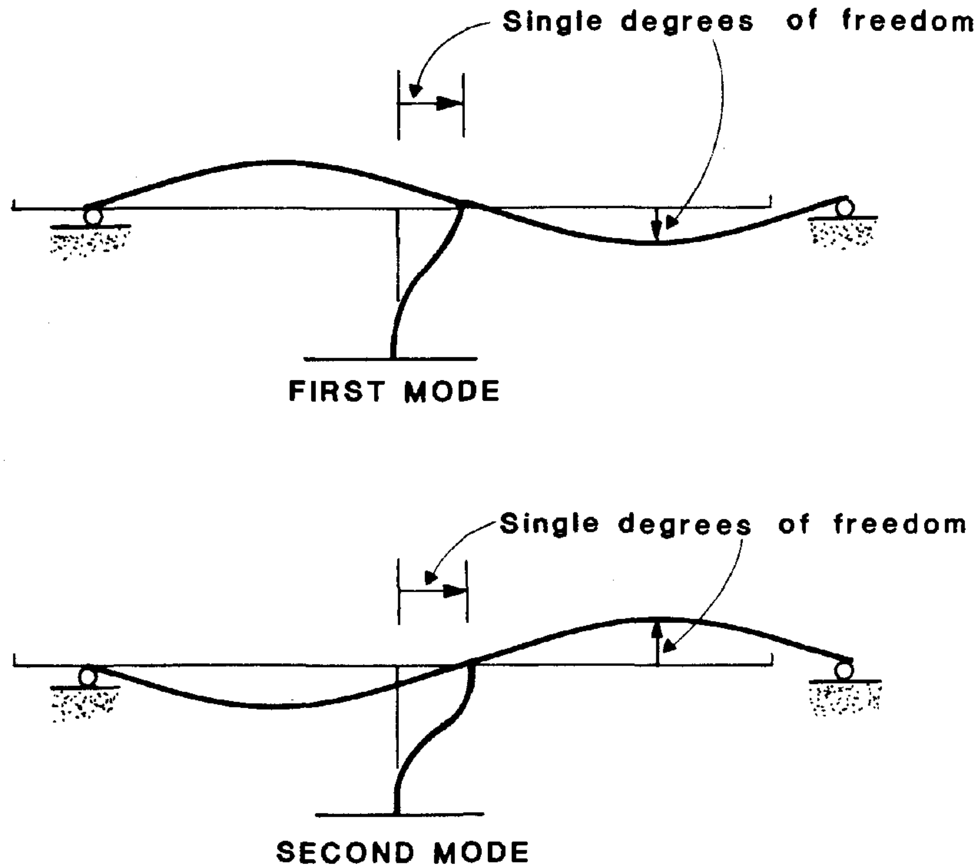


Figure 8.- Continuous two-span bridge. Coupled longitudinal translational and antisymmetric vertical flexural modes of vibration.

deck is not assumed to be rigid in its own plane and lateral flexural motions are expected. These lateral flexural distortions can be approximated adequately as the linear combination of a half-sine wave for the first mode and three half-sine waves, for the second mode (fig. 9). A full sine wave is also a possible distorted shape, but it is not likely to be excited by lateral symmetric ground motions. The lateral flexural distortions are likely to occur in combination with torsional distortions of the bridge deck about a centerline axis parallel to the roadway. The torsional distortions are induced by bending moments at the top of the column, and their existence is dependent upon the torsional stiffness of the bridge deck (the greater the torsional stiffness, the lower the torsional distortions). Since the deck is torsionally restrained only at its ends, it is torsionally most flexible at the column support. At this location, lateral distortion and

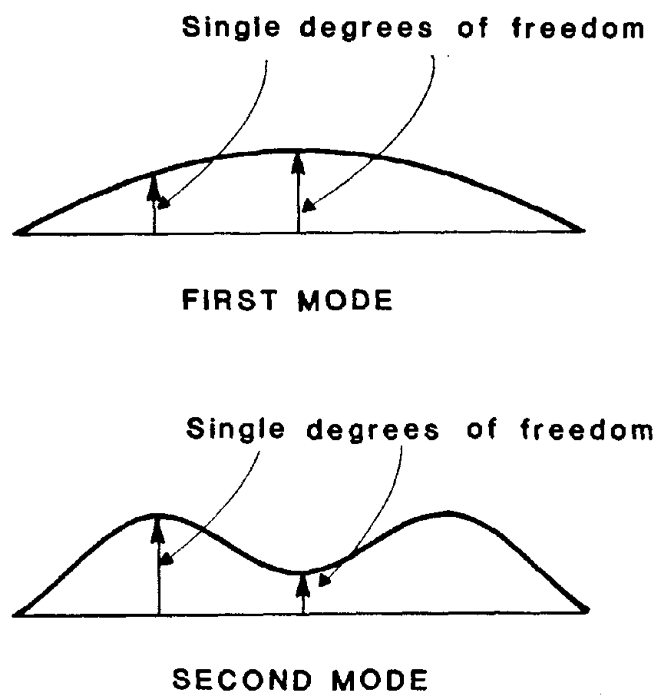
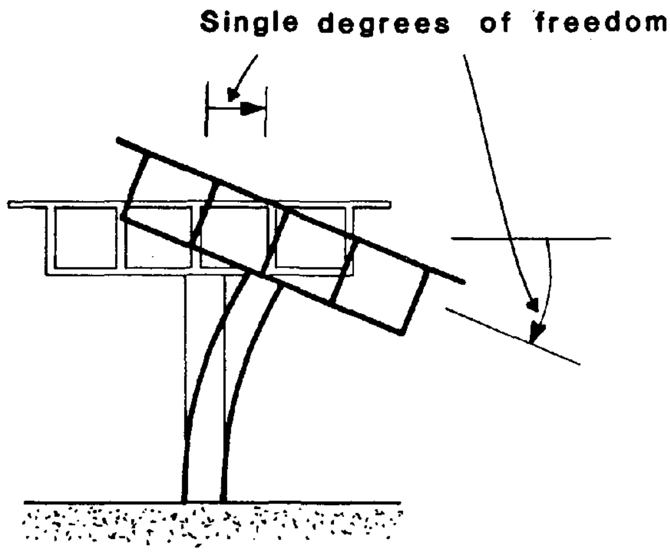


Figure 9.- Continuous two-span bridge. Lateral flexural modes of vibration.

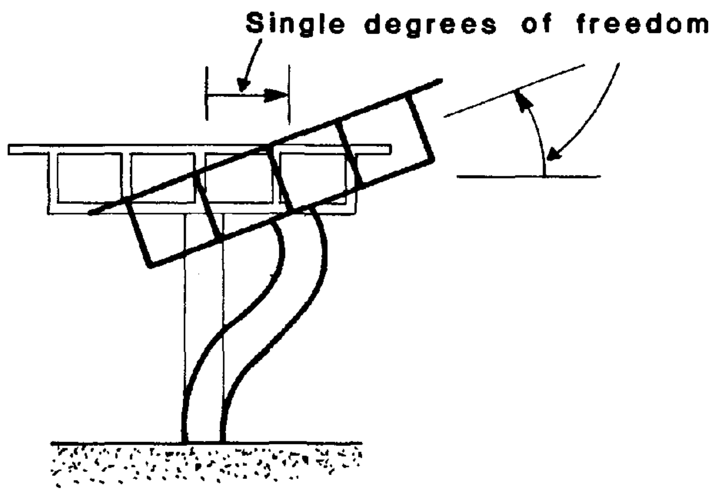
torsional distortion can be expected to combine as shown in figure 10. Again it is likely that torsional distortions along the length of the bridge can be described adequately as some linear combination of a half-sine wave and three half-sine waves. If it is expected that lateral flexural distortions could occur without torsional distortions, the lateral distortions can be described by two degrees of freedom: a lateral translation at the center column and a lateral translation at mid-span of either span (fig. 9). If the lateral distortions are expected to have a torsional component (fig. 10), then two additional degrees of freedom will be required: a rotation at the center column and a rotation adjacent to the mid-span lateral translation.

Assuming the motions of the column and abutments are identical, the only other expected distortions are symmetric vertical flexural distortions. It is likely that the distorted shape of this mode of vibration can be approximated by a vertical dead load deflected shape (fig. 11), and therefore, only one additional degree of freedom is required to describe symmetric vertical flexural distortions: a vertical translation at mid-span on the span not considered in the asymmetric vertical flexure case.

The description of these bridge distortions is complicated if horizontal abutment distortions (relative to the column base) due to a flexible foundation condition are expected. Longitudinal abutment motions are expected to add no significant complexities because the longitudinal roller supports at each abutment are designed to allow free relative movement between bridge deck and abutment. Lateral distortions of the abutments relative to the column base add two additional degrees of freedom: one lateral translation of each abutment. Lateral distortions of the abutment also allow the additional rigid body bridge deck distortions of lateral translation and rotation about a



FIRST COMBINATION OF LATERAL
AND TORSIONAL DISTORTIONS



SECOND COMBINATION OF LATERAL
AND TORSIONAL DISTORTIONS

Figure 10.- Continuous two-span bridge. Combinations of lateral and torsional distortions.

vertical axis. These two rigid body bridge deck distortions are, however, direct functions of the horizontal abutment distortions and do not introduce additional independent degrees of freedom.

In summary, for abutments and foundation material assumed to be rigid, all significant bridge distortions can be described by seven degrees of freedom: one longitudinal translation, two vertical translations, two lateral translations, and two deck rotations about an axis parallel to the roadway center line. If horizontal abutment distortions are also expected, then nine degrees of freedom are required to adequately describe all significant bridge distortions: the above seven plus two lateral abutment translations.

C. Continuous Three-Span Bridge

Consider as the last example the expected distortions of the continuous three-span bridge shown in figure 12. The bridge deck is a continuous reinforced-concrete box girder on simple one-directional roller supports at

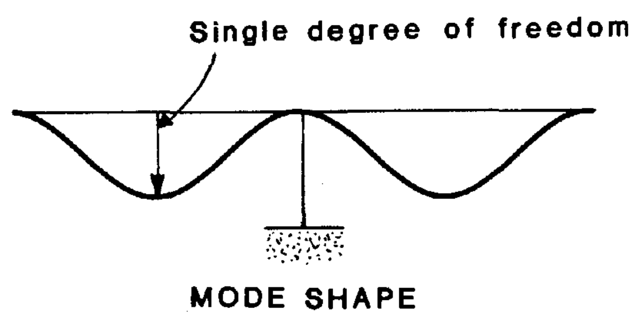


Figure 11.- Continuous two-span bridge. Symmetric vertical flexural modes of vibration.

its ends, and on two intermediate supports. At each intermediate support are two columns cast integrally with the box girder and fixed at their bases. The intermediate supports are not equal in height, making this bridge unsymmetric. Assuming the ground motion is identical at each abutment and at the intermediate supports, the bridge response, too, will be unsymmetric.

Assuming the ground is rigid with respect to the bridge, significant types of distortions expected for this bridge are longitudinal rigid-body translation of the deck (involving flexure of the supporting columns), vertical flexure of the deck, and lateral flexure of the deck. Torsional distortions of the deck about a centerline axis are not expected because the columns at each intermediate support and the roller supports at the bridge ends are assumed to resist torsional motion. Only between supports is the bridge deck free to twist. Since it is anticipated, however, that the ground

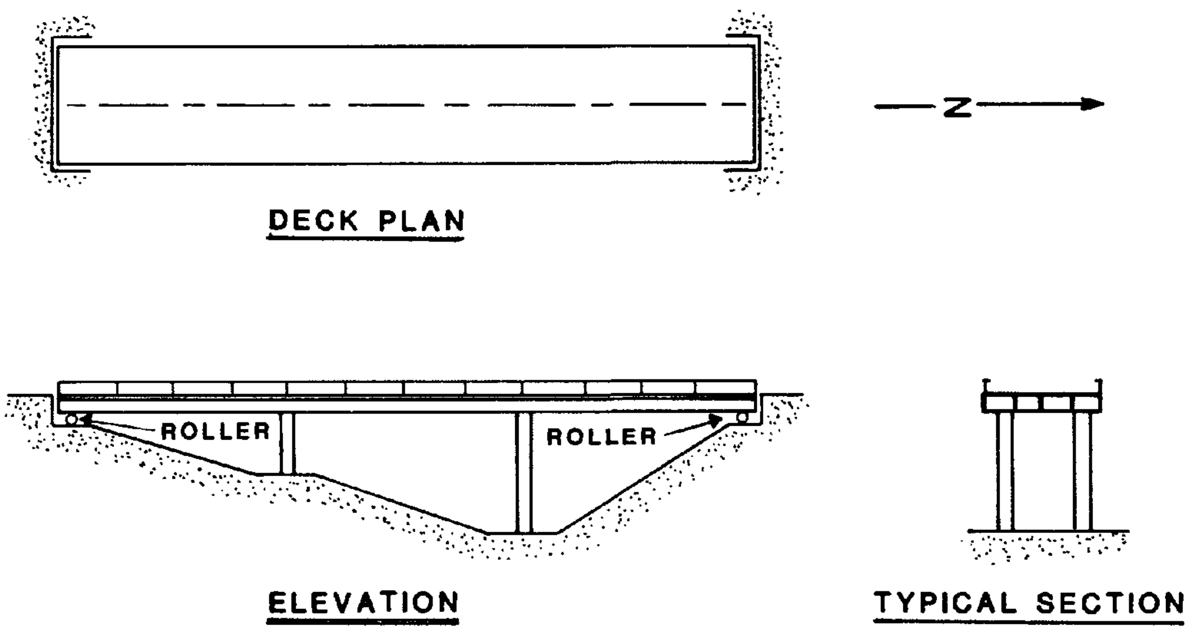


Figure 12.- Continuous three-span bridge.

motion will not contain significant torsional components, these free spans are not likely to be excited torsionally.

If all axial extensions of the bridge deck are ignored, then all significant bridge distortions can be described adequately with seven degrees of freedom. Longitudinal distortions can be described by a single degree of freedom: a longitudinal translation at the top of either intermediate support (fig. 13). Assuming that vertical flexural deck distortions can be approximated as a half-sine wave per span, these vertical distortions can be described adequately with three degrees of freedom: one vertical translation at midspan of each span (fig. 14). Even though continuous boundary conditions between spans will not be satisified with the half-sine approximations, these approximate shapes generally will be sufficiently accurate for the identification of model properties and for the calculation of forces acting on

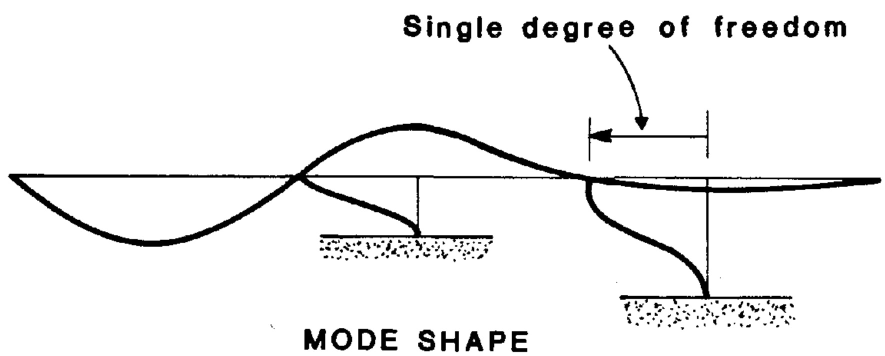


Figure 13.- Continuous three-span bridge. Longitudinal translational mode of vibration (with associated vertical deck flexure and column flexure).

the bridge. Bending moments, however, which are functions of the curvature of these deformed shapes, will be erroneous using these approximations. Assuming that lateral flexural deck distortions can be approximated as the linear combination of a half-sine wave (first mode), full sine wave (second mode) and three half-sine waves (third mode), these lateral distortions can be described with three degrees of freedom: one horizontal translation at mid-span of each span (fig. 15). The reader should note that because the bridge is not symmetric, a distorted shape approximated by a full sine wave is possible even though the ground motion may be symmetric. In other words, symmetry arguments cannot be used for this example bridge.

If the ground is not considered rigid relative to the bridge, then in addition to the distortions for the rigid foundation condition, lateral distortions of the intermediate supports and abutments may be possible.

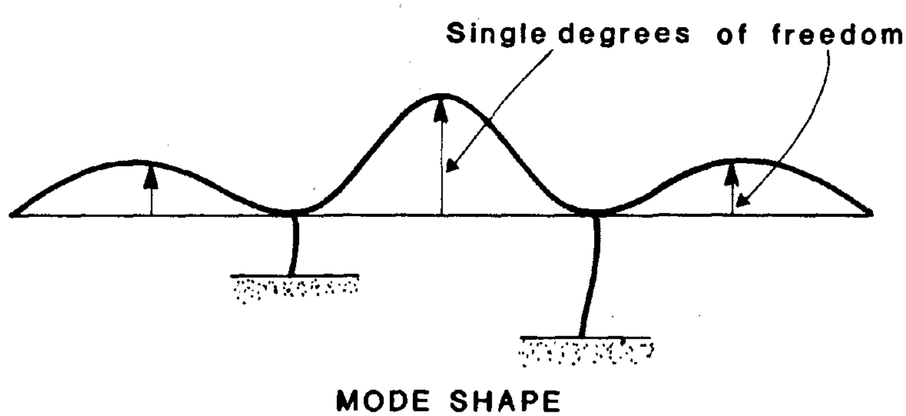


Figure 14.- Continuous three-span bridge. Vertical flexural mode of vibration.

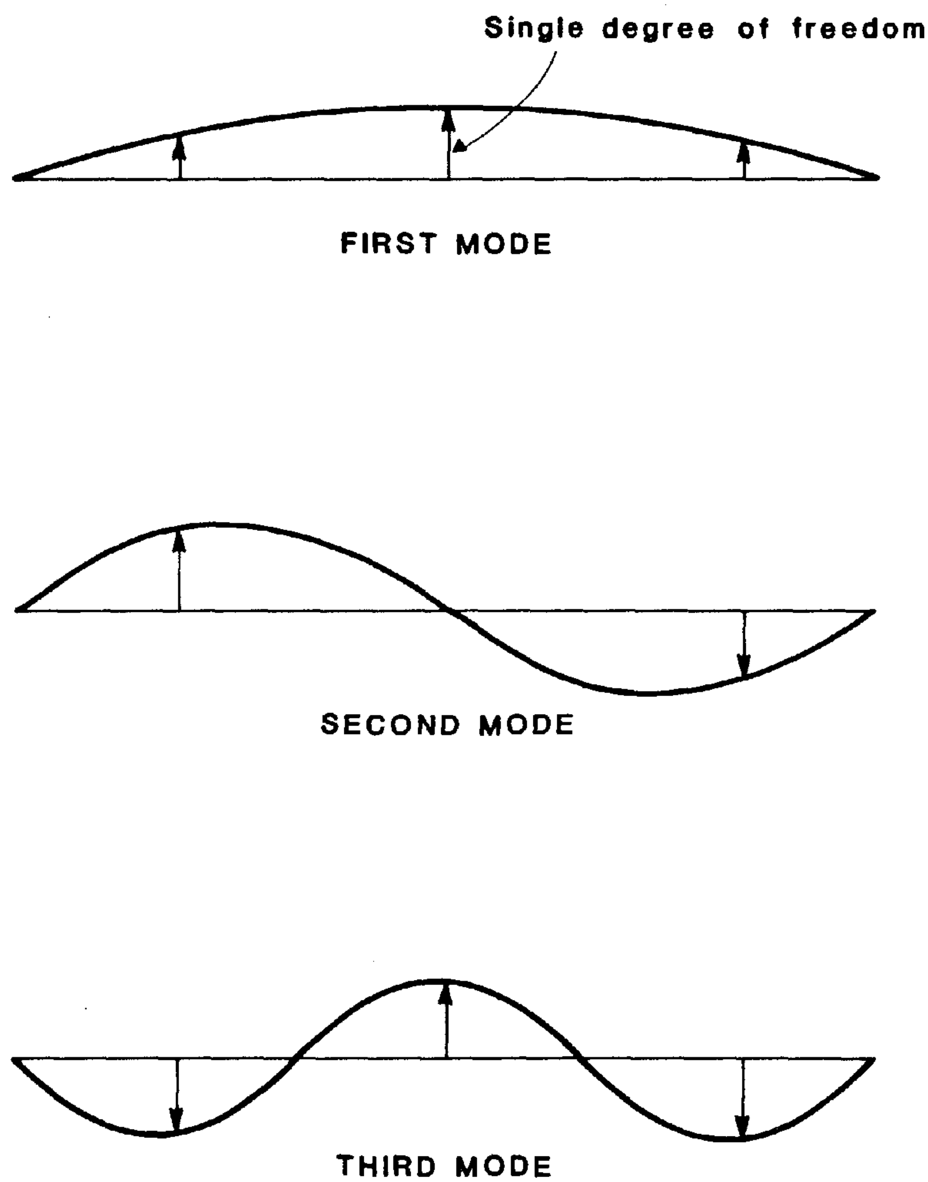


Figure 15.- Continuous three-span bridge. Lateral flexural modes of vibration.

Longitudinal abutment motions, however, are expected to add no significant complexities because the longitudinal roller supports at each abutment are designed to allow free relative movement between bridge deck and abutment. Similarly, the longitudinal flexural and longitudinal rocking distortions of the intermediate column supports normally require no additional degrees of freedom because these distortions can be described collectively by a previously required single degree of freedom: a displacement at the top of either column support in the longitudinal bridge direction. If these distortions are to be differentiated, however, two additional degrees of freedom are required: a rotation about the horizontal axis at the base of each intermediate support.

The lateral distortions that can be expected to occur in the flexible foundation condition consist of four rocking motions, one at the base of each intermediate support and one at each abutment. Lateral rocking distortions at each abutment can be described by a single degree of freedom: the lateral translation of each abutment top (assuming each abutment is rigid). Lateral rocking distortions of the intermediate supports can also be described by one degree of freedom: a rotation at the base of each intermediate support about a horizontal axis parallel to the roadway centerline. Even though the lateral rocking distortions of the intermediate supports allow for the possibility of additional lateral rigid body translation of the deck (that is, translation in addition to that resulting from lateral flexural distortion of the intermediate support columns) as well as warping of the deck about a roadway centerline axis, no additional degrees of freedom are required to identify these additional distortions.

In summary, for abutments and foundation material assumed to be rigid, all significant bridge distortions can be described by seven degrees of freedom: one longitudinal translation, three vertical translations, and three lateral translations. If the foundation material is not rigid and rocking motion is possible, then eleven degrees of freedom are required to adequately describe all significant bridge distortions: the above seven plus two lateral translations (one at the top of each abutment) and two rotations (one at the base of each intermediate support around an axis parallel to the roadway centerline).

V. FAILURE OF HIGHWAY BRIDGES IN THE 1971 SAN FERNANDO EARTHQUAKE

Although the theory of linear dynamic bridge response provides the basic information required to efficiently instrument highway bridges, an understanding of the failure mechanisms of actual highway bridges during past earthquakes is also important. Not only does the historic record of highway bridge failures emphasize the critical aspects of bridge behavior during earthquakes, but it also clearly reveals those types of structures that are most vulnerable to earthquakes, structures that are excellent candidates for instrumentation. Following, therefore, is a discussion of the failure of highway bridges in the magnitude 6.4 1971 San Fernando, California earthquake. That earthquake caused substantially more damage to recently built highway bridges than any other earthquake to date in U.S. history. Moreover, the highway bridges damaged were similar in design and geometric layout to those currently being built or already situated in the seismically active areas of the U.S.

Of the approximately 62 highway bridges damaged by the San Fernando earthquake, approximately 25 percent were heavily damaged, 50 percent sustained moderate damage, and 25 percent sustained relatively minor damage (Elliott and Nagai, 1973). Two of the heavily damaged bridges totally collapsed and three others partially collapsed; all five were located within 3 miles (5 km) of the surface fault rupture zone. Damages to four of the partially or totally collapsed bridges and one other heavily damaged bridge are described in the following paragraphs. These damage descriptions have been extracted from a previously published report on the earthquake's effects on freeway bridges (Elliott and Nagai, 1973). That report also contains detailed descriptions of several other heavily damaged bridges as well as a tabular summary of all bridge damages sustained as a result of the San Fernando earthquake.

San Fernando Road Overhead (Widen) - The San Fernando Road Overhead bridge was a multi-span structure originally built in 1955 and widened just prior to the 1971 earthquake. It was located adjacent to the Route 5/210 Interchange (fig. 16) approximately 2.5 miles (4.0 km) from the surface fault rupture zone. With the exception of one span over a railroad line, the superstructure was of reinforced-concrete box-girder construction; the remaining span was bridged by steel girders (original construction) and precast prestressed girders (widened section) (fig. 17).

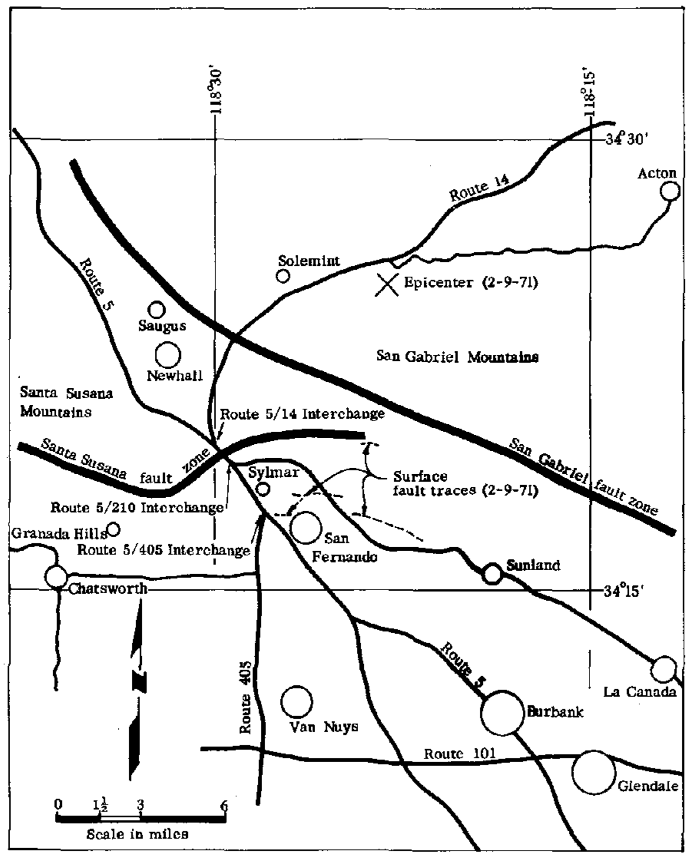


Figure 16.- Freeways in the region affected by the 1971 San Fernando, California earthquake.

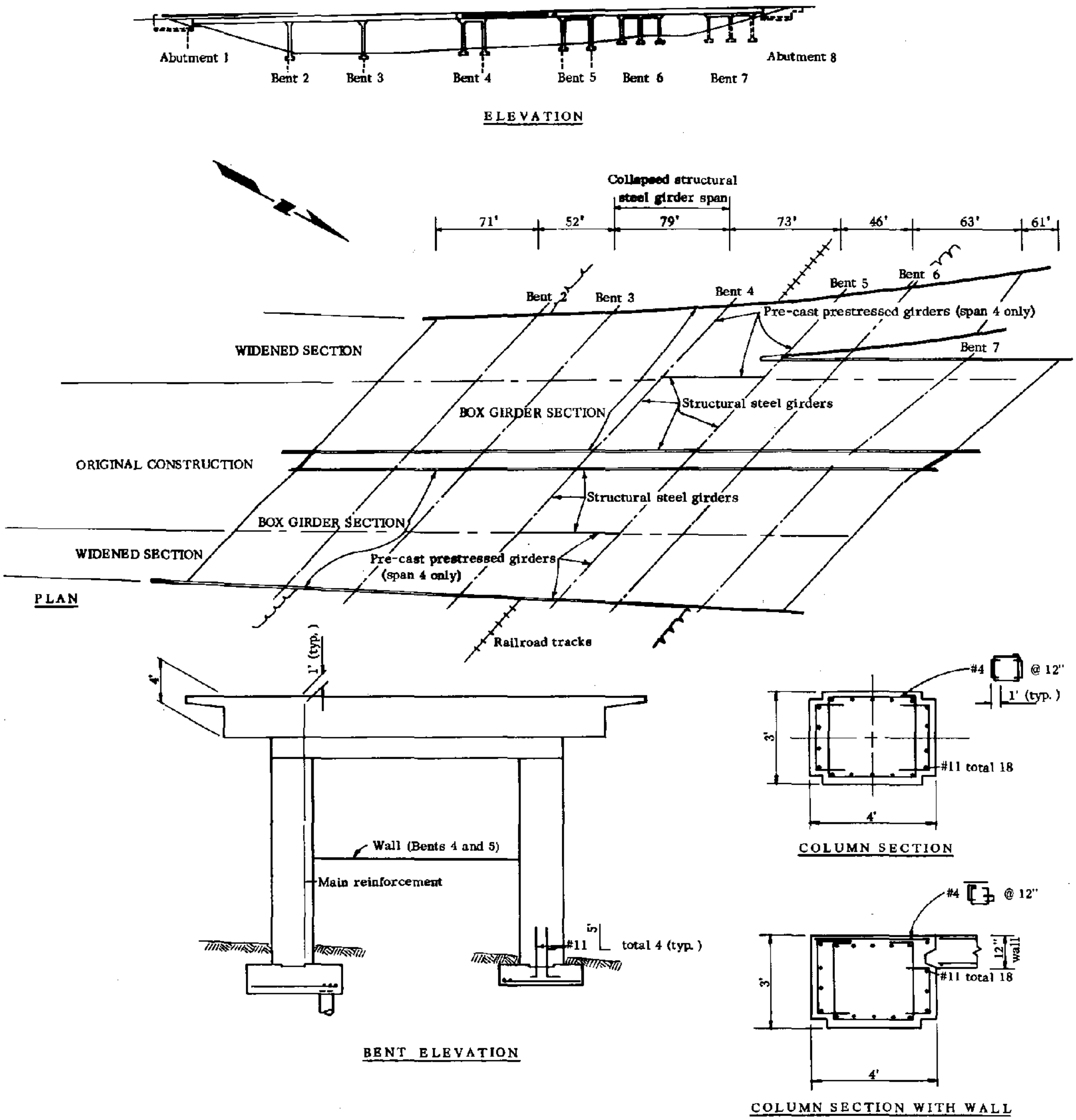


Figure 17.- San Fernando Road Overhead (Widen) bridge. Construction details.

Although the bridge sustained concrete damage at the column deck connection (spalling) and near the column mid-height (shear fracturing) in some columns, the most serious damage was the collapse of the steel girder span (see fig. 18). The steel girders were on bearing bars with the ends of the girder projecting 16 inches (40.6 cm) onto the bent seats. Apparently the distance between the tops of bents 4 and 5 elongated enough during the earthquake to allow the steel girders to fall between the bents. The adjacent precast prestressed girders, however, did not fall; those girders projected 26 inches (66.0 cm) into the seat and apparently that distance was sufficient to keep the girders in place and undamaged.

San Fernando Road Overhead - The San Fernando Road Overhead bridge, located in the Route 5/210 Interchange approximately 2.5 miles (4.0 km) from the surface rupture zone (near the multi-span structure described above, fig. 16), was a

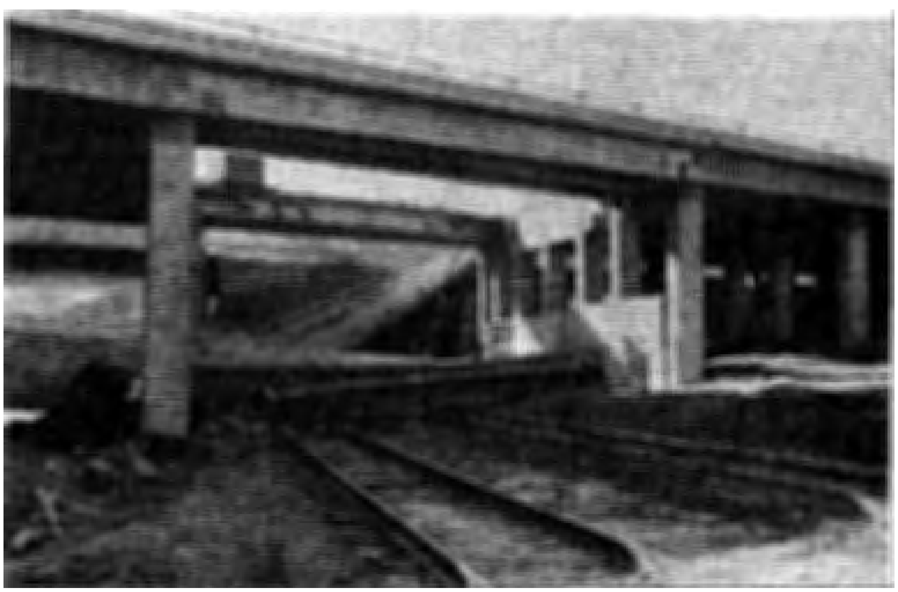


Figure 18.- San Fernando Road Overhead (Widen). Collapsed steel girder span between widening sections of precast prestressed girders. California Division of Highways, Bridge Department photograph.

cast-in-place prestressed structure with two 122-foot-long continuous spans supported by a single reinforced-concrete column (fig. 19).

During the earthquake the base of the supporting column was shattered and the bridge deck sagged approximately four feet (1.2 meters) (fig. 20). The spacing of the hoop ties in the column (#4 @ 12") was apparently too large to contain the concrete and to prevent the vertical reinforcing bars from buckling (figure 21).

Route 5 (Truck Lane)/405 Separation - The Route 5 (Truck Lane)/405 Separation bridge, located less than 0.5 miles (0.8 km) from the surface fault rupture zone (fig. 16), was a cast-in-place prestressed box girder structure with two 177-foot-long continuous spans supported by a two-column bent (fig. 22).

During the earthquake, the bent columns were completely shattered, causing the bridge deck to collapse (fig. 23) and to fracture on both sides of the bent (fig. 24). Either the spacing of the hoop ties in the bent columns (#4 @ 12") was too large or the size or splicing of the hoops was insufficient to contain the concrete and to prevent the vertical reinforcing bars from buckling (fig. 25).

Northwest Connector Overcrossing - The Northwest Connector Overcrossing bridge, located in the Route 5/210 Connector Overcrossing bridge approximately 2.5 miles (4.0 km) from the surface fault rupture zone (fig. 16), was a curved, highly skewed, four-span reinforced concrete box girder structure supported by three four-column bents (fig. 26). At the abutments the bridge superstructure sat on elastomeric bearing pads.

During the earthquake, the superstructure of the bridge rotated between 3 to 5 feet (.9 to 1.5 meters) about a vertical axis, causing one end to drop off the abutment (fig. 27). The tops of the columns of bent 2 (fig. 28) and

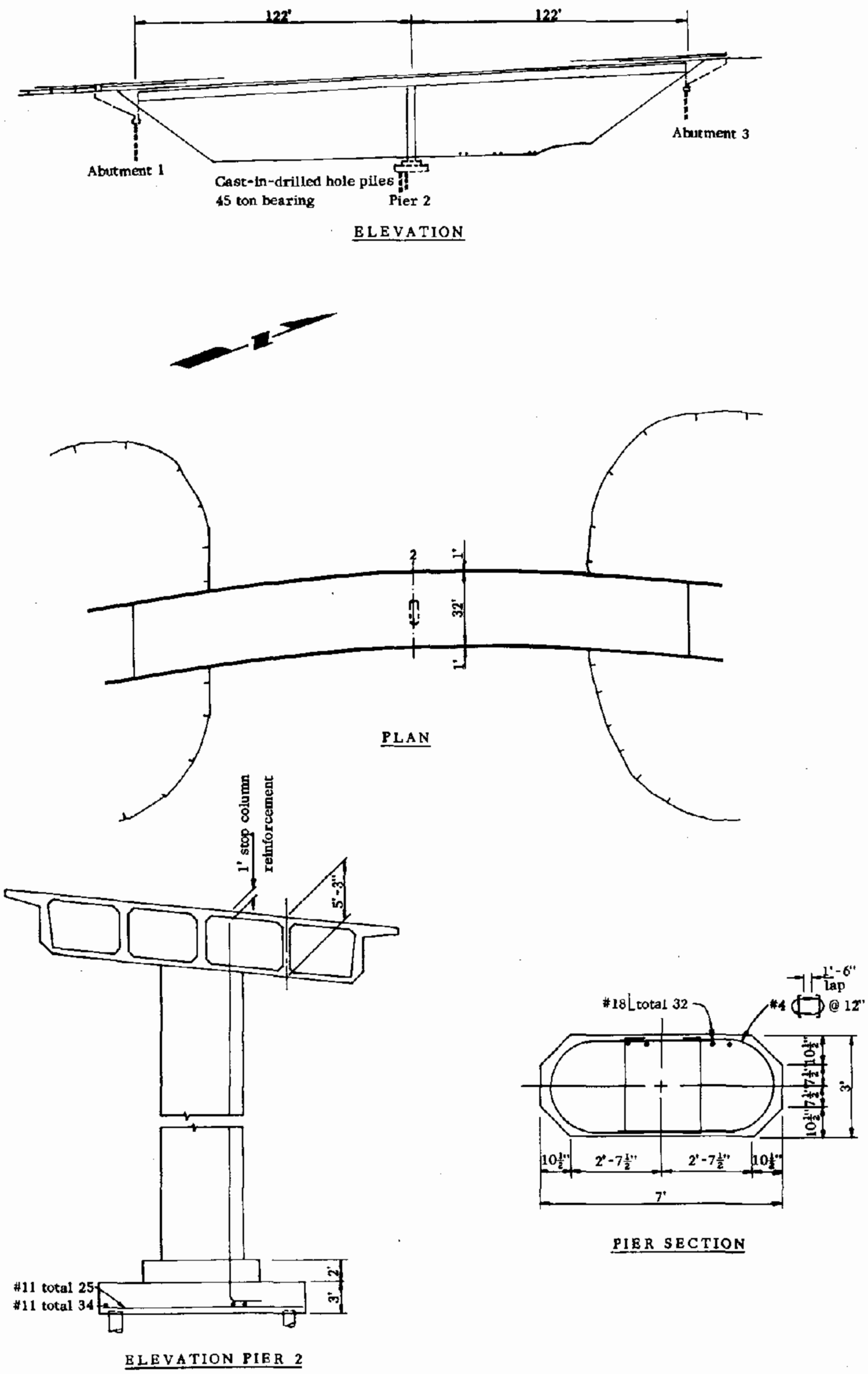


Figure 19.- San Fernando Road Overhead bridge. Construction details.

Figure 20.- San Fernando Road Overhead bridge. Damaged column. Note cracks in superstructure on either side of bent cap. California Division of Highways, Bridge Department photograph.

Figure 21.- San Fernando Road Overhead bridge. Closeup of column base. Note flareout of vertical reinforcing bars. California Division of Highways. Bridge Department photograph.

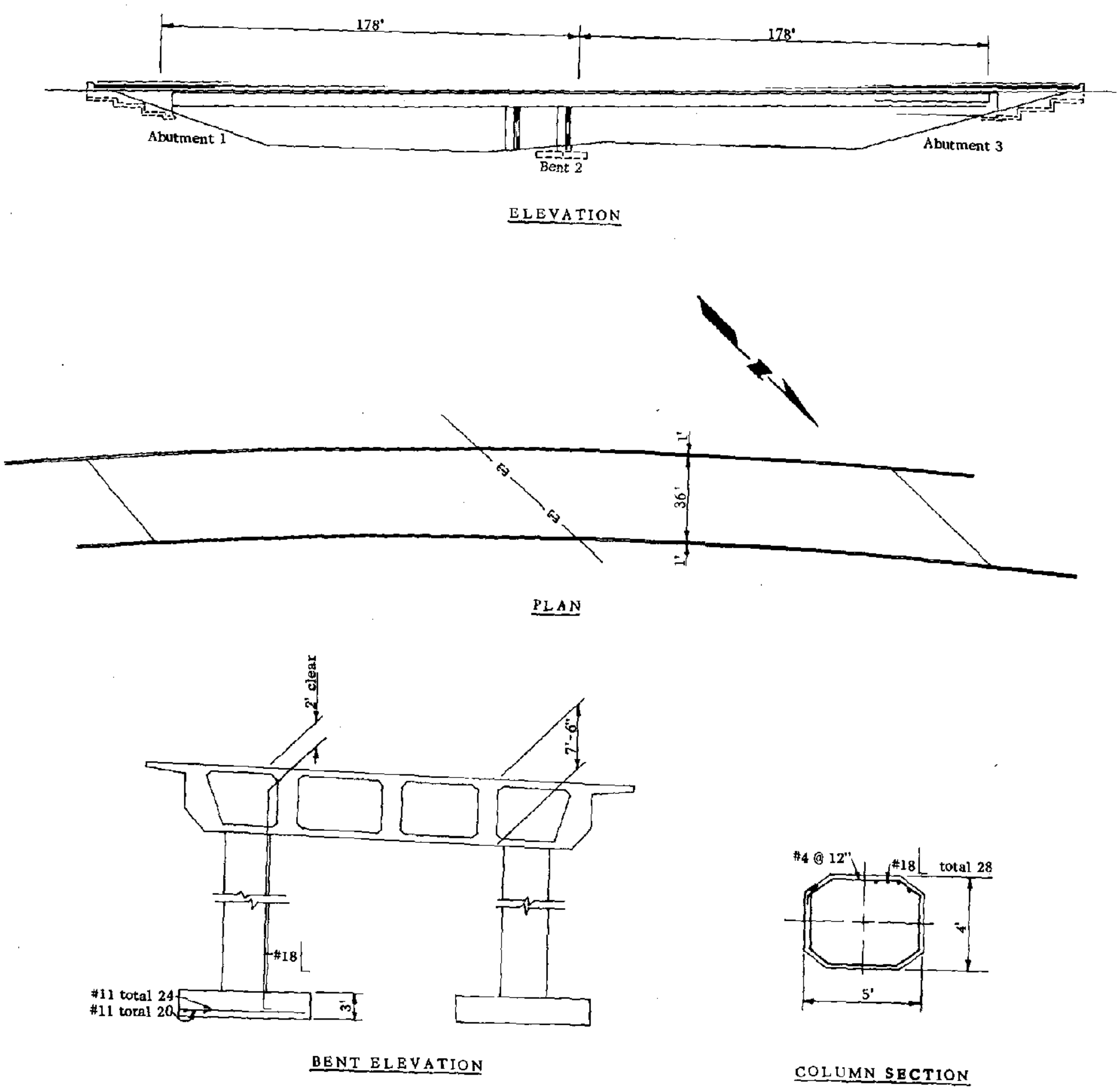


Figure 22.- Route 5 (Truck Lane)/405 Separation bridge. Construction details.

Figure 23.- Route 5 (Truck Lane)/405 Separation bridge, looking north. California Division of Highways, Bridge Department photograph.

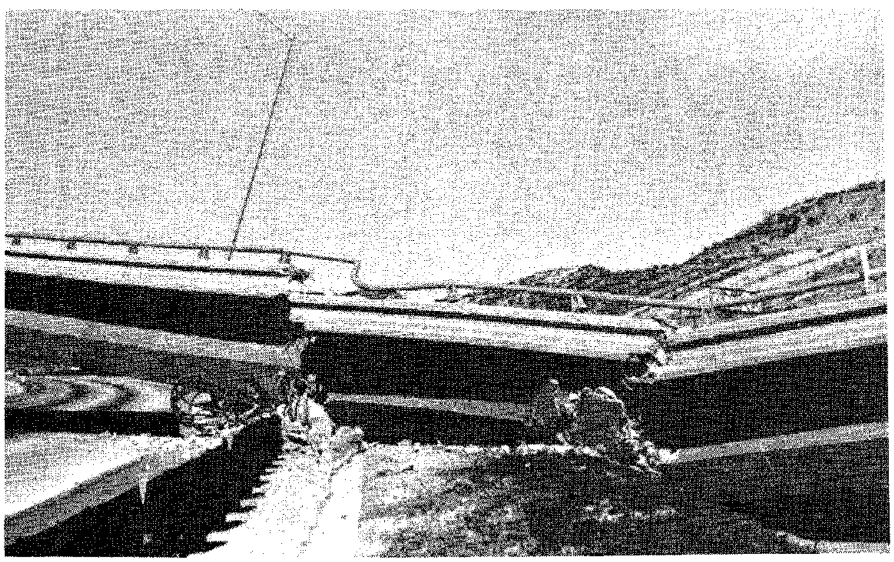


Figure 24.- Route (Truck Lane)/405 Separation bridge. Breaks in superstructure on both sides of shattered bent columns. California Division of Highways, Bridge Department photograph.

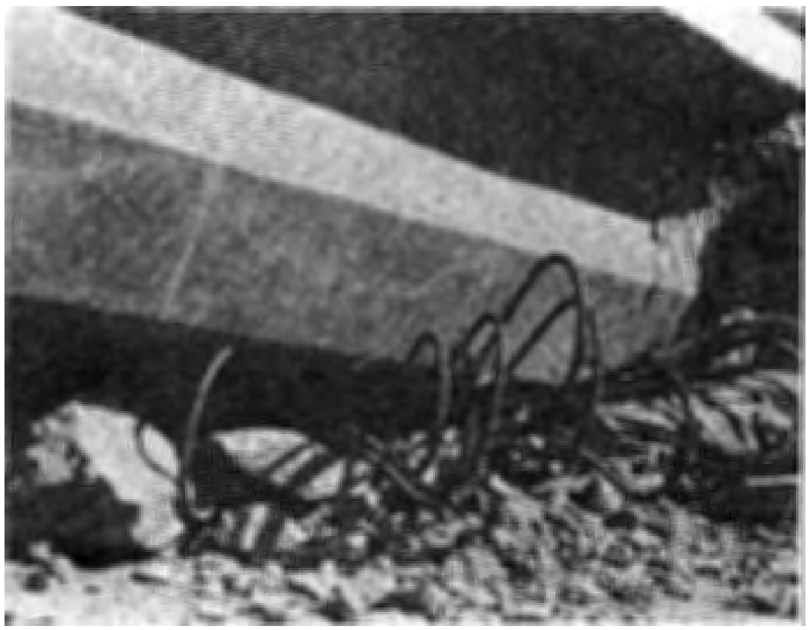


Figure 25.- Route 5 (Truck Lane)/405 Separation. Remains of shattered south column of two-column bent. Note buckled vertical reinforcing bars. California Division of Highways, Bridge Department photograph.

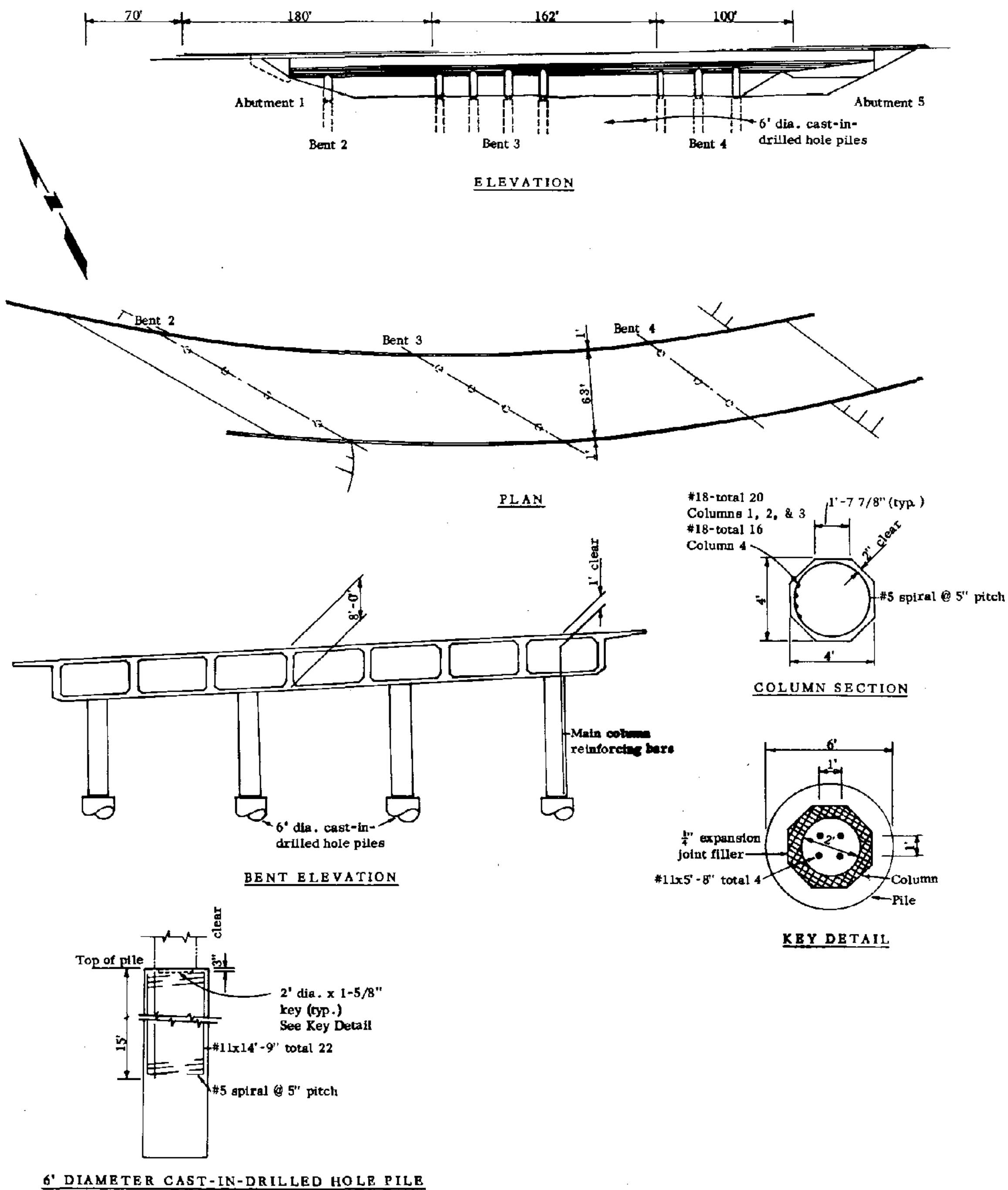


Figure 26.- Northwest Connector Overcrossing bridge. Construction details.

Figure 27.- Northwest Connector Overcrossing bridge. Note that the superstructure has dropped off the seat of abutment 1 in the right portion of the photograph. California Division of Highways, Bridge Department photograph.

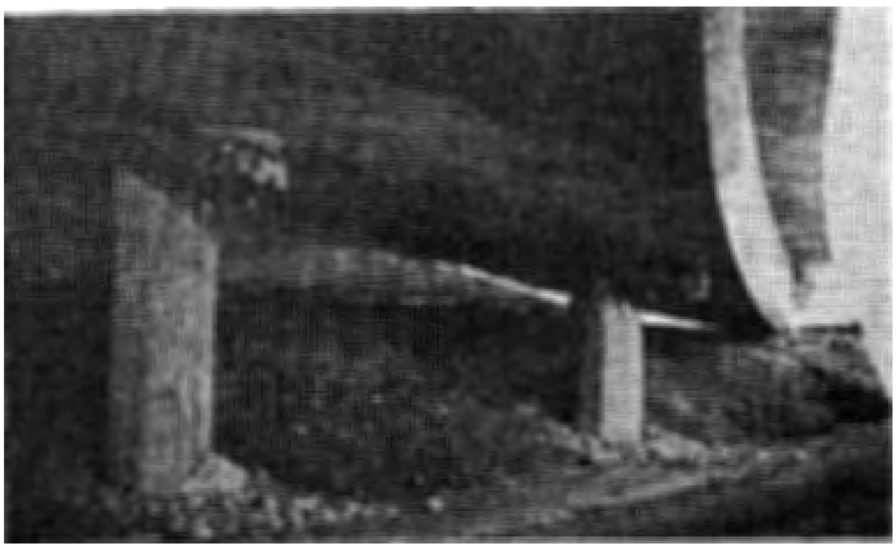


Figure 28.- Northwest Connector Overcrossing bridge. Bent 2 column. Note severe crushing of concrete at tops of columns as well as displacement due to superstructure rotation. California Division of Highways, Bridge Department photograph.

bent 4 (fig. 29) were severely crushed and displaced due to the rotation of the superstructure. The columns of bent 3, located near the center of rotation of the bridge superstructure, suffered only minor spalling damage.

Route 5/210 Separation and Overhead - The Route 5/210 Separation and Overhead bridge, located approximately 2.5 miles (4.0 km) from the surface fault rupture zone (fig. 16), was a seven-span 770-foot-long reinforced concrete girder structure on an 800-foot (240-meter) radius (fig. 30). At the abutments the bridge superstructure sat on elastomeric bearing pads.

During the earthquake, the entire bridge superstructure toppled toward the west (outside of curve) and collapsed in several sections on the ground (figs. 31 and 32). Contributing to the collapse were: (1) the failure of three out of four of the 20-inch-thick north and south abutment wingwalls due to lateral

Figure 29.- Northwest Connector Overcrossing bridge. Bent 4 columns. Note severe crushing of concrete at tops of columns as well as displacements due to superstructure rotation. Note also fallen superstructure of adjacent bridge at right of photo. California Division of Highways, Bridge Department photograph.

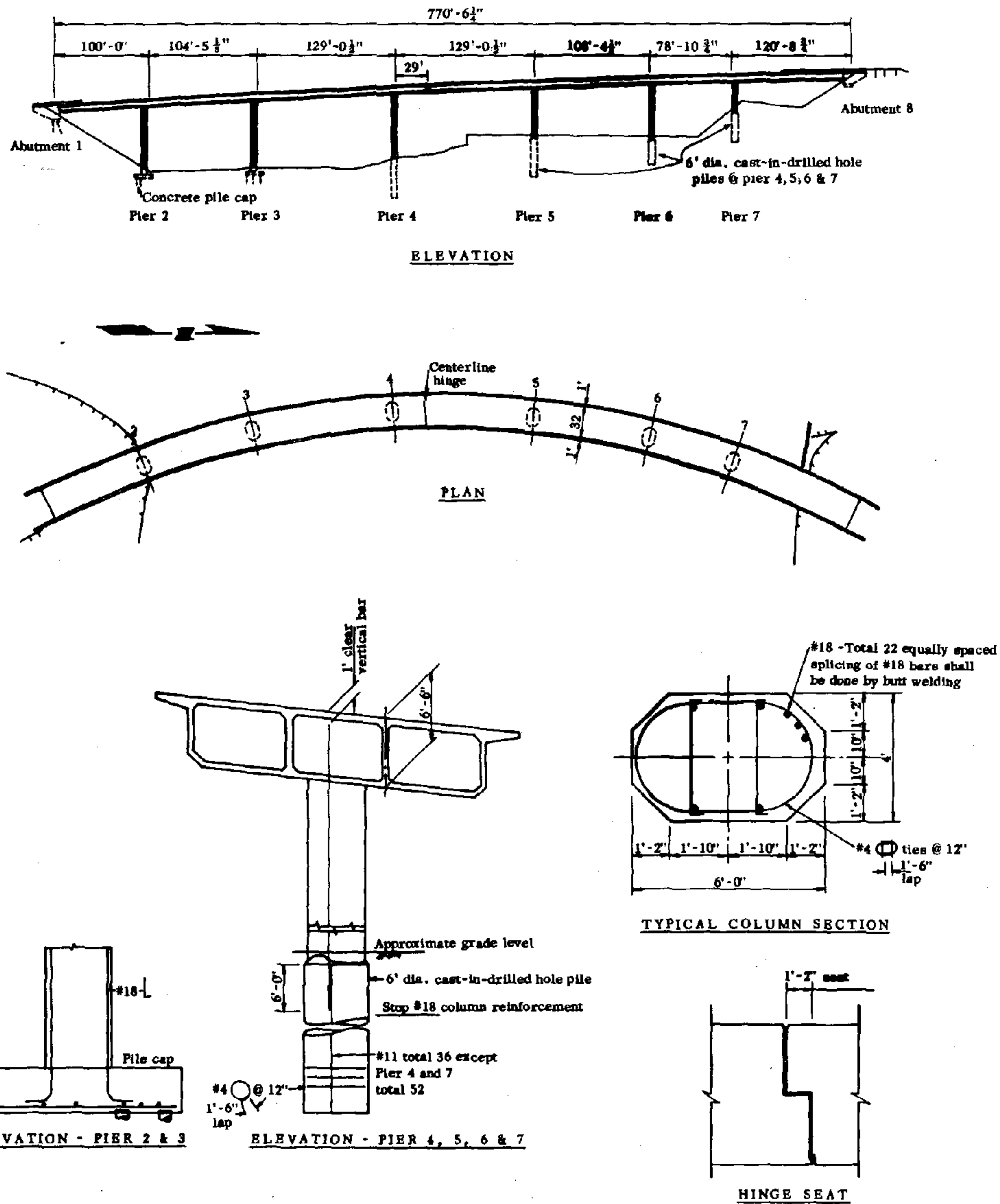


Figure 30.- Route 5/210 Separation and Overhead bridge. Construction details.

Figure 31.- Route 5/210 Interchange, looking northeast. Route 5 crosses lower portion of photograph and Route 210 approaches the interchange from the top. Note collapsed Separation and Overhead bridge near center of photograph. California Division of Highways, Bridge Department photograph.

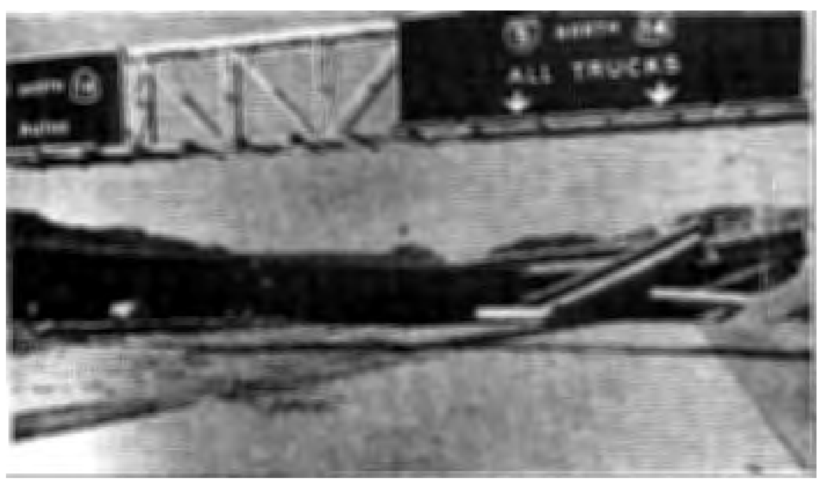


Figure 32.- Route 5/210 Interchange Separation and Overhead bridge. Portion of collapsed superstructure. Columns from left to right are Piers 5, 6, and 7. California Division of Highways, Bridge Department photograph.

battering action of the superstructure (figs. 33 and 34); (2) the bond failure at the base of all columns that allowed the core of vertical reinforcing bars to pull out of the pile caps at piers 2 and 3 (fig. 35) and out of the cast-in-place drilled-hole (CIDH) piles at piers 4, 5, 6, and 7 (fig. 36); and (3) the lack of longitudinal restrainers at the hinge in the bridge superstructure between piers 4 and 5 (fig. 30). A concrete key prevented differential lateral motions at the hinge, but there was no restraint to prevent longitudinal differential motion, i.e., the north and south segments of the superstructure were free to pull away from each other.

In summary, and based on the bridge failures described above as well as the failures in the other approximately 57 bridges damaged by the San Fernando earthquake, it is apparent that insufficiencies in design and construction details have been largely responsible for earthquake induced highway bridge failures. More specifically, the most severe damages appear to have resulted from: (1) inadequate hoop tie bar size, spacing, and splicing in reinforced concrete columns, particularly at the column base; (2) superstructure seats (on bents and abutments) that were too narrow in the longitudinal direction; (3) inadequate horizontal and vertical restraints at these seats as well as at superstructure hinges; (4) the rotation of skewed structures; and (5) the bond failure of vertical reinforcing bars at column bases. It is noteworthy that in the San Fernando experience, the superstructure showed few signs of distress, even in those cases where the supporting columns collapsed or the superstructure was rotated substantially.

Because strong-motion records from damaged bridges are considered to be the most important and desirable types of records that could be obtained,

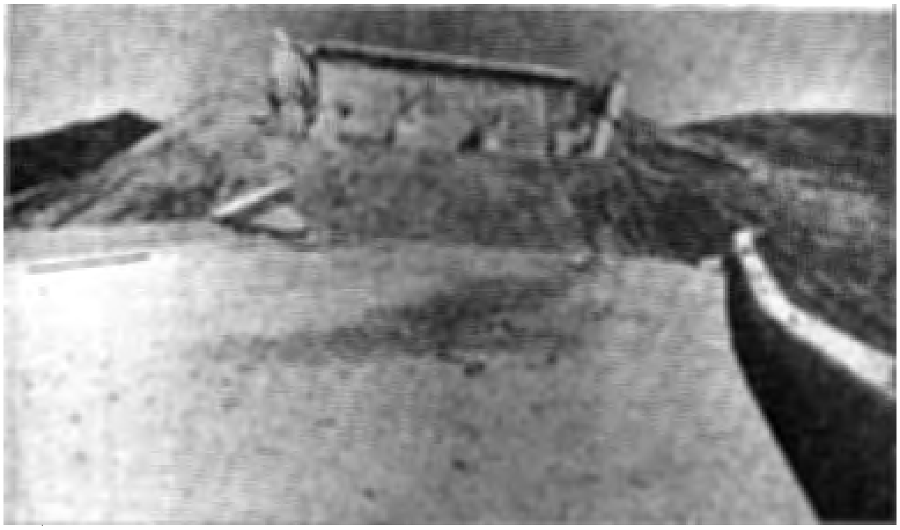


Figure 33.- Route 5/210 Separation and Overhead bridge. Abutment 8 (north abutment). Note shattered wingwall due to lateral battering action of bridge superstructure. California Division of Highways, Bridge Department photograph.

Figure 34.- Route 5/210 Separation and Overhead bridge. Portion of collapsed superstructure and pier 2 column. Abutment 1 (south abutment) is at top of embankment. Note that the right wingwall is completely sheared off whereas the left wall is undamaged. California Division of Highways, Bridge Department photograph.

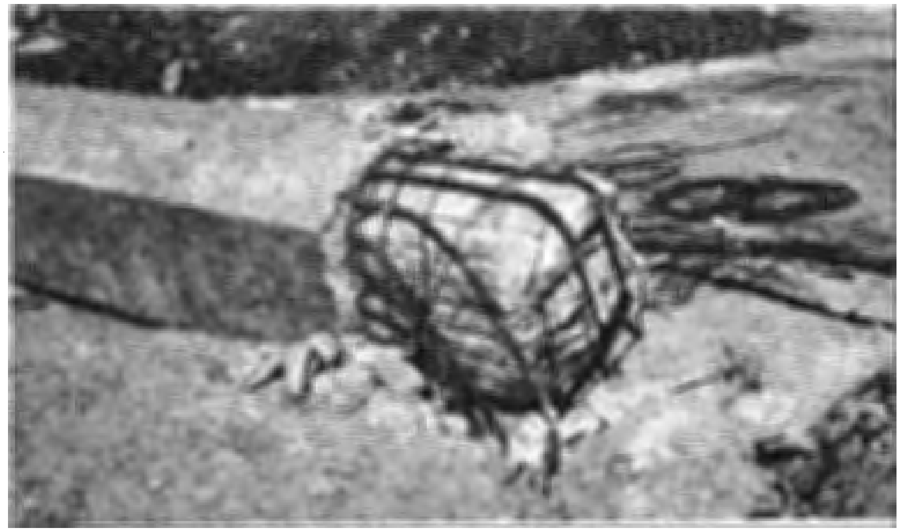


Figure 35.- Route 5/210 Separation and Overhead bridge. Bottom of pier 2 column. Core of column pulled out of pile cap due to bond failure. Califoria Division of Highways, Bridge Department photograph.

Figure 36.- Route 5/210 Separation and Overhead bridge. Bottom of pier 5 column at construction joint. Reinforcing bars were pulled cleanly out of the CIDH pile shaft due to bond failure. California Division of Highways, Bridge Department photograph.

damage summaries such as the one described above provide valuable guidance on which types of bridges are ideal candidates for strong-motion instrumentation. In the case of the San Fernando earthquake, the summaries strongly suggest that skewed bridges, bridges having simply-supported spans, bridges with high non-continuous curved superstructures, and some types of continuous two-span bridges may be highly susceptible to earthquake-induced damage; such bridges, therefore, are certainly appropriate candidates for strong-motion instrumentation. Studies of damages to other bridges in other earthquakes have yielded similar findings.

Damage summaries such as the one described above also illustrate where damages can be expected to occur in a variety of types of bridges and therefore help to identify the locations where displacement and loading histories are the most critical. In the case of skewed bridges, for example, the San Fernando studies indicate that strong-motion instrumentation schemes should be designed to record relative horizontal motions between the bridge deck and each abutment as well as the overall rotation of the deck (around a vertical axis). These studies also indicate that in the case of bridges with simply-supported spans, it is important to record relative motions between the bridge deck and the various supports (abutments and interior piers), whereas in the case of bridges with non-continuous superstructures, and particularly those with superstructures that are relatively high and curved, it is important to record relative motions between deck sections separated by expansion joints. In the case of continuous two-span highway overpass bridges, the San Fernando studies indicate that strong-motion recordings of lateral and vertical motions near the central support bent would provide critically needed information on lateral and vertical loads prior to and during bent failure, should it occur.

VI. RECOMMENDED INSTRUMENTATION SCHEMES

The extent and type of strong-motion instrumentation that should be installed on and near highway bridges are dependent on three primary factors: (1) the objective of the instrumentation program; (2) the expected behavior of the bridge, including potential failure mechanisms; and (3) the quantities of motion that are to be measured. Although all factors must be considered simultaneously in the design of an instrumentation scheme, the first factor is of particular importance, i.e., the instrumentation must yield data that satisfies the objectives of the instrumentation program. If the goal of the instrumentation program is to obtain data that can be used to improve engineering design and construction practice, or to evaluate the potential earthquake hazard of existing bridges, the instrumentation scheme should have one of the following primary objectives: force level determination; mathematical model identification; or mathematical model verification. The amount of instrumentation that is required on the superstructure to satisfy each of these objectives is objective dependent, with the first two objectives (force level determination and model identification) requiring substantially more instrumentation than the third objective (model verification). The amount of instrumentation that is required at ground level, however, is objective independent. Such instrumentation is solely dependent on the expected ground and bridge support motions and will not vary according to the instrumentation scheme objective.

Following are discussions regarding instrumentation schemes recommended for installation at ground level and instrumentation schemes recommended for installation on the bridge superstructure. The recommended instrumentation schemes assume that all bridge motions will be recorded by acceleration recording instruments, which are currently considered to be the most desirable type of instrumentation for recording strong-motion (see Appendix).

A. Ground-Level Instrumentation

In all cases, ground-level instrumentation should be installed at the base of one or more bridge supports and at one or more "free-field" sites. The purpose of the free-field instrumentation is to obtain records of ground motion that are not influenced by (do not include) bridge response. In conjunction with the motions recorded at the base of one or more bridge support(s), free-field motions provide information on the extent to which soil-structure interaction has occurred at the site, that is, the extent to which the bridge mass and stiffness have influenced motion recorded at the base of the bridge support(s). Even in cases where foundation conditions suggest that it is highly unlikely that soil-structure interaction might occur, at least one free-field site should be instrumented in order to validate that assumption. Because of the intended use of free-field recordings, it is important that free-field instrumentation be installed near the bridge on similar foundation conditions, but that it not be located immediately adjacent to the structure. In order to estimate the distance from the instrumented structure for such a site, we recommend that the same criterion be followed for bridges as is presently utilized for buildings in the California Strong-Motion Instrumentation Program. This criterion specifies that sites intended to be free-field should be at a distance from the instrumented structure equal to 1 to 1 1/2 times the estimated wavelength of a shear wave (at the surface) having a period equal to the fundamental period of the instrumented structure. Assuming then, for example, that the surface shear-wave velocity at the site is 700 ft/s and the fundamental period of the bridge is 0.5 s, the free-field site should be located between 106.5 m (213 m/s x 0.5 s) and 160 m (1 1/2 x 106.5 m) from the instrumented bridge.

As a minimum, it is recommended that one orthogonal triaxial package of accelerometers (2 accelerometers oriented horizontally and one oriented vertically) be installed at one free-field site per each instrumented bridge. Ideally, however, the instrumentation of three such sites per instrumented bridge is much more desirable. These sites, each instrumented with an orthogonal triaxial accelerometer package, should be located at the corners of an equilateral triangle with the bridge located at the center of the triangle. Such a scheme would provide important information on the direction of seismic-wave form arrivals, on wave phase lags, and on the overall three-dimensional response of the ground surface in the vicinity of the bridge. It is recognized that in some cases it may not be possible to find even one suitable free-field site due to the proximity of other structures (e.g., in metropolitan areas). In such cases, it must be recognized that the quality of any recorded data will be severely limited; in other words, such structures should be instrumented only if no other alternatives are available.

In selecting the amount of instrumentation to be installed at the base of the bridge supports, it is first necessary to decide which one of the following three basic ground motion conditions is expected to exist at the site: (1) the ground motion is expected to be identical (or nearly so) beneath each bridge support; (2) the ground motion is not expected to be identical beneath each support even though the foundation material is considered to be rigid relative to the bridge structure; and (3) the ground motion is not expected to be identical to the support motion because the foundation material is not rigid relative to the bridge structure and soil-structure interaction is expected to occur.

Condition 1 exists if the entire bridge is situated on firm relatively homogeneous foundation material and if the bridge length (or length of spans of interest) is less than what we shall refer to as the critical distance, L_{cr}. The critical distance is primarily dependent on the following two factors: (1) apparent or actual horizontal propagation velocity of the ground motion; and (2) the fundamental or lowest natural frequencies of the bridge (or spans of interest). These factors provide estimates of the maximum phase lag that can be expected to occur between acceleration histories recorded at various points on the bridge. Such phase lags are considered by the authors to be significant in force determination and mathematical modelling studies if they exceed 6°, which would result in wave amplitude errors as large as 10% at any time. Since components of motion with frequencies close to those of the fundamental frequencies of the bridge are presumed to be the most critical components during earthquake response, we recommend the following equation be used to estimate the maximum spacing between instruments located at the base of bridge supports:

$$L_{cr}(n) = T_m \times c \times 6°/360° \qquad (1)$$

where $L_{cr}(n)$ = recommended maximum spacing between instruments located at the base of bridge supports, for each dominant mode of of response, n,

T_n = natural period of dominant mode of response, n, and

c = apparent or actual horizontal propogation velocity of the dominant seismic-wave forms (c = 3,000 m/s for S-body waves, normally the dominant wave forms at locations close-in to the source region of shallow earthquakes (Fung, 1965); c = 2,700 m/s (approximately) for surface waves, which may be dominant at larger distances).

In the case of $c = 3{,}000$ m/s (S-body waves are expected to be dominant) and $T_1 = 0.5$ s (a typical value for the longitudinal fundamental natural period of some highway overpass bridges (or spans)), $L_{cr}(1)$ is estimated to be approximately 25 m. This estimate suggests, then, that in those situations where the length of such a bridge (or span) is longer than 25 m, the ground motions that are expected to be dominant in exciting longitudinal response are not expected to be in phase beneath each bridge support. That is, condition 1 is not expected to exist.

In those rare instances where the ground motion is expected to be identical (or nearly so) beneath each support (condition 1), we recommend that one orthogonal triaxial package of accelerometers be installed at the base of the bridge support nearest the mid-way point of multi-span bridges, or on one of the abutments in the case of single-span bridges. In the more usual circumstance, however, in which the foundation material is firm but the bridge or span length is longer than $L_{cr}(n)$ for one or more dominant modes of response, condition 2 is considered to exist. In this case, instrumentation should be installed at the base of two or more bridge supports.

Condition 2 also exists if the foundation conditions vary significantly from one end of the bridge to the other. An example of such a condition would be a situation in which one end of a bridge is on hard rock while the other end is on soil. In this case, it is recommended that instrumentation be installed at the base of at least one bridge support on each foundation type and at the base of as many additional bridge supports as is suggested by the overall length of the bridge or spans of interest and $L_{cr}(n)$ for the dominant modes of response.

The number and arrangement of accelerometers to be installed at each support base to be instrummented in condition 2 situations is dependent on the direction(s) of the expected dominant mode(s) of response. In all cases, at least one orthogonal triaxial accelerometer package is required, and in many cases, triaxial packages may be required at every support base requiring instrumentation. In those instances, however, when the natural periods of the dominant modes of response are widely spaced, or when response in one or more of the three orthogonal principal directions is expected to be insignificant due to support constraints, accelerometers oriented in all three principal directions may not be required at every instrumented support base. An example of such a case is a wide non-skewed 2-span continuous reinforced-concrete bridge in which the abutments, central support column or bent, and bridge deck have been poured monothically. In this case, longitudinal motions (i.e., motions parallel to the roadway centerline) are not expected to be significant because of the support constraints, and therefore accelerometers oriented in that direction need be installed at the base of only one support. In addition, if the period(s) of the dominant mode(s) of response in the vertical direction is (are) substantially longer than those in the transverse direction, fewer accelerometers may be required in the vertical direction than in the transverse direction.

In those instances where condition 3 is expected to exist, that is, in those instances where the foundation material is flexible (relative to the bridge) and there is potential for soil-structure interaction, instrumentation should be installed at the base of each support at which that condition exists and at the base of one or more additional supports, depending on $L_{cr}(n)$ for the dominant modes of response and the length of bridge (or spans of interest) on firm foundation material (see discussion on conditions 1 and 2 above).

Obvious classes of bridges for which this condition can be expected to exist are those having pile supports and those having abutments or other supports on fill material. The amount of instrumentation to be installed at the base of each support where soil-structure interaction is expected is dependent on the expected mode of soil-structure interaction. Normally, soil-structure interaction can be expected to occur in the form of horizontal translations or rotations about a horizontal axis. In those instances where horizontal translations of the bridge support base are expected, one accelerometer is required for each principle direction of interest (normally the longitudinal and/or transverse directions). In those instances where rotations about a horizontal axis are expected, either a rotation sensing accelerometer or a pair of vertical accelerometers is required for each principal direction of interest.

Regardless of the foundation condition, it is imperative that the accelerometers at the instrmented support bases be installed on the footing and that they not be attached to the column, bent, or abutment at some distance (several feet) above the footing. This stipulation is required so that the recorded motions do not include structural response motions, that is, so that the recorded motions represent structure input motions only.

B. Superstructure Instrumentation

Because the amount of strong-motion instrumentation that is required on the superstructure of any particular bridge is dependent primarily on the objective of the instrumentation program, the remainder of this section on recommended instrumentation schemes is subdivided into three parts, one for each of the three primary program objectives referred to earlier in this report: force level determination, mathematical model identification, and mathematical model verification. The instrumentation schemes recommended are based primarily on the theoretical linear dynamic behavior of bridges and assumed ground-motion conditions. In some cases, potential failure mechanisms also necessitate the use of additional accelerometers.

Schemes for Force Level Determination - In order to determine force levels in bridges from strong-motion records, it is critical that the bridge be properly instrumented. By proper instrumentation it is meant that there exist a sufficient number of time-correlated accelerations to describe all significant bridge distortions as well as all significant structure input (ground level) motions. This is absolutely necessary if accurate force levels are to be determined from the experimental data without the use of a mathematical model. Based on theoretical linear bridge response considerations, there must exist at least one independently recorded structural motion for each significant type of distortion and, as indicated earlier, at least one set of triaxially recorded base-level input motions. Depending on the expected nature of the ground motion, dynamic properties of the bridge, and potential failure mechanisms of the bridge, the recording of additional base-level and structural response motions may also be required.

As an example of the strong-motion instrumentation required to compute adequately force levels throughout a bridge, consider the three-span simply-supported bridge shown in figure 37. The foundation material is firm and homogeneous beneath the entire bridge, S-body waves are expected to be dominant, and no soil-structure interaction effects are expected. It is not immediately clear, however, whether the ground motion is expected to be identical and in phase (or nearly so) beneath each of the bridge supports.

In regard to the structural characteristics of the example bridge (fig. 37),

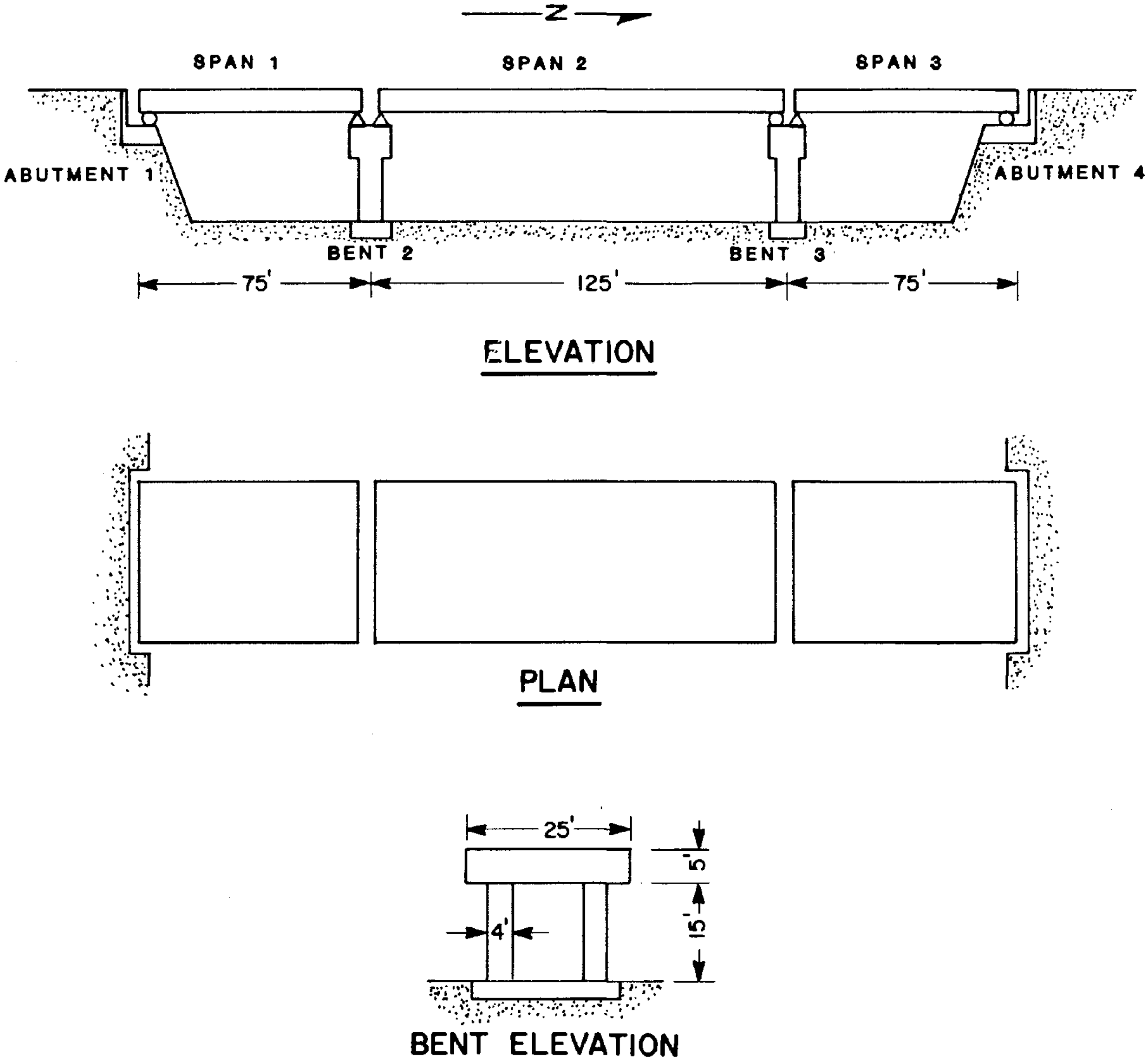


Figure 37.- Three-span simply-supported bridge, plan, and elevation views.

it is assumed that the three deck elements will remain rigid in their own plane, are restrained longitudinally at one end, and are restrained vertically and laterally (transversely) at both ends. It is therefore expected that lateral (transverse) and longitudinal flexural distortions of both bents as well as vertical flexure distortions of each deck element will be the dominant modes of response. It is also assumed that the vertical and lateral distortions of the end spans (spans 1 and 3) are expected to be similar due to symmetry, that the vertical flexural motions are uncoupled from the lateral and longitudinal motions, and that torsional motions about the bridge centerline axis can be ignored for the following reasons: (1) torsional components of the ground motion are not likely to be large; and (2) secondary torsional motions of the bridge due to lateral translations will not be exicted at the bents because the pair of columns at each bent prevents the pier cap from rotating about the centerline axis.

Having made the above assumptions regarding the bridge's probable response, we consider first the number, location, and orientation of accelerometers required to record fully structure input motions. Because the bridge is on firm homogeneous foundation material, equation 1 ($c = 3{,}000$ m/s) will be used to identify which support bases require instrumentation. This requires that the fundamental periods of the expected dominant modes of response be estimated. In this case, the fundamental periods, which have been derived from a previously carried out ambient vibration study of the bridge, are estimated as follows: for the longitudinal flexural distortions of bents 2 and 3 (modes 1 and 2), 0.5 and 0.3 s, respectively; for the lateral flexural distortions of both bents (mode 3), 0.2 s; for the vertical flexural distortions of spans 1 and 3 (mode 4), 0.1 s; and for the vertical flexural distortions of span 2 (mode 5), 0.3 s. On the basis of equation 1 then, $L_{cr}(n)$ for modes 1 through 5 is estimated to be: 25 m (mode 1); 15 m (mode 2); 10 m (mode 3); 5 m (mode 4); and 15 m (mode 5). Considering the symmetrical aspects of the bridge and the fact that longitudinal rollers are located at the bridge ends, these $L_{cr}(n)$ estimates suggest that vertically- and laterally-oriented accelerometers should be installed on the footing of abutment 1, bent 2, and bent 3 (accelerometers 1-6, fig. 38), and that longitudinally-oriented accelerometers should be installed on the footing of bent 2 and bent 3 (accelerometers 7, 8, fig. 38). Due to the anticipated similar behavior of spans 1 and 3, vertically- and laterally-oriented accelerometers are not required at abutment 4. Longitudinally-oriented accelerometers at abutments 1 and 4 are also not required because the longitudinal rollers at each abutment are assumed to prohibit the longitudinal abutment motions from affecting longitudinal bridge superstructure response. Nevertheless, it is recommended that longitudinally-oriented accelerometers be installed at each abutment (accelerometers 9, 10, fig. 38) in order to provide information on possible connection failures that may occur at these locations. The purpose of accelerometer 9 is to provide information on relative motion between abutment 1 and the top of bent 2 should span 1 move off abutment 1 and collapse, and that for accelerometer 10 is to provide similar information at abutment 4. In total, then, ten accelerometers are required to record fully all important structure input motions.

The number, location, and orientation of accelerometers required on the superstructure are dependent on: (1) anticipated dominant modes of structural response; (2) structure symmetry; (3) the location and orientation of accelerometers required to record structure input motions; and (4) potential

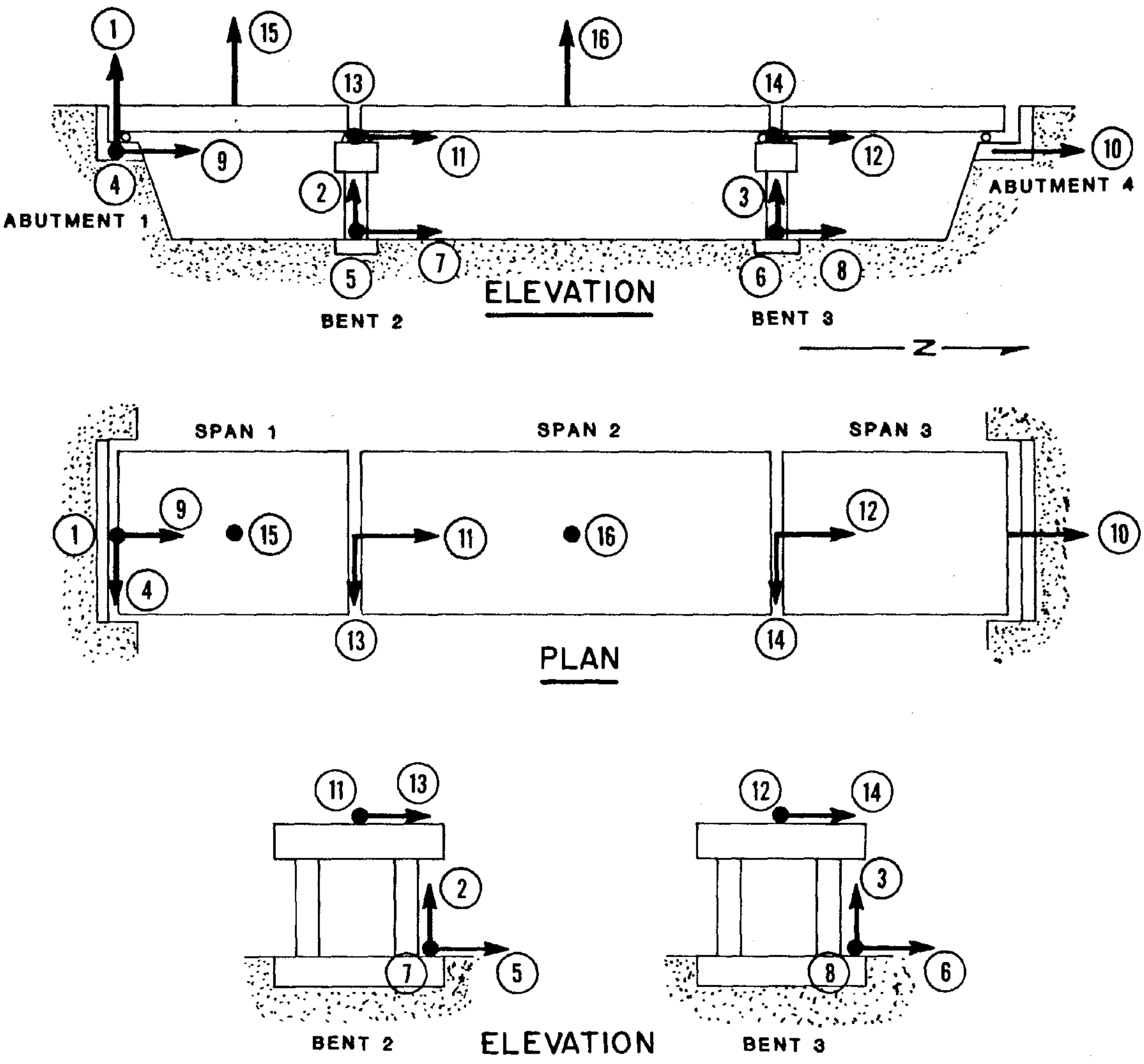


Figure 38.- Three-span simply-supported bridge. Strong-motion instrumentation scheme for force level determination. Accelerometer locations and orientations shown by arrows and dots, which represent arrows perpendicular to plane of figure.

failure mechanisms. The most important rule-of-thumb for selecting the locations and orientations of accelerometers to be placed on the bridge superstructure is that each accelerometer should be positioned so that it records the maximum response amplitude of the dominant mode that it is intended to record. For the simply-supported bridge shown in figure 37, then, the recommended locations and orientations for the superstructure accelerometers are as follows: a longitudinally-oriented accelerometer at the top of bent 2 (accelerometer 11, fig. 38) to record mode 1, the longitudinal deck motion of spans 1 and 2 and longitudinal flexural distortions of bent 2; a longitudinally-oriented accelerometer at the top of bent 3 (accelerometer 12, fig. 38) to record mode 2, the longitudinal deck motion of span 3 and longitudinal flexural distortions of bent 3; laterally-oriented accelerometers at the top of bents 2 and 3 (accelerometers 13 and 14, fig. 38) to record mode 3, deck lateral translational motion and bent lateral flexural distortions (it can not be assumed that the lateral motions at the top of both bents will be identical in time because of the possible phase lag in motions recorded at the base of both bents); and a vertically-oriented accelerometer at mid span of spans 1 and 2 (accelerometers 15 and 16, fig. 38) to record modes 4 and 5, vertical flexural distortions of each of these spans (it is assumed that these distortions can be approximated as half-sine waves). In total, six accelerometers are required on the superstructure to record fully structural response motions.

In summary, a total of fourteen time-correlated accelerations are required to determine force levels in the example simply-supported three-span bridge; two additional accelerometers are also required to provide information on relative motions at the abutments where connection failures are possible. For more complex bridges, the procedures for developing a suitable instrumentation plan are identical.

Schemes for Mathematical Model Identification - The principal objective of any system or identification procedure is to identify the mathematical model of dynamic bridge behavior. Once a realistic model has been identified through the analysis of strong-motion records from an instrumented bridge, that model can be used to: (1) evaluate the assumptions made in the formulation of any mathematical model used in the design stage, and (2) predict all significant bridge distortions for the instrumented bridge as well as similar bridges during future earthquakes.

In order to formulate a mathematical model through the analysis of a set of strong-motion records from an instrumented bridge, it is imperative that the strong-motion instruments be located both at the base of the bridge (to record structure input motions) and on the bridge superstructure (to record all significant structural response or output motions). If it has been concluded that all significant bridge distortions can be described by a model with n independent degrees of freedom, then by definition of their being independent, at least n independent motions (time-correlated accelerations) must be recorded. In addition, as indicated earlier in this report, at least one set of triaxially recorded structure base input motions is required. Strong-motion instrumentation schemes for mathematical model identification, then, have the same general requirements as those for force level determination.

As an example of the strong-motion instrumentation required to identify the mathematical model that best describes the observed distortions, consider the continuous two-span bridge shown in figure 39. As is the case for force

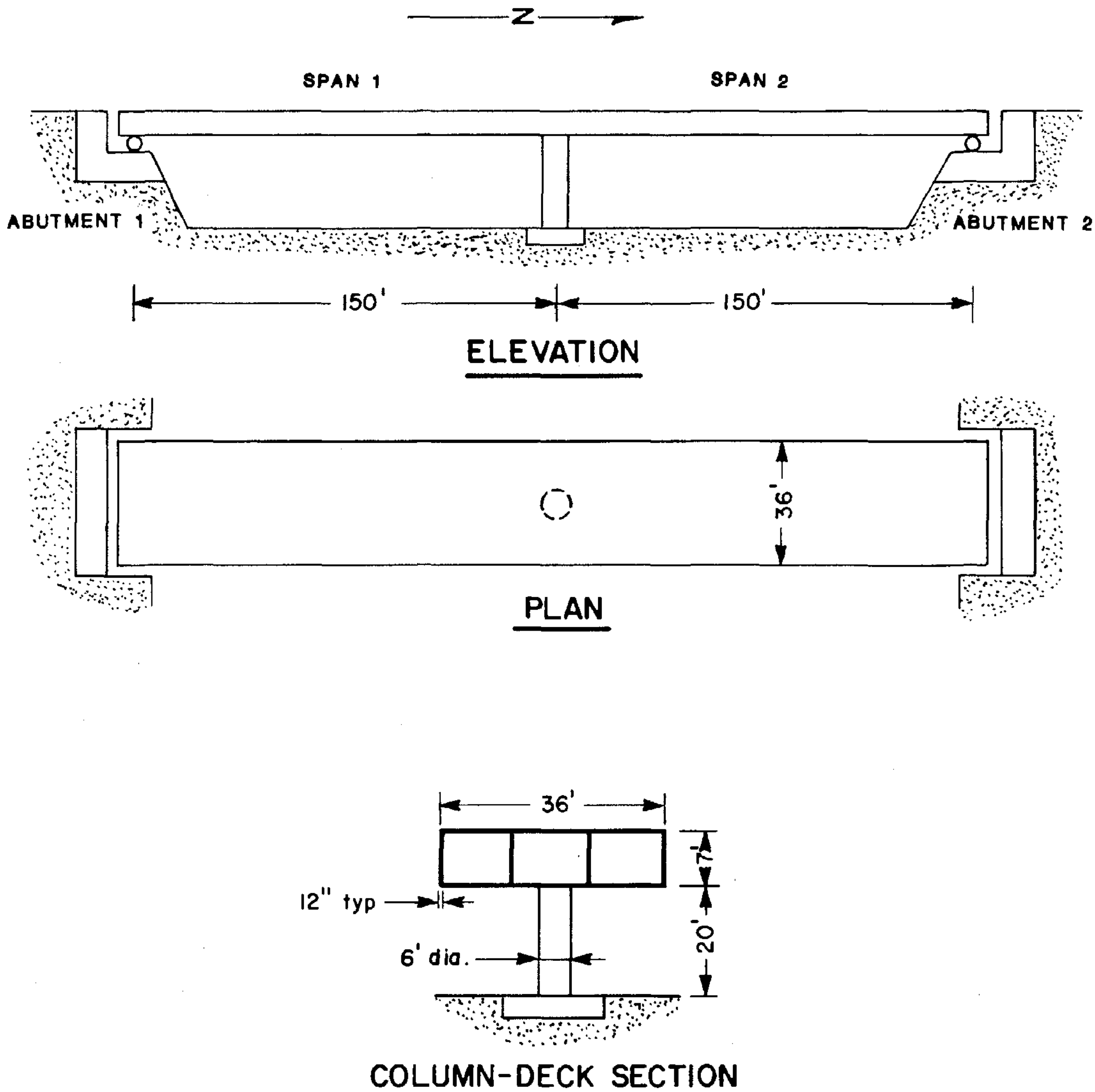


Figure 39.- Continuous two-span bridge plan, section, and elevation views.

level determination schemes, the design of the instrumentation scheme is dependent upon the expected nature of the ground motion, dynamic properties of the bridge, and potential failure mechanisms of the bridge. Assumptions regarding these factors, therefore, need to be made at the outset.

For the bridge shown in figure 39, the foundation material is not rigid beneath each of the supports, nor is it homogeneous throughout. The center column is situated on natural foundation material, which is assumed to be rigid, but the abutments are on non-rigid fill material. In other words, soil-structure interaction is expected to occur at each abutment site. As a result, the motions of each abutment are expected to be different from each other as well as different from the motions at the base of the center column.

In regard to the structural characteristics of the example bridge shown in figure 39, it is assumed that the continuous reinforced-concrete box girder is restrained vertically and laterally at each end, that the single support column at mid span is fixed to the bridge at the top and to the foundation at the bottom, and that the bridge deck is torsionally flexible at the column support. The following dominant modes of response are therefore expected to occur: (1) two modes of longitudinal rigid body motion of the bridge deck coupled with antisymmetric vertical bridge deck distortions (modes 1 and 2); (2) two modes of lateral (transverse) flexural distortions of the deck coupled with torsional distortions of the deck about a centerline axis parallel to the roadway (modes 3 and 4); (3) one mode of symmetric vertical flexural distortions of the deck (mode 5); and (4) rotation of the bridge deck about a vertical axis coupled with torsional distortions of the deck about a centerline axis parallel to the roadway (mode 6). This last mode of response would occur as a result of antisymmetric abutment motions.

Having made the above assumptions regarding the bridge's probable response, we consider first the number, location, and orientation of accelerometers required to record fully structure input motions. Because soil-structure interaction affects are expected to occur at each abutment site and because the foundation material beneath the central support column is assumed to be rigid, the ground motions are not expected to be identical beneath any two supports. This requires that an orthogonal triaxial package of accelerometers be installed at the center column (accelerometers 1-3, fig. 40) and that laterally- and vertically-oriented accelerometers be installed at each abutment (accelerometers 4-7, fig. 40). Longitudinally-oriented accelerometers are not required at each abutment because the longitudinal rollers are assumed to prohibit longitudinal abutment motions from affecting longitudinal bridge superstructure response. As in the case of the previous example for force-level determination studies, however, it is recommended that longitudinally-oriented accelerometers be installed at each abutment (accelerometers 8, 9, fig. 40) in order to provide information on possible connection failures that may occur at these locations. The purpose of accelerometer 8 is to provide information on relative motion between abutment 1 and span 1 should span 1 move off abutment 1 and collapse, and that for accelerometer 9 is to provide similar information at abutment 2. In total, then, nine accelerometers are required to record fully all important structure input motions.

As is the case for force-level determination schemes, the number, location, and orientation of accelerometers required on the superstructure are dependent on: (1) anticipated dominant modes of structural response; (2) structure symmetry (or lack thereof); (3) the location and orientation of

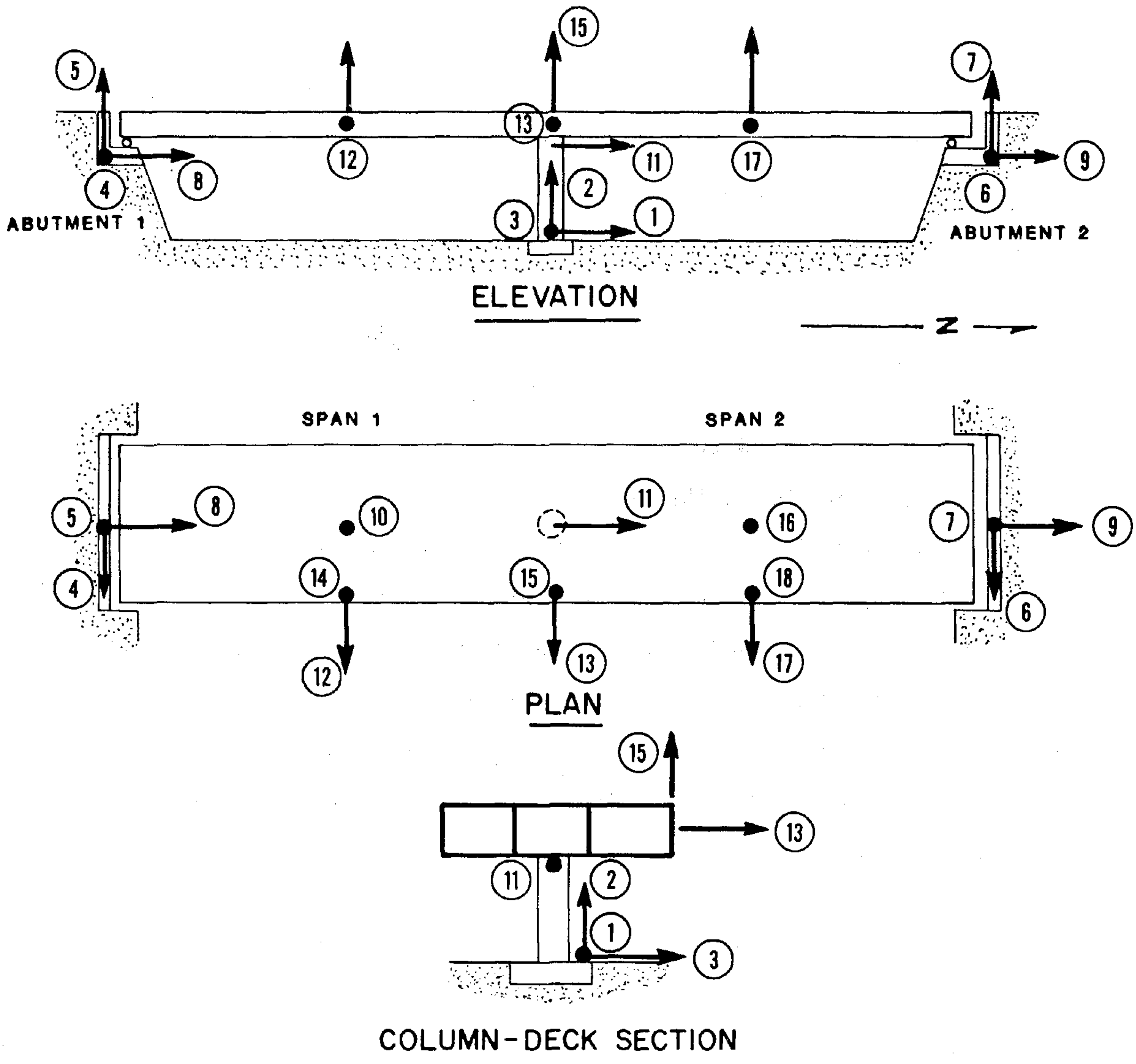


Figure 40.- Continuous two-span bridge. Strong-motion instrumentation scheme for mathematical model identification. Accelerometer locations and orientations shown by arrows and dots, which represent arrows perpendicular to plane of figure.

accelerometers required to record structure input motions; and (4) potential failure mechanisms. Again, the most important rule-of-thumb for selecting the locations and orientations of accelerometers to be placed on the bridge superstructure is that each accelerometer should be positioned so that it records the maximum response amplitude of the dominant mode that it is intended to record. For the two-span continuous bridge shown in figure 39, then, the recommended locations and orientations for the superstructure accelerometers are as follows: a vertically-oriented accelerometer at mid span of span 1 at the deck centerline and a longitudinally-oriented accelerometer at the top of the center column (accelerometers 10 and 11, fig. 40) to record modes 1 and 2, longitudinal rigid-body motions of the bridge deck and the corresponding coupled antisymmetric vertical bridge deck distortions (it is assumed that the distorted shape of the vertical motions can be approximated as a sine wave); a laterally-oriented accelerometer at the top of the center column and at mid span of span 1 along with a vertically-oriented accelerometer along the outside edge of the bridge deck at each of these locations (accelerometers 12-15, fig. 40) to record modes 3 and 4, lateral (transverse) flexural distortions of the deck and the corresponding coupled torsional distortions of the deck about a centerline axis parallel to the roadway (it is assumed that the distorted shapes of the lateral and torsional motions can be approximated as a linear combination of a half-sine wave for the first mode and three half-sine waves for the second mode); a vertically-oriented accelerometer at mid span of span 2 (the span not instrumented to record modes 1 and 2) at the deck centerline (accelerometer 16, fig. 40) to record mode 5, symmetric vertical flexural distortions of the deck (it is assumed that the distorted shape of the vertical motions can be approximated by a vertical dead load deflected shape); and, finally, a laterally-oriented acclerometer at mid span of span 2 and a vertically-oriented accelerometer along the outside edge of the bridge deck also at mid span (accelerometers 17 and 18, fig. 40) to record mode 6, rotation of the bridge deck about a vertical axis (due to antisymmetric abutment motions) as well as the corresponding torsional distortions of the deck about a centerline axis parallel to the roadway. In total, nine accelerometers are required on the superstructure to record fully structural response motions.

In summary, a total of sixteen time-correlated accelerations are required to identify a mathematical model of the example two-span continuous bridge; two additional accelerometers are also required to provide information on relative motions at the abutments where connection failures are possible. For more complex bridges, the procedures for developing a suitable instrumentation plan are identical.

As is illustrated by this example, the complete instrumentation required for model identificaton purposes for a simple continuous two-span bridge is quite extensive. Obviously the instrumentation required for model identification purposes for multi-span bridges with many expansion joints would be formidable indeed. Much can be learned from model verification rather than model identification, however, and as will be discussed, the required instrumentation for purposes of model verification can be far less than that required for purposes of model identification.

Schemes for Mathematical Model Verification - The model-verification approach requires that at least one acceleration history be recorded on the superstructure (output motion) and that a complete set of time-correlated accelerations be recorded at the base of the structure (input motion). In other words, although the same amount of instrumentation is required to define input motion in model-verification schemes as in force-determination or model-identification schemes, substantially less instrumentation is required (in model-verification schemes) to record output motion on the superstructure. In the data analysis stage, the proposed theoretically-formulated mathematical model of the bridge is subjected to the recorded input motion, and the recorded accelerations from the bridge superstructure are then compared to the motions theoretically generated for the same location using the proposed model. If the comparison is acceptable, the model is verified and little more need be done. If on the other hand, the comparison is not acceptable, then little can be done to improve the model in a systematic manner. By perturbing certain parameters, the effect those parameters have on the theoretically-generated response may be observed, but the likelihood is not great that the model can be improved significantly in this manner, particularly in the case of large systems.

Literally any response or set of responses may be used for model verification, but some response motions are preferable over others, depending upon the criterion used to verify the model. Some locations on a bridge are likely to respond with comparable amplitude in many natural modes of vibration, whereas others may respond almost exclusively in a single mode of vibration (usually the fundamental). If the criterion used to verify a model is the cumulative goodness of fit of many modes of vibration, a response recording in one of the former locations would be preferable over one of the latter. If, on the other hand, the criterion is the precise goodness of fit of a single mode of vibration, then a response recording in one of the latter locations is preferable over one of the former. In general, we recommend that the number, location, and orientation of accelerometers to be installed on the superstructure be sufficient enough to provide information about all modes of dominant response, but that as few accelerometers as possible be utilized. In most cases, only three accelerometers will be required, one in each of the three orthogonal principal directions, but in some cases, more or less than three may be required, depending on the directions of motion of the dominant modes of response or their complexity.

As an example of the strong-motion instrumentation required for mathematical model verification, consider the continuous three-span reinforced-concrete box-girder bridge shown in figure 41. The bridge is founded on firm homogeneous foundation material (S-body waves are expected to be dominant) and is supported by an abutment at each end and by two intermediate supports that are unequal in height, making this bridge and its response unsymmetric. Each intermediate support consists of two columns cast integrally with the box girder and fixed at the base. Torsional distortions of the deck about a centerline axis are not expected because the columns at each intermediate support and the roller supports at the bridge ends are assumed to resist torsional motion as well as vertical and lateral motion. Only between supports is the bridge deck free to twist. Since it is anticipated, however, that the ground motion will not contain significant torsional components, these three spans are not likely to be excited torsionally.

Based on a previously formulated mathematical model of the bridge that is to be verified using data from the strong-motion instrumentation scheme developed in this example, the following dominant modes of reponse are expected to occur: (1) three modes of lateral (transverse) flexural

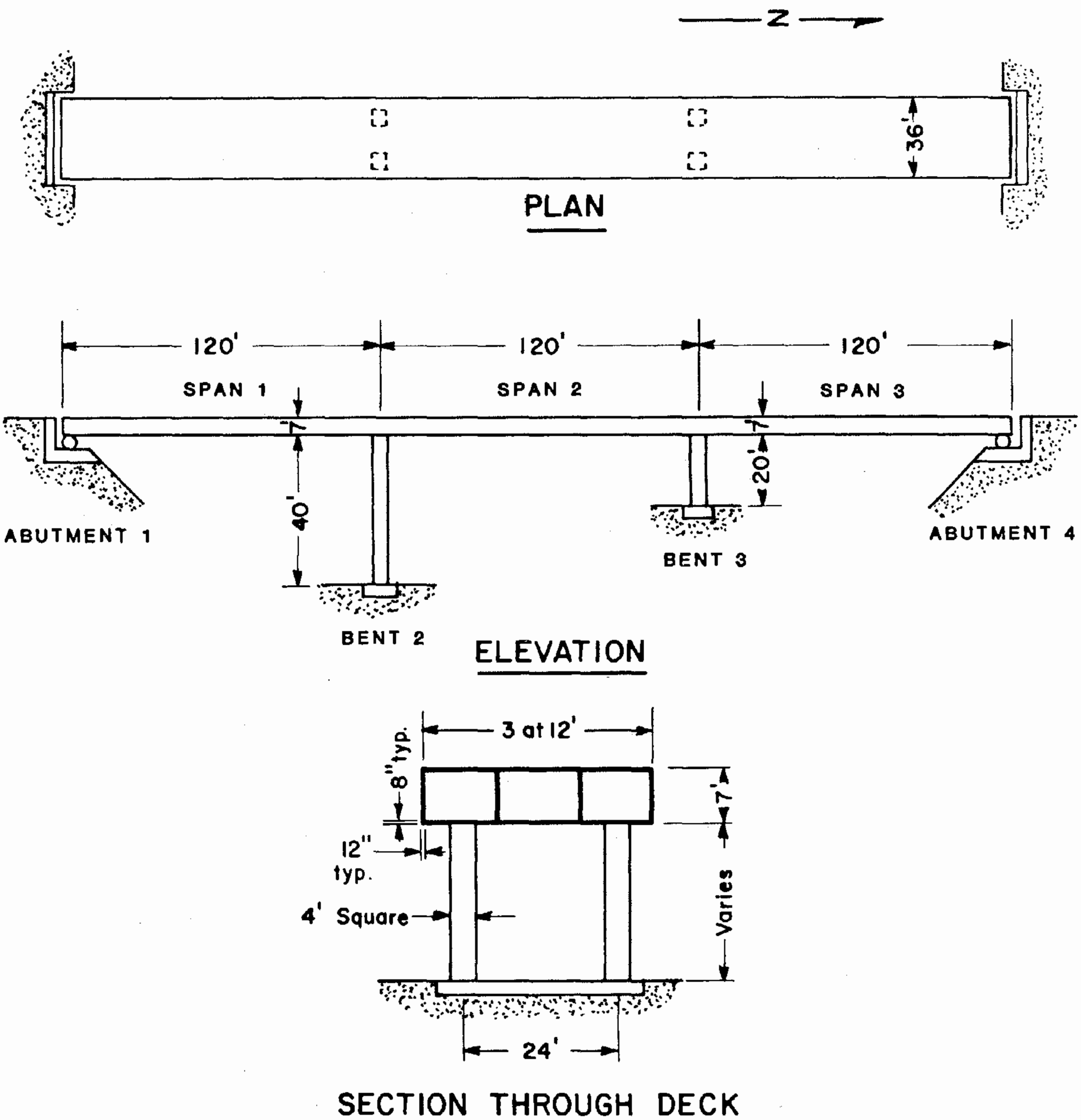


Figure 41.- Continuous three-span bridge plan, section, and elevation views.

distortions of the deck (modes 1-3, fig. 42); (2) one mode of longitudinal rigid body motion of the bridge deck coupled with antisymmetric vertical bridge deck distortions (mode 4, fig. 42); (3) and three modes of vertical flexural distortions of the deck coupled with small longitudinal rigid body motions (modes 5-7, fig. 42). The predicted natural periods of these 7 modes are: 0.35 s (mode 1); 0.17 s (mode 2); 0.10 s (mode 3); 0.46 s (mode 4); 0.32 s (mode 5); 0.27 s (mode 6); and 0.20 s (mode 7).

Having identified the expected dominant modes of response and their natural periods, we next consider the number, location, and orientation of accelerometers required to record fully structure input motions. Because the bridge is on firm homogeneous foundation material, equation 1 (c = 3,000 m/s) will be used to identify which support bases require instrumentation. On the basis of this equation, $L_{cr}(n)$ for modes 1 through 7 is estimated to be: 17.5 m (mode 1); 8.5 m (mode 2); 5 m (mode 3); 23 m (mode 4); 16 m (mode 5); 13.5 m (mode 6); and 10 m (mode 7). Considering the fact that longitudinal rollers are located at the bridge ends, these $L_{cr}(n)$ estimates suggest that vertically- and laterally-oriented accelerometers should be installed on the footing of abutment 1, bent 2, bent 3, and abutment 4 (fig. 43, accelerometers 1-8), and that longitudinally-oriented accelerometers should be installed on the footing of bent 2 and bent 3 (fig. 43, accelerometers 9, 10). Longitudinally-oriented accelerometers at abutments 1 and 4 are not required because the longitudinal rollers at each abutment are assumed to prohibit the longitudinal abutment motions from affecting longitudinal bridge superstructure response. Nevertheless, it is recommended that longitudinally-oriented accelerometers be installed at each abutment (accelerometers 11, 12, fig. 43) in order to provide information on possible connection failures that

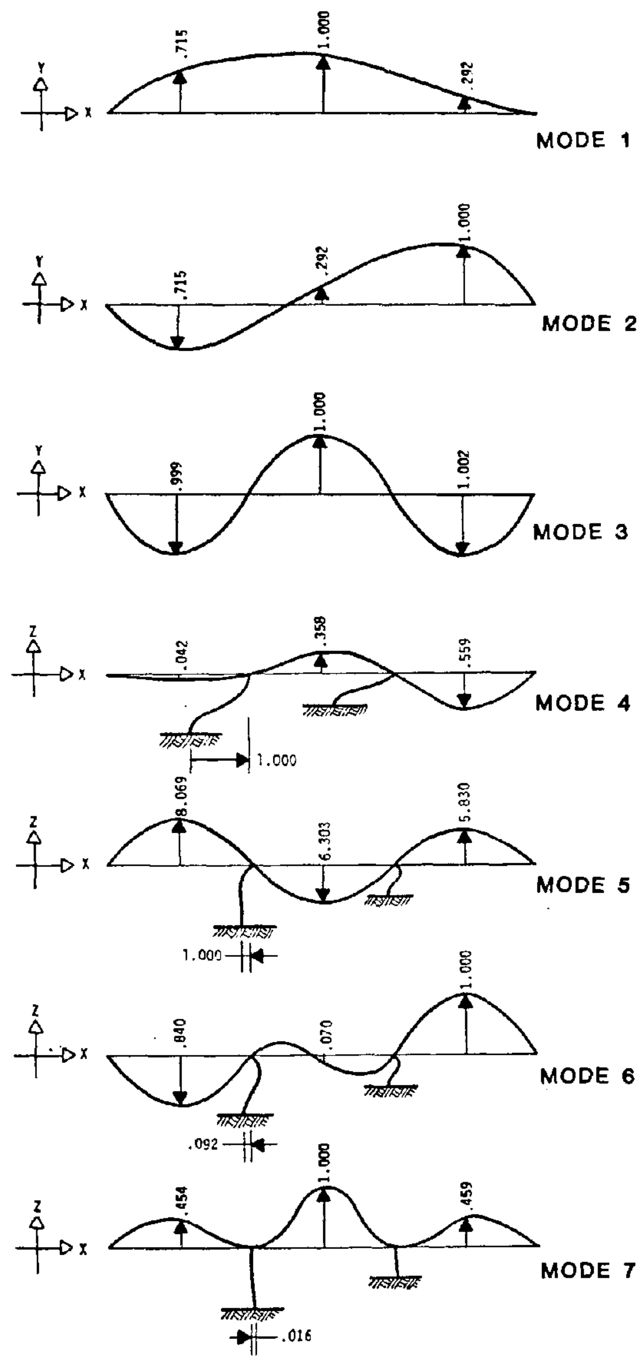


Figure 42.- Mode shapes for the example continuous three-span bridge, based on a previously determined mathematical model. Longitudinal, lateral, and vertical directions denoted by x, y, and z, respectively.

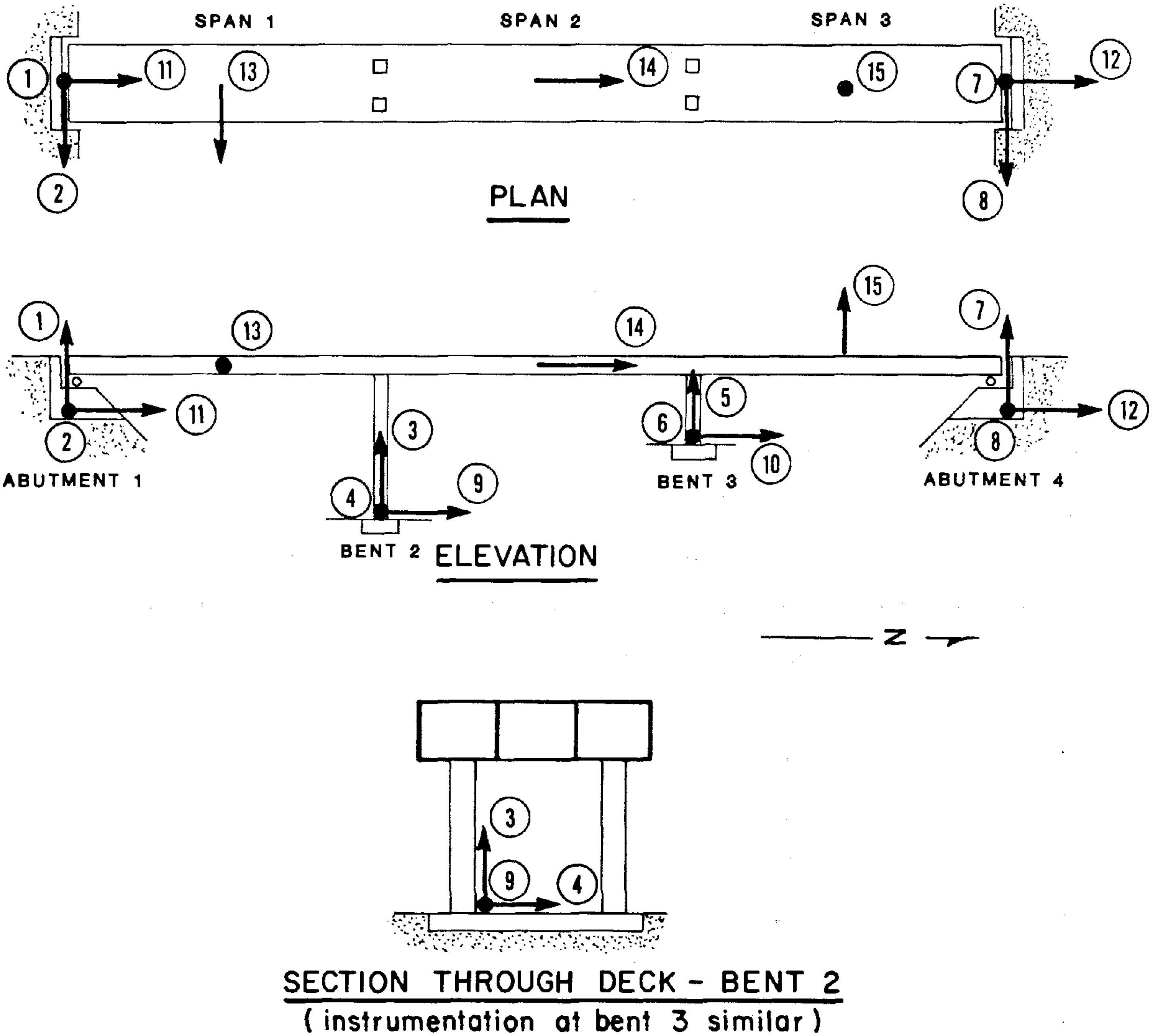


Figure 43.- Continuous three-span bridge. Strong-motion instrumentation scheme for mathematical model verification. Accelerometer locations and orientations shown by arrows and dots, which represent arrows perpendicular to plane of figure.

may occur at these locations. The purpose of accelerometer 11 is to provide information on relative motion between abutment 1 and span 1 should span 1 move off abutment 1 and collapse, and that for accelerometer 12 is to provide similar information at abutment 4. In total, then, twelve accelerometers are required to record fully all important structure input motions.

As indicated earlier, the number, location, and orientation of accelerometers required on the superstructure are primarily dependent on the shape and orientation of the expected dominant modes of response. The mode shapes (fig. 42) identified by the mathematical model to be verified using data from the strong-motion instrumentation scheme developed in this example suggest that: (1) one accelerometer is required for each of the three orthogonal principal directions; (2) the most information about the lateral modes of response (modes 1-3) could be obtained if the laterally-oriented accelerometer were located at mid span of span 1 (location where all three modes have relatively high amplitudes of response); (3) the most information about the vertical modes of response (modes 5-7) could be obtained if the vertically-oriented accelerometer were located at mid span of span 1 or 3 (locations where all three modes have relatively high amplitudes of response); and (4) the most information about the longitudinal mode of response, which includes coupled vertical motions, (mode 4) could be obtained if a vertically-oriented accelerometer were located at mid span of span 3, in addition to the longitudinally-oriented accelerometer that could be located anywhere along the bridge deck longitudinal axis. For the three-span continuous bridge shown in figure 41, then, the recommended locations and orientations for the superstructure accelerometers are as follows: a laterally-oriented accelerometer at mid span of span 1 at the deck centerline (accelerometer 13, fig. 43); a longitudinally-oriented accelerometer at mid span of span 2 (accelerometer 14, fig. 43); and a vertically-oriented accelerometer at mid span of span 3 (accelerometer 15, fig. 38). In total, three accelerometers are required on the superstructure to record fully structural response motions.

In summary, a total of thirteen time-correlated accelerations are required to verify a mathematical model of the example three-span continuous bridge; two additional accelerometers are also required to provide information on relative motions at the abutments where connection failures are possible. For more complex bridges, the procedures for developing a suitable instrumentation plan are identical.

Because strong-motion instrumentation schemes for model verification require substantially less instrumentation than those for force-determination or model-identification studies, the installation of model verification schemes is often necessitated in situations where economic resources are severely limited. Even though there are serious shortcomings associated with such an approach, these schemes do provide the opportunity to evaluate the validity of any proposed mathematical model of the bridge under study. When successfully carried out, such studies yield valuable results for a minimal cost.

VII. RECOMMENDED INSTRUMENTATION TYPE

As indicated in an earlier section of this report, the analog and digital accelerographs in use today in the U.S. are available in two principal varieties: as triaxial self-contained instruments or as remote-accelerometer central-recording systems. In the self-contained variety (fig. 44), all major components--accelerometers, recorder, and trigger--are housed in one container, whereas in the remote-recording type (fig. 45), these elements can be physically separated but are interconnected by low-voltage data cable. Of these two varieties, the remote-accelerometer central-recording type is well-suited for installation both on highway bridges and at adjacent free-field sites. The self-contained variety, on the other hand, is normally suitable only for installation at free-field sites.

Remote-accelerometer central-recording systems have several important

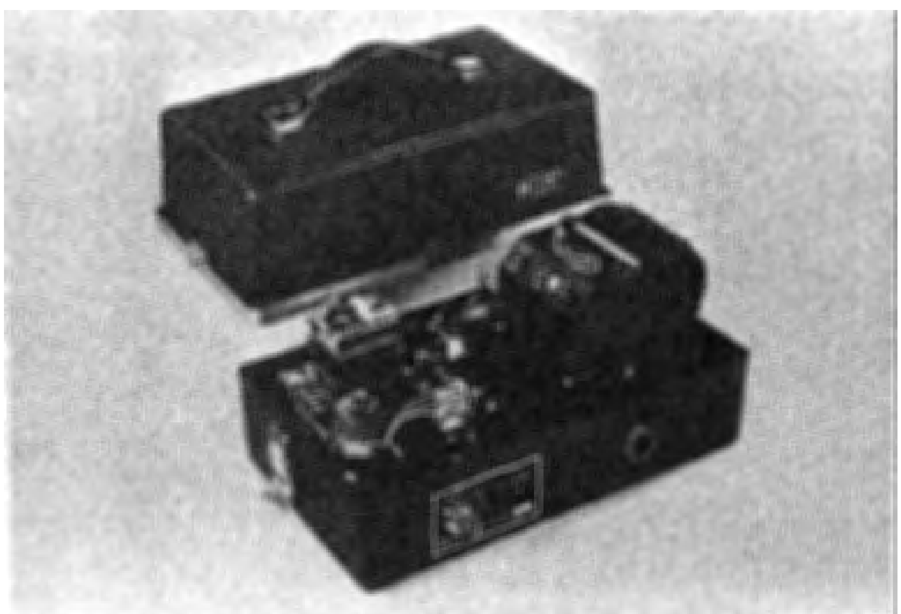
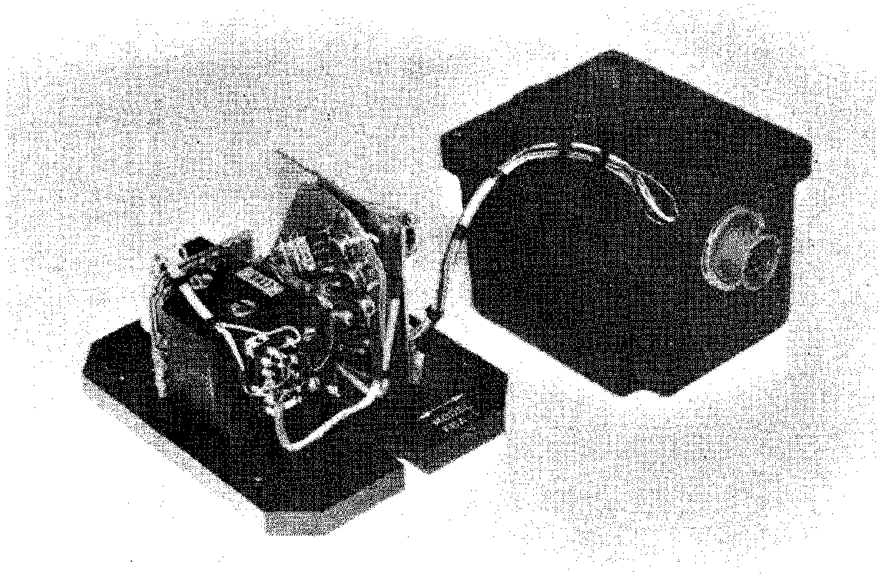
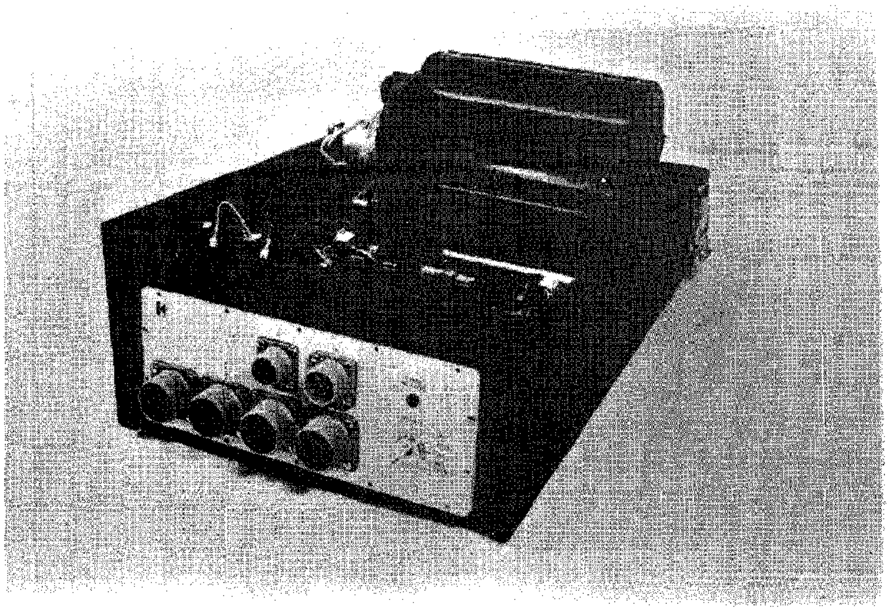


Figure 44.- SMA-1 triaxial self-contained accelerograph.



FBA-1 single-axis accelerometer



CR-1 central recorder

Figure 45.- CRA-1 remote-accelerometer central-recording accelerograph.

characteristics that make them ideal instruments for installation on highway bridges. The accelerometers in such systems are small, require little maintenance, and can be mounted right-side-up, up-side-down, or on vertical surfaces almost anywhere on the bridge, including inaccessible or remote locations. In addition, the recorder(s) can be centrally located for easy maintenance and record retrieval. Self-contained instruments, on the other hand, require more spacious and secure sites (these instruments are several times larger than the accelerometers in remote-recording systems), must be installed right-side-up on a horizontal surface, and must be accessible for routine maintenance and record retrieval. Such instruments, therefore, are not recommended for installation on highway bridges because their size and space requirements would prohibit them from being installed at many of the locations required on the basis of expected bridge response.

Since the remote-accelerometer central-recording systems and the self-contained instruments can be purchased with either an analog or digital recording capability, the next question that must be addressed is that regarding the suitability of recording in analog or digital form. Ideally it would be more desirable to use digital recorders rather than analog recorders in order to eliminate the costly and time-consuming step of digitizing the recorded data. There are, however, several factors that make currently available digital recorders much less desirable than their analog counterparts. Firstly, the digital recorders are 50% to 75% more expensive than the analog recorders, and secondly, and more importantly, there are major technical problems with currently available digital recorders. It is questionable, for example, whether the sample rate in today's digital recorders is adequate for state-of-the-art processing, and whether satisfactorily precise common timing amongst several such recorders, normally available in three-channel configuration, can be achieved. (The need for precise common timing is especially critical when several recorders are used to record both bridge ground-motion input and structural response data.) Moreover, as exemplified by the extremely poor operational performance of digital recorders on three recently instrumented bridges in the State of Washington (all recorders performed so poorly that they are currently being replaced with analog recorders), the long-term reliability of digital recorders in often hostile field environments is highly questionable. Until these technical obstacles are overcome, digital recorders are not recommended for installation on highway bridges. The one exception to this recommendation, of course, is the situation in which one of the objectives of the instrumentation scheme is to evaluate the instrumentation itself.

The most ideal analog-recording instruments (for highway bridge and adjacent free-field site installations) known to the authors and currently available on the market are the Kinemetrics SMA-1 self-contained accelerograph (fig. 44), or its equivalent, and the Kinemetrics CRA-1 remote-accelerometer central-recording system (fig. 45). Both types have been heavily utilized in the instrumentation programs of the U.S. Geological Survey and the California Division of Mines and Geology and both have proven themselves to be reliable field instruments.

The SMA-1 accelerograph (fig. 44), which records three orthogonal components of data on 70-mm photographic film, is battery powered, is normally triggered by vertical motion that equals or exceeds 0.01 g, and is designed to record acceleration with frequency components nominally within the range 0-25 Hz and with maximum amplitudes of 1 g. (One-quarter, one-half, and 2 g instruments are also available, but are normally not recommended for installation at free-field sites near instrumented bridges.) The instrument is capable of recording up to 25 minutes of earthquake data, with real time for each recorded event provided by a radio WWVB receiver or an internal time-code generator. The system also records one or two fixed traces on each earthquake record (two recommended) as well as an internal time mark every half second (@ 1/2 cm spacing).

The CRA-1 accelerograph system (fig. 45), which records up to 13 channels of data on 178-mm (7 in.) photographic film, is also battery powered, is normally triggered by vertical motion that equals or exceeds 0.01 g, and is designed to record acceleration with frequency components nominally within the range 0-50 Hz and with maximum amplitudes of 1 g. The system consists of a CR-1 multi-channel analog recorder and single-axis, biaxial, or triaxial force-balance (FBA) accelerometers. The FBA accelerometers have a natural frequency of approximately 50 Hz and are available in standard or water-tight down-hole casing; the latter type is designed for installations in wet environments such as ground vaults or on the tops of footings below the ground surface. Like the SMA-1, the CRA-1 system is capable of recording up to 25 minutes of earthquake data, with real time for each recorded event provided by a radio WWVB receiver or an internal time-code generator. The system is also capable of recording several fixed traces on each earthquake record (one fixed trace for every pair of data traces is recommended) as well as an internal time mark every half second (@ 1/2 cm spacing).

VIII. INSTRUMENT INSTALLATION AND MAINTENANCE

Strong-motion instrument installation and maintenance require careful planning and execution, as operational success is directly related to the extent to which these activities are properly carried out. There are numerous well-known examples of instrumentation systems that have failed in the field as a result of improper installation or maintenance, and in each of those cases, irreplaceable valuable data were lost. It is imperative, therefore, that the importance of these operations not be underestimated.

A. System Installation

In order to simplify our discussion on strong-motion instrumentation installation, we have subdivided the subject into the following two subtopics: (1) installation of free-field instrumentation; and (2) installation of bridge instrumentation, which includes remote-accelerometers, central-recorder(s), and accelerometer-to-recorder interconnect cable.

Free-field instrumentation installation - In most cases, the basic components of a typical free-field installation are the accelerograph (or triaxial accelerometer package) itself, the foundation to which it is attached, the housing that shelters the instrument, the electrical-power source, and arrangements for security. In many cases, the design of the foundation and the size and weight of the instrument shelter are particularly critical because these elements, in conjunction with the site soil characteristics, define the extent to which soil-structure interaction (foundation compliance) may affect (amplify or attenuate) any recorded strong-motion data (Bycroft, 1978). When the free-field site is founded on competent rock, of course, soil-structure interaction will not occur and the design of the foundation and housing for the instrument is not critical. On the other hand, when the site is founded on incompetent soil, which can be characterized as soil having a low shear-wave velocity, the design of the foundation and housing for the instrument is critical. In the latter situation, soil-structure interaction will be minimized if the instrument is placed in a small ground vault (fig. 46 A, B), an arrangement that requires the installation of a standard or down-hole remote-accelerometer package. If a self-contained acceleograph (e.g., an SMA-1) is to be used, soil-structure interaction can also be minimized but to a lesser extent than for the underground vault arrangement. In this case, the instrument shelter should be designed so that the moment of inertia for the overall system is small. This requires that the reinforced-concrete foundation pad be relatively large in plan compared to its depth (e.g., 1.2-m x 1.2-m x 0.2-m deep), that the instrument be attached directly to the pad and not be situated on top of a tall monolithically-poured pedestal (as has been done at some sites in the past), and that the shelter housing be constructed of a light-weight material such as fiberglass. A typical design for such a system is shown in figure 46C.

The amount of accessory equipment required at the free-field site also depends on the type of accelerograph system to be installed. If the instrumentation is to consist only of a triaxial remote-accelerometer package (e.g., down-hole FBA-3), the only required additional equipment is the recorder interconnect cable (cable specification described below). If the instrumentation is to consist of a self-contained accelerograph (e.g., SMA-1), a more complete complex of equipment will be required. In addition to arrangements for security (fence and door lock), the additionally required equipment include an external electrical-power source to charge constantly the accelerograph internal power supply (normally two 6-volt rechargeable

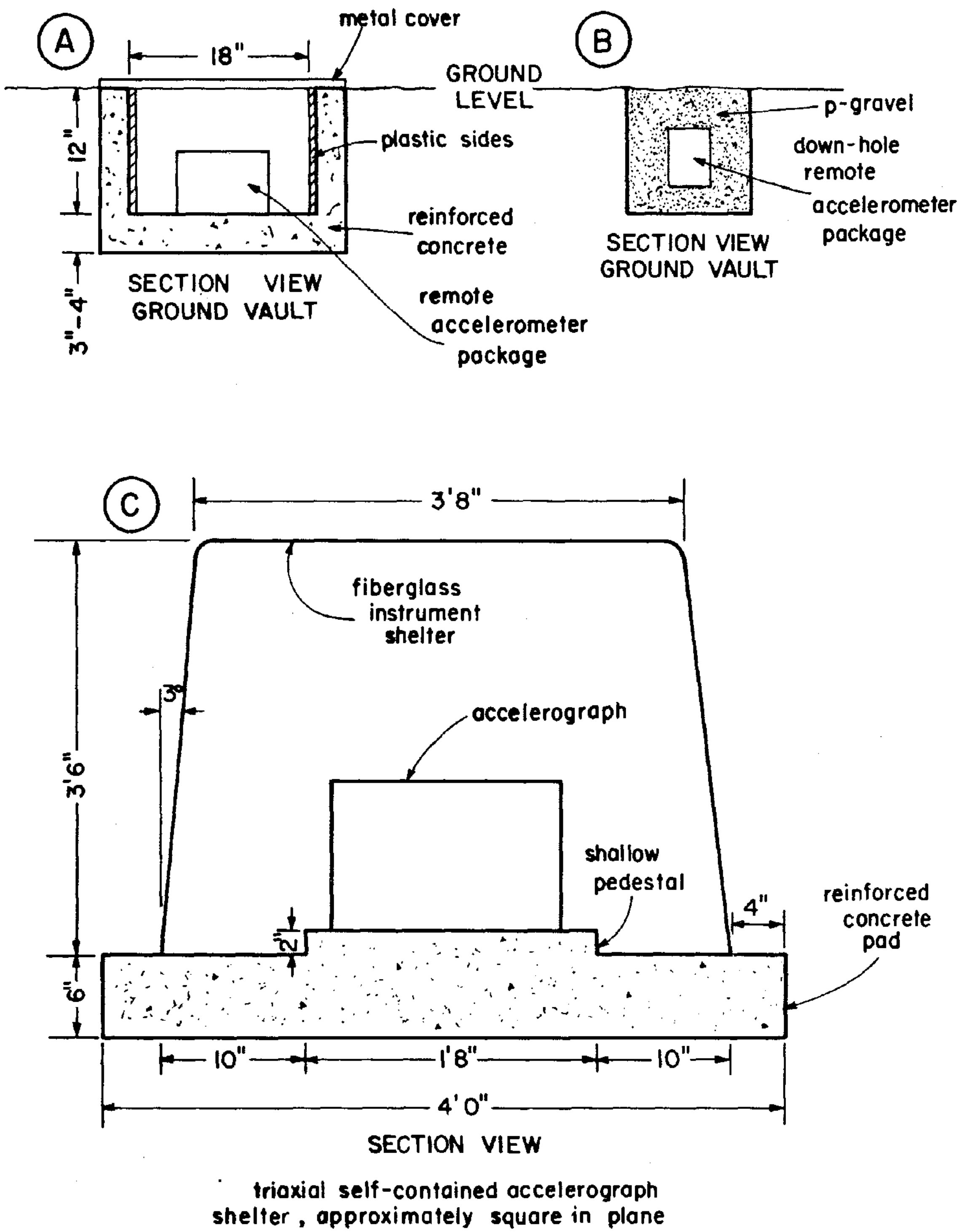
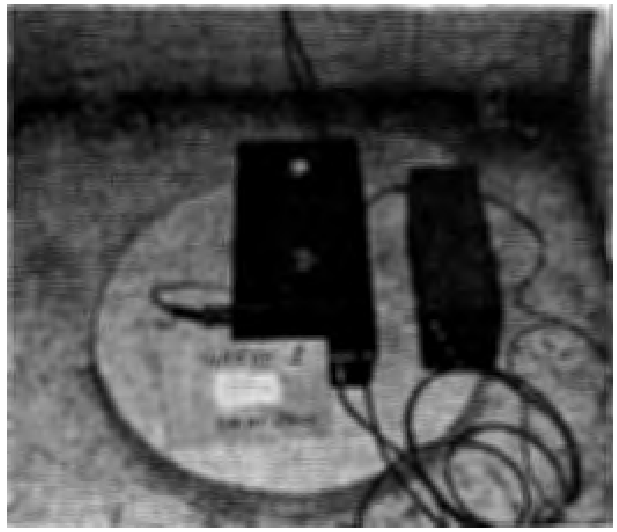


Figure 46.- Recommended designs for free-field installations.

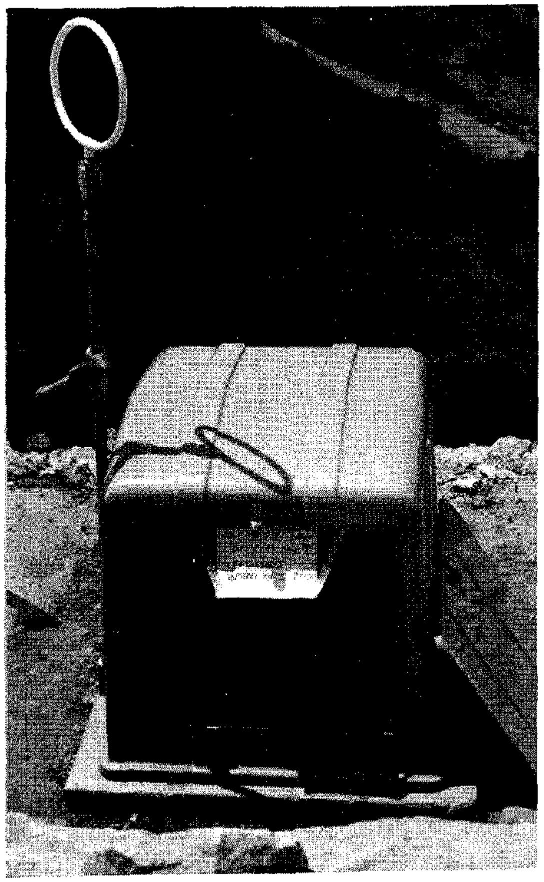
batteries, fig. 47A), and, in some cases, a radio antenna to receive WWVB time (fig. 47B), if this is the means by which real time will be coded on the recorded data. The external power source can either be the normal 110-volt alternating-current supply provided by public utilities, or solar panels (fig. 47C) (14.0-volt, 1-watt presently recommended). The former is preferred. If the 110-volt ac supply is utilized, a trickle charger that provides a tapered charge that maintains the batteries at the voltage specified by the instrument manufacturer will be required (Globe-Union gel/cell charger type GRC 12060 CDF, or equivalent, presently recommended for SMA-1 and CRA-1 systems).

Because common timing for data recorded at the free-field site and on the bridge is also highly desirable, we recommend that interconnect cable be installed between these locations wherever possible. In those cases where a remote-accelerometer package is installed at the free-field site, no additional cable is required because this instrument would presumably already be interconnected to the same recorder as the accelerometers attached to the bridge (hereafter referred to as the bridge recorder). In those cases where a self-contained accelerograph (SMA-1) is installed, however, we recommend that the self-contained accelerograph and the bridge recorder(s) be interconnected via 18-gauge 4-conductor shielded color-coded waterproof cable suitable for direct burial. Such cable should be installed in water-tight underground conduit.

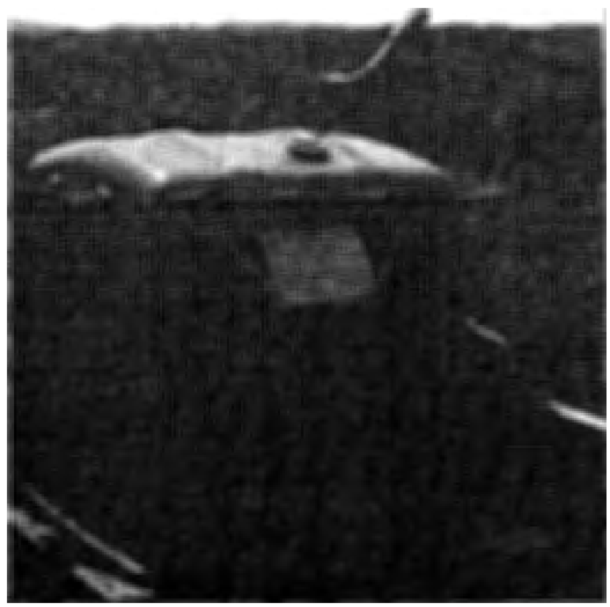
It is critical at all sites that the accelerograph (or remote-accelerometer package) be properly anchored to the base of the instrument shelter so that the instrument cannot be moved in any direction relative to the base to which it is attached. For this purpose we recommend



A, Self-contained accelerograph anchored to shelter foundation. Note battery case to right of accelerograph.



B, Shelter with WWVB radio receiver antenna.



C, Shelter with solar panel (just above top of shelter).

Figure 47.- Typical free-field self-contained accelerograph installations.

that Phillips red head self-drilling anchors, bolt size 1/4", be used.

Bridge instrumentation installation - The installation of instrumentation on and near the bridge is a much more complex operation that the installation of free-field instrumentation. In general, however, the techniques for anchoring, protecting, and interconnecting the instrumentation, are quite similar.

The remote-accelerometers to be installed on the bridge (fig. 48), whether they be single-axis, bi-axial, or triaxial units, should be securely anchored to the structure (Phillips red head self-drilling anchors, bolt size 1/4", are recommended) and should be protected from water or water vapor damage. In some cases, this may require an additional housing around the accelerometer package, and in other cases, it may require the installation of water-tight down-hole accelerometers, which are designed to be installed under water. As

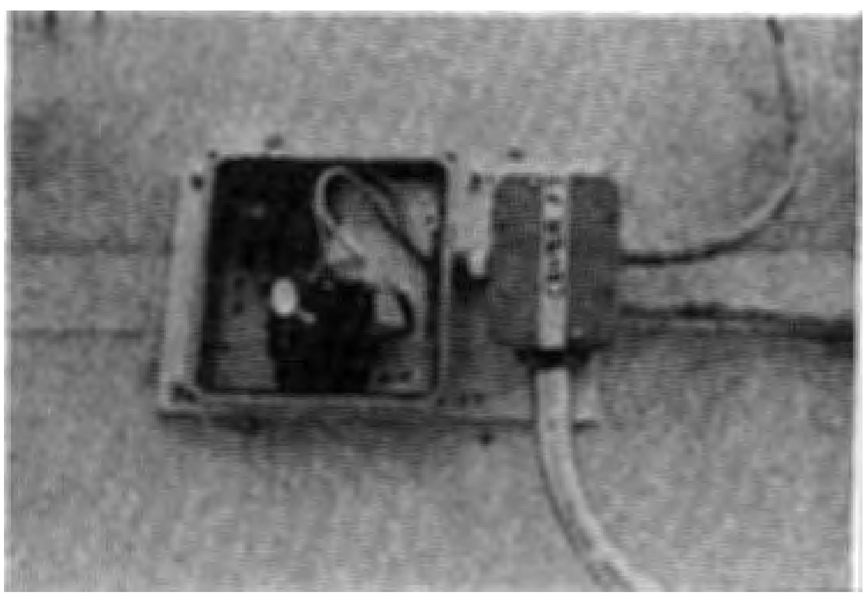


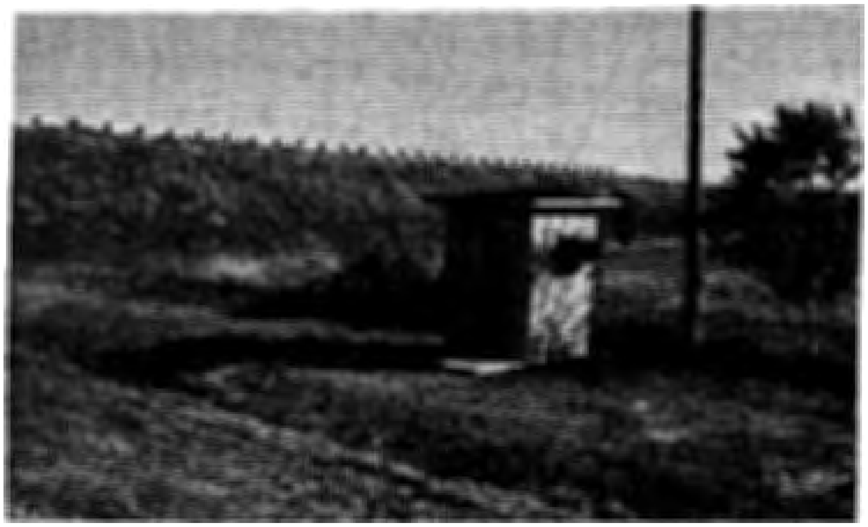
Figure 48.- Remote accelerometer (FBA-1) anchored to underside of bridge deck.

mentioned previously, accelerometer packages to be installed on footings beneath the ground surface should normally be of the down-hole variety.

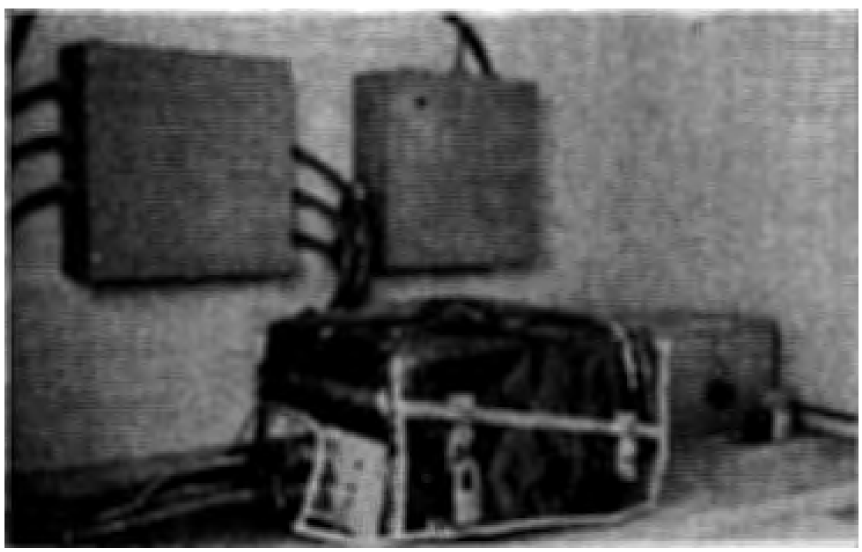
Low-voltage accelerometer-to-recorder interconnect cable is required between each remote-accelerometer and the recorder site. For CRA-1 systems, we suggest that the same size cable be utilized to service all accelerometers, whether they be single-axis, biaxial, or triaxial, in order to simplify the design of the cable layout as well as installation (18-gauge 12-conductor cable, shielded in pairs, Alpha #6024 or equivalent, is currently recommended). It is imperative that all runs of this interconnect cable be unspliced and that it be installed in metal conduit in order to protect the cable from environmental degradation and vandalism; PVC or plastic conduit is normally unacceptable because it has a high coefficient of thermal expansion and is therefore subject to serious buckling.

The recorder(s) should be housed in a protected shelter (permanent shelter preferred) in a nearby location convenient for routine maintenance (fig. 49A). In order to simplify and facilitate cable installation and connection, the interconnect cable from the remote-accelerometers should terminate at terminal boards inside the instrument shelter (fig. 49B); from there other cables would lead to the recorder(s), which should be securely anchored to the floor or work bench (also to be securely anchored). In some cases, a heater or air conditioner may also be required if environmental temperature variations are expected to be severe enough to affect instrument operation (fig. 49A).

Because the installation of accelerograph equipment on and near highway bridges requires special skills and equipment and is often a complex operation, we recommend, if funding permits, that such instrumentation be



A, Shelter housing CR-1 recorder system; note air conditioner attached to side of shelter.



B, Terminal boxes and CR-1 recorder anchored to wall and work table inside instrument shelter.

Figure 49.- Typical CR-1 recorder installation.

installed by an agency or instrument manufacturer that has a substantial amount of experience in installing such equipment. Currently such organizations include the Seismic Engineering Branch of the U.S. Geological Survey (Menlo Park, California), the California Division of Mines and Geology Office of Strong-Motion Studies (Sacramento, California), and Kinemetrics, Inc. (Pasadena, California).

B. Maintenance Requirements

The amount of maintenance required by an accelerograph system is dependent primarily on the degree of sophistication of the instrument, the overall instrument design, the number of years in service, and the amount of instrument-improvement effort put forth by the instrument manufacturer and/or organization that maintains the instrument. Existing instruments such as the SMA-1 and CRA-1, which have been extensively utilized in the field and have undergone significant design changes to improve operational reliability, require minimum maintenance. Present versions of the SMA-1 accelerograph and CR-1 recorder (recorder for the CRA-1 system), for example, are maintained at 6-month and 4-month intervals, respectively, and the remote accelerometers (FBA's) for the CRA-1 systems require only rare maintenance (in most cases, no maintenance). Newer and more sophisticated instruments like the current models of digital instruments, on the other hand, require frequent checkout and maintenance because they have not yet established themselves as reliable field instruments.

The maintenance operation for the currently-used analog accelerograph systems (e.g., SMA-1's and CRA-1's) can easily be carried out by thoroughly trained experienced electronics technicians. On a typical inspection visit, each such accelerograph system undergoes an overall checkout, which includes a check on the accelerometer characteristics, battery charge, film age, trace alignment and focus, lamp voltage, proper instrument anchorage, and other operational parts of the system. Based on the experience of the Seismic Engineering Branch of the U.S. Geological Survey, which has maintained strong-motion instruments nationwide for approximately 50 years, we estimate that the maintenance of strong-motion instrumentation at a typical instrumented bridge site, which may contain as few as 13 channels or as many as 35 channels, or more, will require approximately one man-month per year. If an organization instruments a bridge but does not have the capability to maintain the instrumentation, we strongly recommend that that organization purchase maintenance services from the instrument manufacturer or another organization that has demonstrated its capability to properly maintain such instrumentation. In any case, it is absolutely critical that the instrumentation be routinely and properly maintained by highly competent personnel.

IX. AN EXAMPLE INSTALLATION: MELOLAND OVERCROSSING NEAR EL CENTRO, CALIFORNIA

The Meloland Road-Interstate Route 8 overcrossing, a two-span reinforced-concrete highway overpass bridge located 10 km east of El Centro, California and 0.5 km southwest of the Imperial fault, was designed in 1968-69 by the California Department of Transportation. The bridge is a continuous two-span cast-in-place reinforced-concrete box girder with 104-ft (32-m) spans (fig. 50) carrying two lanes of local traffic over Interstate Route 8. The bridge superstructure, which weighs about 7,100 lb per linear foot (10,570 kg per linear meter), is supported by an abutment at each end and a single column at the center of plan (fig. 50). The central support column is a 1.5-m-diameter reinforced-concrete column resting on a 4.5-m-square stepped 25-pile footing. The column is reinforced with 18 #18 bars confined by #5 spiral bars at 5-in. (12.7-cm) pitch. The bridge abutments are poured

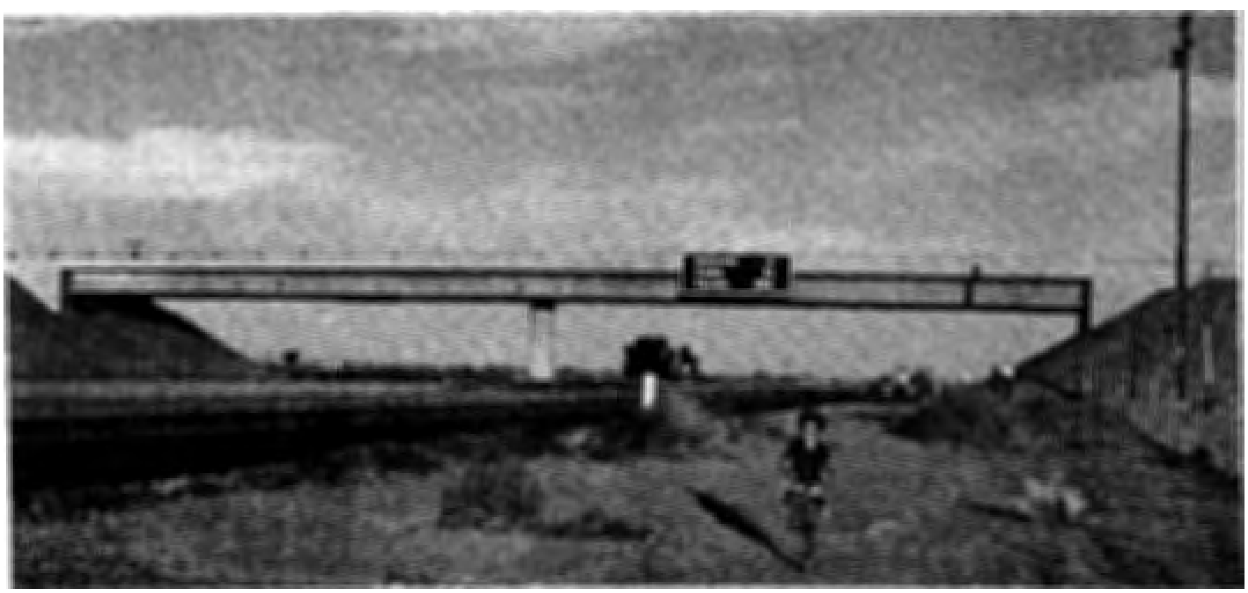


Figure 50.- View eastward of Meloland Road-Interstate Route 8 overcrossing.

monolithic with the superstructure. Each 18-in. (45.7-cm) abutment wall, which is nominally reinforced with #7 bars at 12 in. (30.5 cm) vertically and #4 bars at 18 in. (45.7 cm) horizontally, is about 1.5 m high and rests on a .9-m-by-.5-m seven-pile footing. The piles are 45-ton timber piles that extend 12 to 14 m into the alluvial foundation material. Logs from three soil borings at the site indicate that the foundation material consists of soft to stiff silty clay.

The bridge was selected for instrumentation under the California Strong-Motion Instrumentation Program because of its structural characteristics, size, and location in a known highly active seismic area. It was instrumented in November 1978 with two 13-channel Kinemetrics CRA-1 remote-accelerometer central-recording accelerograph systems that were installed in accordance with the recommendations of the following two groups: the California Seismic Safety Commission, Subcommittee on Instrumentation for Transportation and Other Lifeline Facilities; and an ad hoc site-visitation committee composed of the second author and J. H. Gates of the California Department of Transportation. The system was installed under the supervision of J. T. Ragsdale of the California Division of Mines and Geology (CDMG) and is consistent with the bridge strong-motion instrumentation guidelines described herein. The instruments are maintained by the CDMG Office of Strong-Motion Studies.

Instrumentation at the site consists of fourteen FBA-1 single-axis and one FBA-3 triaxial package of force-balance accelerometers on the bridge structure, two FBA-3 accelerometer packages on embankments adjacent to each abutment, one FBA-3 accelerometer package at a ground site intended to be free-field approximately 60 m west of the bridge, and two 13-channel central recording units and one VS-1 vertical starter at ground level beneath the bridge (fig. 51). The recorders are interconnected for common timing and real time is provided by a WWVB radio receiver.

The 13-channel recording units are housed in two fiberglass instrument shelters adjacent to the bridge's central support column (fig. 52A, B); the two FBA-3 accelerometer packages on the embankments and the free-field FBA-3 accelerometer package are housed in ground vaults just beneath the surface (fig. 53). The embankment sites are intended to record ground motion on the fill embankment material adjacent to each abutment; they are not intended to be free-field sites.

The FBA accelerometer locations on the bridge structure (actual installations are shown in figs. 54, 55) were selected to provide information on overall bridge response as well as base input motion. The primary purpose of the five east-west-oriented accelerometers and one north-south-oriented accelerometer on the bridge super-structure (accelerometers 3, 5, 7-9, 13, fig. 51) is to obtain and isolate east-west translational, north-south translational, torsional (about a vertical axis), and in-plane bending response of the roadway deck. In conjunction with the accelerometers on the footing of the central support column (accelerometers 2, 4, fig. 51) these accelerometers provide information on overall east-west and north-south modal response as well as relative motion between the base of the central support column and the bridge superstructure. The vertically oriented accelerometers on the roadway deck (accelerometers 6, 16-22, fig. 51) and on the footing of the central support column (accelerometer 1) provide information on vertical flexure (bending) of the roadway deck as well as twisting of the bridge deck about the roadway centerline.

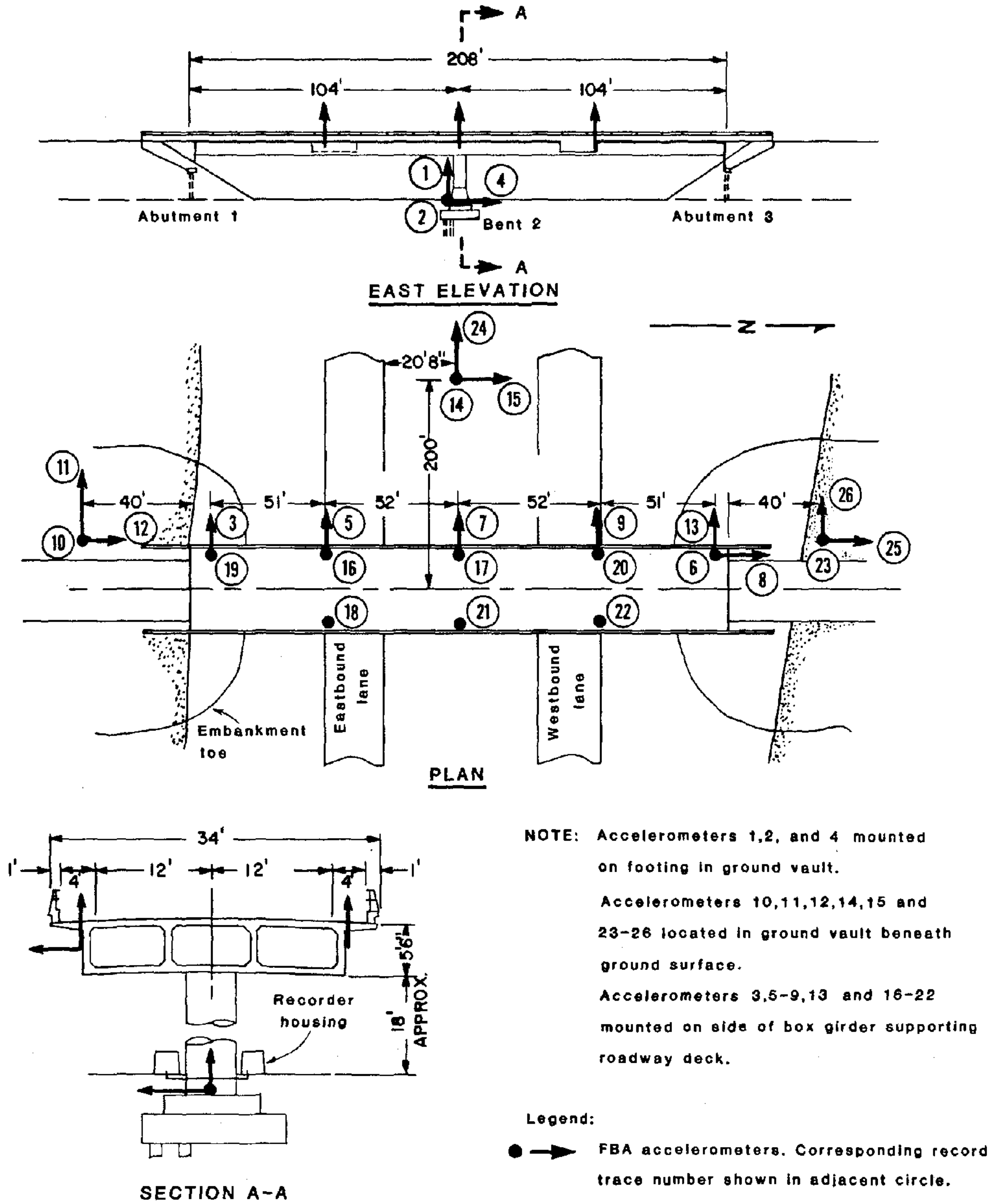
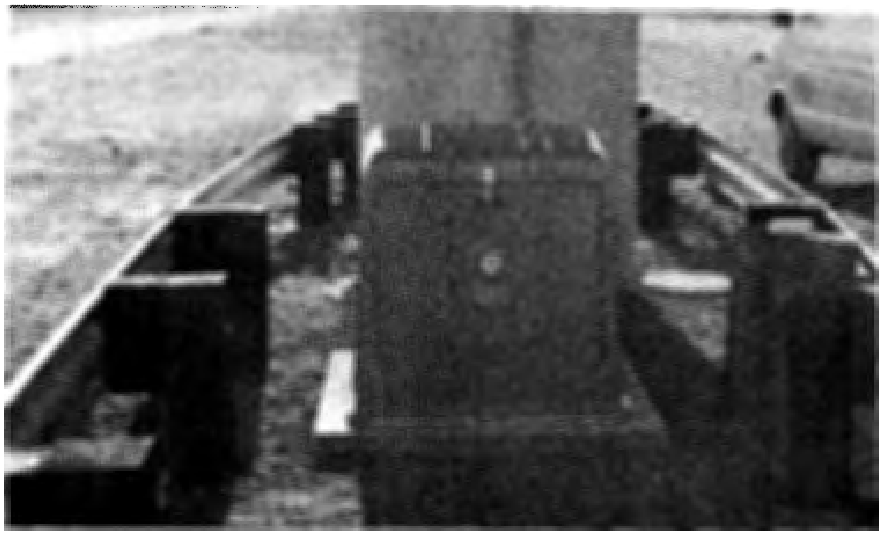
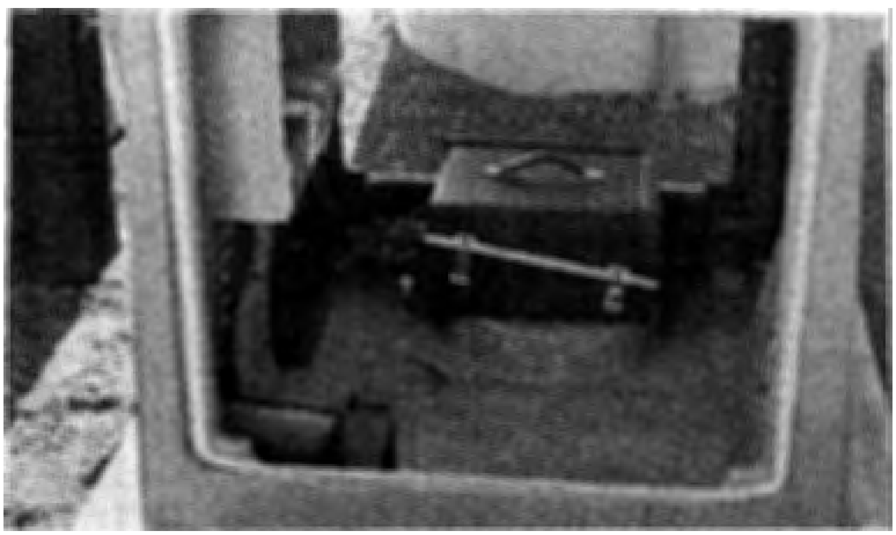


Figure 51.- Meloland Road-Interstate Route 8 overcrossing elevation, plan, and section showing location and orientation of FBA accelerometers.



A, Fiberglass shelter mounted on reinforced-concrete pad. Shelter located next to bridge central support column.

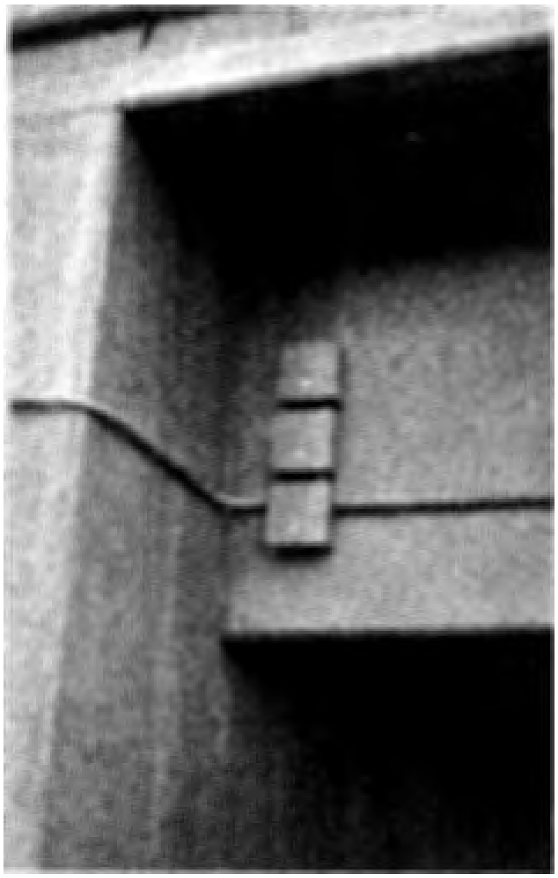


B, CR-1 recorder anchored to reinforced-concrete pad. Recorder is inside fiberglass shelter shown in A.

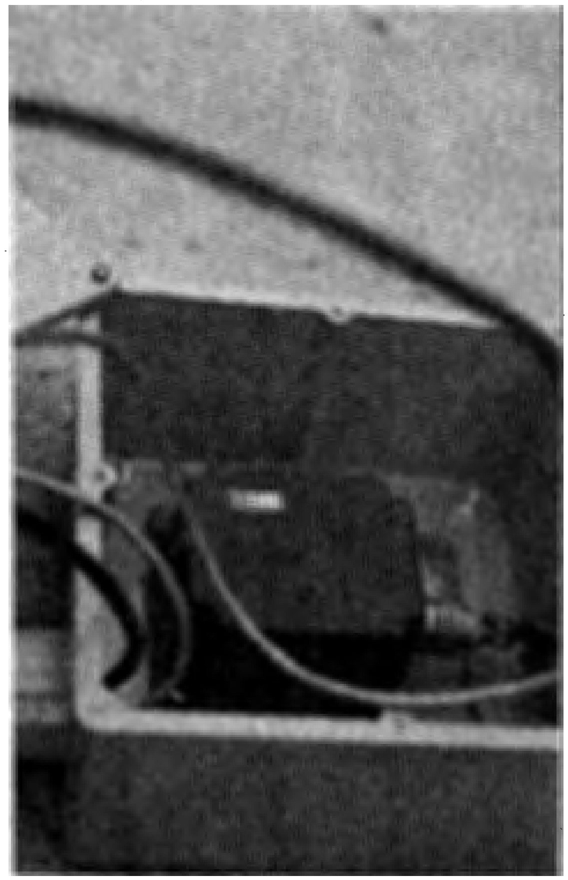
Figure 52.- Meloland overcrossing CR-1 recorder housing.

Figure 53.- Meloland overcrossing. View downward of FBA-3 accelerometer package anchored to small reinforced-concrete pad at bottom of ground vault.

Figure 54.- View southeastward of Meloland overcrossing. Note two pairs of FBA-1 accelerometers, which are encased in metal boxes (fig. 55B). PVC conduit used here to protect interconnect cable found to be highly unsatisfactory due to thermal expansion and contraction.



A, View eastward of three FBA-1 accelerometers in metal boxes anchored to deck near abutment 3.



B, Close-up view of FBA-1 accelerometer in metal-box housing.

Figure 55.- FBA-1 accelerometers attached to side of deck of Meloland overcrossing.

The instrumentation was triggered by the shallow-focus, magnitude 6.7, October 15, 1979 earthquake that was accompanied by surface movement on the Imperial Fault approximately 0.5 km from the bridge and the epicenter of which was located approximately 18 km to the southeast (Rojahn and others, 1980). Although the bridge did not sustain any significant structural damage during the earthquake, the strong-motion records from the bridge, included herein for information purposes, constitute an important data set. This is the first time strong-motion data had been obtained from an extensively instrumented structure situated less than 1 km from the surface-rupture zone of a moderate-magnitude earthquake that severely damaged some structures (not the bridge).

Although both accelerograph systems operated during the October 15 earthquake, the record from one system was flawed to a limited but significant extent because the recorder film transport stalled during the earthquake. The malfunction, which the manufacturer subsequently discovered could be avoided by using an aluminum rather than a plastic film take-up magazine, occurred at intervals of roughly 3 s during the first 30 s of operation and is characterized by overtracing on the record (fig. 56). The CDMG hopes to extract from this record a reasonably accurate facsimile of the actual waveforms (L. D. Porter, oral commun., 1980); waveforms from corresponding channels on the other 13-channel record (fig. 57) will then be used to evaluate and improve the restored data. The fact that corresponding components at the two embankment sites (traces 10-12, fig. 56; traces 23, 25, 26, fig. 57) were not recorded on the same system should help in this regard. In its original state, the flawed record (fig. 56) provides peak acceleration data and frequency characteristics as well as an overall sense of the

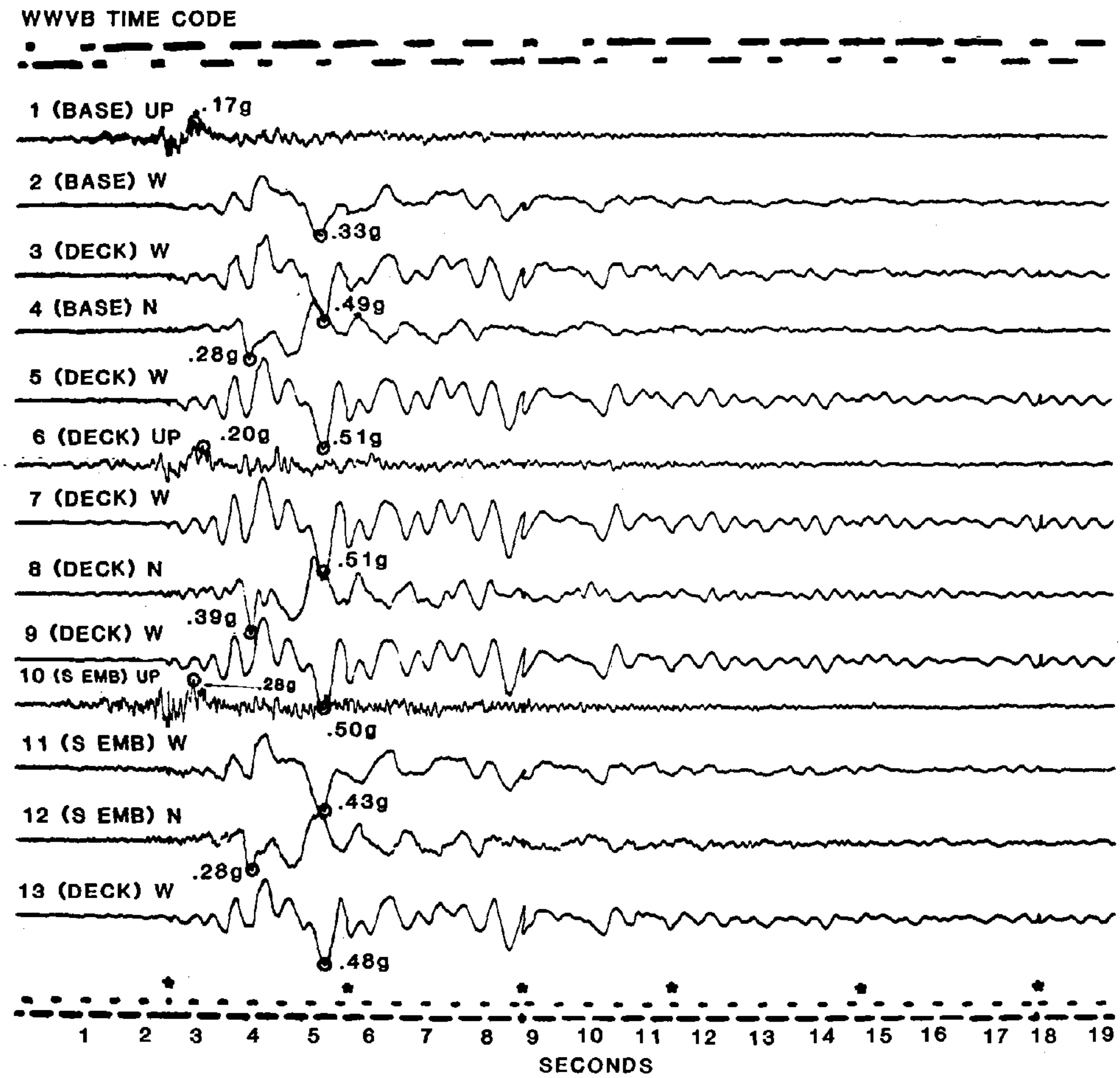


Figure 56.- First 19 seconds of October 15, 1979 strong-motion accelerogram recorded at Meloland Road-Interstate Route 8 overcrossing on master recorder. N (north), W (west), or Up in trace identification denote direction of positive acceleration. Accelerometer locations are shown in figure 51. Asterisks denote times at which recorder film transport system stalled.

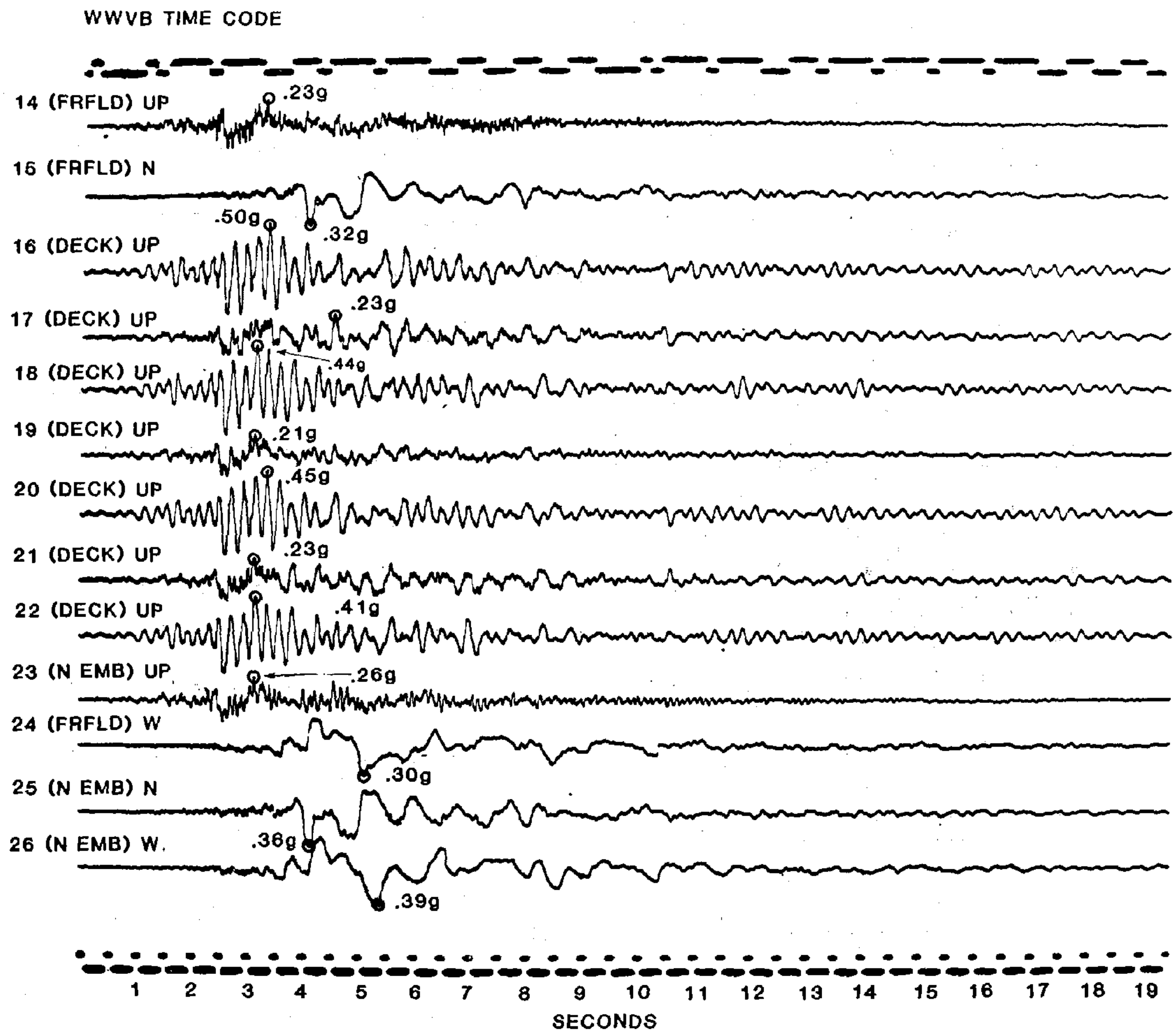


Figure 57.- First 19 seconds of October 15, 1979 strong-motion accelerogram recorded at Meloland Road-Interstate Route 8 overcrossing on slave recorder. N (north), W (west), or Up in trace identification denote direction of positive acceleration. Accelerometer locations are shown in figure 51.

strong-motion at the site.

Peak accelerations at the free-field site 60 m west of the bridge were 0.32, 0.23, and 0.30 g, respectively, for the north-south, vertical, and east-west components (traces 15, 14, 24, fig. 57), whereas peak accelerations for comparable components at the base of the bridge's central support column were 0.28, 0.17, and 0.33 g, respectively (traces 4, 1, 2, fig. 56). Although the peak values for the column-base horizontal components differed slightly from those recorded at the free-field site, the signatures of corresponding components are generally the same. The signatures of the vertical components at these sites (trace 1, fig. 56; trace 14, fig. 57), however, differ significantly. The peak vertical accelerations are similar, but high-frequency motions in the free-field record are substantially larger in amplitude than their counterparts in the column-base record. This is to be expected since the high frequency motion of the base of a structure is suppressed by the presence of the structure itself.

Corresponding motions recorded on the embankment sites adjacent to each abutment differ somewhat from those recorded at the base of the bridge's central support column. In general, the waveforms are the same: the east-west components (trace 11, fig. 56; trace 26, fig. 57) contain the approximately 1-second-long acceleration pulse between seconds 2 and 4 that is apparent in the column-base record. The records differ in that peak accelerations recorded at the embankment sites are generally higher than those recorded at the base of the central support column. They therefore suggest that the embankment motions were strongly affected by the presence of the embankments themselves.

Among the most notable features of the acceleration time histories recorded on the bridge deck are: (1) a dominant frequency of approximately 2.3 Hz in the east-west components (traces 3, 5, 7, 9, 13, fig. 56); and (2) a dominant frequency of approximately 4.5 Hz in the midspan vertical components (traces 16, 18, 20, 22, fig. 57). We also note the coherence of waveforms in several groups of traces from comparable locations and components (traces 16, 18, 20, and 22; 17 and 21; 19 and 6; 3, 5, 7, 9, and 13; 11 and 26). The waveform similarities within each of these groups reflect semi-rigid body motion. The striking differences in waveforms between groups, however, reflect the modal response of the bridge during and after the time of strongest ground shaking. Between seconds 2 and 4, for example, the vertical-component time histories recorded at the center of each span (traces 16, 18, 20, 22, fig. 57) contain high-amplitude cyclic motion at a frequency of approximately 4.5 Hz (vertical-deck modal response) that does not occur in the vertical deck components recorded adjacent to the central support column and near the abutments (traces 17, 21, 19, 26, fig. 57). Similarly, between seconds 3 and 6 the east-west component time histories recorded on the roadway deck (traces 3, 5, 7, 9, 13, fig. 56) contain approximately 2.3 Hz cyclic motion (lateral-deck modal response) that does not occur in corresponding-component time histories recorded at the north and south embankment sites (trace 11, fig. 56; trace 26, fig. 57). The cyclic motion (deck response) in both the vertical and east-west directions continued at smaller amplitudes after the period of strongest ground shaking had ended. In addition, deck rotation about a horizontal axis parallel to the roadway centerline is evident in the vertical deck components recorded adjacent to the central support column (traces 17, 21, fig. 57) because, starting near second 5 and continuing through second 19, the two vertical motions are 180° out of phase.

X. CONCLUDING REMARKS

This report, which has dwelled extensively on suggested guidelines for the placement of strong-motion accelerograph instruments on highway overpass bridges, has also included discussions on criteria for selecting highway bridges for strong-motion instrumentation, recommended instrumentation types, instrument installation and maintenance, and an example installation on an actual bridge near El Centro, California. As of the time of this writing, 8 highway overpass bridges nationwide have been instrumented in accordance with the principles presented herein: 4 in the State of California, 3 in the State of Washington, and 1 in the State of Missouri. Three other bridges--a suspension bridge near Long Beach, California, a cable-stay bridge near Sitka, Alaska, and a 3-span approach section to a suspension bridge near Massena, New York--have also been instrumented in accordance with these principles. Data obtained from these bridges should not only enhance our understanding of bridge behavior, but should also tell us how we can improve strong-motion instrumentation schemes in the future.

Continued field experience with the current generation of strong-motion instruments will also provide significant new lessons regarding instrument performance and reliability. Recent experience has shown, for example, that remote-accelerometer central-recording systems may be seriously damaged if the bridge on which the system is installed is struck by lightning. Unexpected as it was, this problem has occurred three times (twice at the same site) and is believed to be directly related to the fact that the affected bridges were constructed entirely or substantially of steel. Recent experience has also revealed instrument deficiences, such as the film transport problem with the CRA-1 system and numerous technical problems with today's digital-recording instruments. When such problems have been found and are resolved, as in the case of the CRA-1 film transport system, instrument reliability is enhanced. When such problems have been found but cannot be resolved (at least in the short run), alternate courses of action, which may offset the severity of the problem, often become immediately apparent. In the case of the lightning problem, for example, one obvious course of action would be to instrument steel bridges only in areas where the occurrence of lightning is rare. Similarly, the problems that have been experienced with digital-recording instruments simply suggest that such instruments are not yet field worthy.

Other lessons about bridge behavior and strong-motion instrumentation will undoubtedly be learned as more data are obtained from instrumented bridges. Eventually, new generations of instruments and new data analysis techniques will be utilized, and these may require that the guidelines recommended herein be altered substantially. In the meanwhile, these guidelines should provide important state-of-the-art technical information for those organizations charged with the responsibility of selecting highway overpass bridges for strong-motion instrumentation, designing the instrumentation schemes for those bridges, and installing and maintaining that instrumentation.

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APPENDIX

A DISCUSSION ON THE POSITIVE AND NEGATIVE ASPECTS OF RECORDING VARIOUS QUANTITIES OF MOTION

The dynamic response of a bridge to seismic induced ground motions is governed by an equation of the form

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\} = -\{M_x\}\ddot{U}_{xg}-\{M_y\}\ddot{U}_{yg}-\{M_z\}\ddot{U}_{zg}$$

where

$[M]$	= mass matrix;
$[C]$	= damping matrix;
$[K]$	= stiffness matrix;
$\{M_x\}$, $\{M_y\}$, $\{M_z\}$	are participation factors;
$\{U\}$	= vector of bridge displacements relative to the ground;
$\{\dot{U}\}$	= vector of relative bridge velocities;
$\{\ddot{U}\}$	= vector of relative bridge accelerations; and
$\ddot{U}_{xg}$, $\ddot{U}_{yg}$, $\ddot{U}_{zg}$	are three mutually perpendicular ground accelerations.

The matrices of coefficients and vectors of participation factors may be weakly time dependent, and the response may be dependent upon more than the translational accelerations at a single point. In general, however, the equations of bridge motion are roughly in this form. This form should be kept in mind when one considers which quantities of motion should be recorded. Following is a discussion on the positive and negative aspects of recording each of the four basic measurable quantities of motion: absolute acceleration, absolute velocity, absolute displacement, and relative displacement.

A. Absolute Acceleration

Most commercially available strong-motion recording systems record absolute acceleration. The primary reason for this is that absolute accelerations can be recorded easily and accurately (to within ± 3 percent) with a reasonably small recording package. There are also experimental reasons why absolute acceleration is a desirable quantity to record.

One of the primary experimental reasons for recording absolute acceleration is that seismically induced input to the bridge vibration is best recorded in this quantity. Earthquakes do not produce forces on bridges and other structures above ground; rather, they produce ground surface motions. Forces in the bridges initially are only inertial, resisting those ground motions. Since bridge masses are constant, those inertial resisting forces are proportional to the accelerations. Mathematically, this is evident from the equation of motion written previously. The only measure of earthquake input needed for analysis is absolute ground accelerations; hence, it is the most logical input quantity to record.

From an experimental point of view, absolute bridge accelerations are also meaningful quantities to measure because the forces acting on bridge components are proportional to absolute accelerations. In fact, if finding the total forces acting on the bridge is the principal objective of the instrumentation program, the absolute bridge accelerations are clearly the best quantities to measure.

There are certain problems, however, in recording absolute bridge accelerations if the objective is to identify or verify mathematical models of dynamic bridge behavior. Needed in the model identification procedure are relative (to the ground) bridge accelerations, relative bridge velocities, and relative bridge displacements. In order to obtain these quantities, much processing of the data is required. In the first place, ground accelerations must be subtracted from the recorded accelerations in order to produce relative accelerations. Time correlation between the records must be nearly exact to minimize errors. These relative accelerations must then be integrated once to yield relative velocities and twice to yield relative displacements. Integration naturally tends to filter out high frequency errors, but at the same time tends to greatly emphasize low frequency errors. These low frequency errors, which are most often present, must then be filtered from the velocities and displacements. High pass digital filtering always introduces additional errors of its own. Therefore, from what are acceptable absolute accelerations (± 3 percent) are computed velocities and displacements with questionable accuracy. These velocities and displacements are vitally necessary for model identification. Furthermore, stresses and strains, which are the ultimate measure of the integrity of the bridge, are computed from relative bridge displacements. If these displacements are not measured accurately, then experimental stresses and strains cannot be expected to be measured with accuracy either. One way around this dilemma is possible. Model verification can be accomplished by comparing theoretical accelerations to recorded accelerations. For such cases, computed relative velocities and displacements are not necessary, so the errors associated with these computations may be avoided. Then, <u>if</u> the model is verified, it can be used to compute theoretical stresses and strains which are likely to be close to those actually occurring. If the model is <u>not</u> verified, however, then it may be difficult to improve the model without resorting to model identification procedures which invariably will require relative bridge velocities and displacements.

One last word of caution about recording acceleration. Acceleration per se is not recorded, but recorded instead is an electric or optical signal that is proportional to the response of a simple damped oscillator which in turn is assumed to be proportional to absolute acceleration over a specific frequency range. When a quantity is recorded in such an indirect manner, there is always the possibility of the introduction of some spurious error.

B. Absolute Velocity

There is little physical justification for recording absolute velocity. Inertial input and response forces are not proportional to it. Elastic bridge forces are not proportional to it. Only damping forces, which are typically an order of magnitude less than inertial and elastic forces, are proportional to relative bridge velocities.

With all these shortcomings, there are mathematical advantages to recording absolute velocity. The first is that displacements may be computed from one integration only, and accelerations from one differentiation. Both computations involve errors, but far less than the errors that are likely to be associated with double integration of acceleration to obtain displacement. The second advantage is that low as well as high frequency signals are recorded with comparable magnitudes. Accelerations typically are dominated by high frequency signals and many important low frequency signals are masked by noise. Velocities are neither dominated by low or high frequency signals, so both are likely to be an order of magnitude greater than background noise.

Even with these advantages, absolute velocity is rarely recorded. Perhaps the principal reason for this is that the high amplitudes of velocity associated with strong-motion (150 cm/s or more) are difficult to measure.

Velocity transducers for these amplitudes of motion are difficult to build, primarily because there are severe technical design problems resulting from the fact that it is necessary to have a flat velocity response curve in the frequency range of interest in earthquake engineering. In fact, at present there simply does not exist in this country a reliable stable velocity measuring device appropriate for recording strong earthquake motion.

C. Absolute Displacement

Absolute ground and bridge displacements are desirable quantities to measure, but it is difficult to do so with any acceptable accuracy. Moreover, there are problems regarding the required physical size of the displacement measuring device (meter) for measuring strong-motions on the order of several feet. Displacements are easy to measure if the frequency of vibration is high because it is not required that the physical size of the displacement meter be large. For the case of bridges, however, which have natural frequencies that are typically low (1 hz or lower), any acceptable displacement meter must be large and cumbersome, and therefore impractical.

D. Relative Displacement

Relative displacement is perhaps the best bridge response to measure in conjunction with absolute ground acceleration. These quantities can be used to verify and identify mathematical models of bridge behavior with the least computational preprocessing necessary. Relative bridge velocities and bridge accelerations can be computed quite easily from the recorded relative displacements. Twice differentiating a displacement record is likely to be far more accurate than twice integrating the acceleration record. Strains can be computed from the relative displacements directly yielding a direct and accurate measure of the integrity of the bridge.

Although there are some problems, relative displacement is not particularly difficult to record. The flexure of a small beam, for example, can be measured electrically using electrical resistance strain gauges, or the relative displacement between two points may readily be recorded mechanically through a simple linkage system. The primary difficulty in recording relative displacement is the need for the two points, between which the displacement is to be recorded, to be physically connected. The resulting network of wires, rods, and cables may be unacceptable while at the same time being particularly vulnerable to intended or unintended damage. Another drawback is that if the entire motions of a structure are to be recorded experimentally (rather than a point or two for model verification), then a large network of relative displacement gauges must be employed throughout the structure. The simplicity of the gauge itself may make the use of several instruments in one structure quite feasible. And should a complete strain picture of a bridge be recorded during an earthquake, then a vast amount of information would, in fact, be obtained. Such extensive instrumentation, however, is probably beyond the realm of practicality. A more practical approach would be to supplement an array of absolute acceleration measuring devices with a few relative displacement meters. Such displacement meters could span across expansion joints or between conveniently located elements of the bridge and could, in conjunction with accelerations recorded on the structure at ground level, constitute an economically optimum recording system.

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