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Proceedings of A Workshop on

EARTHQUAKE RESISTANCE OF HIGHWAY BRIDGES

January 29-31, 1979

ATC APPLIED TECHNOLOGY COUNCIL

Sponsored by

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Proceedings of a Workshop on

EARTHQUAKE RESISTANCE OF HIGHWAY BRIDGES

Held on January 29, 30 and 31, 1979

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PREFACE

These proceedings document the results of a workshop on Earthquake Resistance of Highway Bridges held on January 29-31, 1979. The workshop was funded by the National Science Foundation and conducted by Applied Technology Council (ATC). The objectives of the workshop were to summarize the current state of knowledge and to establish research needs on seismic aspects of highway bridge design.

The proceedings contain the recommendations for future research, the twenty-three state-of-the-art and state-of-the-practice papers and four appendices. The appendices consist of the program, the participants, the members of the working groups, and a compilation of research publications.

ATC would like to thank all those individuals who contributed to the success of the Workshop. We are most appreciative of the efforts of Dr. John B. Scalzi, Program Manager of the Earthquake Engineering Program of the National Science Foundation, for his assistance during the planning of the workshop and for his continuous support and cooperation throughout the project. Our thanks to Mr. Richard Christopherson, ATC President, and the ATC Board of Directors for their helpful advice and support. Special thanks go to the Steering Committee—James Cooper, James Gates, James Libby, Joseph Penzien, and Robert Scanlan—for their assistance in organizing the workshop and their extensive input and efforts as chairmen of the working groups. Wilma Chappell, Administrative Assistant of ATC, deserves special mention for her assistance and dedication in handling all the organizational details and making the workshop an enjoyable experience for all participants. Her help in typing and editing the proceedings is also gratefully acknowledged. Finally, our sincere appreciation goes to all authors of the papers and all participants who took time from their busy schedules to contribute to the success of the workshop.

Funding for the workshop was provided by Grant No. PFR-7811802 from the National Science Foundation and their support is gratefully acknowledged. These proceedings constitute the final report to the National Science Foundation. The conclusions and recommendations expressed herein do not necessarily reflect the views of the National Science Foundation.

RONALD L. MAYES
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November 1979

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INTRODUCTION

Significant advances have been achieved during the last decade in the design and construction of seismic-resistant bridges. This progress has resulted from analytical and experimental research conducted at various institutions, as well as from lessons gained by inspecting damage caused by recent earthquakes.

Despite advancements in this field, significant gaps still remain in the understanding of the seismic behavior of bridges, and numerous areas exist in which specialists, both researchers and practitioners alike, continue to disagree. This is not surprising in view of the complexity of the seismic response of bridges and the multitude of structural systems, configurations, and details encountered in practice. Although additional research on seismic behavior is needed to solve these problems, this may not be sufficient by itself, since achievement of efficient seismic-resistant construction requires integration of knowledge obtained from many diverse fields. This integration is difficult because of the limited communication between experts working independently in different areas. Most of the available information has been published in widely dispersed publications or presented orally, and little effort has been made to assemble and integrate the data in a form that encourages its systematic discussion, evaluation, and dissemination among the various specialists in the field.

To improve this situation, it was felt that researchers, professionals, and representatives from industry and government working in the field of earthquake-resistant design of bridges should be brought together in a workshop to discuss and evaluate the available information and to determine priorities for future research needs.

OBJECTIVES

The main objectives of the workshop were to (1) evaluate current knowledge and practice in the planning, design, and construction of earthquake-resistant bridges; (2) examine needs and priorities for immediate, as well as long-range, research required to minimize gaps in current knowledge and to improve current practice; and (3) improve communication and cooperation (at both national and international levels) between research and professional organizations, and between different research groups.

CONTENTS AND ORGANIZATION

To achieve these objectives, forty-seven specialists from professional and research disciplines were invited to attend and participate in a workshop held in San Diego, California, during January 29-31, 1979. The Workshop was conducted by Applied Technology Council (ATC) and sponsored by the National Science Foundation (NSF). It was organized by Ronald L. Mayes and Roland L. Sharpe of ATC and by a project steering committee whose members were selected on the basis of extensive knowledge and experience in the field.

Workshop activities were divided into two parts. Part 1 included the presentation and discussion of state-of-the-art and state-of-the-practice papers in seismic design of bridges. Experts in various areas were requested to present research and state-of-the-art papers. Open discussion followed each presentation. In Part 2, five working groups met to assess ongoing research in the seismic design of bridges, define research needs, and establish priorities for future research. The edited recommendations were circulated to all participants for their comments and priority ratings. Each working group chairman then reviewed and approved the final recommendations for his group. The final recommendations included in these Proceedings have been distilled from the discussions of the various participants and working groups and do not constitute an individual endorsement by a particular participant or organization.

Participation in the Workshop was by invitation. Forty-seven participants were selected on the basis of their experience in the field of seismic design of bridges, knowledge of current research programs in the field, and awareness of research needs or practical problems in the general field of earthquake engineering.

There were two classifications of participants: MAIN PARTICIPANTS were requested to prepare a comprehensive state-of-the-art or state-of-the-practice paper; REGULAR PARTICIPANTS were invited to participate voluntarily in the discussions of the papers and working groups. All participants were assigned to serve on one of the five working groups. Participants were also requested to submit a list of draft recommendations to be considered by the appropriate Workshop working group. These draft recommendations were distributed to the other participants, along with preprints of the technical papers, prior to the Workshop.

The Workshop Proceedings include the state-of-the-art and state-of-the-practice papers, the final Recommendations, a list of the steering committee and participants (Appendix A), a list of the working group members (Appendix B), and a compilation of research publications (Appendix C) related to the field. The publication compilation contains only those references supplied by the participants and is not comprehensive. It is provided to serve as a directory of current research. It is hoped that this directory can be completed and updated in the future for the benefit of researchers and practitioners working in this field. Responsibility for the contents of the papers rests solely with the individual authors. The texts and illustrations of the papers have been reproduced from camera-ready originals supplied by the authors.

SUMMARY OF RECOMMENDATIONS: IDENTIFICATION OF HIGH PRIORITY NEEDS

The Recommendations formulated during the Workshop deal with a wide variety of research, development, and other needs for improving the seismic design of bridges. Priorities have been assigned to these recommendations by the participants. It is hoped that recommendations in this form will serve as guidelines to researchers and sponsoring agencies for current and long-term research needs.

After reviewing the final Recommendations, the steering group attempted to identify needs of highest overall priority, or of common concern to several working groups. Among those identified, the following deserve special mention.

1. Cooperation and Communication

Every effort should be made to improve cooperation and communication between researchers and professionals, as well as between researchers themselves. Effective exchange of research information should be accomplished on both a national and an international basis.

2. Evaluation and Dissemination of Available Data

Effective methods are needed for reviewing and evaluating available data and disseminating pertinent design-oriented technical information in simple, comprehensive terms. Effective evaluation and dissemination will require precise definition and agreement on the main parameters controlling bridge performance, and formulation of guidelines for collecting and reducing data and presenting

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FINAL RECOMMENDATIONS

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WORKING GROUP 1

ANALYTICAL PROCEDURES AND MATHEMATICAL MODELLING

Working Group 1 discussed research needs related to analytical procedures and mathematical modeling. The group emphasized that analytical and experimental research should be integrated and recommended that, wherever possible, both simple and sophisticated methods of analysis be verified by experiments or field measurements. In general, research is required for both simple and advanced methods of analysis and the research and development needs are divided into three major categories. Category A includes five recommendations related to computer-program and analytical-procedure developments. Category B includes three recommendations related to mathematical modelling. Category C includes seven recommendations for parameter studies.

A. COMPUTER PROGRAM AND ANALYTICAL PROCEDURE DEVELOPMENTS

1. DEVELOP A USER-ORIENTED COMPUTER PROGRAM SPECIFICALLY FOR THE DYNAMIC ANALYSIS OF BRIDGES USING SEGMENTS OF EXISTING PROGRAMS

General analysis computer programs currently available to bridge designers tend to be all-inclusive, requiring knowledge in both structural dynamics and structural mechanics. The current trend in developing enhancements on existing and new programs is toward even more generality which makes dynamic analysis even more difficult for both the experienced and new user. In developing a program specifically for bridges, initial consideration should be given to linear dynamic analysis capabilities. Future expansion should include nonlinear capabilities.

The program architecture should provide incremented processing capabilities with an accessible data base to define and modify input data, examine intermediate results for possible errors, and store final output results for possible post-processing. Preprocessing to generate the models required to idealize the characteristics unique to bridges should be included in the program.

2. DETERMINE THE RANGE OF APPLICABILITY OF THE RESPONSE SPECTRUM METHOD OF DYNAMIC ANALYSIS

The methodology currently recommended by AASHTO for the dynamic analysis of complex bridges is the response spectrum method of analysis. The relative simplicity of this method, compared with time history response analysis, makes it much more usable for design purposes. Recent parameter studies on curved highway bridges, conducted at the University of California, Berkeley, indicate that the response spectrum method yields unreliable results for bridges having a high degree of curvature. The results of Recommendation A2, when available, should be used to determine the range of applicability for the response spectrum method of analysis.

3. DEVELOP ACCURATE METHODS TO COMBINE MODAL CONTRIBUTIONS TO THE MULTIMODE METHOD OF ANALYSIS

Various statistical methods used to combine modal contributions in the multimode method of analysis provide greatly varying results. An effective means of combining modal contributions is required to make the response spectrum method of dynamic analysis as accurate as possible.

4. DEVELOP A RATIONAL METHOD FOR DESIGNERS TO COMBINE THE EFFECTS OF EARTHQUAKE LOADINGS IN THREE ORTHOGONAL DIRECTIONS

Bridges with curved horizontal and vertical alignments have coupling between the component directions within each mode of vibration. Seismic provisions should consider the simultaneous application of earthquake loadings to to yield more realistic member forces.

5. DEVELOPMENT OF ADVANCED ANALYTICAL METHODOLOGIES

Research efforts should be directed toward the further development and generalization of methodologies for analyzing the three-dimensional (3-D) dynamic response of bridges to traveling seismic waves. Methodologies that merge continuum solutions (for representing foundation/soil intereaction effects) with finite element techniques (for representing the superstructure) should be emphasized in such development efforts, because of their strong capabilities for analyzing 3-D traveling wave effects. Such methodologies should eventually be extended to incorporate such features as embedded footings and pile foundations, deformable footings, visco-elastic layered soil media, topographic irregularities, structural nonlinearities (e.g., expansion joints), and transient excitations (through the use of Fast Fourier Transform techniques).

B. MATHEMATICAL MODELLING

1. DEVELOP A PRACTICAL AND ACCURATE METHOD TO ESTIMATE THE FUNDAMENTAL PERIOD OF BRIDGES

The fundamental period is required to obtain seismic coefficients for use in the single-mode spectral method of analysis. A simple, but reasonably accurate, method of estimating the period is essential.

2. CORRELATION OF VIBRATIONAL CHARACTERISTICS OF EXISTING BRIDGES WITH ANALYTICAL RESULTS

Although vibrational characteristics of bridges can be calculated, it is not certain that the basic assumptions or approximations inherent in a mathematical idealization give accurate results. Sensitive seisometers can be sued to obtain information from ambient testing and these results correlated with analytical results. In addition, when strong motion records of bridge response become available, these should be correlated with the ambient tests and the analytical results.

3. REVIEW AND DEVELOP (IF NECESSARY) SYSTEM IDENTIFICATION PROCEDURES FOR THE ANALYSIS OF LINEAR AND NONLINEAR RESPONSE MOTIONS TO SINGLE AND MULTIPLE INPUT MOTIONS

System identification procedures now exist to determine structural parameters from the measured or recorded response from single input motions. There are also procedures available for the analysis of nonlinear motions to a single input. These procedures have been developed primarily for the analysis of buildings and are only moderately successful when used to analyze bridges. It is recommended that existing procedures be reviewed and, if required, new procedures developed to analyze linear and nonlinear bridge response to single and multiple input motions to determine structural parameters of interest.

C. PARAMETER STUDIES

1. CONDUCT PARAMETRIC STUDIES FOR THE SEISMIC RESPONSE OF COMMON TYPES OF BRIDGES TO DETERMINE THE EFFECTS OF GEOMETRY AND CONSTRAINT ON OVERALL SEISMIC RESPONSE

Bridges should be classified by their gross seismic performance. Effects of span length, column height and stiffness, curvature, skew, type, material, restraint, etc., on the overall seismic response should be identified.

Recommendations can then be made for the use of appropriate types of structures which provide increased seismic resistance.

2. DEVELOP NONLINEAR COMPUTER CAPABILITIES FOR SHORT STIFF BRIDGES WHICH INCLUDE THE EFFECTS OF SOIL-STRUCTURE INTERACTION AT ABUTMENTS AND BENTS AND CONDUCT SENSITIVITY STUDIES TO PROVIDE GUIDELINES FOR DESIGNERS USING ELASTIC ANALYSIS TECHNIQUES

The effect of soil-structure interaction at the abutments and bents significantly influences the overall response of short stiff structures. These effects should be studied in greater detail to develop guidelines for the designer. Approximate spring and damping systems which can be incorporated in an elastic analysis should be developed for the designer.

3. DEVELOP IMPROVED NONLINEAR THREE-DIMENSIONAL DYNAMIC ANALYSIS CAPABILITIES AND CONDUCT SENSITIVITY STUDIES TO PROVIDE THE DESIGNER WITH GUIDELINES FOR DUCTILITY REDUCTIONS

Current earthquake design criteria for bridges are based on the philosophy that the bridge should remain functional after a maximum credible earthquake and that, where possible, damage that does occur should be readily detectable and accessible for inspection and repair. This philosophy implicitly relies on post-elastic deformation of columns to dissipate most of the seismic energy input during a major earthquake.

The nonlinear behavior of a bridge is a complex problem involving the interaction of nonuniform progressive yielding of columns, the nonlinear behavior at intermediate expansion joints and bearings, and the effect of the nonlinear behavior of the foundation. Nonlinear three-dimensional computer capabilities with stiffness and/or strength degradation should be further developed to provide realistic models for the cyclic loading that a bridge experiences during an earthquake. This program should also include the nonlinear behavior that occurs at the foundation, in the columns, at the expansion joint as gaps open and close, bearings slide, restrainers take up and yield, and energy absorption devices deform. Sensitivity studies should then be conducted using the developed program to study the overall energy absorption characteristics of bridges and the ductility demands imposed on the columns. The effects of yielding during moderate-level earthquakes in structures with nonuniform columns should also be considered. Information obtained from these studies would be used to establish guidelines for ductility demands that would ensure adequate design for both the maximum credible earthquake and moderate earthquake.

4. PERFORM DYNAMIC ANALYSES TO INVESTIGATE THE INFLUENCE OF FOUNDATION AND BEARING COMPLIANCE ON DUCTILITY DEMAND OF BRIDGE PIERS

The range of earthquake and structural characteristics considered in the dynamic analysis of bridge systems with additional flexibility resulting from foundation compliance and bearing deformation should be extended.

5. DETERMINE ANALYTICALLY THE BOUNDS OF RESPONSE LIKELY TO BE EXPERIENCED BY BRIDGE COMPONENTS

A major analytical task is to establish guidelines for experimental work. Naturally, it should be directed at commonly used components first. The task should then be aimed at identifying the ranges of the problems rather than solving them.

The study should attempt to narrow the ranges of interest for combinations of the following parameters.

a. Axial load, bending, shear
b. Size and shape of cross section
c. Cross section varying along length (nonprismatic)
d. Material properties
e. Loading program (include effects of all motions)
f. Ground motion intensity and characteristics

6. PERFORM AN IN-DEPTH STUDY TO IDENTIFY THE TYPES OF BRIDGE BEARINGS CAPABLE OF RESISTING LOADS ASSOCIATED WITH AREAS OF VARYING SEISMIC RISK

Most commonly used bearings are not designed to resist large lateral loads. The upper bound of lateral resistance for each type bearing should be determined. The bearings should then be categorized for areas of appropriate seismic risk.

7. PERFORM DYNAMIC ANALYSES TO INVESTIGATE THE RESPONSE OF PRESTRESSED CONCRETE BRIDGES TO VERTICAL SEISMIC ACCELERATIONS

Because of low damping and high dead load/live load ratios, some prestressed bridges may be susceptible to superstructure damage under vertical ground accelerations, particularly if prestress overbalances dead load. Increased pier axial loads may also influence available ductility capacity.

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WORKING GROUP 2

FOUNDATIONS, ABUTMENTS AND GROUND MOTION EFFECTS

Working Group 2 reviewed and discussed current knowledge and practice as it relates to the design of foundations and abutments. The Working Group also considered ground motion effects as they relate to bridges. The recommendations are presented in four categories. Category A contains four recommendations on the needs of expected ground motions and their effects related to bridges. Category B consists of five recommendations regarding foundations. Category C identifies two recommendations related to abutments. Category D lists one recommendation regarding soil properties. Working Group 2 did not spend a significant amount of time on Category D, as a workshop was recently held on this aspect for all civil engineering structures.

A. GROUND MOTION EFFECTS

1. DEVELOP ELASTIC DESIGN SPECTRA FOR VARIOUS DEGREES OF DAMPING BASED ON EFFECTIVE (DESIGN) PEAK VALUES OF ACCELERATION, VELOCITY AND DISPLACEMENT

2. DEVELOP EFFECTIVE (DESIGN) VALUES OF PEAK GROUND MOTION TO PERMIT ESTIMATES OF TRANSLATIONAL AND ROTATIONAL MOTION AT AN ABUTMENT OR SUPPORT POINT, AND TO PERMIT ESTIMATION OF RELATIVE MOTIONS BETWEEN SUPPORT POINTS

3. DEVELOP TIME HISTORIES, OR SUGGESTED GUIDELINES FOR ESTIMATING TIME HISTORIES, FOR CARRYING OUT NONLINEAR AND LINEAR CALCULATIONS WHOSE RESULTS MAY BE FACTORED INTO THE DESIGN PROCESS AS PART OF OVERALL PARAMETER STUDIES

The seismic design of bridges is dependent upon a sound knowledge of the expected ground motions at the site and expected variations, if any, at and below the site surface. There should be an ongoing program of research by personnel of governmental agencies, private firms, and universities to review and upgrade our seismological and geological bases of design in the light of engineering needs and applications. The upgrading studies should be based on existing and newly acquired data. For the design of bridges, the result of such studies should be in the form expressed in the above three recommendations.

4. EVALUATE EFFECTS OF SPATIALLY VARYING GROUND EXCITATIONS ON THE BEHAVIOR OF BRIDGES DURING EARTHQUAKES

Spatially varying ground excitations that propagate with various directions of incidence in the vicinity of a bridge structure can, under certain conditions, have significant effects on the bridge response. In view of this, further research is recommended to increase our understanding of such effects. This research should involve development of results from suitable analytical procedures and from field measurements of the earthquake induced response of bridges using suitably designed arrays of bridge instruments. These results should be used to:

a. Identify bridge response phenomena that results from spatially-varying ground excitation.

b. Assess how these phenomena are influenced by the bridge configuration, soil properties, and characteristics of the ground excitations (e.g., predominant wave types, etc.).

c. Using a parameter study, develop simplified design procedures that include effects of spatially-varying ground excitations. The procedures should include guidelines regarding when such effects should be considered in the bridge design process and when they can be neglected.

B. FOUNDATIONS

1. DEVELOP IMPROVED NONLINEAR AND EQUIVALENT LINEAR DEFORMATION RELATIONSHIPS WHICH ACCOUNT FOR THE INTERACTION BETWEEN SOILS AND PILES FOR PILED FOUNDATIONS: FULL SCALE TESTS ARE RECOMMENDED TO VALIDATE THE RELATIONSHIPS

The flexibility of piled foundations can have a significant effect on the seismic response of a bridge. The effect is usually considered, in analytical models, through the use of various linear and nonlinear spring and dashpot systems, finite elements and continuum models. The properties of these systems require considerable refinement and additional research. The use of cyclic load tests on full-scale specimens is recommended to verify the analytical techniques and models developed.

As this information becomes available, it is recommended that it be used in parametric studies to evaluate the effects of foundation flexibility of piled foundations on the overall seismic response of a larger bridge.

2. DEVELOP PRACTICAL DESIGN MITIGATION RECOMMENDATIONS FOR BRIDGE FOUNDATIONS AT SITES HAVING A HIGH LIQUEFACTION RISK

Where siting restrictions and the economics of remedial measures to improve resistance to liquefaction necessitate foundation design for a high risk liquefaction area, it is desirable to minimize the risk of structural failure by using appropriate foundation types and design methods. By undertaking a series of parametric studies using a dynamic soil-pile-structure interaction model incorporating degradation in lateral resistance in liquefiable soil zones, it would appear possible to provide guidelines for selecting various design methods. It is envisaged that one potential solution is to found piers and abutments on long ductile steel pipe piles, and to provide adequate setback of abutments and piers from stream, fill or cut slope faces. Results of such a study would have immediate application to routine bridge design where detailed dynamic studies are not warranted.

3. PERFORM AN ANALYTICAL STUDY COUPLED WITH PARAMETRIC EVALUATIONS AND FULL SCALE TESTS ON BRIDGE TYPE FOUNDATIONS TO BETTER DEFINE THE DYNAMIC STIFFNESS CHARACTERISTICS OF SHALLOW ISOLATED SPREAD FOOTINGS FOR BRIDGE STRUCTURES. THIS SHOULD INCLUDE THE NONLINEAR EFFECTS OF ROCKING AND TORSION AT HIGH LEVELS OF LOAD

As with piled foundations the flexibility of spread foundations can have a significant effect on the seismic response of a bridge. When necessary this effect is usually considered through the use of linear and non-linear spring dashpot systems in analytical models. It is the properties of these systems that require refinement and experimental verification.

4. DEVELOP GUIDELINES ON THE ADVANTAGES AND DISADVANTAGES OF VERTICAL AND BATTER PILES SUBJECTED TO SEISMIC GROUND MOTIONS

Both vertical and batter piles are used in bridges subjected to seismic ground motions. The decision making process on the pros and cons of each system is generally based on the experience of the designer. Detailed case studies on the effect of the two pile types on the seismic performance of a bridge is generally not economical. Thus a series of parameter studies would highlight the advantages and disadvantages of each system. The guidelines developed as a result of such a study would enable a designer to quickly assess the type of pile system best suited for his bridge type and soil conditions, and give him a better basis for decisions and implications of each system.

5. ADDITIONAL INFORMATION IS REQUIRED ON THE EXPECTED RELATIVE HORIZONTAL DEFORMATIONS OF DIFFERENT TYPES OF SUB-SOIL CONDITIONS SUBJECTED TO SEISMIC MOTIONS.

Soil layers may be subjected to significant relative horizontal deformations when subjected to seismic motions. The degree of this deformation is not well understood and thus requires further study. Its effects on piles is important in that it may cause significant curvatures in piles which should be accounted for in design.

C. ABUTMENTS

1. PERFORM LARGE-SCALE OR FULL-SCALE TESTS ON REPRESENTATIVE ABUTMENT TYPES INCLUDING MONOLITHIC, JOINTED, TIED-BACK AND FREE-STANDING TYPES, TO DETERMINE THEIR FORCE/DEFORMATION AND ENERGY DISSIPATION CHARACTERISTICS UNDER CYLIC LOADING: THE TEST PROGRAM SHOULD BE COUPLED WITH A CORRELATION STUDY USING SIMPLIFIED ANALYSIS TECHNIQUES

Observed damage in past earthquakes indicates that abutments are susceptible to damage. However, their response to strong ground shaking and their interaction with the superstructure is not well understood. Furthermore, little information is available on both the force-deformation and energy dissipation characteristics of abutments and their backfill. When the information is available, it is recommended it be used to study force levels and relative deformation of actual abutment damage observed in past earthquakes.

2. PERFORM A STUDY COMPARING THE EXPECTED PERFORMANCE OF END DIAPHRAGM ABUTMENTS HAVING SACRIFICIAL WING WALLS, WITH AND WITHOUT PILES, WITH THE ANTICIPATED PERFORMANCE OF BACKWALL-TYPE ABUTMENTS HAVING AND NOT HAVING ENERGY DISSIPATORS

A major area of concern in the seismic design of bridges is that of abutments. To avoid significant damage during earthquakes, comparative studies of viable alternatives indicated in this recommendation are needed.

D. SOIL PROPERTIES

1. DEVELOP APPROXIMATE CORRELATIONS BETWEEN RESULTS FROM EXISTING WELL-ESTABLISHED FIELD TESTING METHODS (SHEAR WAVE VELOCITY, STANDARD PENETRATION TEST, CONE PENETRATION TESTS) AND THE NONLINEAR STRESS-STRAIN CHARACTERISTICS OF SOIL UNDER CYCLIC LOADING.

The nonlinear stress-strain or modular characteristics of soils under cyclic loading are an essential starting point for dynamic soil-structure interaction studies, and form the basis for assessing foundation or abutment stiffness factors. A number of empirical correlations already exist in the literature between results from the above field tests and deformation or strength characteristics of soils. Such correlations need to be critically examined with respect to cyclic loading properties, and additional field and cyclic laboratory tests performed to improve correlations where necessary. When available this will provide a rapid and field-oriented procedure for providing nonlinear soil stiffness parameters for input into design procedures which require foundation stiffness coefficients.

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WORKING GROUP 3

EXPERIMENTAL INVESTIGATIONS

Working Group 3 discussed research needs realted to experiments on actual bridges, large-scale laboratory experiments, and correlation of analytical and experimental investigations with observed earthquake damage. It was agreed that dynamic behavior of materials, simplified structural systems, and composite structures are preferably examined through laboratory (or field) experiments employing vibration excitors, actuators, or shaking tables. To maximize the usefulness of model and field experiments, it is recommended that appropriate international cooperation be developed among research groups in various countries concerned.

The concensus of the Working Group was to divide the research and development needs into three major categories: Laboratory Testing, Field Testing, and Instrumentation. Each category is subdivided and relative priorities are listed.

The Working Group felt strongly that high priority should be given to evaluating and upgrading existing bridges in seismically active areas. Selection of such bridges should be based on careful analysis of life safety and cost benefit studies, and the details are covered in Working Group Four's recommendations.

A. LABORATORY TESTING

Large-size, multi-directional shaking tables are extremely useful for the study of many problems of bridge aseismic design, such as the modeling of soil properties, soil-foundation interaction, dynamic behavior of complex bridge systems, etc. The small number and high cost of such facilities should encourage international cooperation in their development and use.

1. CONDUCT STUDIES TO DETERMINE DUCTILITY CAPACITY OF REPRESENTATIVE CONCRETE BRIDGE COLUMNS AND PIERS

There is a pressing need for experimental and analytical studies to determine the reserve capacities of various bridge components. Much of the considerable research work on column behavior has been done on relatively small specimens and has been extrapolated for bridges from tests of columns typically used in buildings. Bridge columns are larger and lower stressed than building columns and this does not permit easy extrapolation from the present wealth of building column data. Therefore, work is needed to determine whether the behavior of small sections can be extrapolated to larger cross sections. For example, the efficiency of transverse reinforcement may not be the same for a large section as for a small one.

Specific research needs that fall under this category are:

a. The dynamic stress-strain characteristics of confined concrete.

b. The inelastic flexural response of confined circular, rectangular, and hollow-section bridge columns under biaxial lateral loads and axial loads expected during earthquakes.

c. The shear strength of bridge columns and, in particular, the extent to which the shear contribution of concrete can be relied upon.

Recent tests have given a measure of confidence to the likely performance of well-detailed and -confined circular reinforced concrete bridge columns. However, more testing is required to investigate the ductility capacity of:

a. Columns at medium to high axial load levels.

b. Columns of other section shapes (rectangular, elliptical, hollow box section).

c. Plastic hinge zones where longitudinal reinforcement is lapped.

d. Columns subjected to seismic displacements in two orthogonal directions.

e. Columns with high moment/shear ratios.

2. OBTAIN EXPERIMENTAL DATA ON THE PERFORMANCE OF COMPLETE BRIDGE SYSTEMS TO CHECK THE VALIDITY OF ANALYTICAL PROCEDURES OVER THE TOTAL RANGE OF RESPONSE

Entire bridge systems must be tested to investigate:

a. The interaction of superstructure components with each other and with the substructure (including the effects of shear keys, restrainers, bearings, and expansion joints and energy dissipating devices).

b. The interaction of the substructure with the supporting foundation. The current limitations of shaking table facilities and scaled tests are recognized; i.e., problems of scaling, lack of differential ground motion, and the inherent size and frequency limitations of available facilities. However, only through development of tests of this type can the required data be obtained.

3. CONDUCT STUDIES TO DETERMINE RESPONSE CHARACTERISTICS OF JOINTS AND CONNECTIONS

There is a real practical need for development and publication of proven seismic resistant details which could be included directly into design criteria. Proven details such as column ties, reinforced shear keys, bearings, abutment details, column connections, girder connections, footings, etc., need to be presented in a form which will eliminate the need for an engineer to perform a lot of computations and yet will permit him to obtain, with some confidence, a reasonably seismic resistant bridge. Some of the specific needs that are required follow:

a. Perform Studies to Determine the Anchorage and Splice Requirements Under Cyclic Loads of the Large Diameter Bars Commonly Used in Bridges.

Beam-column joints in buildings have received considerable attention. The types of details and joints used in bridges do not have correlatable counterparts in buildings and so the data accumulated cannot be directly transferred. Special attention must be given to anchorage of large bars, particularly where a number of bars are anchored in close proximity. Very little work which is applicable to bridges has been done on splices of large-size bars under reversed cyclic loads. Special attention must be given to lapped and mechanical splices of large bars.

Bridge supports typically are constructed using #14 and #18 bars. Bars must be anchored in the footing and in the bent cap or superstructure depending on the type of construction. In addition, it may be necessary to splice bars in the pier. Very little experimental work has been done regarding the anchorage and development of large-diameter reinforcing bars. There is a need for tests simulating the connection between piers and footings and between piers and bent caps or superstructure. Tests should determine the free-deformation (slip) characteristics and strength of anchored bars or groups or bars in typical bridge configuration under cyclic load reversals. The performance of splices and the influence of splices on the force-deformation relationships for the piers should be examined. Alternatives to splices, such as mechanical anchors, should be studied. The experimental results can be used to develop design guidance for detailing large bar anchorages and for developing behavorial models which can be used in analytical studies.

b. Perform Studies to Determine the Dynamic Characteristics of Bearings Commonly Used in Bridges

Tests and studies are needed on the many critical hardware and detail components such as bearings, restrainers, and keys. The need to design for non-collapse conditions will require reliance upon these components to perform without failure.

The performance of bearings can significantly affect the seismic behavior of bridges. Research studies are needed to establish reliability of performance and force-deflection characteristics for use in analysis. Bearing studies should include elastomeric-rubber and low-friction sliding-type bearings. Similarly, the testing of restrainer-type devices should be pursued.

c. Conduct Studies to Determine the Dynamic Properties and Appropriate Application of Mechanical Energy-Absorbing Devices

Application of mechanical energy-dissipating devices to bridges has considerable potential although it is still in its developmental stages. Further dynamic testing of the devices is required, using earthquake acceleration records with large amplitude motions.

Several types of damping devices have been developed which are thought to be effective in reducing seismic forces in the superstructure and substructure through their energy absorbing capabilities. Performance of such damping devices when installed with bridge bearings should be determined from research studies including studies of their dynamic behavior when installed on actual bridges. Standardized criteria and guidelines are needed for design and installation of such devices.

d. Determine the Capacity of Anchor Bolts Embedded in Concrete Subjected to Cyclic Loads

Little work has been done to assess the capacity of anchor bolts embedded in concrete under cyclic loading. Present values are based on estimated factors of safety, generally derived from static loading, yet failure of this component can be as critical as failure of an embedded reinforcement bar. This research should include inserts, expansion bolts, and bolts with combined bending and shear.

e. Determine the Experimental Properties of Common Types of Expansion Joints and Identify How They are Modelled Analytically

During earthquakes, expansion joint components are subjected to complex stress states and their behavior is often crucial to the overall seismic performance of bridge structures. Many factors are involved in determining expansion joint requirements for bridge deck structures. Often thermal or construction considerations dictate their need. The potential for damaging effects of multiple impacting at expansion joints is recognized. How to account for these effects in design is not clear. Research is needed to clarify how to analytically model expansion joints and also to determine if they can be eliminated or significantly reduced in number. Representative components for existing structures should also be tested to determine their seismic response and whether retrofit measures are deemed necessary.

4. PERFORM SHAKING TABLE TESTS TO INVESTIGATE THE DYNAMIC CHARACTERISTICS OF COMPLETE BRIDGE STRUCTURES

Shaking table tests afford the possibility of evaluating the overall dynamic characteristics of a bridge. This is particularly important in more complex bridges where the interaction of components is difficult to determine analytically. Experimental data can be obtained on behavior of critical components such as connections, intermediate hinges, restrainers, columns, etc., as they interact with other components in the overall response of the bridge. Shaking table tests should be carried out on as large a scale as possible to ensure that scale effects do not become significant.

B. FIELD TESTING

It is essential that the results of laboratory tests on scaled components and structures be verified by field testing of existing bridges where feasible. Much can be learned from nondestructive testing on existing bridges; large-amplitude testing on bridges designated for demolition can also be very informative. The types of bridges to be tested should include single-span, double-span, and multi-span of reasonably long lengths.

1. LARGE-AMPLITUDE FORCED-VIBRATION TESTS ON FULL-SCALE BRIDGES

To determine structurally meaningful damping properties from full-scale tests, it is desirable to subject a bridge structure to vibration amplitudes at or near yield levels. As a practical matter, this requires that large-amplitude dynamic testing be carried out on bridges scheduled for demolition due to highway rerouting, etc., because of the possible damage that may result from the experiments. The effect of stiffness degradation could also be studied in experiments of this type.

2. NONDESTRUCTIVE DYNAMIC TESTS OF EXISTING BRIDGES

a. Ambient and moderate-level forced vibration tests should be conducted on full-scale bridges to provide basic data on their dynamic properties such as natural frequencies, mode shapes, and damping ratios. Where possible particular attention should be given to isolating the effects of soil structure interaction. The data derived from these experiments are required to calibrate analytical models used for theoretical seismic response calculations.

b. A class of bridges which should be sujected to nondestructive dynamic testing is that of bridges which are instrumented with strong-motion instruments for measuring seismic response during future earthquakes. Their dynamic properties may be obtained by pull testing, by mechanical vibration, or at least by ambient vibration studies. Future accelerograph recordings will be of greatest value if the actual dynamic response characteristics of bridges are known prior to any possible dynamic motions. If the properties of the bridge are known prior to the occurrence of damaging motions then it can be determined how damaging such motions were.

c. Another class of bridges which should be investigated is that of bridges under construction. Selected bridges should be subjected to dynamic testing at various phases of construction. For purposes of isolating the effects of soil-structure interaction, it would be very desirable to subject individual piers and bents to careful dynamic experiments. This could only be done during construction of the bridge. Other useful information regarding dynamic properties of the superstructure could also be obtained.

C. INSTRUMENTATION

Various types of bridges in seismic areas should be instrumented so as to promote further insight into the behavior of bridges during earthquakes. Although vibrational characteristics of bridges can be calculated, it is not certain that the present basic assumptions will give results comparable to actual response during an earthquake. Selection of the type and location of instruments to measure the response of bridges to traveling seismic waves should be guided by analytical studies and by prior observations of bridge response characteristics. In some cases dense arrays of instruments may be required to provide sufficiently detailed measurements to fully interpret the nature of the bridge response.

Instrumentation to measure response of bridge components and actual ground motion over appropriate distances from the bridge are required. The out-of-phase effects of ground motion may be very important, particularly for long bridges.

Instrumentation should be installed on and near bridges to measure the following types of motions and/or response:

a. Influence of abutments on response of small bridges.

b. Effects of pier foundation-ground interaction.

c. Periods and mode shapes of the bridge and its components.

d. Rotational components or ground motion and/or torsional response of the bridge.

e. Free field motions.

The above instrumentation should be installed on one or more of each of the following:

a. One-, two-, or three-span bridge structures.

b. Multispan bridges (six or more spans).

c. Long-span (500-600 feet or more) bridges.

Actual measurement of bridge response during an earthquake is the only reliable way to obtain data to compare with theory and analysis procedures including period calculations.

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WORKING GROUP 4

RETROFITTING EXISTING BRIDGES

Working Group 4 discussed methods for: identifying existing bridges that may be potentially hazardous; evaluating the nature and degree of the hazard; and developing procedures to improve their seismic resistance.

The Working Group first developed an outline and flow chart that identified the steps in the retrofit process - Figures 1 and 2. The figures were used as a basis for identifying nine research recommendations which are subdivided into three categories. Category A includes three recommendations for the development of criteria for the selection of bridges to retrofit. Category B includes procedures for the evaluation of the resistance of existing bridge components and Category C includes procedures for improving the resistance of existing bridges.

Some of the research recommendations of Working Group 4 significantly overlap recommendations outlined by Working Group 1, Analytical Procedures, and Working Group 3, Experimental Investigations. The overlap signifies the overall importance of the recommended research.

A. CRITERIA FOR THE SELECTION OF BRIDGES TO RETROFIT

1. DEVELOP A METHODOLOGY FOR THE PRELIMINARY SCREENING OF BRIDGES FOR RETROFIT

When dealing with large numbers of bridges, the evaluation of their structural integrity for the purpose of deciding if retrofit is warranted can be a time-consuming and costly task. It is therefore desirable to have a ranking procedure that allows for the classification of bridges based on a screening process which involves an examination of bridge plans, design specifications, bridge site visits, site factors, etc. For example, site factors would be used to assign an importance rating to the bridge in terms of the following parameters: the distance of the bridge from causitive faults, the probable earthquake magnitude, maximum credible site accleration, and soil conditions.

2. DETERMINE APPROPRIATE DESIGN LEVEL FOR RETROFITTING BRIDGES

It will not be economically feasible to increase the strength of existing bridges to the levels required for new bridges by new design specifications. A basis must be established for the selection of some lower level of design for retrofitting. The basis will establish a minimum level below which the bridge would be recommended for replacement rather than retrofit. Currently, standard retrofit measures, such as installation of restraining devices, are utlized by the states. These may not be adequate in regions of potentially strong ground motion.

3. DEVELOP CRITERIA FOR DEFINING THE WORTH OF A BRIDGE TO A ROAD NETWORK

Before a decision is made to retrofit a bridge which is structurally inadequate, it must be decided if the bridge is important to the continuity of societal needs in the immediate post-event period. Retrofit budgets must be allocated to the more critical bridges. A decision procedure is required.

B. PROCEDURES FOR THE EVALUATION OF THE RESISTANCE OF EXISTING BRIDGE COMPONENTS

1. DETERMINE THE ULTIMATE STRENGTH CAPACITY OF TYPICAL LARGE-SIZE COLUMNS IN EXISTING BRIDGES

Most reinforced concrete column research has been conducted on small test specimens. Large bridge-size columns should be tested to determine the horizontal and vertical load-carrying capacities and ductilities. The information is not available for most existing columns, particularly those designed by pre-1971 AASHTO criteria. When available, this information would be used in evaluating existing column design to determine the overall seismic resistance of existing bridges. The type and extent of column retrofitting required for existing bridges would be based on findings from such studies.

2. IDENTIFY THE ULTIMATE STRENGTH CAPACITIES OF TYPICAL EXISTING BRIDGE COLUMN/FOUNDATION CONNECTIONS

Bridge column/foundation connections have proven to be a weak detail in past earthquakes. The ultimate strength and deformation capacity of typical connections for standard columns must be established. Appropriate retrofit measures can then be developed for strengthening this detail. Once the strengths of typical existing connection details are determined, deficiencies including insufficient embedment length and bar anchorage can be corrected.

3. EVALUATE THE EFFECTIVENESS OF COLUMN RETROFITTING TECHNIQUES

Many existing bridge columns are seismically deficient and may need to be retrofitted in order to obtain a desired minimum level of seismic resistance. Several methods of retrofitting columns have been proposed, but none have been used. The effectiveness of these measures is unknown. Analytical and experimental studies are required prior to using retrofit concepts in the field, as the concepts must be shown to be practical and cost effective.

4. DEVELOP MODEL RETROFIT CRITERIA

There is a need for model criteria for retrofit which contains comprehensive guidelines for the evaluation of existing bridge components and establishes strength and displacement design criteria for retrofit. These basic items are not defined at the present. The intent of the model criteria would be to ensure a basic uniformity of practice in retrofit, and it would be useful for all agencies contemplating or undertaking retrofit.

C. PROCEDURES FOR THE IMPROVEMENT OF THE RESISTANCE OF EXISTING BRIDGES

1. DEVELOP IMPROVED RETROFIT CONCEPTS

Typical structural systems for bridges should be investigated and structurally feasible concepts developed for enhancing the seismic resistance of these systems in existing bridges. Retrofit concepts developed would be applicable to typical existing structural systems and all structural materials. The concepts developed should include the use of isolating devices, dampers, and restraining devices and techniques for strengthening existing structural members.

2. DEVELOP RETROFIT CONCEPTS FOR LONG-SPAN BRIDGES

Longspan bridges frequently form critical links in the highway network. Many are old and hence have not been designed to resist strong ground motion. Retrofit details or concepts for long span bridges, where displacements may become excessive, are lacking and require identification. Most major long span bridges should be investigated on a case-by-case basis. However, specific retrofit details, developed from general concepts, are required to control excessive structure displacement.

FIGURE 1

OUTLINE OF THE RETROFIT PROCESS

1. Initial Screening

   a. Identify primary routes
   b. Identify structures required to remain in service
   c. Evaluate alternate available routes
   d. Investigate likelihood of damage due to seismic exposure
   e. Evaluate structural vulnerability based on:
      (1) age
      (2) condition
      (3) code under which designed
      (4) structure type
      (5) details
   f. Make preliminary selection

2. Develop Retrofit Analysis/Design Criteria

   a. Seismic forces
   b. Displacement
   c. Load combinations
   d. Allowable element capacities

3. Perform Structural Investigation of Selected Bridges

   a. Simplified analysis
   b. Computer analysis
      (1) Linear
      (2) Nonlinear
   c. Evaluate analytical results

4. Identify Retrofit Measures

   a. Superstructure
   b. Substructure
   c. Soil condition

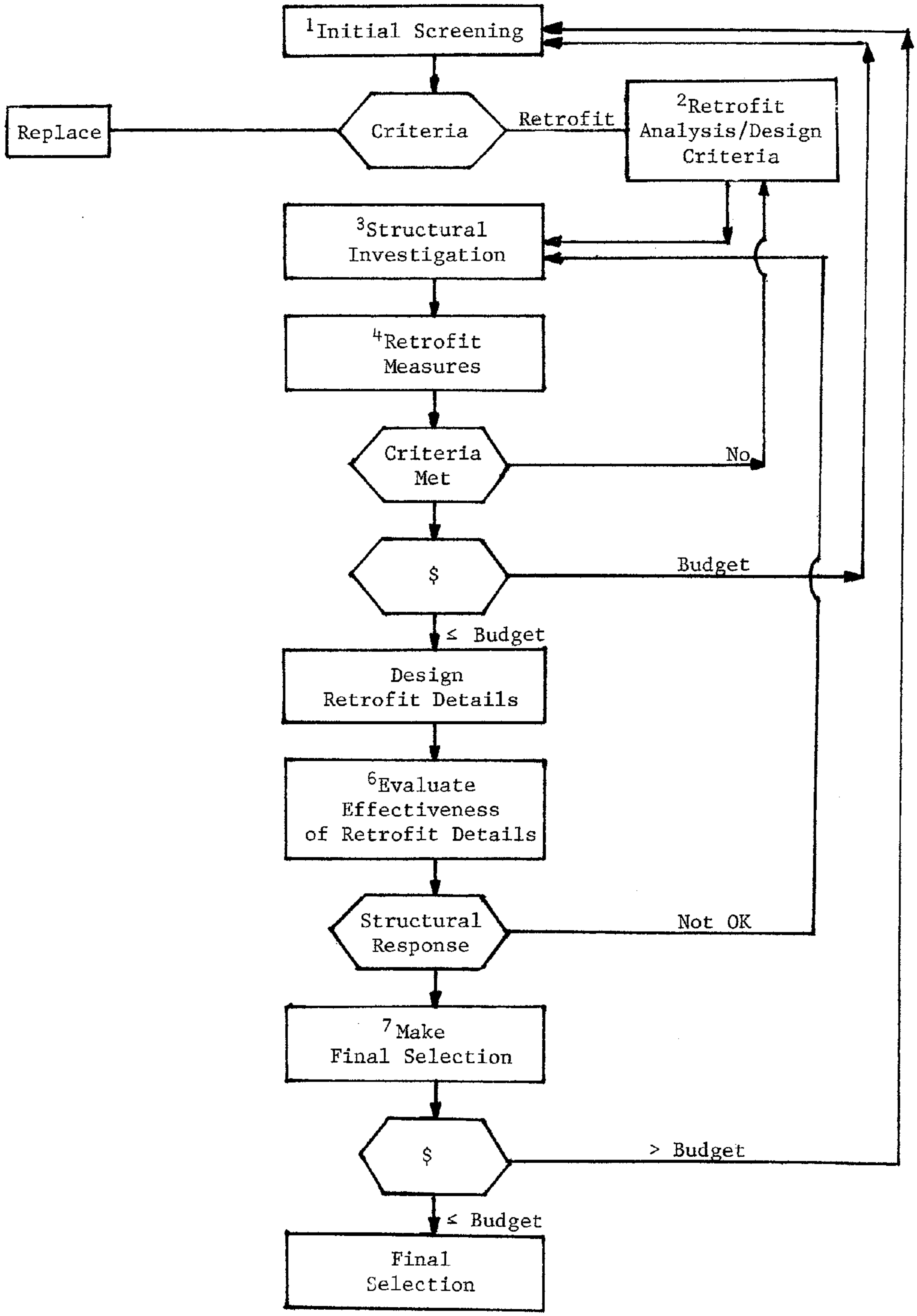
5. Design Retrofit Details

6. Evaluate Effect of Retrofit Details on Overall Seismic Response of Bridges

7. Make Final Selection of Bridges for Retrofit Based on Economic Considerations

FIGURE 2

(Numbers refer to steps in Figure 1)



WORKING GROUP 5

PROFESSIONAL USER NEEDS

Working Group 5 was primarily concerned with the following problems: (1) to assess current knowledge and practice; (2) to evaluate the relevance of present research results to actual user needs; (3) to recommend means for more effective cooperation between researchers and professional users; and (4) to identify and develop strategies for more rapidly disseminating, evaluating, and screening research findings which may be beneficial so that they can be implemented in design practice.

After discussing and developing recommendations for the above issues, the Working Group compiled them in three categories. Included in Category A are recommendations regarding means for improving cooperation between researchers and professional users and for more rapidly disseminating, evaluating, and screening research findings. Category B included areas where guidelines on specific aspects of bridge design should be developed for the professional user. In Category C, the Working Group identified the research that should be performed after a major earthquake.

A. COMMUNICATION AND COOPERATION BETWEEN RESEARCHERS AND PROFESSIONALS

1. ESTABLISH EFFECTIVE MEANS WHEREBY PRACTITIONERS CAN INFORM RESEARCHERS OF PROBLEM AREAS THAT NEED TO BE INVESTIGATED

Short courses or meetings should be organized by universities, professional associations, local engineering groups, or others, to permit and encourage dialogue between practicioners and researchers. Such discussions might be organized around a specific topic of interest or problem area. Periodic surveys by universities or professional engineering groups of questions and suggestions regarding research may also be useful.

2. DETERMINE EFFECTIVE MEANS OF TRANSMITTING RESEARCH RESULTS TO PRACTICING ENGINEERS

There is a great need to collect, distill, and present research results for practical application. Special reports, booklets, journal articles, specialty meetings, and short courses containing practical design examples may serve this purpose. Regular research reports containing practical design information should also be widely disseminated and published in journals.

3. ENCOURAGE THE ORGANIZATION OF INTERNATIONAL EFFORTS FOR THE INTERCHANGE OF DESIGN METHODOLOGIES AND CRITERIA AND RESEARCH FINDINGS BETWEEN COUNTRIES CONCERNED WITH SEISMIC DESIGN OF BRIDGES

It was recognized that distances between research centers have hindered cooperation on a worldwide basis. It is recommended that national and international organizations take steps to improve cooperation in the areas of both research and practical design. The format of a workshop is a good example to follow.

B. <u>DEVELOPMENT OF GUIDELINES AND REPORTS FOR DESIGNERS</u>

1. DEVELOP GUIDELINES OF BOTH RECOMMENDED AND NOT RECOMMENDED CONSTRUCTION DETAILS FOR USE IN BRIDGES LOCATED IN SEISMICALLY ACTIVE AREAS

Such guidelines should be published and distributed to professionals. The adequacy of the guidelines should be evaluated before their distribution by appropriate technical professional committees or others of demonstrated compentence.

2. ESTABLISH TYPICAL CONSTRUCTION DETAILS FOR BRIDGES OF ONE TO THREE SPANS FOR USE IN AREAS OF LOW TO MODERATE SEISMICITY

Adequate seismic resistance can probably be achieved with minimal design effort and construction cost. This would avoid the necessity of design engineers in these areas acquiring the knowledge to perform more detailed structural analysis for seismic loads.

3. PREPARE A SUMMARY OF THE DYNAMIC BEHAVIOR AND CHARACTERISTICS OF BRIDGES OF DIFFERENT TYPES

Many bridge engineers are not familar with the dynamic behavior and characteristics of different bridge types and sites. Such a summary would be most helpful in the preliminary design phase of a bridge in that it would inform bridge engineers of the good and bad characteristics of different bridge types and sites.

4. DEVELOP GUIDELINES AS TO WHAT CONSTITUTES AN ADEQUATE SOILS AND GEOLOGIC FOUNDATION INVESTIGATION FOR THE SEISMIC DESIGN OF BRIDGES

Considerable variations exist in such reports because of the significant advances in research in this area in recent years. Such a report should be prepared by a knowledgeable multidisciplinary committee.

C. POST-EARTHQUAKE RESEARCH

1. CONTINUE POST-EARTHQUAKE INSPECTION AND INTERPRETATION OF DAMAGE CAUSED BY MAJOR EARTHQUAKES THROUGHOUT THE WORLD

Earthquake inspection requirements are broad in scope and dependent on the characteristics of individual earthquakes. They can, however, be divided into three types: (a) field observations immediately after the earthquake, (b) preliminary interpretation of earthquake damage; and (c) comparison of measurements of earthquake response with results of experimental and analytical investigations. Each of these efforts requires different characteristics of personnel and has different funding requirements. For example, the effectiveness of post-earthquake field observations requires that funding and modes of operation and cooperation be pre-established and maintained.

In addition, studies of this nature should examine and report on both damaged and undamaged bridges in the areas of significant ground shaking. Too often undamaged structures are not discussed in reports on major earthquakes.

PRIORITIES OF THE RESEARCH RECOMMENDATIONS

A summary of the specific recommendations developed by each of the five Working Groups was presented at a plenary session of the participants. The recommendations were then edited and circulated to all participants for their comments and priority ratings. A rating of 10 was assigned the highest priority and was defined as an urgent research or development need that should receive immediate attention. A rating of 1 was assigned the lowest priority and was defined as a research or development need that would improve the state of knowledge but does not require immediate or near-term attention.

Twenty-one of the forty-five participants rated the recommendations and the average of these ratings is given in Table 1. Each participant that rated the recommendations was classified in either the design or research area. The average rating for each recommendation is given in Table 1 with respect to the average of all twenty-one responses and to the average of the ten and eleven participants in design and research areas, respectively.

The Steering Committee recommends that the ratings be used with caution in that many of the specific recommendations are interrelated and research should not necessarily be confined to a specific recommendation. In many cases, comprehensive research programs incorporating several of the recommendations, regardless of the individual ratings, is preferable to a series of smaller independent research programs. The ratings are therefore presented as a guide to research priorities and the Steering Committee believes that the range within which a particular rating falls is more important than its specific value.

TABLE 1

AVERAGE RATINGS OF RESEARCH RECOMMENDATIONS

Working Group	Recommendation	Avg. Rating of All Participants	Avg. Rating of Designers	Avg. Rating of Researchers
1	A1	6.5	6.1	6.9
	A2	6.2	5.5	6.9
	A3	5.7	5.1	6.4
	A4	4.8	5.1	4.5
	A5	4.7	4.5	4.9
	B1	7.8	8.4	7.3
	B2	6.2	6.7	5.8
	B3	3.6	3.5	3.8
	C1	7.1	5.8	8.4
	C2	6.6	7.3	6.0
	C3	6.1	6.6	5.6
	C4	5.9	5.2	6.6
	C5	5.5	4.7	6.6
	C6	5.4	4.6	6.2
	C7	4.3	4.2	4.4
2	A1	6.2	6.7	5.8
	A2	6.3	6.7	5.9
	A3	5.2	4.8	5.6
	A4	5.5	4.6	6.5
	B1	7.2	7.0	7.3
	B2	6.7	6.0	7.3
	B3	6.3	7.0	5.6
	B4	6.2	6.7	5.8
	B5	4.7	4.3	5.1
	C1	7.1	7.4	6.9
	C2	5.7	6.2	5.2
	D1	5.8	5.3	6.3
3	A1	8.2	8.5	8.0
	A2	7.4	6.6	8.1
	A3	5.6	5.2	5.9
	A4a	7.5	7.7	7.3
	A4b	6.8	5.7	7.8
	A4c	6.3	6.6	6.1
	A4d	6.3	5.6	6.9
	A4e	5.9	4.9	6.8
	B1	6.5	7.2	5.8
	B2	6.2	5.7	6.6
	C1	8.4	7.9	8.8

TABLE 1 (concluded)

Working Group	Recommendations	Avg. Rating of All Participants	Avg. Rating of Designers	Avg. Rating of Researchers
4	A1	6.5	5.0	7.9
	A2	6.3	6.1	6.5
	A3	5.0	3.7	6.1
	B1	7.6	8.0	7.3
	B2	7.2	7.4	7.0
	B3	7.2	7.4	7.0
	B4	5.0	3.8	6.2
	C1	7.0	6.8	7.1
	C2	5.3	4.8	5.9
5	A1	7.6	7.7	7.6
	A2	7.3	7.4	7.3
	A3	7.2	6.7	7.6
	B1	8.0	8.2	7.8
	B2	7.1	7.5	6.8
	B3	7.1	6.8	7.3
	B4	6.7	6.5	6.8
	C1	9.2	9.1	9.2

AN OVERVIEW OF THE STATE OF PRACTICES IN EARTHQUAKE RESISTANT DESIGN OF HIGHWAY BRIDGES IN JAPAN

by

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INTRODUCTION

Since the Kanto Earthquake of 1923 Japan has been experiencing a number of severe earthquakes, and the incidence of damages to highway bridges has been considerable. With consideration of the damages caused by the Kanto Earthquake seismic forces have been first taken into account in the design of highway bridges in 1926. The seismic coefficient method was developed and introduced in the practical design of structures at that time. After experiencing severe damages during consecutive strong earthquakes, seismic regulations were reviewed and amended several times. With view of damages caused by the Niigata Earthquake of 1964 the current specifications for earthquake-resistant design of highway bridges were issued in 1971. Furthermore, extensive efforts are now undertaken to establish more rational seismic criteria for highway bridges, with consideration of recent advancements of earthquake engineering associated with bridges and also the damage experience due to the Miyagi-Ken-Oki Earthquake of 1978.

This paper briefly describes a history of seismic design codes of highway bridges in Japan, and introduces the current specifications (1971) and also the new specifications (draft, 1979) for seismic design of highway bridges.

HISTORY OF EARTHQUAKE RESISTANT DESIGN PROVISIONS FOR HIGHWAY BRIDGES IN JAPAN

The Ministry of Home Affairs stipulated in 1926 "Specifications for Design of Road," which are parts of "Road Laws." In the specifications seismic forces were first taken into account in the design of highway bridges, in consideration that several highway bridges sustained substantial damage during the Kanto Earthquake of 1923. The specifications provided that highway bridges shall be designed in accordance with the seismic coefficient method, in which horizontal seismic coefficients were taken from 0.1 to 0.4. The values of the coefficients were dependent on areas and ground conditions.

For bridges to be constructed in Tokyo and Yokohama, seismic coefficients of 0.3 or more were recommended. This seems due to the substantial damage to bridge structures in the areas during the Kanto Earthquake.

The Ministry of Home Affairs issued in 1939 "Specifications for Design of Steel Highways," which took place of the former ones in the design of highway bridges. The specifications stipulated that seismic forces shall be taken into account in accordance with the seismic coefficient method, considering a horizontal coefficient of 0.2 and a vertical coefficient of 0.1, simultaneously.

The specifications were revised in 1964 by the Japan Road Association, with a commission from the Ministry of Construction. The revised specifications specified that horizontal coefficient of 0.1 to 0.35 depending on areas and ground conditions and vertical coefficient of 0.1 shall be considered in the aseismic design, simultaneously.

In view of technological advances in bridge engineering, earthquake engineering and other scientific fields, the Japan Road Association, also with a commission from the Ministry of Construction, drew up in January 1971, comprehensive specifications exclusively for earthquake-resistant design of highway bridges. In the current specifications two methods are provided for the aseismic design. One is the conventional seismic coefficient method for rigid structures, where the horizontal coefficient ranges between 0.1 and 0.24 depending on areas, ground conditions, and importance. The other is the modified seismic coefficient considering structural response for comparatively flexible structures, where the horizontal seismic coefficient vary from 0.05 to 0.3 depending on the fundamental natural periods in addition to the above three factors.

On the other hand, characteristic criteria have been proposed, between 1966 and 1967, tentatively for the aseismic design of highway bridges relating to specific projects administrated by the Japan Highway Public Corporation (JHPC), the Metropolitan Expressway Public Corporation (MEPC), the Hanshin Expressway Public Corporation (HEPC) and the Honsyu Shikoku Bridge Authority (HSBA). Moreover, the Japanese National Railways (JNR) stipulated in 1968 its own criteria for aseismic design of railway bridges.

Table 1 tabulates briefly the above mentioned history of design loads (primarily seismic loads) for highway bridges in Japan[7].

Table 1 History of Design Loads for Highway Bridges in Japan (Primarily Seismic Loads)

Year	Name of Regulations	Design Live Loads: Class	Design Live Loads: Truck Roller Streetcar Loads	Design Live Loads: Uniform Loads	Design Live Loads: Line Loads	Impact Loads	Seismic Loads k: Horizontal Seismic Coefficient	Other Seismic Regulations (k: seismic coef.)	Major Earthquakes
1) 1886	Order No.13 Ministry of Home Affairs (MHA)			U-454 kg/m	-	not considered	not considered		(M>7) after Rika Nenpyo(1971)
2) 1906	Order MHA		R-13.6 t S-12.7	U-400 600 kg/m (carriage way) U-270 400 kg/m (footway)		not considered	not considered		
3) 1919	Road Laws MHA		R-13.6 t T-11.2 t S-30 t	Same as 2)	-	not considered	not considered		
4) 1926	Specifications for Design of Roads, Road Laws, MHA	1st	T-12 t	U-600kg/m		considered	Seismic Coefficient Method k=0.15∿0.4 depending on location and ground condition (k≥9.3 advised in Tokyo, Yokohama)	1924.Building Law (k≥0.1) 1950. Kawasumi's Seismic Map.	1923.Kanto (M=7.9)
		2nd	T-8 t	U-500 kg/m	-				
		3rd	T-6 t	U-500 kg/m					
5) 1939	Specifications for Design of Steel Highway Bridges, MHA	1st	T-13t	U-500 kg/m		considered	Seismic Coefficient Method k_h=0.2, k_v=0.1	1950. New Building Law (k=0.2, H≤16m increase 0.01 per an increase of 4m above 16m)	1946.Nankai (M=8.1) 1948.Fukui (M=7.3) 1952.Tokachi-oki (M=8.2)
		2nd	T- 9t	U-400 kg/m					
6) 1956 (and 1964)	Revision of Specifications for Design of Steel HIGHWAY Bridges, JRA	1st	T-20 t	-	L-20(5t/m)	considered	Seismic Coefficient Method k=0.1∿0.35 depending on location and ground conditions		
		2nd	T-14 t	-	L-14 (3.5t/m)				
7) 1964 to 1971	Specifications for Design of Substructures of Highway Bridges,JRA	same as 6)					same k as 6) Detailed calculation methods	1964. High-Rise Building. (H≥45m) k=(0.18∿0.36) /T(T:period) (0.05≤k≤0.2)	1964.Niigata (M=7.5)
8) 1966* (and 1970)	Design Standards, JHPC	same as 6)					Design Code for High-Rise Bridges, (Increase in k with the height of the piers)		
9) 1967*	Specifications for Earthquake Resistant Design of Honshu-Shikoku Bridges,JSCE	Special Live Loads and Impact Loads for Suspension Bridges are Considered					Modified Seismic Coefficient Method (Basic Coef. k=0.2)	1965. Report of JSCE on Civil ERD of Engineering Structures.	
10) 1967*	Specifications for Earthquake-Resistant Design of Metropolitan Expressway Public Corporation, MEPC	same as 6)					Seismic Coef. Method k=0.2∿0.3	1968. Report of JSCE on Earthquake-Resistant Design of Substructures of Railway Bridges.	1968.Tokachi-oki (M=7.9)
11) 1968*	Aseismic Design Standards, HEPC	same as 6)					Seismic Coef. Method k=0.2∿0.28		
12) 1971	Specifications for Earthquake Resistant Design of Highway Bridges, JRA	same as 6)					Seismic Coef. Method k=0.1∿0.24 (Rigid) Modified SCM. k=0.05∿0.3 (Flexible)	1977. A Proposal for Earthquake Resistant Design Methods (MOC)	
13) 1979	Part V Earthquake Resistant Design, Specifications for Highway Bridges (New Specifications - DRAFT), JRA	same as 6)					Seismic Coef. Method k=0.1∿0.24 (Rigid) Modified SCM. k=0.05∿0.3 (Flexible) Earthquake Response Analyses (Very Flexible Bridges)		1978. Miyagi-ken-Oki (M=7.4)

*) Specifications from 8) to 11) are for highway bridges related to special large projects.

CURRENT SPECIFICATIONS (JRA - 1971)

Current Specifications for Earthquake-Resistant Design of Highway Bridges[2] were issued in January 1971, by the Japan Road Association, which apply to the design of highway bridges with spans not longer than 200 meters, to be constructed on expressways, national highways, prefectural highways and important municipal highways.

The specifications basically stipulate to employ seismic coefficient methods and provide two methods in determining design seismic coefficients. One is the conventional seismic coefficient method that applies to the design of relatively rigid structures. The other is the modified seismic coefficient method considering structural response that applies to the design of relatively flexible structures. The followings are the principal points of the specifications.

(1) The horizontal design seismic coefficient for a rigid structure is determined systematically, depending on the geographical location of the bridge site, the ground conditions at each substructure site, and the importance of the bridge. The horizontal design coefficient for a flexible structure is determined depending on the fundamental natural period of each structural system.

(a) In the seismic coefficient method that is employed for relatively rigid structures, the horizontal design seismic coefficient (k_h) shall be determined by

$$k_h = \nu_1 \nu_2 \nu_3 k_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where

k_h: horizontal design seismic coefficient,
k_0: standard horizontal design seismic coefficient (=0.2),
ν_1: seismic zone factor,
ν_2: ground condition factor,
ν_3: importance factor.

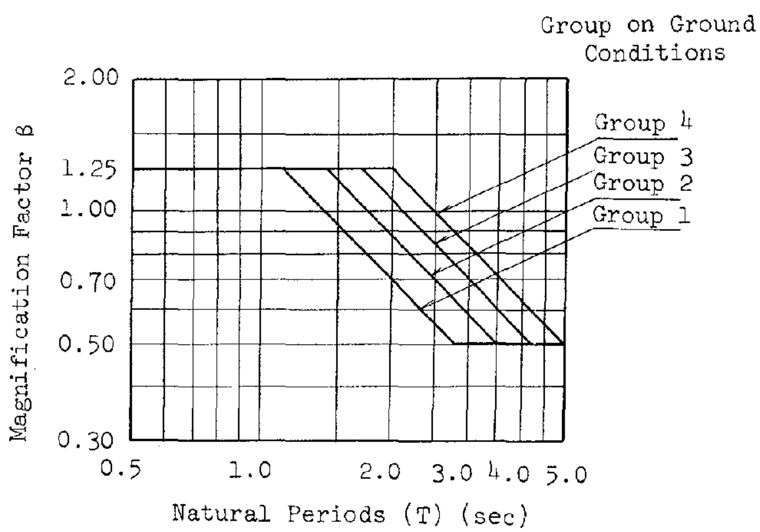


Fig. 2 Magnification factor (β) for general highway bridges

Table 2 Seismic Zone Factor ν_1 for General Highway Bridges

Zone[1]	Value of ν_1
A	1.00
B	0.85
C	0.70

Note: 1) Zones A, B and C are illustrated in Fig. 1.

The values of ν_1, ν_2, and ν_3 are shown in Tables 2, 3 and 4, respectively. The definitions of classification are specified in the provisions. The minimum value of k_h shall be considered as 0.10.

(b) In the modified seismic coefficient method considering structural response that is employed for relatively flexible structures such as a bridge with highrise piers higher than 25 m or a bridge with a fundamental period longer than 0.5 seconds, the horizontal design seismic coefficient (k_{hm}) shall be determined by

$$k_{hm} = \beta k_h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Table 3 Ground Condition Factor ν_2 for General Highway Bridges

Group	Definitions[1]	Value of ν_2
1	(1) Ground of the Tertiary era or older (defined as bedrock hereafter) (2) Diluvial layer[2] with depth less than 10 meters above bedrock	0.9
2	(1) Diluvial layer[2] with depth greater than 10 meters above bedrock (2) Alluvial layer[3] with depth less than 10 meters above bedrock	1.0
3	Alluvial layer[3] with depth less than 25 meters, which has soft layer[4] with depth less than 5 meters	1.1
4	Other than the above	1.2

(Notes) 1) Since these definitions are not very comprehensive, the classification of ground conditions shall be made with adequate consideration of the bridge site.
Depth of layer indicated here shall be measured from the actual ground surface.

2) Diluvial layer implies a dense alluvial layer such as a dense sandy layer, gravel layer, or cobble layer.

3) Alluvial layer implies a new sedimentary layer made by a landslide.

4) Soft layer is defined in Section 3.7 "Soil Layer Whose Bearing Capacities are Neglected in Earthquake Resistant Design."

where

k_{hm}: horizontal design seismic coefficient in the modified seismic coefficient method considering structural response,

k_h : coefficient given by eq. (1),

β : a factor depending on the fundamental period of the bridge, and obtained by Fig. 2.

For structures whose fundamental periods are shorter than 0.5 seconds, β may be considered as 1.0.

The minimum value of k_{hm} shall be 0.05.

Table 4 Importance Factor ν_3 for General Highway Bridges

Group	Definitions	Value of ν_3
1	Bridges on expressway (limited-access highways), general national highways and principal prefectural highways. Important Bridges on general prefectural highways and municipal highways.	1.0
2	Other than the above	0.8

Note: The value of ν_3 may be increased up to 1.25 for special cases in Group 1.

(2) The vertical design seismic coefficient may generally be considered as zero, except for special portions such as bearing supports.

(3) The horizontal design seismic coefficient for structural parts, soils and water below the ground surface may be considered as zero.

(4) Hydrodynamic pressure during earthquakes are specified in the specifications. Earth pressures during earthquakes, however, are specified in the related specifications.

(5) A special attention is paid to very soft soil layers and soil layers vulnerable to liquefaction during earthquakes. The bearing capacities of these layers are neglected in the design, in order to assure high earthquake-resistance for structures that are built in these layers.

(6) A special attention is also paid to the design of structural details, in consideration of the damage previously experienced to bridge structures. To this aim provisions are specified for bearing supports and devices for preventing bridge girders from falling.

(7) Increases in allowable stresses of materials may be considered in the earthquake-resistant design, magnitudes of increases for various materials are specified in the several related specifications. The increasing rates are as follows:

concrete in reinforced concrete structures:	50%
reinforcements in reinforced concrete structures:	50%
structural steel for superstructures:	70%

structural steel for substructures:	50%
concrete in prestressed concrete structures subjected to compressive forces:	65%
foundation soils:	50%

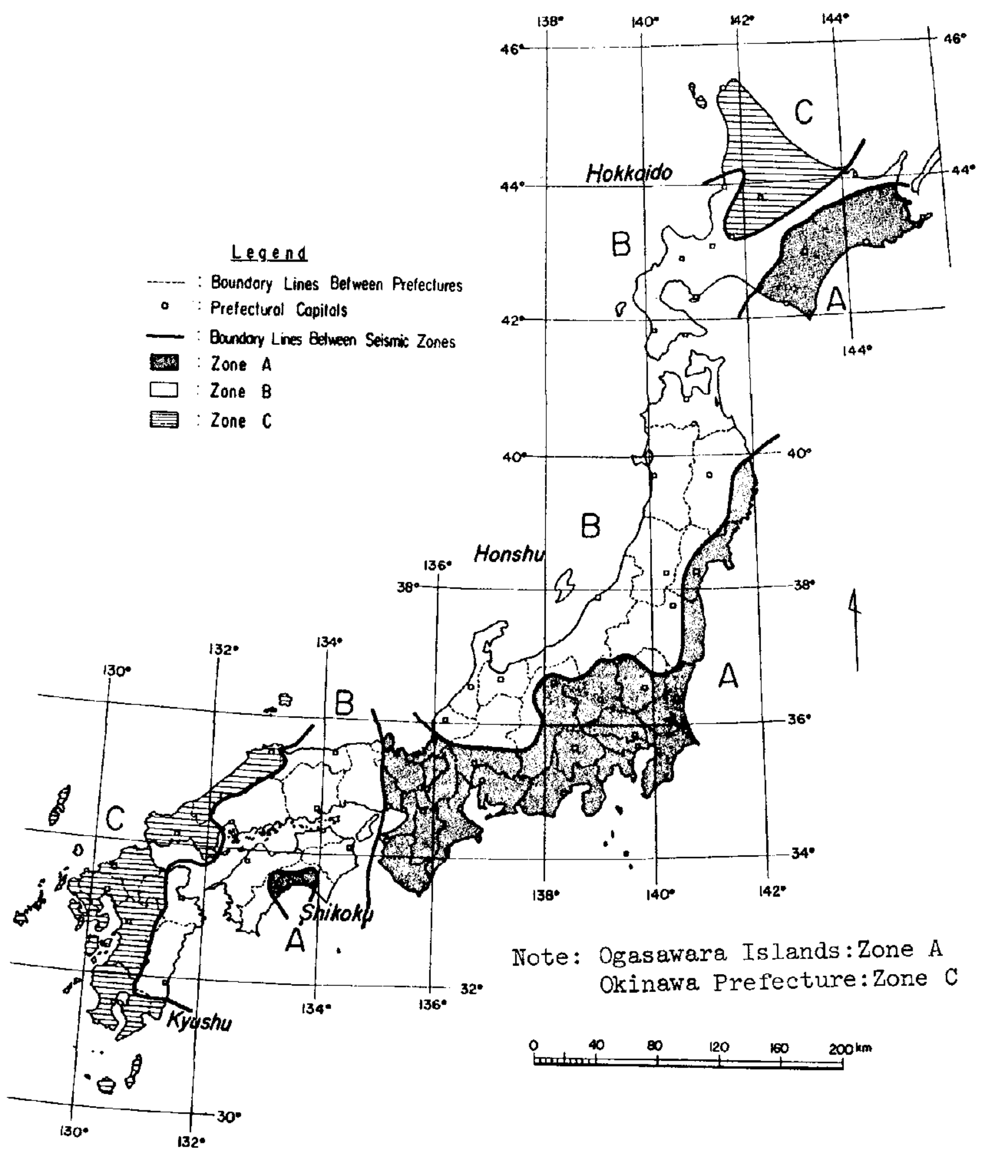


Fig. 1. Current Seismic Zoning Map

NEW SPECIFICATIONS (JRA DRAFT - 1979)

The design of general highway bridges in Japan is done according to "Specification for Highway Bridges" issued by the Japan Road Association. The Specifications consist of five parts, and up to the present Part I General Specifications [1972], Part II Steel Bridges [1972] and Part III Concrete Bridges [1977] were already issued. Preparations of Part IV Substructures and Part V Earthquake Resistant Design (New Specifications) are now undertaken for future release.

Currently the effects of earthquakes in designing highway bridges are stipulated very briefly in one of sections of General Specifications, Part I of the "Specifications for Highway Bridges." In this section it is specified that the effects of earthquakes shall be referred to "Specifications for Earthquake Resistant Design of Highway Bridges," which was issued by the Japan Road Association in 1971 (Current Specifications described in the previous section in this report). After the enforcement of the Current Specifications, extensive investigations and researches have been undertaken in earthquake engineering fields. One of the recent major advantages is the fruit of efforts devoted in a comprehensive research project done in the Aseismic Technology Development Committee which was established at the Technology Center for National Land Development from 1972 through 1976, with a commission from the Public Works Research Institute and the Building Research Institute, Ministry of Construction. The results of investigations achieved in this project were put into a unified form of provisions on earthquake resistant design for civil engineering structures and buildings, namely "A Proposal for Earthquake Resistant Design Methods," which was issued by the Ministry of Construction in March, 1977[3]. The Proposal was formulated with special considerations on the following principles:

(1) Standardization of Fundamental Design Criteria

Since earthquake resistant design methods currently applied to various structures have their own backgrounds and have so many variations according to types of structures, design seismic forces are unbalanced among various type of structures. Consequently in the project, existing earthquake resistant design methods were thoroughly reviewed and the fundamental criteria for designing various structures such as bridges, soil structures, underground structures and buildings were proposed.

(2) Clarification of Design Procedure

Current earthquake resistant design has an inclination that they force designers to devote so complex calculations that the designers sometimes forget fundamental principles of earthquake resistant design. Therefore, the Proposal intends to make clear the fundamental design procedures.

(3) Development of Rational Design Procedures

Earthquake resistant design is required to be achieved by the proper procedures in accordance with subsoil and structural conditions. The Proposal also intends to develop a variety of rational design procedures which can be adequately selected according to the structural types.

Efforts for revising the Current Specifications with the form of Partart V of "Specifications for Highway Bridges" are now undertaken, and a draft of the New Specifications (1979) was tentatively formulated in the fall of 1978. The contents of the New Specifications (Draft) are presented in Appendix in this paper.

The principal features and improvements of design methodology in the New Specifications are described in the following.

Seismic Zoning Map

The newly developed seismic zoning map as illustrated in Fig. 3 was adopted replacing the current seismic zoning map shown in Fig. 1. The newly introduced seismic zoning map is based on the Proposal for Earthquake Resistant Design Methods with aim of unifying seismic zoning maps currently applied for civil engineering structures and buildings, and slight modifications were introduced on the proposed original map from the viewpoint of administrative considerations.

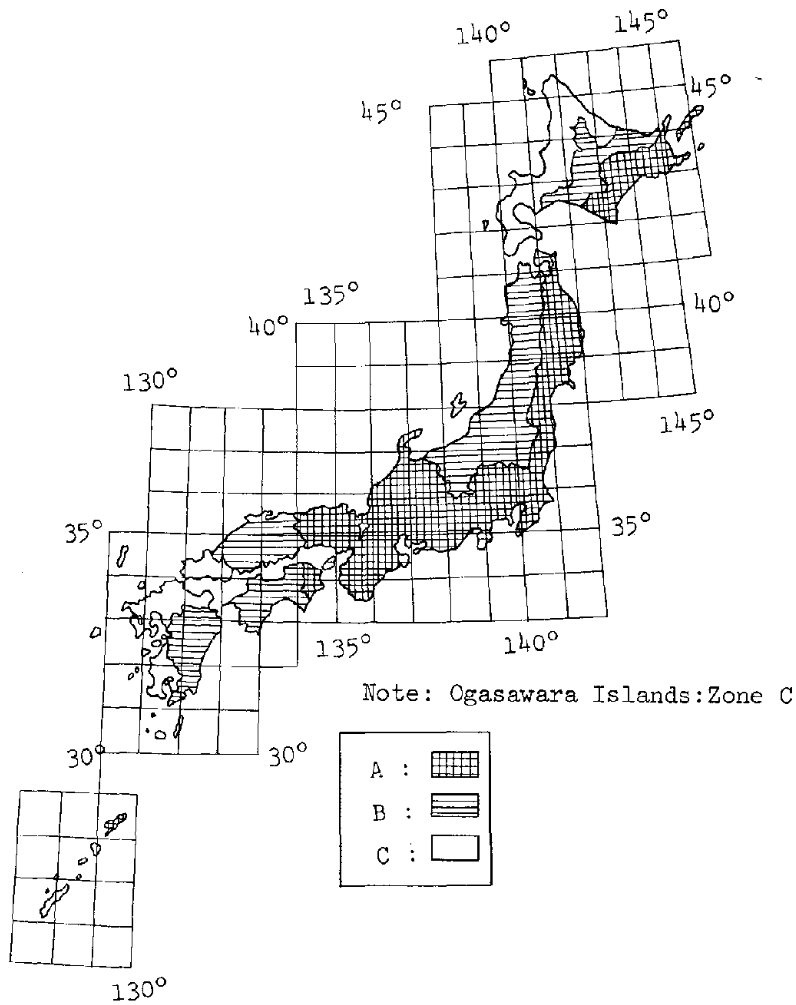


Fig. 3 Seismic Zoning Map (New Specifications)

Classification of Ground Conditions

In the Current Specifications, classification of ground conditions are determined in accordance with geological conditions as tabulated in Table 3. However, since subsurface ground responses during earthquakes would generally be more largely affected by the predominant period of the ground, it is considered more reasonable to classify grounds into some groups in terms of the period of the ground. Consequently in the New Specifications, the ground conditions will be classified into four groups according to Table 5, in which characteristic value of ground T_g is stipulated to be principally calculated by the following equation:

$$T_g = \sum_i \frac{4H_i}{V_{si}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where

T_g : Characteristic value of ground (second)
H_i : Thickness of i-th subsoil layer (m)
V_{si}: Shear wave velocity of i-th subsoil layer at low strain (around 10^{-4}%)

Table 5 Classification of Ground Conditions

Group	Characteristic Value T_g (second)
1	$T_g < 0.2$
2	$0.2 \leqq T_g < 0.4$
3	$0.4 \leqq T_g < 0.6$
4	$0.6 \leqq T_g$

As for the shear wave velocities, it is recommended that it is directly measured through site investigations. The baserock for calculation of Eq. (3) is stipulated to take on the soil layer that has a shear wave velocity at low strain equal to 280 m/sec or higher and is not underlaid by materials having significantly lower shear wave velocities.

The characteristic value T_g implies a natural period of subsurface ground at low strain level. The classification of T_g shown in Table 5 was proposed based on numerical seismic analyses of many types of subsurface grounds. Such analyses revealed that the natural period T_s of subsurface ground at high strain level which would be expected to occur during strong earthquakes can be approximately obtained by the following equation.

$$T_s = 1.25\ T_g \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

It was also found that the ground conditions could be adequately classified into four groups by taking T_s as $T_s < 0.25$ seconds, $0.25 \leq T_s < 0.5$ seconds, $0.5 \leq T_s < 0.75$ seconds and $T_s \geq 0.75$ seconds. The characteristic values T_g presented in Table 5 were thus obtained by substituting the above mentioned T_s into Eq. (4).

Fig. 4 is one of representative results of analyses showing a relationship between the characteristic values T_g and thicknesses of soil deposits. It is apparent from the result that the classification of ground conditions determined by the characteristic value T_g as shown in Table 5 can also be approximately estimated by thicknesses of alluvial and diluvial layers. It is therefore recommended to use this relation to classify the ground condition when T_g cannot be obtained.

Fig. 5 shows a comparison of the ground classifications which are provided in

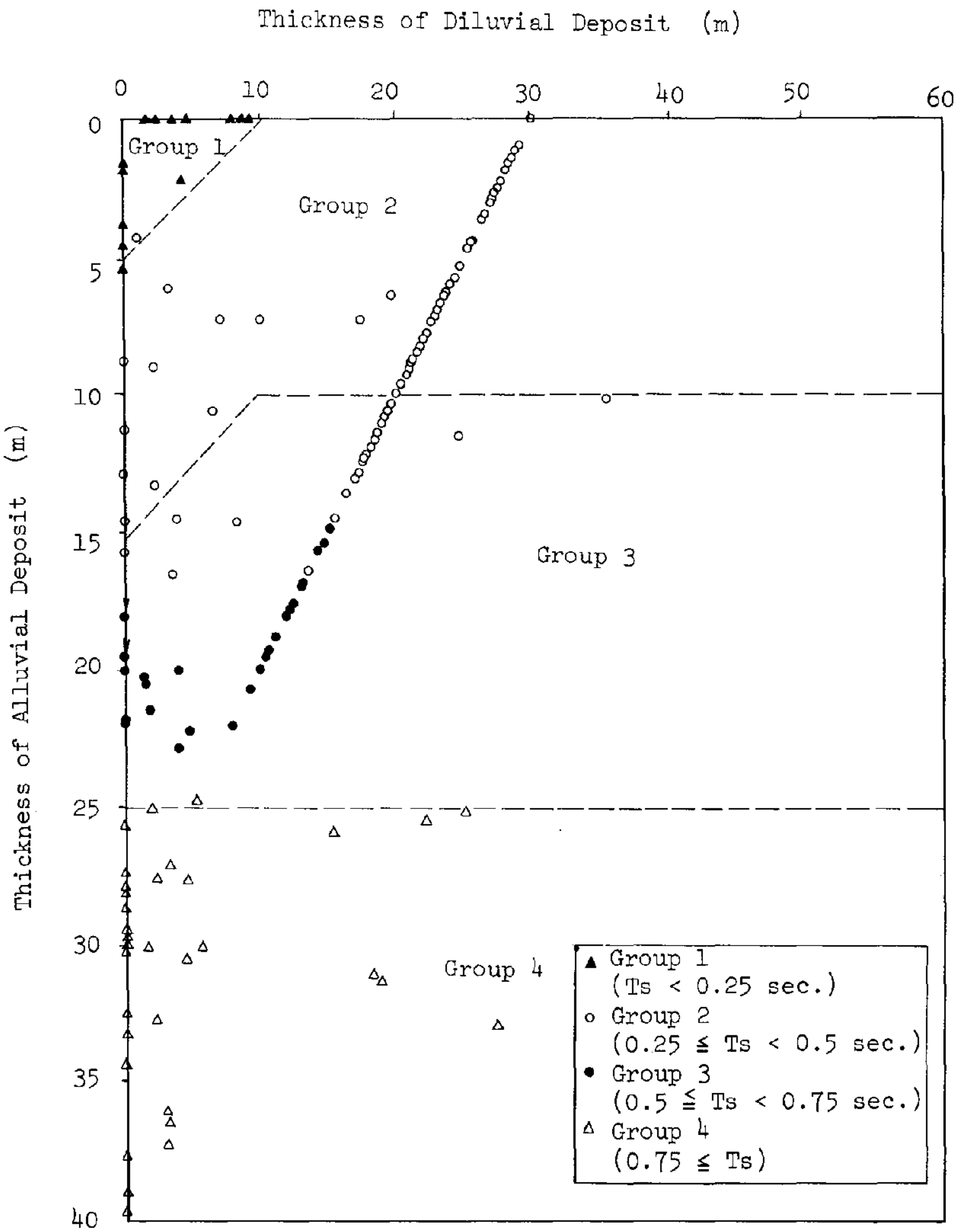


Fig. 4 Classification of Ground Conditions in Terms of Thicknesses of Alluvial and Diluvial Soil Layers

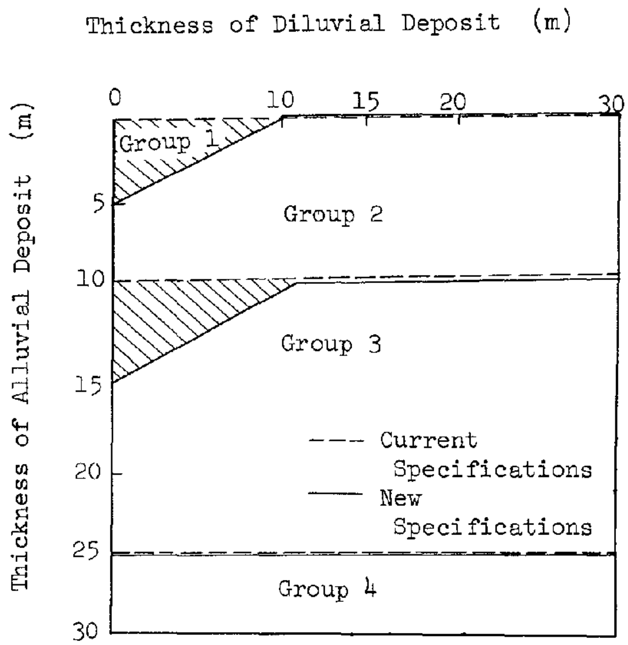


Fig. 5 Comparison of Ground Conditions Provided in Current and New Specifications

the Current Specifications and in the New Specifications. It is understood from the results that some parts of grounds which are evaluated as Groups 1 and 2 in the Current Specifications turn into Groups 2 and 3, respectively, in the New Specifications.

Liquefaction of Sandy Soil Layers

In the Current Specifications, it is stipulated that saturated sandy soil layers which are within 10 meters of the actual ground surface, have a standard penetration test N-value less than 10, have a coefficient of uniformity less than 6, and also have a D_{20}-value on the grain size accumulation curve between 0.04 mm and 0.5 mm, shall have a high potential for liquefaction during earthquakes, Bearing capacities of these layers shall be neglected in design.

After the Niigata Earthquake, comprehensive studies have been conducted to assess vulnerability of saturated sandy soils. Based on these studies, the provisions for liquefaction are improved in the New Specifications as follows:

(1) Sandy Soil Layers Needed to be Checked for Liquefaction - Saturated sandy soil layers which exist under water table and do not coincident with any of the following conditions are vulnerable to liquefaction, and liquefaction potential of these layers shall be estimated according to item (2).

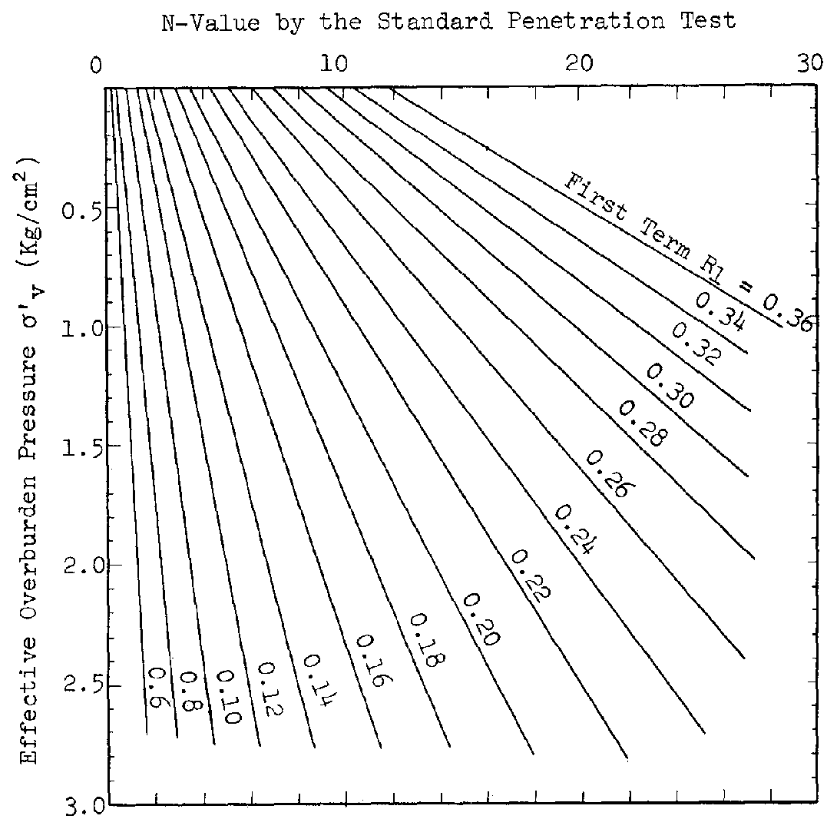


Fig. 6 R_1-Value (First Term of Resistance R)

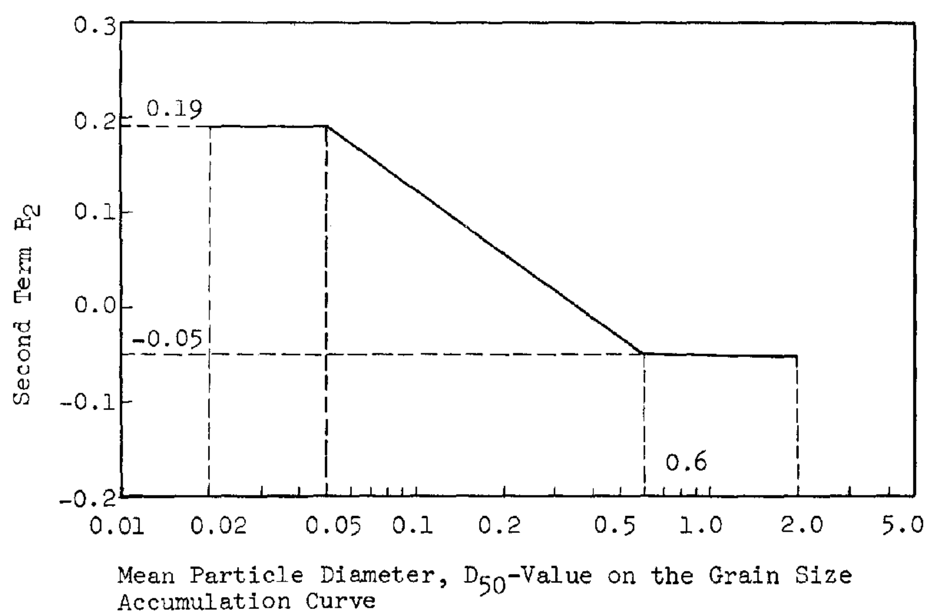


Fig. 7 R_2-Value (Second Term of Resistance R)

1) Soil layers which exist 20 meters below actual ground surface or deeper.

2) All the soil layers in the case where the water table exists deeper than 10 meters below actual ground surface.

3) Soil layers which have a D_{50}-value on the grain size accumulation curve either smaller than 0.02 mm or larger than 2 mm.

4) Soil layers formed in the diluvial era or older.

(2) Estimation of Liquefaction - For those soil layers which are judged to be vulnerable for liquefaction, liquefaction potential shall be checked based on liquefaction resistance factor F_L defined by the following equation.

$$F_L = \frac{R}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

where

F_L: liquefaction resistance factor

R : resistance of soil elements to dynamic loads, and

$$R = R_1 + R_2 \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

R_1 and R_2 shall be determined in accordance with Figs. 6 and 7, respectively.

L : dynamic loads to soil elements induced by earthquake motion

$$L = r_d k_s \frac{\sigma_v}{\sigma_v'} \quad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

$$r_d = 1.0 - 0.015z \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

z: depth from the actual ground surface (m)

k_s: seismic coefficient for evaluation of liquefaction, and shall be determined by the following equation:

$$k_s = \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot k_{so} \quad \ldots\ldots\ldots \quad (9)$$

ν_1, ν_2, ν_3: seismic zone factor, ground condition factor and importance factor, provided in Tables 2, 3 and 4, respectively.

$k_{so} = 0.15$

σ_v : total overburden pressure

σ_v': effective overburden pressure at the static condition.

Soil layers having the liquefaction resistance factor F_L smaller than 1.0 shall be judged to liquefy during earthquakes. Figs. 6 and 7 are graphic illustrations of R_1 and R_2 represented in the following equations which were proposed based upon the results of laboratory dynamic triaxial tests on soil specimens taken from several sites in Japan[10].

$$R = \begin{cases} 0.0882\sqrt{\dfrac{N}{\sigma_v' + 0.7}} + 0.19 & (0.02\text{mm} \leqq D_{50} \leqq 0.05\text{mm}) \\ 0.0882\sqrt{\dfrac{N}{\sigma_v' + 0.7}} + 0.225 \log_{10}(\dfrac{0.35}{D_{50}}) & (0.05\text{mm} < D_{50} \leqq 0.6\text{mm}) \\ 0.0882\sqrt{\dfrac{N}{\sigma_v' + 0.7}} - 0.05 & (0.6\text{mm} < D_{50} \leqq 2.0\text{mm}) \end{cases} \quad (10)$$

(3) Treatment of Soil Layers which were Judged to Liquefy – For those soil layers which were judged to liquefy by the above estimation and are within 10 meters of the actual ground surface, bearing capacities and other soil constants shall be either neglected or reduced in the seismic design, by multiplying the original bearing capacities by reduction factors D which are determined in accordance with F_L-values and tabulated in Table 6.

Table 6 Reduction Factor of Bearing Capacities of Soil Layers

F_L	Reduction Factor D	Remarks
$1.0 < F_L$	1.0	Not Reduced
$0.8 < F_L \leqq 1.0$	2/3	Reduced
$0.6 < F_L \leqq 0.8$	1/3	
$F_L \leqq 0.6$	0	Neglected

Modified Seismic Coefficient Method

In the Current Specifications, the modified seismic coefficient method is provided to apply to bridges which have flexible piers and long fundamental periods (longer than 0.5 seconds), such as those with piers taller than 25 meters above the ground surface. Accounting for seismic responses, magnification factors (β) for the modified seismic coefficient method are stipulated and displayed in Fig. 2. However, it has been pointed out that fundamental natural periods sometimes exceed 0.5 seconds even for those bridges with pier lower than 25 meters above the ground surface. Basing on experimental data on the relationship between fundamental natural periods and pier heights, it is modified in the New Specifications that the modified seismic coefficient method shall apply to bridges which have flexible piers and long fundamental periods, such as those with piers higher than 15 meters above the ground surface.

In addition to the above change, the following two modifications were also introduced:

1) The magnification factors (β) are modified as shown in Table 7 and Fig. 8 so as to avoid a sudden change of β-value at a period of 0.5 seconds.

Table 7 Magnification Factor β for Modified Seismic Coefficient Method

G. C.	β-value			
Group 1	$\beta = 2T$ $0.5 \leq T \leq 0.625$	$\beta = 1.25$ $0.625 \leq T \leq 1.1$	$\beta = 1.40/T$ $1.1 \leq T \leq 2.8$	$\beta = 0.50$ $T \geq 2.8$
Group 2	$\beta = 2T$ $0.5 \leq T \leq 0.625$	$\beta = 1.25$ $0.625 \leq T \leq 1.4$	$\beta = 1.75/T$ $1.4 \leq T \leq 3.5$	$\beta = 0.50$ $T \geq 3.5$
Group 3	$\beta = 2T$ $0.5 \leq T \leq 0.625$	$\beta = 1.25$ $0.625 \leq T \leq 1.7$	$\beta = 2.10/T$ $1.7 \leq T \leq 4.2$	$\beta = 0.50$ $T \geq 4.2$
Group 4	$\beta = 2T$ $0.5 \leq T \leq 0.625$	$\beta = 1.25$ $0.625 \leq T \leq 2.0$	$\beta = 2.50/T$ $2.0 \leq T \leq 5.0$	$\beta = 0.50$ $T \geq 5.0$

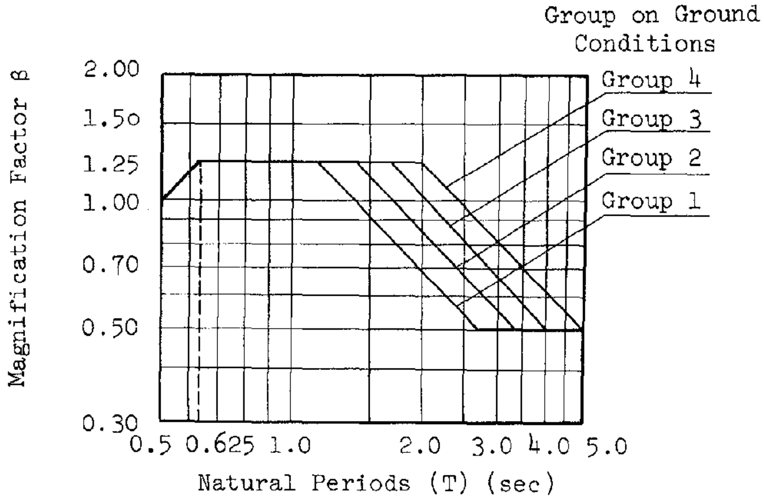


Fig. 8 Magnification Factor for the Modified Seismic Coefficient Method (New Specifications)

2) In the current Specifications, effects of subsoil condition are not precisely considered in estimating fundamental natural periods. Since the effects of subsoils would be predominant in calculating fundamental natural periods, especially for bridges with short piers, it is stipulated in the New Specifications that the effects of subsoils shall be taken into account for those bridges which are constructed into the soft grounds. It is recommended to estimate the fundamental natural period for the individual system consisting of each substructure and the part of superstructures supported by it by the following equation.

$$T = 2\sqrt{\delta} \quad \text{.......................} \quad (11)$$

where

T: Fundamental natural period in seconds of the system consisting of a substructure and the section of the superstructures supported by it.

δ: Maximum horizontal displacement (in meter) of the pier when subjected to the dead weight of the section of superstructure supported by the substructure and also to 80 percent of the dead weight of the substructure above ground surface assumed in earthquake resistant design.

SEISMIC MOTIONS IN THE EARTHQUAKE RESPONSE ANALYSES

In the Current Specifications, it is stipulated that dynamic earthquake response analyses shall be adopted for those bridges for which detailed investigations are required. In the New Specifications, an article is newly introduced concerning seismic motions to be utilized in the dynamic response analyses. The principal aspects of the provisions are as follows:

(1) The earthquake response analyses apply to those bridges which are designed either by the seismic coefficient method or the modified seismic coefficient method, in order to investigate precisely the earthquake resistivity of bridges in terms of ductilities and maximum bearing capabilities. The earthquake response analyses are needed for those bridges having structural systems which are significantly different from those assumed in the seismic coefficient method or the modified seismic coefficient method, those bridges having new structural types for which the experiences of damages accumulated through the past earthquakes cannot be adequately extended, those bridges which are constructed on extremely soft soil deposites and are expected to deform considerably during earthquakes, and those bridges for which detailed investigations on requirements of ductility of structures are needed.

(2) Two types of earthquake response analyses, i.e., response spectrum analyses and time history analyses can be used.

(3) Input motions used for the time history analyses shall be selected from strong-motion acceleration records with consideration of dynamic characteristics of bridges and characteristics of the records.

In determining input seismic motions, two procedures are proposed. One is to estimate expected intensities at the side based on the lifetime of the bridge and recurrence period of earthquake occurrence. Another procedure is to estimate the expected ground motions by assuming the locations and the magnitudes of earthquakes around the site. In the second ground motions can be evaluated either by the theory of seismic gaps or the statistics of the past historical earthquakes. It is also recommended to select the input seismic motions according to objectives of earthquake response analyses. It is described that bridges shall maintain their functions for those motions which are expected to occur two or three times during their lifetimes, and the bridges shall survive those motions which are expected to occur once or rare at the site.

(4) In utilizing seismic ground motions recorded on the soft soil deposits which have appreciably different ground conditions as compared with those at a construction site concerned, it is recommended to take account of such effects in the analyses. For such purposes, earthquake response analyses based on the baserock motions are recommended.

(5) Input earthquake response spectra used for the response spectrum analyses shall be determined in view of the response spectra calculated from strong-motion accelerations. In the appendix of the New Specifications, the results of statistical analyses of strong-motion acceleration records are presented.

Some of the representative earthquake response spectra and relation between maximum horizontal accelerations and epicentral distances are presented in Figs. 9 and 10, respectively.

Design Seismic Coefficient Considering Ductilities

In order to avoid brittle failure during earthquakes, it is extremely important for reinforced concrete structures to have adequate ductilities. A provision that stipulates the seismic coefficient used for the design of reinforced concrete piers with ductilities is newly introduced in the New Specifications. It is stipulated that the design seismic coefficient considering ductilities shall be determined according to the following equation.

$$k_{hd} = \nu_4 \cdot k_h \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

where

k_{hd}: design seismic coefficient with consideration of ductilities

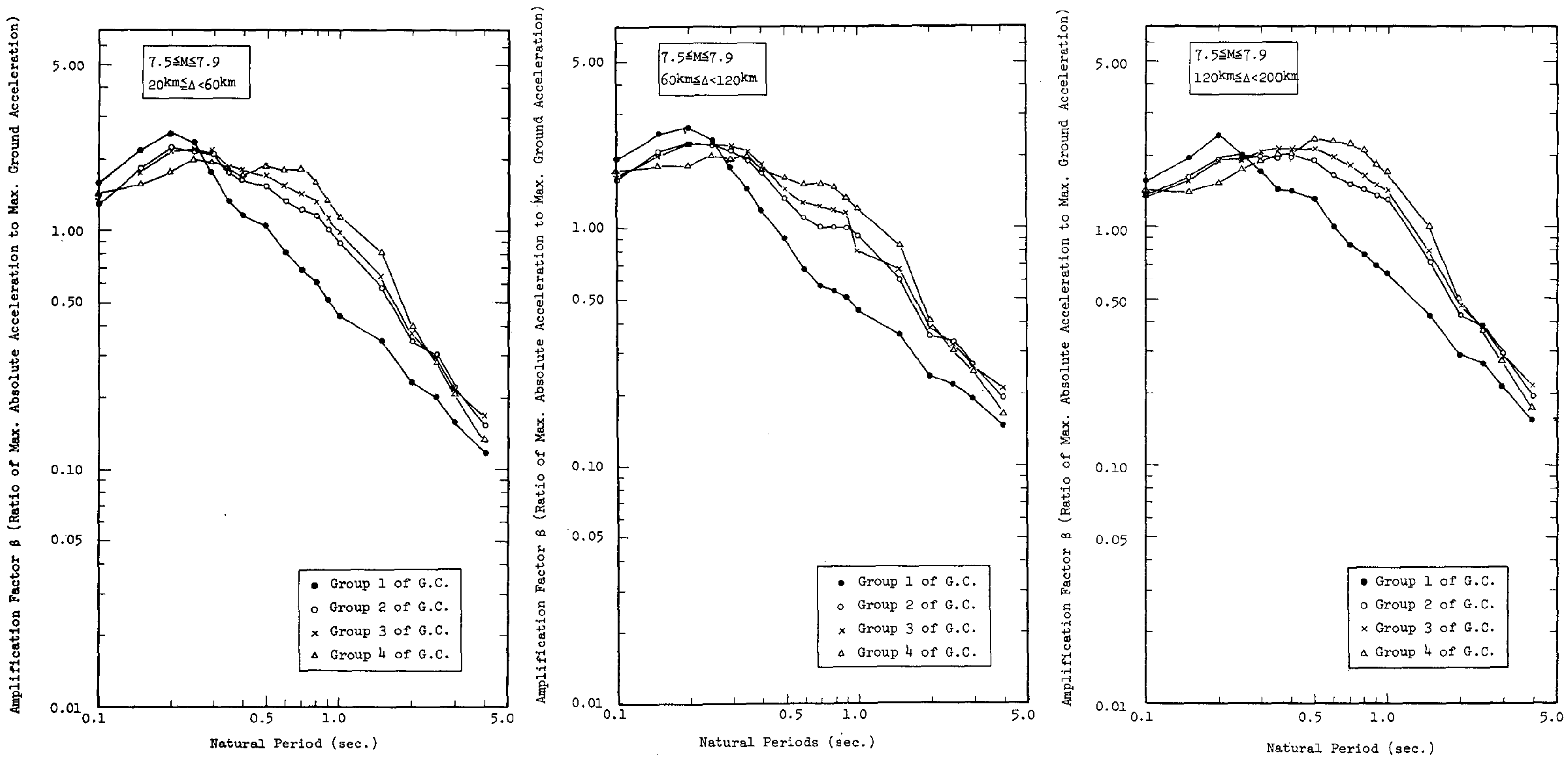
ν_4 : structural characteristics factor (greater than 1.3)

k_h : horizontal design seismic coefficient provided in Eq. (1).

Table 8 Maximum Ductilities from the Analyses of RC Bridge Piers

Section		Maximum Ductility	Number of Piers Examined
Circle-Shaped Column		6.4 ∿ 8.1	6 Specimens
Hollowed-Circle Shaped Column		5.8 ∿ 6.8	6 Specimens
2-Rectangular Column	Longitudinal	5.6 ∿ 10.5	6 Specimens
	Transverse	5.7 ∿ 8.6	4 Specimens
Oval-Shaped Column	Longitudinal	5.3 ∿ 7.3	3 Specimens

In Table 8 are tabulated maximum ductilities of ordinary RC bridge piers which were analytically determined accounting for defomations due to bendings of piers and deformations of reinforcements pulled out from footings, in which the critical strains of concrete were assumed as 0.35%. It can be recognized from these results that the maximum ductilities of bridge piers which are normally designed by the seismic coefficient method can be taken as approximately 6. However, since values of maximum ductilities are derived from analytical calculation for a half cycle loading, it is considered desirable to take maximum design ductilities to be smaller than 6. With considerations of the fact that the ductilities of concrete piers decrease significantly under alternatingly repeated loading conditions[15], one third of the values tabulated in Table 8, which lead to about 2, is recommended as the ductility factor for the design purpose.



(a) 20km ≦ Epicentral Distance Δ < 60km

(b) 60 km ≦ Epicentral Distance Δ < 120km

(c) 120km ≦ Epicentral Distance Δ < 200km

Fig. 9 Response Spectrum Curves β in case of $7.5 \leqq M \leqq 7.9$

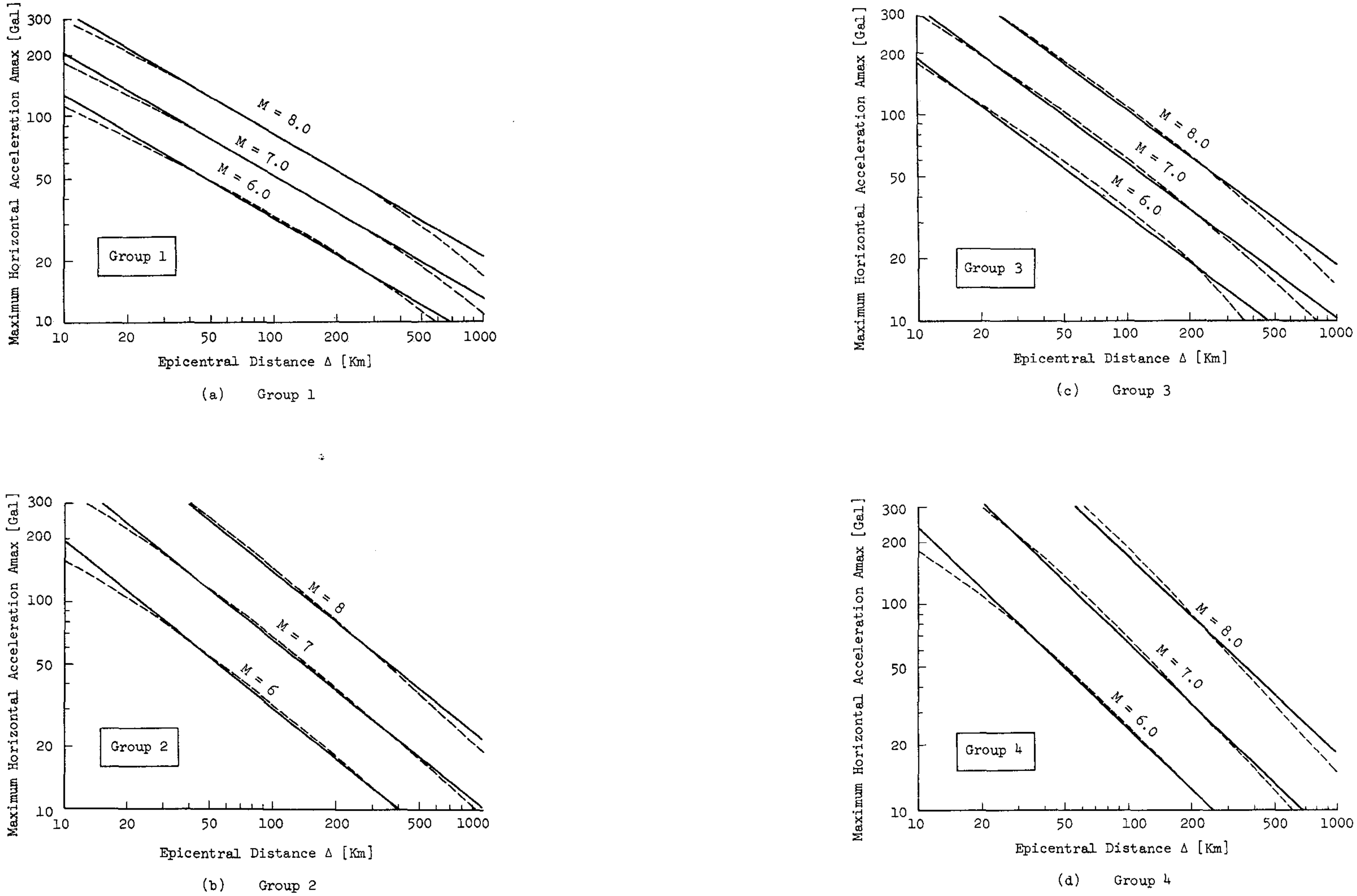


Fig. 10 Relationship Between Maximum Horizontal Acceleration (Amax) and Epicentral Distance (Δ) Obtained by Statistical Analyses of Strong-Motion Acceleration Records

CONCLUSIONS

Earthquake resistant design criteria and practices of the design of highway bridges in Japan are briefly described with emphasis on improvements and modifications in the Draft of New Specifications (JRA-1979). In view of the history of earthquake resistant design of highway bridges, it is considered necessary in the future to concentrate comprehensive investigations on the following subjects.

Analysis of Seismic Behavior of Substructures in Liquefied Soil Layers

Due to the comprehensive researches conducted after the Niigata Earthquake of 1964, it became practically possible to determine the vulnerability of saturated sandy deposits and to judge insitu liquefaction potential. Furthermore, it will be needed to conduct the investigations on the seismic behavior of substructures in the liquefied layers and to develop suitable earthquake resistant design procedures of bridges under such conditions.

Analysis of the Effects of Soil-Structure Interactions

Recently it becomes frequent that bridges are constructed on deep and soft soil deposits. From the evidences of the past extensive earthquake damages, it is well recognized that the influences of surrounding subsurface grounds are very important for the seismic responses of substructures, especially for substructures which are embedded into the deep soft ground. Consequently, considerable interests have been concentrated on the soil-structure interaction effects on such structures through model experiments and theoretical calculations. However, very limited researches have been undertaken for investigating seismic responses of actual substructures during strong seismic excitations. It is encouraged to investigate the effects of soil-structure interactions, by utilizing strong-motion records obtained at actual bridges. For this purpose, it is recommended to extend the strong motion observations, especially simultaneous observations both on bridges and surrounding subsurface grounds are desired.

Experiments on Ductilities of Bridge Piers

Seismic damages to bridges were most commonly caused by pier and foundation failures. It is, therefore, extremely important to prevent brittle failures of substructures. Up to the present, very limited experimental studies have been conducted on hysteretic behavior of bridge piers under cyclic loading. Such a lack of data on the hysteretic response of piers is one of major obstacles to introduce limit design of bridges accounting for ductilities of the members. It is recommended that extensive efforts be devoted into accumulations of such experimental data.

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APPENDIX CONTENTS OF PART V EARTHQUAKE RESISTANT DESIGN, SPECIFICATIONS FOR HIGHWAY BRIDGES [DRAFT - 1979], JAPAN ROAD ASSOCIATION

Contents		Same As the Current Spec. (include minor modification)	Modified		Newly Introduced
			Slightly	Completely	
Chapter 1	General 1.1 Scope	O			
	1.2 Definitions of Terms	O			
Chapter 2	Basic Principles for Earthquake Resistant Design		O		
Chapter 3	Loads and Conditions in Earthquake Resistant Design 3.1 General	O			
	3.2 Seismic Effects	O			
	3.3 Seismic Forces Transmitted from Superstructures to Substructures		O		
	3.4 Earth Pressures during Earthquakes		O		
	3.5 Hydrodynamic Pressures during Earthquakes	O			
	3.6 Classification of Ground Conditions in Earthquake Resistant Design		O		
	3.7 Soil Layers Whose Bearing Capacities are Reduced in Earthquake Resistant Design			O	
	3.8 Ground Surface Assumed in Earthquake Resistant Design			O	
Chapter 4	Design Seismic Coefficient 4.1 General		O		
	4.2 Design Seismic Coefficient in the Seismic Coefficient Method	O			
	4.3 Factors for Modifying the Standard Horizontal Design Seismic Coefficient		O		
	4.4 Design Seismic Coefficient in the Modified Seismic Coefficient Method Considering Structural Response			O	
	4.5 Earthquake Motions in the Earthquake Response Analyses				O
	4.6 Design Seismic Coefficient Accounting of Ductilities				O
Chapter 5	General Provisions for Design of Structural Details 5.1 General		O		
	5.2 Devices for Preventing Superstructures from Falling		O		
	5.3 Vertical Seismic Forces for Design of Connections between Superstructures and Substructures	O			
	5.4 Methods for Transmitting Seismic Forces at Connections between Superstructures and Substructures		O		
	5.5 Devices Expected for Decreasing Seismic Forces	O			
Appendices I.	References on Liquefaction			O	
II.	Examples of Classification of Ground Conditions			O	
III.	Example of Estimation of Design Horizontal Seismic Coefficient	O			
IV.	References on Earthquake Motions in the Earthquake Response Analyses			O	
V.	Example of Calculating Natural Periods Utilized in the Modified Seismic Coefficient Method			O	
VI.	Example of Calculating Ductilities				O
VII.	Major Related Provisions Concerning Earthquake Resistant Design		O		
VIII.	Detailed Practices			O	
IX.	References			O	

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AN OVERVIEW OF THE STATE OF PRACTICE IN EARTHQUAKE RESISTANT DESIGN OF BRIDGES IN NEW ZEALAND

by

H.E. Chapman
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ABSTRACT

The paper briefly summarizes previous New Zealand design requirements governing earthquake resistant design of bridges, before setting out the present methods in more detail. Treatment is split into sections on Design Philosophy, Design Methods, and Problems Encountered in Implementation. Most of the content comprises discussion of design methods used, including the requirements for strength and for structure ductility. Methods by which designers can assess structural capacity for these two aspects are set out. The design approach adopted for smaller structures with indeterminate response to shaking is discussed and consideration of current methods of design of retaining walls, bridge abutments and foundations is also included. The final part of the design section contains information on details used in typical bridges, together with reproductions of some of the relevant details.

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INTRODUCTION

General

New Zealand is a small country with a low population density. Roading and railways have played a major part in its development over the last 140 years of European settlement. The country is mountainous in nature, and a mountain chain, ranging up to 3,764 metres, forms a backbone of the two main islands, which together with the smaller Stewart Island, extend over 1,600 km north to south but are no more than 450 km wide at their widest point. In spite of its population being only just over three million, there are over 95,000 kilometres of road, of which 39,000 km have a sealed surface and 11,200 are designated as state highways or motorways. There are also approximately 4,800 km of railways. The mountainous nature of the land results in bridging being of considerable significance in the transport network.

The seismicity of New Zealand is regarded as moderate and does not approach that of other parts of the circum-Pacific system such as Japan, Chile or the Phillipines. The frequency of earthquake occurrence throughout the country varies, but within the most active area covering most of the North Island and the northern part of the South Island, a shock of magnitude 7 can be expected about once every 10 years, magnitude 6 about once a year and magnitude 5 about ten times a year. In recent times the strongest recorded shocks have registered $7\frac{3}{4}$ on the Richter scale, with a shock in 1855 believed to have reached magnitude 8.

By nature of the small population there exists in the fields of civil engineering design and research a close liaison which allows development and application of new approaches to proceed in a relatively rapid way. This results from the good communication possible within a small group of participants. Much effort has been directed in the last 8 years towards developing rational and practical methods of designing bridges for earthquake resistance. These have been based on results of research carried out both overseas and within New Zealand.

RECENT HISTORY OF DESIGN METHODS

The need to design for horizontal loading arising from earthquake ground motions was recognised in New Zealand for several decades, but reasonably sophisticated methods were not introduced until about 10 years ago. Initially the design requirement, as specified in the *Bridge Manual* [1], was for adequate strength to be provided to resist a horizontal force of 0.1 times the structure's weight at 133% of normal permissible stresses. No emphasis was placed on the care necessary in choosing the form of the structure for optimum seismic behaviour, nor on the need to detail elements with care.

In 1965 a new New Zealand Standards Association loadings code [2] was published, in which recognition was given to the dynamic behaviour of structures and likely response to what was judged to be a reasonable 'design' ground motion spectrum [3]. The code, which was primarily for buildings, included allowances both for structural dynamic properties and for, crudely, the probability of occurrence of the 'design' earthquake by introducing 3 seismic zones for design purposes. This was a contentious issue, as the seismologists questioned whether a building owner would thank the designer for a building that would collapse only less often. They maintain that a severe earthquake, in excess of the design motion, could strike in any part of the country. However, from an economic viewpoint, the logic of attempting to even out probabilities of structures reaching their design strength throughout the country is still favoured. Zoning is still used, although zone boundaries are open to question in view of more recent work [4].

It has been noticeable in published literature and in the proceedings of international conferences, that the problem of earthquake resistant design of bridges has received little attention in comparison with that devoted to building structures. While design techniques for buildings were being advanced to prevent both expensive secondary damage and loss of life or injury, bridge designers were slow to recognise the principles adopted, and to adapt them to their type of structure. This probably resulted from the inevitable specialisation that occurs in both building and bridge design areas. Furthermore, ingrained habits of using elastic design methods die hard and these hindered appreciation of likely seismic behaviour. In New Zealand interdisciplinary transfer of personnel has been beneficial in stimulating consideration of bridge design methods relative to the results of research work on building structures. While bridge structures are in some ways 'simpler' than buildings they have particular seismic problems of their own. Transfer of design ideas from buildings to bridges is not straightforward and for this reason seismic research work in New Zealand has recently been directed to bridges in a specialised manner. This has been co-ordinated and supported financially through the Structures Committee of the Road Research Unit, which is part of the NZ National Roads Board. Much progress has been made through research projects in the last 5 years and some of the results are reported to this workshop by Blakeley [5] and Priestley [6].

Earthquake resistant design standards for highway bridges in New Zealand are specified in the *Highway Bridge Design Brief* [7]. This was first issued in 1971 and has been updated periodically. The current design approach for earthquake resistance included in the design brief is summarised and discussed below.

PHILOSOPHY

General

The overriding principles adopted in New Zealand for earthquake resistant design of bridges are:-

(a) At least small to moderate earthquakes should be resisted within the elastic range of the structural components, without damage.

(b) Exposure to shaking in excess of the 'design' earthquake should not cause collapse of all or part of the bridge. The 'design' earthquake for the most active zone is assumed to have a smoothed elastic response spectrum of magnitude similar to that obtained from the strongest horizontal component of the El Centro 1940 earthquake record.

(c) The bridge should be usable, at least by light traffic, as soon as possible after the 'design' earthquake has occurred. It is accepted that some minor repair or temporary measures may be necessary to permit this.

(d) Any structural damage occurring should preferably be visible, but in any case should be readily accessible for inspection and repair.

In New Zealand moderately sized bridges form the bulk of the annual bridge construction programme. It has been found that with conventional structural forms it is not normally economically justifiable to design such bridges with sufficient horizontal strength in the foundations and piers for them to remain elastic during the design earthquake. Costs of foundations, in particular, increase steeply as the imposed design horizontal load increases. Consequently the design philosophy currently favoured is one of limiting, or at least reducing, the horizontal force which can develop between the foundation and the structure above during earthquake shaking. In parallel with continuing research [6,8] the New Zealand Ministry of Works and Development Civil Engineering Division has been developing methods of providing earthquake protection of bridges. Two main approaches have been used:

(i) The first method, to which emphasis will be given in this paper, ensures that the structure can deform in a ductile manner beyond its elastic limit, thereby limiting the seismic loading that it will be required to react. A plastic mechanism must be able to form and, due to the nature of bridge structures, the plastic hinges would usually form in the piers or foundations. These must therefore be capable of considerable plastic deformation. Design precautions are normally taken to cause hinging to occur in the piers rather than in the foundations wherever possible, thus reducing the likelihood of damage occurring in less accessible parts.

(ii) The second method more fully described elsewhere in this workshop [5] is based on increasing energy dissipation during earthquake motions by introducing specially developed devices between the piers and the superstructure. Such devices effectively increase the damping in the structure, thereby reducing its elastic response to earthquake shaking. In such cases to date, the principles used in (i) above have been applied to design of the piers to give improved performance in the event of excessive earthquake motions. The ductile demand on such piers would not normally be very great, however.

In order to encourage designers to keep in mind that structures must contend with a dynamic cóndition during earthquakes, three structural types have been defined. These are acknowledged as being idealisations but it is advantageous to use such classification when specifying loading, design displacements etc.

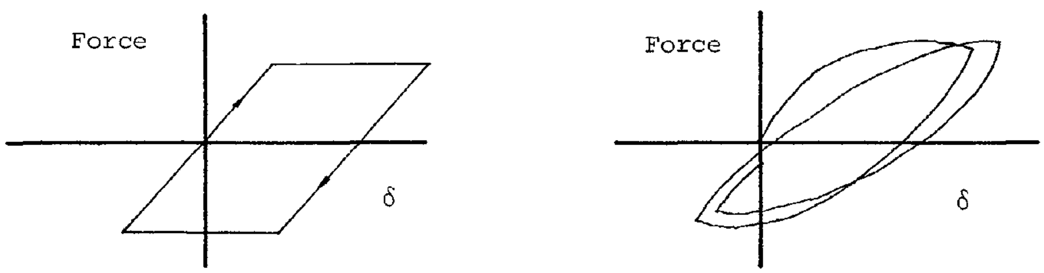
Definition of Structure Types

For the purposes of design structure types are defined with reference to the relationship between the applied horizontal force and the resultant displacement (δ) of the centre of mass of the structure.

'Fully-ductile' structures are those in which a plastic mechanism can form in the structure. The relationship is essentially one where, after 'yield', the resultant displacement increases without appreciable increase in applied horizontal force (see Fig 1). In addition, the relationship must apply for reversing loads and over at least several cycles, to ensure hysteretic dissipation of energy.

'Partially-ductile' structures are those where some of the earthquake resisting elements (eg piers in flexure) yield while others (eg elastomeric bearings at abutments) remain elastic. With increasing displacement the applied force increases although at a decreasing rate (see Fig 2). As for 'fully-ductile' structures the relationship must apply for reversing loads to ensure hysteretic dissipation of energy.

'Non-ductile' structures are those where no earthquake resisting elements yield and the force/displacement relationship develops neither a yield 'plateau' nor a hysteretic energy dissipating capability. The force/displacement relationship includes elastic behaviour leading to sudden and irreversible reduction of load capacity (see Fig 3).



(a) Idealised (b) Actual

FIGURE 1 : 'FULLY-DUCTILE'

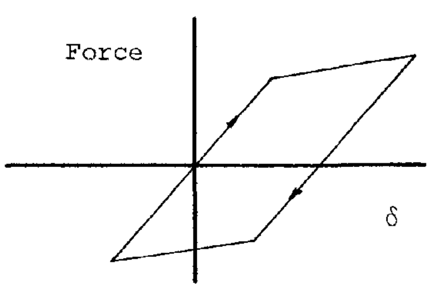


FIGURE 2 : 'PARTIALLY-DUCTILE

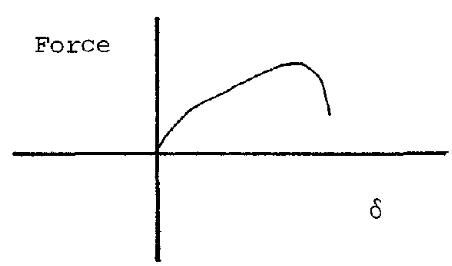


FIGURE 3 : 'NON-DUCTILE'

General Design Approach

Ideally, bridge structures should be designed so that earthquake induced energy will be dissipated by members acting in a ductile manner, avoiding brittle shear failures. These may be main members (eg piers) or supplementary members (eg energy dissipating devices). The ease with which this can be achieved varies considerably with the type and form of the structure and in some cases it may be uneconomic to design to this criterion. A compromise is then necessary and in such cases it is acceptable for secondary parts of the structure to fail in a brittle manner, provided that the risk of collapse is not increased unduly and provided that with such damage the bridge would at least carry light traffic. Design should ensure that any damage occuring during seismic motions is minimal. Where possible it should be designed to occur in a predictable and accessible position.

Design Sequence

The following design sequence is recommended:

(a) The economics of providing a ductile structure capable of resisting the minimum specified design loads should be examined as the preferred method of providing the required earthquake resistance.

(b) In structures where ductile action will not occur at minimum specified loads, provision of additional shear capacity, to allow the resistance of greater loads in a ductile manner should be examined, up to the maximum equivalent to the theoretical elastic response load, which is taken as 6 times the specified design load.

(c) As an alternative, where the designer judges that the economics justify it, provision may be made for a lower standard of seismic performance based on a 'non-ductile' approach, accepting that some brittle failure damage will occur during strong earthquakes. The loading discussed later and set out in Table 2 should be used as a minimum design value in this case.

Design Implementation

'Fully-ductile' or 'Partially-ductile' Structures — A ductile structure may be provided either by designing for plastic hinges to form (usually in the piers rather than in the superstructure) or by introducing ductile energy dissipating devices between the superstructure and its supports. In either case, the likely variations in properties of the materials used must be considered. The resisting members must have the capacity to accommodate the maximum as well as the minimum post-elastic moments and shears resulting from these variations. For example, for this purpose a maximum frictional coefficient of at least 0.15 is normally assumed for stainless steel/PTFE sliding bearings.

Research work recently carried out in New Zealand and reported by Tyler [9] has given useful design data for PTFE/stainless steel bearings under dynamic conditions. The report shows frictional forces to vary with temperature, sliding speed and pressure intensity. The adopted values have been chosen as the likely extremes under normal conditions.

'Non-ductile' Structures — Although for many structures designed to the criteria for 'fully-ductile' structures the financial investment is acceptable, there are forms of bridge for which the resultant investment becomes unjustifiably large. This is mainly the result of the large design forces necessary when a plastic mechanism cannot form at the specified design load in a particular direction in parts or all of the bridge, (eg bridges with short piers; abutments giving transverse restraint to superstructures; one or two span bridges where longitudinal restraint is necessarily at abutments). In such cases a lower standard of earthquake resistance is acceptable in that direction in the part of the bridge concerned. Structures designed as being in the 'non-ductile' category should be detailed in such a manner that the lower level of earthquake resistance is achieved mainly in the form of a lower standard of post-elastic behaviour, but not by an appreciable increase in risk of collapse. Any strong motion damage occurring should be accessible for detection and repair. For safety and accessibility reasons, it is generally preferable for failure to occur in shear keys rather than in piers or foundation elements below water or ground level.

It is acknowledged that in the type of structure generally likely to fall into this category it is difficult to predict with certainty the point of weakness. This is due to the uncertainty of relative maximum capacities of elements such as elastomeric bearings in shear or of piled foundations in shear or pull-out. Consequently it is the designer's responsibility to use a structural form with as predictable behaviour as is feasible so that the likely distribution of forces through the structure will lead to the structure behaving as well as possible during earthquake shaking.

It is desirable that members (eg such as foundation piles) should be detailed in such a way that ductile behaviour will be encouraged under unpredictable earthquake displacements even though a structure may be considered to be in the 'non-ductile' class. Attention to provision of increased shear strength in such members is likely to afford worthwhile value in protection against sudden brittle failure. Where economics justify it, it is accepted that a design may be based on the occurrence of pier base rocking or pile pull-out, as both types of behaviour are preferred to failure of an important member in a brittle and irreversible manner. It is considered that under dynamic loading pile pull-out or base rocking would be limited to small amplitude movements. This approach is however only favoured as a means of avoiding brittle failures or where very expensive foundations would otherwise result. Undesirable permanent displacements of the foundations and superstructure are very likely to occur with this type of behaviour and foundation damage would be particularly difficult to rectify.

General Comments on Philosophy

As is evident from the foregoing summary, the aim is to achieve as good a design as possible, while taking investment in earthquake resistance into account. Achievement of this aim is very dependent upon the designer's judgement and on his understanding and following of the spirit of intent of the 'design brief' which is written on this basis. It has not yet been found feasible to write into code form one set of design rules which always lead to good results without excessive cost. This arises from the wide variation of site conditions and geometry for which bridges must be built. Provided the overriding principles governing collapse, serviceability after an earthquake and location of damage, as previously set out, are observed the designer carries the responsibility for obtaining the best solution available for reasonable earthquake resistance investment. Choice of suitable structural form is regarded as the most far-reaching aspect of the engineering of the structure in achieving this aim. The proportioning of members is also of importance in influencing the structure's dynamic response.

DESIGN METHODS

Design of 'Fully-ductile' Structures

The adopted philosophy is one which recognises that structural strength and consequent extent of inelastic behaviour during the 'design' ground motions are inter-related. Design practice in New Zealand places emphasis on ensuring that structures possess both adequate strength and adequate ductility. Procedures have been developed for each aspect and are in common use. They are summarised below:

Strength — Design for strength uses 'capacity design' principles (Park and Paulay) [10] assuming inelastic behaviour to occur in predetermined locations in the structure. The aim is twofold:

(i) To ensure a minimum *dependable* strength for the plastic hinging intended to develop during strong ground motions. This then prevents undue damage occurring during the more frequent moderate shaking;

(ii) To recognise that the plastic hinges developing are likely to possess flexural strengths in excess of the minimum dependable values. Thus, other members in the structure intended to remain elastic are designed on the basis of the plastic hinges developing their *overstrength* flexural capacities.

The following definitions are applied to sections of a structural member:

The *ideal strength* S_i is the theoretical limit strength, based on the section geometry as detailed and on the nominal minimum material strengths.

The *dependable strength* S_d is related to the ideal strength by the capacity reduction faction ϕ.

$$S_d = \phi S_i$$

where ϕ is less than 1.

The *probable strength* S_p takes account of materials usually being stronger than the nominal minimum strengths. Thus

$$S_p = \phi_p S_i$$

where ϕ_p, the probable strength factor, is greater than 1. For design convenience for reinforced concrete sections S_p is calculated in the same way as is S_i, but reinforcement yield stress is assumed to be 1.15 fy, which reflects the usual margin of actual yield stress over the minimum specified value.

The *overstrength* S_o takes account of all the possible factors that may contribute to section strength, such as overstrength reinforcement, increased reinforcement stress due to strain hardening at large deformations and a concrete strength higher than specified minimum. Thus,

$$S_o = \phi_o S_i$$

where ϕ_o, the overstrength factor, is greater than 1.

For reinforced concrete sections S_o is calculated in the same way as is S_i, but reinforcement yield stress is usually assumed to be 1.25 f_y. Thus the structural analysis and design procedure comprises two stages, summarised in basic form below:-

(i) Design plastic hinge sections to have the minimum required flexural strengths.

> Decide structural form and choose desired location of plastic hinges to allow a plastic mechanism to develop;
>
> ↓
>
> Carry out elastic analysis under specified loads included in load effect combinations (discussed later in paper);
>
> ↓
>
> Hence determine minimum flexural strengths required for plastic hinges and design these sections to have dependable strengths to match requirements.

(ii) Design all sections other than the plastic hinges for shear and flexure. Design plastic hinges for shear.

> Calculate overstrength flexural capacities of plastic hinges as designed in (i) above;
>
> ↓
>
> Analyse structure assuming all plastic hinges to have developed their overstrength flexural capacities. Hence determine shear and moment capacities required for all sections other than the plastic hinges, and design sections accordingly. Design plastic hinges for shear.

Because economics of foundations are so sensitive to the applied flexure and horizontal loading, recommendations have been developed so as to ensure that the structure not only behaves as intended, but also that excessive compounding of safety factors is avoided. For example, it is not considered necessary to match the overstrength moment capacity of a plastic hinge at the bottom of a pier with the dependable strength of the piles supporting the pier, but rather the $\frac{\text{probable strength}}{1.15}$ is recommended. The 1.15 value is intended as a 'safety factor' against the plastic hinging developing in the piles. Thus it is accepted that all structural members would be working in flexure near to yield values, but design factors are applied to encourage the plastic hinging to occur in the desired location. In aspects where more certainty is needed (for example in protecting against shear failures), the dependable member strength in shear is matched against the shear force which would develop in the member with the plastic hinges developing overstrength moment capacity.

While it is recognised that plastic action may occur at the top of foundation piles, it is not normal practice for 'fully-ductile' structures to be designed on the assumption that a complete plastic mechanism will form in the foundation members. Uncertainties of soil properties considerably reduce confidence in predicting the horizontal shear necessary to cause the second plastic hinge to form, lower down in the pile.

Ductility — The following definitions have been adopted in New Zealand bridge design practice:

(a) *Displacement ductility* (structure ductility) gives a measure of the extent to which the centre of mass of a structure may be displaced beyond the yield displacement.

$$\text{Structure ductility} = \mu = \frac{\text{limit ('ultimate') displacement of the centre of mass of the structure}}{\text{displacement at yield}} = \frac{\delta u}{\delta y} \qquad \text{(Figure 4)}$$

(b) *Curvature ductility* (section ductility) gives a measure of the extent to which the curvature of a section may be increased beyond the yield curvature.

$$\text{Section ductility} = \frac{\text{limit ('ultimate') curvature of the section}}{\text{curvature at yield}} = \frac{\phi u}{\phi y} \qquad \text{(Figure 5)}$$

It should be noted that Figures 4 and 5 illustrate idealisations of relationships applying in practice. The actual form of the curves is shown in Figure 6. Departure from the elastoplastic idealisation occurs because:

(i) steel is unlikely to reach yield stress simultaneously throughout the section. For example, in circular or rectangular sections with side steel, the bar layout precludes this; and

(ii) properties of concrete vary with strain, depending on the extent of cracking and on the effect of confining reinforcement.

To simplify specification of the required structure ductility, 'yield' deflection of a structure is taken to be δy (Figure 6) rather than $\delta' y$, which is equivalent to the initiation of reinforcement yielding (point A, Figure 6). Thus 'yield' deflection is found at the intersection of the line projected from the origin and passing through point A, with the horizontal line through the ultimate strength for the structure. This approach is more convenient for practical design purposes. The ratio $\delta y/\delta' y$ depends on the shape, reinforcement and average axial stress in the pier sections where plastic hinging occurs.

A similar definition is adopted in the case of section ductility.

Ability to calculate available structure ductility is clearly desirable for structures designed to a philosophy in which it is one of the parameters. For this reason a design publication [11] has been produced which summarises the methods for calculating ductility and includes design charts. The methods have been described elsewhere [12] but an extract is included below. Design methods are based on results of research work undertaken in various places, but much of it has been performed at University of Canterbury, New Zealand.

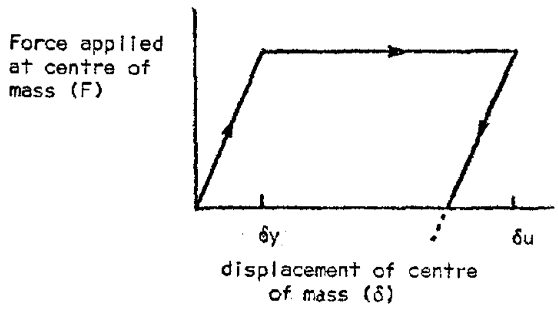


FIGURE 4

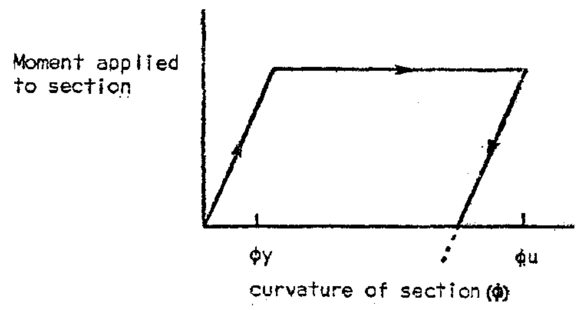


FIGURE 5

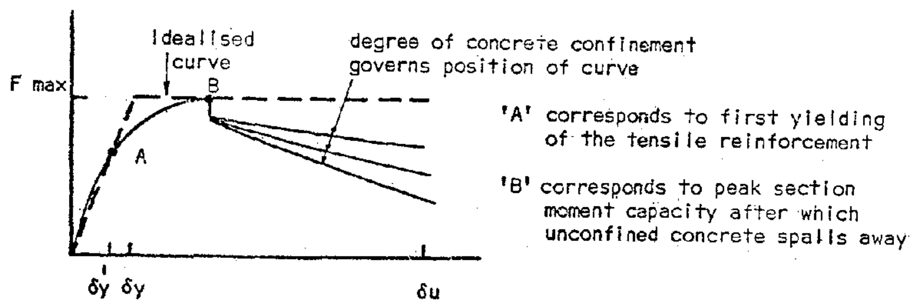


FIGURE 6

Park [13] outlined the approach and this was used, together with results of later work such as that investigating the material stress/strain curves [14].

Calculation of available structure ductility considers:

(a) the section ductilities of each section of pier in which plastic hinging is intended to form, and

(b) the consequent structure ductility available.

Section ductility was defined previously as $\frac{\text{limit curvature of the section}}{\text{curvature at yield}}$ which may be expressed as $\phi u / (\phi y' \times \frac{\phi y}{\phi y'})$ where $\phi y'$ = curvature at first yield of the reinforcement. $\phi y'$ can be calculated from the relationship of the steel strain at first yield and the distance between the tensile steel and the neutral axis. The ratio $\phi y/\phi y'$ depends mainly on the reinforcement layout, the section shape and the average axial stress in the section.

The value of ϕu is calculated from the relationship of the limit concrete strain and the depth to neutral axis when the limit strain is reached. The design charts take account of the variables involved and allow both the strain and the depth to be readily derived.

An alternative method for defining and predicting limit curvature is discussed by Park and Paulay [10 pp.594-599]. It refers to moment/curvature relationships of a section derived analytically. The limit criterion suggested is the curvature achieved when the moment of resistance has reduced to 85% of the ultimate value. While reducing an approximation inherent in the present approach, this method does introduce another variable for the designer to take into account. However, with more test results to support the analytical approach developed, and with presentation appropriate to design office use, this method offers prospects of improvement to this aspect of the present approach.

Whichever method is used for determining the limit section curvature ϕu, a relationship must be established between section and structure ductilities so that the latter can be calculated.

Reference 11 is based on the concept of an equivalent plastic hinge length discussed more fully by Park and Paulay [10 pp.244-250]. Suitable design charts and examples of the analysis of various structural forms of bridge are included in reference 11. The approach used in all cases of design application is to develop the force/displacement relationship for the centre of mass of the structure. It is important to take into account the flexibility of all components, such as foundations and elastomeric bearings, since their effect in reducing available structure ductility can be considerable.

It is acknowledged that the current approach includes approximations which make the result little more than an estimate of structure ductility. Consequently it would be unreasonable to use it for anything more than the single degree of freedom structure. Compounding of the approximations involved in calculating required and available structure ductilities for a multi-degree of freedom structure would lead to inaccuracies which would not warrant the amount of calculation involved. In spite of the inherent approximations it is felt that the introduction of such a procedure into design office practice has the following benefits:

1. For design of the substructure and piers, where earthquake requirements usually govern, consideration of ductility becomes as much a part of the process as does consideration of strength.

2. Presentation of design aids allows the relative earthquake performance of alternative structural forms to be more readily assessed during preliminary studies.

3. The designer is better able to investigate the effect of various parameters and hence to optimise the structure.

4. Bridge design is itself a specialist subject. Designers involved are not always in a position to keep abreast of all details of research in the complex area of seismic design. A method such as has been discussed allows the designer to assess the effect of various parameters while avoiding detailed involvement in complex background material. This should lead to structures which behave better in earthquakes.

Design of 'Non-Ductile' Structures

Structures of this type respond elastically to earthquake shaking, provided the shaking does not induce a loading which exceeds the structure's capacity. The elastic response of such a structure to the 'design' earthquake adopted would normally considerably exceed the design loads specified for 'fully-ductile' structures, although equivalent damping of 'non-ductile' structures is likely to be greater than the value of 5% assumed in deriving the loading curves for 'fully-ductile' structures.

Structures in the 'non-ductile' category are designed by strength design methods but because of their nature, the capacity design approach and ductility checks are not normally applicable. An elastic analysis of the structure is therefore performed, applying the design loading and load combination specified for 'non-ductile' structures (discussed later). Members are designed to resist the forces and moments assigned to them from this analysis. It is stressed that designers must consider how the structure would behave in more severe shaking - for instance slab type piers founded on piles would behave elastically in a transverse direction until, usually, the foundation piles yielded in flexure. For this reason piles or cylinders are always detailed with confining steel over the upper two diameters at least, to ensure that they have ductile capacity, even though it is not subject to calculation.

Design of 'Partially-Ductile' Structures

This category of structure covers a wide range between 'fully-ductile' and 'non-ductile' types. The proportionate effects of the elements which behave elastically and those with ductility determine where in the range a particular structure lies.

Because of the wide range of structures covered, it is difficult to specify recommended design loads for this group. The designer's judgement must play a major part in deciding the forces for which such a structure should be designed, taking into account the cost and benefit gained in seismic protection. Any improvement over the 'non-ductile' standards discussed previously for reasonable cost, can be considered worthwhile. Design principles used for designing the ductile parts of a structure are the same as for those in 'fully-ductile' structures.

'Partially-ductile' structures are likely to fall into two main categories:

(i) those in which structural members enter the plastic range in flexure, while other members contributing to the seismic restraint remain elastic;

(ii) those in which energy-dissipating devices act in conjunction with other members which remain elastic. Such structures may also include members which are designed to enter the plastic range in flexure during more severe earthquake disturbance.

Design Loading

The minimum elastic strength required for structures subject to earthquake loading is derived from the load effect combination specified in *Highway Bridge Design Brief* [7]:

$$U = 1.08 \left(KDL + 1.25\ (EP + OW) + SG + ST + 1.25EQ + 0.33TP\right)$$

where U = design load for strength design method
K = 1.2 or 0.8, whichever is more severe, to allow for vertical acceleration effects
DL = dead load + superimposed dead load effects
EP = earth pressure effect
OW = effects of ordinary water pressure and buoyancy (as distinct from flood water pressure)
SG = shortening effects
ST = settlement effects
EQ = earthquake load effect
TP = overall and differential temperature effects

Earthquake load is calculated with the formula

$$V = CFW$$

V = total minimum seismic base shear force due to earthquake in the direction being considered
C = basic seismic coefficient. Values of C for 'fully-ductile' structures are shown in Figure 7; values of CF for 'non-ductile' structures are shown in Table 1.
F = importance factor, shown in Table 2.

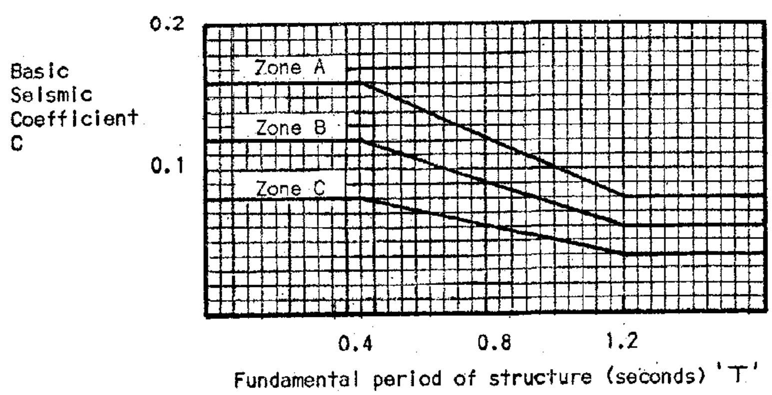


FIGURE 7 : BASIC SEISMIC COEFFICIENT FOR 'FULLY-DUCTILE' STRUCTURES

Category	Zone A	Zone B	Zone C
1	0.24	0.18	0.12
2	0.20	0.15	0.10
3	0.17	0.13	0.09

NOTE: Categories are defined in Table 2

TABLE 1 : VALUES OF CF FOR 'NON-DUCTILE' STRUCTURES

Category	Description	Minimum Value of F
1	Bridges carrying more than 2,500 vehicles per day and all bridges under or over motorways or railways	1.0
2	Bridges carrying between 250 and 2,500 vehicles per day	0.85
3	Bridges carrying less than 250 vehicles per day	0.7

NOTE: Choice of category in Table 2 should be based on the average number of vehicles per day current at the time of design. Judgement must, however, be exercised - for example, lack of alternative routes or importance in spite of low traffic usage may warrant use of a higher factor.

TABLE 2 : IMPORTANCE FACTOR

The basic seismic coefficient specified for 'non-ductile' structures is more severe than for the equivalent 'fully-ductile' structure. This is intended to impart more inherent strength to structures which, because of their form, are likely to perform less well during earthquakes. It also encourages the designer to use the more satisfactory 'ductile' structural type by offering an inducement, in the form of less severe basic loading.

Calculation of Fundamental Period of Structure

For 'fully-ductile' structures the value of C is related to the fundamental period of the structure T in the direction of the loading being considered (see Figure 7). T is calculated from the formula:

$$T = 0.063\sqrt{\Delta}$$

where Δ is in millimetres
T is in seconds

Δ = the horizontal displacement of the centre of mass of the superstructure under the following sets of loads: The horizontal load at each level (x) is obtained by multiplying the mass at that level by a factor

$$\left(g\frac{h_x}{h_n}\right)$$

which varies linearly from zero at the base to g at the top. Δ should include the effects of foundation rotation and displacement within the supporting soils and of flexibility of other elements such as elastomeric bearings.

g = standard gravity
h_x = height of level 'x' above assumed base of structure
h_n = height of the centre of mass of the superstructure above the assumed base.

For bridges it is usually sufficiently accurate to consider only the mass of the superstructure, or to increase the mass of the superstructure by that of the cap at the top of the pier plus half of the pier stem. Δ can then be calculated as the deflection of the centre of mass of the superstructure, supported on a pier of negligible mass, with an applied horizontal acceleration of 1 g.

The value of Δ calculated is closely dependent on the stiffness assumed for the pier stem. Current practice is to calculate T on the basis that the pier stem reinforcement has just reached yield stress at the displacement Δ. For design convenience an equivalent value of section rigidity EI, applicable to the full pier stem length, is useful in calculating Δ for this condition. Research tests at University of Canterbury [15] indicate that, for a prismatic pier section, a reasonable approximation is to use the EI value for the section at first yield of the tensile reinforcement. Design aids [11] are used for this purpose.

Required Structure Ductility for 'Fully-Ductile' Structure

The specified design loading is based on the assumption that the structure can attain an adequate displacement ductility factor in the direction being considered. This is defined in general terms as:

$$\text{structure ductility} = \mu = \frac{\delta u}{\delta y}$$

where δu = limit ('ultimate') displacement of the centre of mass of the structure

δy = displacement at yield, (see Figure 6).

The ductility demand on a structure is dependent on the degree of damping inherent in the structure and its foundations. A bridge with rigid foundations on firm ground may be expected to have an equivalent viscous damping of 5% for elastic vibration, and require a structure ductility factor of about 5 for the design strengths and 'design' earthquake intensity considered appropriate for design. In order to provide a safety factor against reaching limit displacement under 'design' earthquake conditions, a structure ductility factor of 6 is considered necessary at the design strengths specified in the load effect combination section of *Highway Bridge Design Brief* and discussed previously. Where the characteristics of the structure or its foundations are such that there is greater damping, the ductility demand would decrease. A structure ductility factor of 4 is regarded as a minimum design value to safeguard structures generally. This may be difficult to achieve in structures with very flexible foundations, and in certain cases a reduced design ductility requirement (and hence a lower degree of protection) may be considered justifiable on economic grounds.

When the structure yield capacity V' exceeds the minimum specified value of CFW, the required structure ductility may be reduced. Curve A on Figure 8 shows the appropriate reduced values. Curve B represents an advisory curve where the reasonable minimum value of 4 should grade towards 6 as the structure's behaviour remains more within the elastic range.

NOTE: V' should be calculated as the actual *dependable* strength of the structure.

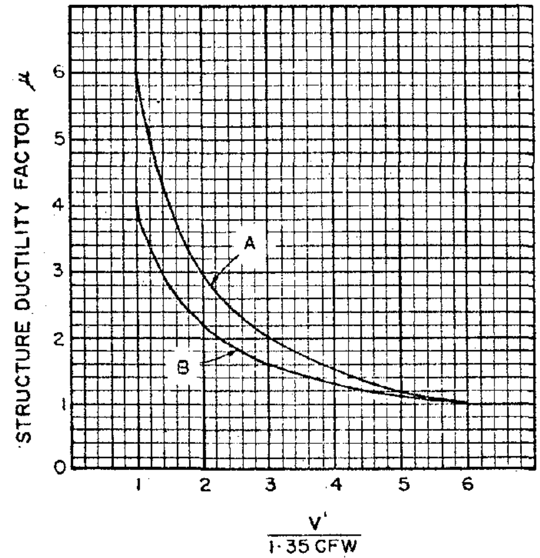


FIGURE 8 : MAXIMUM DUCTILITY FACTOR FOR OVERSTRENGTH STRUCTURES.

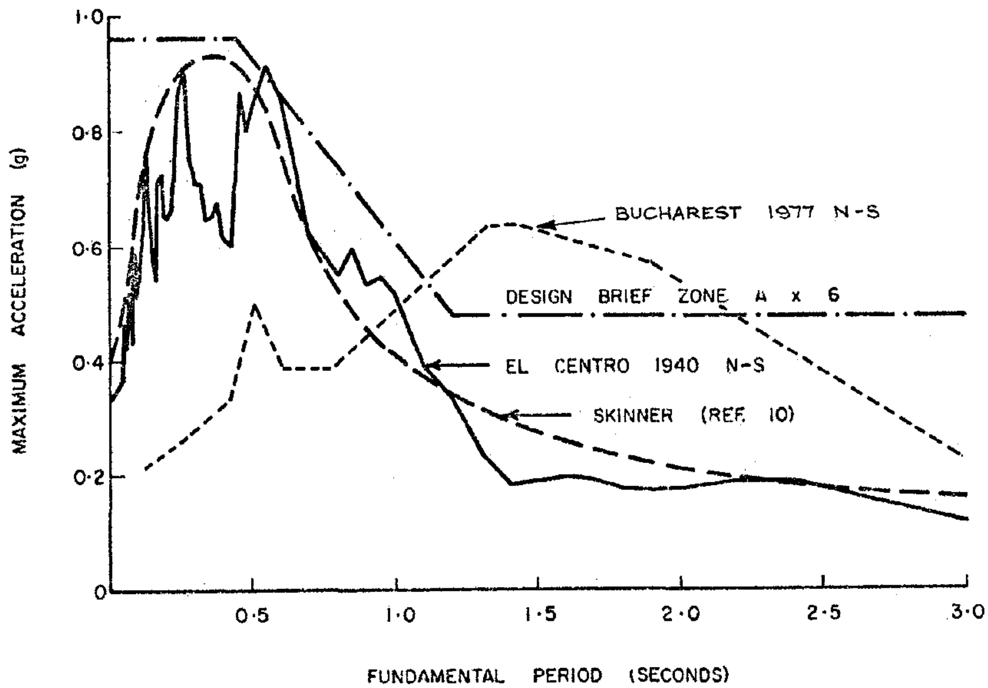


FIGURE 9 : COMPARISON OF ACCELERATION RESPONSE SPECTRA FOR 5% CRITICAL DAMPING

Importance Category	Zone A	Zone B	Zone C
1	0.24	0.18	0.12
2	0.17	0.13	0.09

TABLE 3 : SEISMIC COEFFICIENTS CF FOR EARTH RETAINING STRUCTURES

For comparative purposes an 'equivalent elastic response curve' has been approximately derived for a ductile structure by multiplying the yield strength by the ductility factor. This assumes equal maximum displacement responses for the elastic and inelastic systems. Figure 9 shows the equivalent elastic response spectrum for the zone A specified loading curve from Figure 7, assuming an importance factor of 1.0 and a required ductility factor of 6. This may be compared with the various response spectra for 5% damping also plotted. It can be seen that for long period structures provisions appear conservative. Recent inelastic response analysis work undertaken has shown this conservatism exists, but for short period structures the assumption of equal displacement response is invalid. Greater ductility factors than 6 are necessary for these structures. It is likely that increased values will be specified when work in this area is completed.

Design of Retaining Walls

Retaining wall design has been based for some years on *Retaining Wall Design Notes* [16], which presents a summary of commonly used design methods, with charts, in a form particularly convenient for design office use. Earthquake effects are taken into account using the Mononabe-Okabe pseudo-static approach. Design seismic coefficients to be applied to earth pressure calculations are shown in Table 3. They are determined without regard to the dynamic characteristics of the retaining structure or soil, but are dependent on the seismic zoning and the importance of the structure. Three importance categories are specified for earth retaining structures:

Category 1: Major retaining walls supporting important structures, developed property or services and the like, and where failure would have disastrous consequences such as cutting vital communications or services, serious loss of life, etc.

Category 2: Free-standing structures of at least 6 metres in height in locations other than covered by Category 1 and which would be difficult or costly to replace and/or where other consequences of failure would be serious.

Category 3: For all other retaining structures no specific provision for earthquake loading need be considered except that the seismic coefficient to be applied for earth pressure on bridge members should be as specified elsewhere (see Figure 10).

The specified coefficients are used at 'working load' levels and for section design, their effects are factored in the load effect combination:

$$U = 1.08\,(KDL + 1.25\,(EQ + W))$$

where

- DL = dead load of the structural element
- EP = static earth pressure acting on the element, including the effects of any surcharge loads
- EQ = earthquake earth pressure acting on the element
- W = hydrostatic water pressure
- k = 1.2 or 0.8 whichever is more severe, to allow for vertical acceleration.

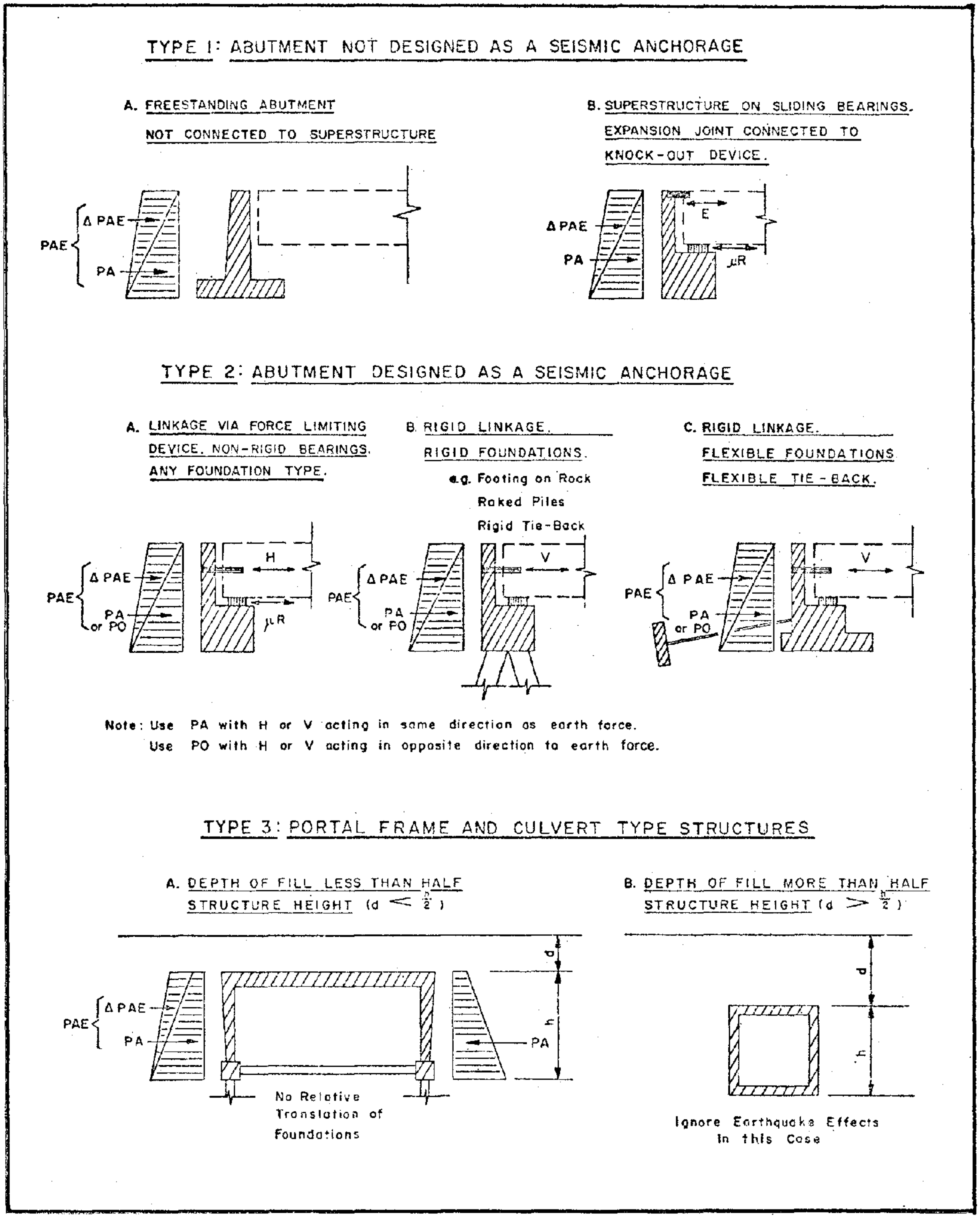


FIGURE 10 : EARTHQUAKE DESIGN FORCE ON EARTH RETAINING PARTS OF STRUCTURES

Design of Bridge Abutments

As with other areas of design involving ground conditions, the formulation of design recommendations for bridge abutments presents problems. Soil/structure interaction effects makes the behaviour complex compared to that of the pier/superstructure system, and rational design methods have been difficult to develop.

Highway Bridge Design Brief [7] contains design recommendations, which are reproduced as Figure 10. The recommendations are, however, based as much on judgement and results of application as on research data, with the aim of at least offering some guidance to the designer in a complex area. Research in this area is clearly needed. The need extends, of course, right up to the point of recommending practical design procedures for providing satisfactory and economically viable earthquake-resistant abutments. Simple design guidance such as is shown in Figure 10 is essential for application to the bulk of bridge designs carried out. In such cases it is not justifiable to undertake complex analyses involving dynamic programs and finite element techniques as may be the case for major projects. Furthermore, improvement of the general standard of earthquake resistance of bridges depends on improvement of this very large group of small to medium-sized structures.

The form chosen for the structure at initial scheme stage greatly influences abutment performance during earthquake - a basic consideration being whether the abutment will attempt to mobilise the superstructure mass or vice versa. For multi-span structures it is general practice to design superstructures to be free to move longitudinally and transversely, depending for stability on flexural capacity of the piers and diaphragm action of the deck slab. In this way it is expected that large inertia forces acting through the abutment should be avoided. Suitable movement gaps are left between abutments and main structural members, and elastomeric buffers are provided in case of impact. While this approach is favoured, other influences such as the preferred use of elastomeric bearings for standard-type beam and slab superstructures, tend to reduce its effectiveness by introducing a stiff shear connection at the abutment. Use of thick multi-layer bearings can reduce this disadvantage and are often adopted. There are many small structures where it is not possible to depend only on the piers for horizontal restraint - for example single or two-span bridges. In these cases it is accepted practice to build the superstructure to act as a prop between the two abutments. The large transverse ground accelerations, equivalent to the 'design' ground motion, can usually be easily accommodated but it is likely that out of phase motions at the abutments would induce rotation damage at the deck end.

In cases where it is unavoidable to anchor the superstructure to the abutment longitudinally, it is advisable to design the anchorage connection to withstand the elastic response of the system (eg maximum likely ground acceleration or more) or to provide a device to act as a 'fuse' or force limiter. Such a connection must function under cyclic loading and some devices have been applied in this way and are described by Blakeley [5]. Anchorage of the abutment on rock is relatively easy in these cases. On soils, abutment anchorage is assisted by using 'friction slabs', which extend for several metres back from the abutment, and are located approximately a metre below the approach fill for the full width of the abutment. Their action depends on the friction developed against the supporting soil under the pressure of the overlying approach fill. An advantage of this detail is that a ramp is provided onto the bridge in the event of approach fill settlement. It is normal practice to provide a 'settlement slab' for this purpose at subgrade level in cases where a 'friction slab' is not used.

In the past it has been observed that earthquake motions tend to cause permanent displacement of the abutments towards the bridge. It is therefore policy to design abutments and any supporting piles so as to minimise the forces developed by settlement of approach fills and to encourage the fills to flow round or under the structure in such circumstances. Wing walls are preferably constructed parallel to the abutment face to constrain fill settlement as little as possible in the transverse direction as this has been a source of damage in the past.

Design of Foundations

Current design practice for foundation members uses a pseudo-static approach. While the designer is encouraged to keep in mind the dynamic nature of the problem, member sizing and design is based on normal member design procedures. Foundation design loads are derived either by the 'capacity design' procedure for 'fully-ductile' structures or, for 'non-ductile' structures, by analysis to resist at least the minimum specified loads equivalent to the seismic coefficients shown in Table 1.

In 'fully-ductile' structures the foundations should provide sufficient flexural capacity to allow the energy-dissipating plastic mechanism to develop as intended. Support conditions for the foundation members are determined from results of site investigation tests such as the standard penetrometer. The pressuremeter or Camkometer hold potential in this area although they are not widely used at present. Translation into equivalent values of modulus of horizontal subgrade reaction is used to derive equivalent spring stiffness values for analysing piles by the beam-on-elastic-support approach. The acquisition of soil parameters for analysis is therefore crude and development of better methods in this area is needed. Translation into support properties is also crude - and more so as the dynamic effects on the modulus are not known. However, resultant pile moments are not very sensitive to the value of horizontal subgrade modulus, but displacements are. In addition to approximations in this area, two other relevant aspects are at present not specifically taken into account in our bridge design practice. Flexibility of subsoils affects site surface response. Input motion applied to the structure may therefore be affected by amplifying the effects on longer period structures. However, a counteracting influence could be expected from increased soil/structure energy dissipation. Current practice therefore, only takes account of flexible subsoils via their effect on the horizontal stiffness of the structure and hence its calculated natural period T, on which the design loadings are based (Figure 7).

Because of the uncertainty with which ground behaviour and its effect on bridges can be predicted, it is desirable to impart as tolerant a character to foundation elements as possible. Toughness under induced curvature and shears is required, and for this reason piles such as steel H sections and concrete-filled steel-cased piles are favoured. Piles and cylinders from which steel casing has been withdrawn during concreting are not used. Driven precast piles are constructed with considerable spiral confining steel to ensure good shear strength and tolerance of yield curvatures, should these be imparted by the soil or by structure response. In addition, wherever possible, use of the 'capacity design' principle limits, or at least reduces, the inertia forces likely to be induced into the foundations by the superstructure mass. Where not practicable and the 'non-ductile' approach has to be accepted, more thought should be given to behaviour of piles during severe shaking. For example, raked piles, while economically attractive for resisting horizontal loads, would behave in a brittle fashion if arranged so that only axial loads are induced. Spacing further apart, while still being raked, allows the piles to act in flexure and axial load and thus to yield in flexure if necessary. This offers a compromise for economy between rigid, raked piles and desirably flexible but less economic vertical piles. It is considered particularly important in river situations to protect piles against shear damage because such damage could go undetected until flood and deep scour conditions find such a weakness.

For sizing of spread footings and assessment of strengths generally, strength design methods (as used in ACI 318-77) are applied also to soil materials. Thus, the same load effect combination as shown in the Design Loading section is used for the earthquake case, and the following capacity reduction factors are applied to ultimate strengths of the soils, depending on the way in which the material properties were assessed:-

Test loaded piles or plate-bearing tests	0.9 to 0.7
Measured soil parameters	0.7 to 0.6
Visually assessed soil parameters	0.6 to 0.5
Dynamic pile formula (eg Hiley)	0.7 to 0.6

Details Used

It is generally accepted in earthquake engineering that careful structural detailing goes a long way to providing a structure capable of behaving well in earthquakes. Details which have commonly been used in New Zealand bridge design practice are discussed below and their application is illustrated in Figures 11, 12 and 13, which are reproduced from reference [17].

Shear Keys and Linkage Bolts - With many of the bridge superstructures being of simply supported standard units, it is essential to provide a reliable means of preventing spans dropping from their supports. Concrete shear keys projecting from the pier top, between the two end diaphragms of adjacent spans, have commonly been used in the past. More recently, these have been made of rectangular hollow steel sections cast into the pier top and filled with concrete. In either case linkage bolts of 20, 32 or 40 mm diameter are passed through cored holes in both diaphragms and shear key. The linkage bolts are detailed to be capable of ductile behaviour in tension - either by using a rolled rather than a cut thread at each end, or by machining a length of flat on one side to reduce the area to less than that of the thread root. Nuts and rubber packers are used on each end of the linkage bolt (see Figure 11). Provision is made for linkage bolts to be withdrawn for inspection or replacement.

Linkage Slabs - As a means of eliminating deck joints and of providing seismic continuity both longitudinally and transversely for simply supported spans, up to 3 or 4 adjacent spans are interconnected with a slab capable of tolerating differential vertical movements in adjacent span ends. Longitudinal

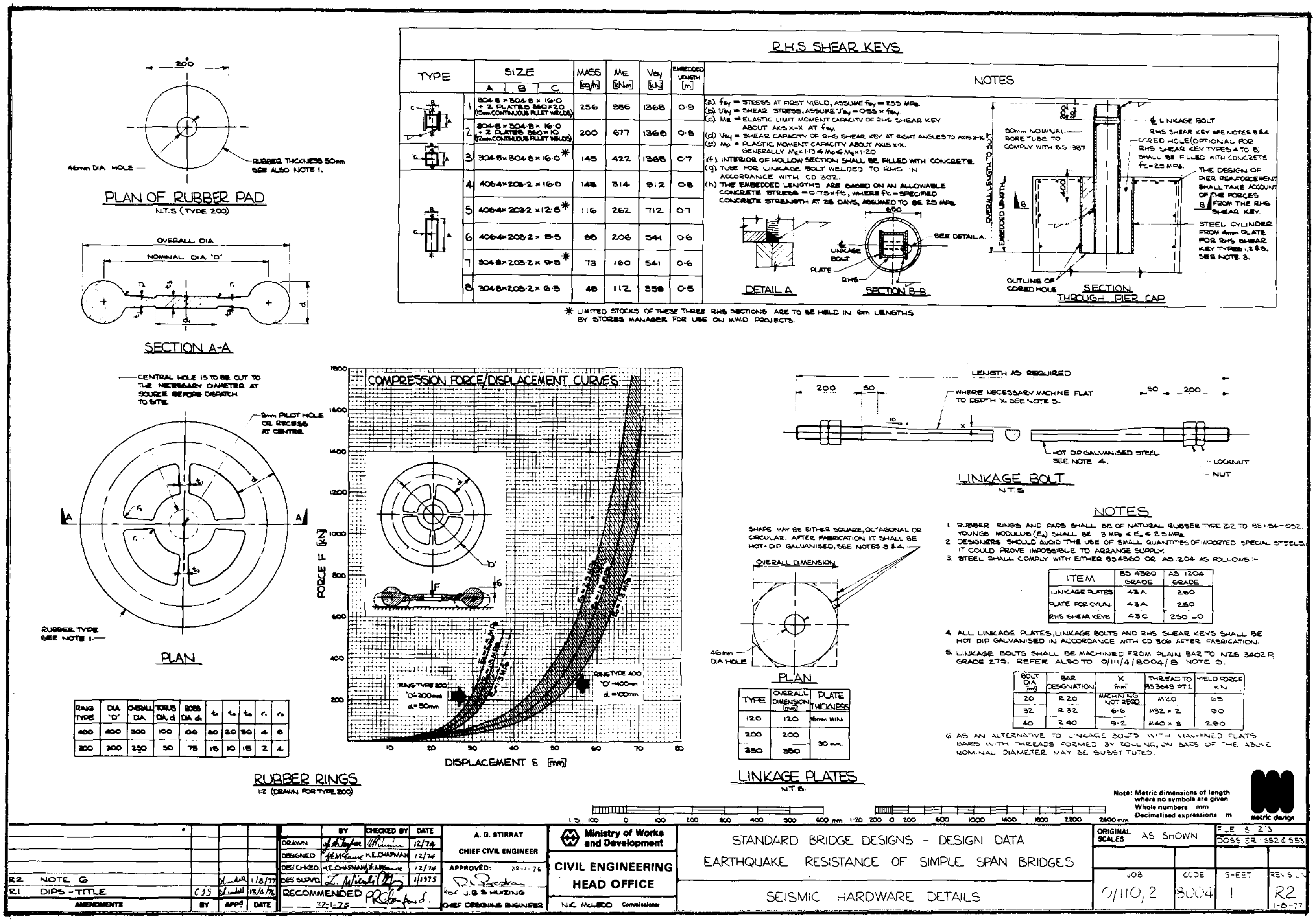


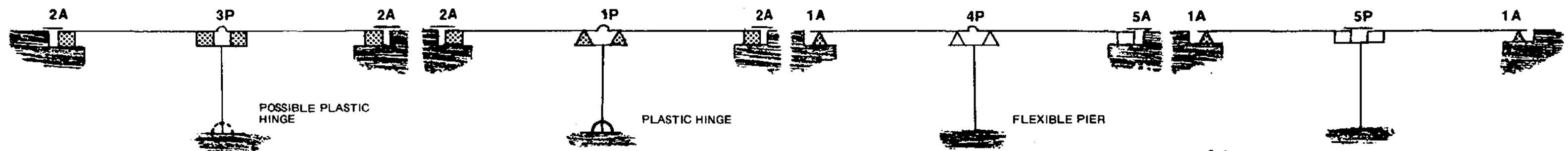
FIGURE 11 : DETAILS OF SHEAR KEYS, RUBBER BUFFERS AND LINKAGE BOLTS

ONE SPAN BRIDGES

1.1 SEISMIC RESTRAINT VIA ELASTOMERIC BEARINGS, IN LONGITUDINAL AND TRANSVERSE DIRECTIONS AT BOTH ABUTMENTS. MOVEMENT JOINTS AT BOTH ABUTMENTS.

1.2 RIGID LONGITUDINAL SEISMIC RESTRAINT AT L.H. ABUTMENT. TRANSVERSE RESTRAINT AT BOTH ABUTMENTS, VIA SHEAR KEYS. MOVEMENT JOINT AT R.H. ABUTMENT.

TWO SPAN BRIDGES



2.1 SEISMIC RESTRAINT VIA ELASTOMERIC BEARINGS, IN LONGITUDINAL AND TRANSVERSE DIRECTIONS AT ALL SUPPORTS. MOVEMENT JOINTS AT BOTH ABUTMENTS. PIER STIFF IN BOTH DIRECTIONS.

2.2 LONGITUDINAL SEISMIC RESTRAINT AT PIER, WHICH IS DESIGNED TO HINGE UNDER SEVERE EARTHQUAKE. RIGID TRANSVERSE RESTRAINT AT ALL SUPPORTS. PIER STIFF TRANSVERSELY. MOVEMENT JOINTS AT BOTH ABUTMENTS.

2.3 RIGID LONGITUDINAL SEISMIC RESTRAINT AT L.H. ABUTMENT. RIGID TRANSVERSE RESTRAINT AT ALL SUPPORTS. PIER STIFF TRANSVERSELY, BUT FLEXIBLE LONGITUDINALLY. MOVEMENT JOINT AT R.H. ABUTMENT.

2.4 RIGID LONGITUDINAL SEISMIC RESTRAINT AT BOTH ABUTMENTS. RIGID TRANSVERSE RESTRAINT AT ALL SUPPORTS. MOVEMENT JOINT AT PIER. PIER STIFF TRANSVERSELY

NOTES

1. SCHEMATIC DETAILS OF THE VARIOUS NUMBERED SUPPORT TYPES ARE INCLUDED WITH THE DRAWINGS OF EACH STANDARD BEAM TYPE.
2. ALTHOUGH BEAMS WILL NORMALLY BE SUPPORTED ON ELASTOMERIC BEARINGS, THE USE OF SLIDING BEARINGS MAY OFFER ADVANTAGES IN SOME CASES (e.g. TYPES 5A, 5P, 6P)
3. THE EXAMPLES SHOWN ARE ONLY A SELECTION OF POSSIBILITIES. DIFFERENT COMBINATIONS WILL BE APPROPRIATE IN OTHER CASES.
4. PIERS SHOULD BE DETAILED TO HAVE REASONABLE DUCTILITY, EVEN IF THEY DO NOT FORM PART OF THE PRIMARY SEISMIC RESTRAINT.
5. FOUNDATIONS ARE ASSUMED TO HAVE ADEQUATE STRENGTH TO ENSURE PLASTIC HINGES FORM IN THE PIERS. THIS MAY NOT BE POSSIBLE IN SOME CASES, AND PLASTIC HINGING OF PILES, OR PILE PULLOUT MAY BE THE LIMITING FACTOR. THE FORM OF EACH STRUCTURE MUST BE CONSIDERED INDIVIDUALLY TO PRODUCE THE BEST SOLUTION IN EACH CASE.
6. FOR MULTISPAN BRIDGES, SPANS LINKED IN THREES ARE CONSIDERED SUITABLE GENERALLY. THIS NUMBER MAY BE VARIED PROVIDED PIER AND BEARING DESIGN ALLOWS FOR SUPERSTRUCTURE LENGTH CHANGES.
7. "RIGID" RESTRAINT WILL GENERALLY BE VIA SHEAR KEYS AND/OR LINKAGE BOLTS, BUT SOME RUBBER MAY BE INTRODUCED TO PROVIDE FLEXIBILITY.
8. USE OF DUCTILE FORCE LIMITING DEVICES IS NOT SHOWN IN ANY OF THESE EXAMPLES
9. "RIGID", "STIFF" AND "FLEXIBLE" ARE ONLY QUALITATIVE INDICATIONS OF THE RELATIVE HORIZONTAL STIFFNESS OF THE CONNECTION OR MEMBER. IN PARTICULAR, ABUTMENTS ARE ASSUMED TO BE "STIFF".
10. THESE EARTHQUAKE RESISTING SYSTEMS SHOW SOME POSSIBLE WAYS OF COMPLYING WITH THE SEISMIC PROVISIONS OF THE HIGHWAY BRIDGE DESIGN BRIEF, JULY 1973. IT IS INTENDED THAT THESE DRAWINGS WILL BE REVISED AND THAT A FULLER EXPLANATION WILL BE GIVEN IN A NEW PUBLICATION, EARTHQUAKE RESISTANT DESIGN OF BRIDGES. THIS WILL BE PUBLISHED IN CONJUNCTION WITH A REVISED EDITION OF THE HIGHWAY BRIDGE DESIGN BRIEF.

SYMBOLS

"RIGID" LONGITUDINAL CONNECTION TO SUPPORT.

BEARING WHICH ACCOMMODATES LONGITUDINAL MOVEMENT.

SPANS RIGIDLY LINKED, PREFERABLY BY IN-SITU LINKAGE SLAB.

EXPANSION JOINT.

SHADED — SYMBOLS DENOTE SUPPORT PROVIDES SIGNIFICANT LONGITUDINAL SEISMIC RESTRAINT.

OPEN — SYMBOLS DENOTE SUPPORT PROVIDES ONLY MINOR LONGITUDINAL SEISMIC RESTRAINT

R1. DIPS-Note 10-Title.	PRS	18-7-77
AMENDMENTS	BY	APPD. DATE

DES'N — DATE 12/74 — A. G. STIRRAT, CHIEF CIVIL ENGINEER — J. B. S. HUIZING, CHIEF DESIGNING ENGINEER — RECOMMENDED 16-12-74 — APPROVED 16/12/74

Ministry of Works and Development — CIVIL ENGINEERING HEAD OFFICE — N.C. McLeod, Commissioner

STANDARD BRIDGE DESIGNS - DESIGN DATA
EARTHQUAKE RESISTANCE
EXAMPLES OF STRUCTURAL FORM.
1 AND 2 SPAN BRIDGES

ORIGINAL SCALES	N.T.S.	FILE	
JOB	CODE	SHEET	REVN
0/110/1	8004	1	R1 18-7-77

FIGURE 12a : EXAMPLES OF STRUCTURAL FORM - SEE FIGURES 13a AND b FOR DETAILS USED

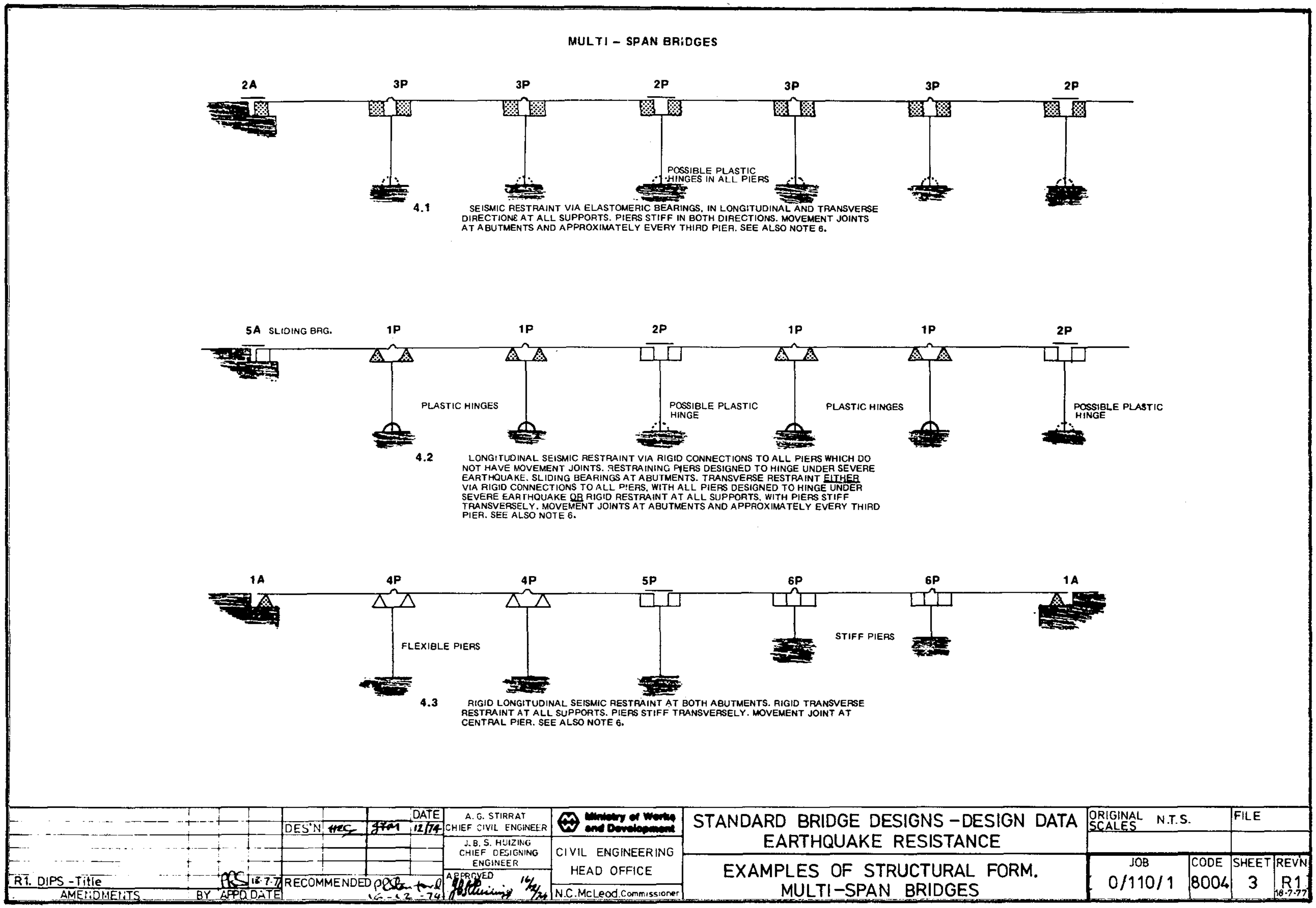


FIGURE 12b : EXAMPLES OF STRUCTURAL FORM - SEE FIGURES 13a and b FOR DETAILS USED

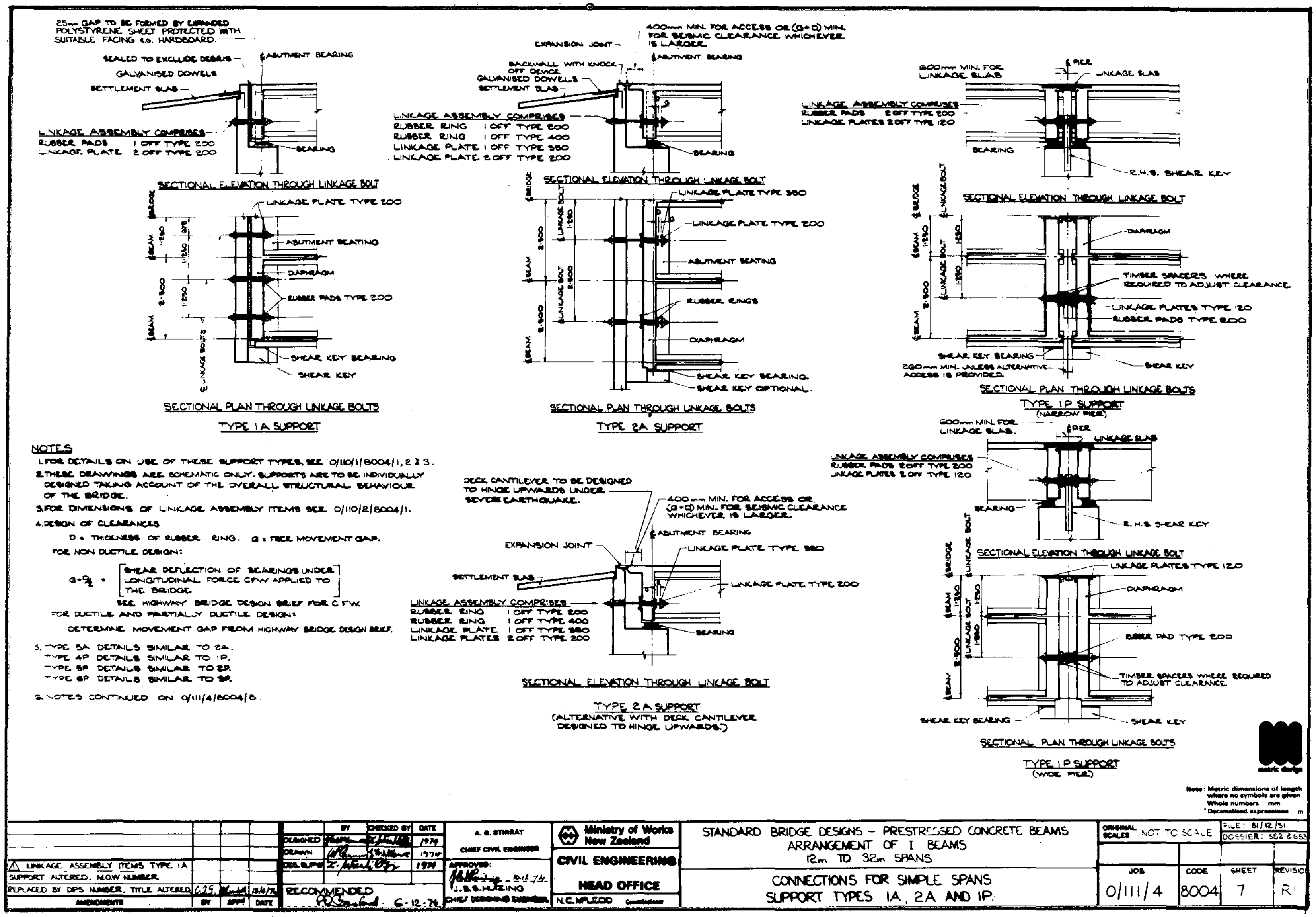


FIGURE 13a : DETAILS OF CONNECTIONS AT PIERS AND ABUTMENTS

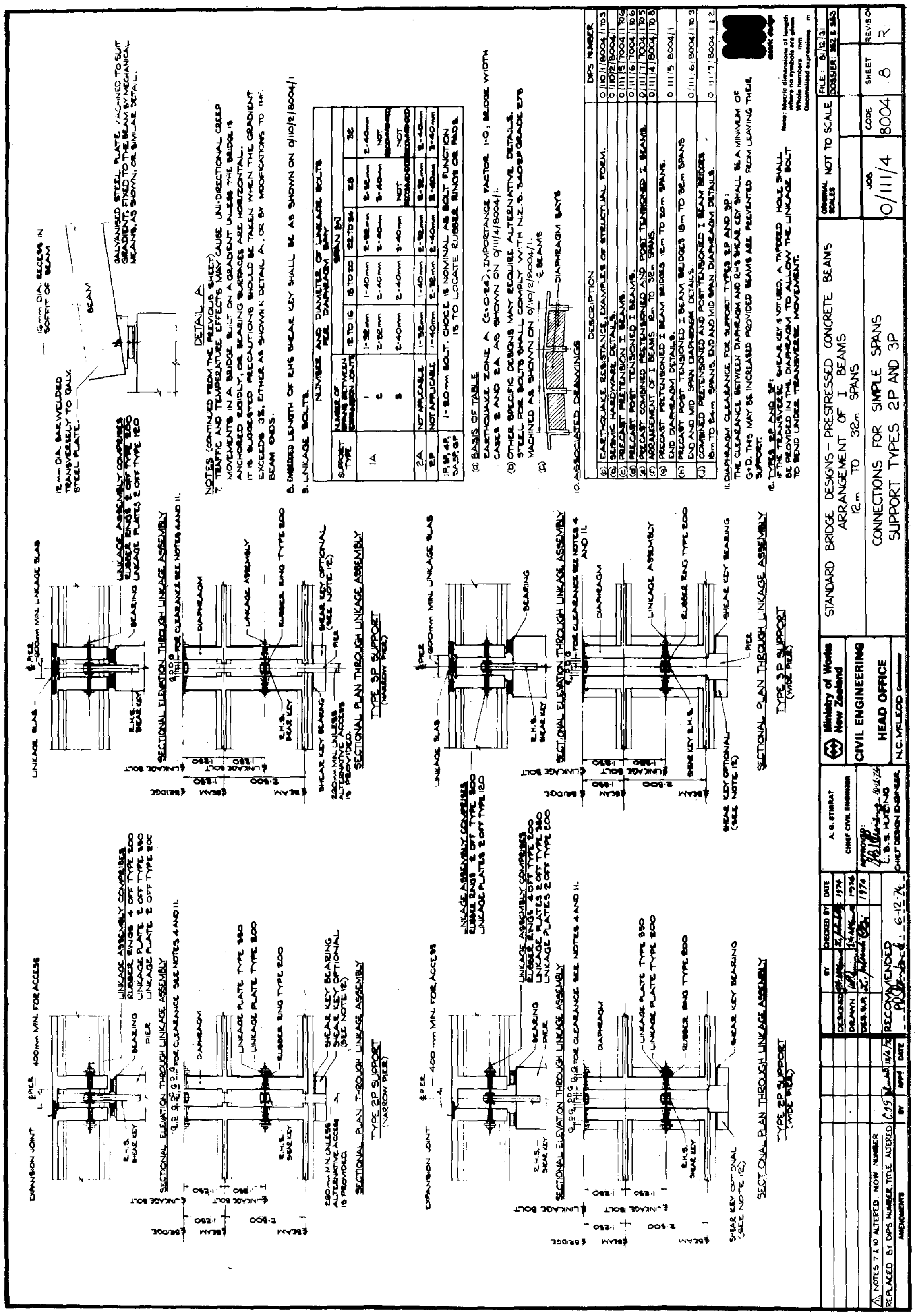


FIGURE 13b : DETAILS OF CONNECTIONS AT PIERS AND ABUTMENTS reinforcement is crossed to form a hinge and a transverse saw cut is used to induce rotation at the desired location (see Figure 13).

'Fuse' Slabs and Backwalls - At points of relative movement in the deck slab, or where the deck meets the abutment backwall, it is not justifiable to provide movement joints which would fully accommodate likely relative displacements under severe earthquake. Deck joints are therefore generally provided to accommodate temperature and shortening displacements and minor shaking, by allowing for 1.5 times the relative elastic deflection of adjacent parts calculated under the minimum specified seismic loading, including out-of-phase considerations. Separation of main structural elements must be at least six times the calculated displacement, to take account of ductile displacement beyond the elastic limit. This can usually be provided quite cheaply. Deck or abutment backwall members are shaped and detailed so that in the event of major shaking an easily replaceable piece of the structure will fail (see Figures 13a and 13b). Such areas could also be quickly bridged for temporary use.

Rubber Buffers (rubber rings) - It is normal policy to fit rubber buffers between main members at points where severe earthquakes may cause impact. The buffers are of circular doughnut shape with a compression stiffness which increases with increasing compression along the concentric axis. This is considered to be a more desirable characteristic than would be provided by a rectangular rubber block, which is generally very stiff in compression, (see Figures 11 and 13a and b).

Bearings - Normal types of bridge bearing are used but provision for jacking is always made, to facilitate bearing replacement if necessary. Elastomeric bearings are designed to deflect with up to 100% shear strain under the specified design seismic loading. Locating dowels are fitted top and bottom to inhibit lateral bearing slip if the structural form is such that appreciable unloading of the bearings is likely. Modification of normal elastomeric bearings by insertion of a cylindrical lead plug for improved energy dissipating characteristics is discussed by Blakeley [5].

Holding-down Devices - With designs intended to allow considerable shear movement in bearings under seismic conditions, it is felt that provision of holding-down devices is likely to adversely affect structural behaviour unless elaborate details for accommodating the movement are used. Because spans are always linked when necessary to prevent loss of support, holding-down devices are currently only used in cases where the reaction due to the specified horizontal seismic load opposes and exceeds 50% of the dead load reaction (eg at the end of cantilever structures). They are arbitrarily designed to withstand at yield stress at least 10% of the dead load reaction which would be exerted if the span were simply supported.

Confining Reinforcement - Reinforced concrete members are detailed to be capable of post-yield flexure wherever it is considered that this is likely to occur. Such confinement is of spiral or closely spaced welded circular hoops for circular members or of closely spaced rectangular hoops for rectangular sections.

PROBLEMS ENCOUNTERED IN IMPLEMENTATION OF PRESENT DESIGN APPROACH

The design methods discussed in this paper differ from previously used methods in three major areas:

(a) The concept of the 'capacity design' approach intended to produce a structure with members proportioned with suitable relative strengths so that the structure will behave as intended.

(b) The introduction of ductility as an aspect requiring detailed consideration.

(c) The need for designers in all cases to identify how a structure will respond dynamically to earthquake motions and to use judgement in determining the optimum structural form to satisfy the design requirements.

Introduction of 'capacity design' principles is relatively straightforward for bridge structures (as compared to building frames). The commonly familiar concept of strength design is used, and provided the designer is presented with recommended strength margins and overstrength values, application is not difficult. Design aids for pier design (axial load/moment interaction curves) are essential for efficient design office procedures. These should preferably be presented in two sets - one with specified capacity reduction factors included and one without. The charts should also be drawn to be independent of material strengths, to allow the designer to take account of variations as required. It is possible to produce the conventional column design charts in a form which is subject only to secondary effects of material strengths. Production of such charts is at present in hand in New Zealand.

Introduction of ductility for consideration in design procedures poses difficulties. The concept is often familiar to designers but because so many variables influence a structure's ductile performance it is a problem for designers to appreciate and evaluate relative effects. For this reason design aids are essential before worthwhile progress can be made in implementing such a design procedure for general use. Design aids [11] were introduced in New Zealand in 1973 to clarify methods and to find out whether problems were likely to exist with structures not being sufficiently ductile. At that time insufficient research had been completed to allow accurate determination of limits for parameters controlling the design charts. It was known that the charts led to conservative results, but their use and the accompanying explanatory notes resulted in standard application of the design method throughout the country. Subsequent research results could be judged against a standard known approach. A revision of the charts is now needed to take account of recent test results, to improve the accuracy of the predicted structural performance. This would be a major undertaking and is unlikely to proceed until adequate pier tests have been completed. Such a test programme is in hand at University of Canterbury at present.

Of the three areas where changes of approach have been introduced, (c) above is the most difficult to implement. It depends for success on individual acquisition of a feeling for structural dynamic behaviour under earthquake conditions. This needs to be coupled with practical experience of designing structures to behave as well as possible within the prevailing design constraints. While design examples can help, the 'feel' for behaviour takes time to acquire in a production design office situation. Knowledge and understanding of background material such as ground motion characteristics etc are also desirable for good design application. We have not yet found a way of producing in code form a set of design requirements which relieves designers of the need to use a great deal of judgement in their designs. It is anticipated that in the area where this is most needed - in the 'non-ductile' class of structure - developments such as mechanical energy dissipators will allow more tolerance to be built into the structures.

CONCLUSIONS

Development of the present design approach for earthquake resistance of bridges in New Zealand has been an evolutionary process, which is still under way. The ultimate goal is to establish stable design methods whereby the bridge engineer can readily design bridges which are seismically satisfactory and in which investment for earthquake resistance is not too great, keeping in mind the relative priorities between the primary function of the bridge and its other secondary required attributes, of which earthquake resistance would be one. Bridge design for traffic loading is a specialist subject, and unless the majority of structures are to be designed by committees, it is important that seismic design methods, while being technically soundly based, are reasonably easy to apply. Design aids form an important step towards the goal by removing the complex detail associated with background material.

The New Zealand National Society for Earthquake Engineering has assembled a bridge design group with the object of independently reviewing all aspects of the present design approach, which has been developed primarily by the New Zealand Ministry of Works and Development, as the major highway bridge specifying authority. Improvements and alternative methods will be recommended where appropriate. It is to be hoped that when the group's work is completed, a comprehensive publication of design aids and design examples can be compiled. This should lead to improvement of earthquake-resistant standards of the small and medium sized structures, which form the bulk of the bridge-building programme. This appears to be the area where effort should be directed to gain most benefit, since larger more significant structures are likely to warrant specialist involvement for their earthquake resistant design.

ACKNOWLEDGEMENTS

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A EUROPEAN VIEW OF THE EARTHQUAKE RESISTANT DESIGN OF BRIDGES

by

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ABSTRACT

Although bridge engineering in Europe is well advanced in many aspects, European codes contain very little information on earthquake-resistant design of bridges. However a survey of bridge damage after recent earthquakes, namely 1976's Friuli, Italy, and 1977's Vrancea, Romania, earthquakes clearly indicates that bridge earthquake-resistant design presents difficult problems in several areas, whose solving requires a coordinated research effort. Codes must consequently be improved, taking advantage of the results obtained in analytical and experimental research studies.

Information on European contributions for the study of soil dynamics, foundation behaviour, soil-structure interaction, behaviour of reinforced and prestressed concrete members under repeated loading, dynamics of seismic isolating systems and dynamic analysis of large bridges is summarized. Comments on the criteria to be followed in code implementation to overcome the actual large gap between available information and code provisions are presented. Needs and priorities for future research in order to improve current bridge earthquake-resistant design practice are briefly reviewed, especially as regards structural concepts, soil dynamics, dynamic analysis and design of structural members.

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INTRODUCTION

Bridge engineering has old and solid traditions in Europe, being a subject of utmost interest to designers and builders for many years. Structural concepts, design criteria, construction and maintenance techniques are dealt with in considerable detail in most European countries.

The progress of bridge engineering has been closely connected with the activity of scientific and technical associations, among which it is imperative to stress the important role of IABSE - International Association of Bridge and Structural Engineering, with headquarters in Zurich. IABSE celebrates this year its 50th anniversary with a colloquium dedicated to general questions of bridge engineering that will provide an excellent opportunity to exchange information in this field.

In spite of this situation, which involves plenty of experience in many aspects of bridge design and construction regulations, European codes contain very little information on earthquake resistant design of bridges. In fact, a survey of West European codes {1} shows a marked enphasis on buildings; some codes (e.g. the Turkish code {2}) explicitly exclude prescriptions about bridges; others, like the Spanish code {3} and the Portuguese code {4} give general rules that can be applied to bridges but that are clearly meant for buildings; very few like the Yugoslavian {5}, contain special provisions for bridges, but even so as an extension of much more developped provisions dedicated to buildings.

Several reasons explain this rather peculiar situation: the moderate seismicity of most West European countries; the specific characteristics of bridge structures, that are necessarily designed for actions similar to the seismic ones (e.g. wind and braking loads); the high degree of specialization involved in bridge design, currently superior to the one involved in building design ; the high priority given by bodies in charge of preparing codes on building earthquake - resistant design since social and economic consequences of earthquake damage in buildings have proved much more serious than the ones resulting from bridge damage.

Reports of damage due to recent earthquakes show however that this feeling of relative security as to bridge aseismic behaviour currently shared by designers, builders and people in charge of bridge operation and maintenance is not quite sound. A survey of bridge damage after earthquakes clearly indicates that bridge earthquake - resistant design presents difficult problems in several areas, whose solving requires a coordinated research effort. Codes must consequently be improved taking advantage of the results obtained in analytical and experimental studies.

The following paragraphs of this report are aimed at stressing and justifying these assertions, on the basis of European information and experience.

EARTHQUAKE DAMAGE IN BRIDGES

A survey of bridge damage caused by recent European earthquakes{6,7,8} brings out scarce yet interesting information on the topic. In fact, most of the reports do not mention bridges. Some just mention that bridges were not damaged. The two following reports deserve serious attention:

Sandi et al {9} report about the Romania 4 March 1977 earthquake, of magnitude M = 7.2 (Richter), with epicenter in the Vrancea region. According to official data 1570 victims have been identified (some 90% of them in Bucharest, at a distance of about 100km from the mainshock epicenter). More than 11 300 persons have been injured. 32 900 dwellings have collapsed or been badly damaged and 35 000 families have lost their shelter. Many schools and hospitals were damaged. A large number of industrial enterprises have also been affected with important production losses.

In contrast with this somber picture, Sandi reports that "bridges have not been seriously affected. Only limited damage of support zones and slight displacements of a few piers, abutments, etc. have been put to evidence in some cases"...

A rather different situation is reported by CNEN - ENEL Commission on Seismic Problems Associated with the installation of Nuclear Plants after the Friuli, Italy, earthquake of May 1976 {8} . According to this report the MM intensity reached was IX and the magnitude probably between 6 and 6.5. Epicenters were located at a densely propulated zone near Udine, the focus depth being smaller than 30km.

In this case all types of constructions in the area were strongly damaged, including bridges. The report states that investigations on the behaviour of highway structures, which were concentrated on viaducts "clearly proved that the behaviour of the piers founded on piles was influenced by the characteristics of the foundation soil, and behaviour of the decks was conditioned by the type of restraint which, being of the support type, allowed a remarkable displacement of the decks. It is worth mentioning that remarkable displacements of the deck beams were observed also in a viaduct designed according to the Italian code for seismic areas of Category 2; this viaduct was equipped with antiseismic blocks that were completely destroyed". The report describes in some detail damages in five viaducts, three being complete and two under construction, as follows:

Viaduct over the Juliense Country Road - three spans with a concrete light-slab deck. Total length about 40m. Abutments and two central piers consisting of columns founded on piles. The deck suffered a residual transverse displacement of about 60m.

Roggia Alta Viaduct - three spans with a light continuous concrete slab deck that at the time of the earthquake had been poured only on one lane. Abutment and central piers consisting of columns founded on piles. The deck slipped longitudinally and transversely by about 50cm.

Viaduct over the Tagliamento River - 28 spans of 45m each, with abutments and piers founded on piles. Deck consisting of prestressed-concrete beams and slab poured in place. The viaduct was designed according to the code for Category 2 seismic zone. Severe damage occurred to the beam supports and to the antiseismic blocks, which were practically destroyed owing to the repeated impacts of the beams.

Two viaducts that were under construction were also severely damaged. In one of them only the central piers and parts of the abutments had been built. The absence of the deck caused the piers to suffer more severe damage than similar piers of the completed viaducts. The other viaduct under construction had four spans, with a deck similar to the Tagliamento River Viaduct. The slab had not yet been poured. The damage was also more severe than in similar completed viaducts, ranging from cracks in the abutments and piers to large displacements of the decks, which in some cases collapsed.

It is interesting to point out the similarities not only between the bridges observed in Friuli and in other recent strong earthquakes, namely Tokachi-Oki 1968 {10} and particularly San Fernando, 1971 {11} but also the similarities between Vrancea and Friuli, although the amount of damage in bridges in these cases was completely different.

This survey clearly indicates that several important problems of bridge earthquake resistant design have not yet been satisfactorily solved. As a result of this most of the present codes must be considerably implemented in order to take into account the real nature of the seismic actions, the dynamic behaviour of soils, an adequate modelling of foundation and superstructure (including supports) behaviour, suitable procedures for the dynamic analysis considering soil-foundation interaction and adequate design criteria for structural members.

This implementations requires a great deal of research. The following section presents information on European contributions for the study of such problems.

EXPERIMENTAL AND ANALYTICAL RESEARCH

A survey of the papers presented to the 5th and to the 6th European Conferences on Earthquake Engineering, held in Istanbul in 1975, and in Dubrovnik in 1978 shows that very few reports deal specifically with bridge earthquake behaviour. There is nevertheless an important amount of research reported on problems that are closely correlated with the subject, such as soil dynamics, behaviour of different types of foundations, namely piles, footings and abutments under seismic actions, behaviour of reinforced and prestressed concrete members under repeated loading and dynamics of seismic isolating systems.

The results obtained in such studies provide a comprehensive view of different aspects that have to be considered in overall seismic bridge studies. Several very interesting reports of this kind using analytical and experimental techniques have been presented to the Dubrovnik Conference {12,13,14,15}. The scope of the present paper justifies a special reference to reports {14} and {15} .

Dynamic Analysis of a Multi-Span Bridge

Parvu, Sandi, Stancu and Teodorescu {14} report analytical studies carried out in connection with the aseismic design of a long two-way railway and four-way highway multi-span bridge over the Danube. Fig. 1 presents a schematic elevation of the bridge. The railway and the highway are supported by the same floor in the zone of the main spans, while they are supported by different floors and piers in the zone of the 50m spans.

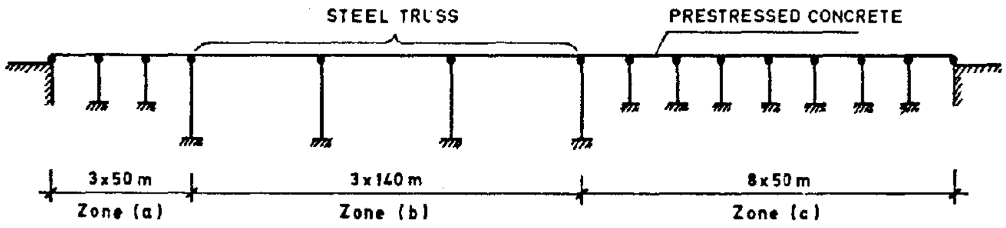

Fig. 1 - Schematic elevation of Danube bridge {14}

The bridge shall be located in a seismic area to which an intensity VII (MSK-64) is assigned by the official zoning map. If has been agreed to consider for the aseismic design of the bridge a methodology that is similar to that used for buildings and industrial structures supplemented by some specific additional topics (essentially: seismic ground pressure, seismic water pressure, non-synchronous disturbances applied at different piers).

The analysis has been based on the assumption of linear dynamic behaviour. The features of the structure permitted to split the spatial problem of dynamic behaviour into two independent problems, corresponding to plane and anti-plane oscillations respectively. The coupling of the two problems that might occur due to accidental non-symmetry of stiffnesses of piers, of ground conditions, of vehicle loading has been estimated to be sufficiently weak not to be considered in analysis.

The length and complexity of the structure permits to qualify it as a large size structure, i.e. a structure for which it is not possible to carry out computations in a single step. So several models have been set up for various computational purposes: for example, the space steel truss adopted for the main spans has been considered as a single, continuous beam for the analysis of oscillations of the systems as a whole, but it has been considered as a space frame in order to determine stresses in its members.

The structure as a whole has been divided into three parts, represented respectively in fig. 1 a,b,c. The lateral zones (a) and (c) of fig. 1 has been considered for plane oscillations as one-DOF systems, for which horizontal oscillations have been investigated. An estimate of the influence of axial flexibility of the floor on fundamental period has been made (the correction was not negligible). The anti-plane oscillations of lateral zones have been analyzed for models corresponding to lumped masses located at the supports offered by piers.

The main span zone (b) of fig. 1 has been considered as a lumped-mass system with 25 masses. Each mass corresponded to two DOF for each kind of oscillations. The DOF considered corresponded to longitudinal and vertical translations for plane oscillations. They corresponded to transverse translation and to rotations with respect to a longitudinal axis in case of anti-plane oscillations respectively. So, 50 dynamic DOF have been considered for each kind of oscillations (to deduce the flexibility matrices of the structure, a static model with 41 nodes and six DOF per free node has been adopted).

The seismic disturbance has been represented, formally, by means of conventional static parameters (conventional inertia forces, accelerations, displacements, etc. induced by the ground motion). Nevertheless, the sense of factors determining the conventional seismic effects has been related to a stochastic model of ground motion.

From the results of the analysis performed according to the above referred assumptions, the authors derive a set of conclusions from which it seems pertinent to stress the following three:

1. Multi-span bridge are complex structures. The analysis of seismic effects on such structures requires a high amount of computations. Even a separate consideration of a three-span part like the main span zone has led to the necessity of considering 40 natural modes (20 modes for each subspace) in order to obtain a sufficiently comprehensive image of seismic effects (e.g., in order to use in analysis at least 2 modes for each of the piers). A lower number of normal modes leads to an acceptable accuracy in evaluating bending moments on piers, but not in evaluating shear forces, which are significant especially for the columns supporting the piers.

2. The considerable length of the structure makes significant the axial deformation of the floor even for some of the lower modes of plane oscillations. The conventional longitudinal horizontal accelerations of the masses of the floor are therefore definitely non-uniform.

3. The fact that differences between natural period for some neighbouring modes are very small (note that these differences vanish when the couples V-VI and XIV-XVI are considered for plane oscillations) implies the need for a special attention when combining the effects corresponding to different modes. The quadratic rule is obviously incorrect for couples of modes with equal periods or very close ones. In case of different modes of plane oscillations with equal periods it becomes rigourously correct to adopt a linear super-position rule.

Seismic studies for the International Guadiana Bridge

Carvalho, Ravara and Duarte {15} report about the analytical and experimental studies performed for the aseismic design of the International River Guadiana Bridge, connecting the villages of Vila Real de St. António in Portugal and Ayamonte in Spain. The main items of the report are the following:

General Information- One of the proposed structural solutions for the bridge is a set of several reinforced concrete twin framed piers spaced 130m center to center with two prestressed cantilever decks spaning 30m in each direction. The connection of these various piers is made by simply supported decks with a span of 30m.

Cross section of the deck, both for the cantilever and the simply suppor ted zone, is formed by two caisson girders connected by a central slab.

The foundation of each pier is made, across alluvium layers with maxi - mum depth of 70m, by two sets of 14 vertical and inclined piles down to the be rock. Diameter of the piles is 1.20m and the concrete is to be enclosed by a 1.2cm thickness steel tube.

A sketch of this structural solutions is presented in fig. 2.

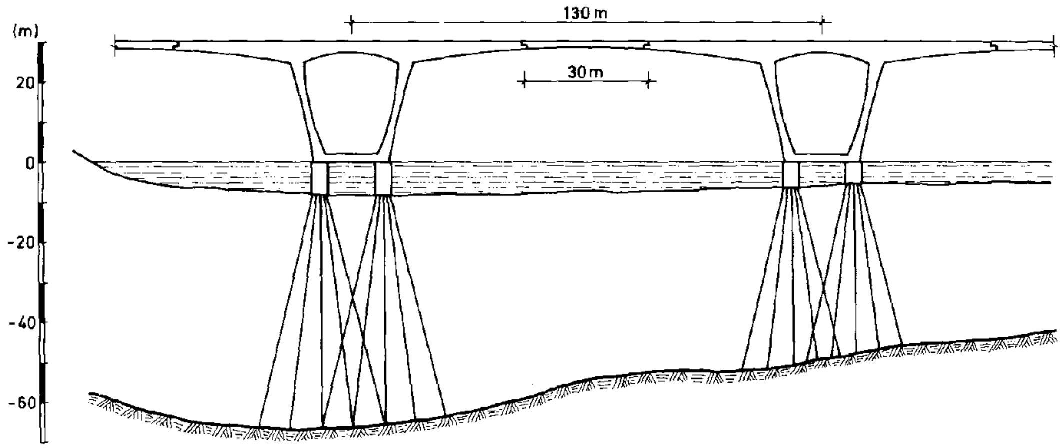

Fig. 2 - Schematic elevation of Guadiana bridge.

Soil characteristics - The soil profile along the bridge axis is presen- ted in fig. 3. Three main types of soil formations are present, in all cases belonging to the recent Quaternarian.

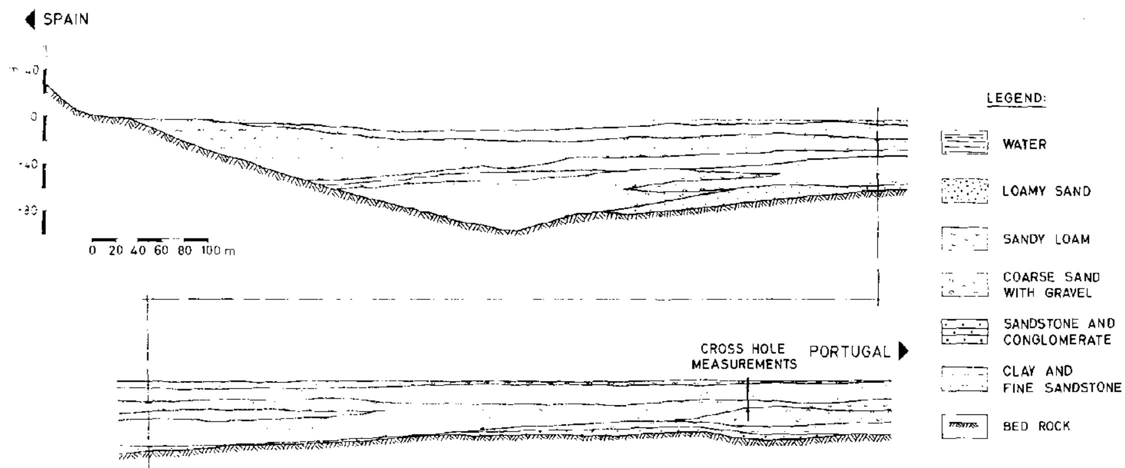

Fig. 3 - Geotechnical profile along the axis of the bridge

Cross hole measurements of longitudinal and shear wave velocities were performed in one of the drillings located in the portuguese bank as indicated in fig. 3. At the time of the measurements the water table at the site was lo cated approximately 1m below the surface. Results are presented in fig. 4.

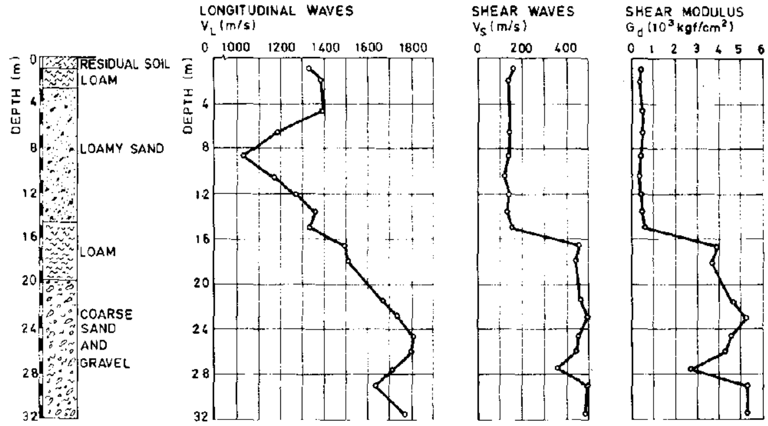

Fig. 4 - Results of cross hole measurements

Analytical Studies - Natural frequencies and mode shapes of the alluvium layers were computed using a plane strain finite element analysis. Results for the two lowest modes are shown in fig. 5. First natural frequency is 1.79Hz and the mode shape corresponds simply to a local vibration of the softer soil on the portuguese bank. Second natural frequency is 1.92Hz and the mode shape corresponds to a global vibration of the alluvium layers especially an the deeper zones under the river bed. Third mode presents the vibration in two zones with opposed ways at the surface and has a frequency of 1.97Hz.

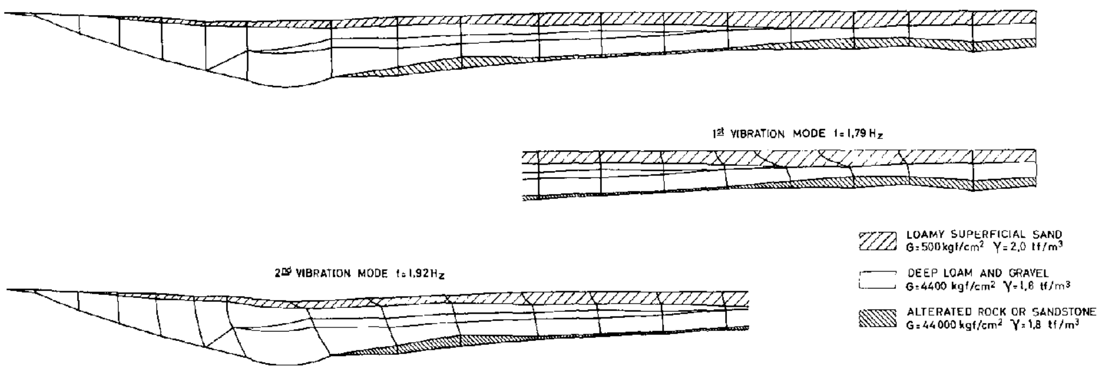

Fig. 5 - Finite element mesh. 1^{st} and 2^{nd} vibration modes

It must be emphasized that displacements and velocities measurements at the surface due to explosive detonations at different depths showed dominant frequencies between 3 and 5Hz. The apparent discrepancy between analytical and experimental results is perhaps due to the fact that while in-situ blast mainly excites longitudinal waves, vibration modes are more associated with shear wave propagation.

Taking this fact in consideration a very simple shear model was used to compute the alluvium frequency at its deepest zone using the same soil characteristics as those used in the finite element analysis. The first natural frequency obtained was 1.74Hz and the corresponding mode shape which is presented in fig. 6 shows good agreement with previous results.

Linear seismic behaviour of that soil profile was evaluated using a probabilistic approach {16} and defining the earthquake as a stochastic process with a suitable power spectral density. For an earthquake with magnitude M=7 and 60km of focal distance the corresponding power spectrum shape was computed {17} and is presented in fig. 6. It corresponds approximately to a maximum rock acceleration of 180 gal which is the maximum expected acceleration at the site for a 1 000 years return period {18}. Maximum displacements and curvatures in a vertical line for an earthquake with 30s duration and assuming a ζ = = 10% viscous damping are presented in fig. 6.

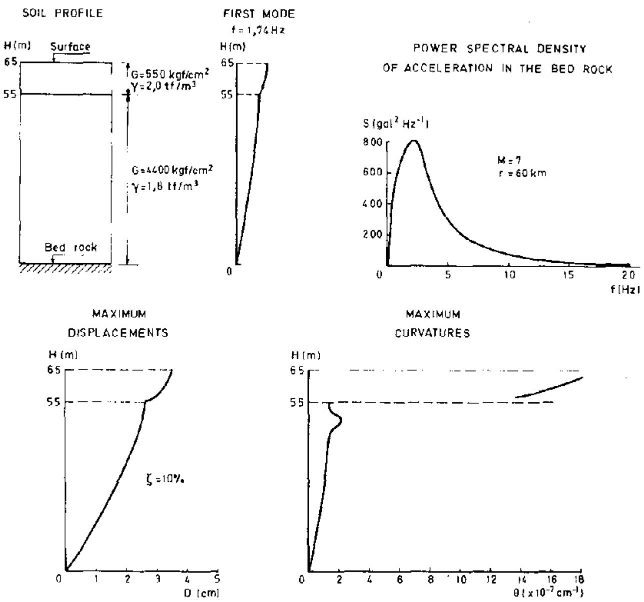

Fig. 6 - Seismic behaviour of a soil profile

Experimental Studies - A 1/100 perspex model of the structure and its pile foundation was constructed. An aspect of the model is presented in fig. 7. About fifty strain gages were bonded to the piles and the model was placed in a 2.0m x 0.7m x 0.7m steel box filled with a suitable material scaling the soil characteristics (fig. 8).

The choice of the filling material was made using as usual in dynamic testing the Cauchy similitude. Length, modulus of elasticity and specific mass scales were fixed by the relations between the perspex model and the real concrete structure and had the following values.

Length $L_P/L_M = 100$

Modulus of Elasticity $E_P/E_M = \dfrac{370\ 000\ \text{kgf/cm}^2}{30\ 000\ \text{kgf/cm}^2} = 12.33$

Specific Mass $\rho_P/\rho_M = \dfrac{2.5\ \text{tf/m}^3}{1.2\ \text{tf/m}^3} = 2.08$

The remaining parameters scales were then imposed by the Cauchy similitude relations.

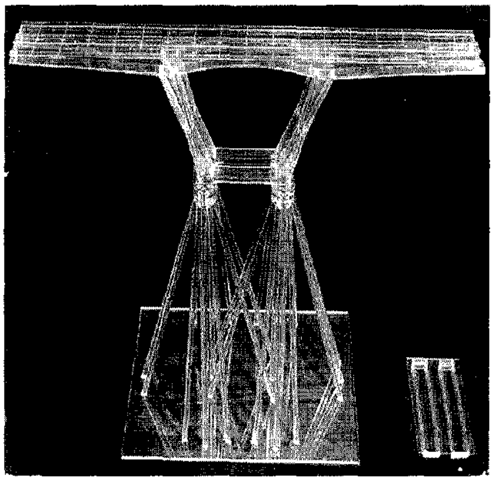

Fig. 7 - 1/100 perspex model

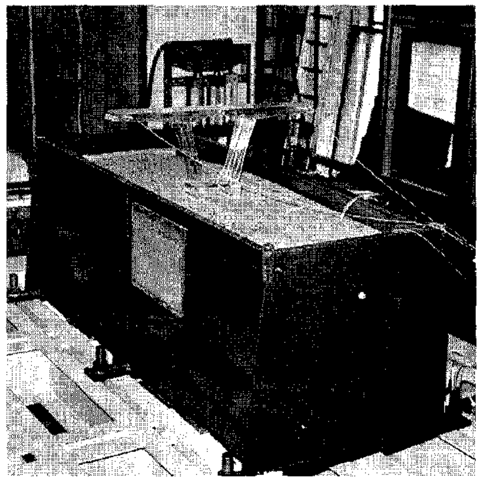

Fig. 8 - Experimental set-up on the shaking table

It was very difficult to find a material scaling simultaneously the specific mass and the shear modulus of the soil. So it was decided to consider shear wave velocity as the scaling parameter. As indicated above, characteristic values for the soil formations shear wave velocities are 150m/s and 450m/s respectively for the soft superficial and for the deeper and more consistent layers which correspond to model values of 62m/s and 185m/s. The simulation of these two types of soil was achieved using a mixture of sand and saw dust for the first and a very fine dry sand for the last. Velocities obtained were respectively 60m/s and 135m/s. In order to avoid the confining effects of the steel box ends two 2cm thick foam plates were placed between the filling material and those ends.

Dynamic tests were performed using the LNEC shaking table and began with the bridge model not yet installed. A two layer situation was created filling the lower 45cm of the box with fine sand and the upper 15cm with the sand and saw dust mixture. Three accelerometers were placed at 0,15 and 30cm depth inside the box and a fourth in the shaking table. A frequency sweep between 5 and 80Hz was made. This system didn't show a peaked transfer function but it was possible to detect maximum amplification between 47.5 and 55Hz with a value of approximately 3.5.

Filling the box only with 60cm of fine sand, a one layer situation was also tested revealing again a maximum amplification frequency around 50Hz. Performing a shock test it was possible to confirm this natural frequency as well as to evaluate through the vibration decay a $\zeta = 12\%$ viscous damping.

The perspex model with the deck oriented transversely to the direction of the shaking table movement was finally placed in the steel box filled with 65cm of sand as presented in fig. 8. Natural frequencies were evaluated pulling the deck of the model and suddenly releasing it. Three natural frequencies were thus obtained with the values of $f_1 = 42$Hz, $f_2 = 47$Hz and $f_3 = 58$Hz respectively for the three following modes shapes: Torsion about a vertical axis; horizontal translation along the deck axis and horizontal translation transversely to the deck. Vibration decay corresponded to a $\zeta = 5\%$ viscous damping. It may be noted that these values agreed very well with an analytical evaluation of the model frequencies assuming that the twin framed piers were built in at the pile caps. Values obtained this way were f_1 = 41Hz f_2 = = 44Hz and f_3 = 55Hz for the mode shapes as described above. Fourth mode determined analytically corresponds to the vertical vibration of the cantilevered parts of the deck and has a frequency of f_4 = 102Hz. Considering the frequency scale the real structure can be expected to present the three lowest natural frequencies in the 1 to 1.5Hz range.

A frequency sweep was again carried out measuring the acceleration at three levels:at the bottom of the box; at the surface and at the bridge deck. The acceleration amplification factors from the soil surface to the bridge deck fitted very well the theoretical transfer function of a system with a natural frequency of 58Hz and 5% viscous damping as can be seen in fig. 9. On the contrary the bottom to the surface amplification factors although presenting a maximum value of 3 for 47.5Hz did not fit well any theoretical transfer function. The slight decrease on the natural frequency of the soil as compared with the former test, is due to the increase from 60 to 65cm in the soil thickness.

As indicated before some piles of the foundation system were instrumented at two cross section 10 and 30cm below the surface of the soil each with three strain gages. Maximum strains measured in one of the piles during the test described above are presented in fig. 10. Location of the pile and of the strain gages appears in the figure. Furthermore it must be said that the frequency sweep was conducted for a shaking table constant acceleration amplitude of 0.14g. In fig. 10 two strain ordinate scales are presented. One for the directly measured strains in the model and the other for the expected prototype strains due to an acceleration amplitude of 0.18g in the bed rock (maximum expected at the site for a 1,000 years return period).

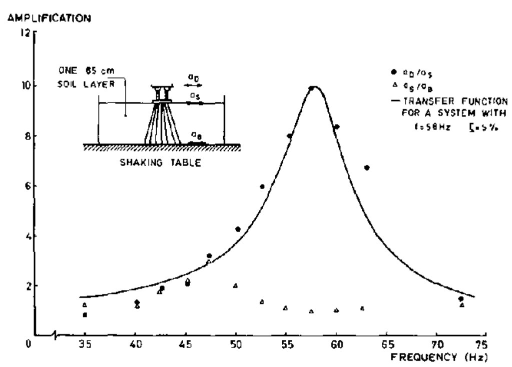

Fig. 9 - Results of a frequency sweep

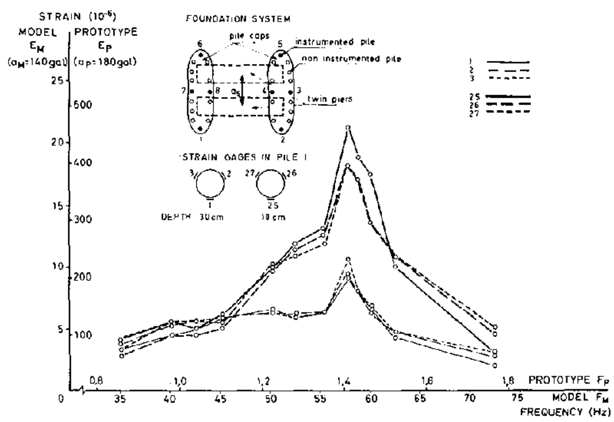

Fig. 10 - Strains in pile 1

Maximum strains are due to the ressonance of the superstructure and it can clearly be seen the influence of the cross section depth on the strain level. Actually the deeper cross section strains are not very much affected by the ressonance while the 10cm deep strain gages present a pronounced peak for the bridge natural frequency. Furthermore the observation of the strains time variation showed an in-phase situation implying that the main internal force acting on the pile is the axial force due to the bridge induced overturning moment acting on the piles' caps. This conclusion should be taken carefully since model test was performed for a one stiff layer situation which certainly doesn't impose great curvatures and consequently, great bending moments in the piles. For several-layered soil profiles and especially for the soft ones, that may be not the case and some bending moment in the piles is to be expected as can be seen in fig. 6.

Finally it must be emphasized that the maximum strain presented in fig . 10 for the prototype with a value of $\varepsilon = 460 \times 10^{6}$ should not be expected to develop in the event of an earthquake with a 180 gal peak acceleration. In fact, while for a system with 5% damping a 10 fold maximum amplification appears in its transfer function, for a real earthquake and the same damping the amplification for peak acceleration is only 2.6 {19}. So it can be expected that for an earthquake with 180 gal of maximum acceleration in the bed rock concrete stress in the piles have values approximately equal to 50 kgf/cm^2.

Final Remarks - The study presented in this paper has been the first attempt in LNEC to conduct a seismic study of a large bridge including its foundations and using simultaneously analytical and model techniques. Very few studies of this kind are published and therefore an effort in this field would be certainly fruitful. The main goal of the study was to achieve an insight in the pile soil interaction in the event of a high intensity earthquake. This goal was only partially fulfilled as the either analytical or experimental modelling of the soil was found to be very difficult, not to mention the incertitude associated with the in situ soil characterization. On the other hand, linear structural analysis of the superstructure is currently feasible using either of the two methods and the present studies results in this field clearly confirm this assertion.

As a final conclusion of the present study, it should be emphasized that, in spite of all the difficulties found along its development, analytical and experimental studies must be carried together in order to provide a better understanding of such a complex phenomenom as the pile soil interaction in deep foundations during an earthquake.

COMMENTS ON CODE IMPLEMENTATION

Although it must be recognized that an adequate solution of many of the problems involved in bridge aseismic design still requires a considerable research effort, there is certainly a large gap between the available information and its implementation in codes. The criteria of code implementation to overcome this gap should consider the following aspects:

Bridge earthquake - resistant design provisions should constitute a specific chapter in aseismic construction codes instead of being treated as an appendix of building regulations as it happens in many present codes.

It is a general tendency of codes to consider three types of approaches in seismic design: static methods using seismic coefficients, simplified dyna mic analysis and dynamic analysis. Codes should carefully typify bridges and indicate for each type the kind of approach to be used. Although bridge types differ considerably from one country to another it is felt that an effort of coordination between code commissions would be particularly worthwhile.

For each type of bridge structure, considering the particular features of superstructure and foundations, codes should point out clearly the proble ms to be dealt with in the design. Different levels of accuracy in soil chara cterization should be considered according to the design approach.

The seismic coefficient approach should be limited to very simple structures, when significant values of dynamic amplification are not to be expected. Simplified dynamic analysis should consider earth pressure in the abutments, foundation flexibility, and the dynamic properties of the superstructure-foun dation system as a whole. Finally, when sophisticated procedures of dynamic analysis are required, codes should clearly prescribe the criteria to be used in: the modelling of ground movements (namely as to the direction, intensity and correlation of the components of ground acceleration, velocities and displacements); the modelling of soil and foundation dynamic properties; the type of dynamic analysis (linear and/or nonlinear) to be carried out; the ductility requirements; the limit states to be considered in the design of the superstructure and of the foundations.

Codes should include guidelines for the use and the design criteria of me chanical energy absorbing devices, which should be adequately checked.

And finally codes should include provisions about seismic instrumentation of important bridges in a way similar to the current practice of many countries for high-rise buildings, as well as provisions about dynamic testing of bridges by means of ambient and forced vibration techniques, to check the design approa ches and the possible deterioration of the bridges.

NEEDS AND PRIORITIES FOR FUTURE RESEARCH

One of the main objectives of this Workshop is the examination of the needs and priorities of future research for improving current practice. The list of authors and papers present at the Workshop fully guarantee that objective, since the papers cover a large scope of important research topics. In the author's opinion some of the more important subjects deserving the attention of researchers can be summarized as follows:

Structural Concepts

A thourough survey of bridge performance during earthquakes should be car ried out in different countries, duly coordinated in order to improve the choi ce of the structural and foundation systems. It is important to bear in mind that structural concepts that can be sucessfully adopted in moderate seismici ty zones are definitely not suited for severe seismicity ones, despite the degree of sophistication used in the dynamic analysis.

It often happens that decisions that apparently are simple, such as the spacing of piers, the connections between piers, deck and abutments, the spacing of joints, etc, are determinant on the bridge overall behaviour.

The use of mechanical absorbers should be studied and implemented, since they are often an efficient and economic way of reducing the transmission of inertial forces from the deck to the piers and abutments.

Soil Dynamics

One of the major sources of uncertainties in the seismic design of bridges is the behaviour of the soil, since it affects the ground movements, the propagation of seismic waves, the flexibility and damping of the foundation , the dynamic earth pressures on the abutments and the overall response of the system. Adequate modelling of soil properties still requires a considerable research effort. The use of experimental facilities namely large size multi-directional shaking tables should be increased in the study of those problems. The small number and big cost of such facilities should encourage international cooperation in this field of research.

Dynamic Analysis

Nowadays it has become possible to perform the dynamic analysis of complex bridge systems, considering several simultaneous independent or correlated components of ground movement, synchronous or non-synchronous, and the influence of the soil near the foundations and abutments, for plane and spatial superstructures. In most cases linear analysis is carried out. Particular attention should be paid to subjects like the mathematical idealization of seismic actions, the damping of the soil and of the superstructure and the criteria for mode superposition specially for modes having very close frequencies. Also the importance of the non-linear behaviour of the soil and the structural members should be investigated. Non-linear analysis procedures or linearization techniques should be further studied. Seismic tests on bridge models, the dynamic observation of bridges under small amplitude vibration or after earthquakes are all very useful means of checking the dynamic analysis procedures and results.

Dynamic analysis of tipified bridge structures should be carried out in a systematic way in order to obtain general conclusions that could be included in codes allowing a seismic design based on the seismic coefficient approach or on a simplified dynamic analysis approach. The dynamic analysis of large and complex bridges provides results that in general are applicable only to the design of the particular bridge analized.

Design of Structural Members

Ductility requirements and design criteria for structural members, made of steel, reinforced concrete or prestressed concrete namely, have been a subject of study in the last years. It seems important to pursue these studies incorporating the information provided by similar studies performed on building structures. An important meeting on this subject, organized by the CEB shall take place in Rome, next May {20} .

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DEVELOPMENT OF HIGHWAY BRIDGE SEISMIC DESIGN CRITERIA FOR THE UNITED STATES

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INTRODUCTION

The 1971 San Fernando earthquake presented a major turning point in the development of seismic design criteria for bridges in the United States. Prior to 1971 the American Association of State Highway Transportation Officials (AASHTO) specifications for the seismic design of bridges were based in part on the lateral force requirements for buildings developed by the Structural Engineers Association of California.[1] In 1973 the California Department of Transportation[2] introduced new seismic design criteria for bridges that included the relationship of the site to active faults, the seismic response of the soils at the site and the dynamic response characteristics of the bridge. In 1975 AASHTO adopted Interim Specifications which were a slightly modified version of the 1973 CalTrans provisions and made them applicable to all regions of the United States. In addition to these code changes the 1971 San Fernando earthquake also stimulated research activity on the seismic problems related to bridges. By 1977 the Federal Highway Administration (FHWA) felt it appropriate that an assessment be made of the 1975 AASHTO Interim Specifications and in June 1977 FHWA funded a program developed by Applied Technology Council (ATC) to:

- Evaluate current criteria used for seismic design of highway bridges.
- Review recent seismic research findings for design potential and use in new specifications.
- Develop recommended new and improved seismic design criteria for highway bridges.
- Evaluate the impact of these criteria and modify them as appropriate.

The status of the work is discussed and the preliminary criteria are summarized in the following text. Discussions are presented of the following:

- Project Objectives and Organization
- Basic Concepts
- Applicability of Provisions to the United States
- Seismic Ground Motion Intensities
- Supporting Soil Effects
- Design Philosophy
- Preliminary Criteria
- Conclusions

PROJECT OBJECTIVES AND ORGANIZATION

The objectives of the work are to evaluate existing seismic design criteria and procedures for bridges and, using the latest research findings, develop comprehensive seismic design guidelines applicable to all regions of the United States with particular emphasis given to seismic risk. The guidelines will consider life safety, protection of property and preservation of essential functions.

It was considered essential in the development of nationally applicable design criteria that representative segments of the bridge design and construction profession be involved. To ensure representative input and adequate consideration of the many factors involved, a Project Engineering Panel (PEP) was assembled and composed of:

- Four AASHTO representatives--California, Idaho, New York and Oklahoma.
- Four private design firms--California, New York and Pennsylvania.
- Three university researchers--California, Illinois and New Jersey.
- Two FHWA representatives.
- An ATC Board member.
- Two ATC staff (Project Manager and Project Technical Director).

See Appendix A for a list of project participants.

The work is being conducted in four phases:

- Phase I
  - Select and organize the PEP.
  - Review 1975 AASHTO Interim Specifications and current specifications in other countries.
  - Review current research findings.

- Phase II
  - Review of current seismic design procedures for bridges.
  - Develop preliminary seismic design criteria including seismic risk maps and design spectra.
  - Select bridges for redesign using preliminary criteria.

- Phase III
  - Negotiate subcontracts with design firms to do redesigns and evaluation of the new criteria.
  - Implement bridge redesign studies.
  - Monitor studies.
  - Review bridge redesign reports.
  - Present proposed criteria to four regional AASHTO meetings.

- Phase IV
  - Assess and evaluate results of Phase III redesign studies.
  - Modify design criteria as appropriate.
  - PEP review of modified criteria.
  - Submittal of final document.

As the work progressed it became evident that a workshop covering seismic problems related to bridges would be of great benefit--thus the present workshop. Phase I has been completed and Phase II will be completed shortly.

BASIC CONCEPTS

Development of the design criteria has been predicated on certain basic concepts including the following:

- Hazard to life be minimized.
- Bridges may suffer damage but have low probability of collapse due to earthquake motions.
- Function of essential bridges be maintained.
- Design ground motions have low probability of being exceeded during normal lifetime of bridge.
- Provisions be applicable to all of the United States.
- Ingenuity of design not be restricted.

APPLICABILITY OF PROVISIONS TO THE UNITED STATES

A basic premise in developing the bridge seismic design provisions was that they be applicable to all parts of the United States. The seismic risk varies from very small to rather high across the country. Therefore, for purposes of design, provisions were developed for four seismic performance categories to which bridges would be assigned based on the map area in which the site is located and their importance classification (IC). See Table 1.

Bridges are classified as to their relative importance--either as an essential bridge or all others. An IC coefficient of II is assigned for essential bridges and I for all others. Essential bridges are determined based on their social/survival and security/defense classification. Guidelines for determining these classifications are presented in the Commentary included with the provisions. Essential bridges are those that must keep functioning during and after an earthquake. Differing degrees of complexity and sophistication of seismic analysis and design are specified for each Seismic Performance Category (SPC). SPC D bridges include those designed for the highest level of seismic performance with particular attention to methods of analysis, design and quality assurance. SPC bridges include those where a slightly lower level of seismic performance is required but the potential for damage is slightly greater than SPC D. SPC B bridges include those where a lesser level of seismic performance is required and a minimum level of analysis and specific attention to support design details are provided. SPC A bridges include those where no seismic analysis is required but attention to certain details for superstructure support is provided.

SEISMIC GROUND MOTION INTENSITIES

The selection of ground motion intensities to be used with the seismic design provisions was carefully reviewed. Fortunately considerable study and effort had recently been made to develop seismic risk maps for the "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC-3-06).[3] The ATC-3-06 maps are based on (1) a realistic appraisal of expected ground motion intensities (2) the probability that the design ground shaking will be exceeded is approximately the same in all parts of the United States and (3) frequency of occurance of earthquakes in various regions of the country. It is possible that the design earthquake ground shaking might be exceeded, although the probability of this happening is quite small.

Figures 1,2,3 and 4 are contour maps of effective peak acceleration coefficient A_a and effective peak velocity-related acceleration coefficient A_v, respectively. Figures 3 and 4 reflect the effect of distant earthquakes and should be used for structures with fundamental periods in excess of about 1.0 second. Motions in high seismic areas near active faults could exceed the values shown especially in locations inside of the 0.4g contour in Figures 1 to 4. Qualified experts familiar with local conditions should be consulted for such areas. The development of Figure 1 was facilitated by the work of Algermissen and Perkins of the United States Geological Survey (USGS) and is in many ways quite similar to their map.[4] Seismologists from various parts of the country were asked to comment on proposed versions of Figure 1 and their suggestions were incorporated. Although the maps were literally drawn by a committee, they are judged to provide the best current estimate of the geographical variation of coefficients for effective peak acceleration and effective peak velocity-related acceleration for purposes of design of structures.

The effects of distant earthquakes on flexible structures have been considered for only a few structures. It is generally recognized that design lateral forces should take into account distance from probable earthquake sources because higher frequencies attenuate more rapidly with distance than the lower frequencies. Figures 3 and 4 were therefore constructed to consider the effects of distant earthquakes. The velocity attenuation study by McGuire and the attenuation of modified Mercalli intensity by Bollinger in conjunction with review of other data from large earthquakes were used to develop the contours.[5,6] It is felt that considerable further study should be given to this area.

After the contour maps in Figures 1 through 4 were prepared, a decision was made to divide the country into seven map areas and to indicate the areas on a county-by-county basis. The seven map areas can best be shown in color and will be so included in the actual provisions. Color reproduction for use in this paper was not feasible. A_a and A_v coefficients as listed in Table 1 apply to over 3,000 counties in the United States.

There is still some disagreement among the PEP members regarding which maps to use. There are maps showing base rock accelerations[2,4] and the ATC-3-06 maps showing coefficients related to effective peak ground accelerations on firm ground. The use of base rock accelerations with site amplification as presently determined does not consider all potentially significant factors. The following discusses this matter.

SUPPORTING SOIL EFFECTS

It is generally recognized that the effects of local soil conditions on ground motion characteristics should be considered in structural design. Three fundamentally different approaches have been used in recent United States studies. The first is based on the concept of potential resonance of a structure with the underlying soil. In the SEAOC requirements[7] the seismic site-structure resonance coefficient varies from 1.0 to 1.5 depending on the ratio of the fundamental building period to the characteristic site period.

In a second approach, CalTrans used the computer program SHAKE[2] to develop the soil amplification factor for the design criteria. The program analyzes a one-dimensional soil column for shear wave motions propagating from the rock level to the top of the column. For the third approach (ATC-3-06)[3], representative spectral shapes were modified to determine corresponding values of effective peak ground acceleration for three typical site conditions. These modifications were based on a study of ground motions recorded at locations with different site conditions and the exercise of experienced judgement in extrapolating beyond the data base.

The ATC-3-06 approach for considering soil effects was used in this study and is further discussed in more detail under Preliminary Criteria. The CalTrans approach is limited because only vertical propagating one-dimensional soil effects are considered, and several parameters which could have significant effects are not considered. The parameters include surface waves, oblique transmission of waves through the soil, and the effects of reflection and refraction at the interfaces of different materials.

DESIGN PHILOSOPHY

The primary basis for development of the seismic design provisions for bridges is to minimize the hazard to life and provide the capability for bridges to survive during and after an earthquake with essential bridges to remain functional. To meet this philosophy, certain principles were followed:

- Small to moderate earthquakes should be resisted within the elastic range of the structural components without damage.
- Realistic seismic ground motion intensities should be used in the design procedure.
- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

Conceptually there are two different approaches that are currently in use to satisfy the above principles. These are principally force design approaches and are the current New Zealand and CalTrans approaches as discussed in detail in references 8 and 9. In the New Zealand Code the bridge is designed elastically so that it can resist small to moderate earthquakes in the elastic range without damage. The New Zealand Code accepts the philosophy that it is uneconomical to design a bridge elastically against a large earthquake. For large earthquakes, in the design of ductile bridges, the philosophy is that where possible flexural plastic hinging in the columns is acceptable but significant damage to the foundation and other joints is not.

In the CalTrans approach the base shear and member forces are determined from an elastic design response spectra for a maximum credible earthquake. The design forces for each component of the bridge are then obtained by dividing these elastic forces by a Z factor. The Z factor is 1.0 and 0.8 respectively for hinge restrainers and shear keys; they are therefore designed for the expected and greater than the expected (in case of shear keys) elastic forces resulting from a large earthquake. Well-confined ductile columns are designed for lower forces than expected from an elastic analysis as Z varies from 8 to 4. This assumes that the columns can deform plastically when the seismic forces exceed these lowered design forces. The end result is similar to the New Zealand approach although the procedures are quite different.

In assessing bridge failures of past earthquakes in Alaska, California and Japan, many of the loss of span type failures are attributed in part to relative displacement effects. Relative displacements arise from out of phase motion of different parts of a bridge, from lateral displacement and/or rotation of the foundations and differential displacements of abutments. Therefore in developing the Draft Guidelines the design displacements were considered to be just as important as design forces and, for SPC C and D bridges, requirements for ties between non-continuous segments of a bridge are specified in addition to minimum support lengths at abutments, columns and hinge seats.

The methodology used in the draft guidelines is in part a combination of the CalTrans and New Zealand "force design" approaches but it also addresses the relative displacement problem. The methodology varies in complexity as the SPC increases from A to D. Three additional concepts are included in the draft guidelines that are not included in either the CalTrans or New Zealand approach. First minimum requirements are specified for support lengths of girders at abutments, columns and hinge seats to account for some of the important relative displacement effects that cannot be calculated by current state-of-the-art methods. A somewhat similar requirement is included in the latest Japanese bridge code. Second, member design forces are calculated to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. Third, design requirements and forces for foundations are intended to minimize damage since most damage that might occur will not be readily detectable.

For SPC A bridges the only design requirement is one of providing minimal support lengths for girders at abutments, columns and expansion joints. For the level of seismic risk of these bridges prevention of superstructure collapse was deemed necessary and hence the requirement. Design for the level of seismic forces in these regions was not considered necessary.

For SPC B bridges the approach is similar to that of CalTrans where the elastic member forces are determined from an elastic design coefficient. Design forces for each component are obtained by dividing these elastic forces by a redution factor (R). For connections at abutments, columns and expansion joints, the R-factor is either 0.8 or 1.0 and they are therefore designed for the expected or greater-than-expected elastic forces. Foundations are also designed for the elastic forces. For columns and piers, the R-factor varies between 2 and 6 and they are therefore designed for forces lower than expected from an elastic analysis and are therefore expected to yield when subjected to the forces of the design earthquake. Design requirements to ensure reasonable ductility capacity of columns in SPC B are not specified whereas they are for SPC C and D bridges.

For SPC C and D bridges the general approach is similar to SPC B; however, several additional requirements are included. For columns, additional requirements are included to ensure that they are capable of developing reasonable ductility capacity. For connections and foundations alternate design forces to those determined by the procedures of SPC B are also permitted. These are based on the maximum shears and moments that can be developed by column yielding. Horizontal linkage and tie down requirements at connections are also provided. For SPC D bridges, settlement slabs are required to ensure use of the bridge after an earthquake.

PRELIMINARY CRITERIA

The preliminary criteria are in the process of being finalized with a final draft currently being prepared for review and approval by the PEP. Therefore the criteria discussed in the following subsections are subject to change.

- Design Procedure
- Site Coefficient
- Lateral Elastic Design Force Coefficients
- Analysis Procedures
- Response Modification Factors
- Minimum Support Lengths
- Design Requirements for Reinforced Concrete Columns
- Foundations and Abutments

Design Procedure

A flow chart of the preliminary design procedures is shown in Figures 5, 5A and B. The first step in the design procedure is to determine the effective peak acceleration coefficient A_a and the effective peak velocity-related coefficient A_v for the bridge site from Table 1. The site coefficient, S is determined from the soil profile - Table 3. These three parameters define the lateral design force elastic spectra and coefficients. A normalized plot is given in Figure 6.

The second step is to establish the importance classification (IC) and the Seismic Performance Category (SPC) for the bridge. The SPC is a function of the bridge site map area and the importance classification of the bridge as discussed previously and shown in Table 2. The SPC governs the complexity of the analysis procedures and design requirements that follow.

For SPC A bridges the design procedure requires no analysis but specifies certain minimum design requirements. Although these have not been finalized, consideration is being given to minimum support lengths at abutments, piers and hinge seats as shown in Figure 7.

The third step in the design procedure is to select the required analysis procedure from Table 4 as a function on the SPC and the number of spans of the bridge. The elastic component forces and displacements are then determined using the required analysis procedure and the elastic design spectra or coefficient. The design forces for the components are obtained by dividing the elastic forces obtained in the previous step by the appropriate component response modification factor R given in Table 5.

The displacements used in design are those obtained from either the elastic analysis or the minimum support requirement of Figure 7, whichever is greater. The bridge components are then designed for these design forces in combination with other prescribed dead and live loads. Special provisions are included for reinforced concrete column design for SPC D and C and ensure that the columns have reasonable ductility capacity.

The design of the foundations is performed with the lesser of the elastic forces obtained from step four or the forces generated by the plastic mechanism of the column or bent. The design of the abutments will either precede or be performed concurrently with the procedure described above for the superstructure and substructure.

Site Coefficient

As discussed previously, the local soil conditions have an effect on the ground motion characteristics to be considered in the design. Three soil profile types are used in modifying the design spectra:

Soil Profile Type S_1: Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2500 feet per second): or stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiffer clays.

Soil Profile Type S_2: Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels or stiff clays.

Soil Profile Type S_3: Soft-to-medium stiff clays and sands, characterized by 30 feet or more of soft-to-medium stiff clay with or without intervening layers of sand or other cohensionless soils.

The soil profile coefficient S used to modify the lateral force elastic design spectra or coeffieient is given in Table 3.

Analysis Procedures

One of the more important aspects of the seismic design process is the determination of the base shear force and its appropriate distribution to the seismic resisting members. Two methods are commonly used in both bridge and building design. The equivalent lateral force procedure or equivalent single mode spectral approach has been the basis of most bridge and building codes. The modal elastic analysis or multimode spectral analysis is being used more frequently for calculating the linear response of complex multidegree-of-freedom bridges and buildings. Both of these methods require the determination of the period of one or more of the fundamental modes of vibration of the bridge to determine the design base shear force.

In assessing the appropriateness of these two methods for bridges with different numbers of spans and degrees of complexity as well as different Seismic Performance Categories, the PEP felt a third method should be added. The third method does not require a period calculation for the bridge but utilizes a seismic coefficient. The minimum applicable procedure for a given type of bridge is shown in Table 4 and is dependent on the number of spans and the Seismic Performance Category where:

Method 1 -- Seismic Coefficient Approach
Method 2 -- Single Mode Spectral Approach
Method 3 -- Multimode Spectral Approach

Each of these methods is discussed in detail in Reference 10.

The Seismic Coefficient Approach was chosen as the appropriate method for use in designing single and simple span bridges in all Seismic Performance categories because of the difficulty in estimating the transverse and longitudinal periods and the fact that the fundamental period of single span bridges generally falls in the flat portion of the design spectra.

Lateral Elastic Design Force Coefficients

The equivalent single mode or multimode spectral design approach requires that a horizontal force be considered in the structural design. For the preliminary design criteria the lateral elastic design spectra or coefficient C_s is given as follows:

$$C_s = \frac{1.2\, A_v S}{T^{2/3}}$$

T is the fundamental period of vibration in the direction under consideration. The value at C_s need not exceed 2.5 A_a for Types S_1 , S_2 and S_3 soils except for Type S_3 soils when A_a is equal to or greater than 0.3 the value of C_s need not exceed A_a. The soil profile coefficient S is given in Table 3. The normalized elastic design spectra corresponding to this equation are shown in Figure 6. To obtain the spectra for a given A_a the ordinate of the graph is multiplied by A_a.

Response Modification Factors

Response modification factors R are used to modify the component forces obtained from the elastic analysis. The values currently being considered are given in Table 5. Because considerable judgement is required in determining these values, they will be varied in the redesign phase of the project to determine their effect on the design of the various components.

The rationale used in the development of these values is as follows: The values of 0.8 and 1.0 assigned to the connections at abutments, expansion joints and columns, respectively, means that the connections are designed for the elastic and greater than elastic forces (in the case of abutments). This was done in part to accommodate the redistribution of forces that occurs when a bridge responds inelastically.[11] The other reason for adopting these values was to maintain the overall integrity of the bridge structure at these important joints. The increased protection obtained by designing for these force levels can be obtained at minimum increase in construction cost.

The values used for the columns and bents in the transverse and longitudinal directions considered the redundancy and ductility capacity provided by the various types of support. As an example, the wall type pier in its strong direction was judged to have little ductility capacity and no redundancy; as a result a value of between one and two is being considered.

For a multiple column bent with well-detailed columns the ductility capacity will be of the same order of magnitude as a single column but the redundancy is greater. Consequently, different R factors are being considered for these types of support.

Minimum Support Lengths

The length of support provided at abutments, piers and hinge seats has to accomodate displacements resulting from the overall inelastic response of the bridge structure, possible independent movement of the abutments, displacements resulting from out of phase motion of different parts of the substructure and out of phase rotation of abutments and piers resulting from travelling surface wave motions.

A reasonable estimate of the displacements resulting from the overall elastic dynamic response of the bridge structure can be obtained from the multimode method of spectral analysis if the flexibility of the foundations is included. Better estimates can be obtained if an inelastic time history method of analysis is performed however this is not recommended in the criteria. Either the elastic or inelastic time history methods of analysis will give reasonable estimates of the out of phase movements of different parts of the substructure whereas the multimode method of spectral analysis will not. The recent work of Elms[12,13] can be used to give the order of magnitude of abutment movement although much research remains to be done in this area. The recent work of Werner[14,15] gives some indication of the effects of travelling waves on the response of a limited number of bridges. However, much also remains to be done in this area.

In summary, the current state of the art precludes a designer from making a good estimate of the displacements to be expected when a bridge is subjected to an earthquake. As a result the PEP believed it was necessary to specify minimum support lengths at the abutments, piers and hinge seats to account for the effects discussed above. Obviously a considerable amount of judgement is required and the proposed criteria will be subject to substantial refinement as the state of the art progresses. Currently the method being considered for specifying the minimum support lengths is based on a combination of the span lengths, similar to that used in the Japanese Code, and the column or pier height. The criteria being considered is shown in Figure 7.

In addition to these minimum requirements consideration is being given to the use of the displacements calculated from the elastic analysis if they exceed the specified minimums. Obviously these values will not include several of the effects discussed above nor will they accurately predict the displacements resulting from the overall inelastic dynamic response of the structure; however, in many cases they will give a reasonable estimate of these displacements.

Design Requirements for Reinforced Concrete Columns

A basic feature of the design philosophy and procedure is the capability of columns to respond to intense earthquake motions in a ductile manner. For small-to-moderate earthquakes the bridge is designed to respond elastically.

Compared to buildings only a small amount of research has been performed on the typical bridge, piers and columns. Most of the experimental work performed has been done at the University of Cantebury in New Zealand and is summarized in references 16 and 17. The New Zealand Ministry of Works Design Brief[8,18] incorporates a slightly conservative methodology for estimating the ductility capacity of columns.

After carefully revieweing the New Zealand and SEAOC approaches to column ductility capacity, the PEP decided to adopt the SEAOC approach used for buildings where design requirements are specified to ensure adequate ductility capacity. An estimate of the ductility provided is not required in the design process.

The following aspects of column design are included in the preliminary provisions to ensure reasonable ductility capacity of columns in SPC C and D bridges:

- Sufficient confining reinforcement at sections where flexural yielding is expected to occur.
- Sufficient transverse reinforcement to prevent brittle-type shear failures.
- Adequate anchorage of longitudinal bars to allow flexural yielding in the columns.
- Foundation design requirements to prevent failure in piles.

Foundations and Abutments

The requirements and approaches being considered for the design of the foundations and abutments are discussed in detail in references 13 and 19. The foundations are designed for force levels that will prevent, wherever possible, significant damage occuring in the foundation system in Seismic Performance Category C and D bridges. The force levels will be determined from the plastic mechanism that is deemed capable of forming in the bridge columns or the elastic forces determined from the analysis; whichever is less. Special attention is required for pile foundations to ensure that wherever possible flexural yielding is forced to occur in the columns and not in the piles.

Guidelines for assessing the liquefaction potential of a site will be included in the commentary to the design criteria. All SPC D bridges are required to have settlement slabs to ensure their functionability after an earthquake.

The state of the art in abutment research and design significantly lags that of other areas and as a result current design prodecures recognize the minimal amount of information available. Recent work by Richards and Elms[12] has provided some new insight into the problem and for non-monolithic abutments their proposed methodology is currently being considered for the preliminary criteria and is described in reference 13.

CONCLUSIONS

There has been a tendency in most parts of the U.S. to view earthquakes as primarily a California problem. However, more than 70 million people are subject to moderate or major earthquake risk. Therefore the need is evident for seismic design provisions that are applicable in all regions of the United States for bridges in addition to buildings.

Review of procedures in other countries have indicated a general similarity in the approach to seismic design of bridges. However, the seismic design of bridges is relatively new and therefore there are a number of areas where considerable additional work and study are needed such as the following:

- Better methods need to be developed for determining the periods of vibration of bridges both transversely and longitudinally.
- The effects of interaction between superstructure and abutments are not well understood.
- The interaction of abutments with the retained soil needs further study.
- Methods for determining required bearing support lengths for the superstructure on piers and abutments need to be developed.
- The ductility capacity of columns and piers used in bridges requires more research.
- The adequacy of typical connection details for seismic loads requires additional research information.

It is planned to test the provisions by making a number of bridge designs using the preliminary criteria. The results of these redesigns will be assessed and where appropriate changes will be incorporated into the final provisions.

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10. Imbsen, R.A., Nutt, R.V., and Penzien, J., "Evaluation of Analytical Procedures Used in Bridge Seismic Design Practice", Proceedings of a Workshop on the Earthquake Resistance of Highway Bridges, Applied Technology Council, Palo Alto, California, January 1979.

11. Imbsen, R.A., Nutt, R.V. and Penzien, J., "Seismic Response of Bridges", UCB/EERC Report No. 78-14, University of California, Berkeley, June 1978.

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14. Werner, S.D., Lee, L.C., Wong, H.L., and Trifunac, M.D., "An Evaluation of the Effects of Traveling Seismic Waves on the Three Dimensional Response of Structures", Agbabian and Associates, El Segundo, California, October 1977.

15. Werner, S.D., Lee, L.C., Wong, H.L., and Trifunac, M.D., "Effects of Traveling Seismic Waves on the Response of Bridges", Proceedings of a Workshop on the Earthquake Resistance of Highway Bridges, Applied Technology Council, Palo Alto, California, January 1979.

16. Park, R., and Blakeley, R.W.G., "Seismic Design of Bridges", Summary Report Volume 3, Prepared for Structures Commiteee, Road Research Unit, National Roads Board, New Zealand, 1978.

17. Priestley, M.J.N., and Park, R., "Seismic Resistance of Reinforced Concrete Bridge Columns", Proceedings of a Workshop on the Earthquake Resistance of Highway Bridges, Applied Technology Council, Palo Alto, California, January 1979.

18. NZMWD, "Ductility of Bridges with Reinforced Concrete Piers", Ministry of Works and Development, New Zealand, Civil Division Publication, CDP810/A, April 1975.

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TABLE 1
DETERMINATION OF COEFFICIENTS A_a AND A_v

Map Area Number	Coefficient A_a	A_v
7	0.40	0.40
6	0.30	0.30
5	0.20	0.20
4	0.15	0.15
3	0.10	0.10
2	0.05	0.05
1	0.05	0.05

TABLE 2
SEISMIC PERFORMANCE CATEGORY

A_v	Importance Classification II	I
>0.29	D	C
0.20–0.29	C	C
0.11–0.19	C	B
0.06–0.10	B	A
≤0.05	A	A

TABLE 3
SOIL PROFILE COEFFICIENT

	Soil Profile Type S_1	S_2	S_3
S	1.0	1.2	1.5

TABLE 4
METHOD OF ANALYSIS

Number of Spans	Seismic Performance Category D	C	B	A
Single or Simple	1	1	1	–
2 or more Continuous	2	1	1	–
2 or more with 1 Hinge	3	2	1	–
2 or more with 2 or more Hinges	3	3	1	–

TABLE 5

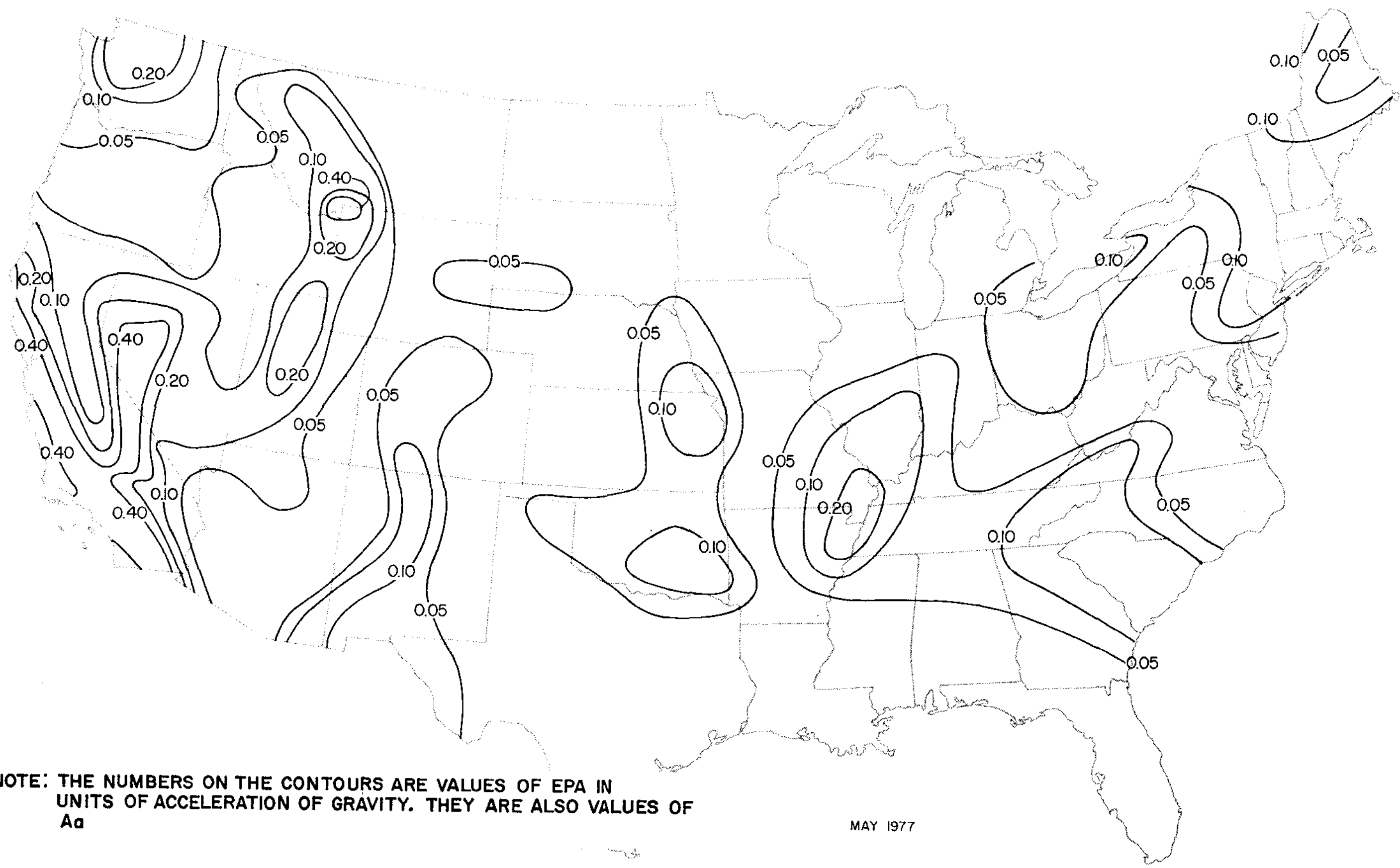
RESPONSE MODIFICATION FACTORS

	Columns or Piers	Connection to/at		
		Abutments[2]	Columns[3]	Expansion Joints
Wall Type Piers[1]	2			
Single Columns	4	0.8	1.0	0.8
Multiple Column Bents	6			

[1]A wall-type pier may be designed as a column in the weak direction of the pier if the provisions for columns are used. In this case the R-Factor for a single column is used.

[2]For single span bridges an R-Factor of 2.5 is to be used for abutment connections

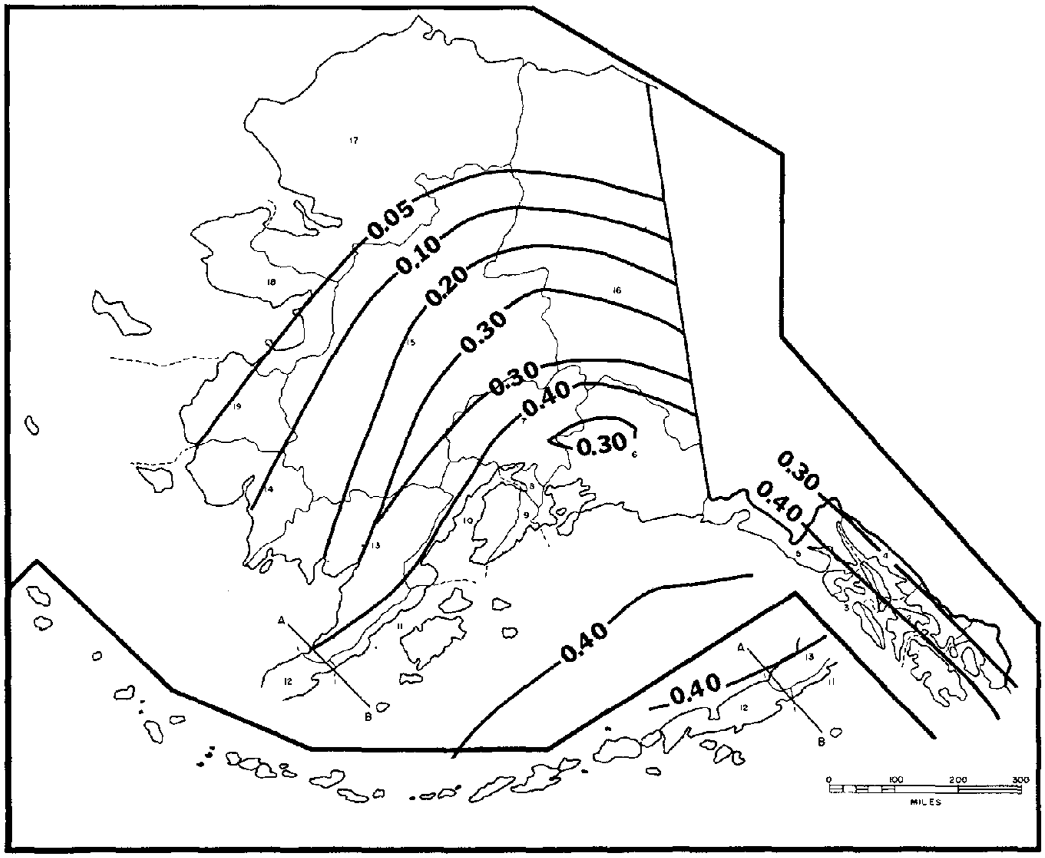
[3]As an alternate the connection to columns may be designed for the maximum forces capable of being developed by plastic hinging at the columns.



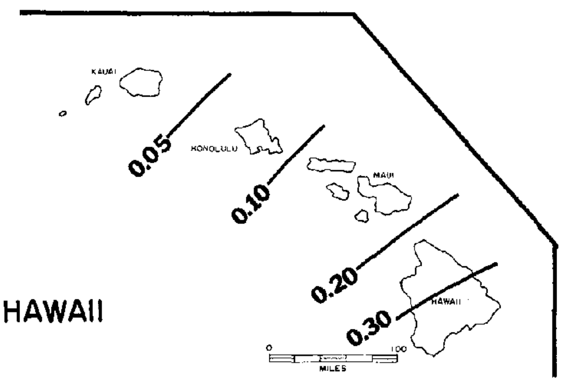
NOTE: THE NUMBERS ON THE CONTOURS ARE VALUES OF EPA IN UNITS OF ACCELERATION OF GRAVITY. THEY ARE ALSO VALUES OF Aa

CONTOUR MAP FOR EFFECTIVE PEAK ACCELERATION
(FROM REFERENCE 3)

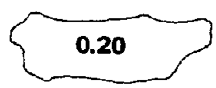
FIGURE 1



ALASKA



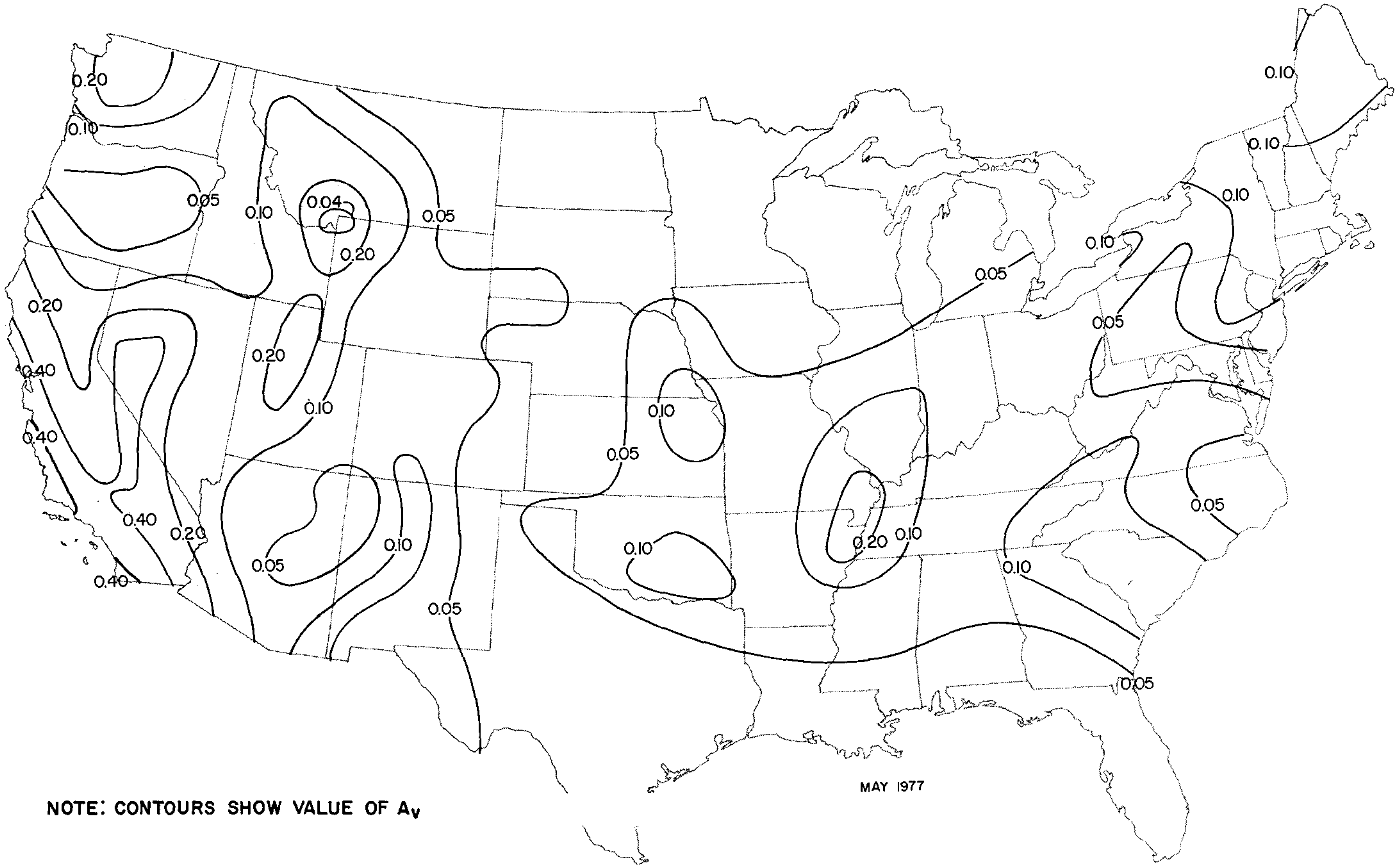
HAWAII



PUERTO RICO

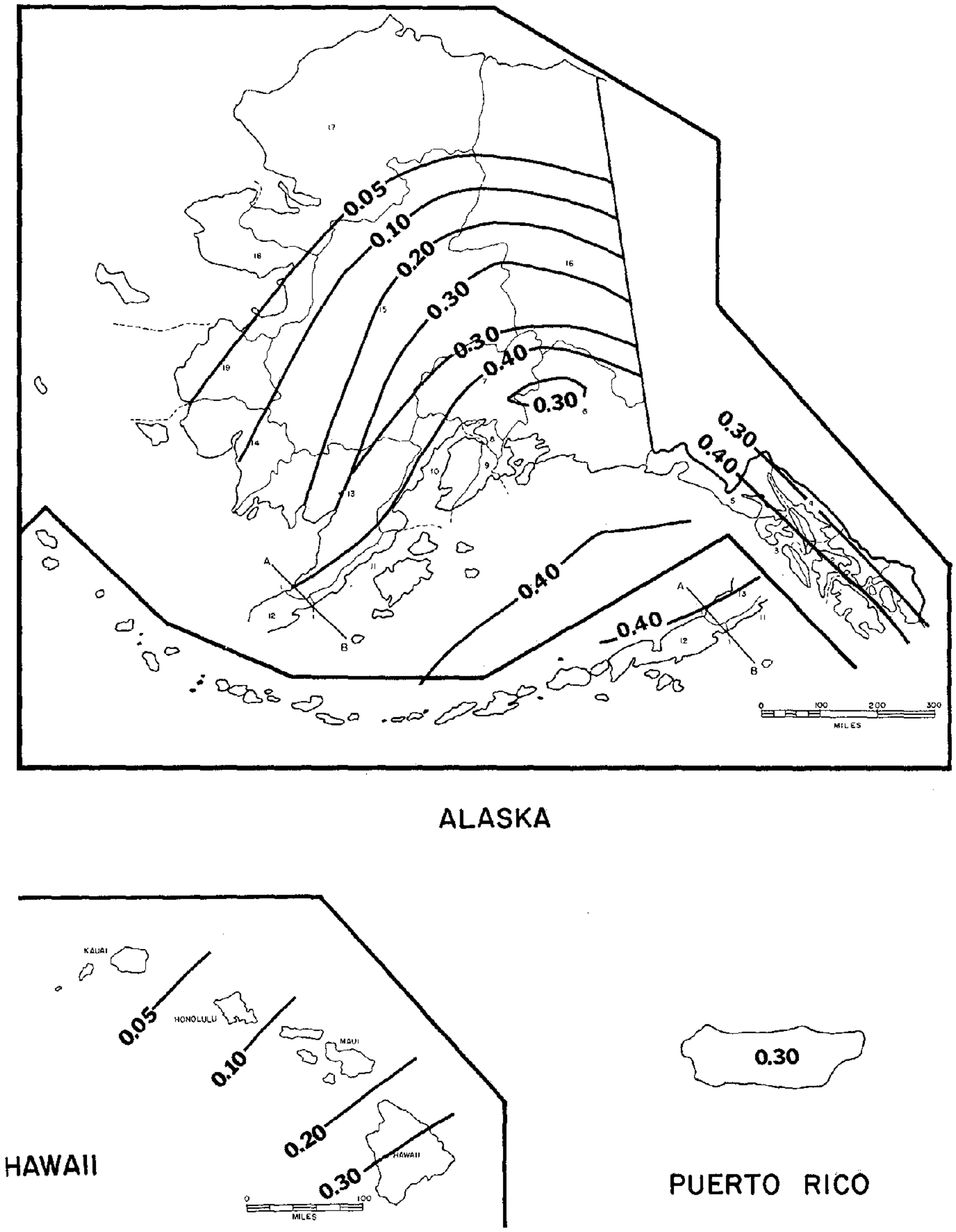
CONTOUR MAP FOR EFFECTIVE PEAK ACCELERATION
(FROM REFERENCE 3)

FIGURE 2



CONTOUR MAP FOR EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION COEFFICIENT
(FROM REFERENCE 3)

FIGURE 3



CONTOUR MAP FOR EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION COEFFICIENT

(FROM REFERENCE 3)

FIGURE 4

FIGURE 5

DESIGN PROCEDURE FLOW CHART

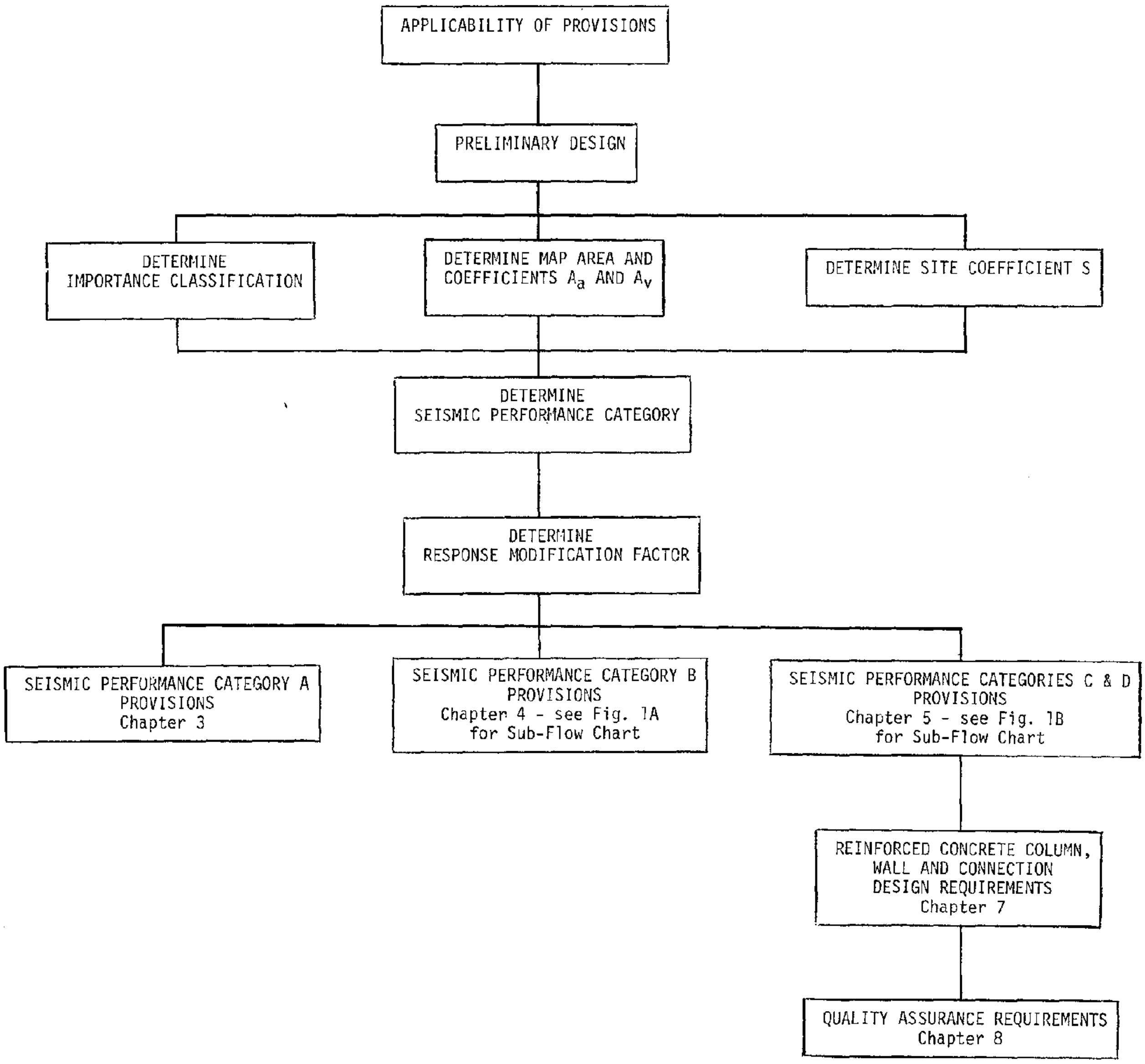


FIGURE 5A

SUB-FLOW CHART FOR CHAPTER 4

SEISMIC PERFORMANCE CATEGORY B

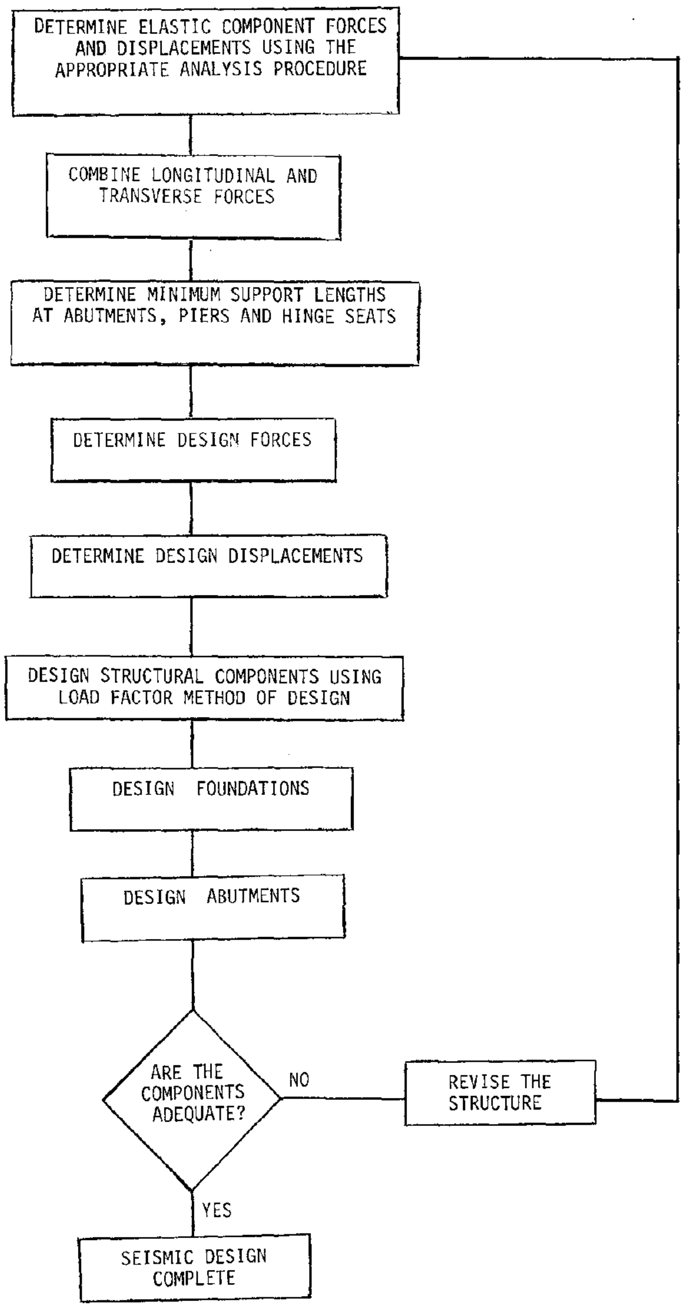
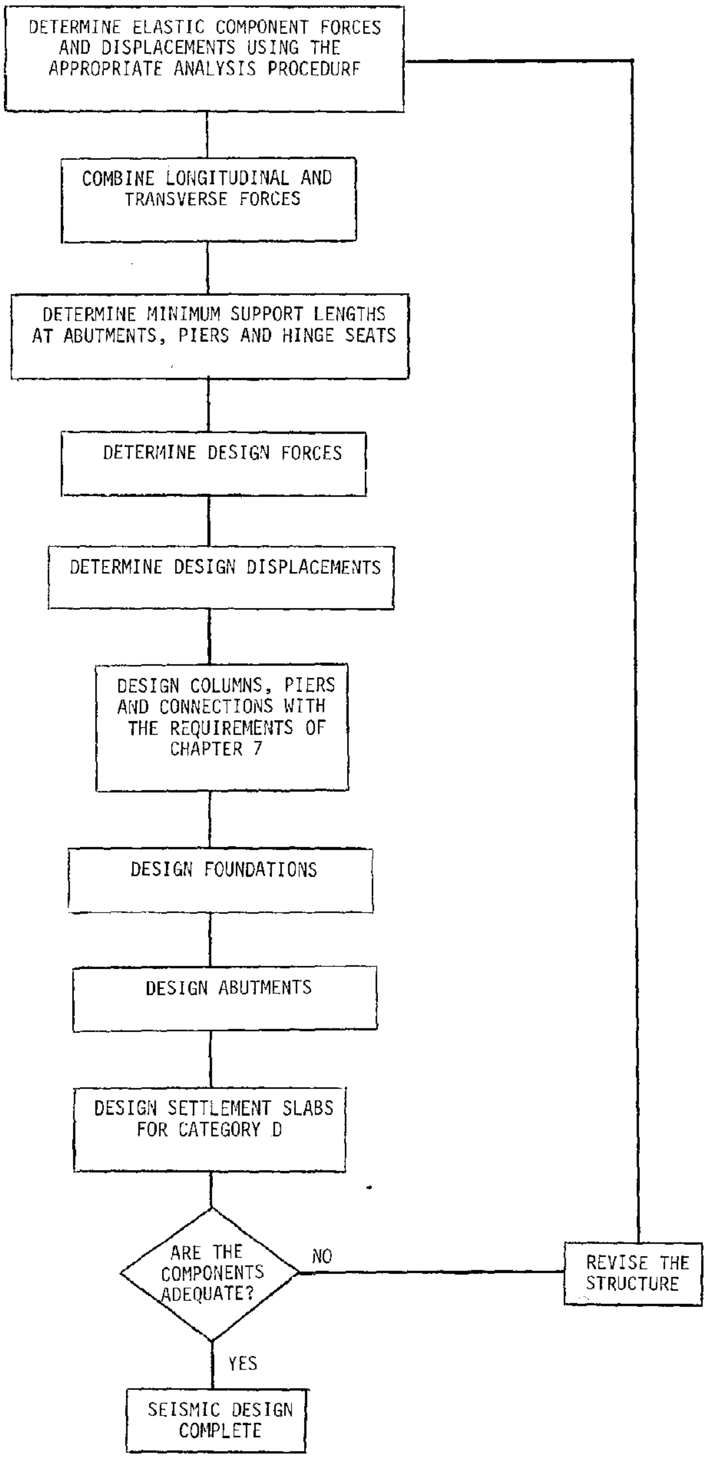
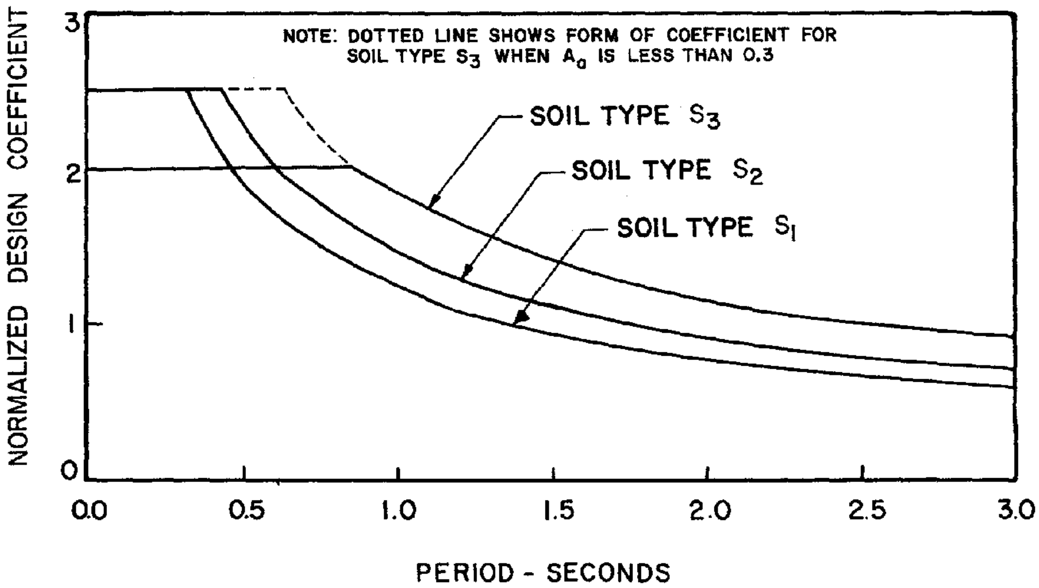


FIGURE 5 B

SUB-FLOW CHART FOR CHAPTER 5

SEISMIC PERFORMANCE CATEGORIES C AND D





NORMALIZED LATERAL DESIGN FORCE COEFFICIENTS
($A_a = A_v = 1.0$)

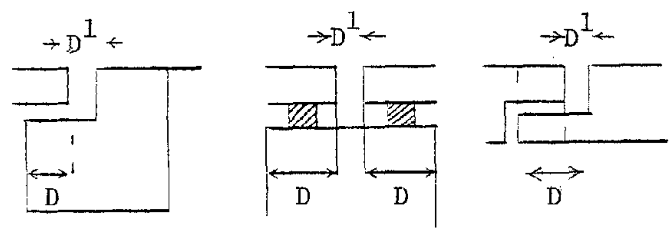
(FROM REFERENCE 3)

FIGURE 6

FIGURE 7

MINIMUM SUPPORT LENGTHS

Substructures supporting the ends of girders shall be designed to provide a minimum support length D (inches) equal to at least that specified below and a gap separation D^1 (inches) shown as a function of D. The gap separation requirement is in addition to that required for creep, temperature and shrinkage.



NON-MONOLITHIC ABUTMENTS — COLUMN OR PIER — EXPANSION JOINTS

$D = 8 + 0.02L + 0.08H$ for SPC A and B

$D = 12 + 0.03L + 0.12H$ for SPC C and D

and $D^1 \geq D/4$

and L = length of the bridge deck to the next expansion joint in feet

and for columns H = column height in feet

for expansion joints H = average column height of the adjacent two columns in feet

for abutments H = average column height of columns supporting the bridge deck to the next expansion joint

APPENDIX A

PROJECT ENGINEERING PANEL

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Professor William J. Hall
University of Illinois

Mr. Edward V. Hourigan
Department of Transportation New York

Mr. Robert Jarvis
Department of Transportation Idaho

Mr. Robert Kealey
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Mr. James R. Libby
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Dr. Geoffrey R. Martin
Senior Engineer, Fugro Inc.

Mr. Joseph P. Nicoletti
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Professor Joseph Penzien
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Princeton University

Project Consultants

Professor David Elms
University of Canterbury

Mr. Roy A. Imbsen
Engineering Computer Corporation

FACTORS CONSIDERED IN THE DEVELOPMENT OF THE CALIFORNIA SEISMIC DESIGN CRITERIA FOR BRIDGES

By

James H. Gates
Senior Bridge Engineer
California Department of Transportation

ABSTRACT

The development of the 1977 earthquake design criteria for bridges in California is described. Factors considered in the criteria are: active fault proximity, effects of local soil conditions, dynamic structural characteristics and force reductions for ductility and risk. The development of a map of rock acceleration for California is described. California Department of Transportation design spectra for rock sites are compared with recent studies including AEC regulatory guide spectrum. The development of elastic response spectra for design by use of standard soil amplification curves is described. The rationale behind current ductility and risk reductions for California bridge components is described. Some of the problems involved in implementing a new seismic criteria in a design environment are discussed.

FACTORS CONSIDERED IN THE DEVELOPMENT OF THE
CALIFORNIA SEISMIC DESIGN CRITERIA FOR BRIDGES

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INTRODUCTION

In California, the February 1971 San Fernando earthquake caused bridge damage far in excess of any previous California earthquake.

From 1933 until 1971 eleven separate earthquakes ranging in magnitude from 5.4 to 7.7 affected approximately 1100 bridge structures in California. In no case were any of these bridges close to the area of intense shaking. The total damage sustained from these events (not including San Fernando) amounted to about $100,000 in today's dollars. This damage was primarily non-vibrational in nature and only affected about 33 bridges.

Most damage observed worldwide before San Fernando was non-vibrational. Penzien, Iwasaki, and Clough (1) in 1972 described this damage as:

(A) Tilting, settlement, and overturning of substructures.

(B) Displacement at supports, anchor bolt breakage.

(C) Settlement of approach fills and wingwall damage.

In San Fernando, however, a significant amount of damage due to vibrational effects such as inelastic column failures were observed in addition to the non-vibrational effects. (Figure 1)

Figure 1 - San Fernando Column Damage - Br. No. 53-1990R

In San Fernando, many modern bridge structures were for the first time located close to and possibly within the causative fault zone. The total damage to highway bridges was about $6.5 Million. This earthquake presented a major turning point in the development of seismic criteria for bridges.

CONCLUSIONS FROM SAN FERNANDO

After evaluation of the observed bridge damage at San Fernando, the following conclusions were drawn (2):

(A) The earthquake force level in San Fernando greatly exceeded the earthquake forces specified by the design criteria.

(B) The vertical acceleration of the earthquake possibly played a part in the cause of the damage. In addition to the large horizontal force exerted by the earthquake, this vertical force generated by the vertical acceleration may have contributed to the collapse of some of the structures.

(C) Skewed structures were highly susceptible to rotational displacement toward acute corners. At some structures, the rotation caused severe damage to columns and abutments.

(D) Tall slender columns performed better than short stiff columns. Shearing and bending fractures that were evident on the short columns were absent in the tall slender columns.

(E) The vibrating action of the earthquake shattered the concrete at the base and footings of many columns. This shattered concrete lost its bonding strength and allowed the column bars to be pulled out causing some structures to collapse.

(F) Deficiencies in details, especially at connections, placed a major role in all of the spectacular and collapse-type failures.

(G) There was considerable ground movement. The ground movement was large enough in some cases to allow spans to drop off.

(H) The fill behind the abutments of many, if not all, structures in the area settled.

Based on these conclusions, the decision was then made to:

(A) Embark on a program to develop a rational design criteria which considered site dependent characteristics and the vibrational properties of the bridge.

(B) Immediately incorporate improved details into all bridges being designed and constructed.

(C) Evaluate and determine priorities for upgrading the earthquake resistance of existing bridges.

FACTORS CONSIDERED IN THE 1977 CALIFORNIA SEISMIC BRIDGES CRITERIA

The following factors, which affect the response of a structure to seismic forces, were selected for inclusion in the criteria:

(A) The location of the bridge relative to active faults.

(B) The effect of a maximum credible earthquake from an active fault.

(C) The effect of overlying soils at a site.

(D) The dynamic responses of the bridge to the ground motion.

(E) The reduction in force level for ductility and risk considerations.

A primary requirement in the development of the criteria was to permit future flexibility. The criteria must be easily modified as new developments are made in earthquake engineering. Therefore, each component of the criteria was designed to represent the independent influence of a different discipline of earthquake engineering.

The four components of the criteria (A, R, S, and Z) were defined as follows:

(A), the peak rock acceleration, is determined from the seismologist's studies of fault activity and attenuation data as gathered from events.

(R), the acceleration spectra in rock, is based on actual earthquake data recorded on rock.

(S), the soil amplification factor, is based on both computer studies and actual recorded data.

(Z), the ductility/risk reduction factor, is based on observed damage and assumed ductility.

The product of three of the factors A, R, and S gives an elastic response spectra curve for the site that would result from a maximum credible event on the closest fault. Division by the factor Z, then gives a design force for the portion of the structure under consideration. The factor Z is component oriented, thus the design force depends not only on seismicity and site conditions, but on the actual structural component being designed.

PEAK ROCK ACCELERATION

Seismic forces tend to dissipate and get smaller (or attenuate) as they radiate outward from the causative fault. Recent studies by Trifunac and Brady (3), which includes San Fernando data, show the peak rock acceleration variation as a function of magnitude and distance from the causative fault, (Figure 2).

The Schnabel and Seed attenuation curves (4) were selected for use in California. These curves still appear to yield reasonable and average values.

For California, the first study of fault activity leading to acceleration contours was performed by the California Division of Mines and Geology (5). This study, originally funded by the California Department of Transportation, was one of the first unified efforts to evaluate fault activity for California. The 1974 version of this study lists 75 faults and fault zones in California that are considered active. For engineering purposes, it was decided that a fault should be termed active if the likelihood of any future movement could cause damage to structures. All evidences of fault activity were evaluated including earthquake history, the amount and age of the most recent prehistoric displacements, topographic expressions and alignments of epicenters. Maximum credible events were estimated for each of the active faults by examining the presently known geological framework and utilizing the magnitude versus fault rupture length data from Bonilla (6). The larger the causative fault length involved in the earthquake, the larger the magnitude of the resulting event. For example, approximately 32 - 48 km (20 - 30 miles) of fault break would be required to generate a magnitude 7.0 earthquake. Using these attenuation and length/magnitude relationships for the 75 selected active faults, a map was then produced (5), which shows the active faults and contour lines of maximum credible peak bedrock accelerations. Figure 3 shows a portion of the 1974 version of the map covering the Los Angeles and San Diego area. The resulting map presents the seismologist's view of fault activity in California and the resulting attenuation of peak accelerations in rock away from maximum credible events on those faults.

HORIZONTAL PEAKS

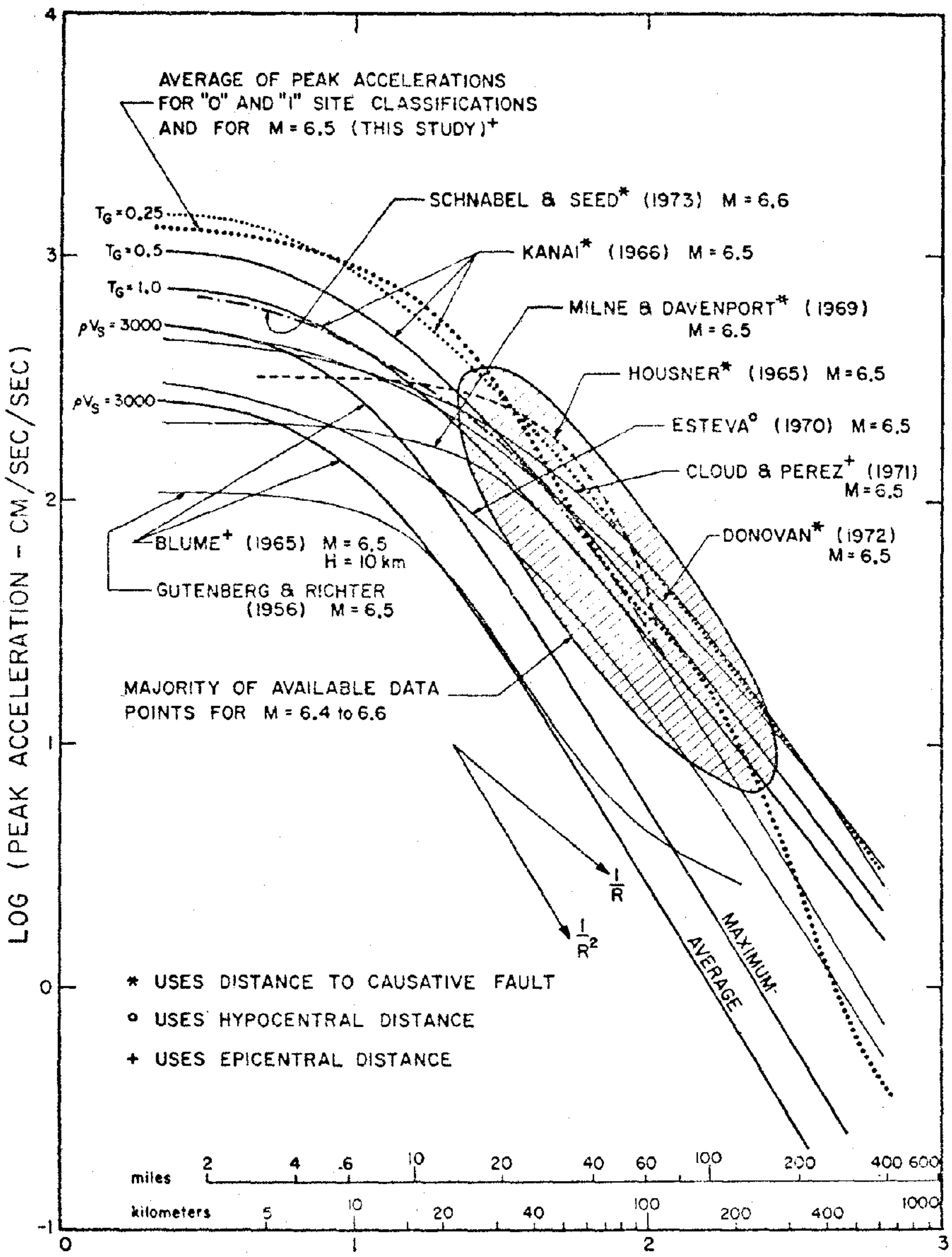


Figure 2 - Comparison of the correlations, for a magnitude 6.5 earthquake, of peak acceleration and distance (3).

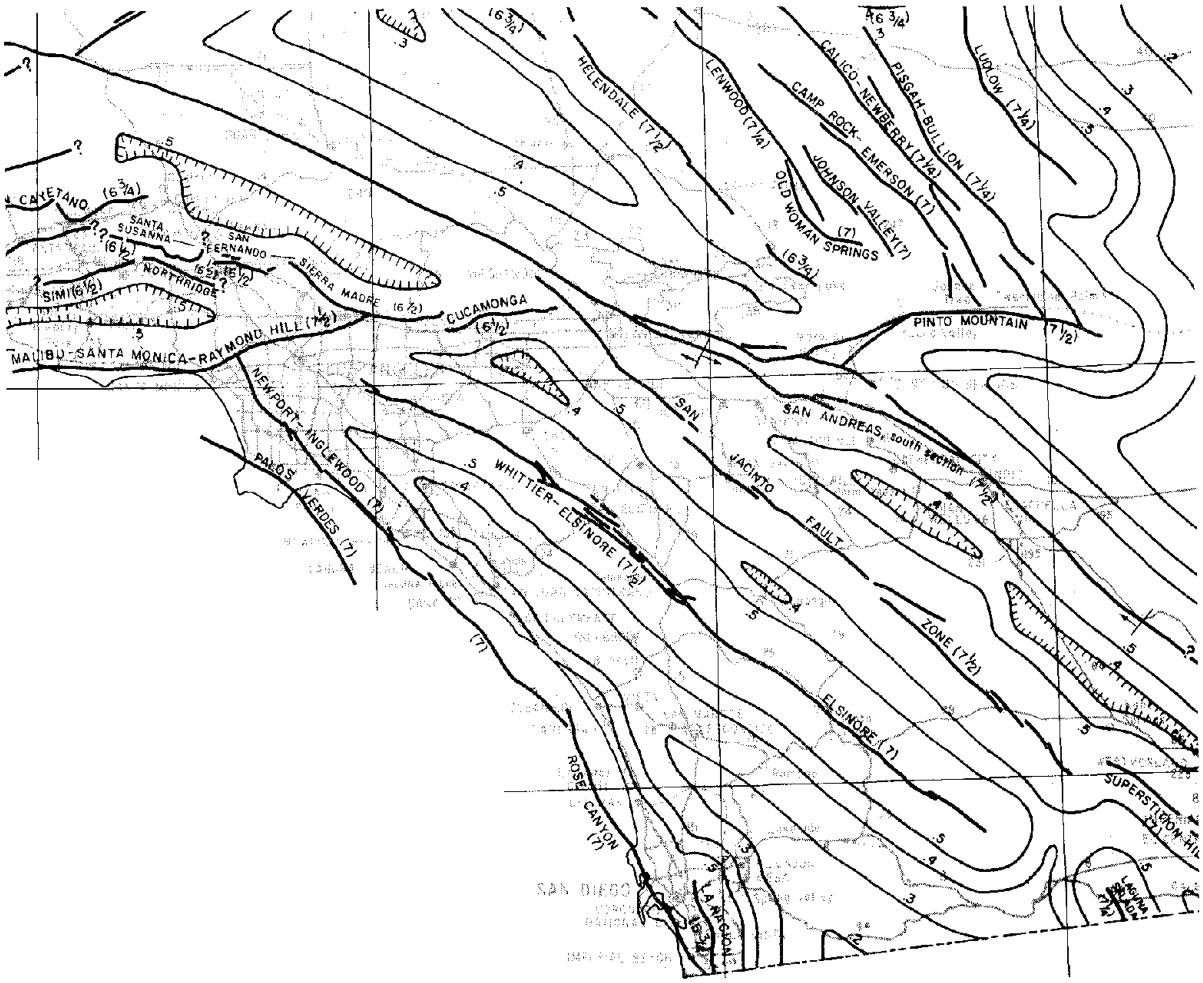


Figure 3 - Portion of Maximum Credible Rock Acceleration Map (5)

The map is admittedly conservative for two reasons:

(1) The determination of fault activity was made without regard to relative activity. If the fault has moved at all in recent geologic time or showed evidence of historic movement or activity, it is termed active.

(2) Total fault rupture was utilized in determining the maximum credible magnitude. This length is determined based on geologic considerations and is estimated on the conservative side.

It is still felt that while some active faults in California are more certain to generate large or near maximum credible earthquakes in the next 50 to 100 years, any current attempt to estimate the reduced probability of occurrence of an earthquake based on the geologic record or the short historic record would be open to serious questions. For this reason the design should be based on the maximum credible event.

In order to be useful, this type of map must be continuously evaluated and updated. This evaluation and updating procedure should be performed utilizing input from a number of seismologists and geologists and should attempt to incorporate all recent data such as newly discovered faults.

For design, the determination of the value of peak bedrock acceleration (A) for a site should be performed by the engineering geologist responsible for the foundations at the site. For sites within the 0.5g contour, a more detailed examination should be made of the fault zone and its relation to the structure and a peak rock acceleration assigned. In California peak rock accelerations up to 0.7g may be assigned.

ACCELERATION SPECTRA IN ROCK

The peak acceleration alone does not completely define the motions at a rock site. The dynamic response of a system of simple single-degree-of-freedom pendulums (a response spectrum) is usually used to indicate the frequency content of an actual seismic event. The actual recorded motions from five rock locations were utilized in developing the R or Normalized Rock Spectrum curve:

(1) Castaic (San Fernando, 1971)

(2) Lake Hughes No. 4 (San Fernando, 1971)

(3) Pacoima Dam (San Fernando, 1971)

(4) Temblor (Parkfield, 1966)

(5) Golden Gate (San Francisco, 1957)

The shape of a response spectrum in rock is primarily controlled by the predominant period of the spectra, which is the region where the spectra maximizes. The predominant period of rock motions were studied by Seed and Schnabel (7). A slight increase in the predominant period is used to denote the recognized lengthening of the waves as they move away from the source.

Normalized 5% response spectra were computed for each of the five records. Predominant periods were obtained by adjusting the time interval to obtain periods of 0.2 sec, 0.4 sec, 0.5 sec, and 0.6 sec. Smooth average curves were then drawn through the spectra using 2.6 as the maximum spectral amplification. (Figure 4)

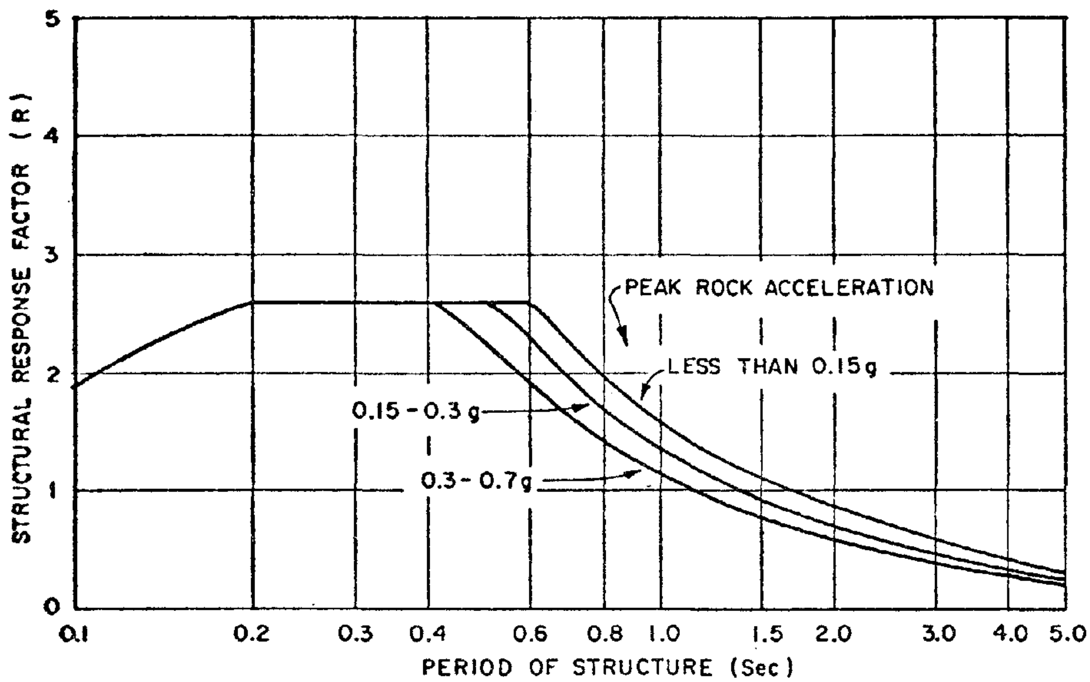


Figure 4 - Normalized Rock Spectra (R)

An elastic spectrum (5% damped) on rock can thus be obtained for any location in California by multiplying the maximum expected bedrock acceleration, A, by the ordinates of the R curve.

Since the CALTRANS R spectra was developed in 1972, several studies have been performed to determine rock spectra based on statistical analyses of various accelerometer records. Newmark, Blume and Kapur performed two independent studies to determine the AEC Regulatory guide spectrum (9). A comprehensive study of 104 accelerometer records was also recently performed in Berkeley in 1974 by Seed and others (8). Most of the records studied were obtained from the Western United States and a small number were from Japanese sites. Figure 5 shows a comparison of the 84 percentile Seed curves for stiff soils and rock, the AEC Regulatory guide spectrum and the CALTRANS R spectra. The CALTRANS spectra appear to adequately define rock motions.

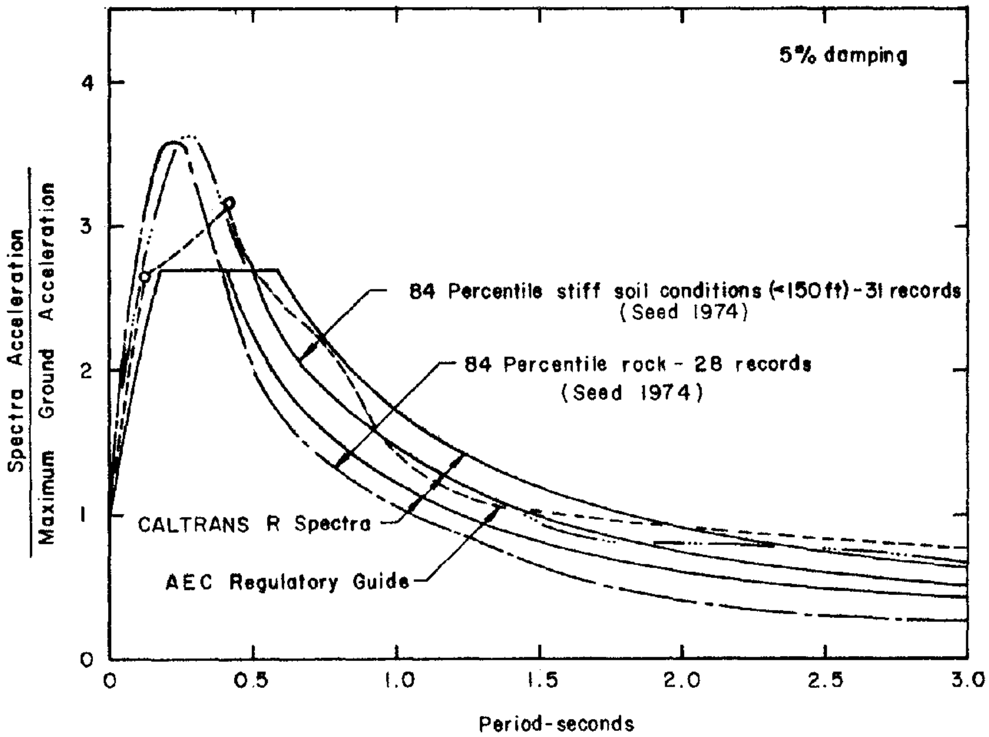


Figure 5 - Acceleration Spectra in Rock Compared

SOIL AMPLIFICATION FACTOR

It is generally recognized that the type and depth of soil over bedrock will modify the rock motion dramatically. In some cases, the accelerations have been intensified by several hundred percent. The computer program SHAKE (10) was used as a basis to develop the S or Soil Amplification Factor for the criteria.

The SHAKE program analyzes a one-dimensional soil column for wave motions propagating from the rock level to the top of the column. By examining the spectral ratios between the computed surface motions and the input rock motion it is possible to compute the amount of amplification (or attenuation) for a number of frequencies. This spectral ratio is defined as the surface spectra divided by the rock spectra on a point-by-point basis, for 5% damping.

It was found that this ratio was primarily dependent on the soil properties and not influenced to a great extent by the frequency content of the input motion. The resultant "S" curve is generally smooth and is used as a modifying factor applied to the bedrock spectra R.

The SHAKE program was utilized to conduct a series of parameter studies to evaluate the various factors which influence the surface motions. A total of 21 computer runs, analyzing 157 soil and earthquake combinations were made. Twenty-four soil columns of varying depth and soil type were analyzed. The soil columns used in the study were of two types, dense granular and compact granular. Both of the soil types contained normal variations in grain size including silty sand. The two groups selected represented alluvial deposits such as those found in the Los Angeles basin or the San Joaquin Valley. Gravels and clays were not included in the study. The study first established a maximum depth soil column of about 180 m (600 ft.). Subsequent runs were then made by moving the "rock-like" base upward through the soil column. The term "rock-like" used describes material that at depth behaves as rock. This material is assumed to have a shear wave velocity of 760 m/s - 910 m/s (2,500 fps - 3,000 fps). As a result of this parameter study and additional computer runs using actual sites, the following conclusions were drawn:

(1) The major variable affecting soil response is the depth of the soil to "rock-like" material.

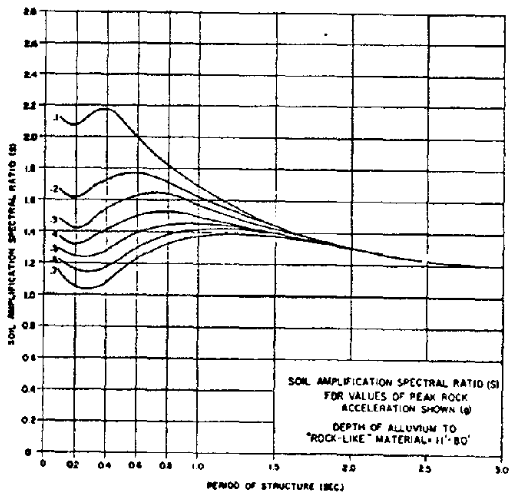
(2) The maximum rock acceleration has the second major influence on the response.

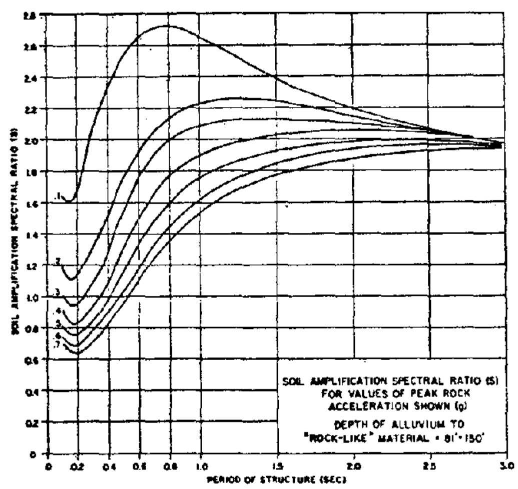
(3) The effect of depth over 76 m - 91 m (250 ft - 300 ft) and the effect of water is negligible.

(4) The use of average amplification curves is an adequate method to obtain site dependent spectra for average California alluvium sites.

Figure 6 shows the standard S curves which were developed as a result of the parametric studies.

NOTE : FOR ROCK SITES AND SITES WITH DEPTH OF ALLUVIUM TO "ROCK-LIKE" MATERIAL UP TO 10 FEET USE S= 1.0





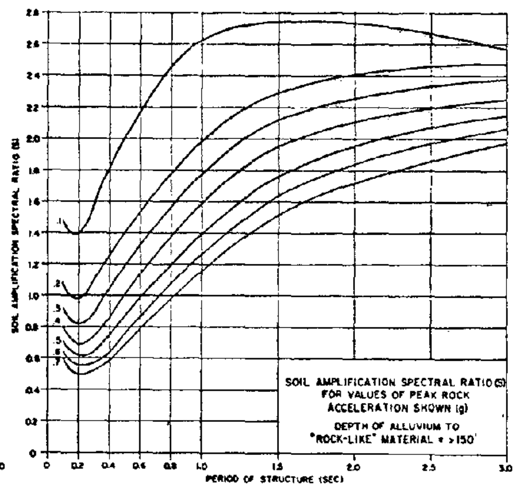


Figure 6 - CALTRANS Standard Soil Amplification Factors (S)

Correlation studies for two sites (11) show the product of the three factors A, R and S produce an elastic 5% damped acceleration spectra, which conservatively envelopes the recorded values.

Recent studies by Bell and Hoffman (12) point out that the standard amplification curves developed at CALTRANS provide a useful tool to determine site response for dense granular soils.

It is recommended that the selection of the proper amplification curve be made by a soils engineer or an engineering geologist who is familiar with the assumptions and limitations involved when using the standard curves.

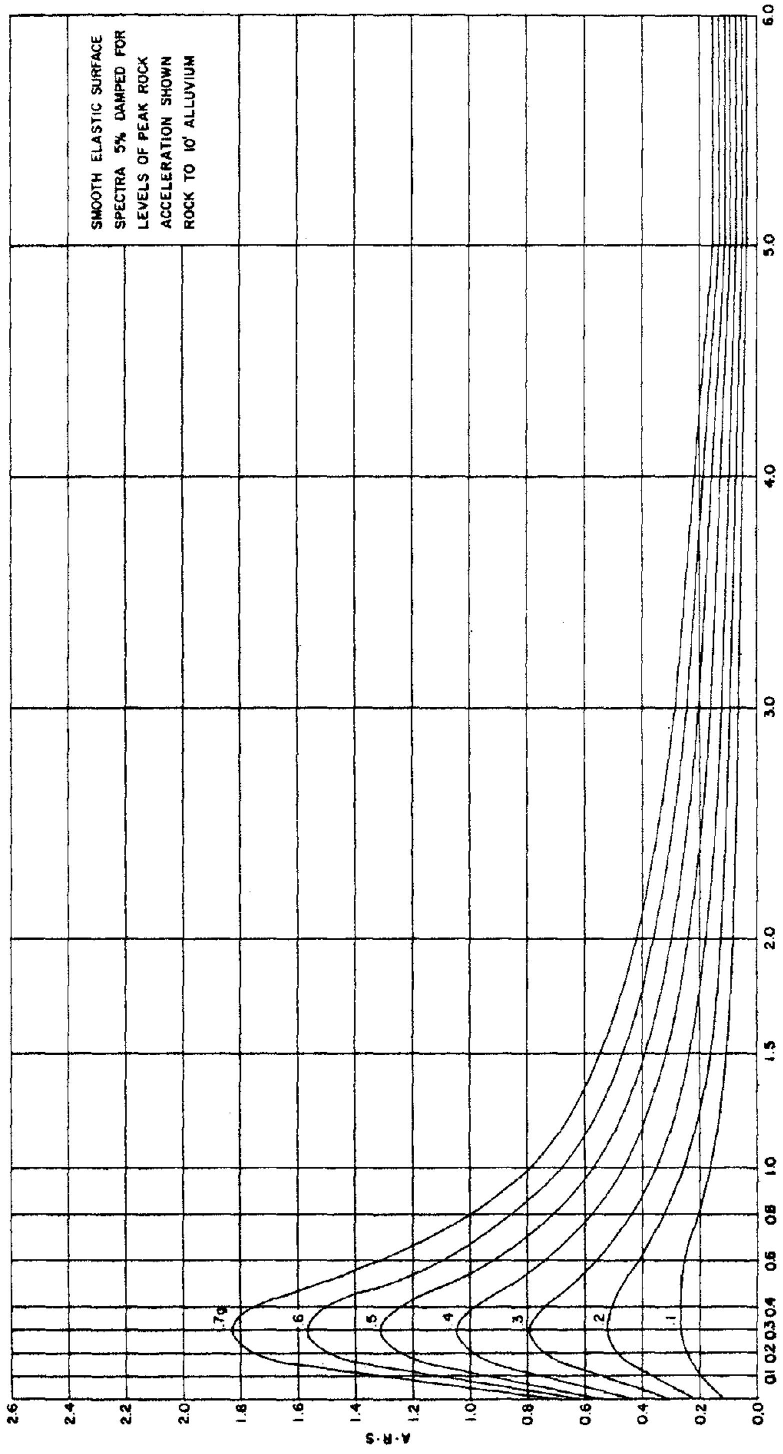
When nonstandard conditions are encountered or where a major bridge structure is planned in a high seismic area, a more detailed ground motion study should be performed considering both the specific site conditions and the local geologic profile. At CALTRANS this nonstandard work is performed utilizing the SHAKE program to analyze representative soil columns for different magnitude events at various distances. Sites which cover a large area may require two or more curves for application to different portions of the structure. The resulting "S" curves are then combined with the standard "R" spectra for rock to arrive at the elastic spectra for use in design.

ELASTIC SITE SPECTRA (A·R·S)

The product of A, R and S results in an elastic response spectra at the site from a maximum credible event. Curves for the standard soil configurations (Figures 7, 8, 9 and 10) represent elastic spectra for average dense granular sites. These spectra are presented in the criteria as elastic spectra and are not reduced to account for ductile behavior.

It has been our experience that the use of pre-reduced spectra to account for ductile behavior is confusing to the engineer and for certain portions of the bridge could lead to highly erroneous results. The use of elastic spectra allows the engineer to cope with realistic deformations in the bridge by presenting him with ultimate loads. For nonstandard sites, elastic site dependent design spectra may be determined by any acceptable method, thus permitting the remainder of the criteria to be utilized directly.

By utilizing elastic site spectra, the criteria retains the capability to use spectra developed for any structure; thus elastic spectra developed for a building or a nuclear power plant could be applied to bridges. This common ground permits the sharing of valuable geotechnical data in high seismic areas and also results in increased communication among seismologists and geotechnical personnel involved in development of site specific spectra.



<u>**Figure 7 - CALTRANS A·R·S Spectra for Rock**</u>

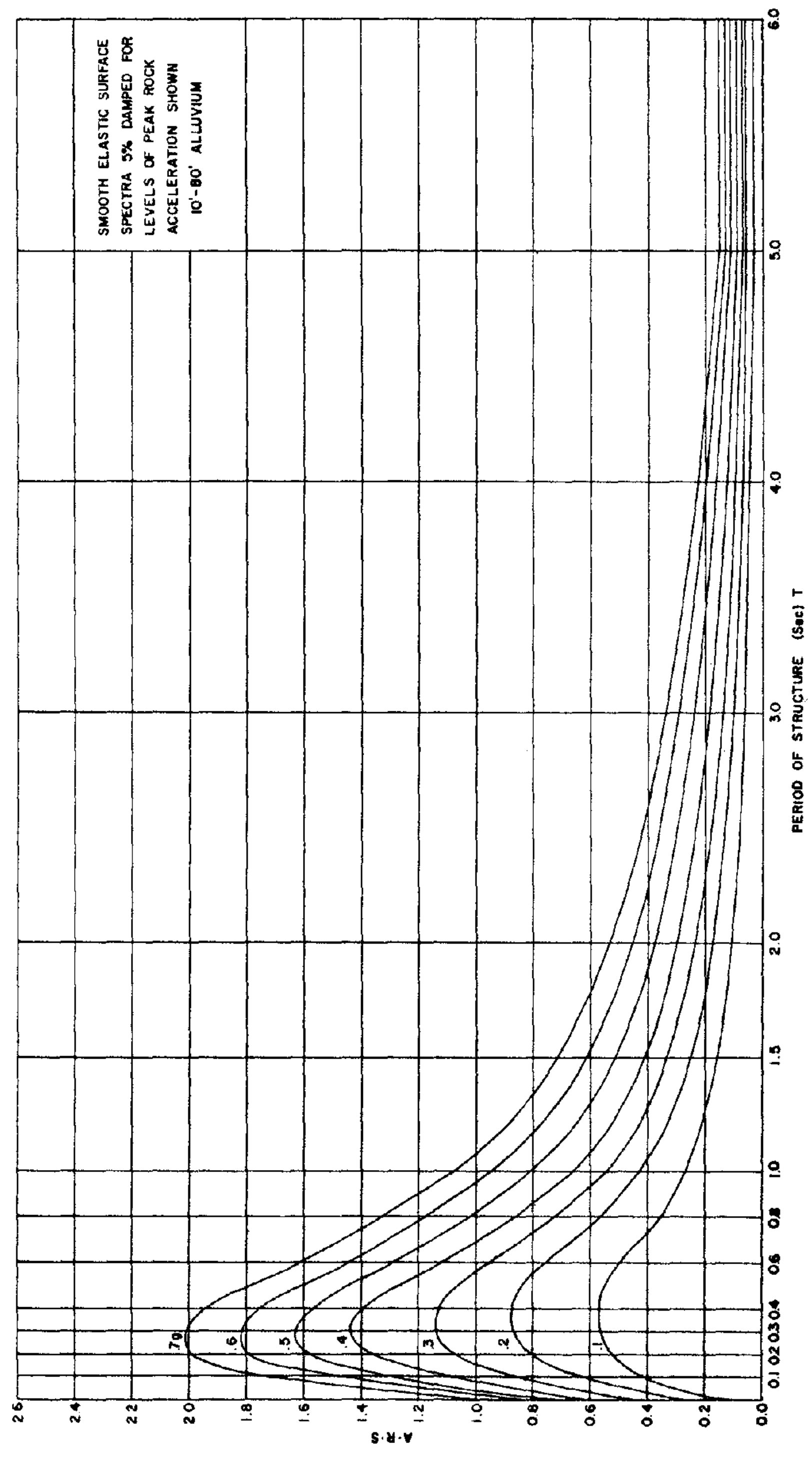


Figure 8 - CALTRANS A·R·S Spectra for 3 - 24 m of Alluvium (10 - 80 ft)

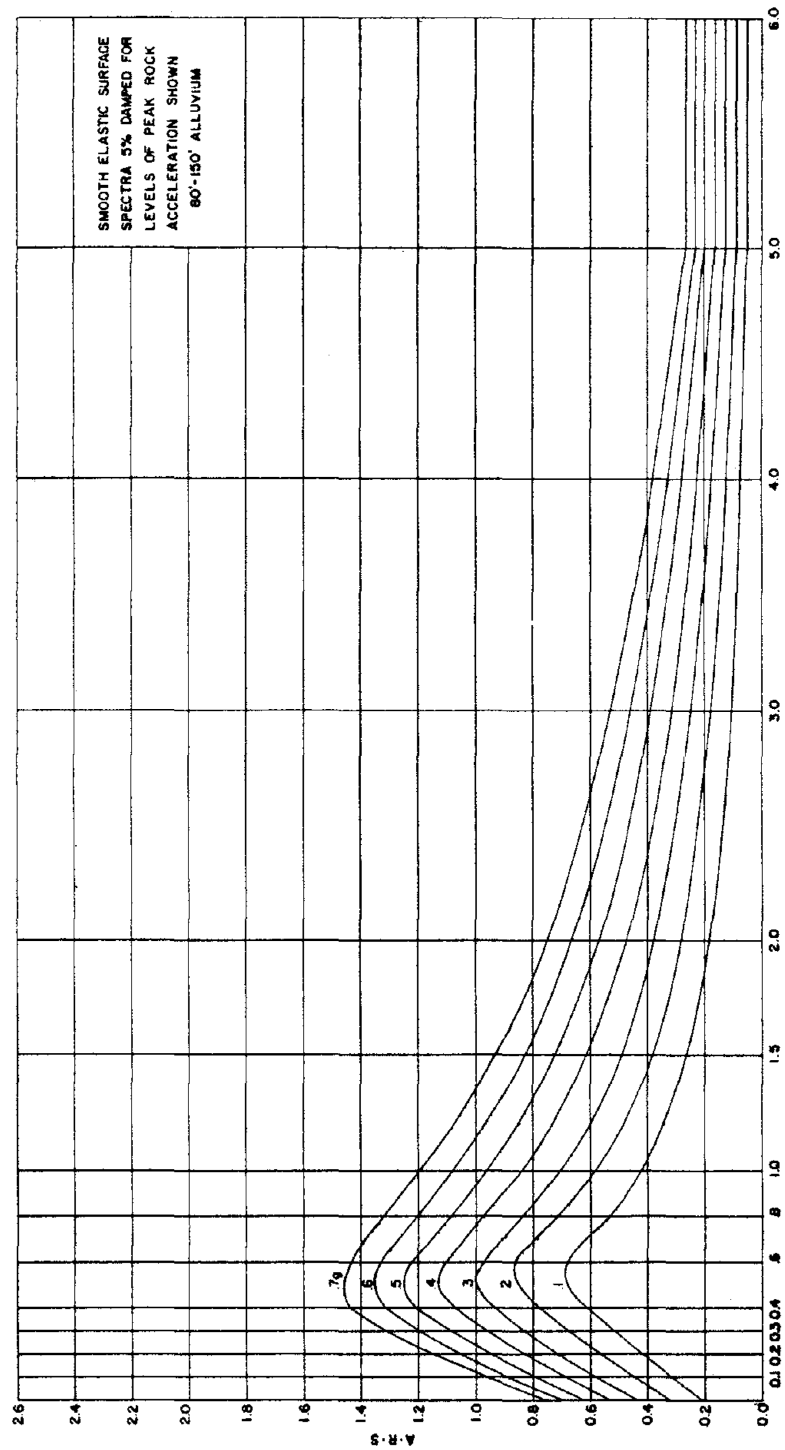


Figure 9 - CALTRANS A·R·S Spectra for 24 - 46 m of Alluvium (80 - 150 ft)

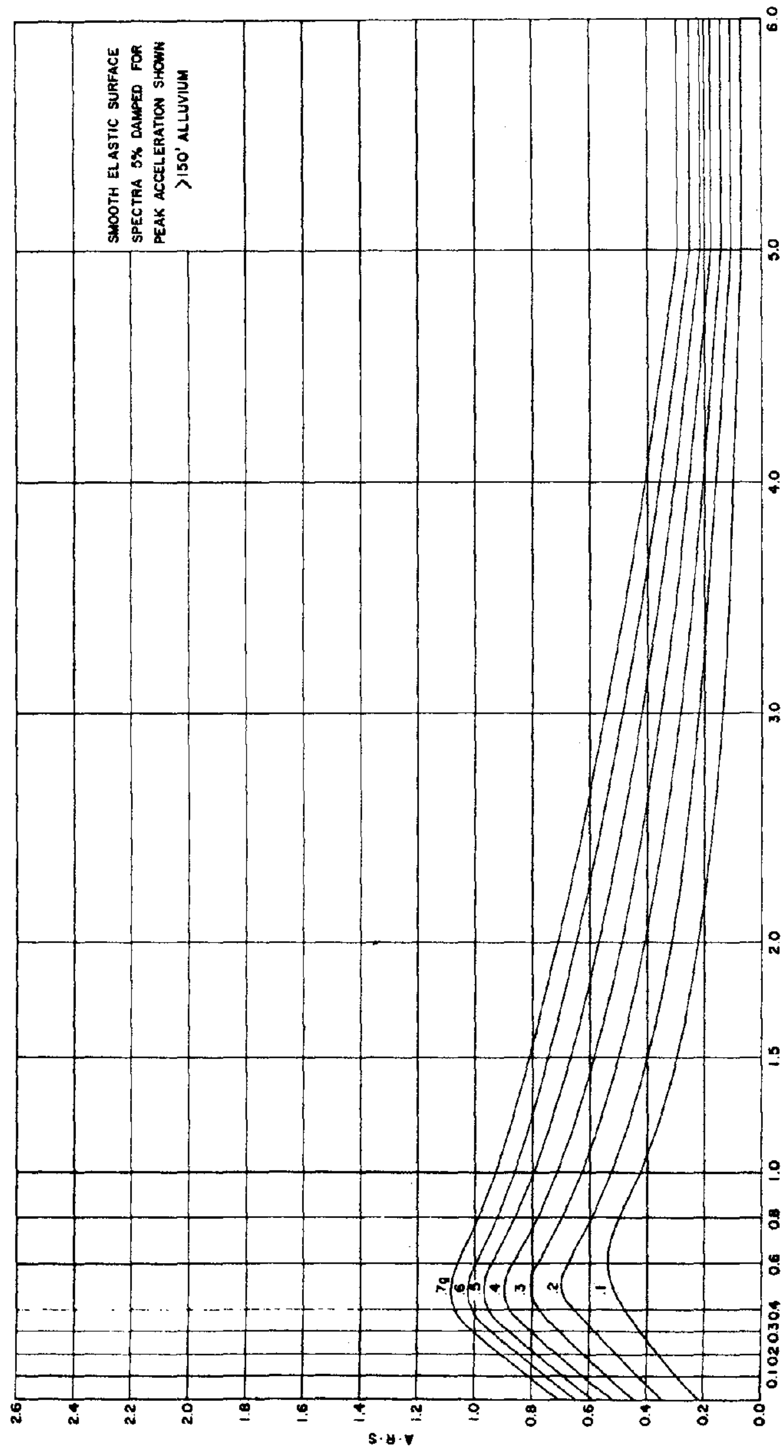


Figure 10 - CALTRANS A•R•S Spectra for Greater than 46 m (150 ft) of Alluvium

DUCTILITY/RISK REDUCTION FACTOR

It is generally assumed that because of the large forces involved in earthquakes and the infrequency of their occurrence, some allowance for inelastic action may be permitted.

Past experience with large earthquake forces in structures has shown that because of inelastic response, increased damping and other factors; a column member is able to resist considerably higher forces than is indicated by ordinary elastic analysis methods.

Ductility is defined as the ratio of maximum plastic deformation to the maximum elastic deformation and represents the degree of elastoplastic action that occurs in a member before failure. The amount of ductility available in a particular component is essentially dependent on the material properties of the member and to some extent on the joint details and system of framing. For example, the available ductility is higher for a bending failure as opposed to a shear failure.

A ductility factor of between 4 and 6 for a well confined, properly detailed reinforced concrete column is normally assumed for a column member that will withstand deformations up to the point of the beginning of visible damage (14).

A basic ductility factor of 4 was selected for the modern well confined reinforced concrete multi-column bent members used on California. This factor is reduced to 3 for the more vulnerable single column members which have less end restraint at the column top and have a higher collapse potential.

Ductility factors of 1.0 and 0.8 were selected for restrainer cables and keys. These low factors insure that these components are not stressed beyond yield. Failure of these components may lead to collapse conditions and thus a lower ductility factor is used.

In addition to ductility the "Z" factor contains a judgment factor for risk. This factor, which is hidden in the Z factor, was provided to permit control over the damage threshold for column members of bridges which exhibited a degree of success in the San Fernando earthquake. The low-level bridges with periods less than 0.6 sec were quite stable and considerably less vulnerable to collapse than the higher, more flexible bridges. Even when the columns were severely damaged, the inherent stability of the structure as a whole, prevented the collapse of the bridge. An additional risk factor of 2 was thus selected for column members only in structures with periods less than 0.6 sec. This risk factor was decreased linearly to 1.0 at 3.0 sec period.

The ductility/risk reductions (Z) currently used at CALTRANS are shown in Figure 11.

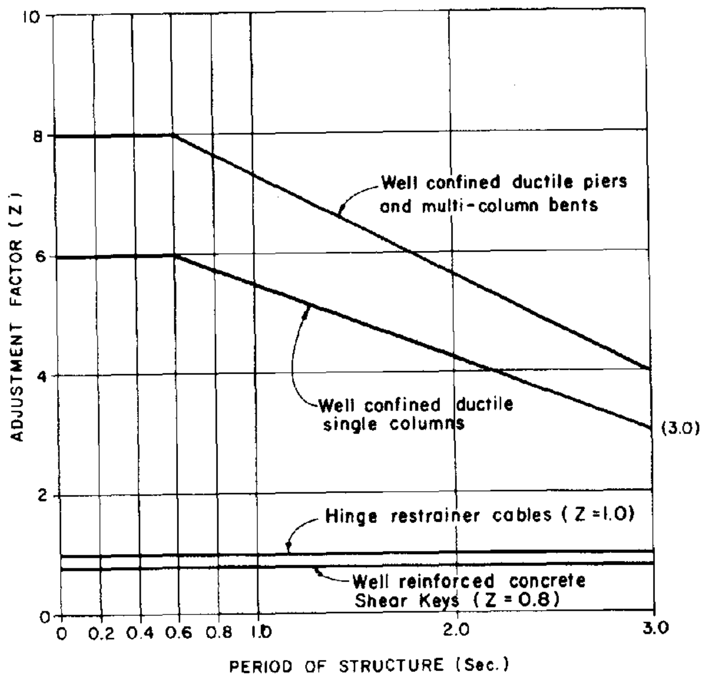


Figure 11 - CALTRANS Ductility and Risk Reductions (Z)

CRITERIA IMPLEMENTATION

The initial version of this criteria presented the engineer with a set of pre-reduced design force curves which incorporated the Z reductions for column members. This pre-reduced version coupled with increasing usage of response spectra modal analysis resulted in much confusion among design engineers.

The removal of Z from the plotted design curves greatly improved this situation. The engineer now is able to look at the bridge and have a fair idea of the total deformations anywhere in the structure. He is able to examine potential collapse mechanisms and even may permit a noncritical component to fail and evaluate the consequences of that failure.

With adequate computer support, training and management this implementation at CALTRANS took about 3 to 4 years.

CONCLUSION

While the emphasis for this paper is aimed at the development and application of the California Seismic Design Criteria, the need for adequate and well thought out details must not be minimized. The designer must carefully examine all connection details and visualize their performance beyond the elastic limit.

A bridge designed for a very high force level can easily be a seismic failure because of a weak or brittle detail. The designer must attempt to foresee and prevent all potential collapse mechanisms, even if extensive damage occurs. The areas for critical detail examination on both new and existing bridges are:

(1) Joint restraint details - primarily the addition of steel cables to prevent excessive separation of expansion joints.

(2) Column details that emphasize the use of increased lateral tie reinforcement at maximum flexural locations.

(3) Shear key details - primarily heavy concrete keys at abutments and expansion joints.

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SEISMIC DESIGN OF BRIDGES -- AN OVERVIEW OF RESEARCH NEEDS

by

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ABSTRACT

This paper contains a brief discussion of criteria applicable to the seismic design of bridges, including selection of the earthquake hazard for design, earthquake ground motion, soil-structure interaction, damping and energy absorption, methods of dynamic analysis for design, and design procedures for bridges. Concurrent with the above is a discussion of those topics for which it is believed major research effort is needed in the decade ahead; foremost among these topics are further consideration of approaches for handling the modelling of bridge structures to permit relatively simple yet rational seismic analysis, and approaches for handling nonlinear effects and relative motions.

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INTRODUCTION

General Purpose and Scope

The purpose of this paper is to provide a brief overview of the seismic design criteria applicable to bridges and concurrently to point out those topics believed to need major research effort in the next decade. Because of the wide variety of types of bridges, each possessing special characteristics, it is difficult to describe explicitly all factors needing study for every bridge type; thus in the paper the discussion is directed to those points needing research which it is believed will have the widest applicability generally in bridge design.

The paper begins with a short section, which follows next, citing selected background studies on bridge damage. Thereafter follows discussions on selection of the earthquake hazard for design, earthquake ground motions, soil structure interaction, damping and energy absorption, methods of dynamic analysis, and some recommendations pertaining to design procedures. Of the many recommendations for research discussed herein among the more important are those pertaining generally to methods for providing for allowance for relative ground motion in design, approaches to permit simple yet rational dynamic analysis of the structures, and approaches for handling nonlinear (energy-absorbing) behavior.

Background

Over the centuries numerous bridge failures resulting from earthquakes have been reported. In recent decades the reports of major earthquakes contain bridge damage descriptions routinely. Among the more comprehensive summary documents pertaining to seismic bridge damage are those contained in Refs. 1, 2, and 3. A review of the literature reveals many different types of damage as having occurred. Much of the damage arises from the vibratory (shaking) aspect of the motion with major damage, in terms of loss of spans (dropping of spans), arising from pier and abutment movement (attributable partially to vibratory motions and partially to relative displacement). That such damage surveys are valuable and effective is apparent when one realizes the many changes in design practice recently undertaken in Japan (Ref. 4) among which is that of providing wider pier caps to help preclude span drop. A survey of the literature suggests that no major long span bridge has yet been subjected to a high intensity earthquake. This observation, if correct, suggests that we may yet have much to learn about the seismic behavior of large long span bridges in actual earthquakes, especially if selected long span structures are adequately instrumented.

Current bridge design specifications and guidelines (for example Refs. 4, 5, 6 and 7) are undergoing rapid changes as a result of intensive earthquake damage assessment studies and of the results of research carried out in recent years. Even so, it is believed that it should be possible to arrive at even better design approaches than currently exist and thus the reason for the workshop and the Applied Technology Council Project ATC-6.

SELECTION OF EARTHQUAKE HAZARD FOR DESIGN

The process of earthquake resistant design requires selection of earthquake hazards as well as estimates of structural strengths, either implicitly or explicitly, as an integral part of the design procedure. Unless these determinations are made in a consistent manner, the final design may be either grossly uneconomical or dangerously unsafe. Both sets of parameters are probabilistic in nature although, for convenience, many aspects of the determination of structural strength may reasonably be approximated as deterministic. However, the earthquake motions themselves for which the design is to be accomplished, or even the occurrence itself of an earthquake affecting the site, must be considered generally as probabilistic matters. Many of the aspects of selection of design earthquakes are discussed in Refs. 8 and 9.

In building structures a single earthquake hazard is employed whereas in the design of nuclear power plants it is generally considered desirable to provide resistance against two earthquakes: (1) a "maximum credible earthquake" which has only a small probability of occurrence during the lifetime of the plant for which the design is made at yield levels or limit conditions, or with some yielding that does not impair the safety of the structures or equipment; and (2) an earthquake having a much higher probability of occurrence, possibly with a "return period" of the order of several hundred years, for which the design is made at lower allowable stresses and in somewhat different combinations of conditions.

For some special large long span bridges it is conceivable that a double earthquake design approach might be desirable or appropriate, depending upon the approach employed in design. For the majority of bridge structures it is believed adequate and desirable to employ a single earthquake criterion as the design hazard basis, much as was done in Ref. 10.

EARTHQUAKE GROUND MOTIONS

Regional Motions

In general, two procedures are available to define the earthquake hazard. In the first, where there is an extensive history of earthquake activity, and geologic and tectonic investigations are feasible, estimates can be made of the possible magnitude and location of future earthquakes affecting a site. In some cases such earthquakes will occur along well defined faults. One can then make estimates of the earthquake motion intensity propagated to the site, taking into account the experimental and observational data available for this purpose, along the lines described in Refs. 11 and 12.

The second procedure for developing the earthquake hazard in a region is used when occurrence of earthquakes is not generally associated with surface faulting, or when insufficient data are available from records and observations. Under these conditions, relationships have been developed for correlating ground motions, generally maximum velocities or maximum accelerations, to a qualitative measure of the intensity of motion. In the U.S.A. the measure of intensity that is used is the "Modified Mercalli Intensity." Although these relations do not appear to be as readily subject to mathematical determination as the relations for earthquake shock propagation, there are sufficient observations to permit useful probabilistic data to be obtained. Such data are summarized in Refs. 13, 14 and 15.

Site Amplification

The regional motions that one derives from the methods just described must be modified to take account of the geologic and stratigraphic conditions pertaining to the site. Although there has been a great deal of study and research involved in this topic, it still must be considered a controversial matter. Nevertheless, it is clear from observations that the type of soil or subsoil has a major influence on the motions that are recorded. In general, for the same earthquake, where the intensity is low (possible maximum acceleration of gravity) the measured accelerations are generally higher on sediments than on rock. However, when the acceleration is high (greater than 0.2 g) then the accelerations measured on rock appear to be higher than those on soil. In most instances the measured velocities are nearly the same. Studies of the nature of the motions on sites of different stiffnesses are summarized in Refs. 16, 17 and 18 in terms of the so-called "response spectra" applicable to the measured records at various sites.

Although analytical methods have been proposed purporting to explain phenomena such as those described in the references previously cited, in most cases these analyses consider a condition not representative of actual conditions. The principal assumption (that the earthquake motions consist of horizontal shear waves propagated vertically upward from some base layer where the motions are defined) is contrary to observations. For example, it is shown in Ref. 19, and it has long been believed by others, that for longer period motions, possibly where the periods are one second or longer, the motions are primarily due to surface waves such as Rayleigh waves or Love waves. It is quite likely, however, that for moderate distances, beyond those corresponding to the depth of focus, surface waves have an important effect even for higher frequencies or shorter period motions, and more complex motions must be considered other than those due to horizontal shears propagated vertically upward. Moreover, vertical motions cannot be accounted for by the simple horizontal shear wave model.

Although peak values of ground motion may be assigned to the various magnitudes of earthquake, especially in the vicinity of the surface expression of a fault or at the epicenter, these motions are in general considerably greater than smaller motions which occur many more times in an earthquake. Design earthquake response spectra are based on "effective" values of the acceleration, velocity and displacement, which occur several times during the earthquake, rather than isolated peak values of instrumental reading. The effective earthquake hazards selected for determining design spectra may be as little as one-half the expected isolated peak instrument readings for near earthquakes, ranging up to the latter values for distant earthquakes.

Relative Motions

It is often important to be able to estimate the relative motion of two points some distance apart in connection with the design of structures supported on or in soil or rock, as for example a bridge and its supports. The source of such motion may arise from ground wave motions or fault related motions. Information about relative motions, as determined from measurements, is often difficult to interpret or assess. It is hoped that time-synchronized field measurements can be made in the future over varying distances to provide a basis for helping estimate relative displacements.

Ground Wave Motion -- Where it is relatively clear that a wave motion is propagated in one direction without interference with other waves in other directions, and where the change in shape of the wave from point to point is relatively small (implicitly this suggests that the distance between points considered is not large), one can make inferences about the relative motions between nearby points in a fairly simple manner as noted earlier by Newmark in Ref. 20. In summarizing the relationships given therein consider two points, point 1 and point 2, at a distance b apart, as shown in Fig. 1, and displacement ρ at point 1 and ρ plus an increment at point 2, as indicated. For forward wave motion of the form given by $\rho = f(x-ct)$, where c is the velocity of this particular wave propagation and t is time, it can be shown that the following relationships exist.

$$\frac{\partial\rho}{\partial x} = -\frac{1}{c}\frac{\partial\rho}{\partial t} \tag{1}$$

and

$$\frac{\partial^2\rho}{\partial x^2} = \frac{1}{c^2}\frac{\partial^2\rho}{\partial t^2} \tag{2}$$

In the case where ρ is in the direction of x, the strain ε is obtained from Eq. (1), and the maximum strain is as follows,

$$\varepsilon_m = -\frac{v_m}{c} \tag{3}$$

where v_m is the maximum particle velocity at point 1.

Similarly in the case where ρ is perpendicular to x, either horizontally or vertically, the maximum curvature at point 1 is given by the following expression,

$$\text{curvature} = \frac{a_m}{c^2} \tag{4}$$

where a_m is the maximum acceleration at point 1.

These relations in Eq. (3) and (4) are often employed to determine the maximum strain that might be experienced by an element extending over some limited distance, or the maximum curvature in an element, as for example a tunnel, pipeline, or other structure.

To provide some indication of the meaning of these expressions, and some perspective as to the order of magnitude of the displacement values, particularly Eq. (3), assume one had a peak ground velocity, v_m, of 60 cm/sec (24 in./sec), which would not be unlikely with a peak ground acceleration of 490 cm/sec^2 (0.5 g), and a velocity of wave propagation c, of 300 m/s (1000 ft/sec), and assume further that the average strain over some distance was about one-half that given by Eq. (3). Then,

$$\varepsilon_{avg} = \frac{60}{2 \times 300 \times 100} = 1 \times 10^{-3} \tag{5}$$

Over a span distance of 100 meters (328 ft), for example, one finds a relative displacement of 0.1 m or 4 in., which obviously is of design significance.

Other relationships are of importance in the case where the motions are caused by more general disturbances than a wave of nearly constant shape transmitted in one direction. For example, it is apparent that the maximum change in the distance between points 1 and 2, δb_{21}, is related to the maximum displacements at points 2 and 1 in the following way:

$$\delta b_{21} \geq u_{max,2} - u_{max,1}. \tag{6}$$

In many instances, this relation may be trivial because the maximum displacements may be nearly equal, but since they do not occur at the same time, it is obvious that the maximum transient change in length must be greater than the difference in the maximum displacements. It is, however, true that the maximum change in length is less than the difference between the maximum displacement at either point 1 or point 2, less the minimum displacement, or the displacement in the opposite direction, at the other point. The minimum displacement would of course be zero, if the displacements do not reverse in direction. This relation is expressed as follows:

$$\delta b_{21} \leq |u_{max,1,2} - u_{min,2,1}|. \tag{7}$$

Similarly, the maximum change in length between points 2 and 1 must be less than the maximum strain anywhere along the line connecting the two points, multiplied by the length, as given in the following relation:

$$\delta b_{21} \leq \varepsilon_{max} b. \tag{8}$$

For the special case where the maximum strain is related to the maximum velocity by Eq. (3), corresponding to a wave transmission situation, then one can derive from the preceding equation the following result:

$$\delta b_{21} \leq \left| (b/c)\ v_{max,2,1} \right|. \tag{9}$$

Fault Displacements -- In a similar manner, Newmark and Hall (Ref. 21) have shown that for total fault motion D, where the angle between the plane of the fault and pipe (bridge) axis is ϕ, for a distance between support points of L, the average strain can be expressed as follows

$$\varepsilon \simeq \frac{1}{8} \left(\frac{D}{L}\right)^2 + \frac{1}{2} \frac{D}{L} \cos \phi \tag{10}$$

If L/D is greater than 10 as would commonly be the case for a bridge, the first term in Eq. (10) is negligible. In order to gain some appreciation for this contribution to relative motion, for a span length of 100 m (328 ft) cutting across a fault experiencing a movement of 1 m (3.28 ft) at 45 degrees, one finds

$$\varepsilon \simeq \frac{1}{2} \frac{1}{100} \cos 45^\circ = 3.5 \times 10^{-3}$$

The relative displacement would be this strain times 100 m or 0.35 m or 14 in. Again, this value is obviously of great design significance, if one knew the bridge crossed a fault. If the bridge were aligned more nearly perpendicular to the fault the displacement would be less and if more nearly parallel to the fault it would be greater.

Ground Rotation -- In the same manner one can explore the possible contribution of rotation.

Assume for example, that a horizontally travelling wave in the vertical plane, in the absence of any other ground motions, had the form of a sine wave, as given by

$$y = y_m \sin \frac{\pi x}{\ell}.$$

where y_m is the peak value of vertical displacement, x is the distance along the wave and ℓ is the half wave length. The maximum slope is given by the expression

$$\theta = \frac{d_y}{d_x} = \pi \frac{y_m}{\ell} \cos \frac{\pi x}{\ell}$$

As an example, if ℓ is taken as 500 m (1600 ft), admittedly short (corresponding to a 0.5 Hz wave motion at 1600 fps), and y_m is 0.5 m (1.6 ft), one obtains for the maximum rotation in one direction

$$\theta = 3.14 \times \frac{0.5}{500} = 3.14 \times 10^{-3} \text{ rad.}$$

For a pier 5 m (16 ft) high the rotation produces a displacement of 0.016 m (0.6 in.) and for a pier 30 m (98 ft) high a displacement of 0.09 m (4 in.). The total displacement from rotation in both directions could be twice as great. For shorter wave lengths or higher piers the effect will be even more pronounced.

The point to the foregoing discussion on relative motions is to illustrate that relative motions should be a consideration in bridge design generally irrespective of whether or not fault motions may be involved. In many cases stream beds follow displacement zones and therefore the bridge may well cross a fault area. In any event, it should be obvious that the longer the span or the higher the piers the more important relative motion considerations become. Additional research on the topic is badly needed, especially field measurements which can be compared to theoretical considerations. And, provisions to account for relative motions along with the normal inertial effects need to be introduced into bridge design guides.

SOIL STRUCTURE INTERACTION

When a structure is founded on a base of soil and/or rock, it interacts with its foundation. The forces transmitted to the structure and the feedback to the foundation are complex in nature, and modify the free-field motions. Methods of dealing with soil-structure interaction have been proposed by a number of writers: (1) involving procedures similar to those applicable to a rigid block on an elastic half space; and (2) finite element or finite difference procedures corresponding to various forcing functions acting on the combined structure-soil complex; and (3) substructure modelling techniques which may or may not include use of the finite element method.

However one makes the calculation, one determines a fundamental frequency and higher frequencies of the soil system which interact with the structure, and effective damping parameters for the soil system taking into account radiation and material damping. Both of these quantities are necessary in order to obtain rational results. Procedures that emphasize one but not the other cannot give a proper type of interaction.

With regard to the design of bridges this topic has not received much attention and may well be of major importance only for quite massive bridges or large long span bridges. For relatively short light bridges the interaction may be minimal in terms of significant effect upon the response. For other bridge systems where retaining walls are involved, or bridges with deep foundations or foundations in soft sediment, the soil-structure interaction effects may be significant.

DAMPING AND ENERGY ABSORPTION

Implications of Damage or Collapse and Modelling Considerations

In considering the response of a structure to seismic motions, one must take account of the implications of various levels of damage, short of collapse, of the structure. Some elements of bridges must perforce remain elastic or nearly elastic in order to perform their allocated function. However, in many instances, a purely linear elastic analysis may be unreasonably conservative when one considers that, even up to the near yield point range, there are non-linearities of sufficient amount to reduce required design levels considerably.

It should be appreciated that the problem of modelling a bridge for analysis purposes can be difficult for in many cases the superstructure is not firmly anchored to the pier and can experience motion. In such cases it is not possible to use without modification the routine methods of analysis commonly employed for building structures. This point is discussed further under methods of dynamic analysis. In some cases in addition to damping and nonlinear material behavior the structural system experiences frictional type resistance to motion, or is fitted with devices to give controlled mechanical energy absorption. There is a pressing research need to investigate common forms of vibration dissipation in bridges along with techniques for handling these approximately in analysis.

Damping -- Energy absorption in the linear range of response of structures to dynamic loading is due primarily to damping. For convenience in analysis, the damping is generally assumed to be viscous in nature and is so approximated. Damping levels have been determined from observation and measurement but show a fairly wide spread.

Damping is usually considered as a proportion or percentage of the critical damping value, which is defined as that damping in a system which would prevent oscillation for an initial disturbance not continuing through the motion. For conservatism, damping values used in design of nuclear plants are generally taken at lower levels than the mean or average estimated values. For convenience, the damping associated with particular structural types and materials as modified slightly from Ref. 22 is given herein in Table 1. The lower levels of the pair of values given for each item is considered to be nearly a lower bound, and is therefore highly conservative; the upper level is considered to be an average or slightly above average value, and probably is the value that should be used in design when moderately conservative estimates are made of the other parameters entering into the design criteria. For normal bridge design the upper set of values in each case in Table 1 is recommended for use.

Ductility -- Energy absorption in the inelastic range is commonly handled through use of the so-called "ductility factor." In general, the ductility factor is defined as the ratio of the maximum useful displacement or the intended design level of deformation to the "effective" value of the elastic limit displacement. Both of these quantities should be determined from an elasto-plastic approximation to the actual displacement-resistance function for the element or the structural system. The approximation requires that the energy absorption be the same for the true and the approximate curves at two points: (1) the "effective" (but not the real) yield point or elastic limit, and (2) the maximum displacement, as illustrated in Fig. 2. Accordingly it is important that the expected resistance function be known for the responding element, both in terms of arriving at the approximate resistance function and in order to be able to assess the margin of safety existant at the maximum deformation level. In some cases in design it requires considerable effort to develop such resistance functions and to be sure that the strength and deformation can be achieved with a sufficient margin realistically. Obviously, local ductility can exceed the "effective" member ductility, and such local ductility may require special consideration from the standpoint of member integrity.

In general the ductility level that is appropriate for use in design in nuclear plants is of the order of 1.5 to 2.5 for concrete loaded heavily in shear or compression; from 2 to 5 for concrete loaded primarily in flexure; from 2.5 to 10 for steel loaded primarily in tension or flexure; and from 1.5 to 6 for steel loaded primarily in compression, with the lower range of values corresponding to members that buckle or wrinkle at or below yielding levels of axial stress. In general one must be careful to employ realistic values. For example, if one is working with a responding member whose behavior can be classed as "stiff" or "brittle", as for example a concrete element that may fail in shear, the ductility level would be expected to be low. Ductility values falling in the same general range would be expected to be applicable to bridge design.

Connections of members that yield should be sufficiently stronger than the member to make yielding or failure occur in the member rather than in the connection. If for some special reason significant deformation needs to be accomodated in the connections then the connection will require special design consideration.

Ductility and Strength of Piers and Abutments

For some time as a result of careful study of earthquake damage to bridges it has been recognized that the strength and ductility of piers needed special attention. The recent studies in New Zealand (Refs. 23, 24, 25, 26) in this area help address this requirement and clearly illustrate the need in design to consider such factors as concrete confinement and type of loading (including interaction) on the pier performance. Experimental data for massive piers will be hard to develop and probably can be obtained only from careful design and later observation in earthquakes.

Of equal importance will be the performance of the pier-footing or pier-piling structural system as a unit. Some damage survey studies suggest that the poor pier performance arises at least in part from failure of the foundation interface when in fact if properly designed, in some cases taking into account backfill pressures, the support unit would have offered adequate supporting capability. This observation is made to suggest that the entire foundation system be studied for strength and deformation capability, including torsional aspects if required.

METHODS OF DYNAMIC ANALYSIS

Response Spectra -- In general, concepts of the response spectrum and its use in dynamic analysis is discussed in some detail in Refs. 27, 28, 29 and others in the list of references. The response spectrum is defined as a graphical relationship of maximum response of a single-degree-of-freedom elastic system with damping to dynamic motions (or forces). The most usual measures of response are maximum displacement, which is a measure of the strain in the spring element of the system, maximum pseudo relative velocity which is a measure of the energy absorption in the spring of the system, and maximum pseudo acceleration which is a measure of the maximum force in the spring of the system.

For a multi-degree-of-freedom system each degree of freedom is treated as an equivalent single-degree-of-freedom system and the responses are compounded by the methods described in the several references cited above. Either one of two methods of compounding are commonly used: (1) an upper bound obtained by taking the sum of the numerical values of the individual response values considering the participation of the various modes in the overall response; (2) a probabilistic determination obtained by taking the square root of the sum of the squares of the individual modal responses.

Time-History -- Alternatively one may make a calculation of response by considering the motions to be applied and the responses computed using a step-by-step numerical dynamic analysis. This implies a deterministic approach since a deterministic time-history is involved. By use of several time-histories independently considered, one can arrive at average or conservative upper bounds of response, at the expense of a considerably increased amount of calculation. In general, however, there is no real advantage in using a time-history as compared with a response spectrum approach for multi-degree-of-freedom systems, unless one is faced with an actual deterministic input.

Motions in Several Directions -- In the real world, earthquake motions occur as random motions in many directions. In other words one can consider a structure to be subjected to components of motion in each of two perpendicular horizontal directions and the vertical direction, and one might also consider three components of rotational motion corresponding to a foundation twist about a vertical axis and two rocking motions about the horizontal axes. These ground motions have, apparently, nearly statistical independence. Consequently if one used time-histories of motion one must either use actual earthquake records or modify them in such a way as to maintain the same degree of statistical independence as in actual records. Consequently, for time-histories that involve inelastic behavior, it is an oversimplification to consider each of the components of motion independently since they all occur at the same time in general. However, there is only a small probability of the maximum responses for each component of motion occurring simultaneously and methods have been derived for handling problems such as this. A summary of these is discussed under design procedures.

RECOMMENDED DESIGN PROCEDURES

Design Philosophy and Methods of Analysis

In the case of bridges most of the mass resides in the superstructure, and with the exception of moderate to highly seismic regions it is expected that the design of the superstructure would be controlled by dead load, live load, and thermal considerations as well as overall performance criteria (for example, general flexibility under traffic loading). Similarly the piers and abutments must be designed to support the massive superstructure and its attendent loads. However in the event of significant seismic excitation the system must be able additionally to perform in a much different sense, that is the piers and abutments as well as the superstructure must be designed to accommodate the generated inertial forces while at the same time the system must be able to accommodate the relative motions (impressed distortions, which may be large, and remain intact (not drop spans). Such performance calls for careful design of the piers, abutments and foundations especially, and for careful attention to the supporting or connecting details between the piers and superstructure, and provisions for accommodating the seismic forces and displacement generated throughout the entire system.

The modelling of bridges for seismic analysis is an extremely difficult problem in the light of the above discussion. The modelling obviously needs to represent as well as possible the actual structural system and its connections, and yet model the loading and resistance parameters as realistically as possible. This matter of modelling and analysis is one needing extensive research.

Whereas complicated models and analysis of bridges may be required for special situations, the goal for routine design is to arrive at simple models which can be used with rational yet explicit approximations to loading or motions, as well as resistance, in the light of acceptable performance criteria. An examination of codes, specifications and research reports to date suggests there is much work yet to be done in this important area.

Modified Response Spectra

Modified response spectra representing average (or some probability above the mean) conditions for earthquake motions are discussed in various papers, including for example Refs. 8 and 29. In general it has been shown that a response spectrum for a particular cumulative probability level can be derived from statistical studies of actual earthquakes, most conveniently as a set of amplification factors applied to the maximum components of ground motion. The probability function which best describes the range of values is one that corresponds to a logarithmic normal distribution. The amplification factors are functions of damping. Equations or tables of values for the amplification factors for the log normal distribution, for both the median or 50 percentile cumulative probability level and the one sigma or 84.1 percent cumulative probability level are given in Refs. 9 and 29.

Effects of Inelastic Action and Design Spectra

The effects on the response of a structure deforming into the inelastic range have been described and/or summarized in Refs. 8, 22 and 29. In general, for small excursions into the inelastic range, when the latter is considered to be approximated by an elasto-plastic resistance curve, the response spectrum is decreased generally by a factor which is one over the ductility factor. If the ductility factor is defined by the symbol μ then the reduction for the low and intermediate frequency portions of the elastic response spectrum (below about 1.5 Hz) is reduced by the factor $1/\mu$ for acceleration. The reduction is made by a factor of $1 \sqrt{2\mu - 1}$ in the higher frequency portions, roughly between 2 and 8 Hz. There is no reduction beyond about 33 Hz. With this concept, one can arrive at design spectra that take account of inelastic action even in the small range of inelastic behavior.

Using the concepts described above, the design spectrum for earthquake motions can be drawn as explained in Refs. 8, 9, 22 and 29 and as illustrated in Fig. 3.

Combined Effects of Horizontal and Vertical Excitation

For design one must consider the combined effects of motion in various directions. Although this can be done in various ways depending upon the method of analysis used, normally it is reasonable to use the response spectrum approach even for multi-degree-of-freedom systems, to arrive separately at the responses in the individual directions, and then to combine the effects in general by taking the square root of the sums of the squares of the individual effects for stress or motion at a particular point in a particular direction for the various components of motion considered. It is considered conservative and simpler, and much more readily defined and calculated, to take the combined effects as 100 percent of the effects in one particular direction and 40 percent of the effects corresponding to the two directions of motion at right angles to the principal motion considered. It is this combination that is recommended for general use.

Relative Motions

In addition to the foregoing approaches for handling inertial effects, the system must be designed to permit accommodation of the relative motions arising from ground motions or fault motions as discussed earlier herein.

Quality Control and Details of Construction

Items which do not lend themselves readily to analytical consideration may have an important effect on the response of structures and facilities to earthquake motions and must be considered in the design. Among these items are such matters as the details and material properties of the elements and components, and the inspection and control of quality in the construction procedure. The details of connections of the structure to its support or foundations, as well as of the various elements or items within the structure or component, are of major importance. Failures often occur at connections and joints because of inadequacy of these to carry the forces or to permit the deformation to which they are subjected under dynamic conditions. Inadequacies in properties of material can often be encountered, leading to brittle fracture where sufficient energy cannot be absorbed, even though energy absorption may have been counted on in the design and may be available under static loading conditions.

Similarly, because bridge structures are exposed openly to the environment, it is necessary to carry out a continuing program of inspection and maintenance to be sure that supports, expansion joints, snubbers, etc. remain fully operational throughout the life of the bridge.

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TABLE 1 RECOMMENDED DAMPING VALUES

Stress Level Combined		Percentage Critical Damping
Working stress, no more than about 1/2 yield point	Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2 to 3
	Reinforced concrete with considerable cracking	3 to 5
	Bolted and/or riveted steel, wood structures with nailed or bolted joints	5 to 7
At or just below yield point	Welded steel, prestressed concrete (without complete loss in prestress)	5 to 7
	Prestressed concrete with no prestress left	7 to 10
	Reinforced concrete	7 to 10
	Bolted and/or riveted steel, wood structures with bolted joints	10 to 15

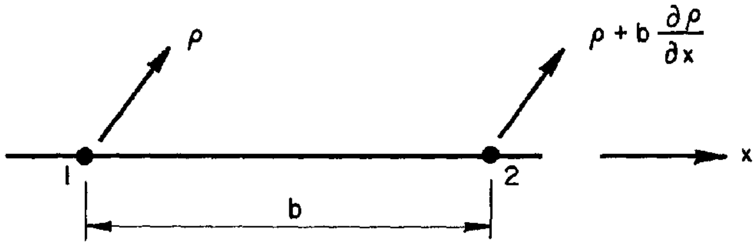

FIG. 1 RELATIVE DISPLACEMENTS

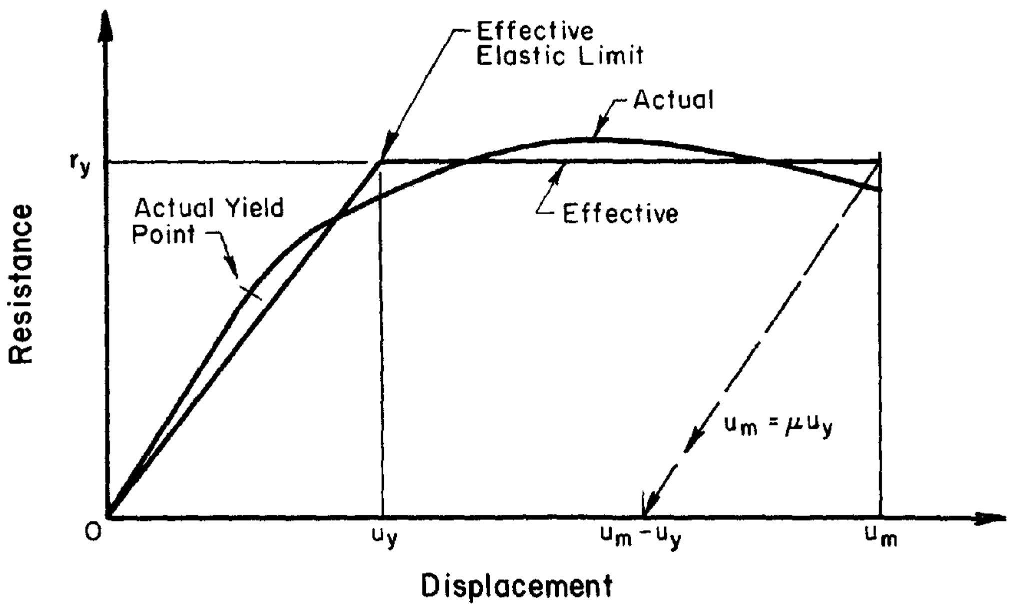

FIG. 2 RESISTANCE-DISPLACEMENT RELATIONSHIP

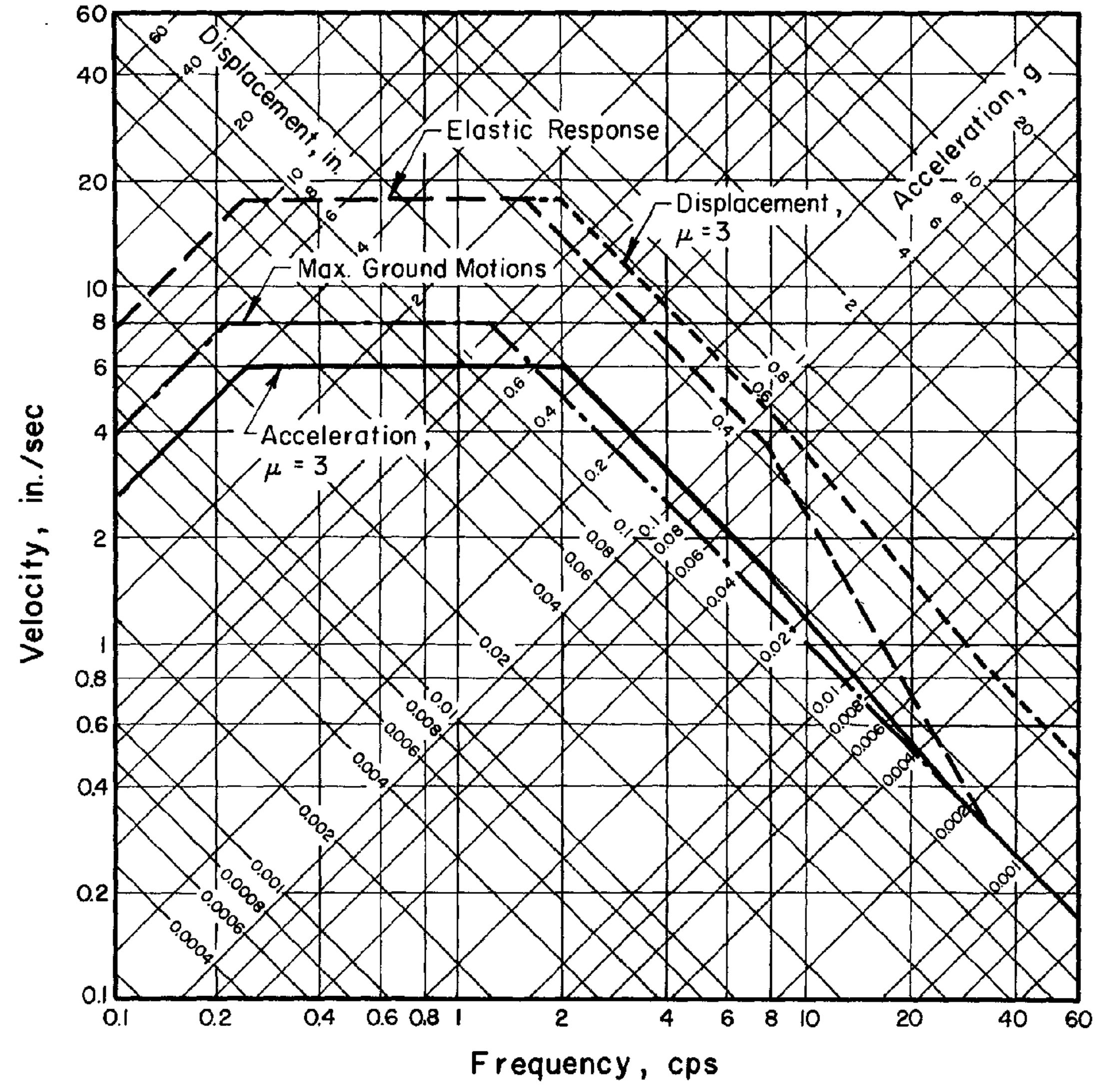

FIG. 3 ELASTIC AND INELASTIC DESIGN SPECTRA

TYPICAL CONFIGURATIONS OF BRIDGES IN THE UNITED STATES

by

James R. Libby, President, James R. Libby and Associates
San Diego, California

and Walter Podolny, Jr., Structural Engineer, Federal Highway Administration
Washington, District of Columbia

ABSTRACT

The details of the more commonly used bridge types found in the United States are described with respect to the superstructure, substructure and bearings between superstructure and substructure. The description of typical details is followed by a discussion of how the more commonly used bridges might be expected to perform under longitudinal and transverse ground shaking. The discussion also includes a description of details frequently found in parts of the country where earthquakes are more frequently experienced and hence, greater emphasis is placed on designing earthquake-resistant structures. It is concluded that many bridges of the more common type have low resistance to earthquake damage. Details of construction commonly used in the areas of higher seismicity are described and is concluded that details of the types used in these areas do result in structures with higher resistance to the effects of ground shaking.

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INTRODUCTION

The task of describing typical bridge configurations used in the United States is not a simple one and in the true sense may be impossible. This can be explained by the fact that the majority of bridges in this country are designed by the Bridge Design Sections of the 50 different state Highway Departments. These Bridge Design Sections operate independently of each other with their only significant tie being that all of them use the Standard Specification for the Design of Highway Bridges published by the American Association of State Highway and Transportation Officials as their basic design criteria.(1) Most have additional criteria and standards developed by their own organizations which either supplement or supercede portions of the provisions contained in the AASHTO Standard Specifications. Each bridge design group has developed methods and details of their own; details which vary widely from state to state for bridges of the same basic type. Because of this, the "typical configurations" described in this paper must be understood to be typical by general classification but regionally typical in detail.

The most commonly used materials of construction in U.S. bridges are concrete and steel yet wood is still used in some regions, notably Alaska. Although substructure construction is generally done in reinforced concrete, abutments and piers of steel and wood are being used in new bridges. Superstructures normally are provided with reinforced concrete decks which may or may not be composite with the beams. Timber decks are still used in some regions. The longitudinal beams or girders in bridge superstructures normally consist of reinforced concrete or steel. Occasionally timber or glued-laminated wood beams are still used in some regions.

This paper is confined to the consideration of the details of the more commonly used bridge types with the emphasis being placed on bridges constructed in areas of assumed moderate to high seismic risk. Only structures composed of concrete substructures with steel or concrete superstructures are considered.

It is customary among bridge engineers in the United States to refer to bridge bearings or bearing devices as being "fixed" or "expansion". Fixed bearings are devices intended to allow the end of the supported beam to rotate in the direction of the span only and not permit translation (displacement) either longitudinally nor transversely. Expansion bearings are devices designed to permit rotation and translation in the direction of the span and allow neither rotation nor translation (of significant magnitude) transversely. Fixed and expansion bearings are used to support simple or continuous superstructure members (beams, girders, trusses, etc.) at abutments and piers (or bents) where it is desired to prevent the superstructure from inducing major shears and moments in the substructure. They can be thought of as "hinges" and "roller-supported hinges" respectively.

Another type of support for bridge superstructures, which is termed "built-in" in this paper, consists of connections between superstructure and substructure that transmit forces and moments between the two and which force them to rotate and deflect equally both longitudinally and transversely at the point of connection. A monolithic joint between the top of a pier and the deck of a cast-in-place reinforced concrete bridge is an example of a built-in bearing or connection.

Long superstructures with built-in connections often require a joint in the superstructure at one or more points between the abutments in order to control the forces and moments that would be induced in the structure by temperature, concrete shrinkage and creep if the joints were not provided. Joints of this type generally are designed to act as expansion bearings in that they allow rotation and translations in the direction of the span but do not permit movements or rotations in the transverse direction. In some instances, they act as fixed bearings and do not allow longitudinal translation. In some regions of the country, joints of this type are referred to as "hinges" but this is a misnomer if they act as expansion bearings and the term "in-span joints" will be used in referring to this type of joint in this paper. With this terminology, one can further describe the bearings as in-span expansion joints or in-span fixed joints.

SUPERSTRUCTURE TYPES

The most commonly used bridge superstructure types can be catagorized as follows:

1. Steel stringer bridges
2. Precast prestressed concrete stringer bridges
3. Cast-in-place post-tensioned or reinforced concrete bridges

Each of these are used in simple and continuous spans and are described further in the following paragraphs.

A typical example of a multi-span bridge having simply supported steel stringers is shown in Figs. 1, 2 and 3. Bridges of this type are normally constructed with cast-in-place reinforced concrete decks as shown in Fig. 2. They are provided with fixed and expansion bearings and have the backwall type of abutment as shown in Fig. 3. Note that the letters E and F are used in Fig. 1, as well as in other illustrations in this paper, to denote expansion and fixed bearings, respectively.

Steel stringer bridges are also made with the stringers continuous over the interior supports as shown in Fig. 4. When made continuous, provision is made for thermal expansion and contraction by providing fixed bearings at some supports and expansion bearings at others. In some instances, in-span joints, which provide vertical support and act as either fixed or expansion bearings, are provided as shown in Fig. 5. Continuous bridges of this type are commonly used in current bridge construction.

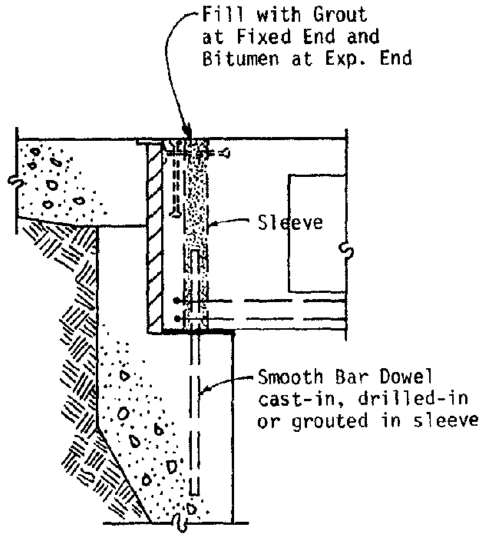


Fig. 7 CROSS SECTION AT BACKWALL-TYPE ABUTMENT OF PRECAST CONCRETE STRINGER BRIDGE (FROM REF. 4)

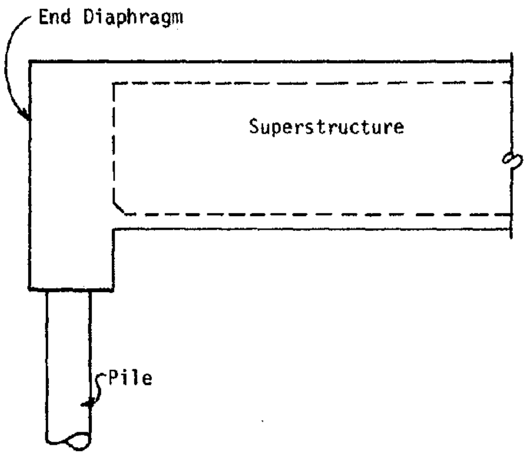


Fig. 8 END DIAPHRAGM-TYPE ABUTMENT

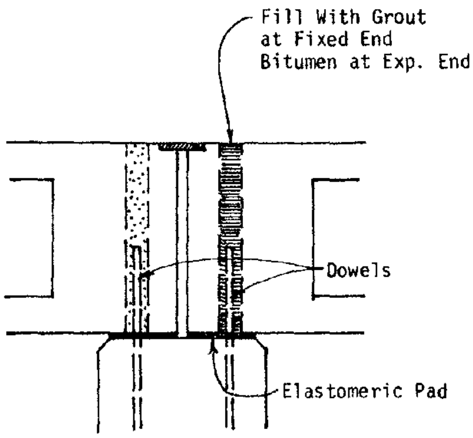


Fig. 9 CROSS SECTION AT PIER WITH PRESTRESSED CONCRETE STRINGERS (FROM REF. 4)

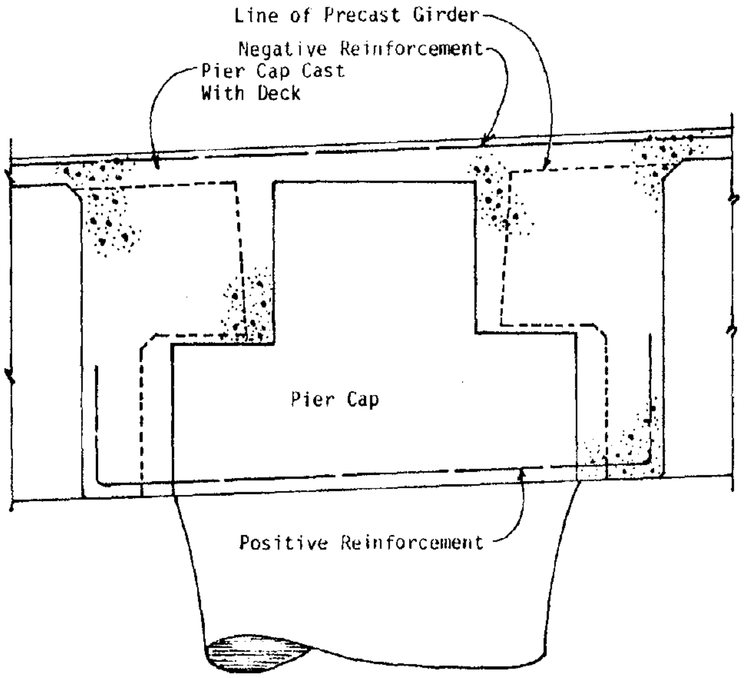


Fig. 10 CROSS SECTION OF BUILT-IN CONNECTION WITH PRECAST STRINGERS

Cast-in-place concrete bridges have been widely used throughout the country but probably to the greatest extent in the western states. This is the most commonly used bridge type in California; perhaps in other western states too. Bridges of this type have superstructures which can be described as slab, T-beam, reinforced concrete box girder and post-tensioned box girder in order of increasing span lengths. Multi-span superstructures of these types are normally continuous and usually have built-in connections with the substructure. They are, however, being extensively constructed with fixed and expansion bearings as described above rather than with monolithic connections. Typical cross sections for T-beam and box girder bridge superstructures are shown in Figs. 11 and 12.

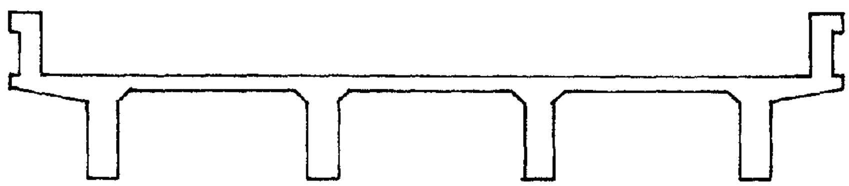


Fig. 11 CROSS SECTION OF A T-BEAM BRIDGE (FROM REF. 3)

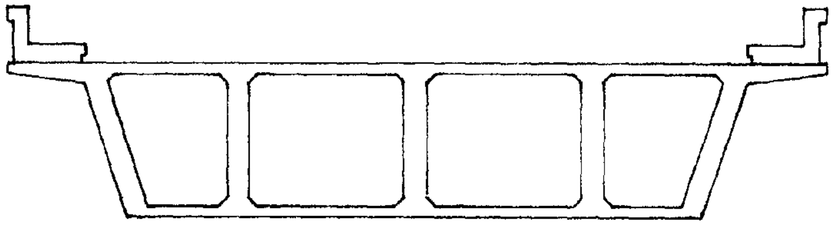


Fig. 12 CROSS SECTION OF BOX GIRDER BRIDGE (FROM REF. 3)

ABUTMENT TYPES

Backwall abutments, as shown in cross section in Fig. 3, have probably been the most commonly used type of abutment. This abutment is characterized by the fact that it is not dependent upon the superstructure for stability. Loads from the superstructure; vertical, transverse and longitudinal, are transmitted to the abutment from the superstructure. The configuration of the abutment is determined by the designer so as to assure stability under all conditions of vertical and lateral loads. Backwall abutments normally are provided with fixed or expansion bearing devices between the abutment seat and the superstructure. Longitudinal earthquake forces may be transferred between superstructure and abutment through fixed bearings or through shear keys provided expressly for this purpose.

End diaphragm abutments of three types are shown in Figs. 8, 13 and 14. The first two types are commonly used in California and are characterized by having the end diaphragm, which is cast monolithically with the superstructure, act as an earth-retaining wall as well as to transmit the vertical end reactions directly into the foundations. In the case of the pile supported abutment of Fig. 8, there are no special provisions for movements from temperature changes, concrete shrinkage or creep; hence, this detail is restricted to bridges of relatively short length. Experience in California has shown that pile supported end diaphragm type abutments perform well with conventional reinforced concrete superstructures up to 122 meters (400 feet) long. The detail shown in Fig. 13 provides for longitudinal shortening of the superstructure through the sliding base detail and is used in longer structures and with post-tensioned superstructures. The third end diaphragm type abutment is a detail that has been used in other western states (Washington and Alaska) with stringer bridge superstructures and is characterized by the end diaphragm, which acts as a retaining wall, being connected monolithically with the superstructure but yet being free to move with respect to the foundation. In the case of the end diaphragm abutments, longitudinal earthquake-induced forces are transmitted to the soil behind the end diaphragm by passive pressure without having to pass through bearing devices.

PIER TYPES

Three types of piers or bents are most commonly used. These are single column, multi-column and wall piers, Figs. 15, 16 and 17. There is no definite transition point at which a relatively wide single column pier becomes a relatively high wall pier. The question of how to define a wall and a column remains unanswered. Single column piers can be a part of a longitudinal frame

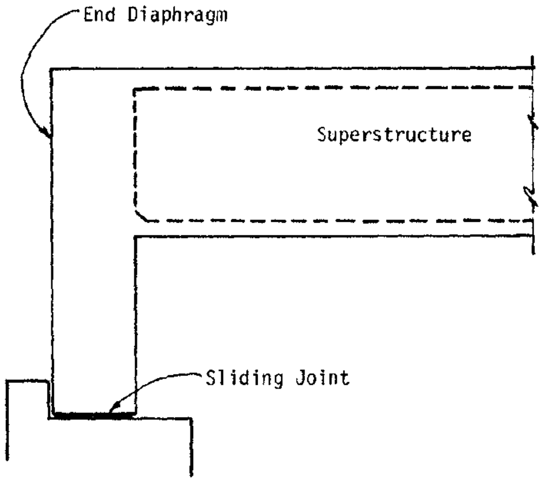


Fig. 13 END DIAPHRAGM-TYPE ABUTMENT WITH SLIDING BASE

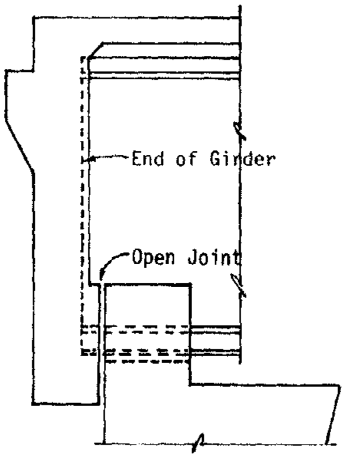


Fig. 14 END DIAPHRAGM-TYPE ABUTMENT

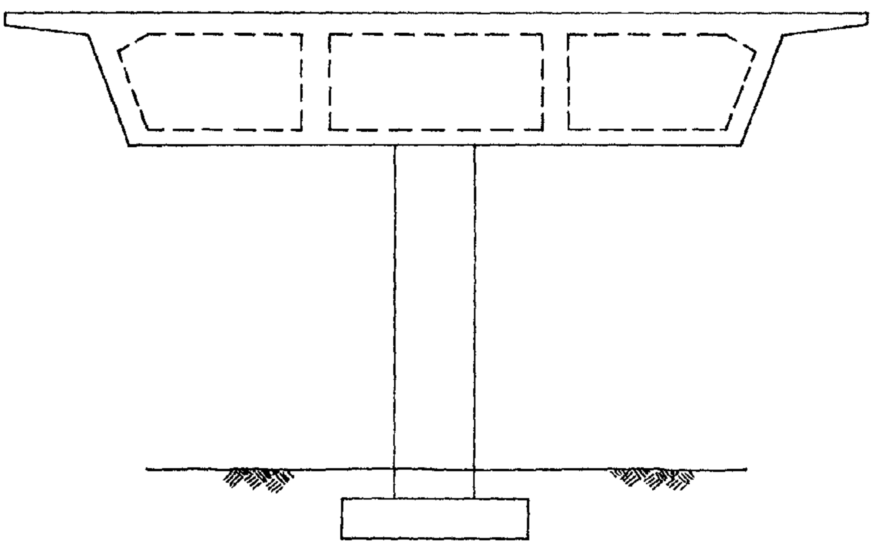


Fig. 15 ELEVATION - SINGLE COLUMN PIER WITH FLUSH CAP

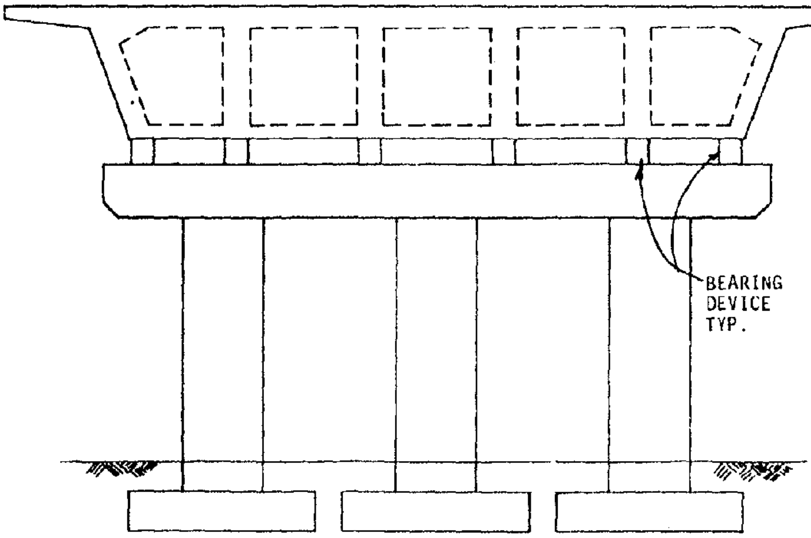


Fig. 16 MULTI-COLUMN PIER

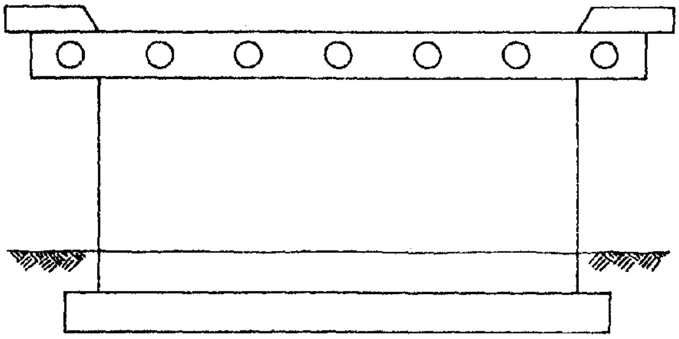


Fig. 17 WALL PIER SUPPORTING SLAB-TYPE BRIDGE SUPERSTRUCTURE

as shown in Fig. 18. If hinged at the top as shown in Fig. 19, the frame does not have as much redundancy as do those having built-in connections. In spite of this, many existing and new bridges are constructed with hinges at the tops of the piers. When loaded transversely, single column piers are restrained against rotation at their tops by the torsional stiffness (if any) of the superstructure and they have low redundancy. Single column piers have been used extensively in the construction of major highways in urban areas because they occupy less space than other types of piers, are attractive (at least are not unattractive) and facilitate construction details for structures on horizontal curves or which are on skews in order to be compatible with objects over which they are spanning. Single column piers or piers are generally used with caps which do not extend below the soffit of the superstructure (flush caps) as shown in Fig. 15.

The principal advantage of multi-column bents or piers over those having a single column is that they can be made into in-plane frames; this is an advantage when considering transverse loading as well as energy-absorption and redundancy under earthquake-induced forces. They are not as convenient as

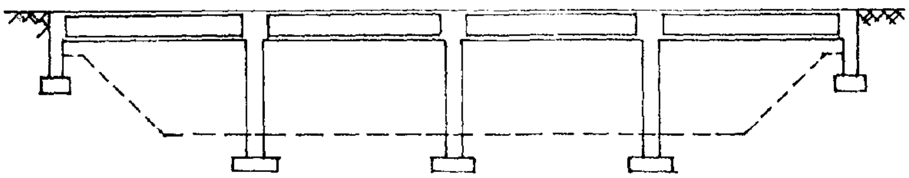


Fig. 18 LONGITUDINAL CROSS SECTION BOX GIRDER BRIDGE WITH BUILT-IN CONNECTIONS AT TOP OF PIERS AND DIAPHRAGM TYPE ABUTMENTS

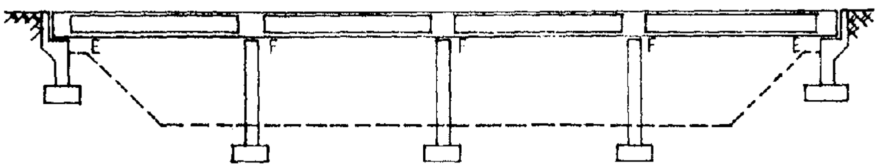


Fig. 19 LONGITUDINAL CROSS SECTION BOX GIRDER BRIDGE WITH FIXED BEARING AT TOP OF PIERS, AND EXPANSION BEARINGS AT THE ABUTMENTS

single column piers in interchange structures but are widely used in bridges of all types. Multi-column piers are easily made with dropped caps as well as with flush caps. They can be made into longitudinal frames by the provision of built-in connections with the superstructure.

Wall piers are frequently used in crossings of waterways or railroads. For in-plane lateral loads, wall piers act as stiff shear walls and have low ductility. Under loads normal to the plane of the walls, the walls can only behave in a ductile manner if they have vertical reinforcing confined in closely spaced ties or spirals; details not normally provided in wall piers.

BEARING DEVICES

Bearing devices of various types are used to transfer vertical and horizontal loads between bridge superstructures and substructures. They are also used to prevent restraint and restraint-induced loads. As explained above, fixed bearings permit the supported superstructure to rotate in the direction of the span and prevent transverse or longitudinal translation. Expansion bearings perform the same functions as fixed bearings except they are designed to move longitudinally to accommodate length changes due to temperature variations and other effects.

Many types and configurations of bearing devices have been used over the years; most are still found in use in existing structures and new ones of most types are put into service annually. The most common types of bearing devices can be classified broadly as metallic, elastomeric or concrete. Several types under these broad classification are described below.

Metallic bearings of several types have been used. Examples are shown in Figs. 20 and 21 (2). The bearings shown in Fig. 20 are referred to herein as "rocker-type" and are characterized by an expansion bearing which relies upon two plates with curved surfaces to permit rotation at the end of the supported beam as well as to allow longitudinal translation; pintels are provided to connect the curved plates to the sole plate and masonry plate and prevent transverse translation. The fixed bearing shown in Fig. 20 has a curved plate at the top only to provide for beam rotation. The masonry plates of both the fixed and expansion bearings are normally connected to the concrete abutment or pier with anchor bolts; the sole plates are connected to the superstructure with either bolts or welds, in the case of steel beams, or with embedded anchors in the case of concrete beams.

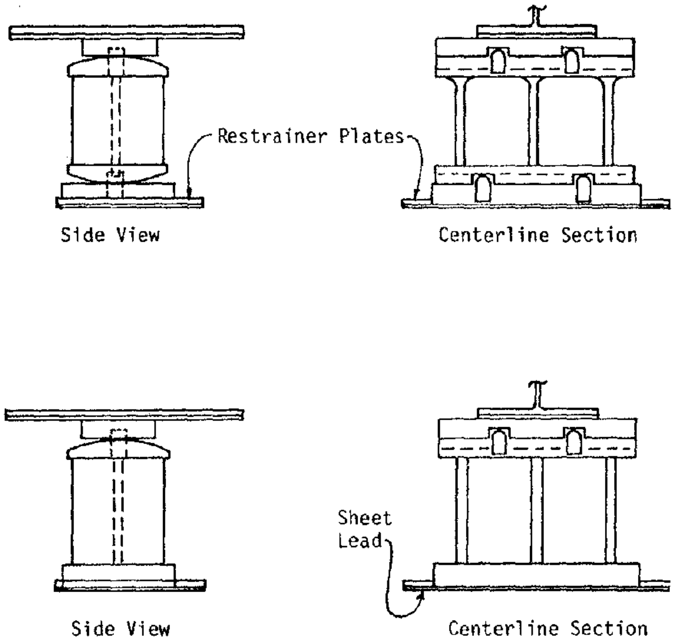


Fig. 20 METALLIC BEARING-ROCKER TYPE (FROM REF. 2)

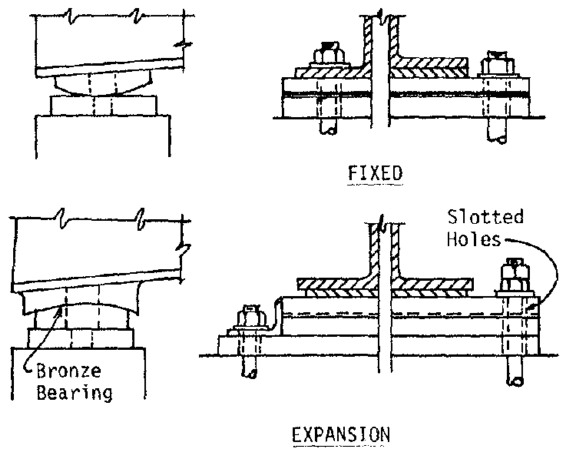


Fig. 21 METALLIC BEARING-SLIDING TYPE (FROM REF. 2)

Bearings of the sliding type are shown in Fig. 21 where it will be seen that a self-lubricating bronze bearing plate is used in the expansion bearing. Transverse translation can be resisted with bearings of this type by providing anchor bolts which extend through the rocker plates or by the provision of steel keeper plates.

Other types of metallic bearings have been and still are used but the above two types, (and variations of them) have been the most commonly used.

Elastomeric bearing pads of two general types have been used extensively in recent years. These might best be classified as "confined" and "unconfined" elastomeric bearing devices.

The most simple of the unconfined type consists of a pad of solid elastomer which is placed between the bearing seat and the supported member. Rotations are accommodated by vertical rotational deformations of the pad and translations are accommodated by shear deformations of the pad. Pads of elastomer alone are limited in the amount of load which they can withstand without excessive bulging of the elastomer; hence, for large loads bearings are made with steel plates laminated between layers of elastomer. The provision of steel plates laminated between the layers of the elastomer restrains the transverse deformation of the elastomer (bulging) and permits larger stresses to be imposed on the elastomer without excessive bulging of the free edges. Steel-laminated elastomeric bearing pads are frequently used between concrete bearing seats and concrete beams without being attached to either; friction is relied upon to keep them in place. They are sometimes bonded to heavy steel plates to facilitate their connection to the structure on which they are to be used. In addition, they are sometimes used in combination with steel plates coated with tetraflouroethylene resin (TFE) to accommodate large horizontal movements. Each of these are shown in Fig. 22.

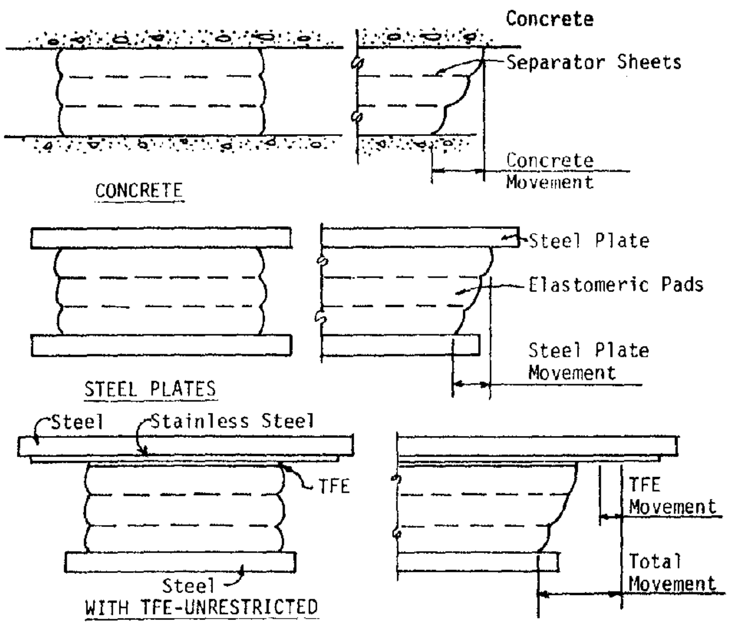


Fig. 22 LAMINATED ELASTOMERIC BEARINGS (FROM REF. 2)

Confined type elastomeric bearings are known by the common name "pot bearings". Pot bearings consist of a round elastomeric pad confined by a steel ring that prevents the elastomer from bulging under vertical load. The load is applied to the elastomer through a circular steel plate slightly smaller in diameter than the inside diameter of the steel ring. With this arrangement, the elastomer acts somewhat like a fluid in a hydraulic ram. The elastomer in a pot bearing allows rotational movements but not translation; hence, steel plates coated with TFE are incorporated in pot bearings used as expansion bearings. The construction of a pot bearing is illustrated in Fig. 23.

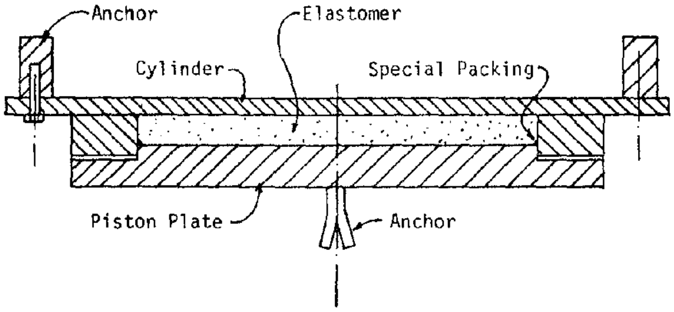


Fig. 23 POT BEARINGS

Concrete bearings can be described as concrete "hinge" and "built-in" bearings. A type of concrete hinge, shown at the base of a concrete column is shown in Fig. 24. The rotation of the hinge results from the large strains in the highly loaded "throat" of the bearing; the shear strength of the concrete resists translations.

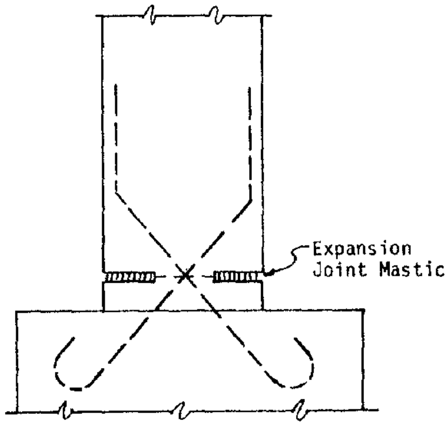


Fig. 24 CONCRETE HINGE (FROM REF. 2)

A built-in bearing, as previously stated, is nothing more than a monolithic or pseudo-monolithic joint.

It should be noted that the steel and elastomeric bearings described above have no strength in the vertical direction and hence, cannot resist uplift forces that might come upon them under extreme ground shaking. (The uplift forces may result from structural response to large loads rather than vertical accelerations which are a part of the ground shaking.) Additionally, it should be noted that the steel bearings and the pot bearings are provided with anchor bolts that have limited capacity to resist horizontal forces. Unconfined elastomeric bearings, as usually used, have little capacity for resisting lateral loads. None of the bearing devices commonly used in U.S. bridge construction are designed for the forces which would be imposed upon them by an earthquake.

DISCUSSION

Based upon the above descriptions of bridge construction details commonly found in the United States, it is interesting to consider the susceptability of bridges having these details to earthquake-induced damage. The types of earthquake damage considered herein is not all-inclusive but is limited to that which most commonly occurs at abutments, bearing devices, bearing seats and in columns.

Longitudinal Shaking

If one considers a bridge composed of one or more simple spans having backwall-type of abutments and expansion and fixed bearings of steel or elastomer, as shown in Fig. 1, it will be concluded that each of the types of damage under consideration would probably be sustained under significant ground shaking. Backwall-type of abutments are earth-retaining structures not normally designed, at least in U.S. practice, for seismically induced earth-pressure forces; hence, they must be expected to move inward (towards the bridge span) under major ground shaking. In addition, if the bearing devices that constitute the substructure-superstructure connections are not strong enough to resist the loads imposed on them by the ground shaking, or if the piers undergo large deformations, backwall damage can be expected due to the superstructure impacting the backwall. Bearing devices of normal design are not in themselves capable of resisting the forces that can be imposed upon them under major shaking and unless other provisions are made for accommodating these forces, bearing failures in the form of sheared anchor bolts, pulled-out anchor bolts, sheared keeper plates, sheared pintels or overturned bearings (or a combination of those) can be expected. If the displacement of the ends of the beams due to the lateral forces exceed the distance the beam ends overlap their supports, collapse of the superstructure can be expected. Finally, the ground shaking may cause pier deformations that exceed their strain capacity causing their failure; this may or may not result in complete collapse.

If the bridge under consideration above were composed of continuous spans as shown in Fig. 4 rather than simple ones, it is apparent that abutment damage of the types described above could still occur as could failure of the bearings and the piers. The fact the superstructure is continuous over the piers would prevent it from losing the support of the piers; hence, this type of failure would be avoided.

Provision of end diaphragm type abutments in either of the two bridge examples described above would tend to avoid damage caused by seismically induced lateral earth pressures and pounding providing the bridge superstructure can act as a strut-tie between the two abutments. In addition, the use of end diaphragm type abutments eliminates the need for conventional bearing devices at the abutments and hence, eliminates the possibility of failure due to inadequacies of the bearing devices or their connections.

The use of built-in connections between the substructure and superstructure at the tops of the piers of the second example would eliminate the need for conventional bearing devices at the piers and would result in frame action. This change would eliminate the possibility of bearing-device-related failures and would add redundancy to the structure.

For structures which have in-span joints, the provision of structural members which tie the superstructure together across the joint, as shown in Fig. 25, can prevent the joint from spreading apart to the extent that support of the suspended end is lost and collapse takes place. The structural ties must be designed in such a way that normal length changes due to temperature, shrinkage and creep can take place without significant restraint; but which resist the large movements associated with out-of-phase vibration due to ground shaking.

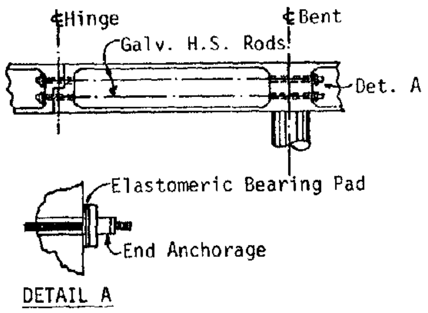


Fig. 25 IN-SPAN JOINT WITH RESTRAINERS (FROM REF. 6)

Transverse Shaking

The connection between the superstructure and substructure at the abutments must be capable of transmitting large forces if the two portions of the structure are to maintain their relative positions under transverse shaking. In the case of backwall-type abutments, it is thought this can best be done by the provision of large shear transferring devices such as the concrete shear keys shown in Fig. 26; conventional bearing devices of usual design are not capable of transferring these forces. With end diaphragm type abutments, the transfer of lateral forces between the superstructure and substructure does not present a problem.

Multi-column piers or bents are generally preferred to single column or wall piers for resisting transverse lateral loads because they can be made to act as frames and hence, with proper detailing, are ductile, energy-absorbing and provide redundancy. The use of a single column pier has lost favor in the more highly seismic areas of the U.S. since the 1971 San Fernando earthquake;

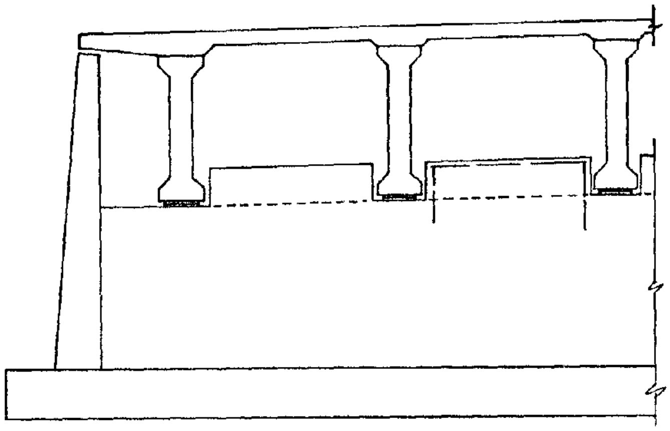


Fig. 26 ELEVATION OF ABUTMENT SHOWING SHEAR KEYS

a number of structures having single column piers sustained severe damage in that earthquake. Wall piers, like shear walls in buildings, behave in a more brittle manner when subjected to large loads. They are generally used for river crossings, railroad overpasses, etc. When used, they are proportioned conservatively with regards to stress levels.

In-span joints are provided with substantial shear keys reinforced in a manner intended to make them ductile. These keys should be similar in detail to those used in backwall-type abutments.

CONCLUSIONS

Many details of bridge construction commonly used in the United States are considered to be undesirable for use in regions of moderate to high seismicity. They have, however, performed well in areas of low seismic activity.

In the areas of higher seismic activity, bridge designers are generally aware of the need for providing details of construction that provide greater ductility and redundancy as well as to eliminate details known to lack the ability to perform well under severe ground shaking. The following principles are followed in the design of bridges located in the higher seismic areas when possible:

1. End diaphragm abutments are used.

2. Multi-column piers or bents are used.

3. Substructure-superstructure connections are built-in to the extent possible.

4. Positive moment reinforcement (either ordinary or prestressed) is provided throughout the length of the superstructure, even through the built-in connections with the substructure, to provide for possible reversals of moment near the supports of longitudinal frames under extreme earthquake loading.

5. In-span joints are used only if necessary.

6. In-span joints are provided with longitudinal restrainers and shear keys detailed to be ductile.

7. Columns are provided with main reinforcement adequately spliced at locations of low moment levels, well anchored in the foundations and superstructure as well as confined by closely spaced spirals or ties to ensure ductile behavior under extreme loads.

CLOSURE

In closing, two bridges are described which are not typical structures but are interesting from the standpoint of showing how some engineers are approaching the seismic design problem for unique structures.

Pasco-Kennewick Intercity Bridge, Washington

This bridge (Fig. 27) is 763 meters long (2,503 feet) and is located in seismic zone 2 as defined in the Uniform Building Code, 1976 edition.(5) The structure is cable-stayed with the deck constructed of precast concrete segments. The deck is fixed longitudinally at the Pasco side only and is free to move with respect to the substructure at all other points. The inside surfaces of the pier towers are provided with vertical elastomer-TFE slide-bearings to permit vertical and longitudinal translation yet transmit transverse loads to the towers. The connection of the deck at the Pasco side is designed to fail in shear longitudinally if subjected to a large seismic force. With the connection at the Pasco side intact, the fundamental period of the structure is estimated to be 0.5 seconds. With the connection failed, the fundamental period is estimated to be 10 seconds with the structure free to sway longitudinally from the tops of the towers. The variable length of the stays is thought to provide considerable damping.

Linn Cove Viaduct, North Carolina

This structure (Fig. 28) is 379 meters long (1,243 feet) having eight spans ranging in length between 30 meters (98 feet) and 55 meters (180 feet). The superstructure, which is to be constructed of precast concrete segments, is on three reversed horizontal curves. It is located in seismic zone 2 of the Uniform Building Code.

A unique detail, shown in Fig. 29, is provided for transfer of lateral forces between substructure and superstructure. This detail includes a reinforced concrete pintel that is a frustum of a circular cone, steel-laminated elastomeric bearing pads and space for placing hydraulic jacks for grade adjustments or future replacement of the bearings. The connection is without vertical ties between the superstructure and the substructure.

Figure 27, Pasco-Kennewick Intercity Bridge, Washington,

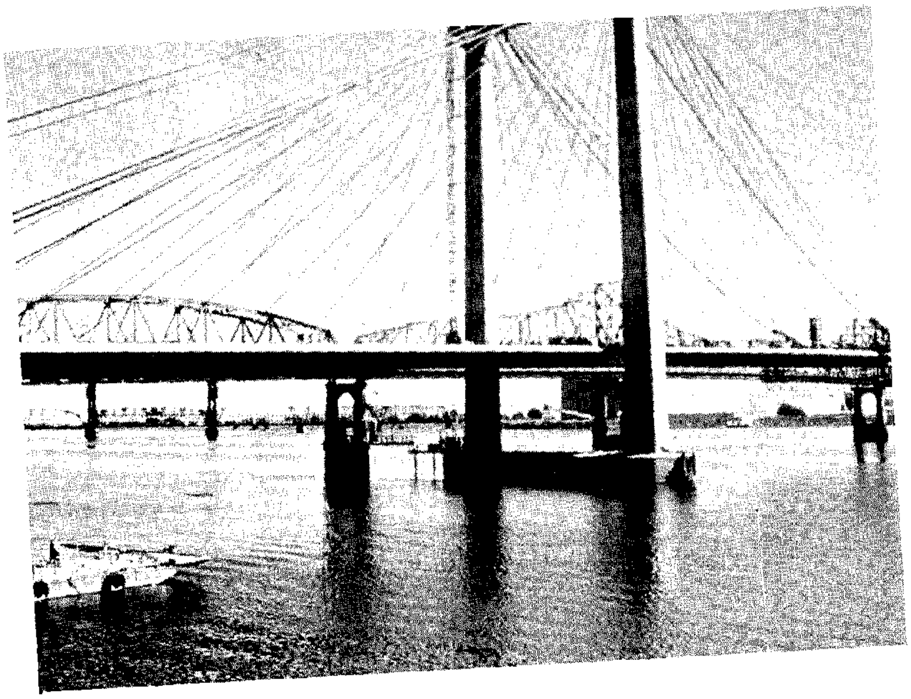
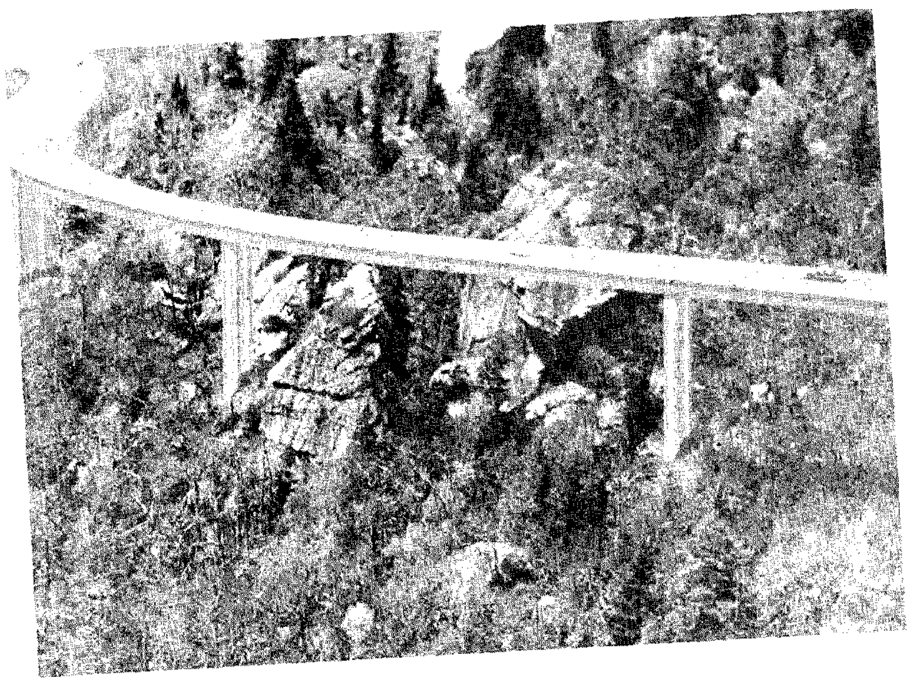


Figure 28, Linn Cove Viaduct, North Carolina



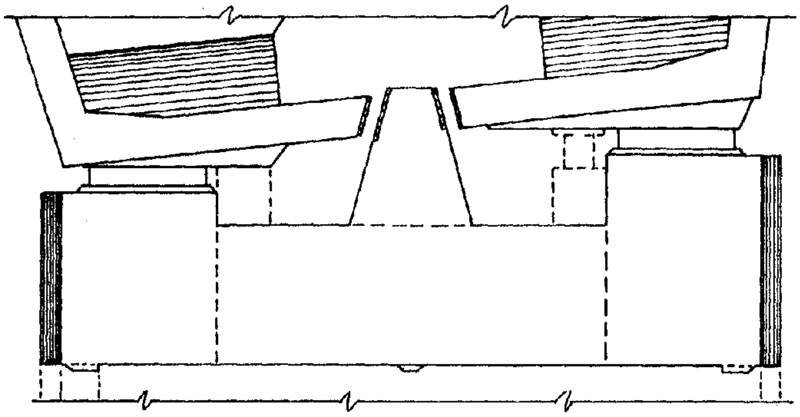


Fig. 29 SUPERSTRUCTURE-SUBSTRUCTURE CONNECTION DETAIL FOR THE LINN COVE VIADUCT

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SEISMIC DESIGN CONSIDERATIONS FOR BRIDGE FOUNDATIONS AND SITE LIQUEFACTION POTENTIAL

By

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ABSTRACT

The nature of damage to bridge structures arising from liquefaction of saturated cohesionless foundation soils is illustrated with reference to several case histories. Methods for evaluating liquefaction potential are briefly outlined, and design philosophies for bridge construction in liquefaction susceptible areas discussed. The calculation of foundation stiffness parameters is reviewed, and the importance of inclusion of non-linear and degradation behavior for soft soils is noted with reference to the case of lateral loading of piles. Brief comments are also made on the significance of free field displacements and soil-pile-interaction in the development of pile bending stresses.

SEISMIC DESIGN CONSIDERATIONS FOR
BRIDGE FOUNDATIONS AND SITE LIQUEFACTION POTENTIAL

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INTRODUCTION

Liquefaction of saturated granular foundation soils has been a major source of bridge failures during historic earthquakes. For example, during the 1964 Alaska earthquake, 9 bridges suffered complete collapse, and 26 suffered severe deformation or partial collapse. Investigations indicated that liquefaction of foundation soils contributed to much of the damage, the loss of foundation support leading to major displacements of abutments and piers. To illustrate the nature of liquefaction induced damage, several case histories from past earthquakes are considered in the subsequent section. Clearly, design of bridge structures to withstand liquefaction failure poses major if not insurmountable difficulties, and a design philosophy based on "calculated risk" at least for non essential bridges may be appropriate.

To provide background to the liquefaction evaluation problem, the two basic approaches for determining the liquefaction potential at a site are outlined. In one approach, in situ liquefaction strengths are assessed by way of correlations between sites which have and have not liquefied and standard penetration test blowcounts. The second approach involves the laboratory determination of liquefaction strengths using cyclic loading tests on undisturbed samples.

Whereas bridge foundation failures in past earthquakes are not common at sites where liquefaction does not occur, the magnitude of foundation compliance arising from cyclic displacements or rotations may have a significant influence on the distribution of loads in the bridge structure itself. The methods used to compute foundation stiffness parameters are briefly outlined, and emphasis is placed on the need to consider the potential effects of non-linear behavior and stiffness degradation with cycles of loading, particularly for softer soils and strong levels of ground shaking. For the case of lateral loading of piles, the American Petroleum Institute recommendations which incorporate non-linear subgrade reactions support are outlined. Finally, the significance of dynamic soil-pile-structure interaction is discussed, where studies have shown that pile curvatures induced by free field displacements may impose significant bending stresses at depth.

DAMAGE TO BRIDGES ARISING FROM LIQUEFACTION

Damage to bridges arising from liquefaction of abutment or foundation soils is characterized by movement of abutments, spreading and settlement of abutment fills, horizontal displacement and tilting of piers, severe differential settlement of abutments and pier, and failure of foundation members. Typical examples of such damage during past earthquakes, are given below.

Niigata 1964

The widespread liquefaction during the 1964 Niigata earthquake (magnitude 7.5) generated many bridge failures, of which perhaps the Showa Bridge collapse is the best known. The bridge site was about 55 km from the epicenter of the earthquake, and the nature of the failure is illustrated in Figure 1.

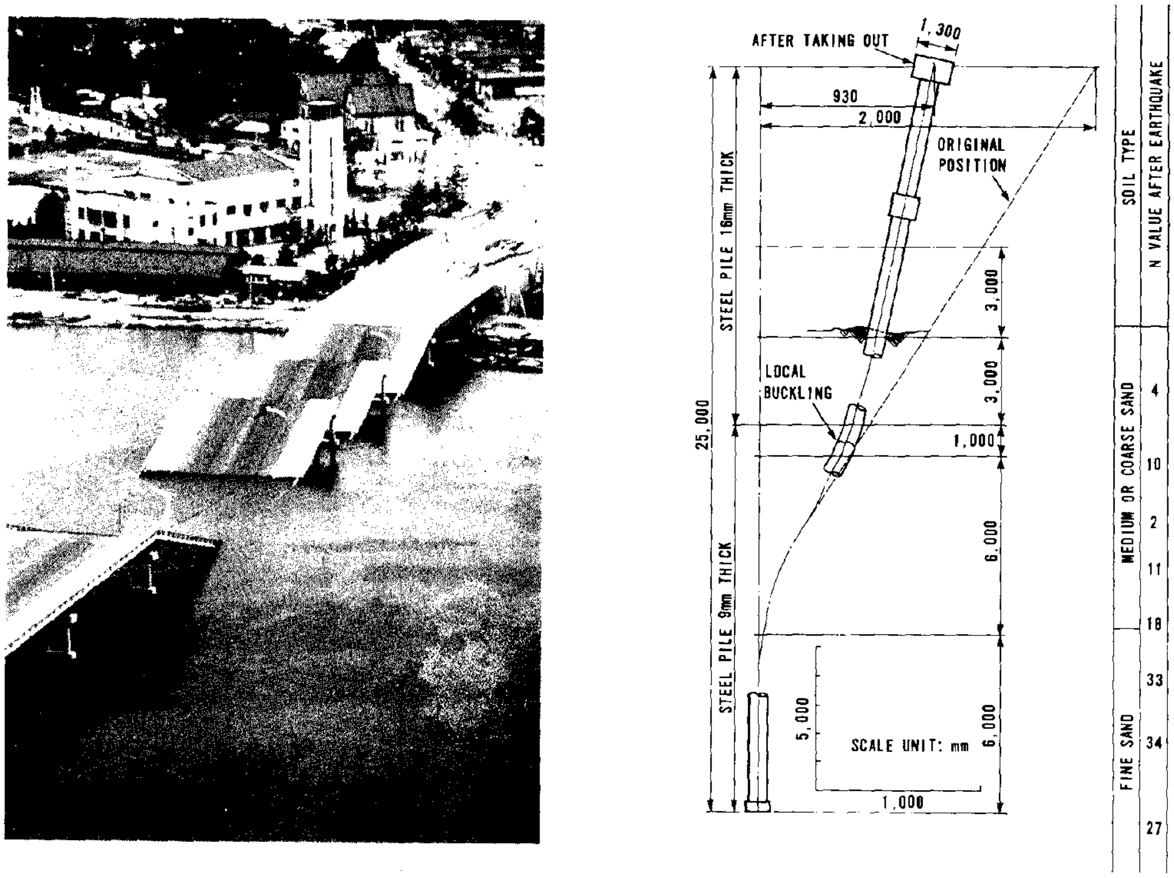


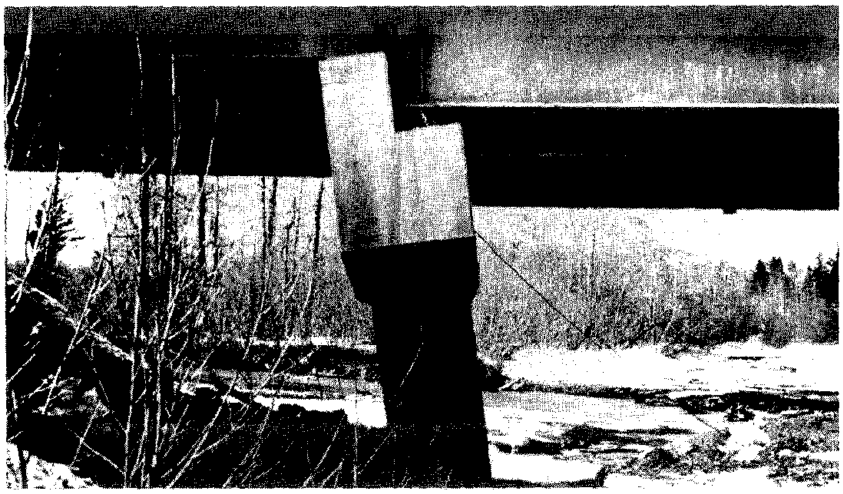
FIG. 1 COLLAPSE OF SHOWA BRIDGE (NIIGATA EARTHQUAKE, 1964) SHOWING SOIL PROFILE AND PERMANENT DEFORMATION OF STEEL PILE REMOVED AFTER EARTHQUAKE.

Bridge piers were supported on steel piles driven through loose sands to medium dense fine sands about 50 feet (15m) below the mudline. Distortions of piles resulting from the loss of lateral support from the liquefied loose sands allowed the unconnected simple spans to fall off the pier supports. After the failure, a study was made [3] where the 50 foot long piles from pier 4 were removed and measurements made of pile distortion as shown in Figure 1.

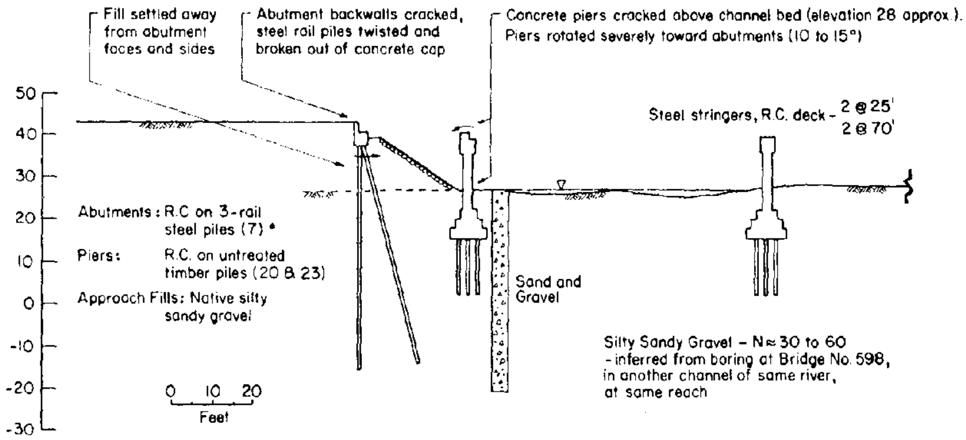
Alaska, 1964

The 1964 Alaska earthquake (magnitude 8.4) damaged numerous bridges, particularly a series of pile supported bridges along the Seward and Copper River Highways. Ross et. al. [4] provide a well documented summary of the damage, which in most instances could be attributed to liquefaction of abutment fills or foundation soils. Typical examples are noted below.

Figure 2 illustrates damage to Bridge 596 on the Resurrection River, located at the southern end of the Seward Highway approximately 60 miles (≈100 km) from the epicenter of the earthquake.



SOUTHERLY PIER LOOKING UPSTREAM. NOTE TILT.



CENTERLINE SECTION LOOKING UPSTREAM

FIG. 2 DAMAGE TO BRIDGE 596: RESURRECTION RIVER. (ALASKA EARTHQUAKE, 1964)

FIG. 3 BUCKLING OF BRIDGE DECK DUE TO INWARD MOVEMENT OF ABUTMENTS. (ALASKA EARTHQUAKE, 1964).

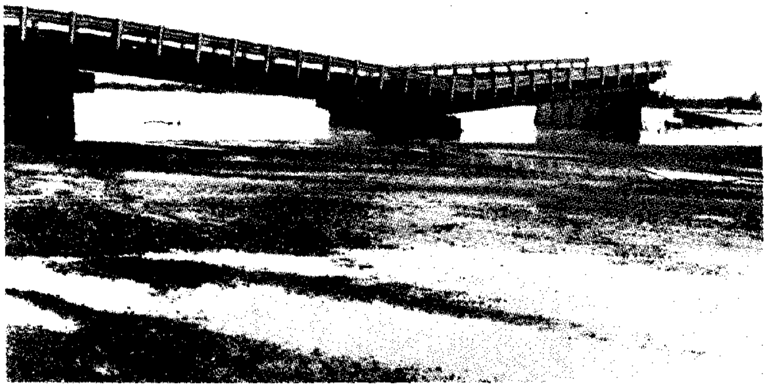


FIG. 4 BRIDGE 334: COPPER RIVER 5 (ALASKA EARTHQUAKE, 1964). NOTE SEVERE DIFFERENTIAL SETTLEMENT OF STEEL-RAIL-BENT PIERS. WEBWALLS ARE REINFORCED CONCRETE.

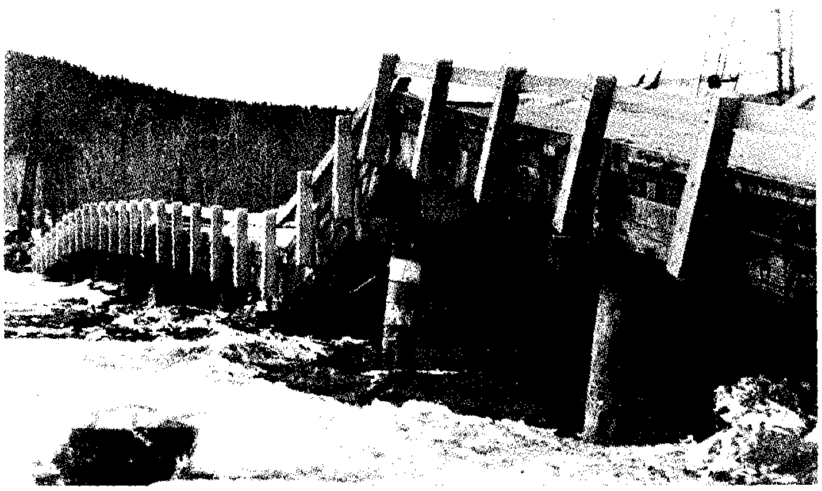


FIG. 5 BRIDGE 605: SNOW RIVER 3. (ALASKA EARTHQUAKE, 1964). POST-EARTHQUAKE VIEW, LOOKING DOWNSTREAM.

The pier rotation has been attributed to high lateral loads on the pier footing and piles arising from horizontal deformations of the abutment fill. Standard penetration tests gave blowcounts of 30-60 for the silty sandy gravel in the area. Such values would not normally be considered conductive to liquefaction failures of a 20 foot high embankment. However, only a few tests were performed and blowcounts are not a good indication of the relative density of sands in gravelly material. It seems possible that high pore pressure buildup in sand lenses or partial liquefaction may have contributed to the lateral spread of abutment fill. It is noted that a similar bridge 1500 feet away suffered little damage. However, in this case, a clearance of 20 feet existed between the toe of the abutment fill and the pier, so that the effects of lateral spread of abutment fill would have been less severe.

Further examples of damage during the Alaskan earthquake are shown in Figures 3, 4, and 5. Figure 3 shows the longitudinal buckling of a bridge deck due to inward movement of abutments. Once again, it has been suggested that the effects of increased pore pressures or liquefaction was a major contributing factor to the large observed displacements of abutment soils. Figure 4 shows bridge damage arising from differential pier settlement of a Copper River bridge. Extensive deposits of sand and gravel dominate the Copper River region, where there was considerable evidence of liquefaction in the form of fissures and subsidence craters with adjacent ejected soil. The timber bridge deck shown in Figure 5 was supported by timber 4-pile bents. The piles extended 40-60 feet into a loose fluvial soil consisting of sands and silts with a standard penetration resistance N = 5-10. Liquefaction of the foundation and abutment soils resulted in the observed failure, where buckling of the deck was initiated by inward movement of abutment fill.

Tangshan, 1976

The 1976 Tangshan earthquake (magnitude 7.8) in China resulted in numerous bridge failures which have been attributed to liquefaction of the very fine sands in a delta region [5] . The general pattern of failures were very similar to those observed in Alaska described above. Many abutment failures occured, the mechanisms of failure either being rotational or lateral as shown in Figure 6.

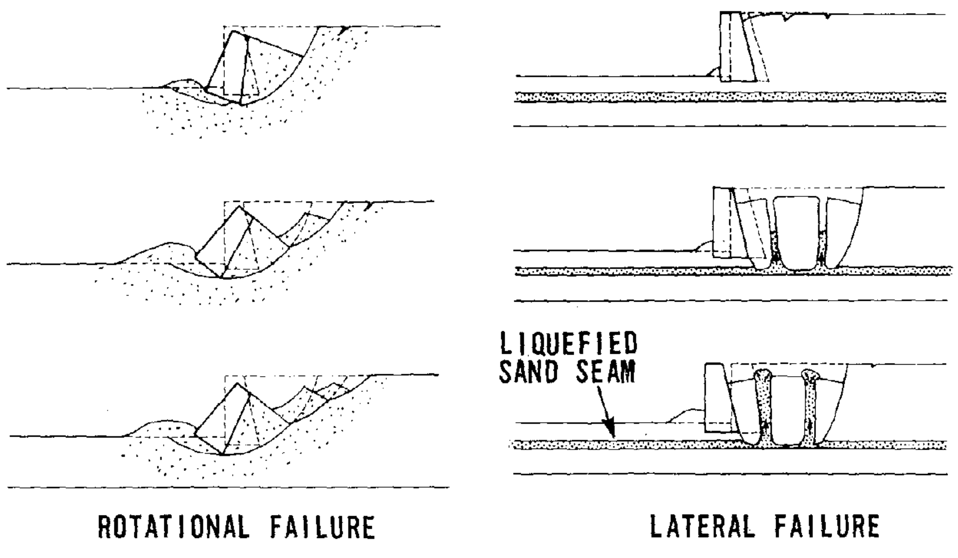


FIG. 6 ABUTMENT FAILURE MECHANISMS, TANGSHAN EARTHQUAKE, 1976.

Rotational failures occurred where the strength of saturated sand backfill progressively degraded as a result of pore water pressure increases. Lateral failures were initiated by the presence of a loose sand layer at some depth below a bridge abutment.

General Discussion

The 1964 Alaskan earthquake demonstrated a good cross section of the types of failure which can be expected where foundation soils liquefy. The greatest concentrations of damage occurred in regions characterized by thick deposits of saturated cohesionless soils, where field evidence indicated that liquefaction probably played a major role in the development of foundation displacements and bridge damage. A correlation between foundation displacements and foundation support conditions is given in Figure 7 below, and reflects the influence of liquefaction on observed damage.

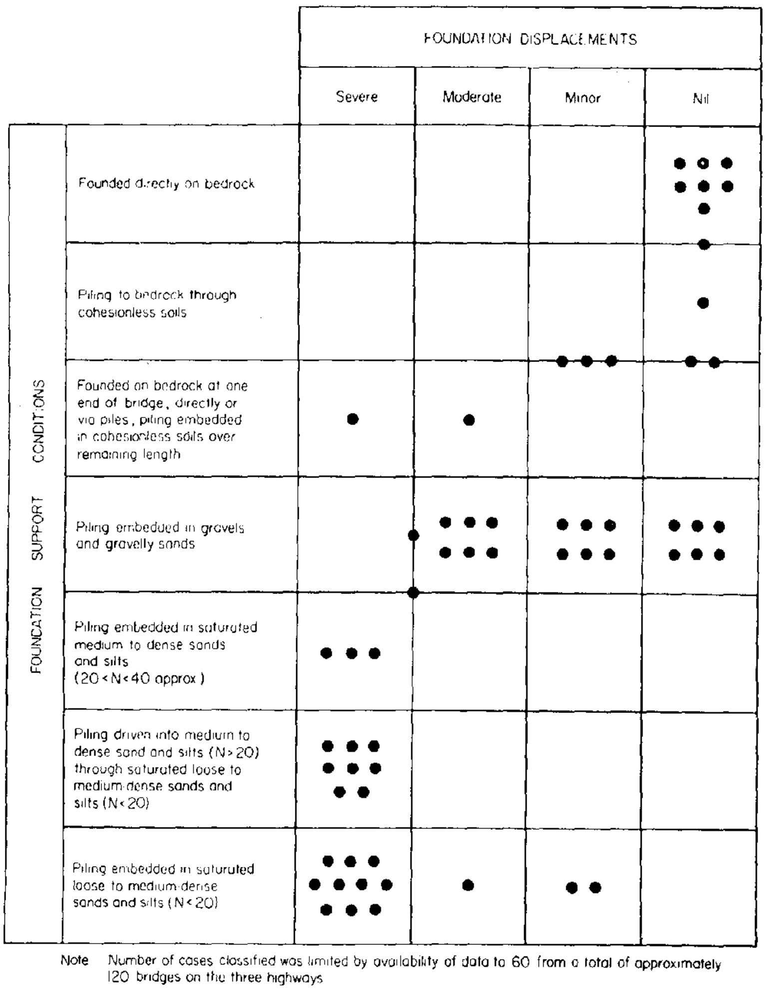


FIG. 7 CORRELATION BETWEEN FOUNDATION DISPLACEMENTS SUSTAINED AND FOUNDATION SUPPORT CONDITIONS AT BRIDGES ON THE SEWARD, STERLING, AND COPPER RIVER HIGHWAYS: ALASKA EARTHQUAKE, 1964. (AFTER ROSS ET. AL. 1973)

The severely damaged foundations in loose to medium dense saturated sands and silts included many structural types from light all timber bents to heavy reinforced concrete piers on long battered steel and concrete piles. This indicates the great difficulty in providing sufficient lateral resistance with conventional pile foundations where liquefaction occurs.

The improved foundation performance for piling embedded in gravels, probably reflects the influence of the rapid dissipation of earthquake induced pore water pressures and the consequent prevention of potential liquefaction. It should be noted, however, that liquefaction can occur in gravels where drainage is impeded by impervious layers. Such cases are discussed by Finn and Yong [6] for conditions in Alaska, where a frozen soil crust can impede drainage. The presence of a frozen layer may also lead to higher than normal static hydrostatic pressures, which will reduce liquefaction resistance.

It is clear that the design of bridge foundations in liquefaction susceptible soils poses difficult problems. Given a choice, the best design measure is to avoid deep loose to medium dense sand sites where investigation shows that liquefaction risks are high. Where dense of more competent soils are found at shallow depths, stabilization measures such as densification may be economic. The use of long ductile vertical steel piles to support bridge piers could also be considered. Calculations for lateral resistance would assume zero support from the upper zone of potential liquefaction, and the question of axial buckling would need to be addressed. Overall abutment stability would also require careful evaluation, and it may be preferable to use longer spans and anchor abutments well back from the end of approach fills.

A further design philosophy for bridges in liquefaction susceptible areas might be one of "calculated risk", at least for those bridges regarded as being less essential for communication purposes immediately after an earthquake. In this respect, C. P. Smith, Chief Bridge Engineer for the Alaska Department of Highways in 1964, made the following statement on the permanent replacement of bridges damaged in the Valdez district [7] :

> "The design for permanent replacement bridges envisioned a series of simple composite steel-beam-concrete-slab spans on pile bent piers and abutments, incorporating every practicable device to insure a simplicity of detail and modular application to various sites. Twenty-five standard drawings were developed covering 50, 60, and 80-foot spans for 28, 30, and 41-foot roadway widths, using either H-pile or cast-in-place concrete pile bents, elastomeric bearings, standard rolled beams, and bolted field connections. The details received favorable comment from contractors and fabricators. Because these simple, economical structures will be applicable to the continuing normal Alaska Highway program, it is contemplated that the designs will later be refined to incorporate welded girder sections. Aside from modification of some detail to prevent minor damage,...it becomes increasingly evident that design against the type of forces experienced in an earthquake of this magnitude is uneconomical and falls in the calculated risk category,...the elimination of damage entirely is impractical."

Hence, for some bridges, design to survive a large earthquake in a liquefaction environment without significant damage may not be economically justifiable. As noted by Sturman [8] , it may be possible to optimize a design so that the cost of potential damage caused by a given design earthquake does not exceed the cost of remedial measures and additional construction needed to avoid the damage.

EVALUATION OF LIQUEFACTION POTENTIAL

Whereas it is beyond the scope of this paper to discuss in detail the methods for evaluating liquefaction potential at a given site, in view of the many bridge failures attributable to liquefaction, it is pertinent to outline generally accepted approaches to the liquefaction problem. A recent review of methodologies has been presented by Seed [9], where two basic approaches for evaluating the cyclic liquefaction potential of a deposit of saturated sand subjected to earthquake shaking are identified:

(i) Empirical methods based on field observations of the performance of sand deposits in previous earthquakes, and correlations between sites which have and have not liquefied and Relative Density or Standard Penetration Test (SPT) blowcounts.

(ii) Analytical methods based on the laboratory determination of the liquefaction strength characteristics of undisturbed samples and the use of dynamic site response analysis to determine the magnitude of earthquake induced shearing stresses.

For conventional evaluations using a "total stress" approach, the two methods are similar, but differ only in the manner in which the field liquefaction strength is determined. In the "total stress" approach, liquefaction strengths are normally expressed as the ratio of an equivalent uniform or average cyclic shearing stress τ_h acting on horizontal surfaces of the sand to the initial vertical effective stress σ_o'. As a first approximation, the cyclic stress ratio developed in the field due to earthquake ground shaking may be computed by an equation given by Seed and Idriss [10], namely:

$$(\tau_h)_{av}/\sigma_o' = 0.65 r_d (a_{max}/g)/(\sigma_o/\sigma_o') \qquad (1)$$

where

a_{max} = maximum or effective peak ground acceleration at the ground surface

σ_o = total overburden pressure on sand layer under consideration

σ_o' = initial effective overburden pressure on sand layer under consideration

r_d = a stress reduction factor varying from a value of 1 at the ground surface to 0.9 at a depth of about 30 feet.

Empirical Methods

Values of the cyclic stress ratio defined by equation (1) have been correlated for sites which have and have not liquefied, with parameters such as relative density based on SPT data [Seed and Peacock (11)] or some form of corrected SPT data [Seed et. al. (12), Castro (13)]. The latest form of this type of correlation [after Seed (9a)] is expressed in Figs. 8 and 9. N_1 is the measured standard penetration resistance of the sand corrected to an effective overburden of 1 ton per square foot using the relationship

$$N_1 = C_N \cdot N \qquad (2)$$

where C_N is a function of the overburden pressure as shown in Fig. 9.

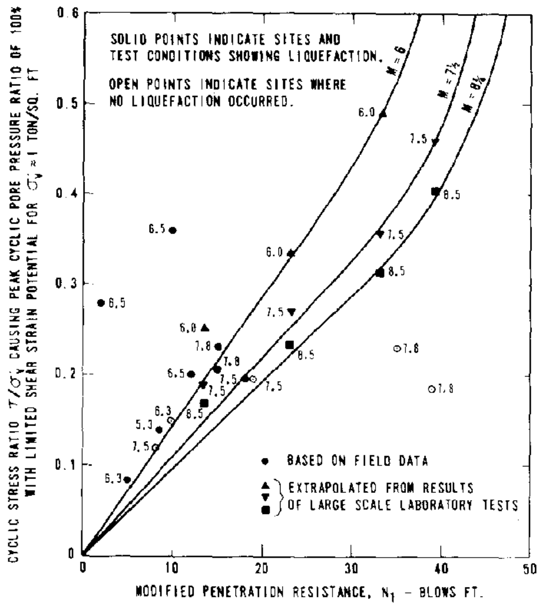


FIG. 8 CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOUR AND PENETRATION RESISTANCE

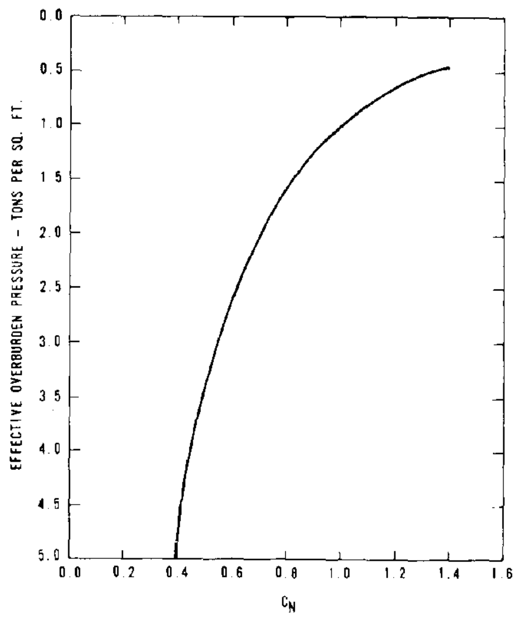


FIG. 9 RELATIONSHIP BETWEEN C_N AND EFFECTIVE OVERBURDEN PRESSURE

Thus, for a given site and a given maximum ground surface acceleration, the average stress ratio developed during the earthquake $(\tau_h)_{av}/\sigma_o'$ at a given depth may be determined from equation (1). This value may then be compared with the value of $(\tau_h)_{av}/\sigma_o'$ at which liquefaction may be expected to occur, as expressed by the empirical correlations shown by Fig. 10. The correlations for different magnitudes reflect the influence of earthquake duration on liquefaction potential. The factor of safety against liquefaction can be determined by comparing the stress ratio required to cause liquefaction with that induced by the design earthquake. It is suggested that a factor of safety of 1.5 is desirable to establish a reasonable measure of safety against liquefaction for the case of important bridge sites.

The calculation of factors of safety using the empirical SPT approach described above, is illustrated with reference to the failure of the pier foundations for the Snow River Bridge 605A [Ross (4)] during the 1964 Alaskan earthquake. A partial centerline section of this bridge is shown in Fig. 10. The pier foundations which were under construction at the time of the earthquake were founded on concrete-fill steel-tube piles extending to an average depth of 90 feet below stream bed level. As a result of site liquefaction during the earthquake, these piers displaced laterally about 8 feet downstream and tilted upstream about 15°, as indicated in Fig. 11. In the site region, maximum accelerations have been estimated to be on the order of 0.15g. Using the average of blowcounts N measured at the site, values of N_1 were computed for each depth and factors of safety against liquefaction computed as outlined above. Values of safety factor are shown plotted against depth in Fig. 12, where it may be seen that factor of safety <1 occur to depths of about 70 feet. This correlates reasonably well with the observed damage. Fig. 12 also shows

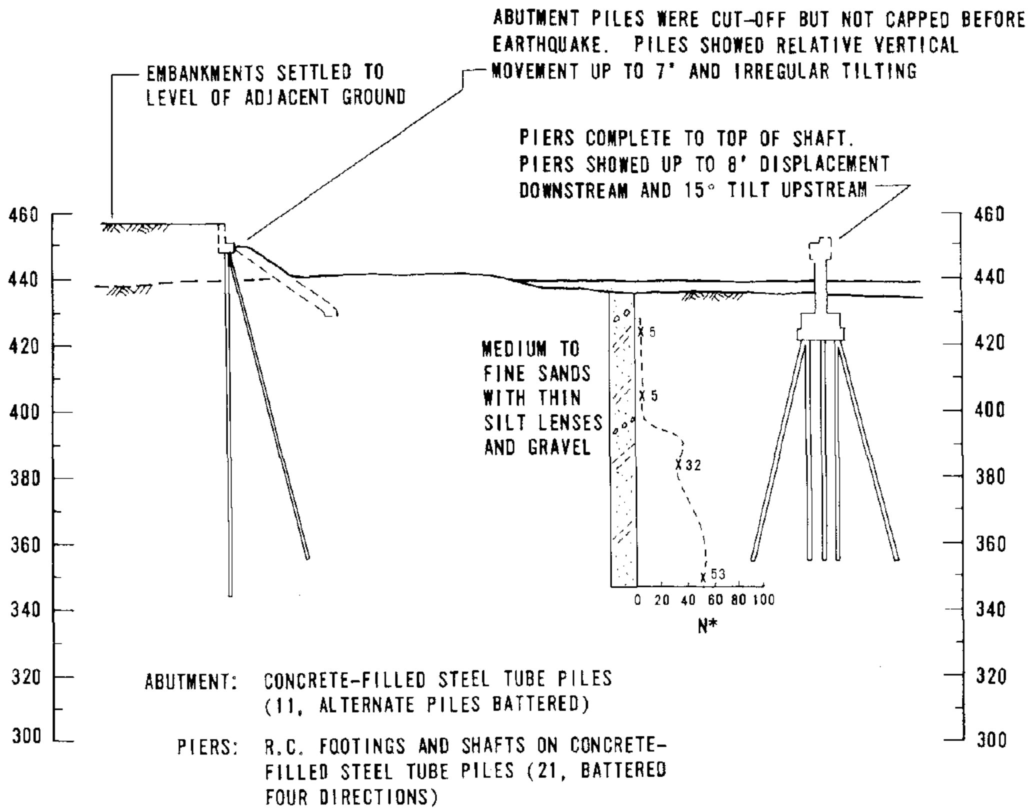


FIG. 10 BRIDGE 605A: SNOW RIVER. PARTIAL SECTION LOOKING DOWNSTREAM.

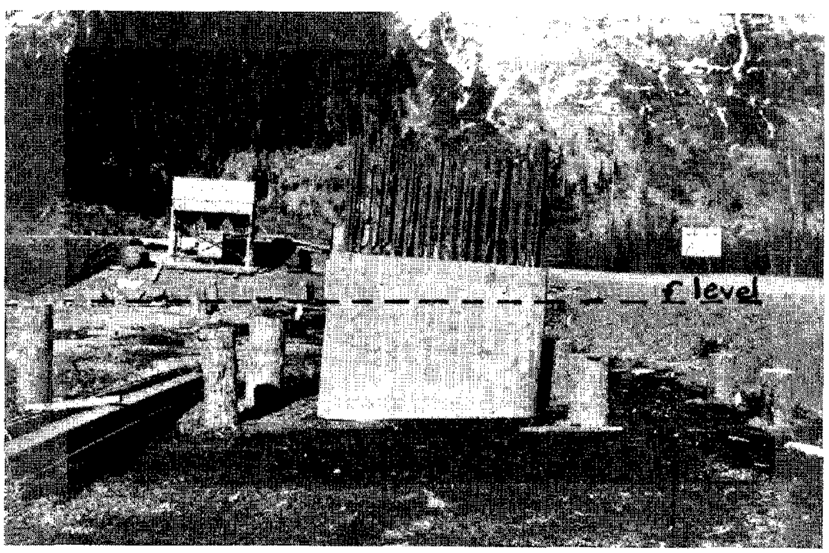


FIG. 11 BRIDGE 605A: SNOW RIVER. PIER TILT AFTER EARTHQUAKE.

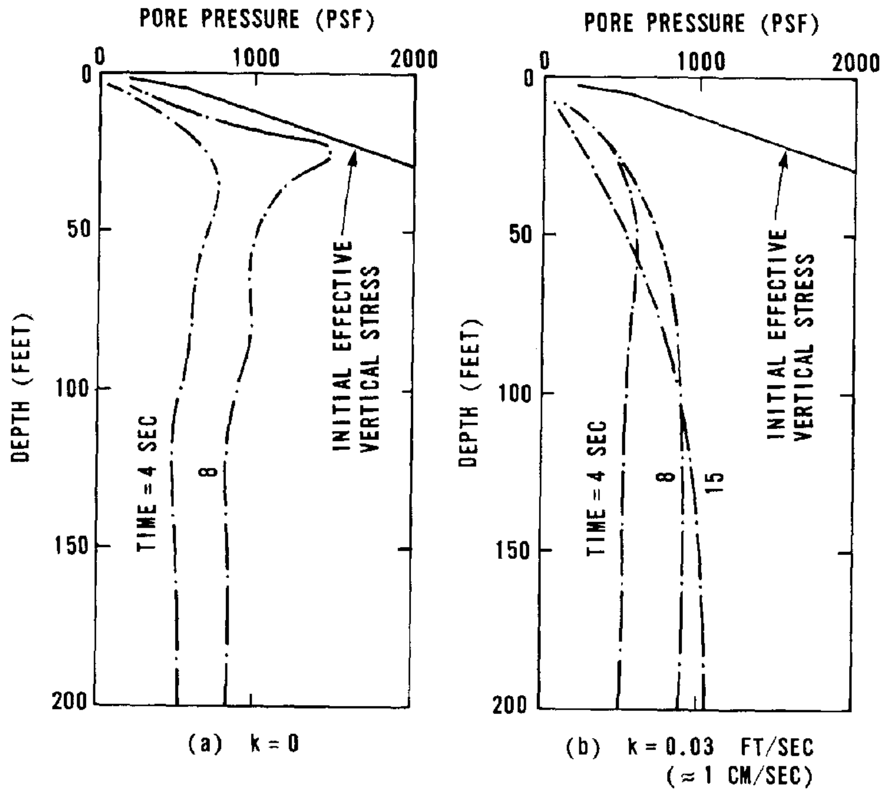


FIG. 14 EFFECTIVE STRESS APPROACH TO LIQUEFACTION EVALUATION SHOWING EFFECT OF PERMEABILITY (AFTER FINN ET. AL., 1977)

The figure shows results of dynamic response computations using the program DESRA [17]. Whereas for the undrained case shown (k=o) liquefaction occurs at a depth of about 25 feet after 8 seconds, if the permeability is 0.03 ft/sec (coarse sand), the effects of drainage and dissipation during the earthquake overide pore water pressure increases, and liquefaction will not occur.

Both empirical and analytical methods require the level of ground accelerations at a site to be defined as a prerequisite for assessing liquefaction potential. This is often established in detailed studies from relationships between earthquake magnitude, distance from the epicenter and peak acceleration.

It is of interest to note that a rough indication of the potential for liquefaction may be obtained between empirical correlations established between earthquake magnitude and the epicentral distance to the most distant field manifestations of liquefaction. Such a relationship has been described by Youd and Perkins [18], (Figure 15), and has been used as a basis for preparation of liquefaction-induced ground failure susceptibility maps.

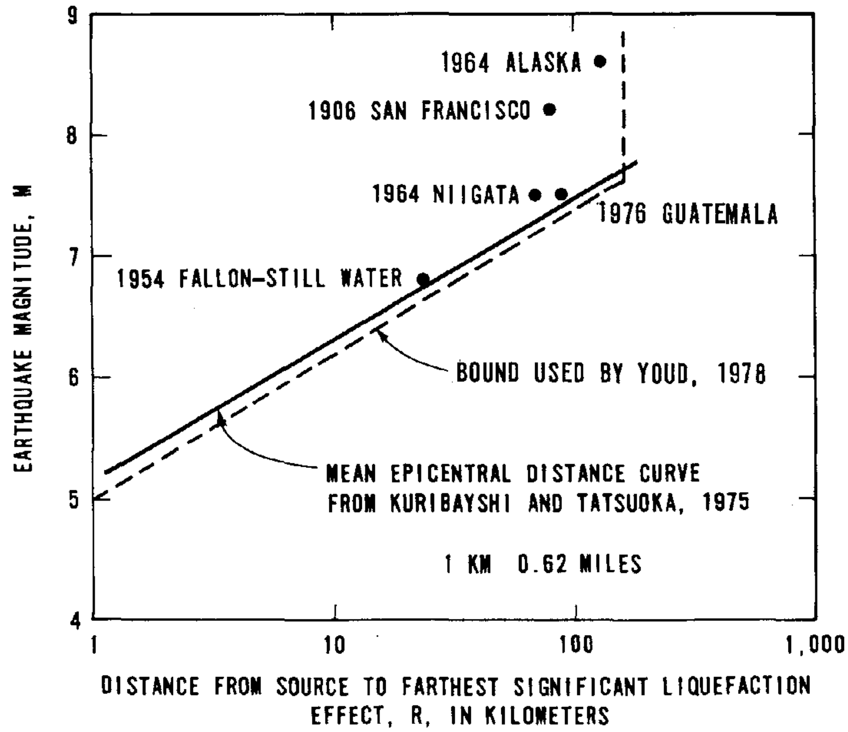


FIG. 15 MAXIMUM DISTANCE TO SIGNIFICANT LIQUEFACTION AS A FUNCTION OF EARTHQUAKE MAGNITUDE.

FOUNDATION DESIGN CONSIDERATIONS

General Concepts

The commonly accepted practice for the seismic design of foundations is to utilize a pseudo static approach, where earthquake induced foundation loads are determined from the reaction forces and moments necessary for structural equilibrium. Whereas traditional bearing capacity design approaches are also applied, with appropriate capacity reduction factors if a measure of safety against "failure" is desired, a number of factors associated with the dynamic nature of earthquake loading should always be borne in mind.

Under cyclic loading at earthquake frequencies, the strength capable of being mobilized by many soils is greater than the static strength. For unsaturated cohesionless soils the increase may be about 10%, while for cohesive soils, a 50% increase could occur. However, for softer saturated clays and saturated sands, the potential for strength and stiffness degradation under repeated cycles of loading must be recognized.

As earthquake loading is transient in nature, "failure" of soil for a short time during a cycle of loading may not be significant. Of perhaps greater concern is the magnitude of the cyclic foundation displacement or rotation associated with soil yield, as this could have a significant influence on structural displacements or bending moment and shear distribution in columns.

As foundation compliance influences the distribution of forces or moments in a structure and affects natural period computations, equivalent stiffness factors for foundation systems are often required. In many cases, use is made of the various analytical solutions which are available for footings or piles, where it is assumed that soil behaves as an elastic medium. In using these formulas, it should be recognized that equivalent elastic modulii for soils are a function of strain amplitude, and for high seismic loads modulus values could be significantly less than those appropriate for low levels of seismic loading. The nature of shear modulus changes with shearing strain amplitude in the case of sands is shown in Figure 16. Values of the maximum shear modulus G_{max} can be established from shear wave velocity measurements in the field using say downhole geophysical tests.

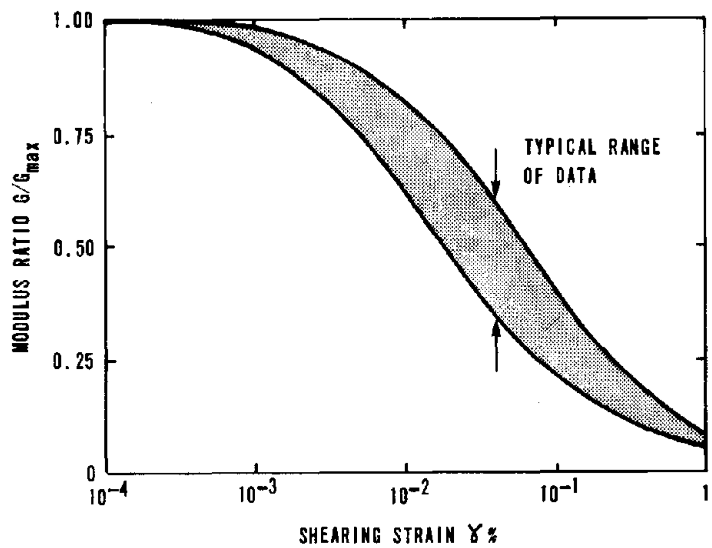


FIG. 16 VARIATION OF SHEAR MODULUS WITH SHEARING STRAIN FOR SANDS

In many cases it has been found preferable to directly incorporate the non-linear deformation behavior of soils in solutions for foundation stiffness, by using numerical computation methods. This is particulary the case for lateral loading of piles, and is discussed further below.

Lateral Loading of Piles

Most of the well known solutions for computing the lateral stiffness of piles are based on the assumption of elastic behavior, and utilize equivalent cantilever beam concepts [19], beam on an elastic Winkler foundation method [20] or elastic continuum solutions [21]. However, the use of methods incorporating non-linear subgrade reaction behavior which allows for soil failure may be important for high lateral loading of piles in soft clay and sand. Such a procedure is encompassed in the American Petroleum Institute (API) recommendations for offshore platform design [22]. The method utilizes non-linear subgrade reaction or p-y curves for sands and clays which have been developed experimentally from field loading tests. The general features of the analysis in the case of sands are illustrated in Figure 17.

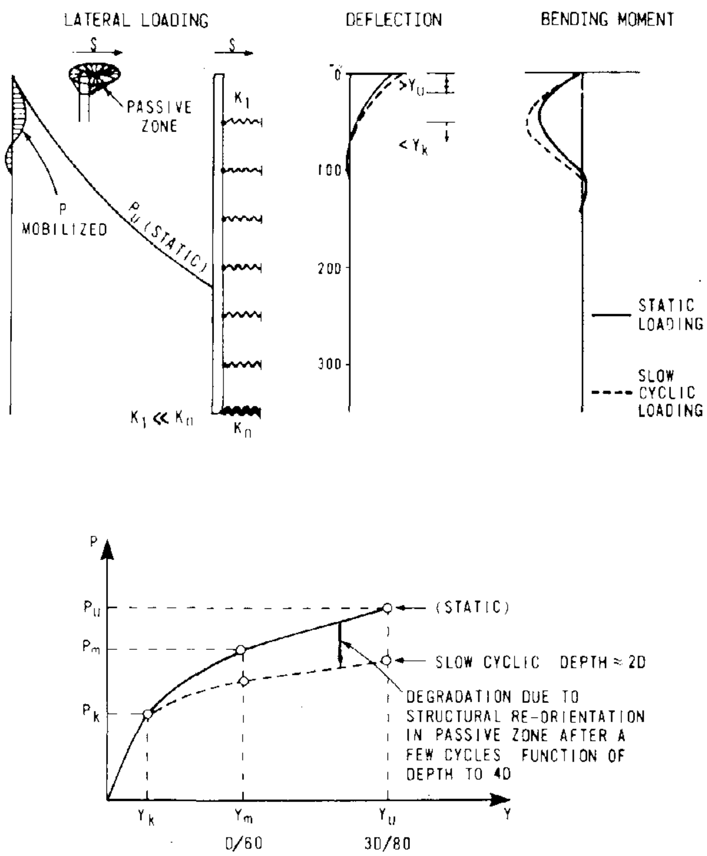


FIG. 17 LATERAL LOADING OF PILES IN SAND USING API CRITERIA

Under large lateral loads, a passive failure zone develops near the pile head, test data indicating that the ultimate resistance p_u is reached for pile deflections y_u of about 3D/80, where D is the pile diameter. It is noted that most of the lateral resistance is mobilized over a depth of about 5D. The API method also recognizes degradation in lateral resistance with cyclic loading, although in the case of saturated sands the degradation postulated does not reflect pore water pressure increases. The degradation in lateral resistance due to earthquake induced free field pore water pressure increases in saturated sands, has been described by Finn and Martin [23]. A numerical method which allows the use of API p-y curves to compute pile stiffness characteristics forms the basis of the computer program BMCOL 76 described by Bogard and Matlock [24].

The influence of group action on pile stiffness is a somewhat controversial subject. Solutions based on elastic theory can be misleading where yield near the pile head occurs. Experimental evidence tends to suggest group action is not significant for pile spacings greater than 4D to 6D.

For battered pile systems, the computation of lateral pile stiffness is complicated by the stiffness of the piles in axial compression and tension. It is also important to recognize that bending deformations in battered pile groups generates high reaction forces on the pile cap as illustrated schematically in Figure 18.

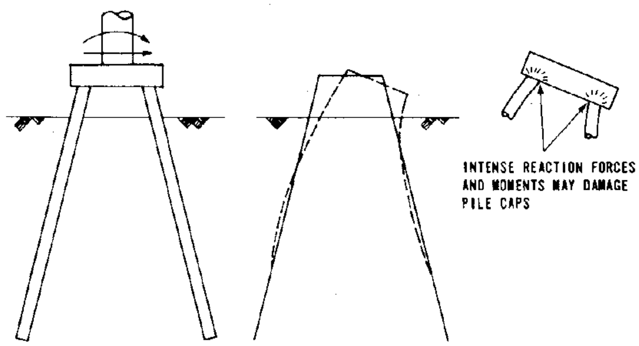


FIG. 18 INTERACTION OF BATTER PILES AND CAPS

Margason [25] reports several cases of damage to battered pile caps during earthquakes due to the bending action.

Due to the potentially high bending moments generated in piles by lateral loading, most codes recognize the need for appropriate reinforcement of concrete piles in the form of minimum requirements for longitudinal steel ratios and closely spaced ties or spiral reinforcement. Clearly, it is desirable to ensure that piles do not fail below ground level, and that flexural yielding is forced to occur in the columns above ground level.

Soil-Pile Interaction

The application of pile stiffness characteristics determined in the manner described above to determine earthquake induced pile bending moments using a pseudo-static approach, assumes that moments are induced only by lateral loads arising from inertial effects on the bridge structure. However, it must be remembered that the inertial loads are generated by interaction of the free-field earthquake response with the piles, and that the free field displacements themselves can influence bending moments. This is illustrated in an idealized manner in Figure 19. The free field earthquake displacement time histories provide input into the lateral resistance interface elements which in turn transfer motion to the pile. Near the pile heads, bending moments will be dominated by the lateral interaction loads generated by inertial effects on the bridge structure. At greater depth (>10D say) where soil stiffness progressively increases with respect to pile stiffness, the pile will be constrained to deform in a similar manner to the free field, and pile bending moments become a function of the curvatures induced by free field displacements.

To illustrate the nature of free field displacements, reference is made to the example shown in Figure 20, where a 200 ft. deep cohesionless soil profile was subjected to the El Centro Earthquake. The free field response was determined using a non-linear one dimensional response analysis, with modulus and damping values characterized by the values shown. From the displacement profiles shown at specific times curvatures can be computed, and pile bending moments calculated if it is assumed that the pile is constrained to displace in phase with the free field response.

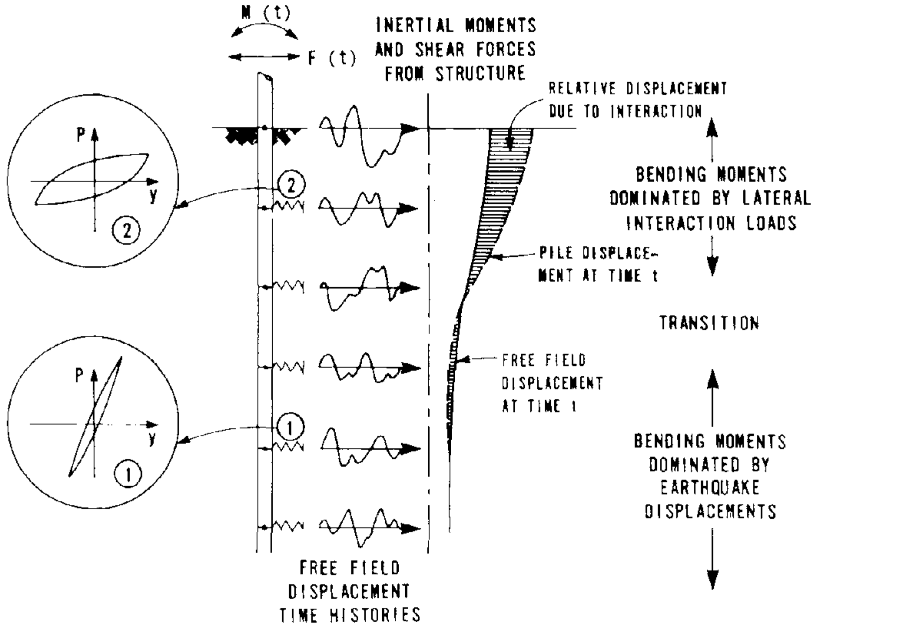


FIG. 19 MECHANISM OF SOIL-PILE INTERACTION DURING SEISMIC LOADING

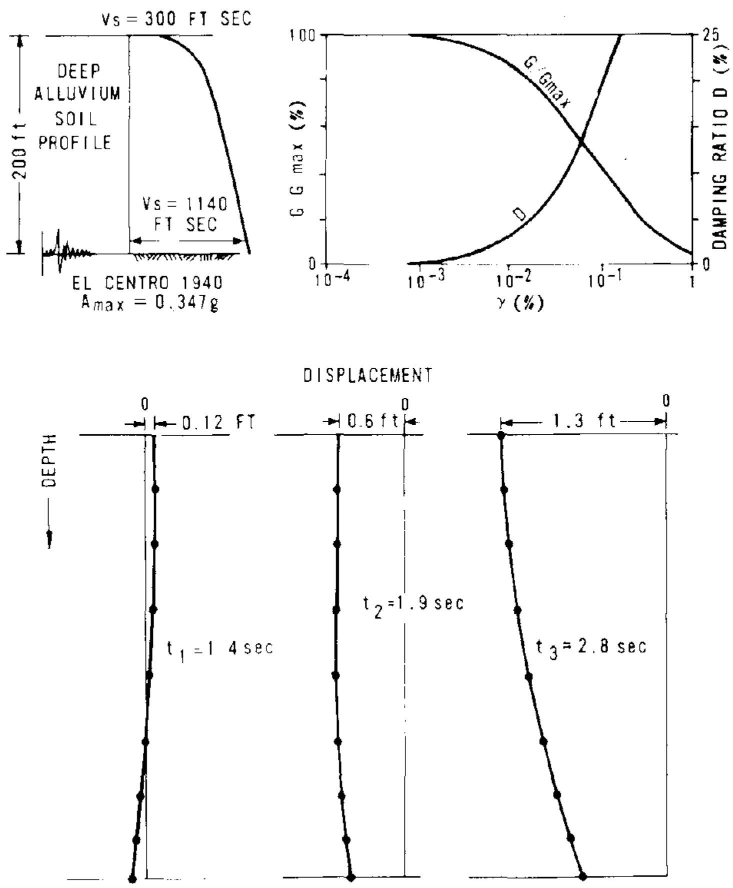


FIG. 20 TYPICAL EARTHQUAKE DISPLACEMENT PROFILES

Large curvatures could potentially develop at interfaces bewteen soft and rigid soils, and clearly in such cases emphasis should be placed on using flexible ductile piles. Margason [25] suggests that curvatures of up to $6x10^{-4}$ inches^{-1} could be induced by strong earthquakes but these should pose no problems to well designed steel or prestressed concrete piles.

Studies incorporating the complete soil-pile-structure interaction system as represented by Figure 19, have been described by Penzein [26] for a bridge piling system in a deep soft clay. A similar but somewhat simpler soil-pile structure interaction system (SPASM) to that used by Penzein, has been described by Matlock et. al. [27, 28]. The model used is in effect a dynamic version of the previously described BMCOL program.

CONCLUSIONS

Field evidence has indicated that damage to bridges arising from earthquake induced liquefaction of saturated cohesionless soils is most severe, being characterized by large vertical and horizontal displacements of abutments and piers. Clearly the best defense against liquefaction is to identify the potential problem before finalizing site selection, particularly if there are options for alternate sites. Given a potentially liquefiable site, design calls for a careful site investigation and evaluation of the liquefaction hazard, an appropriate balance between calculated risk and soil improvement, well anchored abutments and flexible ductile piles extending to depths well below the liquefiable zone.

Foundation design in general should recognize the transient nature of the earthquake loading problem, where the magnitude of cyclic deformations and earthquake induced permanent deformations have a major influence on the structural performance. In assessing foundation stiffness parameters, the non-linear deformation behavior of soils should be taken into account particularly in cases involving strong earthquakes and softer soils. The possible effects of progressive degradation in soil stiffness and strength with cycles of loading should also be evaluated for the latter case. For pile foundations, it should be recognized that in addition to loads imposed by lateral forces from the structure, pile curvatures induced by free field ground displacements may impose significant bending stresses at depth.

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FACTORS INVOLVED IN THE SEISMIC DESIGN OF BRIDGE ABUTMENTS

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ABSTRACT

The nature of damage to bridge abutments in previous earthquakes is described, where it is observed that damage has been primarily related to abutment displacements or transfer of inertia forces from the superstructure. The Mononobe-Okabe pseudo static approach for computing seismically induced lateral pressures is outlined, and the importance of the inclusion of abutment inertia is noted. A recently developed method for computing the magnitude of relative wall displacements during earthquakes is described. Based on this approach, the concept of designing for a specified lateral displacement using a seismic coefficient equal to half the peak value is advanced. Brief comments are also made on the significance of factors such as non-yielding walls, bearing type, and the use of monolithic abutments.

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INTRODUCTION

The numerous case histories of damage or failure to bridges induced by abutment failure or displacement during earthquakes, have clearly demonstrated the need for careful attention to abutment design and detailing in seismic areas. Damage is typically associated with fill settlement or slumping, displacements induced by high seismically induced lateral earth pressures, or the transfer of high longitudinal or transverse inertia forces from the bridge structure itself. Settlement of abutment backfill, severe abutment damage or bridge deck damage induced by the movement of abutments may cause loss of bridge access, and hence abutments must be considered as a vital link in the overall seismic design process for bridges.

The 1971 San Fernando earthquake provided several examples of abutment related damage. Figure 1 illustrates the extent of fill settlement for a typical freeway bridge constructed using diaphragm abutments. Figure 2 illustrates damage to the abutment where high transverse bridge inertia forces have resulted in failure of wing walls, and a lateral movement of the deck relative to the abutment of about 3 feet (≃ 1m). For the latter case the approach fill also slumped several feet. Figure 3 shows slumping of fill and bridge distortion associated with increased retaining wall pressure resulting from the Chilean earthquake of 1960. The displacement of wing walls due to high seismically induced lateral pressures and the associated cracking of fill is illustrated in Figure 4 for a bridge shaken during the 1964 Niigata earthquake.

The nature of abutment movement or damage during past earthquakes has been well documented in the literature. Evans [1] examined the abutments of 39 bridges within 30 miles of the 1968 M7 Inangahna earthquake in N.Z., where 23 showed measureable movement and 15 were damaged. Movements of independent abutments followed the general pattern of outward motion and rotation about the top after contact with and restraint by the superstructure. Fill settlements were observed to be 10-15% of the fill height. Damage effects on bridge abutments in the M7.1 Madang earthquake in New Guinea reported by Ellison [2] , were similar, where abutment movements as much as 20 inches (500 mm) were noted. Damage to abutments in the 1971 San Fernando earthquake are reported by Fung, et. al. [3] . Numerous

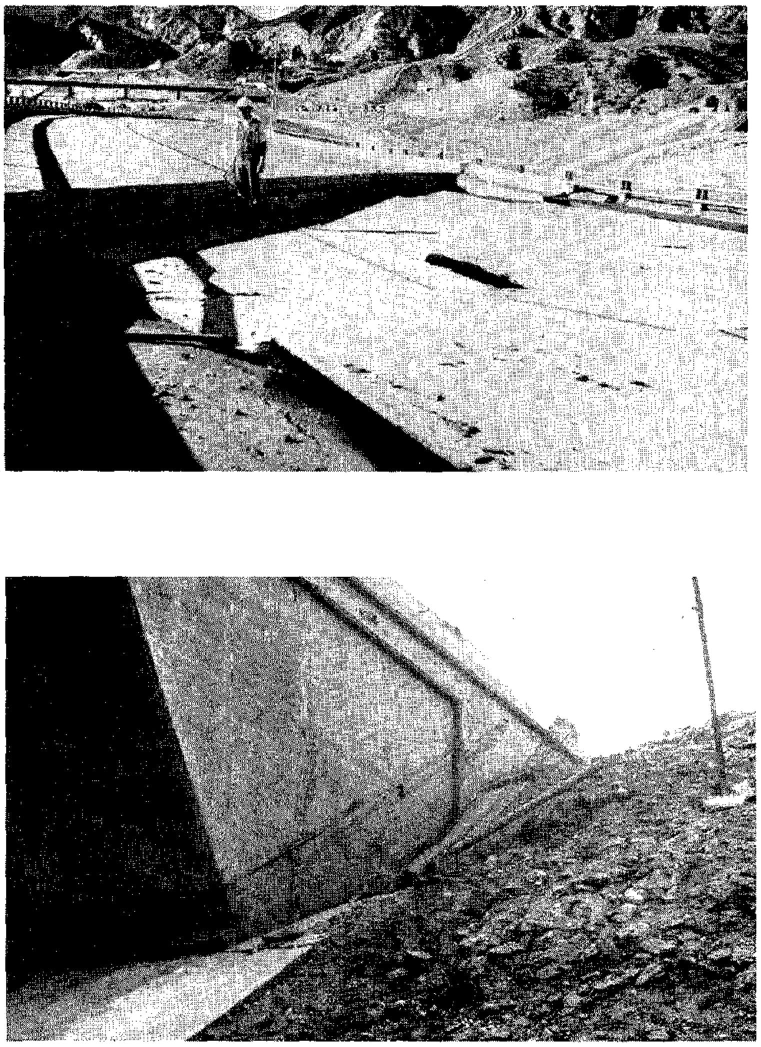
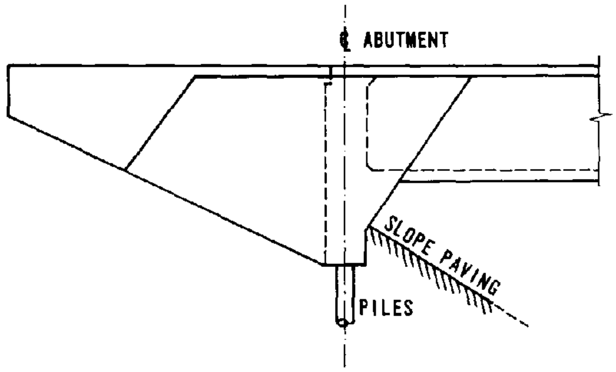


FIG. 1 ABUTMENT AND APPROACH FILL SETTLEMENT - SAN FERNANDO EARTHQUAKE, 1971



SKETCH SHOWING ABUTMENT DETAIL

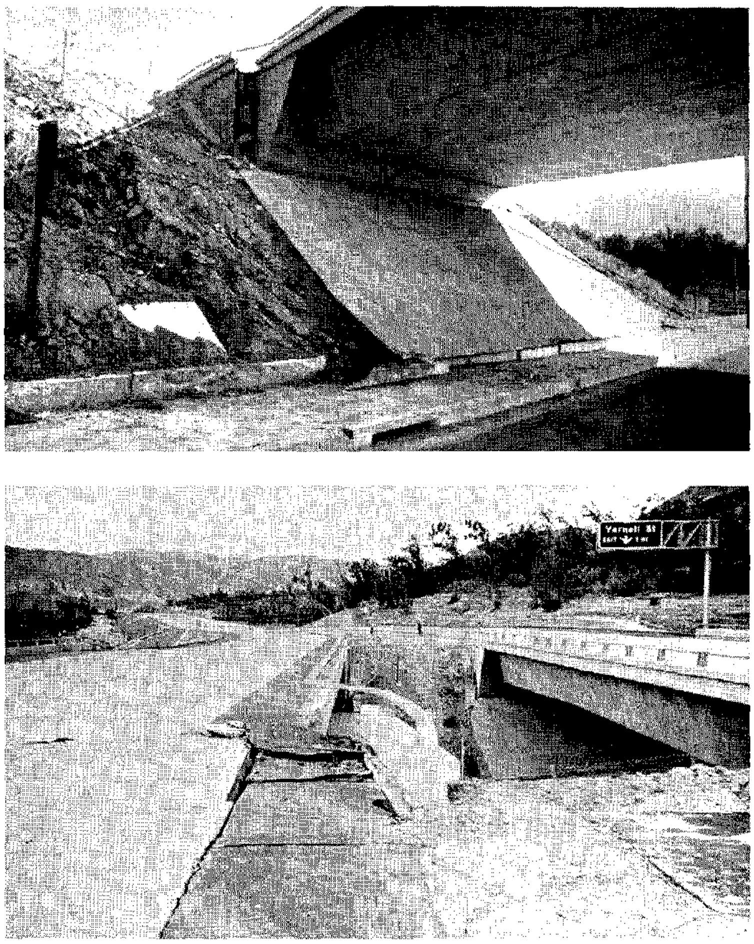
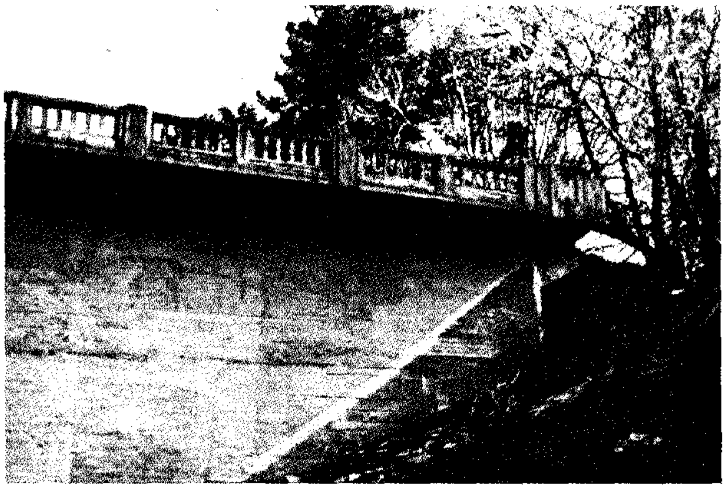


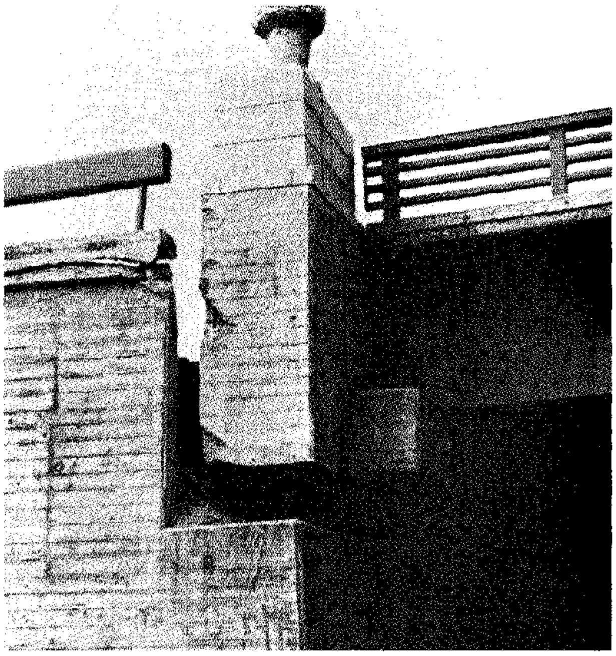
FIG. 2 DAMAGE TO DIAPHRAGM ABUTMENT, ROXFORD ST. UNDERCROSSING, SAN FERNANDO EARTHQUAKE, 1971



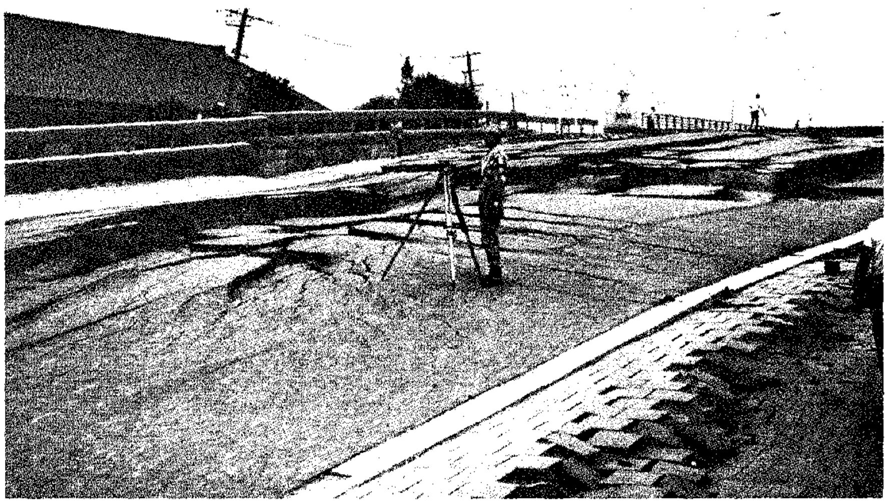
SLUMPING OF APPROACH FILL

DISTORTION DUE TO SOIL PRESSURE ON ABUTMENT

FIG. 3 DAMAGE TO ISLA-TEJA BRIDGE, VALDIVIA, CHILE, 1960



DISPLACEMENT OF WING WALLS



CRACKING OF FILL DUE TO DISPLACEMENT OF WING WALLS

FIG. 4 DAMAGE TO HIGHWAY BRIDGE, NIIGATA, 1964 instances of abutment displacement and associated damage have been reported in publications on the Niigata and Alaskan earthquakes. However, these failures were primarily associated with liquefaction of foundation soils, and are discussed in the workshop paper by Martin [4]. This paper addresses only those design failures related to non-liquefiable foundation soils or fills.

Design features of abutments vary tremendously, and depend on the nature of the bridge site, foundation soils, bridge span length and load magnitudes. Abutment types include free standing gravity walls, cantilever walls, and monolithic diaphragms. Foundation support may use spread footings, vertical piles or battered piles, while connection details to the superstructure may incorporate roller supports, elastomeric bearings or fixed bolted connections. Considering the number of potential design variables, together with the complex nature of soil-abutment-superstructure interaction during earthquakes, it is clear that the seismic design of abutments is a challenging problem. For practical purposes, many simplifying assumptions are necessary.

For free standing abutments such as gravity or cantilever walls, which are able to displace laterally during an earthquake, the well-established Mononobe-Okabe pseudo static approach is widely used to compute earth pressures induced by earthquakes. This approach is reviewed below, and the influence of variables such as the seismic coefficient, soil friction angle and backfill slope is described. The importance of including abutment inertia forces in calculations is also noted.

For free standing abutments in highly seismic areas, the design of abutments to provide zero displacement may be unrealistic, and design for an acceptable small lateral displacement may be preferable. A recently developed method for computing the magnitude of relative wall displacement during a given earthquake is briefly described. Based on this simplified approach, a method is suggested for the selection of a pseudo static seismic coefficient and corresponding displacement level for a given effective peak ground acceleration.

To conclude the paper, brief comments are made on the effects of abutment restraint on lateral earth pressures, and the effects of transfer of superstructure inertia forces in the case of abutments which cannot be regarded as being isolated from the bridge structure.

MONONOBE-OKABE ANALYSIS

Soil Forces - Limiting Equilibrium Values

The most frequently used method for the calculation of the seismic soil forces acting on a bridge abutment is a static approach developed in the 1920's by Mononobe [5] and Okabe [6]. The Mononobe-Okabe analysis is an extension of the Coulomb sliding-wedge theory taking into account horizontal and vertical inertia forces acting on the soil: the analysis is described in detail by Seed and Whitman [7]. The following assumptions are made:

1. The abutment is free to move sufficiently that the soil strength will be mobilized. If the abutment is rigidly fixed and unable to move, the soil forces will be very much higher than those predicted by the Mononobe-Okabe analysis.

2. The backfill is cohesionless, with a friction angle ϕ.

3. The backfill is unsaturated, so that liquefaction problems will not arise.

Equilibrium considerations of the soil wedge behind the abutment (Fig. 5) then lead to a value E_{AE} of the force exerted on the soil mass by the abutment (and vice versa), when the abutment is at the point of failure: E_{AE} is given by the expression

$$E_{AE} = 1/2\gamma H^2(1-k_v).\ K_{AE} \qquad (1)$$

where the seismic active pressure coefficient K_{AE} is

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\ \cos^2\beta\ \cos(\delta+\beta+\theta)\ \left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^2} \quad \text{and where} \qquad (2)$$

γ = unit weight of soil

H = height of soil face

ϕ = angle of friction of soil

$\theta = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$

δ = angle of friction between soil and abutment

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

i = backfill slope angle

β = slope of soil face

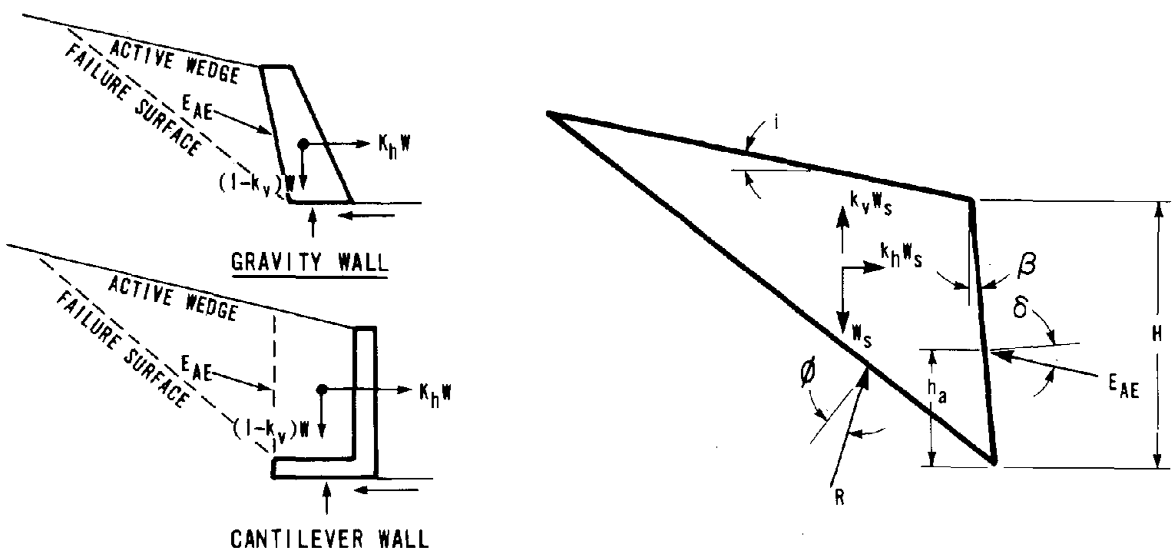


FIG. 5 ACTIVE WEDGE FORCE DIAGRAM

The equivalent expression for passive force if the abutment is being pushed into the backfill is

$$E_{PE} = 1/2\ \gamma H^2(1-k_v).\ K_{PE} \tag{3}$$

where

$$E_{PE} = \frac{\cos^2(\phi-\theta+\beta)}{\cos\theta\ \cos^2\beta\ \cos(\delta-\beta+\theta)\ \left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta+i)}{\cos(\delta-\beta+\theta)\sin(i-\beta)}}\right]^2} \tag{4}$$

As the seismic inertia angle θ increases, the values of K_{AE} and K_{PE} approach each other and become equal when, for a vertical backfill, $\phi = \theta$.

Despite the relative simplicity of the approach, the accuracy of Eq. (1) has been substantiated by model tests [7] and by back calculation from observed failures of flood channel walls [8]; though in the latter case the displacements were large which, as will be seen, can modify the effective values of k_h at which failure occurs.

The value of h, the height at which the resultant of the soil pressure acts on the abutment, may be taken as H/3 for the static case with no earthquake effects involved. However, it becomes greater as earthquake effects increase. This has been shown by tests and by theoretical results derived by Wood [9, 10], who found that the resultant of the dynamic pressure acted approximately at midheight. Seed and Whitman have suggested that h could be obtained by assuming that the static component of the soil force (computed from Eq. 1 with $\theta = k_v = 0$) acts at H/3 from the bottom of the abutment, while the additional dynamic effect should be taken to act at a height of 0.6H. For most purposes it would be sufficient to assume h = H/2, with a uniformly distributed pressure.

Although the Mononobe-Okabe expression for active thrust is easily evaluated for any particular geometry and friction angles, the significance of the various parameters is not obvious. Figure 6 shows the variation of K_{AE} against k_h for different values of ϕ and k_v. K_{AE} is seen to be very sensitive to the value of ϕ. Also for a constant value of ϕ, k_{AE} is seen to double as k_h increases from zero to 0.35 for zero vertical acceleration, and thereafter it increases more rapidly.

In order to see more easily the increase in soil active pressure due to earthquake effects, K_{AE} can be normalized by dividing its static value K_A to give a thrust factor

$$F_T = K_{AE}/K_A \tag{5}$$

Whereas Figure 6 shows that K_{AE} is rather sensitive to changes in the soil friction angle ϕ, the plots of F_T against ϕ in Figure 7 indicates that the value of ϕ has little effect on the thrust factor until quite suddenly, over a short range of ϕ, F_T increases rapidly, and becomes infinite for specific critical values of ϕ. The reason for this behavior may be determined by examination of Eq. 2. The contents of the radical must be positive for a real solution to be possible, and for this, it is necessary that

$$\phi \geq i + \theta = i + \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \tag{6}$$

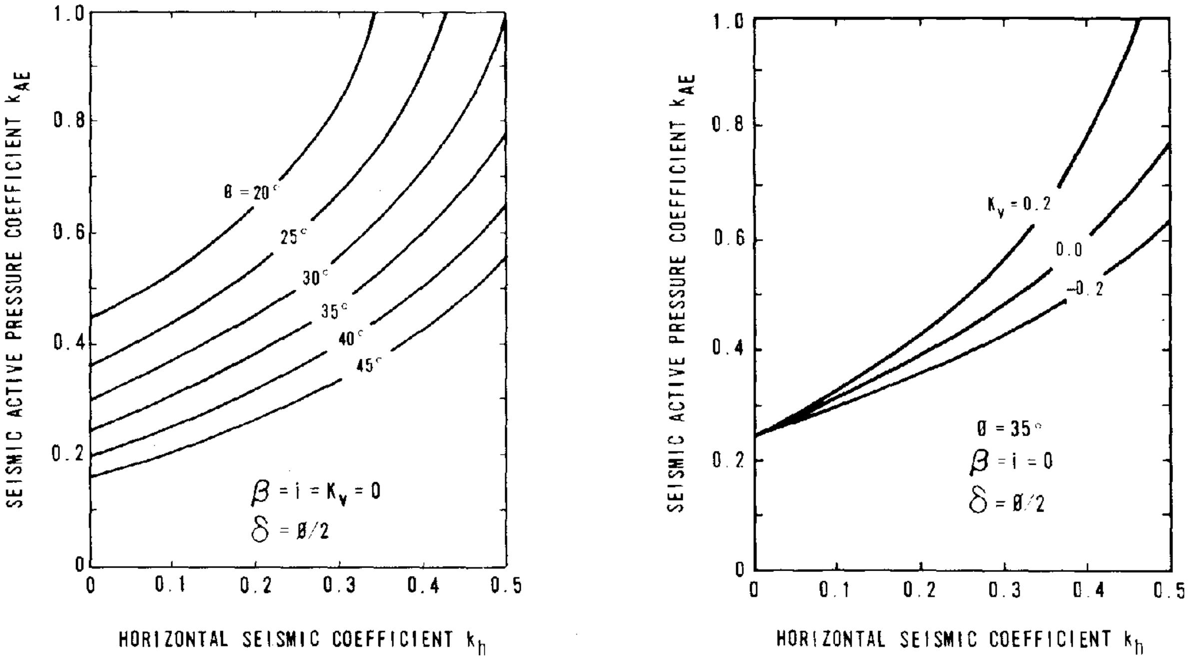


FIG. 6 EFFECT OF SEISMIC COEFFICIENTS AND SOIL FRICTION ANGLE ON SEISMIC ACTIVE PRESSURE COEFFICIENT

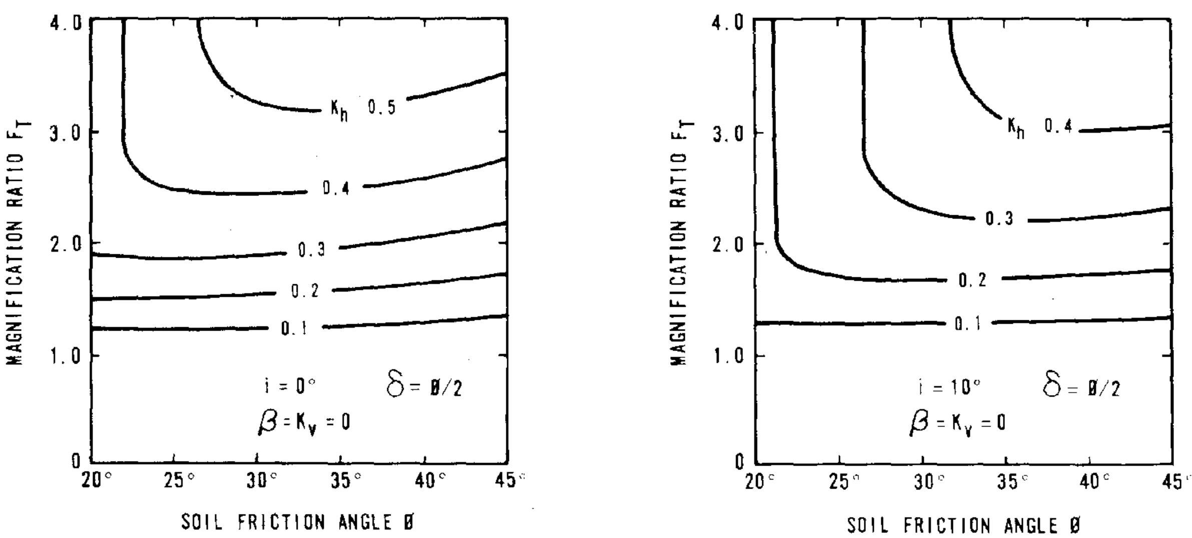


FIG. 7 INFLUENCE OF SOIL FRICTION ANGLE ON MAGNIFICATION RATIO

This condition could also be thought of as specifying a limit to the horizontal acceleration coefficient that could be sustained by any structure in a given soil: the limiting condition is that

$$k_h \leq (1-k_v)\tan(\phi-i) \tag{7}$$

For zero vertical acceleration and backfill angle and for a soil friction angle of 35°, the limiting value of k_h is 0.7. This is a figure of some interest in that it provides an absolute upper bound for the seismic acceleration which can be transmitted to any structure whatsoever that is built on a soil with the given strength characteristics.

Figure 8 shows the effect of changes in the vertical acceleration coefficient k_v on F_T. Positive values of k_v have a significant effect for values of k_h greater than 0.2. The effect is greater than 10% above and to the right of the dashed line.

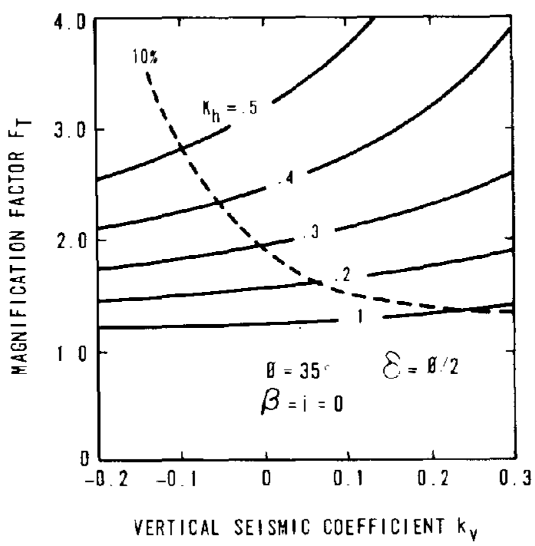


FIG. 8 INFLUENCE OF VERTICAL SEISMIC COEFFICIENT ON MAGNIFICATION RATIO

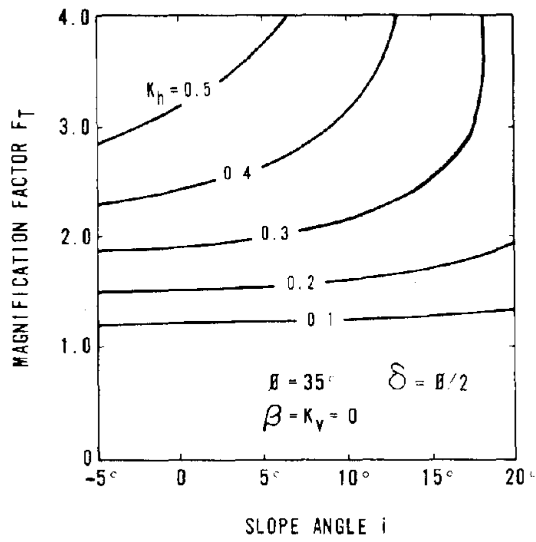


FIG. 9 INFLUENCE OF BACKFILL SLOPE ANGLE ON MAGNIFICATION RATIO

As is to be expected from Eq. 6, K_{AE} and F_T are also sensitive to variations in backfill slope, particularly for higher values of horizontal acceleration coefficient when the limit implied by Eq. 6 is approached, This effect is shown in Figure 9.

Effects of Abutment Inertia

Some current procedures assume that the inertia forces due to the mass of the abutment itself may be neglected in considering seismic behavior and seismic design. This is an unconservative assumption, and for those abutments relying on their mass for stability it is also an unreasonable assumption in that to neglect the mass is to neglect a major aspect of their behavior.

Following earlier work [11, 12], the effect of including abutment inertia effects may be demonstrated by considering the rather simpler case of a gravity retaining wall. Figure 10 shows a free-body diagram of a retaining wall (or free standing bridge abutment) with height h and weight W.

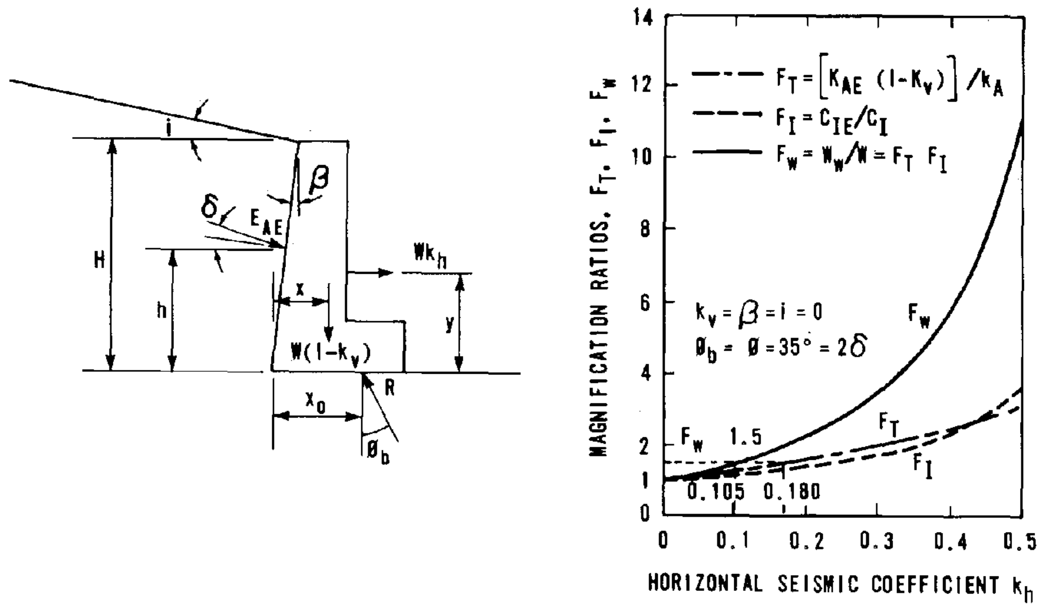


FIG. 10 EFFECT OF ABUTMENT INERTIA ON MAGNIFICATION RATIOS

Resolving the base reaction R into horizontal and vertical components F and N, force equilibrium gives

$$N = (1-k_v)W_w + E_{AE}\sin(\delta+\beta) \tag{8}$$

$$F = E_{EA}\cos(\delta+\beta) + k_h W \tag{9}$$

Now, at the point of sliding,

$$F = N\tan\phi_b \tag{10}$$

where ϕ_b is the friction angle at the base of the wall. Solving for the wall weight W_w that just prevents sliding, we obtain

$$W_w = C_{IE}\,E_{AE} \tag{11}$$

where

$$C_{IE} = \frac{\cos(\delta+\beta) - \sin(\delta+\beta)\tan\phi_b}{(1-k_v)(\tan\phi_b - \tan\theta)} \tag{12}$$

If we now define a wall inertia factor

$$F_I = \frac{C_{IE}}{C_I} \tag{13}$$

where C_I is the static value of C_{IE}, and if we slightly redefine the thrust factor F_T to be

$$F_T = \frac{E_{AE}}{E_A} \tag{14}$$

where E_A is the static active soil force, then the product F_w of these two factors is an amplification factor applied to the mass of the wall to allow for seismic effects on both soil pressure and wall inertia: its value is given by the expression

$$F_W = F_T F_I = \frac{W_w}{W} \tag{15}$$

where W is the weight of the wall required for static equilibrium. As can be seen from Figure 10, F_T and F_I are of the same order of magnitude for most values of k_h. Thus the wall inertia term cannot be neglected for the seismic design of abutments or gravity retaining walls.

Stability Against Tilting

Considering again in this and the next section the rather simpler case of the gravity retaining wall, because Eq. 1 assumes the soil is in a critical state at which failure is just taking place, the horizontal acceleration coefficient k_h should be regarded as the critical acceleration coefficient at which the wall will begin to move. If k_h is exceeded in an earthquake event, displacement of the wall will take place. If movement is going to occur, it is better that the wall should move by sliding outwards rather than by tilting. A condition for this may be expressed in terms of the position of the center of pressure of the forces acting on the base of the wall. Consideration of the equilibrium of the forces acting on the wall (Figure 1) shows that in order that the wall should slide rather than overturn, it is necessary that the value of the distance x_o from the inner toe of the wall to the point of action of the resultant force on the base of the wall should at least be equal to

$$x_o = \frac{E_{AE}h\,[\cos(\beta+\delta) + \tan\beta\,\sin(\beta+\delta)] + W_w[k_h\bar{y} + (1-k_v)\bar{x}]}{E_{AE}\sin(\beta+\delta) + (1-k_v)W_w} \tag{16}$$

where

h = height of resultant soil force (take h = H/2)

$\bar{x}$, $\bar{y}$ = coordinates of wall center of gravity

DESIGN FOR DISPLACEMENT

A difficulty arises that if the wall is designed using a reasonable value of acceleration coefficient, then its mass will often need to be excessively great. An alternative procedure may be used to overcome this problem and produce a more economic design: the wall should be designed for a small predetermined displacement in an earthquake, rather than for no movement at all.

In order to develop such a procedure, it is first necessary to obtain an expression for the maximum displacement of a wall in a given earthquake.

Tests have shown that a gravity retaining wall fails in an incremental manner in an earthquake. For any earthquake record, the total relative displacement is finite, and is calculable by the Newmark sliding block method [13], developed originally for computing displacements of earth dams and embankments. It assumes a displacement pattern similar to that of a block resting on a plane rough horizontal surface subjected to an earthquake, with the block being free to move against frictional resistance in one direction only. Figure 11 shows how the relative displacement relates to the acceleration and velocity time histories of soil and wall. At a critical acceleration coefficient of k_h, the wall is assumed to begin sliding: relative motion will continue until wall and soil velocities are equal. Figures 12 and 13, taken from an earlier publication [11], show the results of a computation of wall displacement for k_h = 0.1 for the El Centro 1940 N-S record.

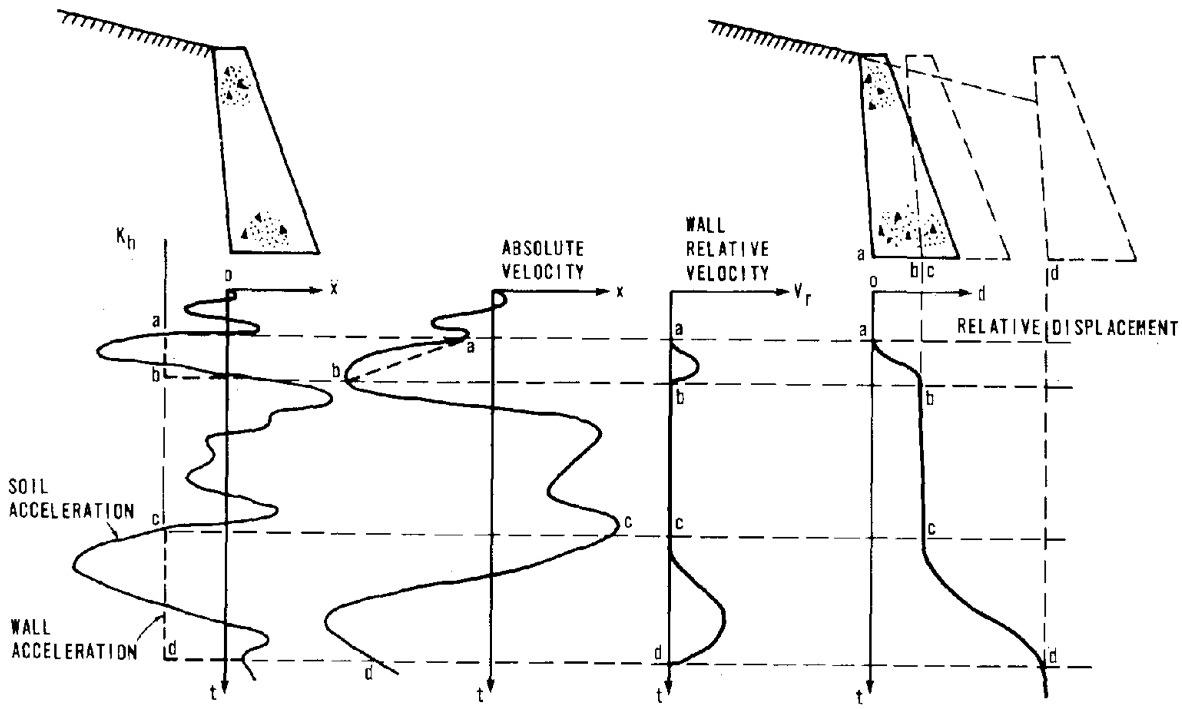


FIG. 11 RELATION BETWEEN RELATIVE DISPLACEMENT AND ACCELERATION AND VELOCITY TIME HISTORIES OF SOIL AND WALL

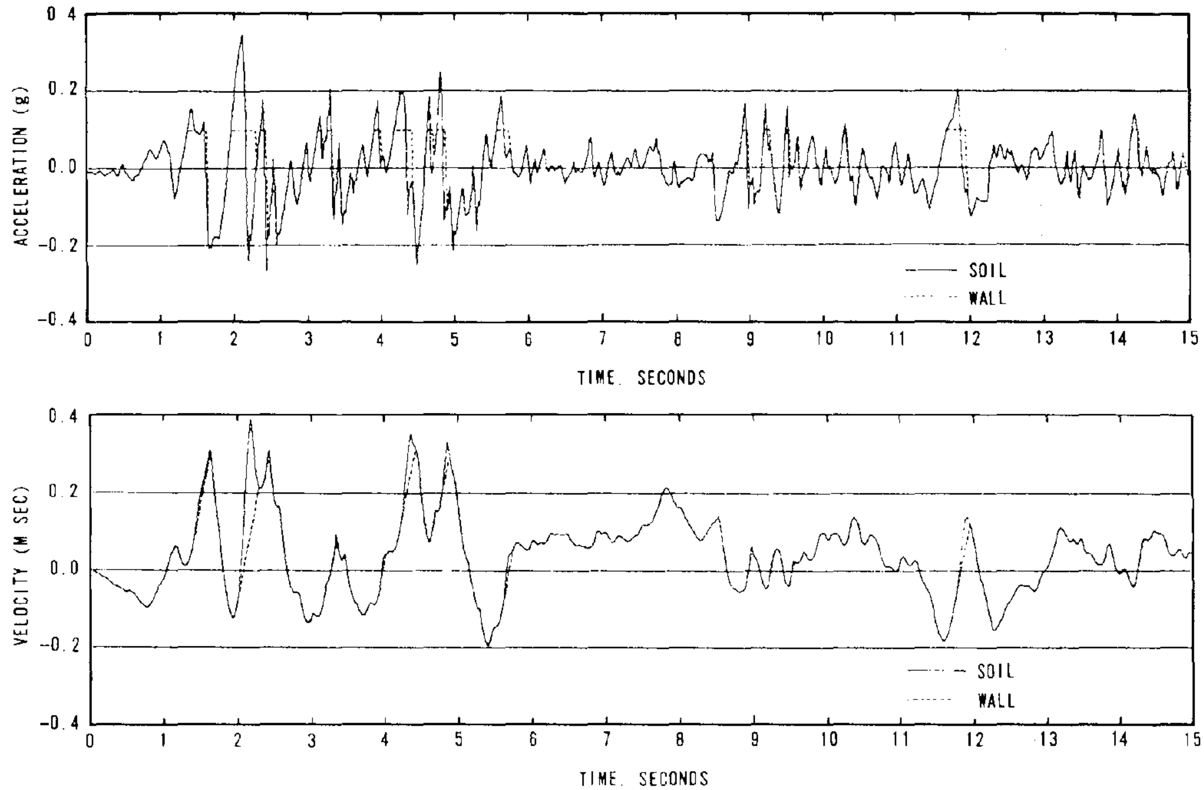


FIG. 12 ACCELERATION AND VELOCITY TIME HISTORIES OF SOIL AND WALL (EL CENTRO 1940 N-S RECORD)

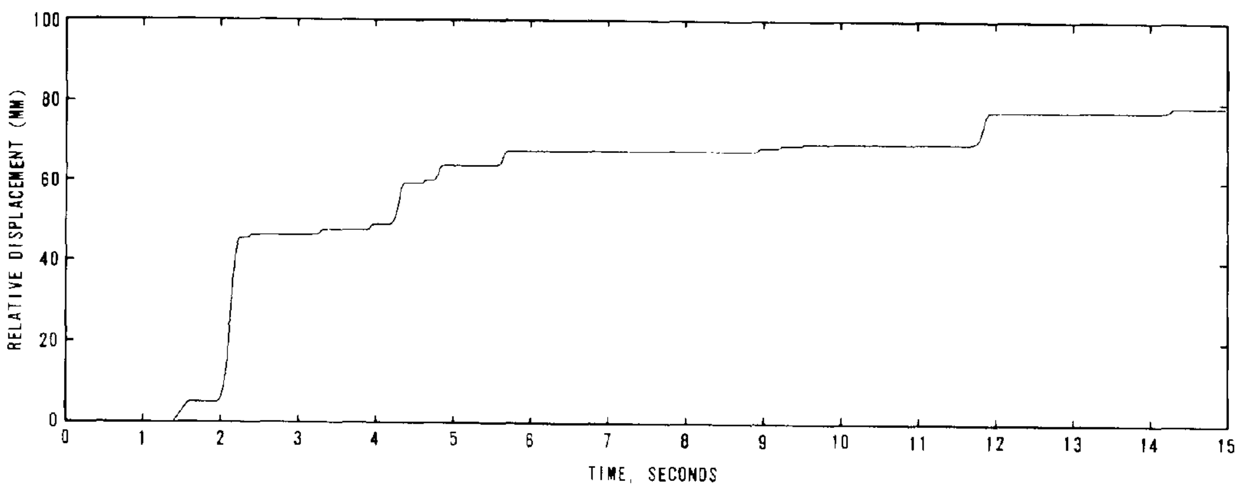


FIG. 13 RELATIVE DISPLACEMENT OF WALL (EL CENTRO 1940 N-S RECORD)

Newmark computed the maximum displacement response for four earthquake records, and plotted the results after scaling the earthquakes to a common maximum acceleration and velocity. Franklin and Chang [14] repeated the analysis for a large number of both natural and synthetic records and added their results to the same plot. Upper bound envelopes for their results are shown in Figure 14. All records were scaled to a maximum acceleration **coefficient** A of 0.5g and a maximum velocity of 30 in/sec. The maximum resistance coefficient N is the maximum acceleration coefficient sustainable by a sliding block before it slides: in the case of a wall, the maximum coefficient is, of course, k_h.

Figure 14 shows that the displacement envelopes for all the scaled records have roughly the same shape.

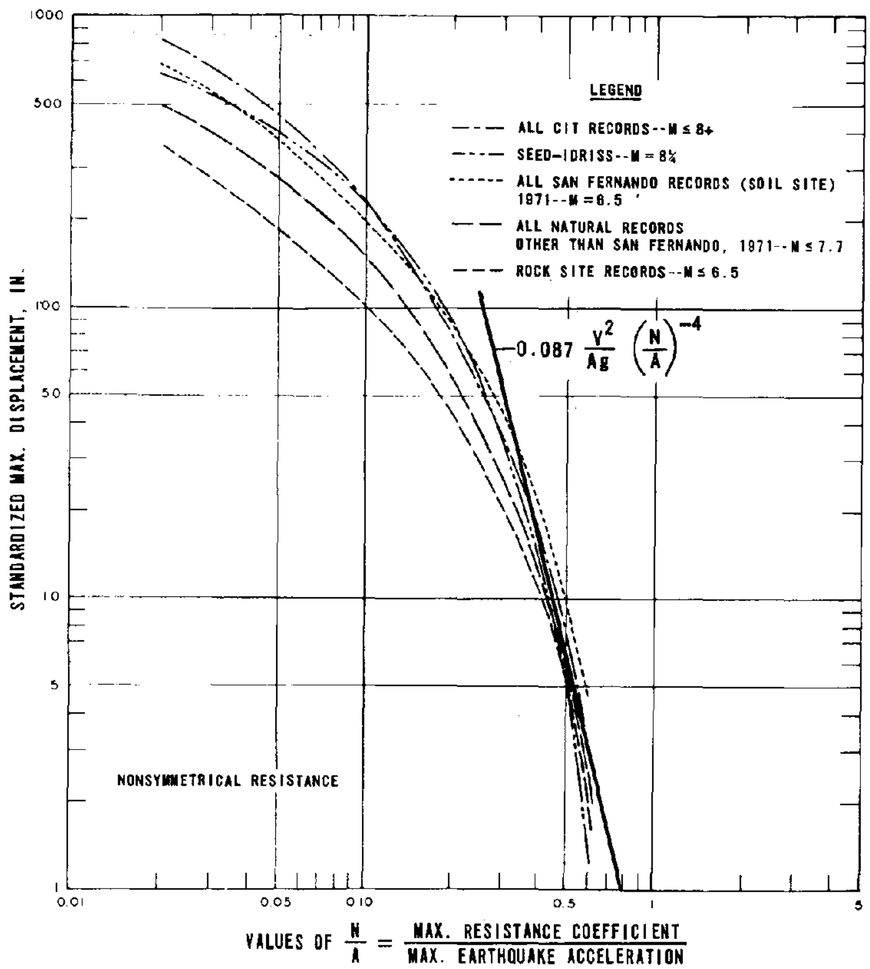


FIG. 14 UPPER BOUND ENVELOPE CURVES OF PERMANENT DISPLACEMENTS FOR ALL NATURAL AND SYNTHETIC RECORDS ANALYSED BY FRANKLIN AND CHANG (1 in. 25.4 mm)

An approximation to the curves for relatively low displacements is given by the relation, expressed in any consistent set of units.

$$d = 0.087 \frac{V^2}{Ag} (\frac{N}{A})^{-4} \qquad (17)$$

where d is the total relative displacement of a wall subjected to an earthquake record whose maximum acceleration coefficient and velocity are A and V. This is drawn as a straight line on Figure 14. Note that as this expression has been derived from envelope curves, it will overestimate d for most earthquakes.

One possible design procedure involves choosing a desired value of maximum wall displacement d together with appropriate earthquake parameters, and using Eq. 17 to derive a value of k_h for which the wall should be designed. The procedure is as follows:

1. Decide on an acceptable maximum displacement d. The wall connections, if any, should be detailed to allow for this displacement.

2. Invert Eq. 17 to obtain the value of k_h corresponding to d. For A use the effective peak acceleration A_a given by the ATC-3 draft recommendations [15]. For all practical purposes, V may be taken as approximately equal to 30 A_a inches per second. We then obtain the expression:

   $$k_h = 0.67\ A_a \{\frac{A_a}{d}\}^{1/4} \qquad (18)$$

   Thus using a specified value of d and the location maps given by the draft ATC-3 recommendations to define A_a, k_h can be computed.

3. Use k_h in Eq. 11 to obtain the required wall weight W_w.

As an example, consider the design of a 16 ft (4.9m) reinforced concrete wall with an inner batter angle β of -5 and with horizontal backfill. Assume that for the soil, $\phi = \phi_b = 33°$, $\delta = 15.5°$ and $\gamma = 100\ lb/ft^3$ ($1600\ kg/m^2$) Assume that, in terms of the draft ATC-3 provisions [15] the location is in map area 5 for which $A_a = 0.2$.

First, attempt to design the wall for zero movement. Using $k_h = 0.2$ and a factor of safety = 1, Eqs. 1 and 11 lead to a required weight of 8,960 lb/ft (11,840 kg/m). This is far too large, and would lead to an average thickness of 3.8 ft (1.17m).

Allow a displacement of 4" (100mm) to occur and detail suitably. From Eq. 18,

$$k_h = 0.2\{\frac{(0.2)(0.2)^2}{(0.2)(0.4)}\}^{1/4} = 0.063$$

Hence, again using a safety factor of 1.0, the required weight is 5,120 lb/ft (6,670 kg/m) - a reduction of nearly 50%. The average thickness required, assuming a unit weight of 150 lb/ft^3 for concrete, is 2.1 ft (0.67m).

Note that if this wall were designed with a factor of safety of 1.5 for static values only, it would require an average thickness of 2'-6" (0.76m). There would thus be no gain in designing the wall for an allowable displacement greater than 4" (100mm).

Displacements for given values of A_a may be obtained by re-ordering Eq. 18 to give:

$$d = 0.2A_a(\frac{k_h}{A_a})^{-4} \quad \text{inches} \tag{19}$$

Thus, if the wall or abutment were simply designed for a static value of, say, half the code acceleration coefficient, i.e., if $k_h = 0.5\ A_a$, then the displacement to be expected for map area 7 for which $A_a = 0.4$ would be $d = (0.2)(0.4)(0.5)^{-4} = 1.3$ in. This is a very small displacement, indicating that the design acceleration coefficient could be lowered further. If we take $k_h = 0.4\ A_a$, then a displacement of 3.1 inches would result. This degree of displacement could easily be allowed for in the detailing of most designs.

Based on the above results and allowing for an additional measure of safety, it is suggested that a design acceleration coefficient for use in the Mononobe-Okabe analysis of 0.5 A_a would be adequate for most design purposes, provided that allowance be made for an outward displacement of 10 A_a inches (250 A_a mm) to occur in the design earthquake.

It is recognized that the above displacement analysis has been carried out for the simple case of a gravity wall. However, in principle, the same approach may be used for more complex abutment systems where additional forces may be imposed by sliding bearings or partial restraints. For more complex abutments, the geometry and size of abutments may be initially determined from non-seismic conditions, in which case one may choose to make an assessment of the magnitude of displacement during a given earthquake. For a given system, once the equivalent pseudo static seismic coefficient has been established for a limiting equilibrium condition, and the ratio of the latter coefficient to the peak ground acceleration coefficient established, then the curves shown in Figure 14 may be used to assess the permanent displacement. Brief comments on the nature of bearings and constraints together with other design factors are given below.

BEARINGS, CONSTRAINTS AND OTHER DESIGN FACTORS

Bearings

Many types of abutment bearings are used to support the superstructure, and are discussed in detail in other contributions to the workshop. For sliding steel bearings or pot bearings, the force diagrams which would be used to study displacement characteristics of a simple abutment are shown in Figure 15. It is noted that once a design is established to satisfy displacement criteria under active pressure conditions, a check may be made to ensure sufficient passive resistance can be mobilized when inertia forces act in the reverse direction. Where bearings comprise unconfined elastomeric pads, the nature of the forces transferred to the abutment become somewhat more complex, as such bearings are capable of transferring significant force. The magnitude of the force initially depends on the relative movement between the superstructure and the abutment, and force magnitudes can become quite large before slip will occur.

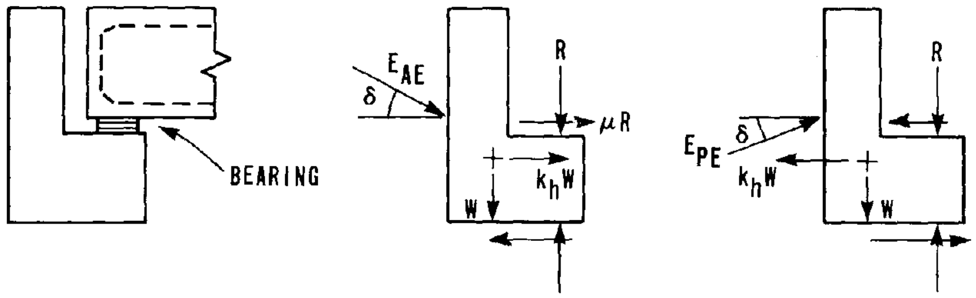


FIG. 15 FORCE DIAGRAMS INCLUDING BEARING FRICTION

A typical abutment support detail used by the New Zealand Ministry of Works is shown in Figure 16. It may be seen that linkage bolts are incorporated to prevent spans dropping off supports. The rubber rings act as buffers to prevent impact damage in the event that the provided lateral displacement clearance is inadequate. The use of a settlement slab is also noted, which has the effect of providing bridge access in the event of backfill settlement. The slab also provides an additional abutment friction anchorage against lateral movement.

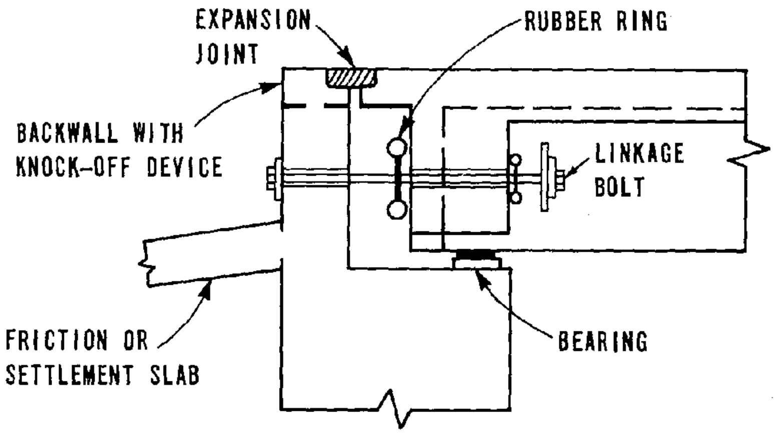


FIG. 16 ABUTMENT SUPPORT DETAIL

Non-Yielding Abutments

As previously noted, the Mononobe-Okabe analysis assumes that the abutment is free to yield laterally a sufficient amount to mobilize peak soil strengths in the soil backfill. For granular soils, peak strengths can be assumed to be mobilized if deflections at the top of the wall are about 0.5% of the abutment height. For abutments which are restrained against lateral movement by tie backs or raked piles, lateral pressures induced by inertia forces in the backfill will be greater than those given by a Mononobe-Okabe analysis. Simplified elastic solutions presented by Wood [9] for rigid non-yielding walls, indicate that pressures could be twice those given by Mononobe-Okabe. The use of a factor of 2 in conjunction with peak ground accelerations is suggested for design, where doubt exists that an abutment can yield sufficiently to mobilize soil strengths. For more detailed design of important bridge structures, it may be justifiable to undertake a dynamic finite element analysis such as described by Wood [10] and Chen and Penzein [16].

Monolithic Abutments

End diaphragm abutments such as shown in Figure 17 are commonly used for single and two span bridges in California. The end diaphragm is cast monolithically with the superstructure, and may be directly supported on piles as shown in Figure 2, or provision may be made for beam shortening during post-tensioning as shown in Figure 17. The diaphragms act as a retaining wall with the superstructure acting as a prop between abutments.

Such abutments have performed well during earthquakes and avoid problems such as backwall and bearing damage associated with yielding abutments, and reduce the lateral load taken by columns or piers. On the other hand, higher longitudinal and transverse superstructure inertia forces are transmitted directly into the backfill and provision must be made for adequate passive resistance to avoid excessive relative displacements. In the case of the transverse forces, closer attention must be paid to wing wall design as indicated by the failure shown in Figure 2.

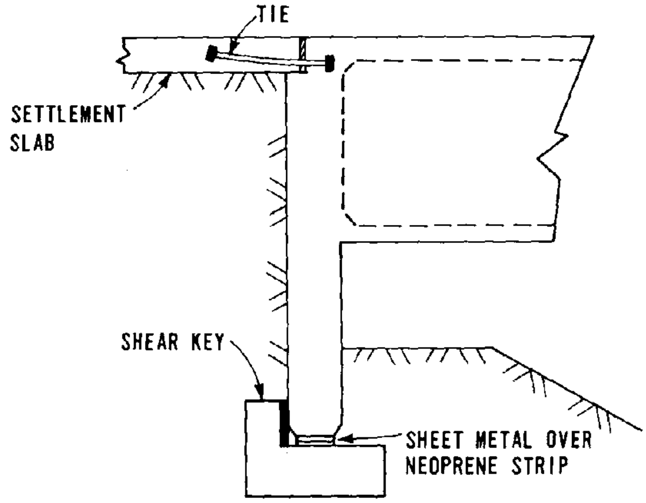


FIG. 17 END DIAPHRAGM ABUTMENT

General Observations

Whereas this paper has primarily focused on a simplified approach to assessing lateral earth pressures and the associated lateral abutment displacements induced by earthquakes, there are clearly many additional practical design considerations which must be borne in mind when making design judgements. In the case of independent abutments which are able to yield, the overall seismic stability of foundation fills must be assessed. Additional contributions to abutment displacement can result if supporting foundation fill slumps during an earthquake. In the case of abutments supported on footings, an approximate estimate of relative displacements over assumed failure planes passing beneath the abutment may be made using Newmark's method. Where abutments are supported by raked piles, such displacements could transfer high lateral loads to the pile system, and it may be preferable to use vertical piles which are able to displace laterally.

The question of appropriate dynamic soil strength parameters to use for the seismic earth pressure and displacement computations described above is also of practical design significance. For dry granular backfill, it is reasonable to use the estimated static angle of friction, although cyclic strengths may be slightly higher. With increasing fines leading through to compacted cohesive backfill, soil strengths capable of being mobilized during cyclic loading may be significantly greater than static strengths.

Another soil factor of importance is that of residual lateral pressures induced in granular backfill by cyclic loading. The tendency for granular soils to compact during cyclic shearing results in a progressive increase in residual lateral or horizontal stress. Such stresses would be superimposed on those induced by inertial effects. However, the effect would be minimized for well compacted backfill.

CONCLUSIONS

Field evidence from past earthquakes has indicated that abutment related damage is primarily related to displacements associated with seismically induced lateral earth pressures, or the transfer of high longitudinal or transverse inertia forces from the bridge superstructure.

Whereas the conventional Mononobe-Okabe pseudo static approach for computing seismically induced earth pressures is an appropriate design method for yielding abutments, the inertia forces from the abutment should be incorporated in analyses, together with consideration of forces transmitted from the superstructure through support bearings.

A method for computing the displacement of independent yielding abutments during earthquakes for a specified peak earthquake acceleration has been described. Based on this procedure, the concept of designing for a specified abutment displacement is advanced. Calculations suggest that the use of a seismic coefficient of half the peak value in conjunction with the Mononobe-Okabe approach, would lead to acceptable lateral displacements of say less than 4 inches (100mm).

For non-yielding abutments, elastic solutions suggest that twice the seismic lateral pressures computed using Mononobe-Okabe in conjunction with peak accelerations would be appropriate for design purposes.

For monolithic abutments and for abutments where significant longitudinal and transverse superstructure inertia force is transmitted to the backfill or foundations, the passive resistance capable of being mobilized by the abutment design should be considered.

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SEISMIC RESISTANCE OF REINFORCED CONCRETE BRIDGE COLUMNS

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ABSTRACT

Common causes of failure of reinforced concrete bridge columns under seismic attack are discussed. Results of dynamic inelastic time-history computer analyses are used to assess the significance of earthquake characteristics, natural period of bridge pier, design lateral force level, hysteresis loop type and additional non-structural flexibility to the ductility demand of bridge columns. The importance of carefully detailed confining steel to adequate section ductility at plastic hinge zone is established. Requirements of different codes and design methods are compared with results of rational analyses based on stress-strain curves for steel and confined concrete, and with results from dynamic and static cyclic load tests of bridge pier models. It is concluded that dependable results can be obtained from carefully detailed bridge columns at displacement ductility factors of six or more. Further theoretical and experimental research needs are outlined.

SEISMIC RESISTANCE OF REINFORCED CONCRETE BRIDGE COLUMNS

by

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INTRODUCTION

Inelastic response of bridge structures under seismic attack will invariably involve plastic hinging of the bridge columns, unless mechanical energy dissipators are incorporated in the design. In general the superstructure design is governed by dead and live load considerations and it is both impractical and undesirable to design for plastic hinging in superstructure members. Behaviour is consequently different in concept from that required of building frames, where a capacity design approach is adopted to ensure beam hinging by specifying column flexural and shear strengths to be higher than the maximum input associated with beam hinges forming at maximum feasible beam strength.

This fundamental difference in philosophy between building frames and bridge frames has meant that much of research on building frames lacks relevence in the bridge seismic design field. For building frames, research emphasis has tended towards providing adequate ductility in the beam hinge regions and solving problems associated with shear and bar slippage in beam/column joints, with little consideration of the available ductility of column sections.

Thus, in terms of research effort, the seismic performance of bridges has been a poor cousin to the building field. This is despite obvious evidence of problems in bridge design philosophy displayed after recent earthquakes. Of particular importance is the example provided by the 1971 San Fernando earthquake, where 42 bridge structures received significant damage, and five structures collapsed [1]. Much of the damage was a consequence of inadequate detailing of the bridge columns, resulting in (a) insufficient ductility capacity to withstand the inelastic displacements imposed, (b) shear failure of columns, or (c) anchorage failure of longitudinal reinforcement in plastic hinges forming at the column bases.

Damage to bridge piers in the San Fernando earthquake highlighted the need for reassessment of existing seismic design practice for bridges, and provided impetus to additional theoretical and experimental research. This paper summarizes recent research into the ductility demand and ductility capacity of bridge piers under seismic attack. Shear design and detailing requirements are discussed, and provisions of relevant codes compared with each other and with results of test programmes.

DUCTILITY DEMAND

In terms of current ductile design, the severity of earthquake attack on a particular structure is normally interpreted to mean the maximum curvature ductility demand on the structure's plastic hinges. For a given bridge pier, this will be a complex function of a number of variables including

earthquake characteristics (intensity and frequency distribution)
design force level
period of structure
shape of the inelastic hysteresis loop
elastic damping coefficient
contribution of foundation and bearing compliance to structural flexibility
plastic hinge length.

The damage potential will also be related in some fashion to the ratio of duration of strong motion shaking to natural period of the structure, which may be expected to be an indicator of the number of yield excursions, and hence of the cumulative ductility. There are some grounds for considering the cumulative ductility to be a more useful index than the peak ductility level - for example, 10 cycles at a curvature ductility factor of 8 might be expected to be more damaging than one yield excursion to a curvature ductility factor of 10 or 12. However, there is little experimental evidence in support or contradiction of this view.

Although a design level of displacement ductility is implicit in design codes specifying seismic design force levels below expected elastic response levels, the actual value is rarely required to be checked. An exception is New Zealand's Highway Bridge Design Brief [2] which requires computation of the design curvature ductility faotor at plastic hinges, and specifies the amount of confining steel required [3] on the basis of the maximum calculated concrete compression strain, found from the curvature ductility. This approach is developed in detail in the workshop paper by Chapman [4].

It is necessary in assessing the significance of the experimental results for available ductility of bridge piers presented in the next section to have comparative theoretical values for required ductility. In discussions below on the significance of the variables listed above on ductility demand, the earthquake characteristics and structural natural period have not been isolated as independent variables because of the obvious interaction between them.

Design Force Level

It is conventional wisdom, on the basis of the equal displacement principle, to assume that the ductility demand will be in inverse proportion to the level of design force adopted. This assumption, explicit in design methods like the CALTRANS approach [5], which reduce the elastic response coefficient by a ductility factor Z, is implicit in design codes, for example the MWD Highway Bridge Design Brief [2] which specify ductile response force levels.

Table 1, extracted from data presented by Gulkan and Sozen [6] for two earthquake records indicates that this is a reasonable assumption for long period structures ($T \geq 1.0$ sec) but may be significantly in error for short period structures. The analyses of Gulkan and Sozen were carried out on simple oscillators with bilinear hysteretic behaviour, and a post-yield stiffness equal to 5% of the initial slope. The anomalous behaviour and high

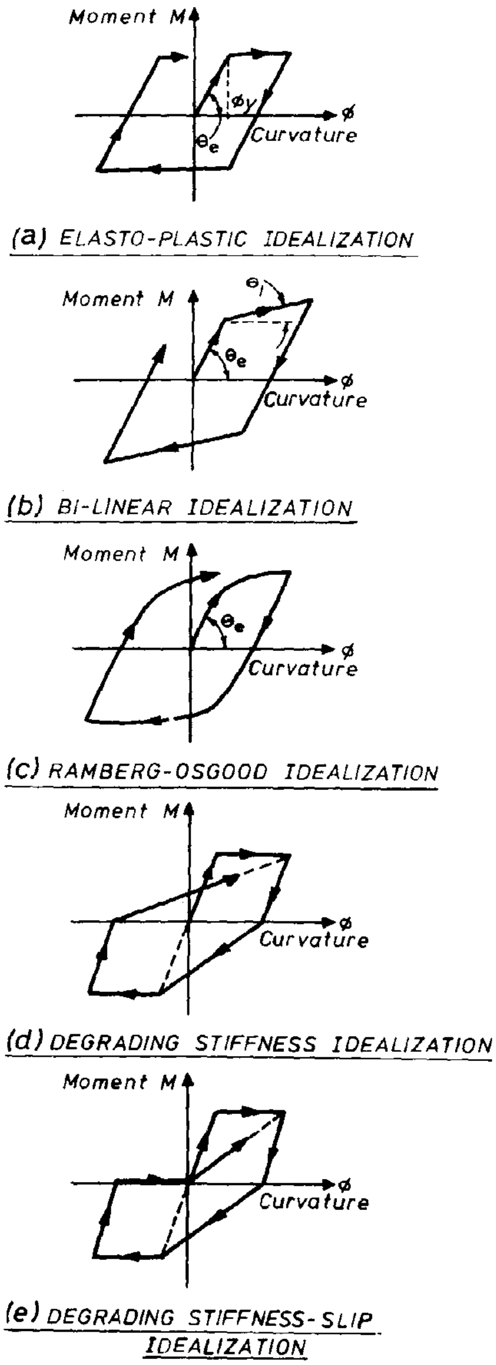


FIG. 1 MOMENT–CURVATURE HYSTERESIS LOOPS FOR REINFORCED CONCRETE

TABLE 1 - INFLUENCE OF BASE SHEAR COEFFICIENT ON STRUCTURAL DUCTILITY FACTOR(6)

Initial Period (sec)	Base Shear Coefficient*	Displacement Ductility Factor μ El Centro 1940 N-S	Managua 1972 E-W
0.15	0.16	28.5	28.8
	0.32	4.1	13.0
	0.48	3.3	5.2
0.50	0.16	8.0	5.6
	0.32	2.9	2.8
	0.48	2.0	2.0
1.0	0.08	4.9	5.6
	0.16	2.5	3.0
	0.24	1.5	1.3
2.0	0.04	4.6	4.5
	0.08	2.0	1.9
	0.12	1.5	1.4

* Design horizontal strength as a fraction of g .

ductility demand apparent in Table 1 for short period structures demonstrate the inadequacies of the equal displacement assumption for short period oscillators. Similar results are reported by Munro [7,8], who investigated the influence of a number of variables on the ductility demand of a circular bridge pier.

Hysteretic Behaviour

A number of common hysteresis loops utilized in inelastic time-history analyses of reinforced concrete structures are shown in Fig. 1. The Ramberg Osgood curve is widely accepted as giving the best representation of the hysteretic behaviour of mild steel, and also of well confined reinforced concrete sections where shear degradation of stiffness is not significant. The degrading stiffness models [9] shown in Fig. 1 are considered to give good representation of members with considerable shear stiffness degradation and shear slip. However, the elastoplastic loop (Fig. 1a), and the bilinear loop (Fig. 1b) with variable post-yield stiffness are analytically more simple, and have in consequence been used more frequently than the others in parameter studies.

Many researchers have investigated the influence of hysteresis type on structural ductility demand of simple oscillators in general, or bridge piers in particular. Results of two studies [10,11] are presented in Table 2, for

TABLE 2 - INFLUENCE OF HYSTERESIS TYPE ON DUCTILITY DEMAND (EL CENTRO N-S 1940)

Nat. Period (sec)	.167	0.262	.369	.462	.30	.30	.60	.60	.90	.90	1.5	1.5	2.1	2.1
Elastic Damping (% crit)	5	5	5	5	2	5	2	5	2	5	2	5	2	5
Base Shear Coefficient*	.216	.216	.216	.214	.168	.168	.151	.151	.118	.118	.084	.084	.084	.084
Reference :														
HYSTERESIS LOOP	MAXIMUM DISPLACEMENT DUCTILITY FACTOR μ													
Elasto-Plastic	2.86	1.63	4.67	2.62	7.28	7.88	5.15	4.36	5.38	4.71	2.13	2.19	2.88	2.71
Bilinear R = .01	2.59			2.48										
Bilinear R = .05	2.50		4.83	2.36										
Bilinear R = .10	2.46	2.05		2.22										
Ramberg Osgood r = 5	2.52			2.36										
Ramberg Osgood r = 10	2.71			2.61										
Degrading Stiffness					15.47	8.96	6.30	5.11	5.07	4.49	2.53	2.35	2.47	2.23
Degrading Stiffness/slip					16.73	11.85	6.64	5.33	4.94	4.50	3.06	2.79	2.49	2.12

* Design horizontal strength as a fraction of g .

response to the El Centro NS 1940 record. It will be seen that elasto-plastic, bilinear, and Ramberg Osgood loops all give very similar results. Significant difference between elasto-plastic and degrading stiffness response is only apparent for short period structures. It thus seems reasonable to adopt the simple bilinear or elasto-plastic characteristics for all except short period structures where shear degradation is expected. Fig. 2 shows

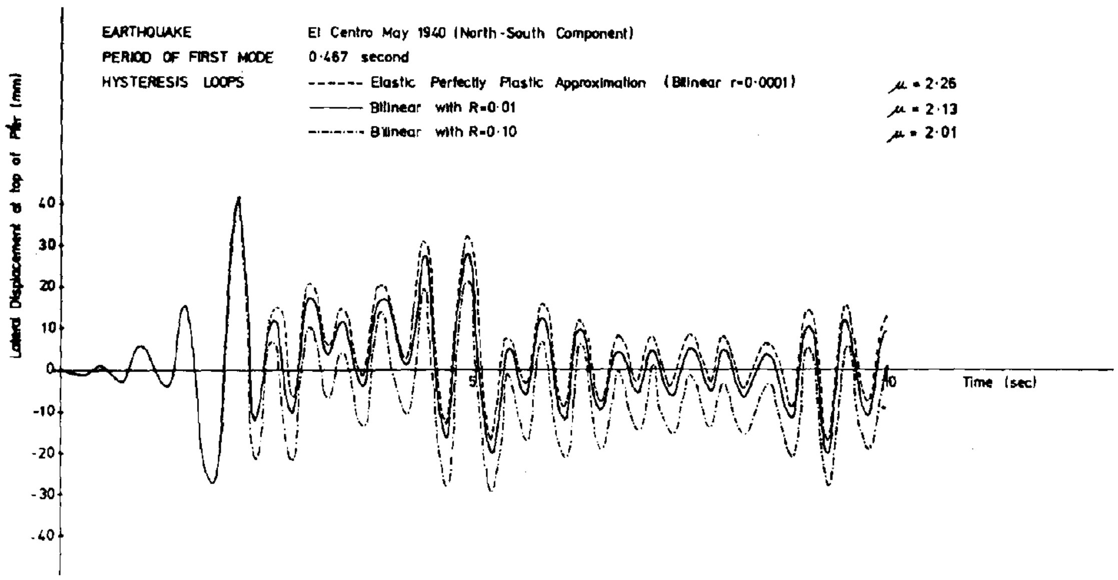


FIG. 2 EFFECT OF HYSTERESIS TYPE ON RESPONSE OF A BRIDGE PIER

a typical time-history response of a bridge pier with different assumed hysteresis types. It will be seen that the greatest influence of hysteresis type occurs at low amplitudes, and the effect on the initial peak displacements is small.

Equivalent Viscous Damping

Choice of the value of equivalent viscous damping, as a measure of elastic damping (as distinct from inelastic hysteretic damping) is a subject of some controversy, resulting mainly from the difficulty in obtaining dynamic measurements at high force levels from actual structures. Values between 2% and 10% critical damping have commonly been used in analyses of bridge systems. However, shake table tests [12] have more recently enabled estimates of damping at levels of load just below that required to induce yield to be made, and indicate that a value of about 5% critical is appropriate.

Table 2 includes comparative results for 2% and 5% critical damping using elasto-plastic and degrading stiffness characteristics. As might be expected, hysteretic damping dominates the response for the elasto-plastic analyses, and the influence of elastic damping is small and inconsistent. Because of the lower energy absorption inherent in the degrading stiffness, and degrading stiffness/slip models, the effect is more significant, particularly for short period structures.

Influence of Foundation and Bearing Compliance

Additional flexibility resulting from foundation compliance or bearing deformation can radically alter the relationship between overall displacement ductility and local curvature ductility at the plastic hinges. Consider the situation represented in Fig. 3. The pier in Fig. 3(a) has a rigid foundation and monolithic pier/superstructure construction. Assuming for simplicity an elasto-plastic yield characteristic, the plastic rotation, considered to be concentrated at the centre of the plastic hinge at the base of the pier, will be

$$\theta_{p_1} = \frac{(\mu - 1)\,\Delta_s}{(L - \frac{L_p}{2})} \qquad (1)$$

where $\Delta_s = \Delta_y$ is the displacement of the centre of mass at yield resulting from shear and flexural deformation of the pier, L_p is the plastic hinge length and μ is the displacement ductility factor.

The average plastic curvature over the hinge length will be

$$\phi_{p_1} = \frac{(\mu - 1)\,\Delta_s}{L_p(L - \frac{L_p}{2})} \qquad (2)$$

Now assuming a triangular bending moment diagram in the pier due to horizontal seismic load concentrated at the centre of mass of the pier superstructure, the displacement due to pier flexure is

$$\Delta_s = \frac{M_y L^2}{3EI}$$

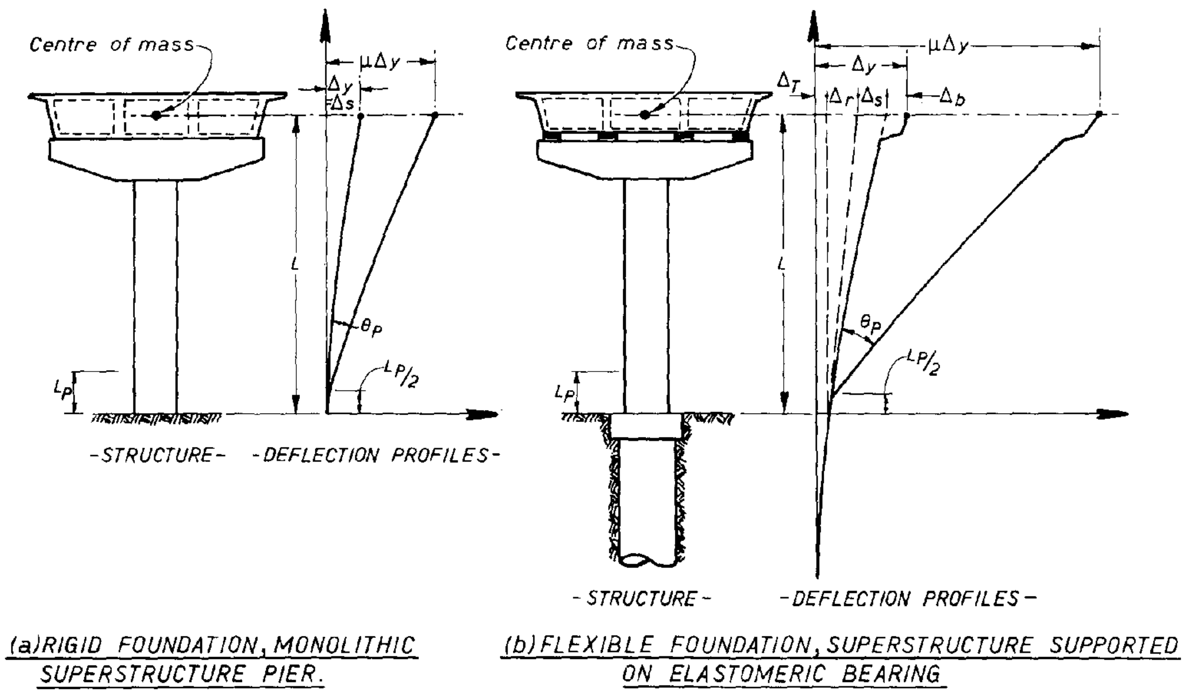


FIG. 3 INFLUENCE OF ELASTIC DEFLECTION COMPONENTS ON PLASTIC ROTATION

where M_y is the yield moment. Therefore the yield curvature is

$$\phi_y = \frac{M_y}{EI} = \frac{3\Delta_s}{L^2}$$

The curvature ductility factor is defined as

$$\left(\frac{\phi_u}{\phi_y}\right)_1 = \frac{\phi_y + \phi_p}{\phi_y}$$

$$= 1 + \frac{(\mu - 1)\,\Delta_s}{L_p(L - \frac{L_p}{2})} \cdot \frac{L^2}{3\Delta_s}$$

$$= 1 + \frac{(\mu - 1)}{1.5\,\frac{L_p}{L}(2 - \frac{L_p}{L})} \qquad (3)$$

The pier in Fig. 3b is founded on a single cylinder pile, and the superstructure transmits vertical and shear loads to the hammerhead and pier through elastomeric bearing pads. The yield displacement is thus increased by translation (Δ_T) and rotation (Δ_r) of the pile cap, and by shear deformation of the bearing (Δ_b). Thus

$$\Delta_y = \Delta_T + \Delta_r + \Delta_b + \Delta_s = C.\Delta_s \qquad (4)$$

where C is a coefficient representing the increase in the elastic flexibility of the system. A capacity design procedure is used to ensure formation of the plastic hinge at the pier base rather than in the foundation cylinder. Assuming a bilinear moment-curvature relationship for the hinge as before, all additional deformation after yield must result from plastic rotation of the pier hinge. For a displacement ductility of μ, then

$$\theta_{p_2} = \frac{(\mu - 1)\, C\, \Delta_s}{(L - \frac{L_p}{2})} = C.\theta_{p_1} \tag{5}$$

and $$\phi_{p_2} = C.\phi_{p_1} .$$

The yield curvature at the plastic hinge remains unchanged, and therefore the curvature ductility factor is

$$\left(\frac{\phi_u}{\phi_y}\right)_2 = 1 + \frac{C(\mu - 1)}{1.5\frac{L_p}{L}(2 - \frac{L_p}{L})} .$$

Hence if the displacement ductility factor μ is the same for the rigid base case and the flexible base case, then

$$\left(\frac{\phi_u}{\phi_y}\right)_2 = 1 + C\left(\left(\frac{\phi_u}{\phi_y}\right)_1 - 1\right) \tag{6}$$

Since for stiff piers the value of C may be as high as 5 or more, the effect on curvature ductility demand may be substantial. For the above argument to be valid, it is of course necessary that the two systems be designed for the same value of μ. This will generally imply a lower design base shear coefficient for the more flexible pier of Fig. 3b as a result of the increase in elastic period.

Ng Kit Heng [13,8] investigated the interaction between additional flexibility, earthquake characteristics and ductility demand for four bridge models, using a simple representation of the effects of foundation flexibility. All models were based on an octagonal stem pier of 1.5 m (4.92 ft) with circularly distributed flexural reinforcement. Pier A was a double stem pier, while piers B, C, and D were single stem piers.

In each case seismic loading acting at right angles to the longitudinal axis of the bridge was analysed. A lumped weight of 4000 kN (900 Kips) was associated with the horizontal degree of freedom and the weight of the stem was taken as 45 kN/m (3.08 Kip/ft). Pier heights were chosen to give natural first mode periods of vibration of 0.2, 0.5, 1.0 and 2.0 sec. for the four models respectively, assuming rigid base conditions and a cracked-section moment of inertia of $I = 0.108\,m^4$ ($12.50\ ft^4$). Yield moment capacities of the plastic hinges were calculated from the base shear coefficient requirements of ref. [2] for Zone A. Salient information on the models is included in Table 3 which summarizes results for this study.

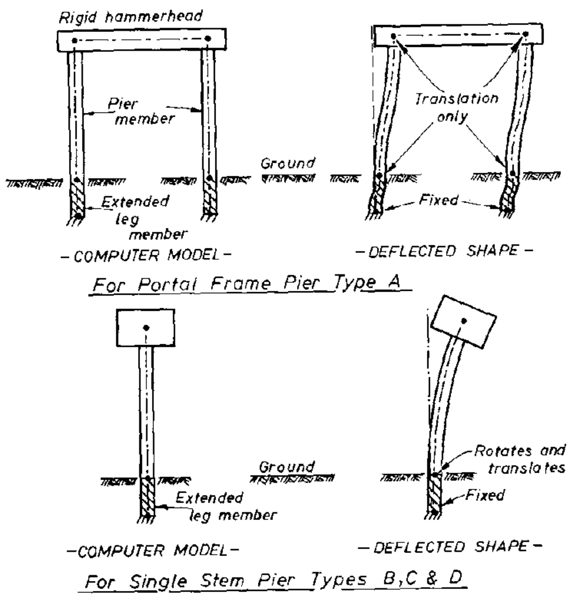


FIG. 4 'EXTENDED LEG' REPRESENTATION OF FOUNDATION FLEXIBILITY

Rather than attempting to accurately model exact soil behaviour, the investigation concentrated on the general consequences of foundation flexibility. The simple computer model adopted to take into account foundation flexibility involved the concept of an 'extended leg' beneath the pier, as shown in Fig. 4, where stiffness was the same as that of the piers, but where strength was sufficiently high to ensure plastic hinge formation at the base of the pier. Increasing the length of the leg increased the flexibility coefficient C defined in eqn. 4.

Analyses were carried out using an inelastic time-history analysis program developed by Sharpe [14,8]. Values for C of 1,2,3 and 4 were considered. Note that C = 1 corresponds to the rigid foundation case. In each case the yield moment of the base hinge was kept at the value appropriate to the rigid base natural period. That is, no allowance was made for change in the base shear coefficient with increase in natural period resulting from foundation flexibility

The resulting values of structural and curvature ductility demand found from the dynamic analyses are listed in Table 3, which includes for comparison calculated requirements based on eqns. 3 and 6 and the displacement ductility specified by ref. 2 for the chosen values of base shear coefficient and natural period. The earthquake records investigated included the Jennings et al. [15] artificial A1 and B1 earthquakes and an early digitization of the Bucharest 1977 N-S record. This latter digitization was very coarse, with a Nyquist frequency of about 2.5 Hz, and results for the stiff type A pier model must be viewed with caution.

For pier type A the curvature ductility factor is shown to increase with increasing foundation flexibility for all four earthquake records run, at a faster rate than predicted by equation 6. This trend is probably due to the fact that the increase in period associated with foundation flexibility moves the type A pier into a region of higher acceleration response on the response spectra for the earthquakes. For pier types B, C and D the curvature ductility demand is comparatively insensitive to increasing foundation flexibility. In many cases the curvature ductility demand shows a decrease rather than the increase predicted by eqn. 6. The exception is the response of Pier B to the Bucharest record, which has a maximum in the acceleration response spectra at about 1.0 - 1.5 sec. It appears that where the increase in foundation flexibility shifts the natural period into a region of reduced acceleration

TABLE 3 - DUCTILITY FACTOR DEMAND OF PIERS WITH FLEXIBLE FOUNDATIONS

Pier Type	Period, Earthquake Record	Maximum Curvature Ductility Factor, ϕ_u/ϕ_y				Maximum Overall Displacement Ductility Factor, μ			
		C = 1	2	3	4	C = 1	2	3	4
A	Natural Period, sec.	0.20	0.30	0.38	0.44	0.20	0.30	0.38	0.44
Base	El Centro N-S 1940	31.40	57.81	70.51	81.05	11.97	11.99	10.28	8.5
Shear	Taft B 69 W 1952	17.09	41.60			6.45	8.28		
Coeff.	Bucharest N-S 1977	12.67	15.69	31.62		5.17	3.93	5.41	
= 0.216g	Artificial B1	49.92	121.74			18.74	22.52		
	Calculated requirements* $\frac{L_p}{L} = 0.116$	16.25	31.50	46.76	62.01	6	6	6	6
B	Natural Period, sec.	0.50	0.67	0.86	0.98	0.50	0.67	0.86	0.98
Base	El Centro N-S 1940	8.86	8.75	6.04	9.47	3.57	2.29	1.70	1.82
Shear	Taft N 69 W 1952	2.68	2.25			1.51	1.08		
Coeff.	Bucharest N-S 1977	6.44	11.59	20.00		2.73	2.78	3.57	
= 0.209g	Artificial B1	16.57	8.58			6.10	2.14		
	Calculated requirements* $\frac{L_p}{L} = 0.105$	17.75	29.7	38.8	46.9	6	5.28	4.51	4.11
C	Natural Period, sec.	1.00	1.36	1.72	1.97	1.00	1.36	1.72	1.97
Base	El Centro N-S 1940	7.47	3.48	1.95	3.45	3.57	1.49	1.13	1.25
Shear	Taft N 69 W 1952	1.02	1.44			1.02	1.05		
Coeff.	Bucharest N-S 1977	18.76	21.19	20.40		8.19	5.32	4.1	
= 0.137g	Artificial B1	8.23	9.01			3.93	2.66		
	Artificial A1	15.57	11.87			6.61	3.31		
	Calculated requirements* $\frac{L_p}{L} = 0.128$	14.91	22.3	33.9	44.8	6	4.73	4.73	4.73
D	Natural Period, sec.	2.00	2.89	3.55	4.11	2.00	2.89	3.55	4.11
Base	El Centro N-S 1940	4.03	2.29	3.75	Elastic	2.25	1.44	1.36	Elastic
Shear	Taft N 69 W 1952	Elastic	Elastic			Elastic	Elastic		
Coeff	Bucharest N-S 1977	7.98	6.05	5.84		3.78	2.05	1.70	
= 0.108g	Artificial B1	3.41	5.48			1.93	1.94		
	Calculated requirements* $\frac{L_p}{L} = 0.125$	15.22	29.44	43.67	57.8	6	6	6	6

* μ from reference (2), $\frac{\phi_u}{\phi_y}$ from μ and equations 3 and 6

response, the requirements of eqn. 6, which assumes an equal displacement ductility factor demand for rigid base and flexible base systems, are excessively severe, and analysis based on a constant plastic displacement (and hence constant curvature ductility factor) might be more reasonable.

Ng Kit Heng [13] also investigated the significance of more accurate foundation modelling by considering the actual bridge pier shown in Fig. 5. Soil behaviour was modelled by a series of inelastic springs whose properties reflected results from measured bore hole data. It is significant that calculations for the pier/pile system indicated a value for the flexibility coefficient of $C = 8.9$. Although dynamic analyses predicted soil yield over the top 3 m of soil, it was found that soil damping, based on elasto-plastic

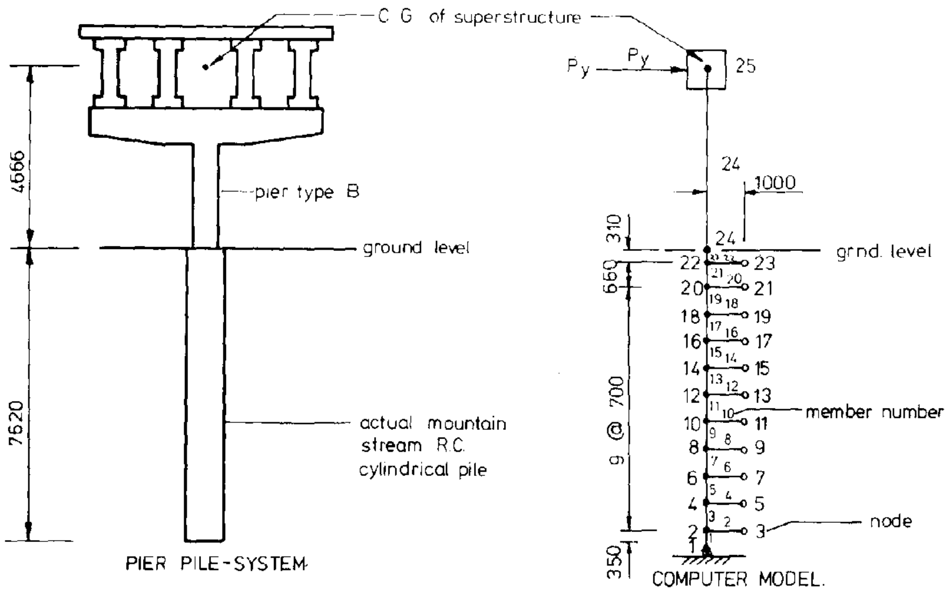


FIG. 5 'REALISTIC' SIMULATION OF MOUNTAIN STREAM BRIDGE FOUNDATION

response of the soil springs was equivalent only to an increase in structural damping from 5% to 9%. However, it should be noted that the elasto-plastic soil model overestimates the area within the hysteresis loop by a factor of approximately two. Hence in practice the damping including the effect of soil yield would have been about 7%. Thus it would appear unwise to rely on a significant reduction in response due to additional damping from soil yield.

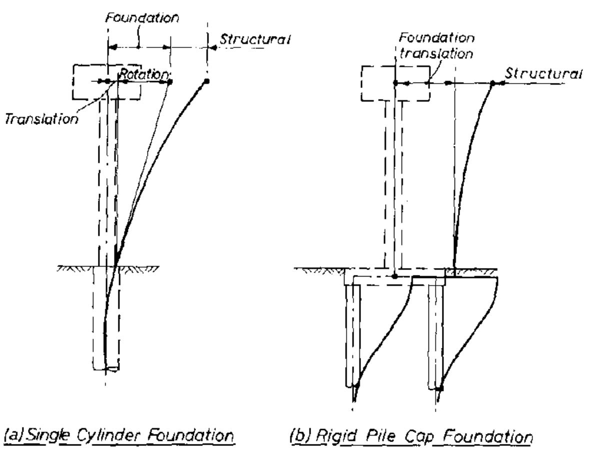


FIG. 6 INFLUENCE OF PILE CONFIGURATION ON FOUNDATION COMPLIANCE

It is probable, however, that the effects of soil damping would have been more significant if the foundation compliance had taken the form of horizontal translation of the pile cap. As Fig. 6a shows, the single cylinder foundation results in a deflection profile where the soil displacement at ground level is small in comparison with the displacement of the centre of mass, and the foundation compliance effect results almost entirely from rotation of the pile cap. Under the circumstances soil damping is unlikely to be effective in reducing response. In Fig. 6b, in which the same centre of mass displacement from foundation compliance is now provided solely by translation of the pile cap, ground deformation will be greatly increased and soil damping may become a significant factor in reducing response. This aspect requires further investigation.

THEORETICAL INVESTIGATIONS OF DUCTILITY CAPACITY

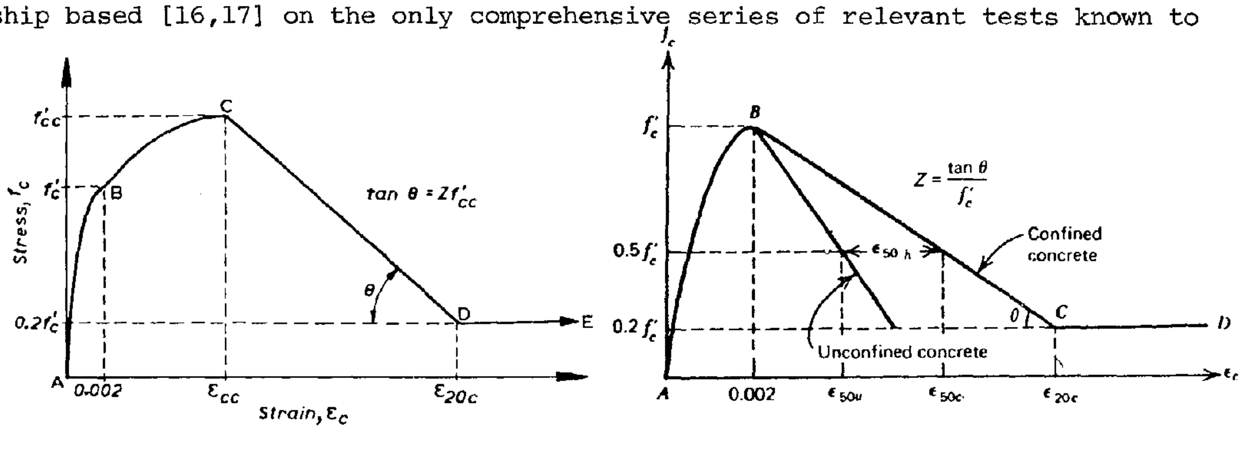
To the authors' knowledge, there is no accepted statement of the conditions representing the limits of available ductility, and hence no generally accepted definition of the available ductility capacity of a given plastic hinge. The N.Z. Ministry of Works and Development design approach [3] adopts a limiting compression strain, dependent principally on axial load level and confining steel content, as the limit condition, but as will be shown, this appears on experimental evidence to be unduly conservative. The authors believe a more rational approach would be to define the available ductility as that existing when the section moment capacity has dropped to (say) 15% below the maximum computed level.

It is clear that, regardless of the definition adopted, the available curvature ductility of reinforced concrete pier sections is very dependent on the stress-strain characteristics of the concrete and the steel, and that some degree of confinement will be necessary to develop the required ductility.

Confined Stress-Strain Characteristics of Concrete

Available stress-strain curves for confined concrete are largely based on the results of monotonically tested axially loaded specimens, and until confirmed by more realistic test programmes, some doubts must remain as to their applicability to concrete subjected to dynamic load reversals and high strain gradients. Figs. 7a and 7b show curves for concrete confined by circular spirals, and rectangular hoops respectively, and are sufficiently different to warrant separate discussion.

Concrete confined by circular spirals: There is a scarcity of experimental data giving complete stress-strain curves for concrete confined by circular steel spirals. The curve in Fig. 7a is an empirical stress-strain relationship based [16,17] on the only comprehensive series of relevant tests known to



(a) Circular Spirals

(b) Rectangular Hoops

FIG. 7 IDEALISED STRESS-STRAIN CURVES FOR CONFINED CONCRETE the authors; that conducted by Iyengar et al. [18] who tested 149 cylindrical reinforced concrete specimens of rather small size (200 mm (8 in) x 100 mm (4 in) dia., or 300 mm (12 in) x 150 mm (6 in) dia.). These tests indicated very large increases in both the strength and the deformation capacity of the concrete, resulting from the radial pressure exerted by the restraining effect of the spiral. The characteristics of the curve are described below.

Region AB: $0 \leq \varepsilon_c \leq 0.002$. A second degree parabola is assumed, following the curve for unconfined concrete.

$$f_c = f'_c \left[\frac{2\varepsilon_c}{0.002} - \left[\frac{\varepsilon_c}{0.002} \right]^2 \right] \tag{7}$$

Region BC: $.002 \leq \varepsilon_c \leq \varepsilon_{cc}$. The region of increasing strength due to confinement is represented by a further second degree parabola

$$f_c = f'_c + (f'_{cc} - f'_c) \left[\frac{2(\varepsilon_c - 0.002)}{\varepsilon_{cc} - 0.002} - \left[\frac{\varepsilon_c - .002}{\varepsilon_{cc} - .002} \right]^2 \right] \tag{8}$$

where
$$f'_{cc} = f'_c \ (1 + 2.3 \ (\rho_s - \bar{\rho}_s) \ \frac{f_y}{f'_c}) \tag{9}$$

$$\varepsilon_{cc} = .002 \ (1 + 23 \ (\rho_s - \bar{\rho}_s) \ \frac{f_y}{f'_c}) \tag{10}$$

ρ_s is the ratio of volume of spiral steel to volume of concrete core, and $\bar{\rho}_s$ is the value of ρ_s when the spiral pitch equals the spiral diameter.

Region CD: $\varepsilon_{cc} \leq \varepsilon_c \leq \varepsilon_{20c}$. A linear falling branch is assumed.

$$f_c = f'_{cc} \ [\ 1 - z \ (\varepsilon_c - \varepsilon_{cc})\ ] \tag{11}$$

where
$$z = \frac{107}{f'_{cc}} \left(\frac{f'_c}{\rho_s f_y} \right)^{1.13} \tag{12}$$

was found from a statistical analysis of the test data of Iyengar, et al. [18], for spiral bars with f_y = 319 MPa. (46,250 psi).

Region DE: $\varepsilon_c \geq \varepsilon_{20c}$. It is assumed that concrete can sustain a compressive stress of $0.2f'_c$ indefinitely.

Fig. 8 shows stress-strain curves predicted by eqns. 7 to 12 for a 508 mm (20 in) dia. bridge column confined by different amounts of confining steel of 19.1 mm (3/4 in) dia.

Concrete confined by rectangular hoops: The curve shown in Fig. 7b was proposed by Kent and Park [19] on the basis of analysis of existing experimental data.

Region AB: $0 \leq \varepsilon_c \leq 0.002$. Equation 7 applies.

Region BC: $0.002 \leq \varepsilon_c \leq \varepsilon_{20c}$.

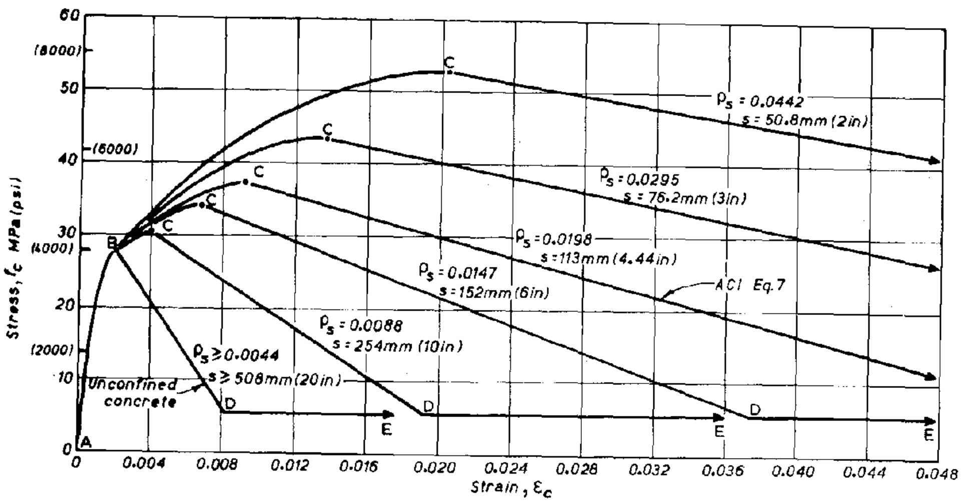


FIG. 8 IDEALISED STRESS-STRAIN CURVES GIVEN BY EQUATIONS 7 to 12 FOR f'_c = 27.6MPa, f_y = 276MPa (508 mm dia column)[17]

$$f_c = f'_c\,[1 - z\,(\varepsilon_c - 0.002)] \tag{13}$$

where

$$z = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - 0.002} \tag{14}$$

$$\varepsilon_{50u} = \frac{3 + 0.29 f'_c}{145 f'_c - 1000} \tag{15}$$

$$\varepsilon_{50h} = \frac{3}{4}\,\rho_s \sqrt{\frac{b''}{s_h}} \tag{16}$$

Where f'_c is in MPa, ρ_s is the ratio of volume of hoops to volume of concrete core, b'' is the width of the confined core and s_h is the spacing of hoops.

Region CD: $\varepsilon_c > \varepsilon_{20c}$. It is again assumed that concrete can sustain a compressive stress of $0.2f'_c$ indefinitely.

Note that as a consequence of the less efficient confinement of rectangular hoops compared with circular spirals, it is assumed that there is no strength increase above the cylinder strength, as was demonstrated by Roy and Sozen by testing prisms confined by square hoops placed at the section perimeter [30]. This assumption for columns with overlapping rectangular hoops is clearly conservative, as evidenced by Fig. 9 which compares results from a recent axial load test on a 2.6 m (8.52 ft) long by 390 x 395 (15.4 x 15.6 in) reinforced column with the prediction of eqns. 13 to 16. The experimental concrete stress plotted in Fig. 9 has been found by subtracting the longitudinal steel contribution from the total load, and dividing by the net concrete area.

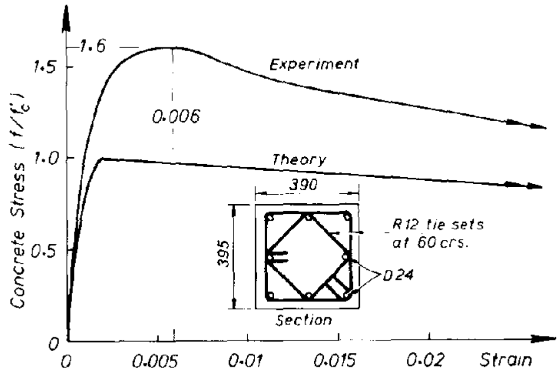


FIG. 9 THEORY/EXP. COMPARISON FOR AXIALLY LOADED COLUMN (f'_c = 29.8MPa, f_y = 300MPa ρ_s = .0368)

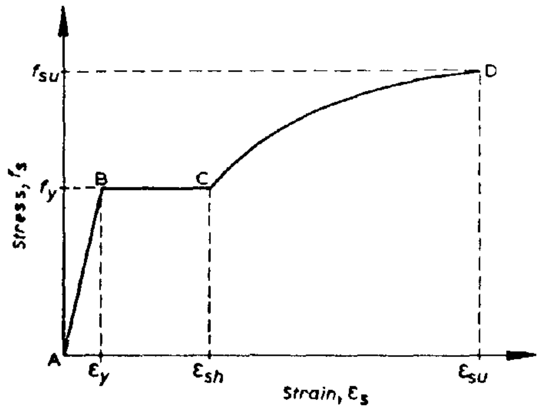


FIG. 10 IDEALISED STRESS-STRAIN CURVE FOR STEEL

Stress Strain Characteristics of Reinforcing Steel

An idealized stress-strain relationship for reinforcing steel is shown in Fig. 10. The behaviour is characterized by 3 regions: A-B elastic, B-C yielding, C-D strain-hardening. Analyses predicting moment-curvature relationships need to model all three regions, including the strain hardening portion. Of particular importance is the length of the yield plateau B-C, which varies widely with strength and chemical composition of the steel. Under cyclic stress reversals the monotonic stress-strain curve has been shown [16] to under-predict stress levels at low strains. This behaviour can best be modelled using Ramberg-Osgood functions based on experimental data.

Codified Confinement Requirements

Because of space limitations, and since circular bridge piers are more common than rectangular or polygonal shapes as a result of the lack of dependence of capacity on angle of seismic attack, discussion below will largely be limited to circular bridge piers.

ACI spiral reinforcement formula: For spiral columns the ACI Code [20] requires that the volume of circular spiral steel per unit volume of concrete core should be at least equal to

$$\rho_s = 0.45 \frac{f'_c}{f_y} \left[\frac{A_g}{A_c} - 1 \right] \qquad (17)$$

where A_g is the gross area of the column section and A_c is the area of the core of the column within the spiral. This requirement is intended to ensure that the axial load carried by the column after spalling of the concrete cover will at least equal the load carried before spalling. Eq. 17 can be derived by calculating the content of spiral reinforcement necessary to enhance the strength of the core concrete sufficiently to compensate for the loss of strength of the concrete cover.

The ACI Code [20] recommends in its special provisions for the seismic design of ductile frames that where the maximum design load of the column, P_e, is greater than 0.4 of the balanced failure load P_b (i.e. $P_e > 0.4P_b$, which in many cases can be taken conservatively as $P_e > 0.1f'_c A_g$) the core of circular columns over the end regions should be confined by spirals so that the volumetric ratio of the spiral steel ρ_s is at least that given by Eq. 17, but is not less than

$$\rho_s = 0.12 \frac{f'_c}{f_y} \tag{18}$$

This additional requirement is not included in the recommendations of the report of ACI Committee 443 [21]. It should be noted that this ACI requirement will not necessarily result in sufficient curvature ductility for the column to survive a severe earthquake, because it is based on the philosophy of preserving the strength of axially loaded columns after spalling of the cover concrete rather than maintaining the ultimate deformability of eccentrically loaded columns. Since the ductility of the concrete is increased by the presence of confining steel, the presence of the spiral will result in improved column behaviour, but can be regarded as only a crude guide to the amount of steel actually required for ductile column behaviour.

Ministry of Works and Development, N.Z. requirements: MWD requirements [2,3] are based on providing a displacement ductility factor for bridges of six. Geometric relationships such as eqn. 3 are used to compute the required section curvature, using Baker's estimate [22] of plastic hinge length, and hence the ultimate required compressive strain ε_{cu}. The stress strain curve adopted for concrete confined by rectangular hoops is that proposed in eqns. 13-16, but for concrete confined by circular spirals, a more conservative relationship than that proposed in eqns. 7-12 is adopted, as no allowance is made for increase in strength resulting from confinement.

Confining steel requirements are found from eqns. 19 and 20 which relate ε_{cu} to the volumetric confining steel content ρ_s and the ratio of effective depth d to neutral axis depth c:

For rectangular hoops: $$\varepsilon_{cu} = .0021\ (1 + 150\rho_s + (0.7 - 10\rho_s)\ \frac{d}{c}) \tag{19}$$

For circular spirals: $$\varepsilon_{cu} = .0033\ (0.29 + 150\rho_s + (0.7 - 10\rho_s)\ \frac{d}{c}) \tag{20}$$

These equations are based on a modified (less conservative) form of Baker's work [22].

Using typical values of $\rho_s = 0.015$ and $\frac{d}{c} = 2.5$, eqns. 19 and 20 result in $\varepsilon_{cu} = 0.010$ and 0.013 respectively. In comparison with the curves of Fig. 8 these ultimate compressive strains are very conservative.

Minimum spiral reinforcement contents are based on the ACI minimum, with a linear increase from $\rho_s = 0.06f'_c/f_y$ at zero axial load to the full ACI value of $\rho_s = 0.12f'_c/f_y$ at $P_e \leqq 0.1f'_c A_g$.

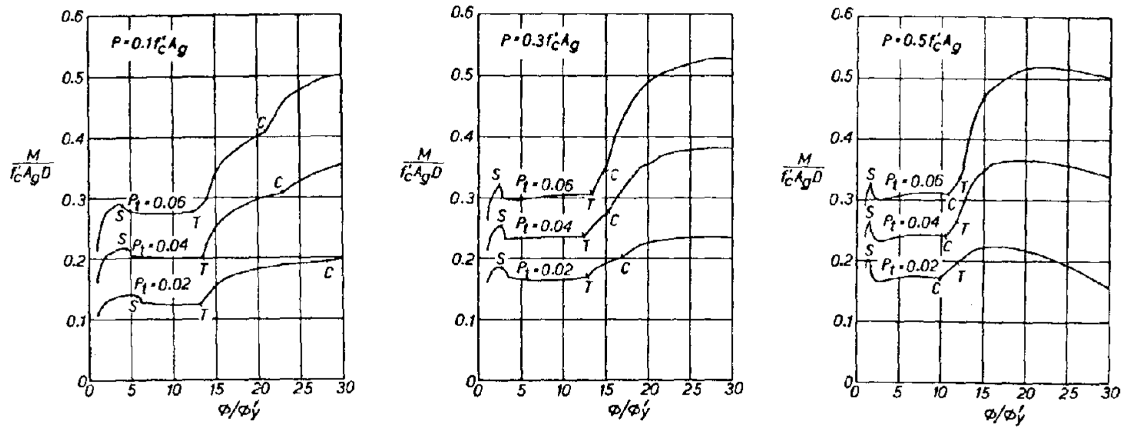
New Zealand Draft Concrete Design Code: Confining requirements in the N.Z. draft Concrete Design Code [23] are based on the SEAOC and ACI [20] requirements modified to take axial load level into account. Thus for spiral columns, the volumetric ratio of confining steel is required to be not less than

$$\rho_s = 0.45 \frac{f'_c}{f_y} \left[\frac{A_g}{A_c} - 1 \right] (.375 + 1.25 \frac{P_e}{f'_c A_g}) \qquad (21)$$

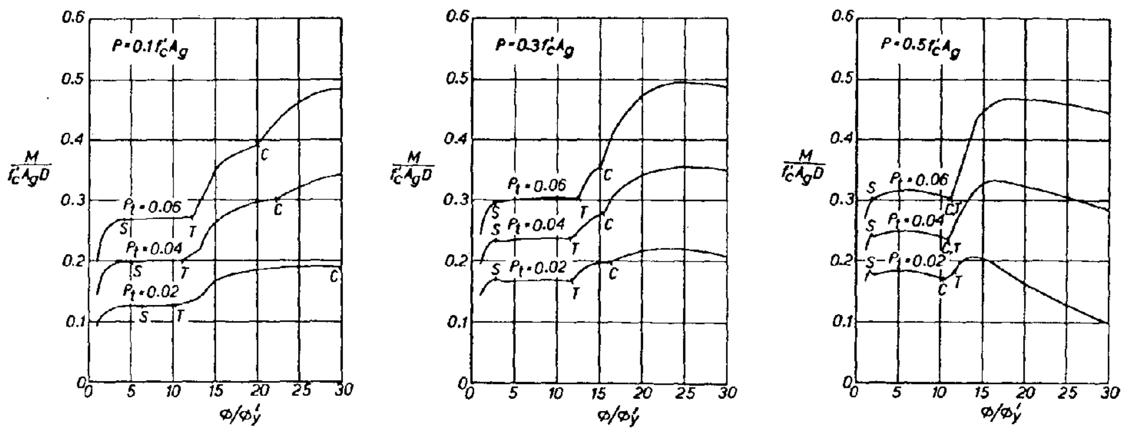
or

$$\rho_s = 0.12 \frac{f'_c}{f_y} (.375 + 1.25 \frac{P_e}{f'_c A_g}) \qquad (22)$$

whichever is greater, where P_e shall not be taken as less than $0.1 f'_c A_g$. Equations (21) and (22) require the same amount as the ACI eqns. for $P_\varepsilon = 0.5 f'_c A_g$, but only half as much at $P_\varepsilon = 0.1 f'_c A_g$. Similar modifications are made to ACI requirements for hoop steel in rectangular columns [23].



(a) Column Diameter = 610 mm



(b) Column Diameter = 1220 mm

FIG. 11 MOMENT CURVATURE DUCTILITY RELATIONSHIPS FOR COLUMNS CONFINED BY ACI SPIRAL (f'_c = 27.6MPa, f_y = 276MPa)

Moment-Curvature Analyses

It is of interest to examine moment-curvature characteristics of columns confined by the ACI spiral to determine the theoretical curvature ductility available. Park and Leslie [17] present results for circular bridge columns based on eqns. (7) to (12) for a variety of column sizes, and load levels and longitudinal reinforcement ratios P_t. Typical results are shown in Fig. 11 for two column diameters, assuming f'_c = 27.6 MPa (4000 psi) and f_y = 276 MPa (40,000 psi). ACI requirements resulted in confining steel contents of $\rho_s = 0.0198$ and $\rho_s = 0.0120$ for the 610 mm (24 in) and 1220 mm (48 in) dia. columns respectively.

For the smaller column size there is a significant reduction in moment at the point where the concrete cover commences to spall away. This effect is not so noticeable for the larger column, since the A_g/A_c ratio is near to unity. The onset of steel strain hardening is clearly seen in most of the diagrams and generally results in a considerable increase in moment carrying capacity. This increase is more significant for higher ratios of longitudinal steel, because a proportionally greater share of the moment is carried by the steel. In spite of assistance from strain hardening of steel the 1220 mm (48 in) dia. column has difficulty sustaining its moment capacity for high axial loads and low longitudinal steel contents.

In Fig. 11 the curvature ϕ'_y is the curvature when the tension steel furthest from the neutral axis first reaches yield strain. If the yield curvature ϕ_y is defined as the curvature at the point of intersection of the tangent to the elastic slope of the moment-curvature curve and a horizontal line at ultimate moment, then ϕ_y would be larger than ϕ'_y. Thus a value of ϕ/ϕ'_y of (say) 30 corresponds to a rather smaller value of ϕ/ϕ_y.

A design approach based on ensuring a satisfactory moment-curvature relationship would form a rational basis for detailing columns for ductility. It seems reasonable to define the ultimate curvature ϕ_u as the curvature when the moment capacity has reduced to 80-90% of the maximum moment capacity. Using this definition and the approach adopted in developing the curves of Fig. 11, the amount of confining steel required for a design level of ϕ_u/ϕ_y could be determined. Such an approach, which determines the content of confining steel necessary to ensure a particular curvature (and hence displacement) ductility factor, is more logical than that followed by the ACI Code [20] which aims to preserve the axial load strength of the column after spalling of the cover concrete, and results in a requirement based on only the A_g/A_c and f'_c/f_y ratios of the column.

On the basis of Park and Leslie's research [17] it appears that the ACI spiral may be overly conservative for low axial load levels, but unconservative for large diameter columns with high axial load and low longitudinal steel content, particularly if the steel does not exhibit significant strain-hardening. The approach adopted by the draft N.Z. Concrete Code [23] (Eqns. 21,22) in which the confining steel content increases with axial load thus appears preferable to the ACI approach.

Static Testing

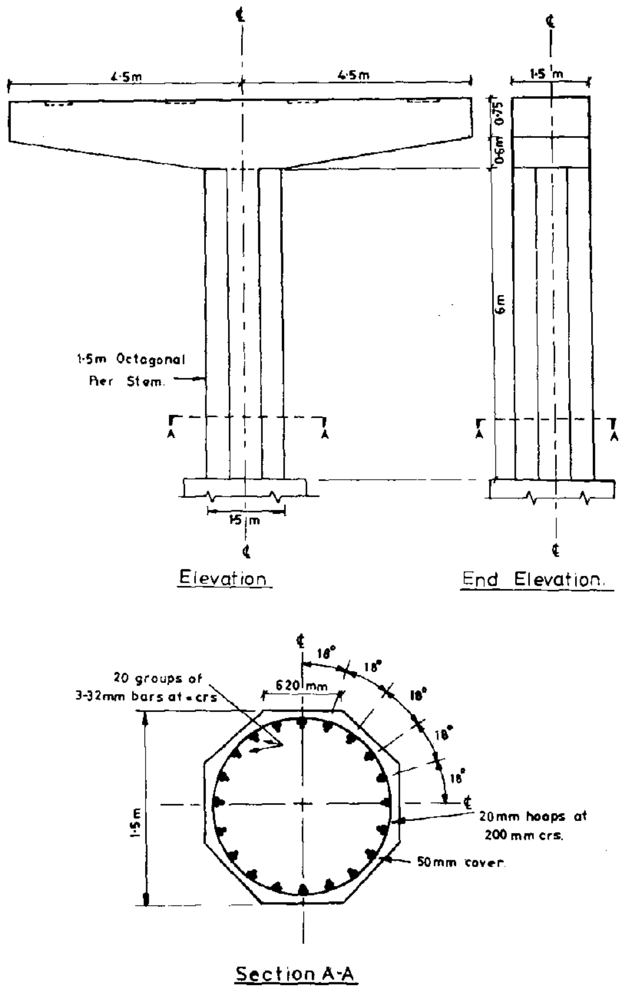


FIG. 12 PROTOTYPE PIER

Davey [24], Munro [7] and Ng [12] investigated the ductility of bridge columns with octagonal cross section by static cyclic load testing of models based on the prototype pier shown in Fig. 12. This was designed according to the MWD Highway Bridge Design Brief [2] in conjunction with the ACI Building Code [20], (the 1971 ACI Code was actually used, but the clauses referred to have not been changed in the 1977 ACI Code).

The superstructure, used for dead load calculations consisted of four MWD standard I-Beams of 20 m (65.5 ft) span. Total superstructure weight was 100 kN/m (6.85 Kips/ft). The base shear coefficient of 0.216g resulted in a pier base design ultimate moment requirement of 7050 kNm (5200 Kip ft) and a design axial load of 3430 kN (771 Kips). Using a capacity reduction factor of 0.75 the required content of longitudinal steel was found to be 2.7%, and could be made up using 20 groups of 3-32mm (1.26 in) dia. bars, as shown in Fig.12.

Six pier models, based on the prototype cross-section and longitudinal steel content were tested. Details of the models are given in Table 4.

Units 1 - 3 modelled the prototype to $^1/_3$ scale and differed only in the effective height of lateral load application expressed in Table 4 in the dimensionless form $M/(V.D)$, where M/V is the lever arm and D is the pier diameter. Longitudinal steel in the models consisted of 20 pairs of 13 mm (0.512 in) dia. deformed bars of yield strength f_y = 372 MPa (54,000 psi), which was 5% less than required for exact model/prototype similitude. Transverse spiral steel was designed in accordance with ACI 318 requirements. The design value of P_e (modelled in units 1 - 3 by applied axial load) was about $0.06\ f'_c A_g$, where A_g is the gross area of the section. With this relatively low axial load level the pier could be designed as a flexural member according to Appendix A of ACI 318 [20]. Using the ACI Code equations the concrete shear resisting mechanism was found to be capable of carrying all horizontal shear for each unit. The minimum web reinforcement allowed at

TABLE 4 - DETAILS OF BRIDGE PIER TESTS (7,13,24)

UNIT	$\frac{M}{VD}$	Nominal Axial Load ($xf'_c Ag$)	Material Strengths (MPa)+ Concrete f'_c	Vertical Steel Yield	Vertical Steel Ultimate	Hoop Steel Yield	Hoop Steel Ultimate	Volumetric Ratio of Confining Steel (%) ACI (20)	MWD (3)*	DZ3101 (23)	Actual
1	5.5	0.06	33.2	373	564	312	421	0.44	2.20	0.64	0.44
2	3.5	0.06	34.8	371	562	312	421	0.44	2.20	0.67	0.44
3	6.5	0.06	33.8	373	563	342	493	0.44	2.20	0.59	0.44
4	5.5	0.03	40.0	305	411	389	493	0.44	2.20	0.62	1.26
5	5.36	0.01	35.1	305	411	263	360	0.44	2.20	0.80	1.86
6	3.72	0.34	33.0	294	444	207	344	1.91	2.70	1.53	2.44

* Calculated for μ = 6.0, + 1 MPa = 145 psi.

the ends of the columns by the ACI code is given by

$$A_v = 0.15\ A'_s\ d/s \quad \text{or} \quad 0.15\ A_s\ d/s \tag{23}$$

whichever is larger, where A_s and A'_s are the areas of tension and compression reinforcement respectively, d is the effective depth and s is the spacing of the transverse reinforcement. Eqn. 23 required a ratio of volume of circular hoop steel to volume of concrete core of ρ_s = 0.44%, which was provided in Units 1 - 3 by 6.5 mm (0.255 in) dia. plain bar circular hoops spaced at 65 mm (2.55 in) centres in the plastic hinge regions. The hoop bars were lapped at least 50 mm (2 in) and joined by a single vee flare weld.

Unit 4 was identical to Units 1 - 3 in scale and longitudinal steel content, but the plastic hinge zone was confined in accordance with minimum MWD requirements, that the volumetric ratio of circular hoop steel should not be less than $\rho_s = 0.12\ f'_c/f_y$, and that hoop spacing should not exceed 100 mm (4 in). This resulted in ρ_s = 1.26%, about 3 times the volumetric ratio required by the minimum shear requirements of ACI 318, and was provided in Unit 4 by 8.0 mm (0.315 in) dia. circular hoops at 34 mm (1.34 in) centres. It should be noted that this is still less than required by ref. [3] to ensure a structural ductility factor of 6.

Unit 5 modelled the prototype section and longitudinal steel content to 1/6 scale, and was in fact a ½ scale model of unit 4. This unit was tested dynamically on a shake-table [12] and subsequently subjected to further static cyclic load testing [13]. Longitudinal steel consisted of ten 13 mm (0.512 in) dia. deformed bars, and the plastic hinge zone was confined by a spiral of 4.4 mm (0.173 in) dia. plain round steel of 14 mm (0.551 in) pitch, welded at the laps.

Units 1 - 5 had the low axial load level of $0.06\ f'_c A_g$, or less. For a tall slender portal frame type of pier structure under combined seismic and gravity loading the axial load in a pier may reach a value of $0.5\ f'_c A_g$ or higher. A high axial load would require greater concrete confinement. Unit 6 was tested with an axial load level of $0.33\ f'_c A_g$ to check spiral reinforcement requirements. Like Unit 5, Unit 6 was a 1/6th scale model. Axial load

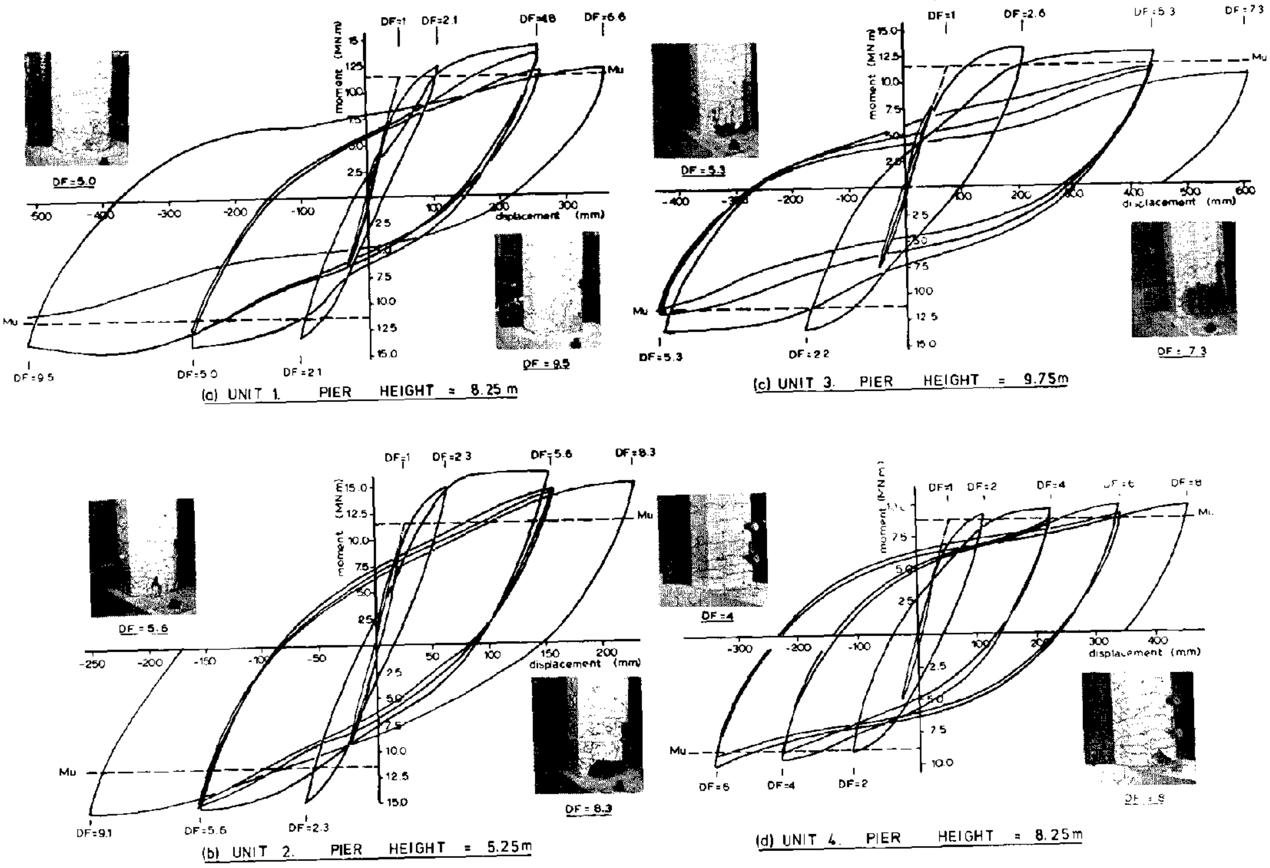


FIG. 13 MOMENT DISPLACEMENT LOOPS, 1/3 SCALE BRIDGE PIERS (SCALED TO PROTOTYPE DIMENSIONS).

was provided by an unbonded 35 mm (1.38 in) Macalloy prestressing bar passing through a central 45 mm (1.77 in) dia. vertical duct. Confinement consisted of a spiral of 4.3 mm (0.169 in) dia. plain round bar at a pitch of 10 mm. This content of spiral steel gave $\rho_s = 2.44\%$. A comparison of actual transverse steel content in the plastic hinge zone with requirements of ACI 318.77 [20], MWD [3] and N.Z. Draft Concrete Code [23] is included in Table 4.

Fig. 13 shows the moment-displacement response of Units 1 to 4 obtained from static cyclic load tests, together with photographs of the plastic hinge condition at salient points in the test sequence. In each case base moments have been scaled to prototype values, and displacements scaled to prototype displacement at the effective theoretical mass centre. Theoretical ultimate moment capacities based on measured material properties and an ultimate compression strain of 0.004 are indicated by dashed lines marked M_u . Displacement ductility factors are based on the experimental yield displacement found by extrapolation of the post-cracking elastic moment-displacement curve to the theoretical ultimate moment capacity, as shown by Δ_y in Fig. 14.

Response of Units 1 to 3, confined in accordance with minimum ACI shear requirements achieved displacement ductility factors in excess of 5. Maximum base moments obtained exceeded M_u by 12%, 24% and 10% for the three units

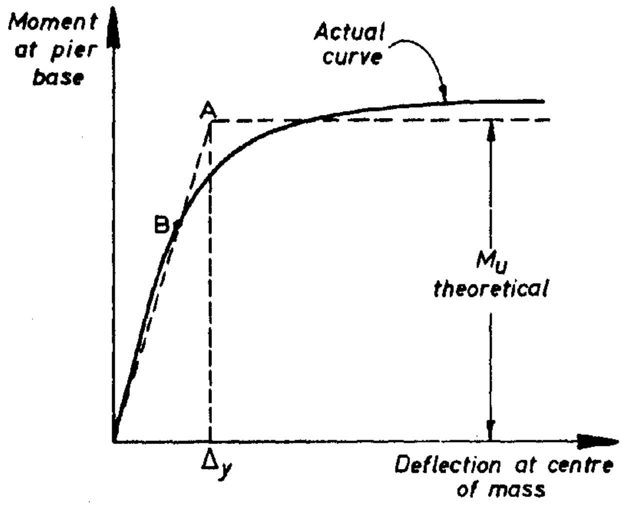


FIG. 14 DEFINITION OF YIELD DISPLACEMENT

respectively. This high excess capacity can be attributed to the short steel yield plateau (measured at 7.3 x yield strain) resulting in early strain hardening of the vertical steel, and an increase in concrete compression strength resulting from confinement by hoop steel and base pad. Despite the generally satisfactory behaviour, hoop steel strains exceeded yield at the first peaks to DF = 4.8, 8.3 and 5.3 for the three units respectively. With this yielding, and loss of cover concrete due to spalling, buckling of compression steel occurred progressively over the next few cycles with subsequent moment and stiffness degradation. This was particularly noticeable for Unit 3 in which the concrete core began to break up and a substantial cavity eventually formed inside the reinforcing cage (see Fig.15). It appears that yielding of the hoops was primarily due to hoop tension developed in restraining the longitudinal bars against buckling, rather than due to shear, since no significantly large diagonal tension cracks appeared across the units. As Fig. 15 shows, buckling of the longitudinal steel occurred over a distance of approximately four hoop spacing rather than in between hoops, indicating a larger hoop size would be required to prevent bar buckling, rather than a closer hoop spacing.

Unit 4 (Fig. 13d) with the heavier transverse steel content behaved exceptionally well. Stable hoops with only minor load and stiffness degradation were obtained at all displacement levels. The lower excess of maximum moment over computed ultimate moment compared with Units 1 - 3 results from a longer yield plateau of the stress-strain curve of the longitudinal reinforcement stress-strain curve. Hoop strains measured during testing of Unit 4 indicated that the hoop steel remained elastic throughout the test, with a maximum of 65% of the actual yield stress of 389 MPa (56,400 psi).

Moment-displacement plots for the two $^1/_6$th scale piers (Units 5 and 6), obtained from static cyclic load tests, are shown in Fig. 16 with results again scaled to prototype dimensions. It is significant that the loops for Unit 5 were obtained after subjecting the model pier to dynamic shaking (on the shaking-table) consisting of eight full El Centro 1940 N-S earthquakes and subsequent cyclic testing to a greater response in an unsuccessful attempt to cause failure. This explains the lower stiffness in the early stages of testing compared

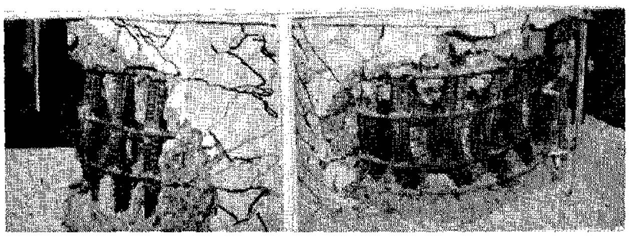


FIG. 15 COMPRESSION BAR BUCKLING UNIT 3[24]

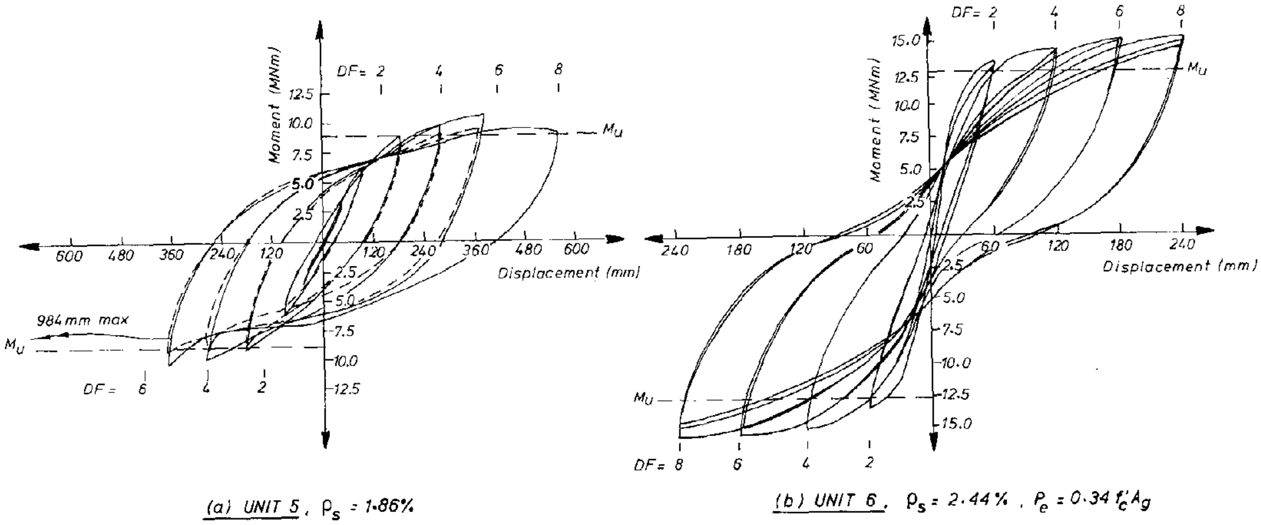


FIG. 16 MOMENT DISPLACEMENT LOOPS, 1/6 SCALE BRIDGE PIERS (SCALED TO PROTOTYPE DIMENSIONS)

with Unit 4. Despite the history of dynamic testing, behaviour at ductility factors in excess of 2 is very satisfactory. At the completion of testing the pier was displaced to 164 mm (corresponding to 984 mm in prototype terms, and a displacement ductility factor of 13.8) without buckling of longitudinal bars, yield of spiral steel, or significant drop in moment capacity.

The hysteresis loops for Unit 6 (Fig. 16b) provide confirmation of the excellent behaviour of the pier observed visually. The hysteresis loops narrow down at the centre (i.e. the loops become pinched) which is typical of columns carrying heavy compression, and occurs because the compression steel yields at a low moment resulting in a closing of cracks in the compression zone. This causes a relatively low stiffness at low moments followed by a stiffening when the concrete becomes effective in compression. However, the energy dissipation of the loops in Fig. 16b is good, ans the stability of the loops is impressive.

Although strains in the confining spiral exceeded yield strain at displacement ductility factors in excess of 4, the extent of yielding was insufficient to cause degradation of the confined core.

The equivalent plastic hinge length L_p can be calculated on the basis of the maximum measured curvature ϕ_u and the measured plastic rotation θ_p as

$$L_p = \frac{\theta_p}{(\phi_u - \phi_y)} \tag{24}$$

It is of interest that the six units had experimental plastic hinge lengths L_p of between 0.45D and 0.60D where D = pier diameter. There was no consistent variation with effective height of pier, scale of model, or axial load level, as would be predicted by Baker's [22] or Corley's [25] equations for plastic hinge length. Because of the method of measurement used, the experimental estimates of L_p are likely to be overestimates. Despite

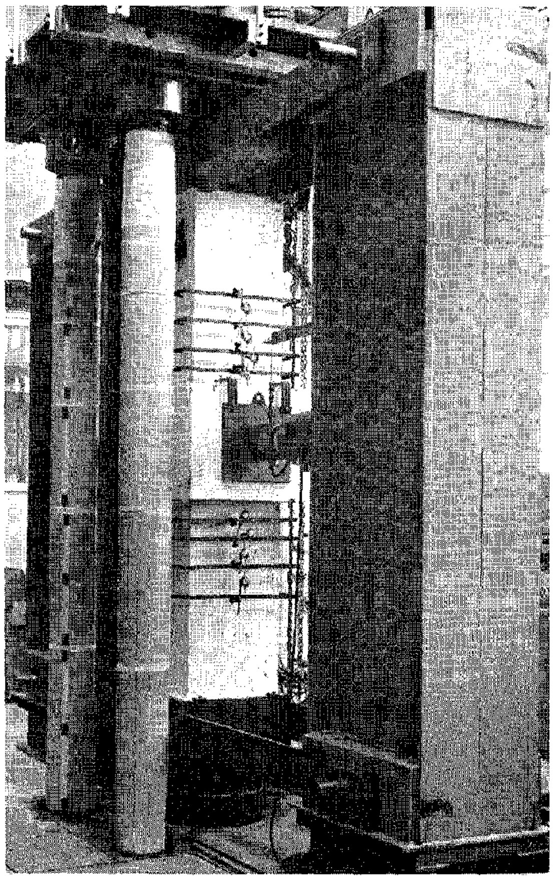


FIG. 17 RECTANGULAR BRIDGE COLUMN IN 10MN TEST MACHINE

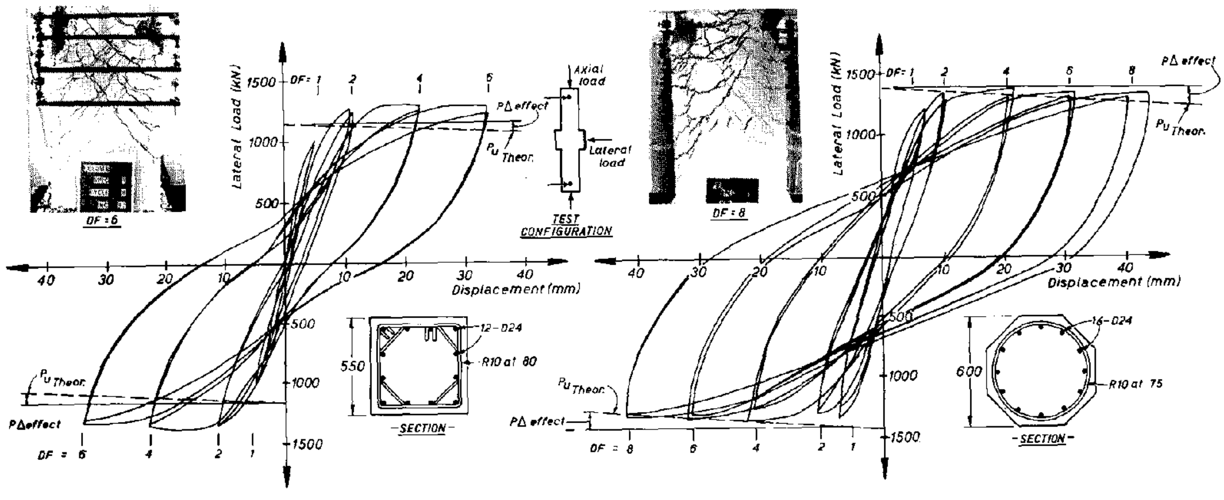
this, the experimental L_p values were smaller than those calculated by Baker's formula, implying higher experimental maximum compression strains. This has significance to the MWD design method [3] which is based on Baker's formula and a limiting ultimate compression strain.

The results from slow cyclic testing of units 1 - 6 indicate that MWD requirements [3] for confining steel content are very conservative for low axial load levels, but that ACI requirements, based on the minimum requirements of eqn. (23) are insufficient to prevent buckling of compression steel in the plastic hinge region of high ductility levels. Requirements of the N.Z. Draft Concrete Code [23] which are intermediate between the other two methods appear to be reasonable at low load levels, but on the basis of Unit 6 the N.Z. Draft Concrete Design Code might be expected to be a little light at higher axial load levels.

To check this supposition further, current test programmes [26, 27] are investigating the performance of square and octagonal columns with axial load levels between $0.15 f'_c A_g$ and $0.60 f'_c A_g$, confined in accordance with the N.Z. Draft Concrete Design Code [23]. The column sections are either 550 mm (21.7 in) square or 600 mm (23.6 in) octagon respectively, with a total height of 3.3 m (10.8 ft). The square columns have transverse steel consisting of overlapping rectangular and octagonal hoops; the octagonal columns have transverse steel consisting of circular spirals. The columns are axially loaded in a 10MN (1120 ton) capacity servo-hydraulically controlled Universal Testing Machine. Lateral load is applied to a stub at mid-height of the column. Fig. 17 shows a square section column under test.

Load-displacement hysteresis loops for the first two units tested in the programme are shown in Fig. 18. In Fig. 18 the theoretical ultimate load decreases with increasing deflection as a result of the P-Δ effect. It will be seen that stable hysteresis loops were obtained at displacement ductility factors of 6 for the rectangular column and 8 for the octagonal column. Straingauge measurements indicated that yielding of confining steel occurred at a displacement ductility factor of 4, but no degradation of the loops resulted.

It is of interest that maximum compression strains of 0.025 and 0.050 were obtained from the two units respectively, compared with ultimate com-



(a) RECTANGULAR COLUMN, $P = 0.26 f'_c A_g$, $f'_c = 23MPa$, $f_y = 300MPa$ (b) OCTAGONAL COLUMN, $P = 0.23 f'_c A_g$, $f'_c = 30MPa$, $f_y = 300MPa$

FIG. 18 LOAD DISPLACEMENT LOOPS, COLUMNS CONFINED TO N.Z. DRAFT CONCRETE DESIGN CODE [26, 27]

pression strains of 0.0092 and 0.0084 predicted by eqns. 19 and 20 respectively for the actual amount of confining steel.

Dynamic Testing

Few data are available for dynamic tests on bridge pier models. As mentioned above, Unit 5 of the series reported in the previous section was subjected to sinusoidal and simulated seismic shake-table testing [12]. Fig. 19 compares static and dynamic moment-displacement curves for Units 4 and 5 respectively, scaled to prototype dimensions. In both cases the curves represent the envelope obtained by joining peaks obtained on first obtaining a new maximum displacement. The dynamic curve is seen to be initially stiffer than the static curve, but the difference is not great. The good agreement between the static and dynamic moment-displacement curves shown in Fig. 19 gives confirmation of the applicability of statically obtained hysteresis loops for determination of seismic response. Theoretical and experimental displacement responses of the centre of mass of the model pier, to the first 10 seconds of the El Centro 1940 N-S record are shown compared in prototype dimension in Fig. 20. The theoretical displacement response

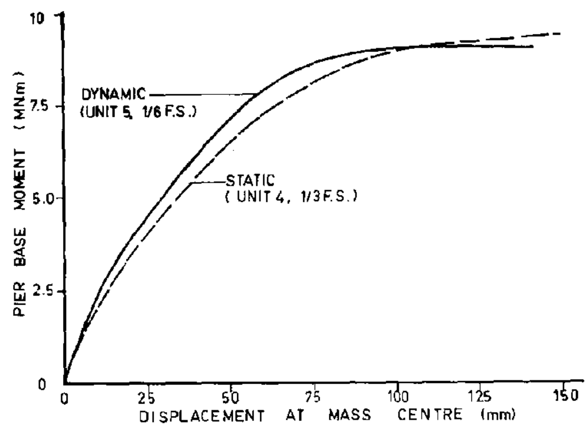


FIG. 19 STATIC AND DYNAMIC MOMENT DISPLACEMENT CURVES

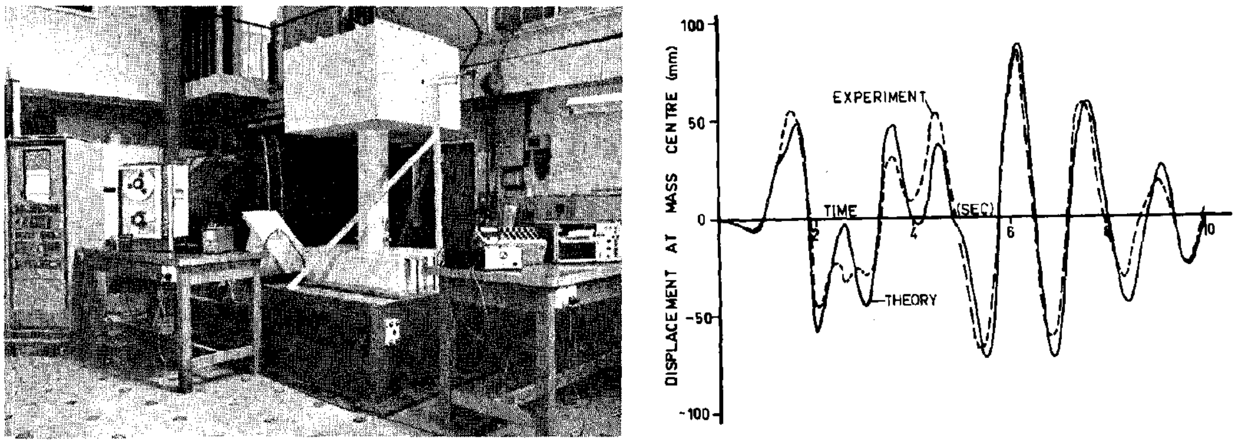


FIG. 20 RESPONSE OF 1/6 SCALE PIER MODEL TO EL CENTRO NS 1940

was determined using Sharpe's [14] computer program assuming 7% critical viscous damping (measured in sinusoidal tests) and a bi-linear moment-curvature relationship. It is evident that the agreement between experiment and theory in Fig. 20 is very satisfactory.

Biaxial Seismic Attack

Under real seismic attack, pier deformation will occur in both longitudinal and transverse directions rather than the simple uniaxial attack simulated by the test programmes described above. Okada [28] has reported results from a series of rectangular reinforced concrete columns 150 mm (6 in) square subjected to constant axial load of $0.1 f'_c A_g$ and either uniaxial, or bi-directional lateral loading. He found that in cyclic tests up to maximum displacement ductility factors of about 3, failure occurred at a significantly earlier stage when displacement was applied as a circular movement of the load point, compared with a uniaxial movement. The ratio of confining steel used by Okada, at 0.56%, may be considered to be low for the axial load level, and anchorage of longitudinal steel at the plastic hinge by welding to end plates may have influenced performance, but the results indicate that caution must be exercised in extrapolating uniaxial tests to bidirectional conditions. Clearly more testing, preferably to a larger scale than Okada's tests, is required to establish the significance of bidirectional deformation on ductility capacity.

DETAILING

The importance of good detailing was clearly demonstrated in some of the pier failures in the San Fernando earthquake [1]. Critical factors include the following.

Lapping of starter bars in plastic hinge zones: From a construction point of view it is desirable to lap longitudinal reinforcement with starter bars at the column base. This is undesirable on two counts. First, the tension splice occurs in a potential plastic hinge region where conditions for bond will be extremely severe. This appears to have been the main cause of failure of one of the bridges of the Golden State-Foothills freeway interchange in the San Fernando earthquake. Second, lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening of the column over the lapping region. This may result in very severe local curvature demand. Testing of this common construction detail is urgently required.

Anchorage of confining reinforcement: Loss of cover concrete in the plastic hinge zone, as a result of spalling, requires careful detailing of confining steel. It is clearly inadequate to simply lap spiral reinforcement by 48 bar diameters, as required by ACI [20,21], if the cover concrete is going to spall off, allowing the spiral to unwind. Under these conditions full strength lap welds are essential. Similarly, rectangular hoops must be adequately anchored by bending ends back into the core. The draft N.Z. Concrete Design Code [23] requires at least a 135° bend with an extension of at least 10 tie bar diameters back into the core, or an equivalent welded anchorage.

Spacing of transverse bars in plastic hinge zones: Spacing of transverse steel must be sufficiently small so that buckling of longitudinal steel is avoided. ACI requirements for spirals (clear spacing between spirals less than 75 mm (3 in)) will generally be adequate for all except very small diameter longitudinal bars, but requirements for tied columns (spacing less than 16 longitudinal bar diameters or 300 mm (12 in)) in Appendix A [20] are unlikely to be sufficient to prevent buckling of compression steel once cover concrete has spalled. For example, tests on beam plastic hinges in reinforced concrete frames have resulted in buckling of compression bars between ties when ties have been spaced more than 6 longitudinal bar diameters apart [29].

SHEAR DESIGN

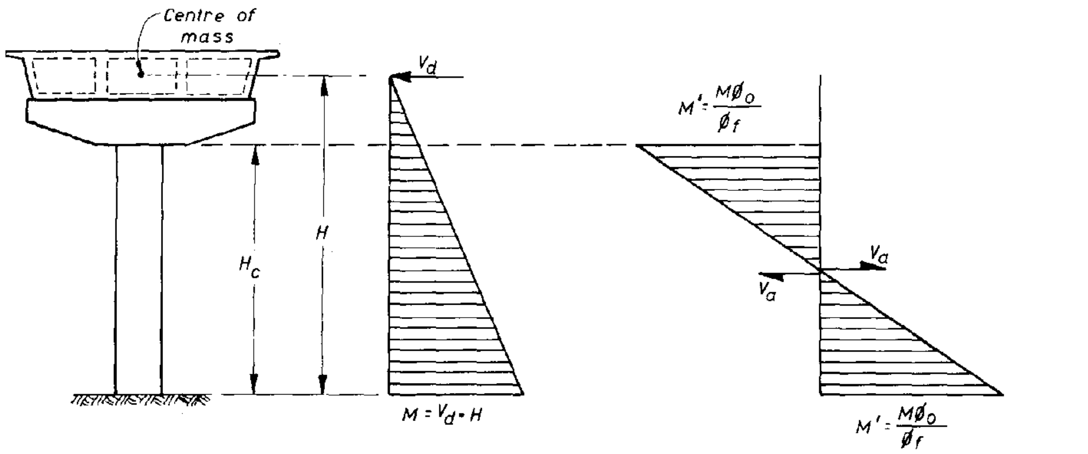
Shear failure of bridge columns must be avoided by a capacity design approach. Dependable shear strength, based on appropriate capacity reduction factors must exceed the maximum probable shear developed at flexural capacity of the column. Overstrength results from material strengths exceeding specified minima, and from effects of strain hardening of flexural reinforcement, and confinement of concrete.

Thus for the single stem bridge pier of Fig. 21(a), the required base moment capacity M_u will normally be based on transverse characteristics, as

$$\phi_f M_u = V_d \cdot H \tag{25}$$

where ϕ_f is the flexural capacity reduction factor, V_d is the design seismic base shear appropriate to the natural period in the transverse direction, and the level of ductility adopted. H is the height of the centre of mass above the column base.

With a ductile design approach it is probable that the actual flexural capacity will be developed both longitudinally and transversely. In the longitudinal direction, plastic hinges may form at top and bottom of the columns with moments possibly reaching the overstrength level of



(a) Structure (b) Base Moment, Transverse Response for Code Level V_d (c) Actual Shear, Longitudinal Response at Pier Overstrength Moments

FIG. 21 SHEAR REQUIREMENTS FOR SINGLE STEM BRIDGE PIER

$$M' = \phi_o M_u$$

$$\frac{\phi_o}{\phi_f} \doteq V_d \cdot H \qquad (26)$$

Thus the maximum actual design shear on the column would be

$$V_a = 2\frac{\phi_o}{\phi_f} \cdot V_d \frac{H}{H_c} \qquad (27)$$

where H_c is the clear column height (see Fig. 21a), and dependable shear strength of the column would have to equal or exceed this value. Note that for typical values of $\phi_o = 1.25$, $\phi_f = 0.75$, and $H/H_c = 1.20$, eqn. 27 yields $V_a \doteq 4.0\ V_d$.

Within the plastic hinge zone, the concrete contribution to shear capacity is undependable, particularly at low axial load levels, as a result of full section cracking under load reversals. The draft N.Z. Concrete Design Code requires the concrete contribution to be neglected for axial load levels less than $0.1 f'_c A_g$, and for higher levels uses the expression in eqn. (28),

$$v_c = 0.25\ (1 + \frac{f'_c}{25})\sqrt{\frac{N_u}{A_g} - \frac{f'_c}{10}} \qquad (28)$$

which approaches the ACI values assymptotically at high axial loads.

It is probable that the use of $v_c = 0$ for low axial load levels is over-conservative, as results from Units 1 - 3 reported above indicated that the concrete in the plastic hinge zone was contributing at least $v_c = 0.1\sqrt{f'_c}$MPa.

CONCLUSIONS

Factors affecting the seismic performance of reinforced concrete bridge columns have been discussed. On the basis of dynamic inelastic time-history analyses, ductility demand for a given bridge column is related in complex fashion to the design force level, natural period and earthquake characteristics, but is not particularly sensitive to the type of hysteresis loop chosen to model the moment-curvature behaviour of the plastic hinges, nor to small variations in the level of elastic damping adopted. Additional elastic flexibility in the form of foundation compliance or bearing deformation was shown to theoretically result in large increases in curvature ductility demand if the displacement ductility factor for the structure remained unchanged. Although dynamic analyses confirmed this result for bridge piers whose natural period was lower than that corresponding to peak response for the earthquake under consideration, the curvaure ductility was comparatively insensitive to the increased elastic flexibility for longer period bridge piers. That is, increased elastic flexibility resulted in a reduced structure displacement ductility demand. More research is needed in this area. Further research is also needed to provide more realistic modelling of soil yield, and to investigate further the influence of soil damping on displacement ductility demand.

Theoretical considerations of moment-curvature curves for confined concrete indicate that confining steel requirements for bridge column plastic hinges should be a function of axial load level as well as material properties and section dimensions. On this basis, ACI requirements are non-conservative for both low and high axial force levels, and somewhat severe for medium axial force levels [$0.1 f'_c A_g \leq P_e \leq 0.3 f'_c A_g$]. New Zealand's MWD requirements based on limiting values for ultimate concrete compression strain result in theoretically conservative designs. More testing is urgently needed to establish the stress-strain characteristics of confined concrete under combined bending and axial load at realistic seismic loading rates. Effects of repeated loading should be investigated.

Experimental results from tests of bridge pier models tend to confirm the observations based on theoretical considerations. Displacement ductility factors of at least 6 were obtained from adequately confined octagonal bridge columns with ultimate concrete compression strains as high as 0.05 being recorded. Lightly confined piers designed in accordance with minimum ACI shear requirements (which governed the design) eventually failed by buckling of compression steel. Further large scale testing is required to investigate available ductility of (a) columns at medium to high axial load levels, (b) columns of other section shapes (rectangular, elliptical, hollow box), (c) influence of lapping of longitudinal steel in plastic hinge zones, (d) columns subjected to seismic displacement simultaneously in two orthogonal directions, (e) columns with high M/VD ratios.

Dynamic shake-table testing should be continued to provide more confidence in results of dynamic inelastic analyses, and in results of slow static cyclic testing.

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SPECIAL CONSIDERATIONS AND REQUIREMENTS FOR THE SEISMIC DESIGN OF BRIDGES IN JAPAN

by

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ABSTRACT

The considerations on special seismic design of bridges are summerized in this paper. Several ways on transmitting horizontal seismic force from superstructure to substructures are introduced as basic seismic considerations. Four types of damping devices using in Japan are presented. These damping devices are effective for distribution of horizontal seismic force to substructures.

Bearing supports need special considerations because they are vurlerable parts in bridges. Installing restrainers at bearing and strengthening anchorage of bearing are described as necessary measures for supporting superstructure safely during earthquake.

As the considerations for preventing superstructures from falling, the installation of restrainers at bearing, the enlargement of bridge seats and the installation of connecting devices are presented. The necessity of the installation of connecting devices for existing bridge is also emphasized.

The considerations for abutment are described as problems of the considerations for the surrounding soil.

Placing reinforcement in column is very significant theme for ductility of column. An example of placing reinforcement and recent laboratory tests on this problem are presented.

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INTRODUCTION

It is a basic concept in seismic design of structures that structures should be remain safe against the strongest earthquake which is anticipated from the past records. Especially in the case of bridges, even if they are partially damaged by seismic force, they are required to let traffic run through just after earthquake without serious trouble, and to fulfil their function. For the above purpose, special considerations should be paid. A bridge type which is adequate to a structure in seismic region should be examined. Design is done following specifications, that is natural. However, good seismic design would not be done without further special considerations.

Two propositions are thought as essential requirements in the considerations. The first is to avoid fall of superstructures. The second is to avoid serious damages which lead to such a situation that bridge can not stand to be use. Adopting some aseismic devices seems to be effective measure for them. And also strengthening structural details in vulnerable parts must be done.

In this paper, systems transmitting horizontal seismic force from superstructure to substructure are discussed at first. It must be basic consideration for doing balanced seismic design when a bridge is constructed in seismic region. At next, some aseismic devices — damping devices, restrainers at bearings and fall-proof devices — are introduced. Structural details are also described. They positively effect not only the ultimate strength and ductility of structural member at earthquake, but also bridge's service for traffic after earthquake. All of them are considerations for seismic design in Japan.

SYSTEMS CARRYING SEISMIC FORCE

In practice, seismic design is done such that substructure carrys horizontal seismic force from superstructure which is calculated previously. The systems carrying seismic force mean the ways transmitting horizontal seismic force from superstructure to substructure. To decide this transmitting system is very important, because, if the choice of the system was not adequate, construction would be more difficult and expensive in cost.

For highway bridge the continuous structures are rather desirable from the viewpoint of automobile driving and of earthquake resistance in comparison with simply supported bridge. However, a continuous structure has a problem that it must be designed for temperature change if it is fixed to many piers. It is one of most important problem in design how to transmit horizontal force from superstructure to substructure, minimizing the influence for structure due to temperature change. Various ways have been tried and developed for it.

Several Ways Transmitting Seismic Force to Substructures

In a continuous bridge, one of the supports is generally designed as a fixed support and others as movable supports, for avoiding restraint against contraction and expansion due to temperature change, creep, dry shrinkage and so forth. In this way, the sectional dimension of the fixed substructure becomes extremely thick in comparison with movable one as illustrated in Fig. 1, because of concentration of longitudinal horizontal seismic force. In result, the design sometimes falls in with difficulty and uneconomy in construction, and also piers surely lose the dimensional balance.

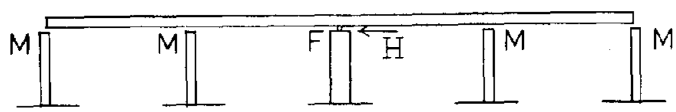


Fig. 1 One intermediate pier carrying horizontal seismic force

In case where sections of piers are requested to be almost same dimension, such as the elevated structure in urban freeway, design mentioned above can not be adopted. Adopting consecutive simple span bridges, the substructures can be easily designed with same dimension as shown in Fig. 2 in this case. A simple span bridge, however, is not good in earthquake resistant design, since a problem on fall of superstructure is more serious.

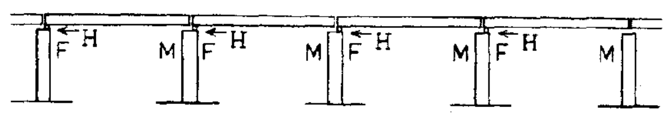


Fig. 2 Consecutive simple span bridges

If piers are considerably high and flexible, the plural supports of a continuous beam are designed as fixed supports as shown in Fig. 3. The horizontal seismic force can be distributed to these piers, while piers softly resist against horizontal movement due to temperature change, because of pier's flexibility. In this design, piers can be designed to be almost same dimension.

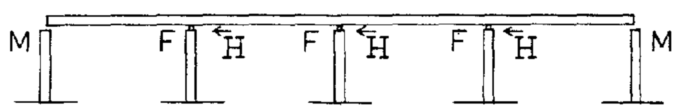


Fig. 3 Plural flexible columns carrying horizontal seismic force

A rigid frame is provided to resist longitudinal horizontal seismic force of a continuous beam by placing it at one end of continuous beam as shown in Fig. 4. In this case, all the spports except one end support can be designed as movable supports.

Therefore, this system can be adopted, even if piers are not flexible such as concrete piers, and the sections can be designed to be almost same dimension. In lieu of the end rigid frame, a relatively large abutment is surely available.

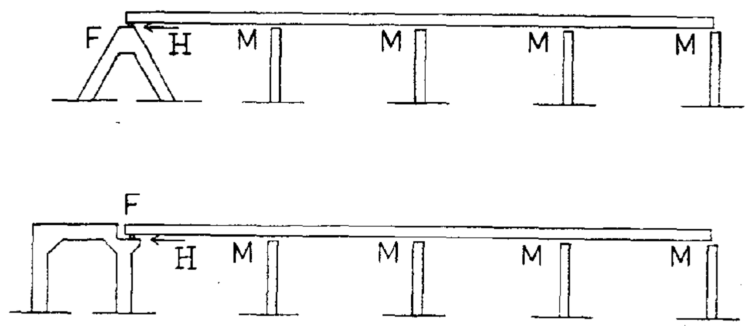


Fig. 4 Rigid frame carrying horizontal seismic force

These systems carrying seismic force must be applied suitably in accordance with conditions of location.

Multi-Span Continuous Bridges

Recently, multi-span continuous bridges have begun to be adopted as the elevated structure for national highway and urban freeway. This is a continuous bridge with long bridge length having about ten or more spans. The adoption of this type of bridge is caused by such environmental consideration as reducing noise problem occurring at expansion joints of bridges. This structural type, however, seems to be better in earthquake resistance of bridge, because of its high degree of redundancy.

Although the design method of this bridge is on an extension of ordinary continuous bridge, there are some difficulties on carrying horizontal forces due to temperature change and earthquake.

Fig. 5 shows one of the examples adopted in the Tokyo Metropolitan Expressway [1]. The superstructure is 12 spans steel continuous box girder (total length of the bridge is 507.5 M). The substructures are reinforced concrete piers and cast-in-place concrete piles with 2.5 - 3.0 M in diameter, 33 - 36 M in length. The geological conditions are not so good involving soft silty clay alluvial layer. The substructure does not have footing but connecting member between piles and a pier (as shown in Fig. 5), so that the substructure including piles flexibly deflects (or resists) against horisontal force due to temperature change and earthquake.

The temperature change is considered $\pm$ 35°C and seismic coefficient is 0.26. The nine fixed substructures deflect and resist for girder's expansion and contraction. And they also carry horizontal force due to earthquake.

In this design, ridigity of substructures is evaluated including displacement of piles. The spring action of sub-soil is considered in lateral and vertical direction. The spring constant between pile and soil surrounding pile in lateral direction can be estimated from the coefficient of lateral sub-soil reaction. The values of coefficient of lateral sub-soil reaction were assumed as 1.0 - 4.0 kg/cm^3 for earthquake load and 0.5 - 2.0 kg/cm^3 for temperature change. The spring constant for temperature change must be different from that of earthquake, because the velocity of loading due to

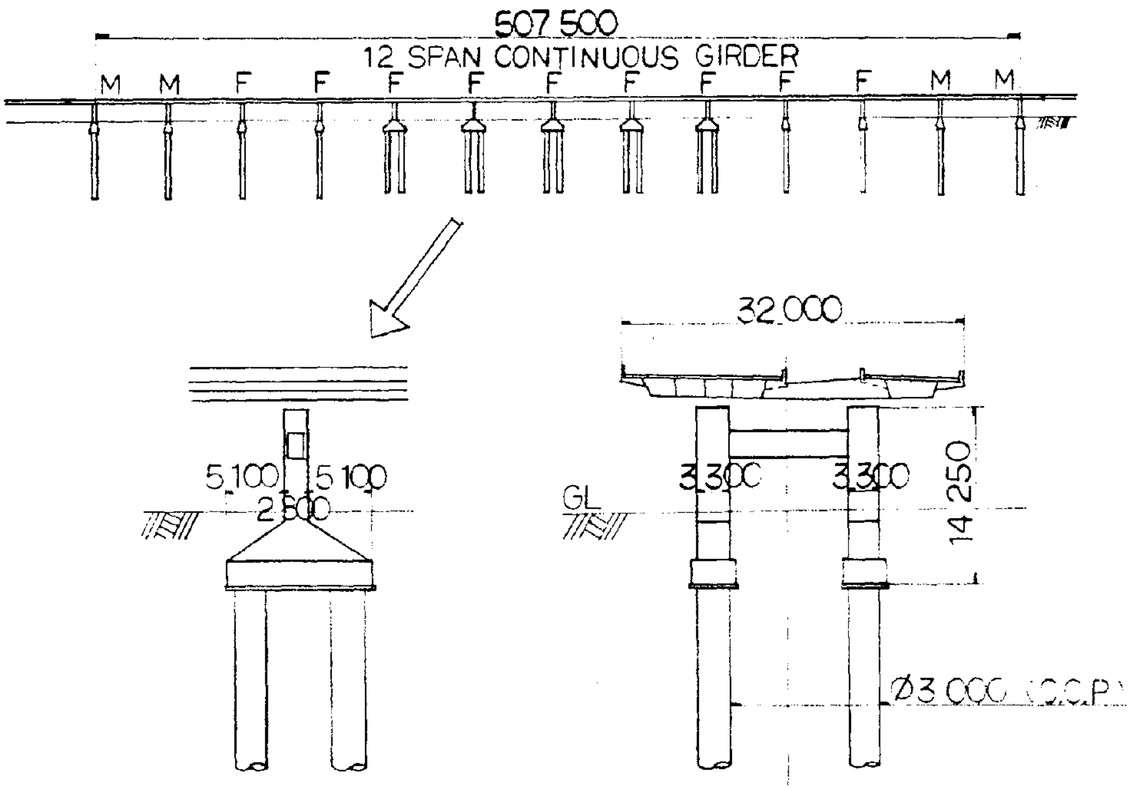


Fig. 5 Multi-span continuous bridge

temperature change is very slower in comparison with that due to earthquake. These values were examined by loading tests and it is confirmed that the spring constants for temperature change are almost one-half of the value for earthquake load. The one of loading test results is showing in Fig. 6 [2].

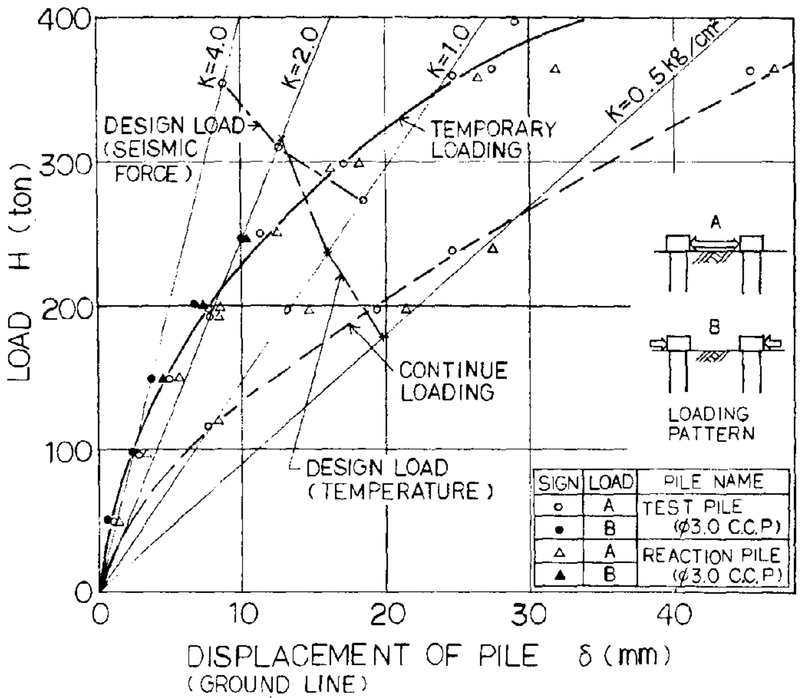


Fig. 6 Loading test result, pile top displacement vs lateral load

In this type of bridge, such a flexible pier must be provided in order to make balanced design against temperature change and seismic force.

The adoption of multi-span continuous bridge will be increased as the current of the world. It is necessary to do further study on the systems carrying horizontal force and dynamic behaviors during earthquake.

On the other hand, such design as considering displacement of foundation implies the developing possibility of design of bridges having many fixed support with flexible piers as shown in Fig. 3. In simple span bridge, two supports can be designed as fixed support in this case. In such a design, the distribution of seismic force to substructures will be able to be done better.

SEISMIC DESIGN USING DAMPING DEVICES

By installing damping devices between a girder and substructures at supports, horizontal seismic force from superstructure can be distributed to the substructures. The vibration of substructure due to earthquake is also controlled and reduced by damping effects. As a result of installing damping devices, all substructures can be designed to carry almost same seismic force, and their displacement are lessened.

Four types of damping devices using in Japan are introduced. The basic idea of them is that the damping devices would not resist for such slow movements as contraction and expansion due to temperature change, but the damping devices would generates resisting force for rapid movement such as can be caused by an earthquake. Therefore a girder contracts and expands freely due to thermal change same as a girder supported on movable supports, and, at earthquake, all supports installing damping devices work like fixed supports for horizontal seismic force.

It is certain that damping devices have damping function for dynamic movement of bridge during earthquake. However, seismic design using damping devices is done expecting its distribution function for horizontal seismic force. Therefore, the adoption of damping devices should be examined in a link of systems carrying horizontal force. And it seems to be useful to apply to multi-span continuous bridge.

Oil Damper

Oil damper is a kind of shock-absorber consists of a cylinder filled up oil and a piston having oriffice as shown in Fig. 7. It is installed beside bearing at movable support as shown in Fig. 8.

A girder is supported by one fixed support and other movable supports installing oil dampers. A model of bridge having oil dampers is illustrated in Fig. 9 where oil dampers are installed at four movable supports. Fig. 10 shows

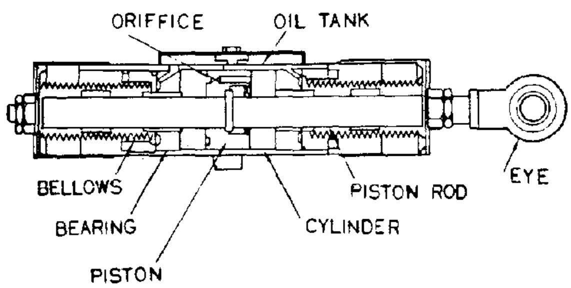


Fig. 7 Oil damper

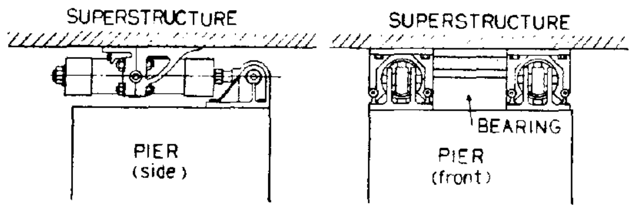


Fig. 8 Oil damper at movable support

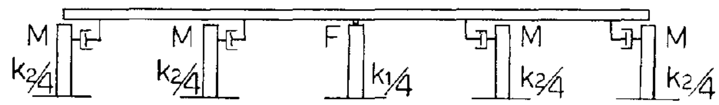


Fig. 9. 4-span continuous girder installing oil dampers

mechanical model of oil damper system. The differential equations of the system is derived as follows

$$M\ddot{x}_2 + C(\dot{x}_2 - \dot{x}_1) + k_1(x_2 - X_0) = 0$$

$$C\ (\dot{x}_2 - \dot{x}_1) - k_2\ (x_1 - x_0) = 0$$

where M = mass of superstructure
C = damping coefficient
k_1 = spring constant of pier with fixed support
k_2 = spring constant of piers with movable support
x_0, x_1, x_2 = displacement as shown in Fig.10

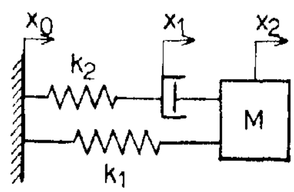


Fig.10 Mechanical model of oil damper system

From these equations relative displacement ratio between x_2 and x_0, or x_1 and x_0 is obtained. Fig. 11 shows displacement response curve of single-degree of freedom system according to the El Centro Earthquake record (1940). ε is damping ratio defined $\varepsilon = C/2\nu M$, where $\nu = \sqrt{k_1/M}$. Optimum damping ratio is such ε that $|(x_2 - x_0)/x_0|$ takes maximum value at constant point on resonance curve of fixed pier [3].

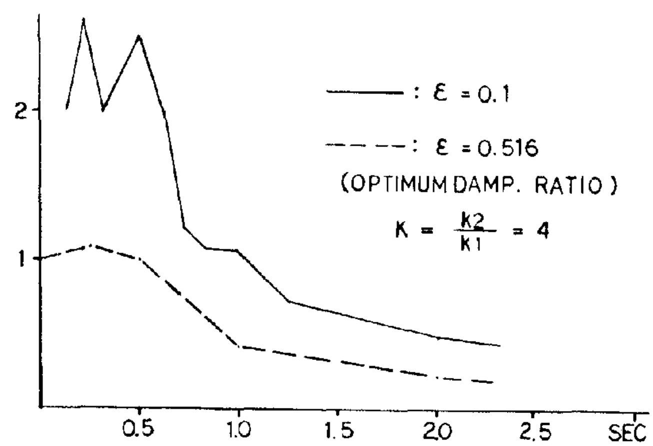


Fig. 11 Response spectrum of El Centro Earthquake (1940)

Relative displacement ratio between fixed support and movable support estimated gives each horizontal longitudinal force carried by fixed support and movable support. In the case of 4-span continuous girder, the calculation results of horizontal force are shown in table [4].

Horizontal Force Carried by Piers

Unit : ton

	Mov.	Mov.	Fix.	Mov.	Mov.
Without Damper	-	-	450	-	-
With Damper	80	80	130	80	80

This kind of damping devices has been already applied to more than ten bridges in Japan after the first application to the Tokyo Metropolitan Expressway at 1962. One of these bridges, Kaihoku Bridge shown in Fig. 12

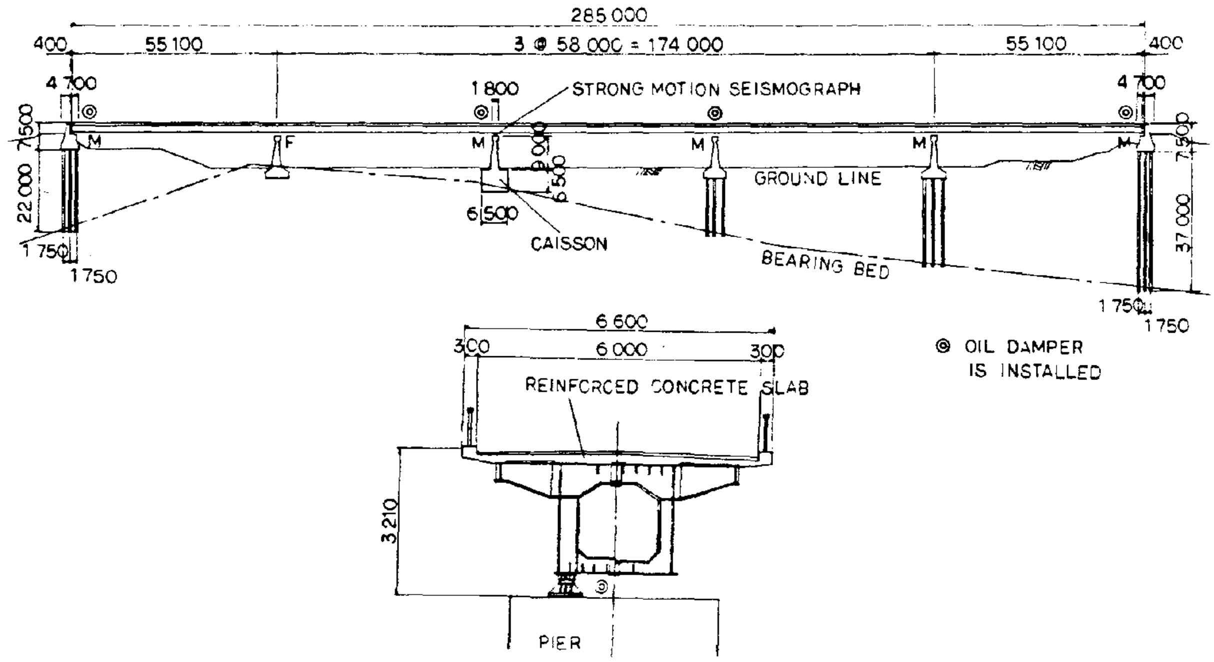


Fig. 12 Kaihoku-bridge

suffered very strong earthquake in June 1978 (Miyagi-ken - Oki Earthquake). The strong motion seismograph installed in this bridge showed peak acceleration value of 192 gals at ground level, more than 500 gals at pier top in longitudinal direction, and 289 gals at ground level, 333 gals at pier top in transverse direction. The bridge, however, was not damaged except slight damage of mortar at bearing bed [5].

Prestressing Wire Damper

A prestressing wire damper system consists of prestressing wire with movable bearings. All supports are movable while the girder is elastically fastened to piers by means of prestressing wires as connecting members as shown in Fig. 13.

The section of wire can be estimated by static calculation [6]. Taking a 4-span continuous beam illustrated in Fig. 13, resistance due to influence of temperature change is calculated based on equilibrium and geometrical conditions of forces after displacement due to temperature change as shown in Fig. 14. Tensile stress of each member or horizontal force to each pier are also easily calculated, by assuming that horizontal seismic force is introduced only by connecting members. The section of prestressing wire is decided by these forces. Fig. 15 shows an example using this damper.

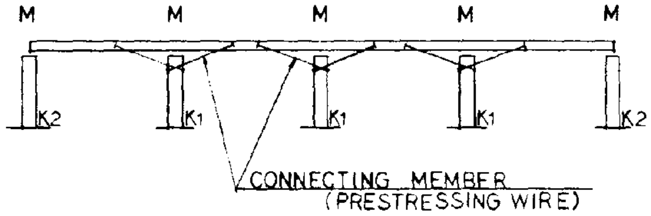


Fig. 13 4-span continuous girder installing prestressing wire damper

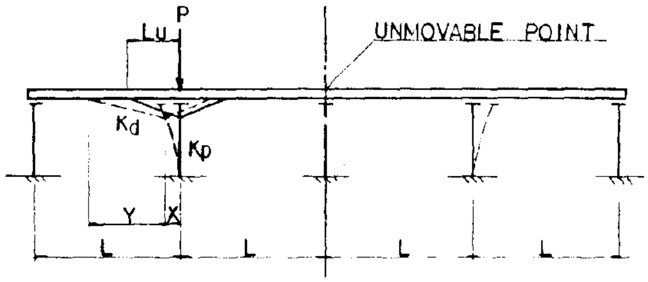


Fig. 14 Deformation of continuous girder due to temperature change at the prestressing wire system

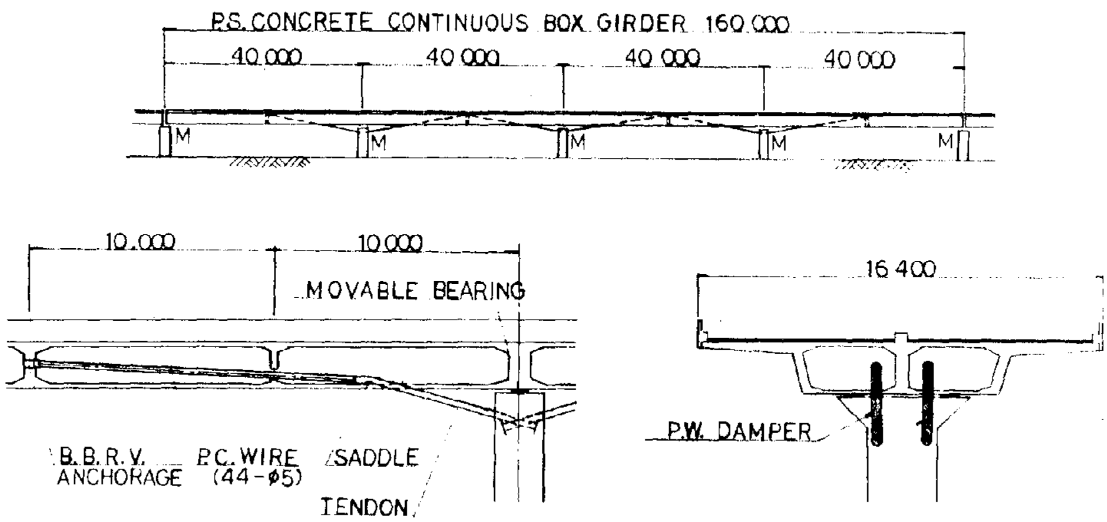


Fig. 15 An example of 4-span continuous bridge adopting prestressing wire damper

Fig. 16 shows mechanical model of this system. The vibration of this system is expressed by the following equation, when the major damping is expected to frictional force at movable supports.

$$(M_1 + M_2)\ \ddot{x}_2 + k_2\ (x_2 - x_1) = 0$$

When sliding occurs, equation can be derived.

$$M_1\ddot{x}_1 + k_1\ (x_1 - x_2) \mp\ F = 0$$

$$M_2\ddot{x}_2 + k_2\ (x_2 - x_0) - k_1(x_1 - x_0) \mp F = 0$$

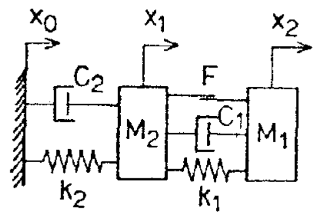


Fig.16 Mechanical model of prestressing wire damper system

where M_1 = mass of superstructure

M_2 = mass of piers

k_1 = spring constant of wires

k_2 = spring constant of piers

x_0, x_1, x_2 = displacement as shown in Fig. 16

F = frictional force

Simulated response of a continuous prestressed concrete bridge to El Centro Earthquake (1940) in the case of frictional coefficient at movable supports μ = 0.1 is shown in Fig. 17 [3].

This damper has been already adopted for elevated structures for the urban freeway in Japan. And it is now planning to applied to 400 m long multi-span continuous girder.

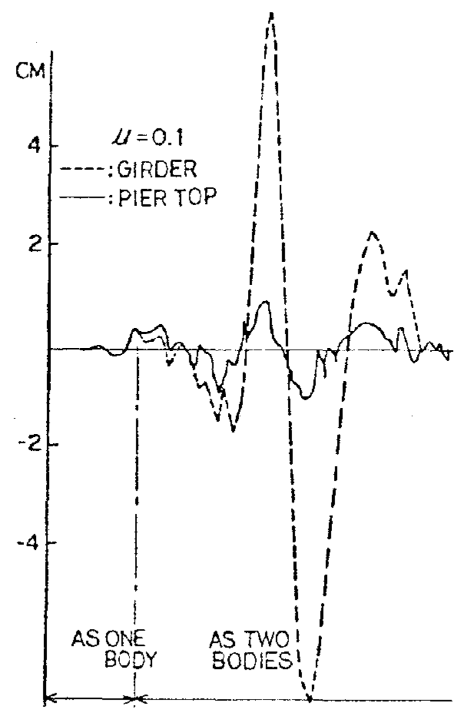


Fig.17 Displacement response curve at μ = 0.1 (Prestressing wire damper system)

Box Stopper

A box stopper consists of a prismatic bar of which upper half part is embedded into concrete of superstructure, lower part of the prismatic bar is inserted into a box which is fixed by concrete of substructure as shown in Fig. 18. The space between them is filled with viscous materials, which has resistance for rapid movement during earthquake, but has neglisible resistance against slow movement. The degree of resistance can be adjusted by amount of gap between the walls. There are box stoppers for fixed support provided plate springs and for movable support as shown in Fig. 18. The whole horizontal force of normal time is resisted only by the fixed pier. When a seismic force hits the bridge, it is efficiently distributed to each pier by viscous action of the fluid [7].

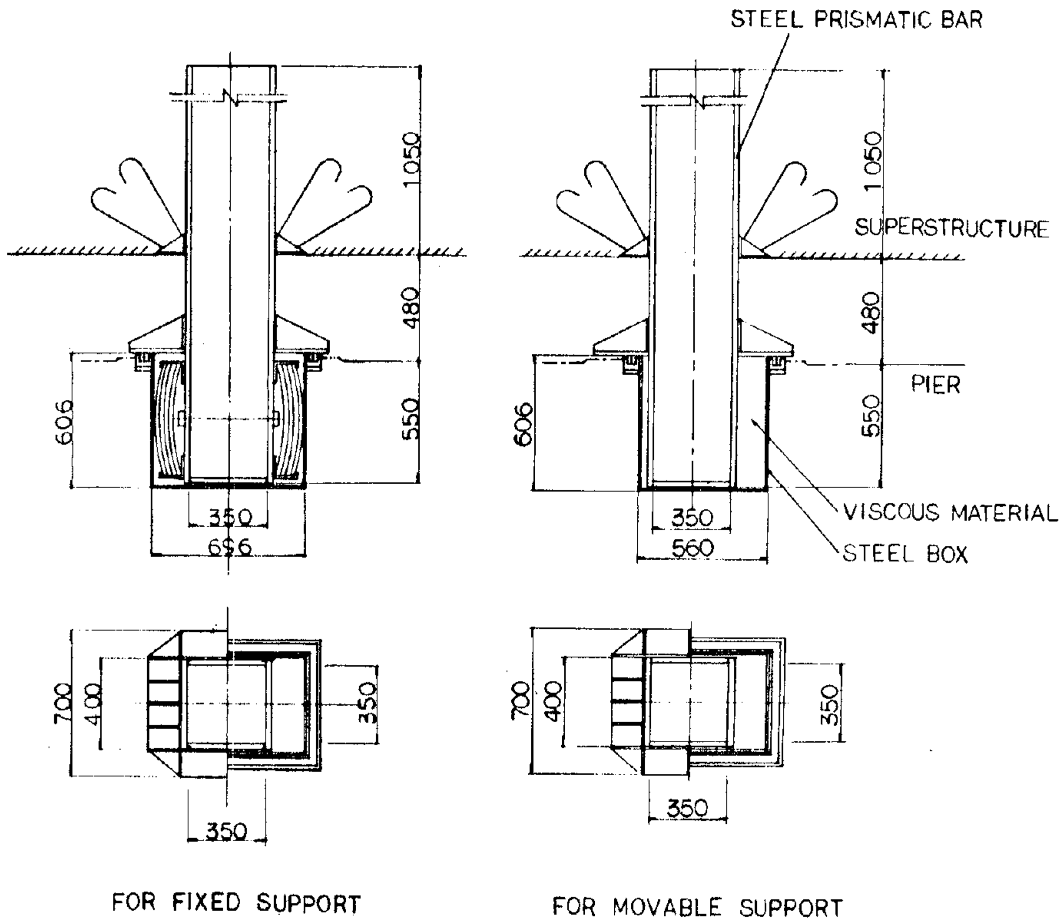


Fig. 18 Box stopper

This stopper is installed in many prestressed concrete bridges for the national railways in Japan. Generally design is done statically, and some of them were examined their design by dynamic analysis considering regidity of piers.

Multi-Shear Stopper

This is a viscous-shear damping devices. In Fig. 19, a rod C is connected with superstructure and a box D is fixed at pier top. Steel plates A are connected with the rod C and steel plates B are put between steel plates A. The viscous material filling the gap resists against a dynamic load by viscous shear. The viscous materials between two planes generate shear-resistance force in accordance with a relative moving velocity leading to absorbtion of vibration energy and to damping vibration [8].

The damping coefficient is controlled by viscocity of materials and area of steel plates. Three bridges were installed this stopper in Japan.

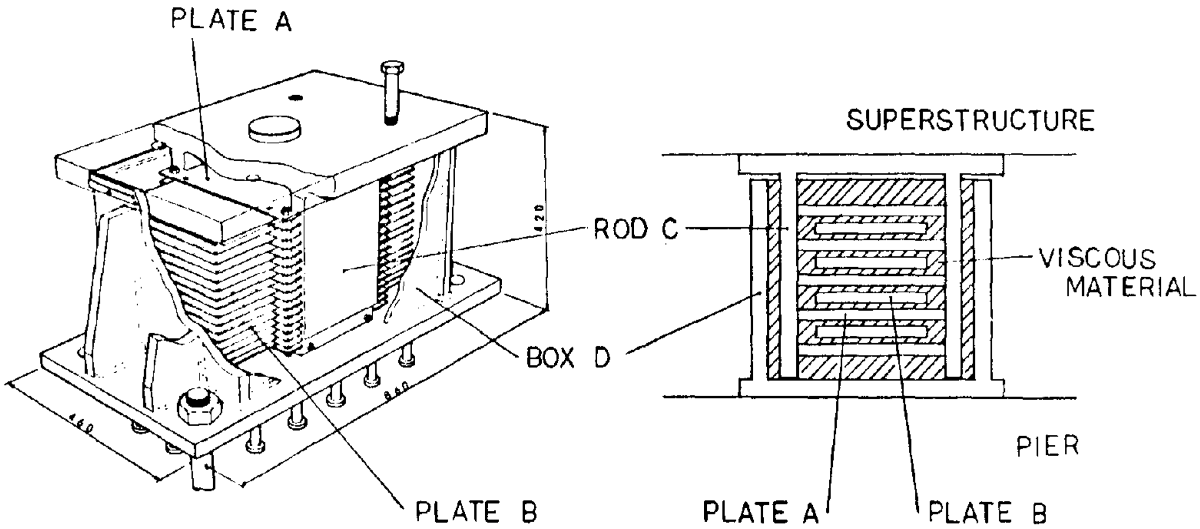


Fig. 19 Multi-shear stopper

BEARING SUPPORTS

There have been many examples of damages in supports of bridges in the past earthquake. The damages can be classified into three categories.

(a) Slipping of a girder out of bearing, and local buckling of a main girder and bracings.

(b) Collapse of bearing itself.

(c) Damages of anchorage of bearing and pier concrete around bearing at pier top.

The countermeasures correspond to each categories are necessary. Such two considerations are done as installing restrainers against excessive movement of a girder at bearings and strengthen details around bearing.

Restrainers at bearings

It is required in the Specifications for Earthquake Resistant Design of Highway Bridges [9] that restrainer against excessive movement in longitudinal direction of a girder at earthquake must be installed in bearing at movable support. Fig. 20 shows standard type of movable bearing for steel girder. Movement in transverse direction is restricted by block projected from lower part of bearing (denoted A in Fig.) as a restrainer. For excessive movement in longitudinal direction at movable support is restricted by the block and projection of soleplate (denoted B in Fig.) as restrainer. These restrainers are designed against horizontal force of 1.5 times as much as

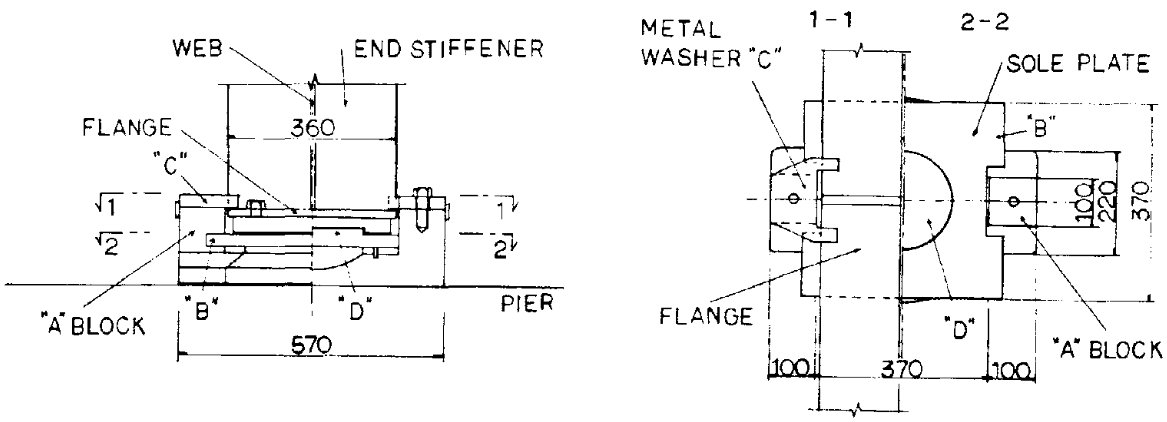


Fig. 20 Movable bearing for steel girder

horizontal seismic coefficient × dead load reaction at support. Metal washer (denoted C in Fig.) is also designed as a restrainer gainst uplift of a girder. A 10% of dead load reaction is considered for this design.

Bearing has to be steadily fixed with superstructure. Fig. 20 shows bearing connected by four bolts, but round projection (denoted D in Fig.) is designed to resist against horizontal force at fix support. In fixed bearing, the projection of soleplate exactly fit the block. The horizontal force from superstructure can steadily transmitted to bearing through these devices. However, in a part of steel girder, for instance lower flange, lateral bracing and sway bracing, there is possibility of occurrence of buckling when the girder is restricted its movement by bearings. Lateral bracing and end sway bracing should be also examined for seismic force.

Anchorage of Bearings

In general design of bearings, they have ribs in bottom as shown in Fig. 21 so as to transmit horizontal force to substructure concrete. The shallow box type hole is previously made in which the bearing will be set. Contractionless mortar will be poured after setting bearing. Anchor bolts which is used for setting bearing and preventing bearings from uplift are also installed previously in anchor bolt holes which are filled by contractionless mortar after setting bolts.

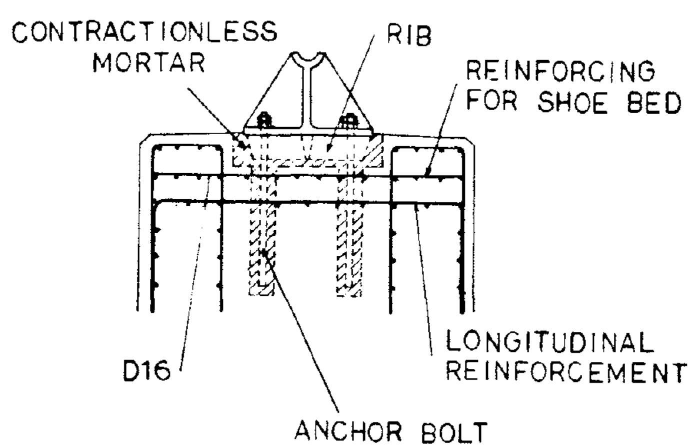


Fig. 21 Anchorage of bearings

Another type of anchorage was developed so as to do steady execution of setting bearing in site. Fig. 22 shows welding type anchorage of bearing using for the Tokyo Metropolitan Expressway. Anchor bolts are previously burried at the same time when pier concrete is plàced. Then steel base plate (30 mm thick) is placed in accurate level and contractionless mortar is poured under the plate. Bearing is connected by welding in accurate position after erecting a girder. By burrying the anchor bolts, a setting frame is used for increasing accuracy of the working.

Reinforcement under a bearing is also important in order to transmit certainly seismic force to substructure. Fig. 21 and 22 show examples of placing reinforcement for strengthening around bearing.

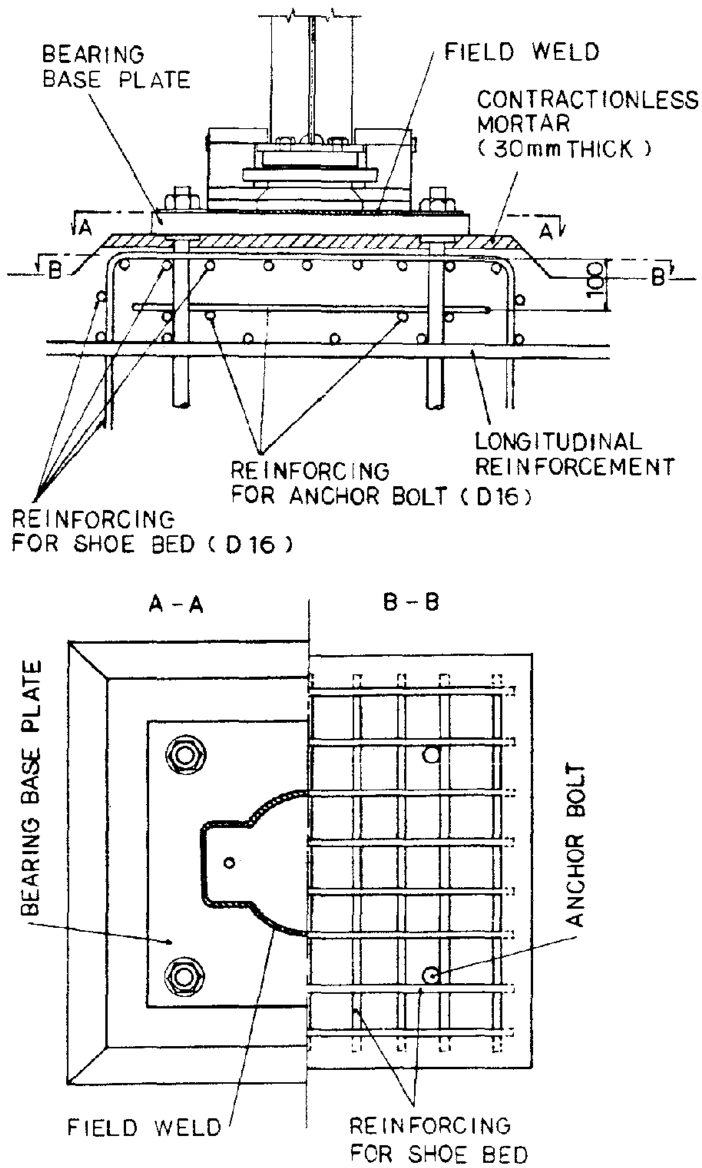


Fig. 22 Welding type anchorage of bearings

Further Considerations on Supports

In spite of the scruplous consideration for design of support, it seems to be still vulnerable in structures. Following two points concerning bearings are thought as its main reason.

(a) The dynamic effect of seismic force is not considered in design. The horizontal reaction due to earthquake must be different from design value.

(b) There are some gaps between blocks of bearing caused by seize margin and setting error. Concentration of larger force than calculated value may occur in one bearing.

Bearing may be broken one by one by these causes.

From thus point of view, the further considerations seem to be necessary. Bearing must be fixed in accurate position considering temperature at setting to minimize error at working.

There is an example to consider accurate setting of bearings. Fig. 23

shows the bearing for 4-span continuous prestressed concrete girder having three fixed supports. The bearing has a steel ring for fixing. The ring is free for horizontal movement until fixing by welding in site after finishing all of elastic displacement due to prestressing and a part of shrinkage and creep. In this method, unexpected horizontal force is hardly transmitted by bearing.

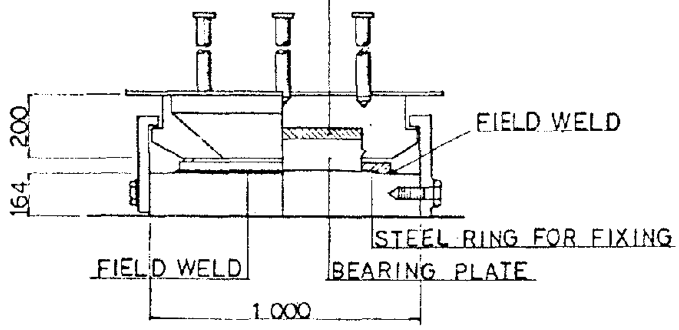


Fig.23 Special type bearing having steel ring for fixing

FALL-PROOF DEVICES

Fall of bridge by an earthquake is obviously serious to human lives and social matters, and in addition, it disturbes the rescue activities after earthquake. Three methods are considered to preventing superstructures from falling off their supports during earthquake as follows.

(a) Installation of adequate restrainer at bearings so as to restrict excessive movement.

(b) Enlargement of bridge seat.

(c) Connecting ajacent girders or girder with pier or abutment.

The first method was already described. If restrainers can not be installed in bearing like rubber bearing, other restrainer should be provided beside bearing. A box stopper is also such a kind of restrainer in a sense.

According to the Specifications for Earthquake Resistant Design of Highway Bridges [9], adopting the combination of method (a) and (b), or alternatively the combination of method (a) and (c) is required as countermeasure for preventing superstructures from falling.

Enlargement of Bridge Seat

Concerning adoption of wide width bearing seat, the Specifications for Earthquake Resistant Design of Highway Bridges [9] provides that the length (in cm as shown in Fig. 24) between the end of the bearing and the end of the substructure is not less than the following value.

$$20 + 0.5\ell \quad \text{for} \quad \ell < 100 \text{ m}$$
$$30 + 0.4\ell \quad \text{for} \quad \ell > 100 \text{ m}$$

in which ℓ is span length in meters. The minimum length between the ends of girders at suspended joint (as shown in Fig. 24) is also specified as 60 cm. These values were determined from the experience in the past and are considered sufficient to prevent collapse of girders and to prevent spalling of

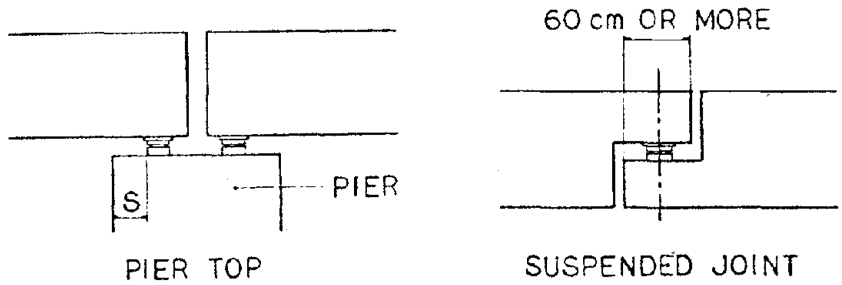


Fig. 24 Enlargement of bridge seats

concrete in the peripheral region of the substructure's crest during strong earthquake disturbances.

Connecting Divices

Even if the pier top is wide enough and the restrainer is installed at bearing, the installation of connecting devices is useful for assurance of safety against fall of bridge.

The examples of connecting devices are shown in Fig. 25, 26 and 27. It is easy to connect steel bridges if the neighboring web is in the same alignment. Fig. 25 shows general types of connecting devices for steel girder. Devices connect with girder's web each other, where one side of the webs is holed elliptically as a countermeasure of girder's horizontal movement due to temperature change and rotation due to live load. Fig. 27 shows connecting device for truss bridge. In the case of concrete bridge, prestressing bars and rubber pads sometimes are installed between neighboring end cross beam as shown in Fig. 26.

Because the current Specifications on Earthquake Resistant Design of Highway Bridges do not mentioned on design load of connecting devices, each corporation specifies it. According to the criteria [10] of Tokyo Metropolitan Expressway Public Corporation, the design load is following.

(a) If pier top is wide enough, a connecting device is designed against horizontal force of a 60% of dead load reaction at support. (60% means two times as much as horizontal seismic coefficient = 0.3.)

(b) If pier top is not wide enough, a connecting device is designed against load of a 100% of the dead load reaction at support. In this case, the design load is thought to be transmitted in vertical direction.

(c) The allowable stress of connecting devices is increased until 1.7 times as much as the allowable stress of ordinary times for (a) case. For (b) case, allowable stress is not increased, considering existance of impact forces at fall of bridge.

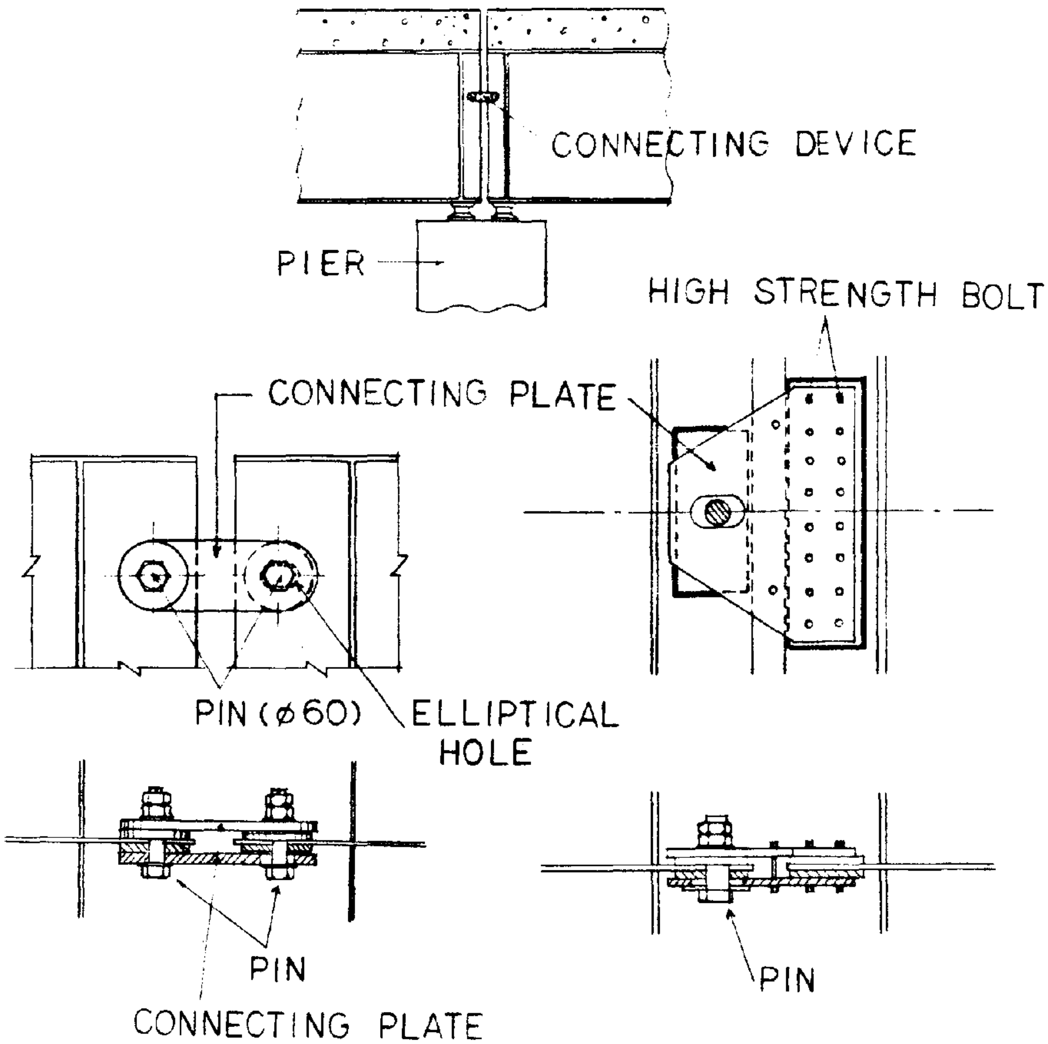


Fig. 25 Connecting divices in steel bridge

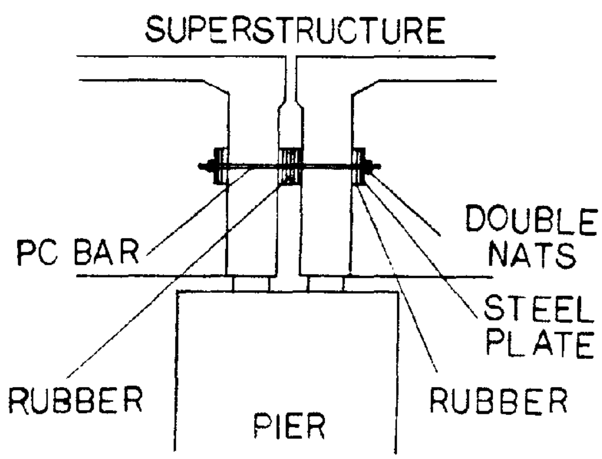


Fig. 26 Connecting devices in prestressed concrete bridge

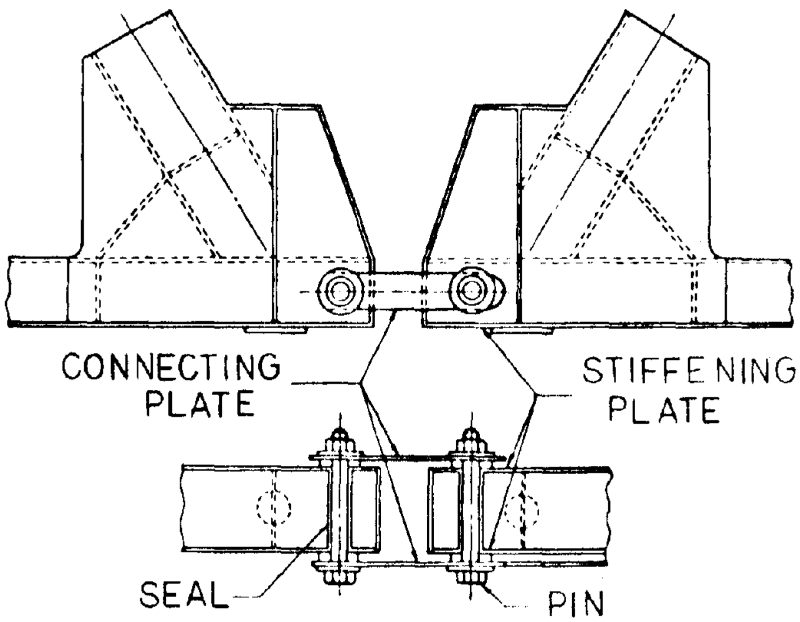


Fig. 27 Connecting device for truss bridge

There are such experiences in Japan that two bridges (named Tengu bridge and Isuzu bridge) installing connecting devices suffered very strong earthquake. Tengu bridge (128.5 M in bridge length) and Isuzu bridge (105 M in bridge length) located in southern part of the Izu peninsular suffered Izu-Hanto-Oki Earthquake whose maximum acceleration was presumed to be about 350 gals at the site where the bridges located, in May, 1974 [11].

Tengu bridge is 2-span continuous truss bridge having 40 M high pier with spread footing as shown in Fig. 28. The bridge had connecting devices connecting end of truss with abutment as shown in Fig. 29. The bridge was damaged limitedly around end supports such that failure of connecting bolts between truss and bearing, and spalling of concrete around anchorage part of connecting devices at abutment. Any other damage could not be found in truss,pier and abutments. So damage was very slight for strength of earthquake.

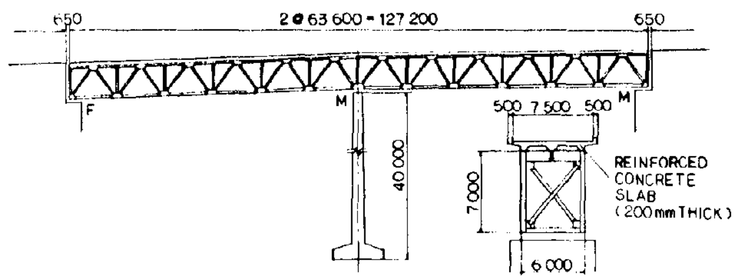


Fig. 28 Tengu-bridge

Isuzu bridge is 3-span continuous curved girder bridge being installed connecting devices at end of the bridge. The damage is almost same as the case of Tengu bridge, failure of connecting bolts between truss and bearing and spalling of concrete of anchorage part of connecting devices.

Judging from such damages of two bridges as the spalling of concrete at anchorage part of connecting devices, the connecting devices are thought to effectively work during earthquake.

Retrofitting of Existing Bridges

There are many existing bridges which have not been paid especially any consideration against fall of bridges. Of course, their pier's top is not wide and they do not have any connecting device. The problem against fall of bridges is more serious in these existing bridges. Therefore, the widening pier top and installing connecting devices have been progressing.

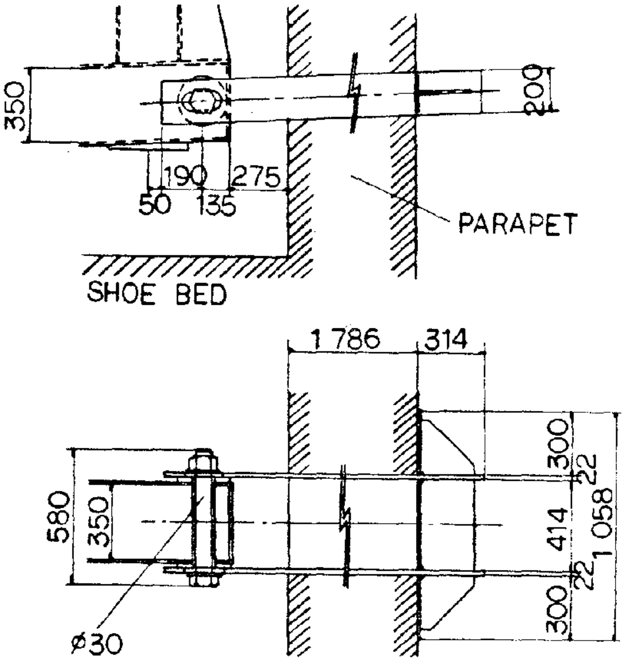


Fig.29 Connecting devices of Tengu-bridge

For steel girder, the connecting devices like that shown in Fig. 23 have been installed. For concrete bridge, other retrofit measures have been devised, because the installation of connecting devices is rather difficult. Fig. 30 shows an example of retrofit measure adopted for the national highway widening pier top with placing concrete. Fig. 31 shows newly developed fall-proof devices for existing concrete bridges [12] for the Tokyo Metropolitan Expressway. This device is designed to allow horizontal movement due to temperature change and rotation due to live load.

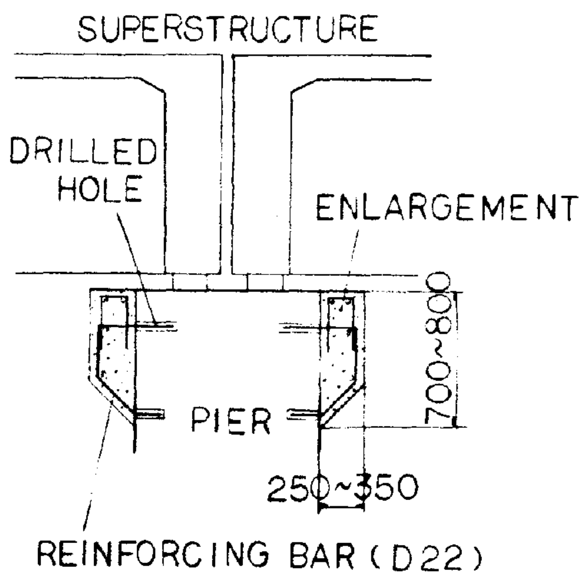


Fig. 30 Widening pier top

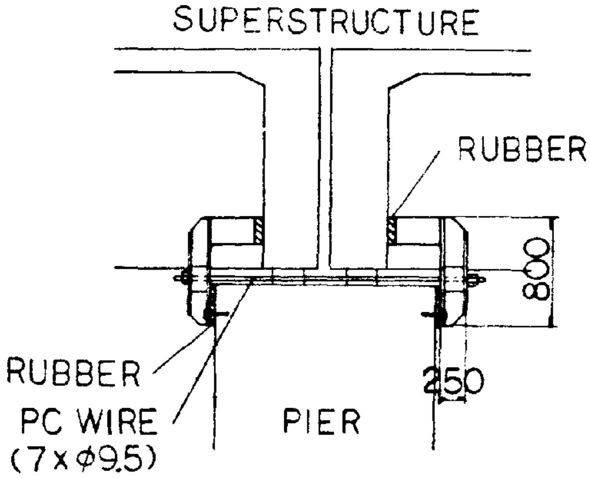


Fig. 31 Fall-proof deivices for existing concrete bridge

ABUTMENT

Abutments adopted for bridges in Japan are mostly wall type like retaining wall to withstand earth pressure during earthquake [13]. It is estimated by modifying earth pressure value at ordinary times based on Coulomb's earth pressure theory introducing such concept that increase of gravity at earthquake is estimated as inclination of the wall and the back fill. The angle of inclination θ_o is

$$\theta_o = \tan^{-1} \frac{k_h}{1 - k_v}$$

where, k_h = horizontal design seismic coefficient
k_v = vertical design seismic coefficient

Earth pressure coefficient during earthquake is estimated considering the angle.

Abutments should be stable including surrounding soil during earthquake. The specifications [9] requires that the failure of the ground layer during earthquake shall be checked for those abutments which are constructed in soft ground layers.

The failure of soft ground layers during earthquake frequently causes large displacement or tilting of the abutment resting in the layer. Adopting inclined piling is effective countermeasure to resist the sliding or large horizontal displacement of abutments. The very soft sub-soil is sometimes replaced with good soil or sand as another countermeasure.

Abutment is the junction between hard structure and soft banking. The safety for collapse between them is different themselves. The bank may settle and fail earlier during earthquake. In case where bank or back fill soil settles, placing approach slab shown in Fig. 32 is effective for serving the bridge for traffic just after shock. This is a reinforced concrete slab which is placed between back fill soil and pavement. Generally two slabs are placed side by side in longitudinal direction of bridge.

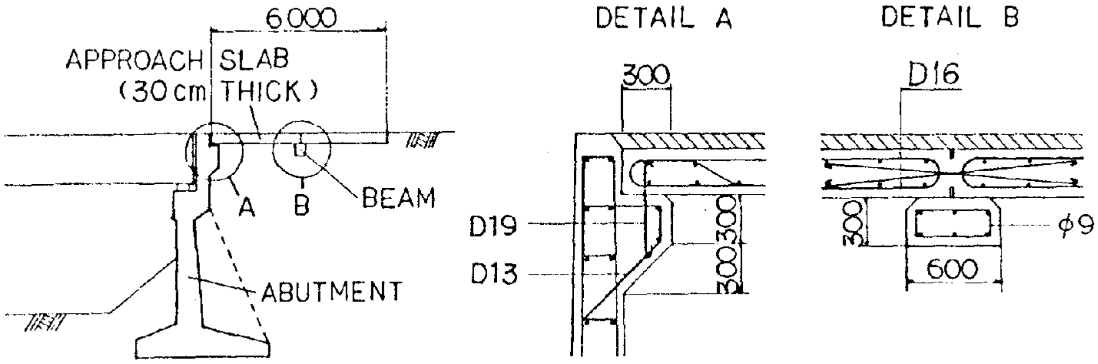


Fig. 32 Placing approach slab at abutment

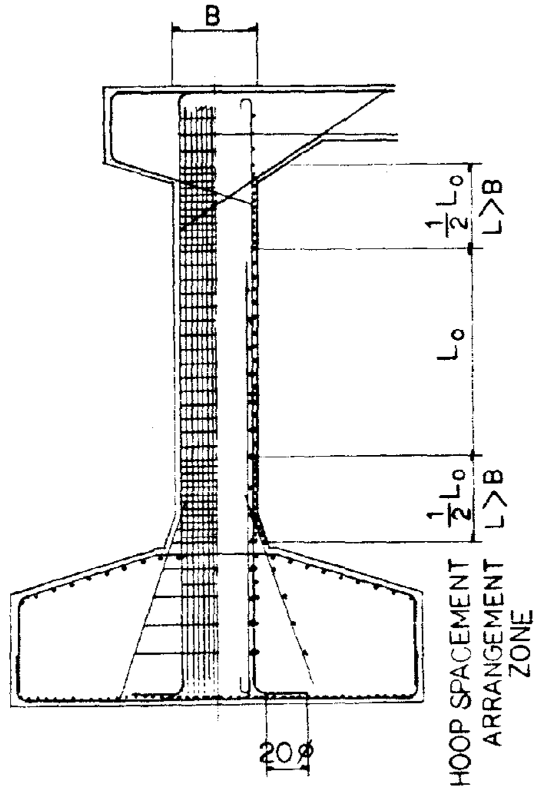
DUCTILITY OF REINFORCED CONCRETE COLUMNS

Increasing ductility of column is very important for the safety of bridge during earthquake. However, the behaviors of reinforced concrete column under dynamic seismic force do not seem to be clear. During earthquake, the fixed pier which carries horizontal seismic force, is subjected by large dynamic bending moment and shear force. It can be regarded as flexural member during earthquake. Therefore it is considered to be necessary to have long development length of longitudinal reinforcement in footing and to place rather much hoops at bottom of pier.

Placing Reinforcement in columns

An example of placing reinforcement in column used for the Tokyo Metropolitan Expressway is illustrated in Fig. 33. The volumes of hoops are twice times as much as necessary volume calculated at the top and bottom of a column. The diameter should be also balanced with the longitudinal reinforcement. The longitudinal reinforcement of column is developed until tail side of footing and bent or hooked there. Footing has the reinforcement at face side and at least one-half of the reinforcement volume at tail side is placed, even if it is needless on calculation.

Static model loading tests (as shown in Fig. 34) show that the existance of the reinforcement at face side footing is necessary for steady anchorage of the longitudinal reinforcement of column and the longitudinal reinforcement has to have L or U type hook to avoid slip out as shown in the test result diagram in Fig. 34 [14].



L_o ; REQUIRED SPACEMENT OF HOOPS

Fig. 33 An example of placing reinforcement in column

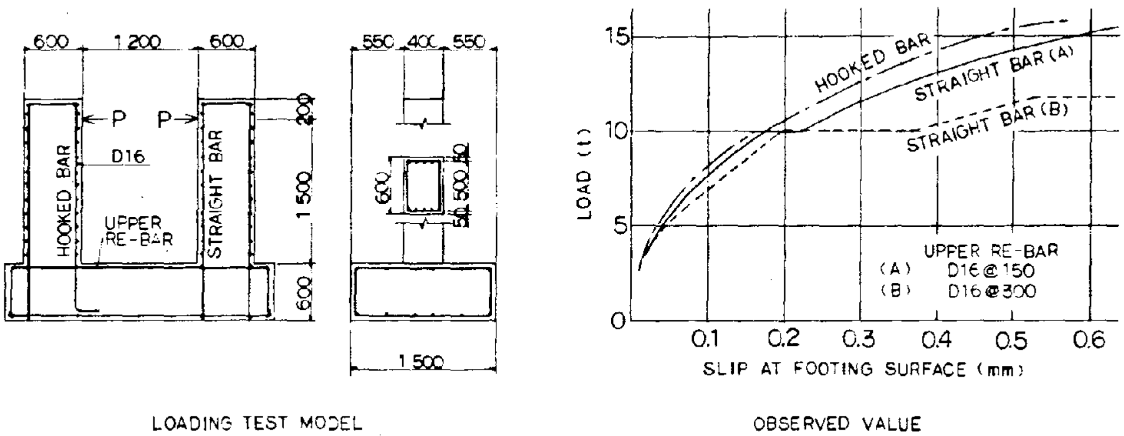


Fig. 34 Static loading test on anchorage of reinforcement in column

Cyclic Load Tests for Columns

Cyclic load tests for column members were done in order to investigate the relationship between size and spacing of placing hoops and ultimate strength or ductility of column [15].

Shapes and dimensions of models are shown in Fig. 35, and loading method is also shown in Fig. 36. The reinforcement were deformed bars with 19 mm in diameter for the longitudinal reinforcement and round bars with 9 mm and 6 mm for hoops.

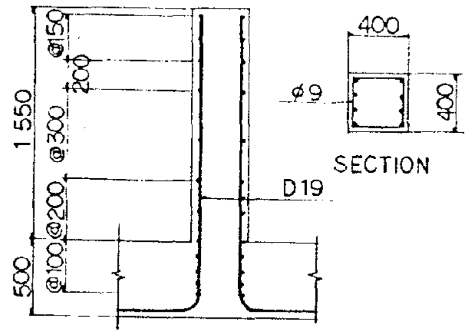


Fig.35 Column cyclic load test model

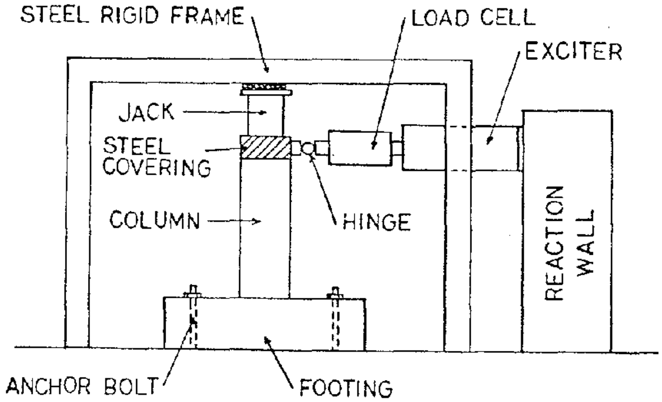


Fig.36 Loading system of column cyclic load test

Axial force was ooaded by oil-jack so that the compressive stress of concrete due to axial force reached almost 10 kg/cm^2 which was almost same as stresses in actual column. The horizontal force was cycled with 0.1 Hz and repeated 10 times for each loading stage. The loading was controlled by force before yielding and by displacement after yielding. The combinations of models, loadings and a part of test results are shown in table. In table, ductility factor means displacement ductility factor μ estimated as a ratio observed displacement to calculated displacement at first yield.
The load-displacement curve is shown in Fig. 37. The sketches of some models at yield stage and ultimate situation are shown in Fig. 38. From the test results, the following discussions were done.

(a) Existance of axial force is useful for the increasing maximum strength, but the degradation of strength will occur rather suddenly at the stage where displacement becomes large. Plastic area is reduced until almost one-half of flexural member by the existance of axial force.

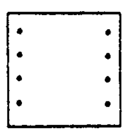
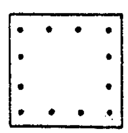
(b) Ductility factor of models subjected to cyclic load is sharply decreased in comparision with model subjected to static load. The column ductility was enhenced by increase of hoop volume. Placing small size hoops, however, does not make with good results, even if hoop spacing is short. Hoops may cut off caused by buckling of the longitudinal reinforcement, if they are small size in diameter.

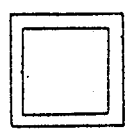
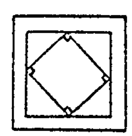
Table on Model Type, Loading, Ultimate Strength and Ductility Factor

Model No.	Longitudinal Reinforcement		Hoop				Loading Method	Existance of Axial Load	Observed Ultimate Strength	Ductility Factor (μ)	
	Dia. Ratio	Placing manner	Dia. Ratio	Spacing	Figure	Splice				*5 $\mu = \frac{\delta u(\frac{1}{2})}{\delta y}$	*6 $\mu = \frac{\delta u}{\delta y}$
1-1	D-19 *1 0.82%	*3 1	ϕ-9 *2 0.16%	20 cm	*4 1	Bend	Static	No Exist.	10.7^t	10.7	7.0
2-1	"	"	"	"	"	"	Cyclic	"	10.2	5.1	3.1
2-2	"	"	"	"	"	Weld	"	"	10.4	8.1	3.3
2-3	"	"	"	"	"	Bend	"	Exist.	12.0	4.0	3.7
2-4	"	"	"	"	"	Weld	"	"	13.4	3.8	3.3
2-5	"	"	"	"	2	Bend	"	"	13.6	4.0	3.7
3-1	"	2	"	"	1	Weld	"	"	16.5	4.8	3.9
3-2	"	"	ϕ-6 0.16%	8.75cm	"	"	"	"	17.5	3.5	2.7
4-1	"	1	ϕ-9 0.32%	10 cm	"	Bend	"	No Exist.	11.2	6.0	2.9
4-2	"	"	"	"	"	"	"	Exist.	12.9	6.9	4.0

*1 D-19 : Deformed bar 19 mm in diameter

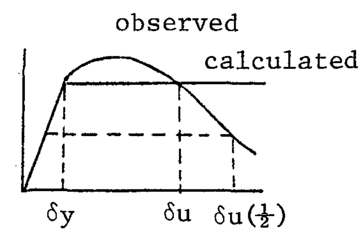
*2 ϕ-9 : Round bar 9 mm in diameter

*3 1  2 

*4 1  2 

*5 $= \frac{\delta u(\frac{1}{2})}{\delta y}$

*6 $= \frac{\delta u}{\delta y}$



Ductility factors of models under cyclic load range from 3 to 4, whose value is small in comparison with the value of models under static load from this experiment.

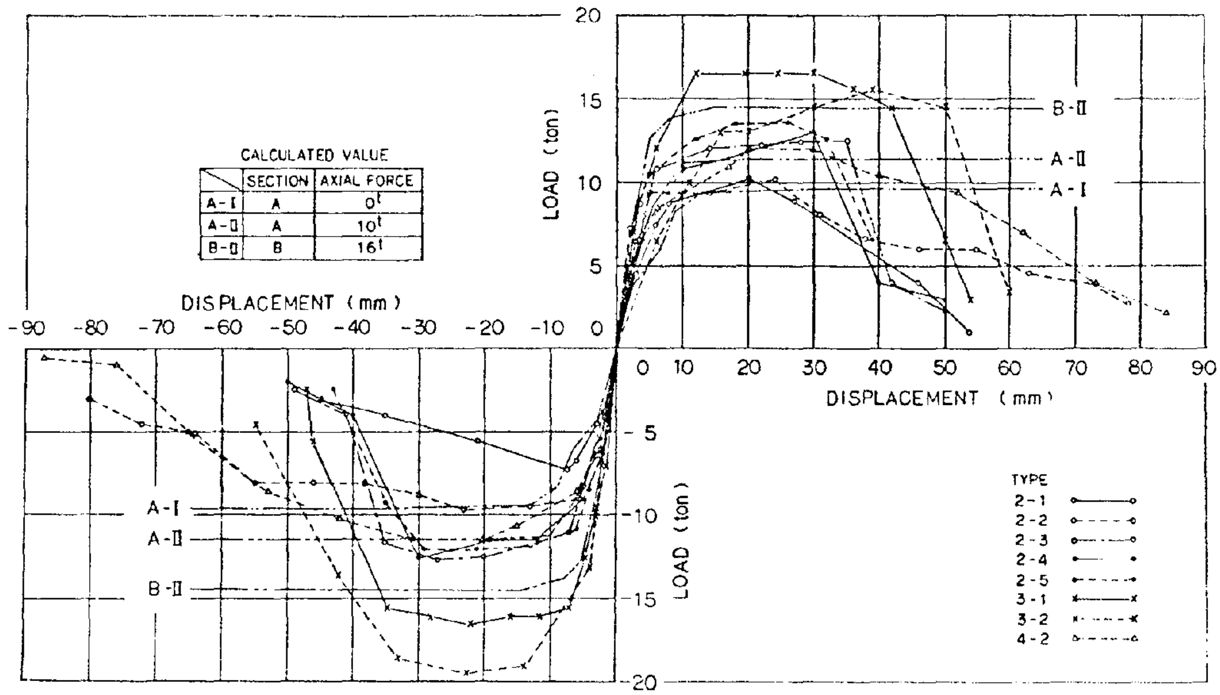


Fig. 37 Load - displacement curve

This is one study on behavior of the reinforced concrete column under cyclic load. It can be emphasized from the results that rather large size hoops are needed at the bottom part of piers. However, the test is not enough to deduce the general conclusion on placing reinforcement. Generally, this kind of experimental study seems to be still insufficient. The research on columns under dynamic load should be done more.

CONCLUSION

The considerations on special seismic design are summerized. They are based upon many experiences of earthquake damages in the past. Ordinary scale bridges may be in almost safe against earthquake by applying the specifications to design and paying the considerations beforehand described in design, although they may have local miner damages. However, there remain many problems to be studied. Further studies are needed for more rational seismic design. Moreover, it is necessary for long span and large scale bridge to examine new ideas of seismic design.

From the considerations reported herein, general conclusions may be deduced as follows.

(1) It is important to choose a system carrying horizontal seismic

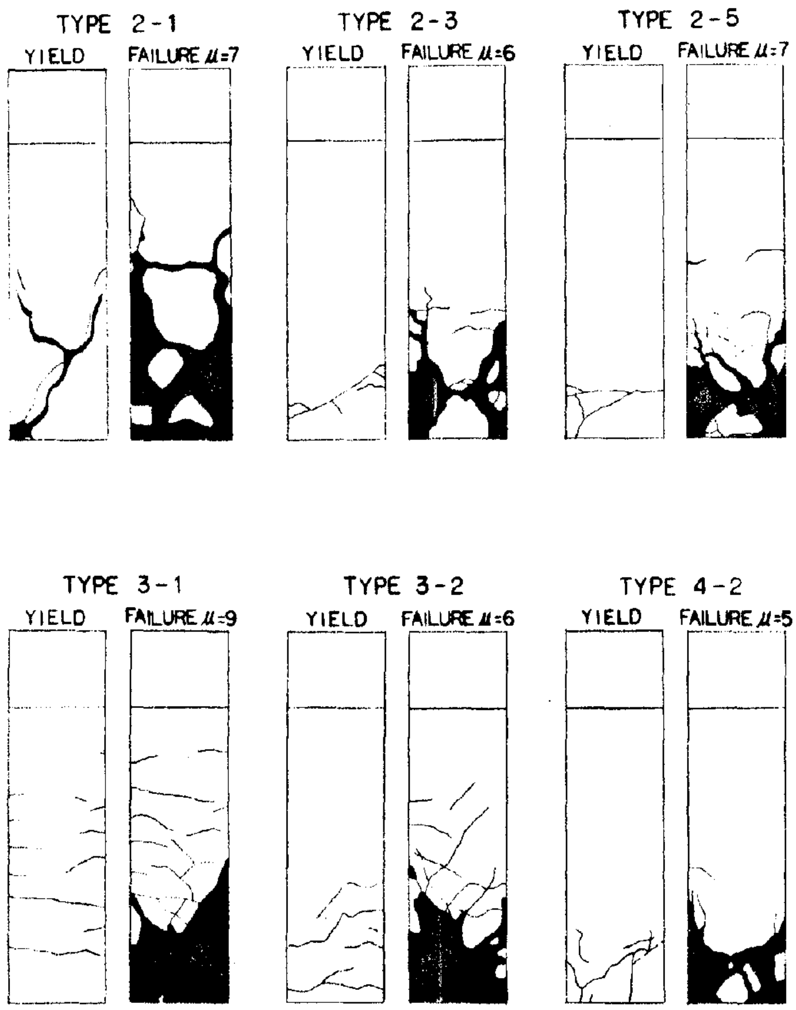


Fig. 38 Sketch of cruck development

(CONCLUSION cont'd)

force which is adequate to not only earthquake but also temperature change, serviceability and etc. at ordinary times. In multi-span continuous bridge, the decision of the system should be examined more carefully taking the use of the damping devices into account. As new idea of the system carry seismic force, it is suggested to consider the behavior of piers cooperating with foundations.

(2) The damping devices are useful for distribution of horizontal seismic force into substructures. There are several types of damping devices. As to the application of these devices for bridges, systematical study should be required.

(3) The bearing supports are designed such that they ristrict girder's excessive movement at earthquake, and connect steadily super-structure with substructure being strengthen anchorage part by

the reinforcement. However, they seem to be still valnerable part in structure judging from damages at earthquake. Rational design and construction should be more studied balancing other parts of bridge member.

(4) Considerations for preventing superstructure from falling are paid by the means of such three methods as enlargement of bearing seat, restrainer at bearing and connecting devices. They are thought to be very useful for their purpose. The considerations should be also paid for existing bridge by installing some fall-proof devices.

(5) At the point of view of stability and serviceability of bridge, the considerations for abutment are very important. In this case, it is necessary to pay cares to surrounding soil conditions.

(6) For bridge piers, the consideration on placing reinforcement are necessary. Development length of the longitudinal reinforcement and size and spacing of hoops should be paid attention. However, the studies on ductility of column under horizontal seismic load seems not to be done enough. More studies are needed on this point.

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ANALYSIS AND DESIGN OF BRIDGES INCORPORATING MECHANICAL ENERGY DISSIPATING DEVICES FOR EARTHQUAKE RESISTANCE

by

R.W.G. Blakeley
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ABSTRACT

Recent developments are presented in an alternative to the current seismic resistant ductile design approach for bridges and other structures. The alternative method is based on two elements; firstly, the structure is supported on flexible mountings, usually elastomeric rubber bearings, to isolate it from the predominant earthquake ground motion frequencies, and secondly, extra damping is provided to keep deflections within acceptable limits. Details of several types of practical mechanical devices developed to provide the extra damping through hysteretic energy dissipation are described. These include devices relying on cyclic flexural or torsional yielding of steel or extrusion or shear of lead. A number of applications of the devices to bridges are described including results of dynamic computer analyses of the time-history of response of the structures with and without energy dissipators, and illustrations of installation details.

Results of dynamic analysis studies to investigate the sensitivity of seismic response to principal parameters for bridges incorporating energy dissipators are described and design charts to determine forces and displacement for various earthquake excitations are presented. Advantages of the approach relative to the conventional ductile design procedure are listed, in particular potential for construction economies and greater protection against earthquake induced damage. A philosophy of design of structures using the approach is proposed.

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INTRODUCTION

Background

It is well known that, when a structure responds to earthquake induced ground motions, very high forces may be generated if the structure is required to remain elastic. For over a decade, common earthquake resistant design practice has been based on limitation of induced forces by yielding of selected structural members, which are designed to deform in a ductile manner. The associated dissipation of energy in the yielding members reduces the seismic response of the structure but entails some form of damage to the structure. This may mean costly repairs and difficulties of restoration of permanent set deflections. The lateral design load coefficients specified by the codes of most countries are such that the threshold of yielding corresponds to amplification of a relatively moderate earthquake, such as may be expected to occur two or three times during the life of a structure. Thus, for any structure so designed there is a high probability of some form of damage during its life. Other disadvantages of the conventional ductile design approach are that design office procedures may be complicated, the required reinforcing steel details may be difficult to place and may be expensive and large deformations may need to be accommodated.

Principles of Base Isolation

In recent years a number of methods of isolating a structure from the effects of earthquake ground shaking have been proposed, such as resting the structure on rollers, floating it in water, or providing a flexible first storey, but have not met general acceptance because of practical deficiencies, in particular the expected excessive displacements under wind or earthquake. However, a system overcoming these deficiencies has been made possible by the development of practical mechanical devices which act as hysteretic dampers. The system of "base isolation" of structures recently developed has two basic elements:

(a) the structure is supported on a flexible mounting so that the period of vibration of the combined structure-mounting system is sufficiently long so that the structure is isolated from the greatest disturbing motions at the likely predominant earthquake ground motion frequencies. The flexible mounting is usually provided by supporting the superstructure on laminated rubber bearings.

(b) sufficient extra damping is introduced into the system to reduce resonance effects and to keep deflections within acceptable limits. The mechanical devices developed by the New Zealand Department of Scientific and Industrial Research serve effectively to provide this damping through hysteretic energy dissipation.

Base isolation may be introduced into an otherwise monolithic bridge structure by supporting the superstructure on elastomeric bearings. Many bridge structures have traditionally been supported on elastomeric bearings and hence the first element of the base isolation system is already present. There is potential however, for varying the degree of isolation, for example by using thicker bearings. Addition of the second element of the base isolation system, the energy dissipators, has the potential advantage of reducing the displacements through increased stiffness and damping, although it may increase the accelerations. Dissipators may also be beneficial in terms of their force-deflection characteristics. At small forces due to wind, traffic braking or minor earthquakes, their elastic stiffness is high relative to that of rubber bearings and deflections are minimised. As earthquake excitation increases, the device "softens" and tends to act as a mechanical fuse or isolator, protecting the structure from damage and itself being readily replaceable, if necessary, after a severe earthquake.

An alternative to provision of extra damping by mechanical energy dissipating devices may be the use of "high loss" rubber bearings. Shaking table tests were conducted at University of California, Berkeley[1] on a three-storey steel frame, mounted on rubber bearings of the same construction but with two vulcanisates having different damping characteristics, namely 3% and 10% of critical damping. The effect of increased internal damping in the rubber bearings was to provide much faster attenuation in the displacement response and a decrease in the maximum building acceleration by about 20% under the El Centro 1940 N-S excitation. The effect of addition of a mechanical energy dissipating device was to increase accelerations but reduce displacements by approximately 40% under the same shaking.

Importance of Earthquake Characteristics

The isolation benefits described above are dependent on the characteristics of the earthquake. Strong motion accelerograms recorded in areas of hard rock or strong alluvial soil have typically exhibited a predominant frequency of about 3 Hz. This characteristic is demonstrated by the El Centro 1940 N-S earthquake as illustrated in Fig 1. For such an earthquake, introduction of flexible mountings to an otherwise stiff system in this period range will dramatically reduce the acceleration response. This could be achieved, for example, by mounting a bridge superstructure on elastomeric bearings at the top of the piers instead of having the superstructure built in to the piers. However, it must be recognised that soft soils may alter the frequency

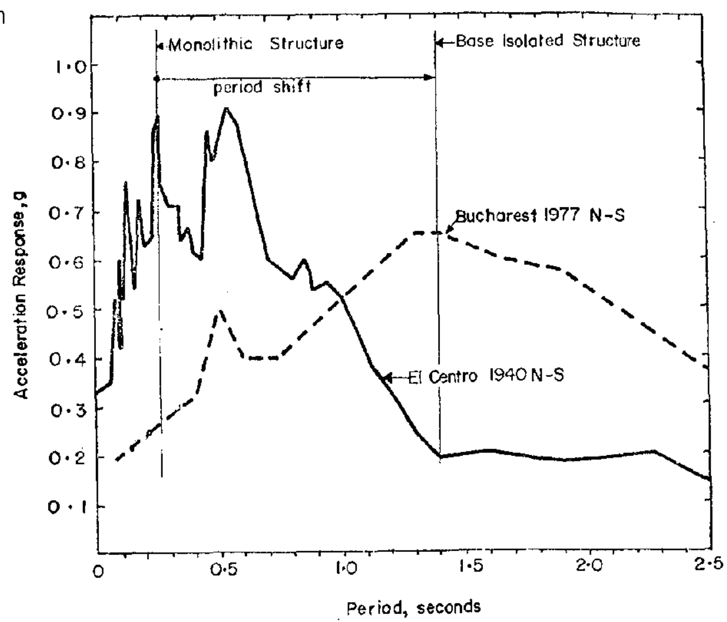


FIG 1 : EARTHQUAKE RESPONSE SPECTRA

response of an earthquake so that more of the energy is transmitted at low frequencies. An example of an earthquake record with predominant low frequencies is the Bucharest, 1977, record shown in Fig 1. For such an earthquake, introduction of flexible mountings will tend to increase rather than decrease response. This effect is common to all bridges mounted on elastomeric bearings with or without energy dissipators. However, a bridge structure incorporating energy dissipating devices will generally be better off because of the extra damping introduced.

DEVELOPMENT AND TESTING OF MECHANICAL ENERGY DISSIPATING DEVICES AND BEARINGS

Energy Dissipating Devices

Details - In recent years a number of practical mechanical energy dissipating devices have been developed by the New Zealand Department of Scientific and Industrial Research. Detailed information is given elsewhere[2,3,4].Devices developed to date are illustrated in Fig 2. These include devices which will act only along one axis and others which are omnidirectional in action. Details are as follows:

Torsional Beam - The solid rectangular section beam is of mild steel and is typically of length 500 mm to 1 m (1'8" to 3'3"). Relative movements between the loading arms at each end of the beam in one direction and the centre of the beam in the other direction induce torsion in the beam. Energy is dissipated by torsional plastic cyclic deformation of the beam.

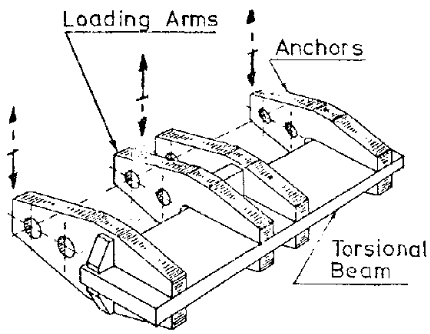
Lead Extrusion - Relative movements between the piston and the cylinder force the enclosed lead through the orifice in the cylinder. Energy is dissipated during cyclic deformations by extrusion of the lead back and forth through the orifice. When the lead is extruded it recrystallises immediately, thereby regaining its original mechanical properties.

Flexural Plate - The tapered plate cantilever was first developed for application between the superstructure of a bridge and the piers or abutments. The mild steel plate is tapered to spread the zone of plasticity and has a buttressed base detail with welding kept well away from the highly strained plate. Energy is dissipated by cyclic flexural yielding of the plate.

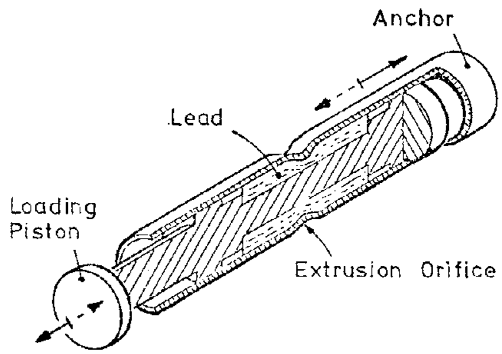
Uniaxial Flexural Beam - The device comprises a uniform mild steel beam and cast steel loading arms free to pivot at the anchor plates. Relative movements at the anchor plates induce uniform moments in the beam. Energy is dissipated by cyclic flexural yielding of the beam.

Omnidirectional Flexural Beam - The short vertical mild steel cantilever is plastically deformed primarily in flexure and will operate for relative movements in any horizontal direction. In some situations two such devices may be used, orientated head-to-head, to comprise a double vertical cantilever-action device.

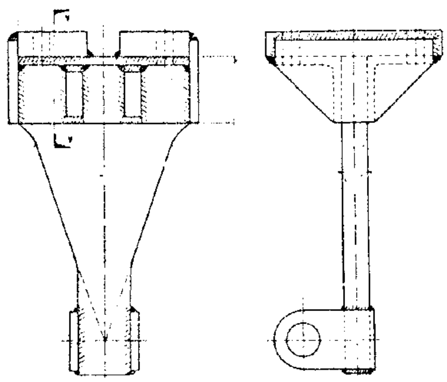
Lead-Rubber - This device comprises an ordinary steel-reinforced elastomeric rubber bearing with a cylindrical lead insert press fitted,or poured molten, in the centre. It may be used as a mounting for buildings or in place of a conventional elastomeric bearing in bridges. When the device is deformed in shear under earthquake loading, the lead is hot worked so that, during its



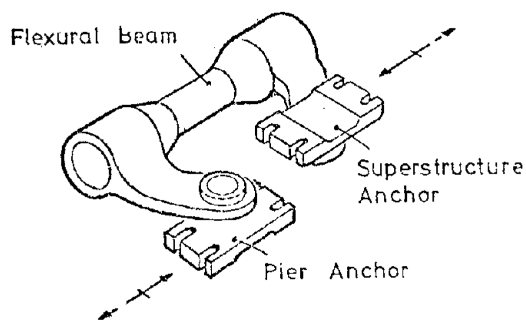
Torsional Beam Device



Lead Extrusion Device

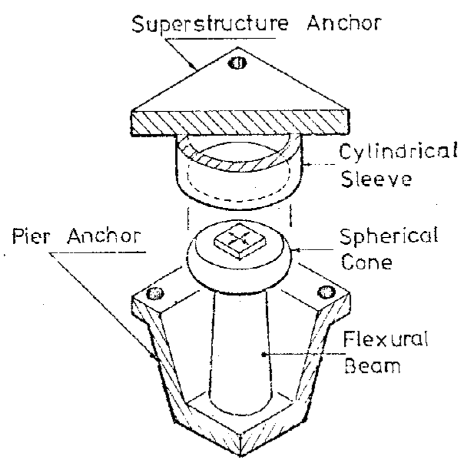


Flexural Plate Device

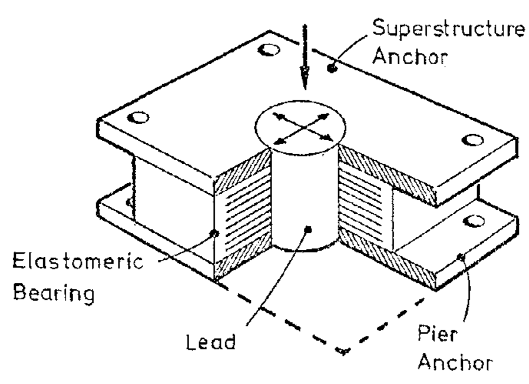


Flexural Beam Device

(a) Uniaxial Action



Flexural Beam Device



Lead—Rubber Device

(b) Omnidirectional Action

FIG 2 : SCHEMATIC DRAWINGS OF MECHANICAL ENERGY DISSIPATORS deformation, energy is dissipated. The lead recovers most of its mechanical properties almost immediately. A particular advantage for long-span bridges is that, at loading rates corresponding to temperature induced lengthening and shortening effects, it will creep without transmitting the magnitude of force corresponding to earthquake induced load rates. Also, it is very simple to install.

Materials - For steel energy dissipators mild steels have been used throughout, either black to British Standard 4360/43A or bright to Australian Standard CS 10308 or CS 10208, having essentially the same chemical composition. Very low carbon steels, pure aluminium, and spheroidal graphite iron have been tried for dissipators in bending and torsion, but with inferior results compared with mild steel. In order to achieve a good fatigue life, welding is kept away from the areas of the device to be highly strained, for example by flaring above the base.

Stress relieving after fabrication is usually carried out, the treatment being for 5 hours at 620°C. Strain age embrittlement is not considered likely to be a problem, on the basis of testing to date.

For the lead energy dissipators, pure lead is used. The lead-rubber device also comprises a conventional steel reinforced laminated rubber bearing.

Testing - The devices have all been tested extensively at earthquake-like frequencies and displacement amplitudes and show stable hysteretic characteristics with lifetimes within the range of 100 to 1,000 cycles at anticipated peak displacement, or equivalent to 10 to 100 major earthquakes.

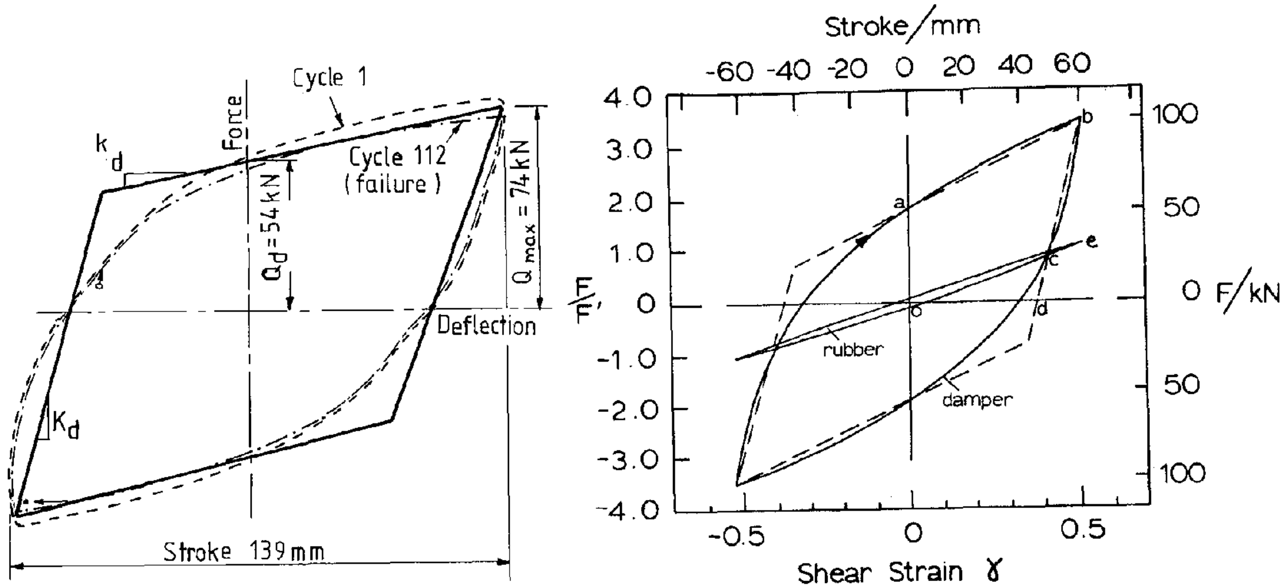
Although the energy absorbed by yielding of mild steel or hot working of lead is eventually dissipated as heat, the temperature rise in the devices is not significant. During testing the temperature rise was only 10 to 15°C for the simulated effects of a major earthquake. A typical force-displacement hysteresis loop for a cantilever flexural beam dissipator is illustrated in Fig 3(a). For analysis purposes, this may be idealised as a bilinear hysteresis loop as shown. For a peak-to-peak strain range of 6%, the stiffness parameters may be represented by:

$$K_d/Q_d = 85\ m^{-1}(2.16in^{-1}) \qquad k_d/Q_d = 5\ m^{-1}(0.127in^{-1})$$

where K_d = initial stiffness on force-displacement idealisation
k_d = post-yield stiffness on force-displacement idealisation
Q_d = force in dissipator at zero displacement ordinate

The corresponding stiffness parameters for the "torsional beam" device are approximately 20% higher than those given above.

The "lead extrusion" device acts as a near Coulomb damper with rectangular hysteresis loops. Typical hysteresis loops for the "lead-shear" device are shown in Fig 3(b). The post-yield stiffness of the device is somewhat greater than the stiffness of the bearing alone, indicating that as well as plastic shear of the lead there are elastic components of the lead plug along its length. The ratio of post yield stiffness of the device to stiffness of the bearing alone is dependent on the size of the lead plug. For the common case of Q_d equal to 5% of the vertical load, the ratio of stiffness ab to oe in Fig 3(b) may be 1.3 to 1.4.



(a) Steel Cantilever (b) Lead-Rubber Device

FIG 3 : FORCE-DISPLACEMENT HYSTERESIS LOOPS
(4.45 kN = 1 kip, 25.4 mm = 1 inch)

Maintenance - In general, all structures incorporating mechanical energy dissipating devices including lead-rubber bearings must be detailed to allow ready access for inspection and replacement, should that be necessary as a result of in-service performance over the years or overstrain during an earthquake. As technical developments permit, consideration could be given to the ideal approach of building in some earthquake protection which can thereafter be forgotten.

The units should also be installed in a dry position under the structure. Hot dip galvanising of steel units is not at present recommended. Instead, it is suggested that steel devices be painted in the normal way or protected with a coat of anti-corrosive grease. The maintenance requirements would then be no more than those for ordinary structural steel components of a bridge.

Cost - The mechanical energy dissipating devices developed by New Zealand Department of Scientific and Industrial Research are patented through Development Finance Corporation of New Zealand, and are manufactured and marketed by a firm or firms selected by the Corporation. Their cost includes any royalty payable to the Corporation at the time of sale. The costs of devices installed in bridge structures to date are given in the next section. At the time of writing, and since the costs were determined for the structures described, the cost structure is under review and costs are likely to be significantly reduced.

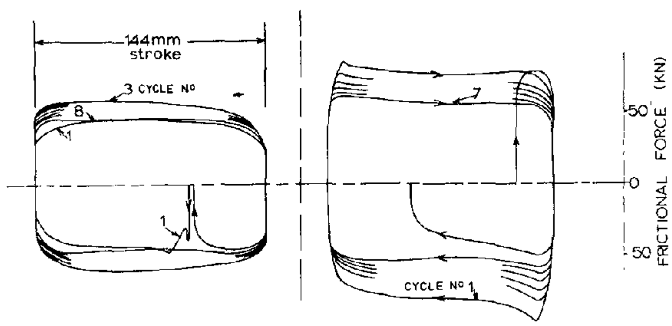
Choice of Dissipator Type - The choice of a particular device depends on cost, technical merit and suitability for the required application. Steel devices have the advantage of toughness and reliable performance, but further testing is required to conclusively discount strain age embrittlement. The lead-extrusion device offers the advantage of a long stroke and true load-limiting capability by virtue of its negligible post-yield stiffness, but has the possible disadvantage of uncertain maintenance requirements. The lead-rubber device is a neat package combining both damping and bearing functions, but is still in its early stages of development.

Other factors which may influence the choice are the installation details, which may be a significant cost item. In this regard the lead-rubber device has advantages of simplicity of installation. Also, it can "creep" to accommodate lengthening and shortening effects of a bridge superstructure, for example due to temperature, whereas for a steel device appropriate clearances will usually need to be provided.

Bearings

Elastomeric - Laminated rubber bridge bearings were subjected to simulated dynamic earthquake loading as described by Tyler [5]. The measured dynamic shear stiffness of one pair of 256 x 356 x 140 mm (10in x 14in x 5½in) bearings at 50% strain in rubber was slightly greater than the shear stiffness nominated by the manufacturer, by margins of 5-18% for varying values of temperature, speed and normal jack pressure. Measured damping as a percentage of critical varied between 3 and 5% from warm to cold at slow speed and double these values at fast speed.

Sliding - Dynamic tests were conducted to determine the coefficients of friction of PTFE sliding bearings at earthquake-like load rates [6]. Measured force-displacement loops are shown in Fig 4, exhibiting typical Coulomb damping characteristics. The tests on pure PTFE sliding layers showed that, for the range of bearing pressures commonly used in bridge construction, namely 15-25 MPa (2140-3510 psi), and for velocities which would occur during severe earthquakes, the corresponding range of maximum friction values at 0°C was 17 to 13%. Tests on a PTFE bearing lubricated with silicone grease showed a coefficient of friction less than 2%. Other tests with cement dust sprinkled over the grease, and also with cement dust sprinkled over dry PTFE layers, in order to simulate extremely dirty conditions on a bridge site, gave friction values up to 40%, which shows the importance of keeping bearings dust-free during service. It is evident from these tests that PTFE sliding bearings can act as a form of energy dissipating device, although the frictional strength may be unpredictable because of inservice conditions of the sliding surfaces.



(a) Slow (b) Fast

FIG 4 : FORCE-DISPLACEMENT LOOPS FOR TESTS ON PURE PTFE SLIDING LAYERS FOR PRESSURE OF 23 MPa (3290 psi) (4.45 kN = 1 kip, 25.4 mm = 1 inch)

APPLICATION OF MECHANICAL ENERGY DISSIPATING DEVICES TO BRIDGES

Introduction

The application of energy dissipating devices to bridges is still in its developmental stages. Design to date has been on an individual basis for each structure and has generally been based on the results of dynamic computer analyses. Following is a description of five bridges incorporating or designed to incorporate mechanical energy dissipating devices. Also described is a detailed consideration for incorporation of devices in a bridge, for which a decision was made against using the devices. Other applications are described in ref 2.

Overpasses on Wellington Urban Motorway

The Bolton Street and Aurora Terrace bridges crossing the Wellington Urban Motorway have been constructed mounted on PTFE glide bearings and incorporating in each case six lead extrusion dampers resisting longitudinal earthquake response. The application is illustrated in Fig 5. The dissipators were required to resist the forces arising during emergency braking of downward moving vehicles, yet yield to limit the forces induced in the abutment from longitudinal earthquake response. There is the possible disadvantage in this application that neither the dampers nor the glide bearings will provide a centring force during seismic response. However, if required after an earthquake the bridges could be jacked back to a central position. Each extrusion damper used is of total length 1.5 metres (4ft 11ins), has a peak-to-peak stroke of 400 mm (16in) and a yield force of 140 kN (31 kips). The cost per damper was approximately US$2,000.

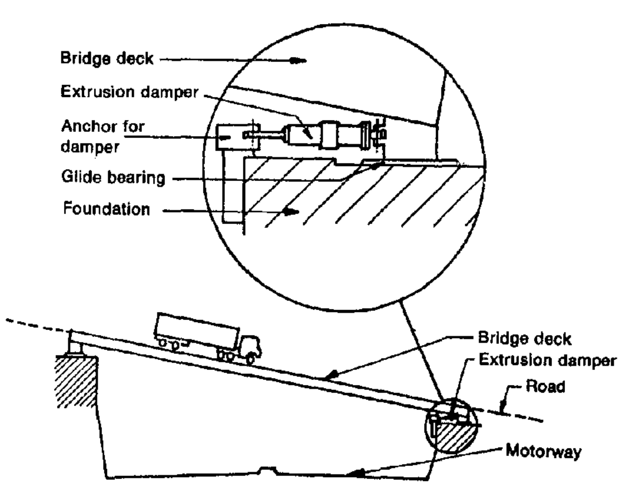


FIG 5 : LEAD EXTRUSION DISSIPATORS IN WELLINGTON URBAN MOTORWAY OVERPASSES

South Rangitikei Rail Bridge

The South Rangitikei Rail Bridge illustrated in Fig 6 is under construction at the time of writing. It is a six-span prestressed concrete box-girder bridge on tall reinforced concrete piers. The design concept is that under earthquake loading transverse to the axis of the bridge the piers will "step", that is rock with each leg alternately lifting from the foundations. The earthquake induced forces are then much less than they would have been if the legs had been rigidly fixed to the foundations. The lateral displacements are limited to acceptable values by energy dissipation in devices of the "torsional beam" type acting between the base of each leg and the foundation. Details of the analysis of this type of structure are given by Beck and Skinner [7].

The central shaft is anchored into the plinth above the foundation. Relative uplift movement between the stepping pier leg and the foundation induces torsion in the energy dissipating devices, the outer arms of which are anchored to the foot of the pier and the inner arms to the foundation. The devices do not act under gravity loads, which are transmitted to the foundations through elastomeric bearings. They will act under uplift due to wind forces but at that level of force the devices are in their high initial stiffness range and

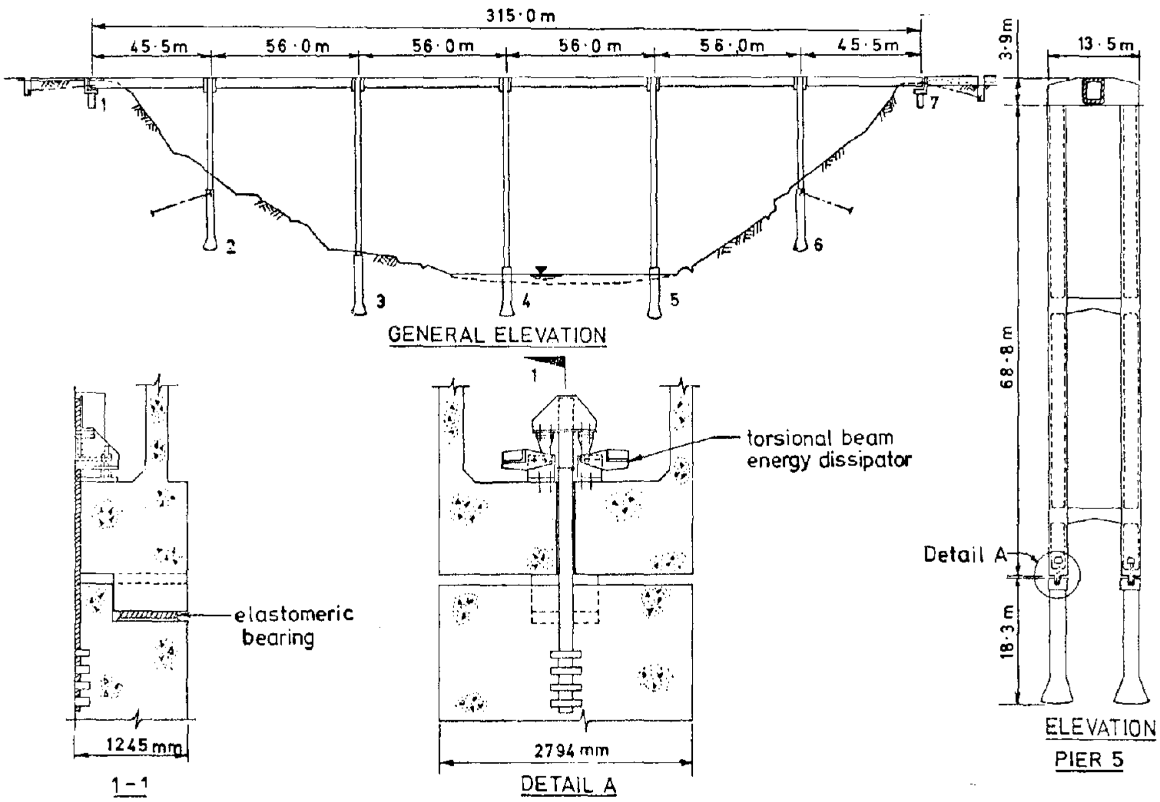


FIG 6 : TORSIONAL BEAM DISSIPATORS IN SOUTH RANGITIKEI RAIL BRIDGE
(25.4 mm = 1in, 1 m = 3ft 3ins)

deflections are minimised. Twentyfour energy dissipating devices have been installed, all with strength at the zero displacement ordinate of 400 kN (90 kips) and a design total stroke of 80 mm (3.1ins). A wind stop is provided in the pier base detail at 127 mm (5 ins).

The cost of the structure with the "stepping" details is comparable to that without, but the benefits lie in concentrating energy dissipation in the devices, thus protecting the structure from earthquake induced damage, and in limiting the axial forces induced in the piers.

King Edward Street Overpass, Dunedin

The King Edward Street Overpass is a three-span twin overpass on the Dunedin-Milton motorway and is under construction at the time of writing. As shown in Fig 7, each structure has a prestressed concrete hollow cell superstructure supported on elastomeric bearings with reinforced concrete piers and abutments. The abutments are free-standing with an independent back wall. Each structure incorporates four energy dissipating devices of the cantilever taper plate type. Those at the tops of the piers act in the transverse direction of the superstructure and those at the abutment act longitudinally. Uni-axial action devices rather than onmidirectional devices were chosen in this case to most effectively use the relative substructure element stiffnessess. The geometry of the slab piers was governed by considerations other than seismic design. They provide a desirably stiff support for transverse action dissipators. In the longitudinal direction the abutments are stiffer than the piers and form a logical location for the longitudinal action dissipators.

Details of the installation of devices on the piers are shown in Fig 7. The orientation with the base fixed to the superstructure was chosen as being most convenient for detailing and access requirements. The embedded channel within the pier is hot dip galvanised. Consideration was given to possible "binding" effects between the bearing cylinder of the plate and the channel, under the real earthquake conditions of response at a variety of inclinations to the principal axes. The effects were not considered to be serious. Even if binding does occur, the abutment and its dissipator are still stiffer than the pier longitudinally. To induce yielding of the pier longitudinally, the surfaces between dissipator and pier would have to generate a coefficient of friction of 0.45 under transverse forces corresponding to an El Centro 1940 N-S earthquake. The consequences of binding would be yielding of the pier at a somewhat lower earthquake intensity than that designed for.

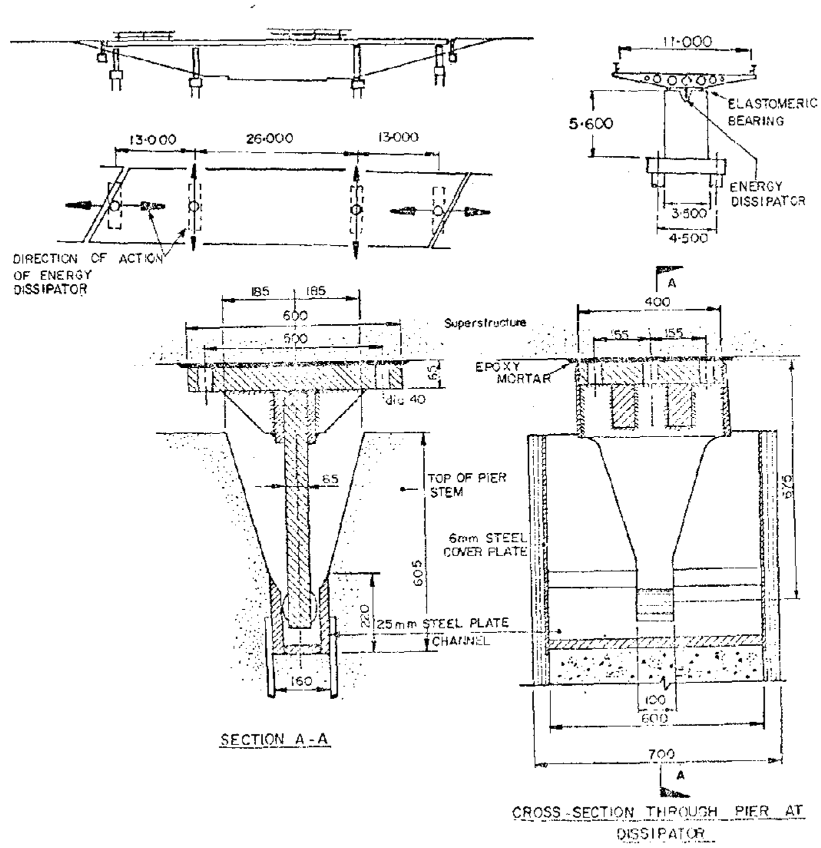
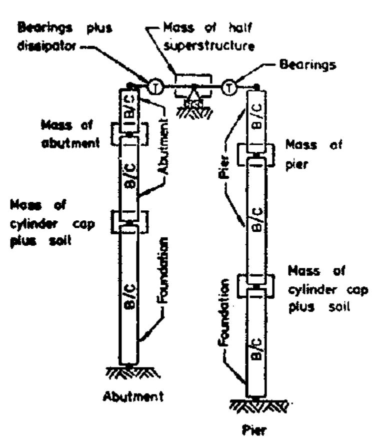


FIG 7 : CANTILEVER TAPER PLATE DISSIPATORS IN KING EDWARD STREET OVERPASS (25.4 mm = 1in, 1 m = 3ft 3ins)

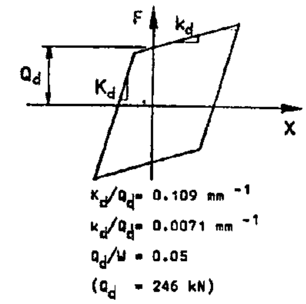
The details at the abutment are similar to those shown for the pier in Fig 7 except that the channel was orientated in the long direction of the abutment and an allowance had to be made for accommodation of lengthening and shortening effects of the superstructure, such as thermal and creep movements, before the device becomes effective. At both pier and abutment the details had to be such as to allow ready access for installation, inspection and, if necessary, replacement.

Extensive dynamic analysis computer studies were carried out during design of the bridge, including comparisons of structural response with and without energy dissipators and examination of the sensitivity of results to various parameters, namely earthquake characteristics, foundation and bearing stiffnesses, structural modelling and member strengths. The results are described more fully elsewhere [8]. The model used for analysis of longitudinal response is shown in Fig 8(a). The flexural stiffness of the foundations and of the pier and abutment were represented by beam/column elements and the shear stiffness of the bearings and the flexural stiffness of the energy dissipator were modelled by axially deformed truss elements. The structural and soil masses were lumped at the nodes. Damping ratios of 4% and 5% equivalent viscous damping were assumed for modes 1 and 2 respectively. The design strength of the dissipators, Q_d, being force at the zero displacement ordinate, was chosen for reasons of practicality and on the basis of previous studies [9] as 0.05 times the weight of the superstructure for the combination of two acting along each principal axis. The bearing and dissipator stiffness characteristics are illustrated in Fig 8(b).

In Fig 8(c) is plotted the time-history of displacement response of the structure to the El Centro 1940 N-S earthquake. Cases (i) and (ii) incorporate energy dissipating devices of equivalent characteristics, but the yield level of the structural elements has been varied. In case (i) the piers have been reinforced so that they would not flexurally yield during an earthquake of this intensity. For case (ii) the strength of the pier has been reduced to achieve construction economies and minor yielding would be expected during an earthquake of this intensity. Case (iii) corresponds to the conventional ductile design approach without energy dissipators, and with pot-stay bearings at the pier and glide bearings at the abutment. The flexural yield level of the piers was determined from the New Zealand Ministry of Works and Development Highway Bridge Design Brief [10] and is 1.5 times greater than that for case (ii). The intervals of the response during which the pier was yielding is shown for each case. Comparison of the results for the three cases shows that the effect of incorporation of the energy dissipating devices is to reduce maximum displacements sustained by the superstructure and to minimise or eliminate structural damage at the "design earthquake" intensity. Although the "fully isolated" system has similar strength requirements in the piers and greater strength requirements in the abutments than the "no dissipators-ductile" system, it avoids damage to the structural members and reduces structural displacements. The greater forces on the abutment do not represent a cost penalty because reinforcement and geometry were governed by minimum



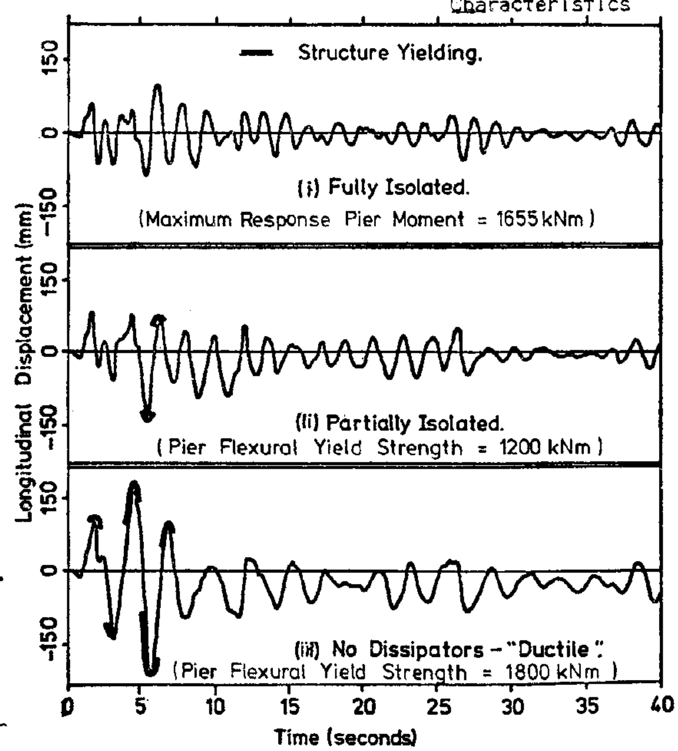
(a) Structural Model



LOCATION	TOTAL SHEAR STIFFNESS
Abutment	3.98 kN/mm
Pier	5.48 kN/mm

Bearings

(b) Dissipator and Bearing Characteristics



(c) Longitudinal Response for Three Structural Alternatives to El Centro 1940 N-S

FIG 8 : DYNAMIC ANALYSIS OF KING EDWARD STREET OVERPASS
(25.4 mm=1in, 4.45 kN = 1 kip, 1.36 kNm=kip ft)

requirements and other considerations. The reduction in structural displacements may represent substantial cost savings in terms of reduced deck joint separation requirements. The "partially isolated" system compared to the conventional "no dissipators-ductile" system achieves construction economies, through reduced strength requirements in piers and foundations, reduced yielding and hence lower ductility demand and damage, and lower structural displacements.

The cost per dissipator was $1,100. This represented for the eight dissipators 1.5% of the total cost of the structure. Because the design forces chosen for the structure were similar to those of a conventional design, the total costs were similar. The extra costs of devices and details was partially offset by savings in joint details and separation requirements at the abutment. The benefit was that the degree of protection against damage was dramatically increased. The return period of an earthquake which would induce yielding in the structural members was increased perhaps ten times compared with a conventional design.

Scamperdown Bridge

The original seismic design concept of this three-span steel universal beam and concrete deck bridge was as illustrated in Fig 9(a). Lateral forces were to be resisted entirely by the piers and raked piles, designed in accordance with the ductile design requirements of the Highway Bridge Design Brief [10]. The foundation material is very soft down to the papa mudstone at depths up to 25 m (82ft). The flexibility of the pier-foundation system was such that the required separation allowance at the abutment was 600 mm (24in). Preliminary dynamic analyses indicated that with either elastomeric or sliding bearings at the abutment, much of the seismic force was being resisted at this point because of its greater stiffness, and pier moments were well below yield.

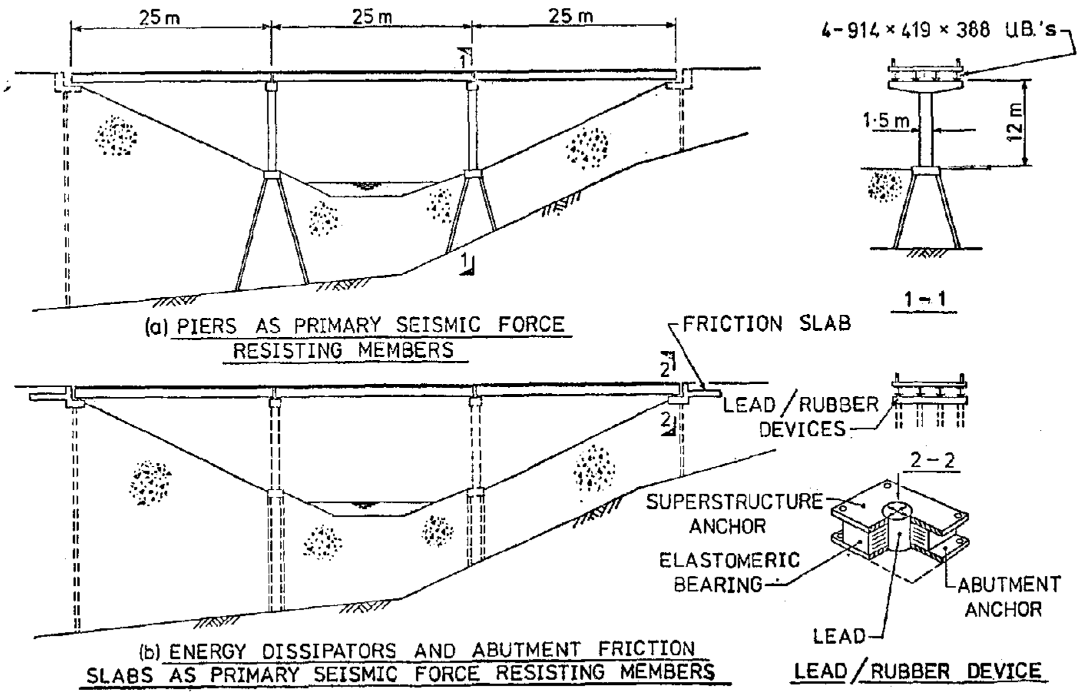


FIG 9 : LEAD-RUBBER DISSIPATORS IN SCAMPERDOWN BRIDGE (25.4 mm = 1in, 1 m = 3ft 3in)

An alternative approach was then investigated, as shown in Fig 9(b), in which energy dissipating devices of the lead-rubber type at the abutments and abutment friction slabs were the primary seismic force resisting members. The piers then acted only as props with design governed by eccentric live load.

A series of numerical integration time-history dynamic analyses were carried out for both longitudinal and transverse response of the bridge. The sensitivity of the results to damping, soil stiffness, energy dissipator characteristics and earthquake acceleration records was studied. After review of analysis results it was resolved to use the lead-rubber device with $Q_d = 0.065W$. Although the superstructure displacements under El Centro 1940 N-S of 70 mm (2.8in) were of the order of 35% greater with these devices than for higher strength devices with $Q_d = 0.13W$, the force to the abutment was 35% lower at 490 kN (110 kips) (0.16 times the weight of half the superstructure). The forces on the piers were similar and not significant. The details of the device were a 406 x 356 x 193 mm (16 x 14 x 7.6in) elastomeric bearing with a lead cylinder of diameter 100 mm (4in). The dead load per bearing was 295 kN (166 kips). The maximum calculated thermal movement was ± 20 mm (¾in). The maximum design wind load was 38 kN (8.5 kips), two-thirds of the "yield" strength of the device.

In view of the importance of stability of the abutment under earthquake shaking corresponding to the "design earthquake", an assessment was made of the possibility of slope failure or liquefaction. Neither seemed likely.

As a result of the change in seismic design concept, savings in the pier and pile systems and joint details were listed as:

(a) a reduction in pier size from 1.5 to 1.3 m diameter;

(b) a change in pile design from 8 steel H-piles, raked at 1:4, to 4 vertical 500 mm dia steel shell piles, which could be reinforced to a capacity exceeding that of the pier. (A reassessment of the original ductile design approach indicated that it was impossible to provide a practical pile system to resist the pier yield moment under the capacity design requirements [10]. Cylinders were not considered a feasible solution).

(c) A reduction in the seismic separation requirements at the abutment from 600 mm (24in) to less than 100 mm (4in) and removal of the need for "knock-off" elements.

(d) Protection of the piers against earthquake induced damage.

Cromwell Bridge

This five span standard steel truss bridge on tall reinforced concrete piers is required to cross a man-made lake as part of the Clutha power development scheme. A proposal was considered for anchoring longitudinal seismic response forces through energy dissipating devices into one rock abutment. The devices proposed are of the uniaxial flexural beam type as shown in Fig 10.

Extensive numerical integration time-history dynamic analyses were carried out studying the sensitivity of the response to: input ground motions, pier stiffness, foundation stiffness, damping, rotational inertia of piers and dissipator yield level. The response of the structure was found to be influenced strongly by the inertias of the piers and hydrodynamic added mass of water,

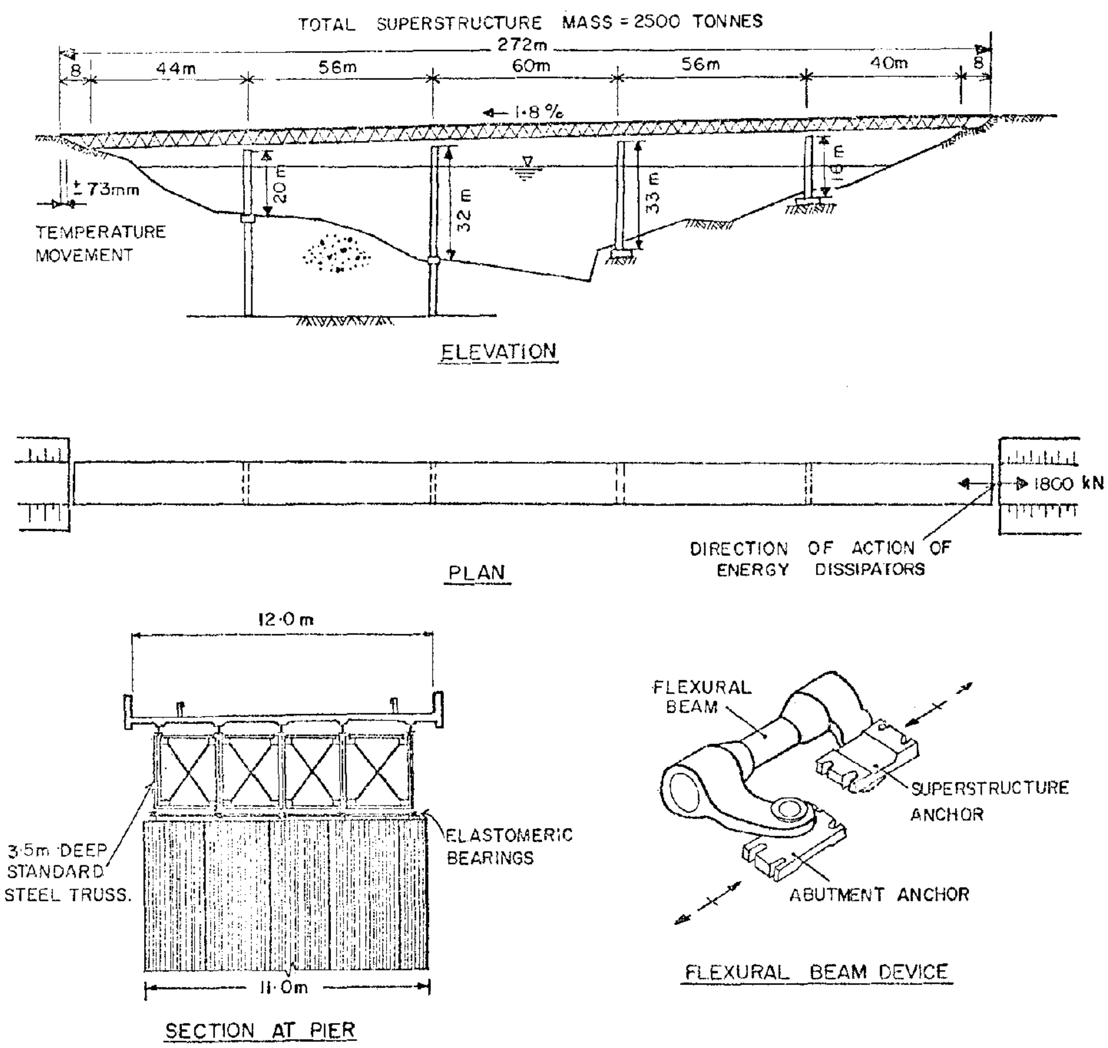


FIG 10 : FLEXURAL BEAM DISSIPATORS IN CROMWELL BRIDGE
(25.4 mm = 1in, 1 m = 3ft 3in, 4.45 kN = 1 kip)

which together were more than the mass of the superstructure. The moments induced in the piers were sensitive to assumptions regarding pier stiffness and damping. Three earthquake records were used; El Centro 1940 N-S, B1 and Parkfield, with increasing severity of effect in that order. Three dissipator yield levels were considered corresponding to total dissipator forces, Q_d, of 0.05, 0.10 and 0.15 times the weight of the superstructure. The general trend with increasing dissipator yield force was reduced displacements of the superstructure, increased shear displacements in the elastomeric bearings on the piers and increased bending moments in the piers. For final design it was resolved to adopt a value of 0.07W as the optimum solution in terms of minimised displacements and practicality of provision of the required force. Forces induced in the dissipators by temperature effects on the superstructure precluded the use of any lower strength dissipators. The dissipator force was provided by six 300 kN (67 kips) yield strength devices.

The comparative longitudinal response of the bridge with and without the energy dissipating devices at the abutment is shown in Fig 11. Time-histories of displacement response of the superstructure and base moment of the 32 mm (105 ft) and 16 m (52 ft) piers are plotted for the B1 earthquake and 5% equivalent viscous damping. The dramatic reduction in superstructure displacement due to the dissipators is evident. Reduction in base moment in the 32 m (105ft) pier was minimal but reduction in base moment in the 16 m (52ft) pier was significant. The ideal flexural strengths of the piers based on minimum reinforcement requirements were 18,300 kNm (13,455 kip ft) for the 32 m (105ft) and 33 m (108ft) piers and 16,900 kNm (12,426 kip ft) for the 16 m (52ft) and 20 m (66ft) piers. Thus only minimal yielding is expected even without dissipators. This is attributable to the flexibility of the structure, with a calculated first mode period without dissipators of 2.5 seconds. The maximum shear deformation of the elastomeric bearings was of the same order for the two cases. The advantage of incorporation of the device is evidently a reduction in superstructure displacements and consequent savings in abutment joint details, and a greater degree of protection against yielding of the piers. In addition, restraint against longitudinal vibration under service load, such as traffic loading, had to be provided. In this application it is desirable to avoid damage below the waterline, under seismic loading,

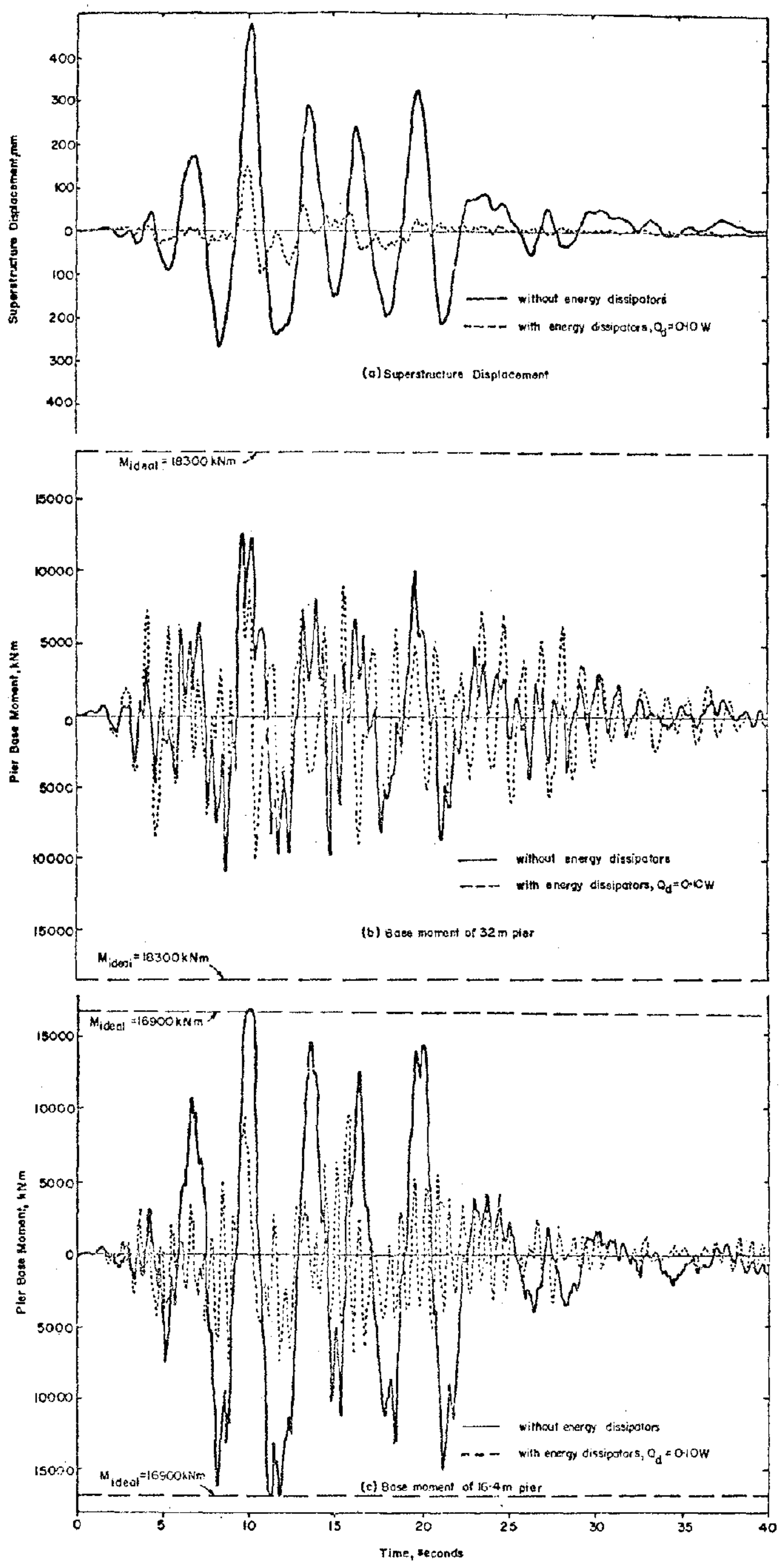


FIG 11: DYNAMIC ANALYSIS RESULTS FOR CROMWELL BRIDGE
(25.4 mm = 1in, 4.45 kN = 1 kip, 1.36 kNm = 1 kip ft)

where it is difficult to inspect and repair.

South Brighton Bridge

A three-span bridge incorporating 22 m I beams and a cast in situ concrete deck was chosen to replace the existing South Brighton Bridge situated just upstream from the junction of the Avon River with the Christchurch Estuary. Priestley and Stockwell [11] report a study of a proposal to incorporate energy dissipating devices of the steel cantilever type. The proposal was rejected as it was concluded that design based on a conventional ductile approach was more economical without significant increase to seismic risk.

The structural details of the bridge are shown in Fig 12. The superstructure is continuous over internal piers with separation achieved at the abutments by deck joints. Elastomeric bearings support the superstructure at abutments and internal hammerheads. Seismic forces are resisted in each direction by single-column octagonal piers acting as vertical cantilevers, from rigid pile caps. These are supported by raking piles founded in dense sand. Initial design was based on 8 omnidirectional cantilever taper dissipators, two at each support, acting between the superstructure and the pier hammerhead or abutment, with a total dissipator force Q_d, equal to 5% of the weight of the superstructure.

Both transverse and longitudinal dynamic analyses were performed. Diaphragm action of the slab was assumed to be ineffective transversely due to existing normal transverse flexural cracking and induced torsional cracking. Thus, the internal piers were conservatively designed for the full contributory mass corresponding to one complete span. The effect of this assumption was that the dissipator force, Q_d, on the pier in the analysis model was only 3.5% of the assumed contributory mass. The base of the pier was assumed to be fully fixed, as initial calculations showed the extent of foundation flexibility to be insignificant. The pier stiffness was based on a cracked section just prior to yield, and yield moment based on probable steel yield stress.

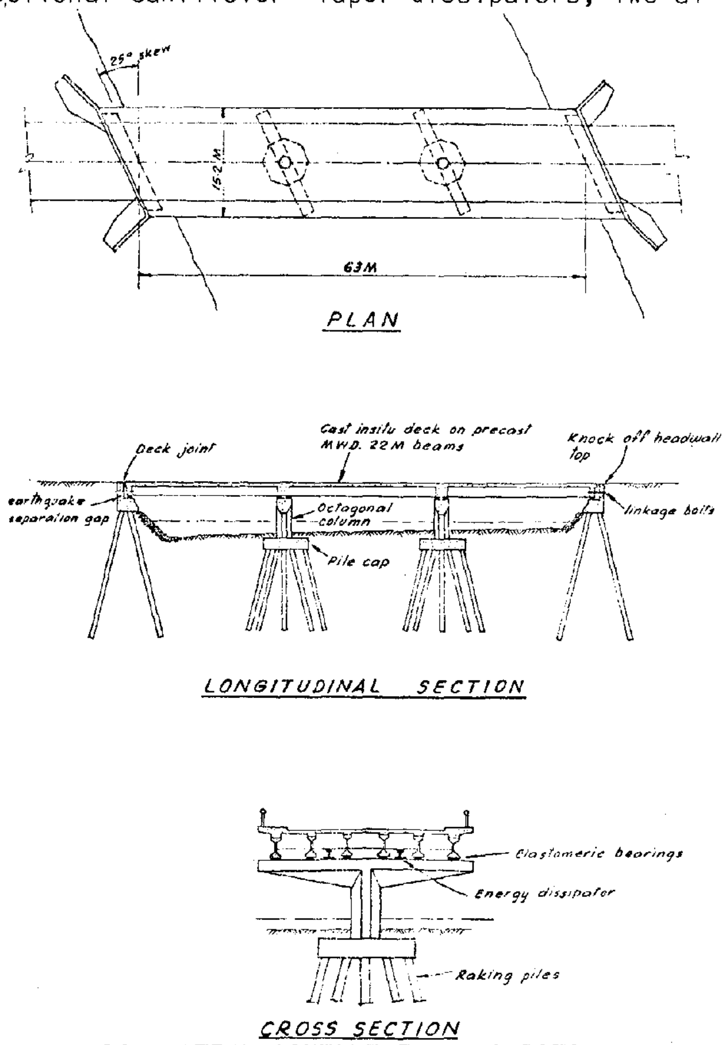


FIG 12 : STEEL CANTILEVER DISSIPATOR PROPOSAL FOR SOUTH BRIGHTON BRIDGE (1 m = 3ft 3ins)

Allowance was made in the model to simulate rotational mass inertia of the superstructure. The bilinear hysteresis loop assumed to represent the combined behaviour of one dissipator and 6 Advanx M11-115-4 elastomeric bearings was narrow since the stiffness of the post-yield portion of the loop was dominated by the stiffness of the bearings. Two earthquake records with different characteristics were chosen, the El Centro 1940 N-S and Bucharest 1977 N-S records. The latter excites unusually high response from elastic structures with natural periods in the 1.2 to 1.7 seconds range.

Comparisons of pier base moment time-histories for El Centro and Bucharest responses, with and without energy dissipators, are given in Fig 13. Also shown is the response of a monolithic design, run only for comparison as this approach would not be feasible because of the short pier stems. Under El Centro the monolithic design reaches yield on seven occasions whereas the models with and without dissipators are both well below yield, the main effect of the dissipators being to attenuate response with increasing time. In the case of the Bucharest record, all three models are subject to two yield excursions, but the monolithic case does not sustain the extent of yielding of both the other systems, which show similar behaviour. Results obtained for pier-top displacements under the Bucharest record showed large plastic sets in the vicinity of 75-100 mm (3-4in). The dissipators had a moderate effect in reducing displacements, whereas the monolithic system was comparatively unaffected by the earthquake.

It was concluded from the results that insufficient reduction in response was obtained from incorporation of the dissipators in this case to warrant their inclusion. Accordingly, the bridge was redesigned without dissipators. It was also noted that "isolation" of a structure by introducing flexible mountings may detrimentally affect response under an earthquake of the nature of the Bucharest record. It may be commented that considerations of more flexible mountings, allowance for transverse diaphragm action and reduced longitudinal displacements is likely to have placed the dissipator alternative in more favourable light.

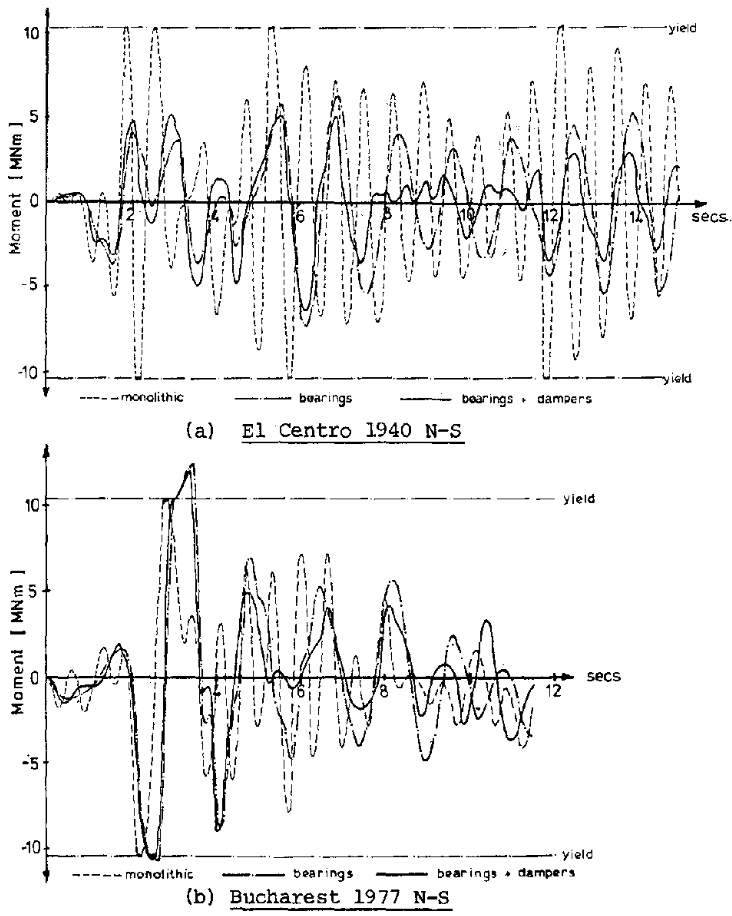


FIG 13 : PIER BASE MOMENT TIME-HISTORIES
(1 MNm = 735 kip ft)

PARAMETER STUDIES FOR DESIGN

Introduction

A series of parameter studies was undertaken in the Civil Design Office of the New Zealand Ministry of Works and Development for the purposes of studying the behaviour of bridges, with and without energy dissipating devices, under earthquake loading. The sensitivity of the seismic response to the principal parameters was investigated. The programme of studies was undertaken with the objective of preparing simple design charts, which could be used in lieu of a dynamic analysis, for design of bridge structures incorporating energy dissipating devices where the structural form did not comprise any unusual features. The results are described more fully elsewhere [2]. Other parameter studies have been made by Sharpe [12] and are summarised in ref 2.

Analysis

The analyses were made using the step-by-step numerical integration time-history of response analysis computer program DRAIN-2D [13]. The basic model used for the studies is shown in Fig 14(b). It could be regarded as simulating half a bridge structure as illustrated in Fig 14(a). The model allows for variability of stiffness of abutment, bearings, pier and foundations. The pier mass is located at 0.65 times the height of the pier, simulating the centre of inertial mass. The mass at the base of the pier represents that due to the pile cap plus a surrounding mass of soil vibrating with the pile cap.

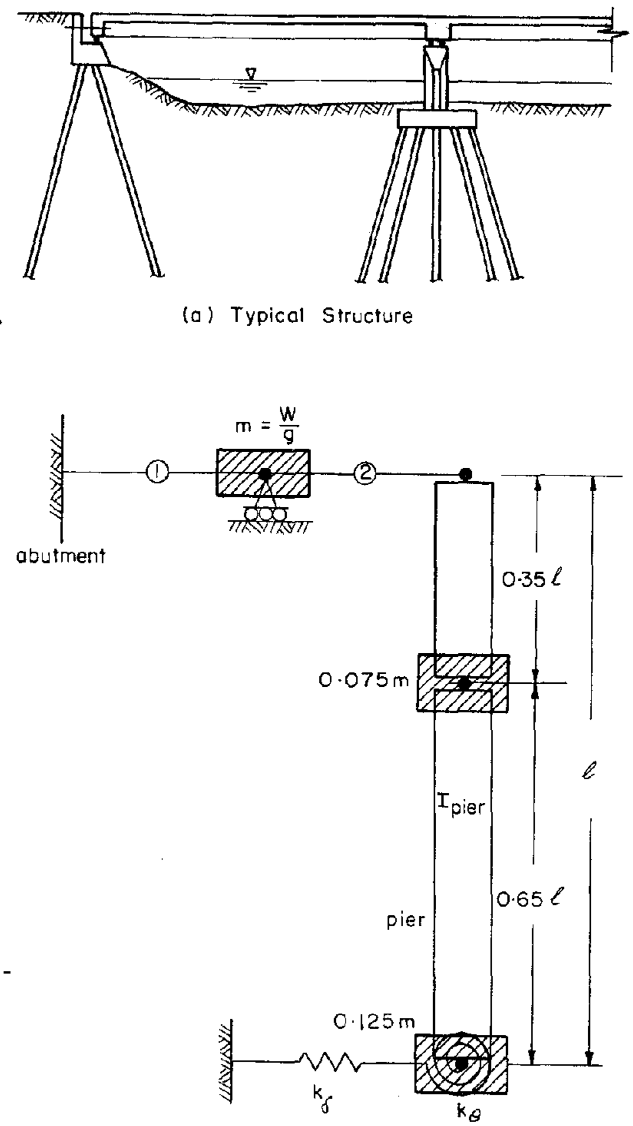


FIG 14 : STRUCTURAL MODEL FOR PARAMETER STUDIES

The model was used to represent longitudinal response and also transverse response where the superstructure could be assumed to act as a rigid diaphragm in plan or where assumptions were made as to the inertial mass contributory to the pier. The model could also be used to consider energy dissipating devices located at either or both abutment and pier, by allocation of suitable force-deflection characteristics to the truss elements 1 and 2 in Fig 14(b).

Variables considered were the strength and stiffness of the energy dissipators, the stiffnesses of elastomeric bearings, abutment, pier and foundations, the flexural strength of the pier and the design earthquakes, namely El Centro 1940 N-S, artificial B1, and Parkfield. Constants were the inertias as shown in Fig 14, damping ratio of 5% in the first mode, and post-yield stiffness ratio of flexurally yielding structural members of 5% of the elastic stiffness. In all cases sensitivity studies were intially made to assess the effect of variations from the assumed constant values.

Results

The results of computed acceleration response for analyses where structural elements were required to remain elastic, and for cases with varying strength dissipators, are illustrated for the El Centro 1940 N-S earthquake in Fig 15 and compared with elastic response without dissipators. The curves for structures incorporating energy dissipating devices cover only that part of the period range consistent with values expected in practice. The curve labelled "Skinner" represents a smoothed curve derived from the response spectra from eight acceleration records scaled to the same intensity as El Centro 1940 N-S [14]. The response spectra of Fig 15 are determined from the "effective period of vibration" for structures with energy dissipators, based on the secant stiffness at maximum displacement for the inelastic system. The effect of the dissipators may be seen to be similar to that of extra equivalent viscous damping; the higher the dissipator strength for a given period the larger reduction in response. In using these curves to compare the effects of different strength dissipators on response of a particular structure, it should be recognised that the effect of increasing the dissipator strength will be to decrease the total period. That is, the period will shift to the left on Fig 15. The effect of this is discussed in the next section. Similar curves for other earthquakes are given in ref 2.

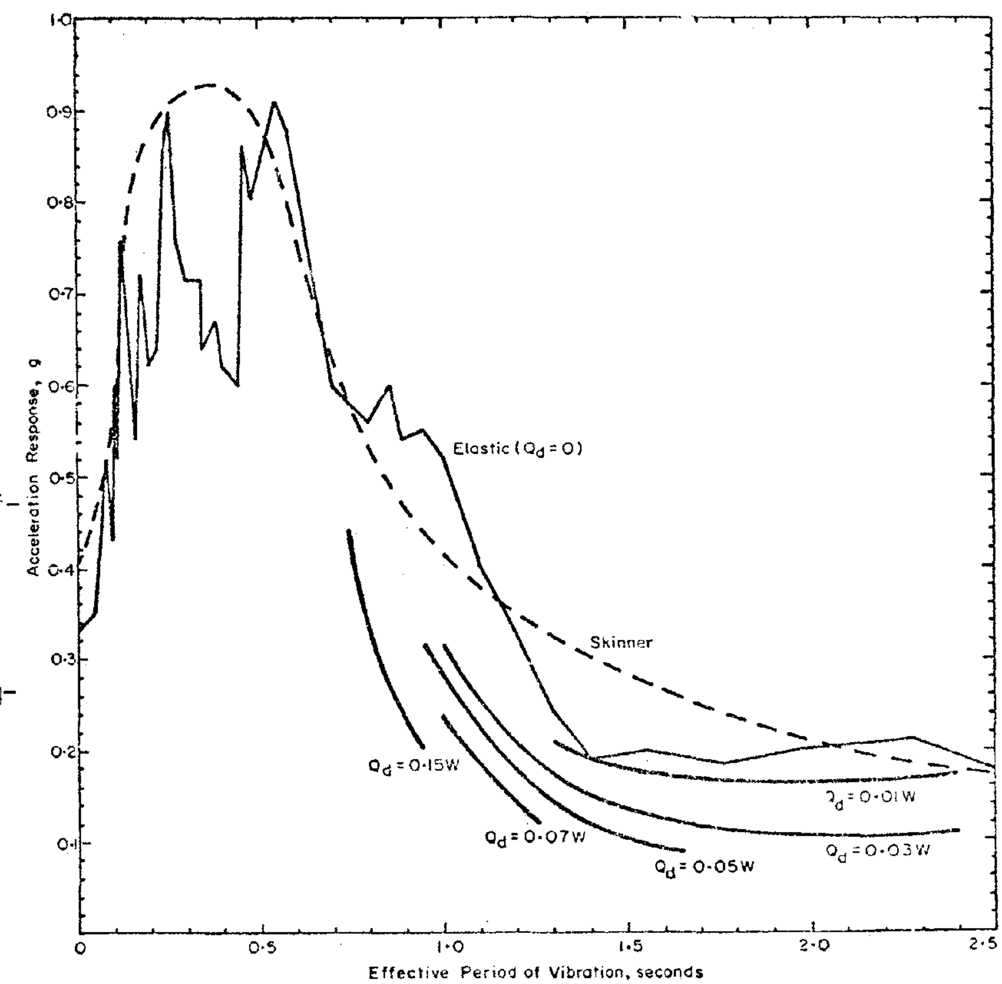


FIG 15 : ACCELERATION RESPONSE WITH ENERGY DISSIPATORS OF VARYING YIELD STRENGTH, EL CENTRO 1940 N-S, λ = 5%

Design Charts for Elastic Structures

On the basis of the parameter studies, design charts were prepared for structures with and without energy dissipators where the substructure is to remain elastic. These charts are presented in full in ref 2 and cover the following cases:

(a) elastomeric bearings only at both abutment and pier
(b) energy dissipators at abutment only
(c) energy dissipators at pier only
(d) energy dissipators at both abutment and pier

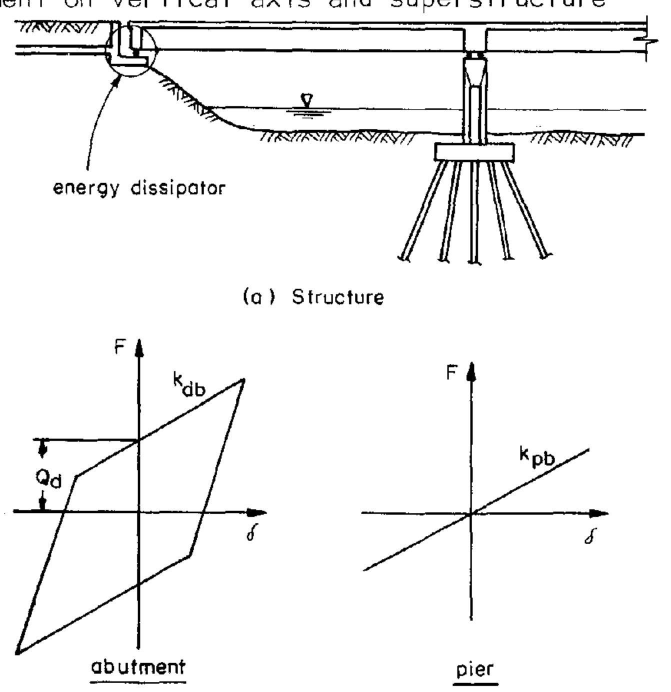
Earthquake acceleration records used were El Centro 1940 N-S, artificial B1 and Parkfield. The charts may be used to assess either longitudinal or transverse response, or if desired response along an axis inclined to the principal axes. As an example, a bridge structure with energy dissipators located only at abutments and elastic restraint at the piers is illustrated in Fig 16. Figs 17 and 18 are design charts for this case where the abutment is rigid, the energy dissipator strength Q_d = 0.05W, and for the El Centro 1940 N-S and B1 earthquakes respectively. The procedure for use of each chart is as follows:

(i) calculate weight of superstructure, W;
(ii) calculate combined stiffness of dissipator plus elastomeric bearings at abutment, k_{db}, and determine k_{db}/W /mm;
(iii) calculate stiffness of pier plus elastomeric bearings (or pier alone where superstructure is built-in to pier), k_{pb}, and determine k_{pb}/W /mm
(iv) from top half of chart, determine intersection of k_{db}/W line and k_{pb}/W curve to give force on abutment on vertical axis and superstructure displacement on horizontal axis;
(v) determine force on pier by either -

(1) multiply superstructure displacement derived from (iv) above by the calculated pier stiffness, k_{pb}, or

(2) from bottom half of chart, determine intersection of k_{pb}/W line and k_{db}/W curve.

The charts illustrate the sensivity of response to the combined stiffness of dissipators plus bearings and to design earthquake characteristics. The apparently surprising decrease of total displacement for increasing flexibility of the pier restraint in Fig 17, is consistent with the dip in the displacement response spectrum for the El Centro 1940 N-S record for periods increasing from 1 to 1.5 seconds.



(b) Force - Deflection Characteristics

FIG 16 : BRIDGE WITH ENERGY DISSIPATORS AT ABUTMENT ONLY

The stiffness parameters and forces in the dissipators, piers and abutments are expressed in terms of the weight of the superstructure, W. The charts may be used considering the whole superstructure or a portion of it. For example, for a three-span bridge with similar stiffness characteristics between each pier and between each abutment, W may be taken as the weight of half of the superstructure and the stiffness parameters then relate to one pier and abutment. For a bridge with more than three spans, or where the elastic stiffness characteristics of the piers differ, W may preferably be taken as the weight of the whole superstructure and the stiffness parameters determined by summing in parallel the stiffnesses of all the pier and the abutment elements. The derived forces then represent total forces on abutments and piers. The design charts include cases for zero stiffness of the pier. This represents the situation when all response forces are resisted by the abutment, for example, for a single span bridge.

Where horizontal diaphragm action of the superstructure may be relied on in plan, the charts may be used in the same way for transverse as longitudinal response. Where such diaphragm action cannot be relied on, use of the charts requires an assumption of a contributory mass of superstructure to the pier or abutment and the curves for zero restraint from the abutment or pier are then used. It is a matter for debate as to when diaphragm action may be assumed. It is fairly clear that a box-girder superstructure will be very stiff in plan over several spans. It is not so clear how a superstructure comprising precast I beams supporting a cast-in situ concrete deck will behave. In such cases although some torsional cracking in the deck may occur because of differences in rotation between the tops of the piers and the abutments, the rigidity in plan is still likely to be high up to 3 spans.

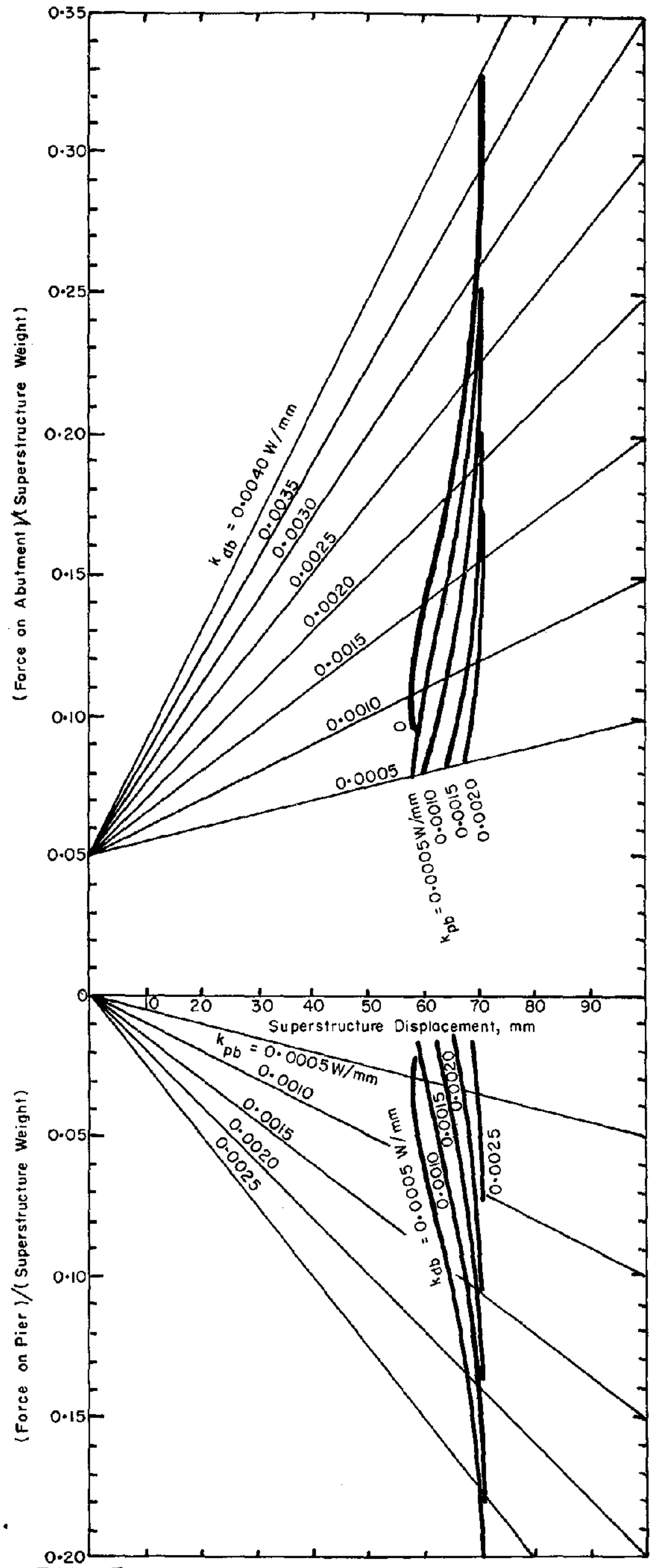


FIG 17 : ENERGY DISSIPATORS ON RIGID ABUTMENT, Q_d = 0.05W, EL CENTRO 1940 N-S (25.4 mm = 1in)

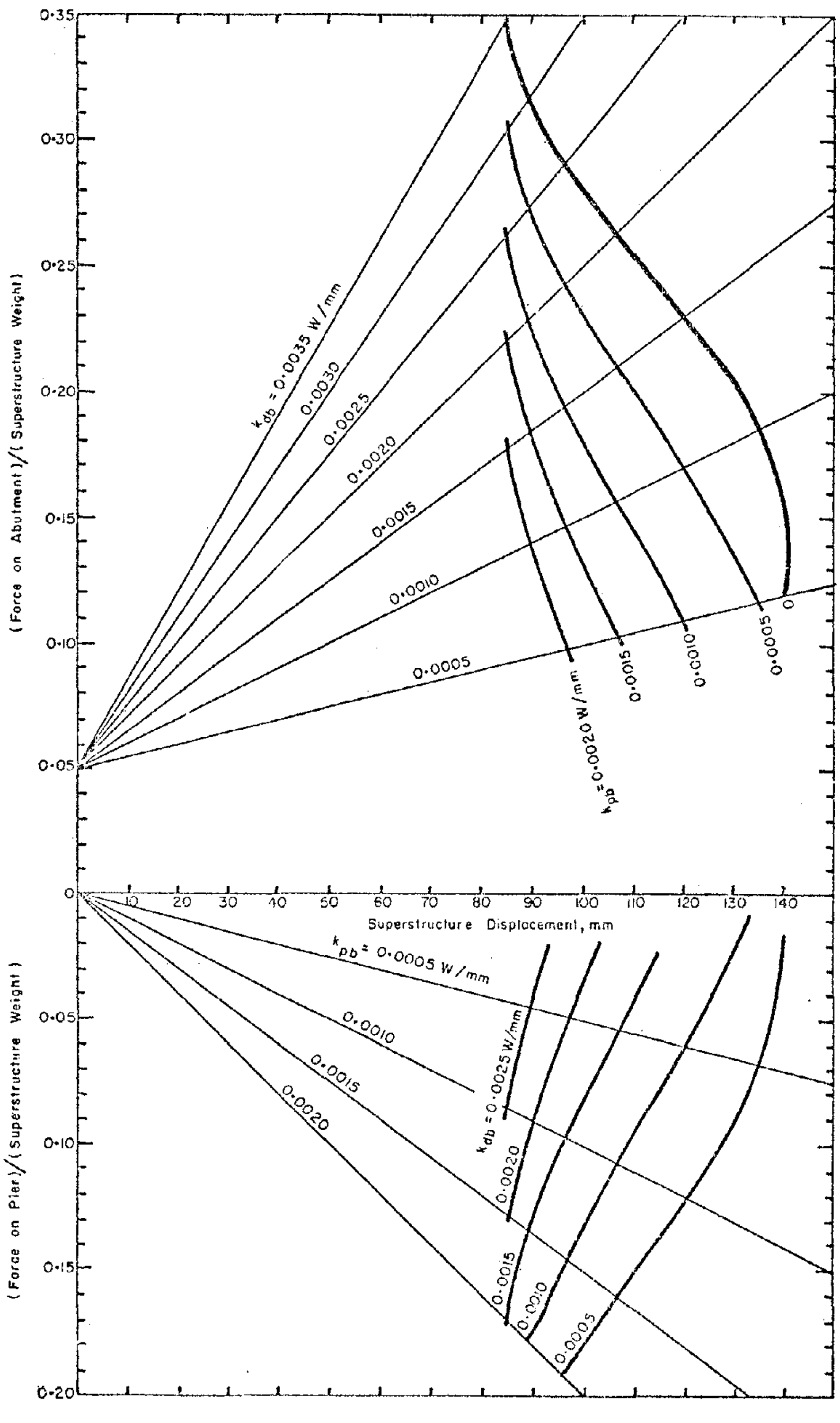


FIG 18 : ENERGY DISSIPATORS ON RIGID ABUTMENT, Q_d = 0.05W, B1
(25.4 mm = 1in)

Fig 19 comprises a chart similar to Fig 18 except that the energy dissipator strength, Q_d = 0.07W. Inspection of these two figures indicates that with increasing dissipator strength both maximum displacement and maximum substructure force are reduced, the former being affected more.

Inelastic Structures Incorporating Energy Dissipating Devices

The effect of incorporation of energy dissipators on the ductility demand on flexurally yielding structures is illustrated in Fig 20. Period of vibration is plotted against μV, where μ is the computed structure ductility demand and V is the design structure seismic shear force, for the El Centro 1940 N-S earthquake. The two cases A and B correspond to conventional structures with different pier flexural strengths; respectively probable strength with importance factor F of 1.0, and dependable strength with F = 0.85, both for seismic zone A [10]. The ratio of strengths A to B is 1.7. The curve labelled B* is for a structure incorporating energy dissipators of strength Q_d = 0.05W but with the same pier yield strength as for the conventional structure labelled B. It may be seen that the μV curves are reasonably close to the elastic response spectrum, that is the "equal displacement criterion" is satisfied, except for short period structures. This reflects the high ductility demand on stiff structures, generally attributed to the tendency of such structures to degrade after yielding into period ranges of increased response. It may be seen that the incorporation of energy dissipating devices has had little effect on reducing response and may even be a disadvantage on flexurally yielding structures.

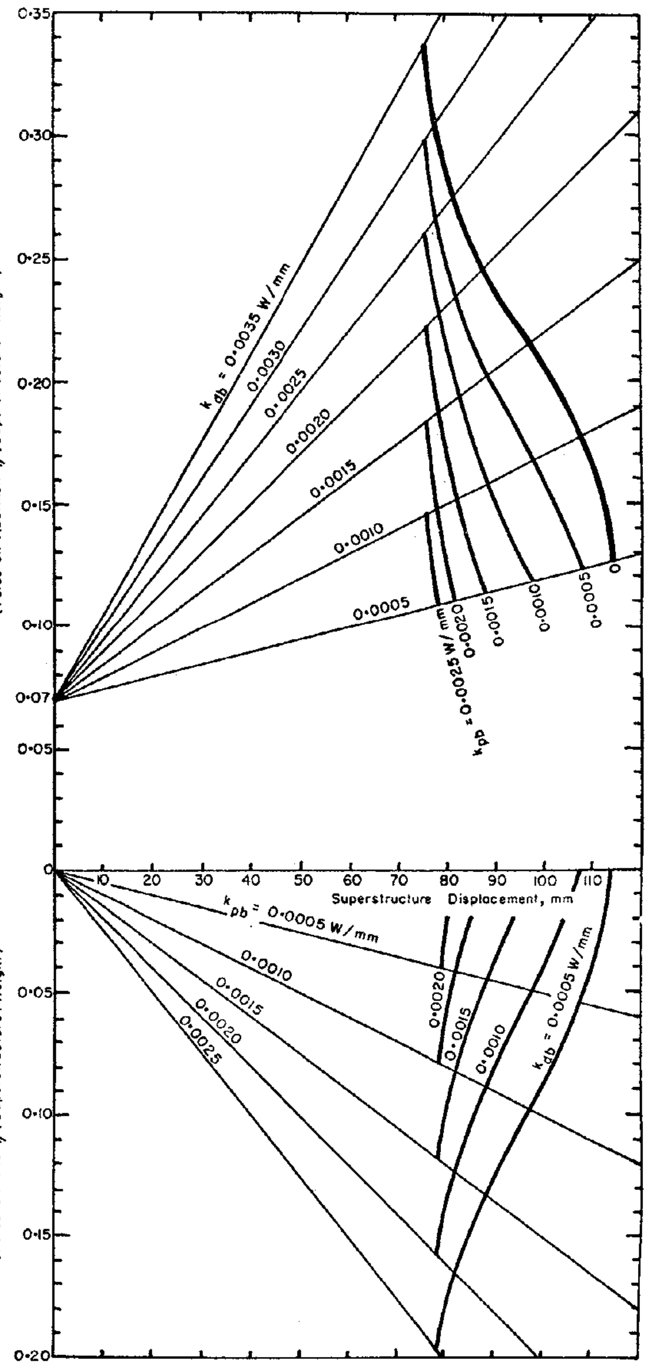


FIG 19 : ENERGY DISSIPATORS ON RIGID ABUTMENT, Q_d = 0.07W, B1
(25.4 mm = 1in)

A group of the New Zealand National Society for Earthquake Engineering preparing recommendations for seismic design of bridges, is proceeding on the basis of specification of design earthquake loadings by means of a design elastic response spectrum. This has the advantage of removing any confusion that may arise, regarding the magnitude of earthquake effects on a structure, caused by use of design coefficients representing an elastic response spectrum divided by some factor representing the required ductility capability of the structure. The designer proportions his structure for:

$\mu V = C.F.W.$

where μ = structure ductility capability
V = design seismic shear
C = coefficient from elastic design spectrum
F = importance factor
W = weight of superstructure.

Any combination of μ and V may then be adopted to satisfy the above condition. If the maximum structure ductility capability is assessed, the minimum V for that period is obtained. Alternatively, if non-seismic structural requirements dictate a higher value of V, then design may take advantage of a reduced ductility demand. As shown in Fig 20 the μV curve will approximately follow the elastic spectrum except for low period structures.

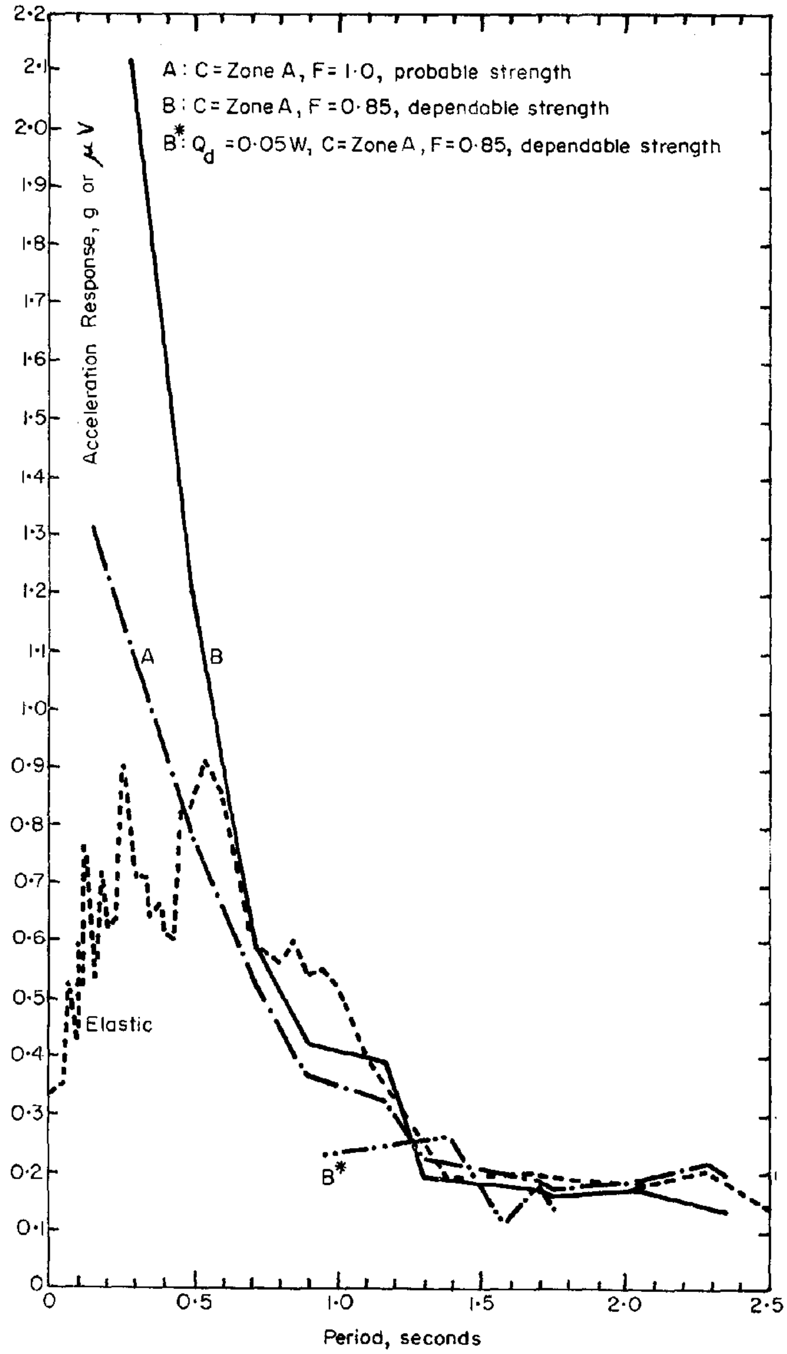


FIG 20 : μV CURVES FOR DUCTILE DESIGN
EL CENTRO 1940 N-S, λ = 5%

DESIGN PHILOSOPHY

Application

The following are applications where incorporation of energy dissipating devices in bridges is most likely to be effective:

(a) in regions of high seismicity;
(b) mounted on a stiff substructure;
(c) mounted on a substructure desired to remain elastic.

The corollary is that energy dissipating devices are unlikely to be effective and may even be a disadvantage in regions of low seismicity or where mounted on a flexible or flexurally yielding substructure.

Potential Advantages

The use of mechanical energy dissipating devices offers a number of potential advantages for the design of earthquake resisting bridges and other structures.

Conceptual Simplicity - The approach is attractive in its conceptual simplicity, that is a concentration of earthquake energy dissipation in components especially designed and developed for this purpose and detailed for easy replacement if necessary.

Design Simplicity - There is a potential for development of standardised solutions for design of bridges incorporating mechanical energy dissipating devices. This is an advantage relative to the substantial design effort required in the conventional ductile approach.

Damage Control - The anticipated frequency of earthquake induced damage in structural members may be significantly decreased without cost penalty.

Economic - The main potential for economic advantage lies in:

- possible savings in abutment separation requirements and joint details:
- redistribution of seismic forces on the substructure, for example control of seismic forces through energy dissipating devicrs at abutments rather than by ductile yielding of piers, provided the abutments have adequate strength;
- use of non-ductile forms or components, with sufficient strength to remain elastic under the expected forces imposed by yielding of the dissipators.

Design Criteria

The draft New Zealand Code of Practice for the Design of Concrete Structures [15] nominates the following criteria for design of structures incorporating flexible mountings and mechanical energy dissipating devices:

(a) the performance of the devices used is to be substantiated by tests;
(b) proper studies are to be made towards the selection of a suitable design earthquake for the structure;

(c) the degree of protection against yielding of the structural members is to be at least as great as that implied in the code relating to the conventional seismic design approach without energy dissipating devices;

(d) the structure is to be detailed to deform in a controlled manner in the event of an earthquake greater than the design earthquake.

A philosophy of design should consider the implications of varying levels of earthquake attack. The following is suggested for bridges and other structures incorporating energy dissipating devices:

1. *Moderate Earthquake:* For a moderate earthquake such as may be expected 2 or 3 times during the life of a structure, energy dissipation should be confined to the devices, and there should be no damage to structural members.

2. *Design Earthquake:* For a "design" earthquake, for example one with a return period of, say 150 years, the designer may adjust the strength levels in the structural members to achieve an optimum solution between construction economies and anticipated frequency of earthquake induced damage. However, the degree of protection against yielding of the structural members should be at least as great as that implied for the conventional seismic design approach without dissipators. In suitable application this may be achieved with construction cost savings. It is recommended that the extent, if any, to which the degree of protection is increased above that minimum, to reduce the anticipated frequency of earthquake induced damage, should be resolved with regard to the client's wishes.

3. *Extreme Earthquake:* For an extreme earthquake there should be a suitable hierarchy of failure of the structural and foundation members that will preclude a brittle collapse. This may be achieved by appropriate margins of strength between non-ductile and ductile members and with attention to detail.

Although the design criteria outlined above encompass three earthquake levels, the design practice need only be based on the "design" earthquake. In the course of that design, the implications of yield levels on response to the "moderate" earthquake would have to be considered, as would also the implications of strength margins and detailing for an "extreme" earthquake.

Structural Detailing

It is recommended in the design philosophy outlined in the previous sections that structures should be detailed to deform in a controlled manner under overload situations. This is regarded as sound engineering practice in view of the uncertainties in modelling and analysis of the structure and in the characteristics of ground shaking. In general, the anticipated lower ductility demand on structures incorporating energy dissipating devices means that simplified detailing procedures appropriate for structures of limited ductility [15] would be satisfactory.

The required controlled post-yield behaviour may generally be achieved by provision of suitable margins of strength between ductile and non-ductile members and by attention to detailing, but without full capacity design procedures. For example, where forces in the substructure are calculated using design charts as shown above or from dynamic analysis, and where it is desired that the structure remain elastic up to "design earthquake" intensity, suitable provisions might be:

(a) Substructure members capable of ductile flexural yielding are to be designed for a probable flexural strength (based on a capacity reduction factor, Ø, of 1.0 and probable yield strength of reinforcing steel of, say, 1.15 times the minimum specified) equal to the calculated "design earthquake" moment.

(b) Non-ductile substructure members, or members in which damage is unacceptable because of inaccessibility for inspection and repair, or all members in shear, are to be designed for a dependable strength (based on appropriate value of Ø [15] and minimum specified material strengths) of 1.10 times the force calculated in that member at the "design earthquake".

(c) The separation details between superstructure and abutment are to allow for a deflection of at least 1.15 times the values calculated at the "design earthquake".

(d) Special reinforcement requirements for confinement of concrete in bridge piers need not be complied with. However, good practice should be followed in the detailing of the transverse reinforcement to enhance ductility in the potential plastic hinge zones. The provisions for design of shear and confinement reinforcement for structures of limited ductility in ref 15 may serve as a guide.

(e) Care should be taken in detailing to ensure the integrity of the structure during earthquake shaking. Positive horizontal linkages should be provided between adjacent sections of superstructure at supports and hinges and between superstructures and their supporting abutments.

CONCLUSION

If energy dissipating devices are used in appropriate applications, they offer some attractive advantages as an alternative to either an elastic design approach or one based on ductile flexural yielding of structural members. The greatest benefits apply when the devices are incorporated on stiff substructures, required to remain eleastic and in regions of high seismicity. The potential for economic advantage is probably greatest where incorporation of the devices allows use of non-ductile substructure components or a change of seismic resisting structural form. In these applications the devices may serve to control loads and deflections. Design charts have been presented for use as a substitute for dynamic analysis of common bridge types. Care in detailing is recommended to encourage controlled behaviour in the event of extreme earthquake loading.

ACKNOWLEDGEMENT

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NOTATION

C = basic seismic coefficient

F = importance factor

g = acceleration due to gravity

I_{pier} = moment of inertia of pier

K_d = initial stiffness of dissipators

k_d = post-elastic stiffness of dissipators

k_{db} = Post-elastic stiffness of dissipators plus elastic stiffness of bearings

k_{pb} = elastic stiffness of pier plus bearings

ℓ = height of pier

m = mass of superstructure

Q_d = force due to dissipator at zero displacement ordinate

V = design seismic shear force

W = weight of superstructure

λ = equivalent viscous damping ratio

$\emptyset$ = capacity reduction factor

μ = structure displacement ductility factor.

RETROFITTING OF EXISTING HIGHWAY BRIDGES SUBJECT TO SEISMIC LOADING-PRACTICAL CONSIDERATIONS

By

Oris H. Degenkolb
Design Engineer
California Department of Transportation

ABSTRACT

The 1971 San Fernando earthquake demonstrated that many pre-1971 built bridges would be severely damaged if subjected to severe seismic shaking. California developed details for retrofitting its older bridges in order to mitigate earthquake induced damage. Retrofitting is not expected to prevent all damage but will raise the level of resistance to major damage.

California's present retrofitting program consists of adding restrainers to hinges and bearings. It was started in 1972 and it is expected that 650 bridges will be retrofitted by 1980. The entire program will strengthen approximately 1050 bridges at a cost of between $25 and $30 million.

Improperly reinforced concrete columns are the second most serious seismic deficiency of pre-1971 bridges. California has considered several details for retrofitting those columns and plans to let a developmental contract to apply them to a structure in 1980. Retrofitting columns may not increase the level of seismic resistance of many structures because of other structural weaknesses. More information is needed before an effective column retrofitting program can be started. It is not likely that such a program would be very extensive because of economics and the probability that it would not significantly raise the level of seismic resistance of many structures.

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BACKGROUND

The 1971 San Fernando earthquake pointed out that the bridge design specifications and practices that were in general use at that time were totally inadequate from a seismic point of view. Although there was a long history of buildings and other structures being damaged and collapsed by earthquakes, seismic damage to bridges in the contiguous 48 states was practically non-existent prior to February 9, 1971. The little bridge damage that did occur before that time was limited to minor spalling and cracking of concrete, damaged bearings and grout pads, and slight shifting of spans. The damage did not cause any serious disruptions to traffic, no lives were threatened, and repairing the damage was a relatively minor nuisance.

The San Fernando event demonstrated that many bridges designed and built before that time have one or more of the following deficiencies:

- Segments of the structure are not adequately connected.
- Columns have too few and improperly detailed ties and spirals.
- Lap splices of main column reinforcement are too short and the surrounding concrete is inadequately confined.
- Footing and bent cap concrete is inadequately reinforced.
- Concrete shear keys are too few or unsufficiently reinforced.
- Design force levels were too low considering the seismicity of the location.

Few existing bridges with these deficiencies can economically be brought up to the same level of seismic resistance as a new bridge.

The lack of adequate connections between segments of a bridge is one deficiency that is quite prevalent and the most readily improved by retrofitting. Fortunately, tying segments of a bridge together is the least expensive of the various deficiencies to correct and, when that is done, it partially alleviates the seriousness of the other deficiencies. Bridges with single column bents are particularly vulnerable when structure segments are not connected.

CALIFORNIA'S RETROFITTING PROGRAM

California started a hinge and bearing restrainer retrofitting program to increase the seismic resistance of its existing bridges in 1972. The first bridges to be retrofitted were on the major lifeline routes in the most densely populated areas of the state which are also the most seismically active areas. The second phase of the program included bridges in the same areas which were initially given lower priorities. This phase of the program was expanded in 1976 to include structures which could be damaged if a "Palmdale Bulge" related earthquake should occur.

A third and final phase of the program is now being formulated to identify and retrofit all seismically deficient bridges in California which are in high seismic but less densely populated locations. This survey considers the proximity of structures to active faults, maximum credible bedrock accelerations, type of foundation material, potential for liquifaction, hinge and bearing details, and flexibility of the structure.

At the present time there are 650 bridges in California which have been, are being, or will be retrofitted with hinge and/or bearing restrainers by 1980. It is estimated that approximately 1050 bridges will be retrofitted with restrainers when the program is completed. The total program will cost between $25 and $30 million.

Inadequately reinforced columns are the second greatest seismic problem of many older bridges. Many reinforced concrete columns have too few and improperly detailed ties and spirals, and main column reinforcement was frequently spliced with short laps in inadequately confined concrete. This is particularly critical in structures with single column bents. Three methods have been devised for retrofitting columns to overcome this deficiency. A scheme for confining concrete around lapped column splices and strengthening footings has also been devised. A developmental contract will be let in 1980 to try out these methods on the single column bents of along freeway to freeway interchange ramp. Because of the wide variety of column and footing details that have been used, these schemes cannot be used universally for all columns and footings.

Because of the high cost and limited or questionable benefits that can be obtained, a column retrofitting program will probably be quite restricted.

RETROFITTING PHILOSOPHY

It is not practical to design bridges that will economically serve normal transportation needs but not be damaged to some extent if subjected to severe seismic shaking. The aim is to make structures seismically resistant to the extent that they may sustain damage but not collapse completely. It is not economically practical to prevent the relatively minor, repairable damage such as was experienced in the August 1978 Santa Barbara, California earthquake. It is also desirable that a bridge should be capable of carrying at least a minimum amount of emergency traffic even though it is damaged. Although retrofitting will increase the seismic resistance of an existing structure considerably, a designer is limited by the capabilities and features of the structure and economics.

Portions of some existing structures have to be strengthened to accommodate the anchorage forces that restrainers require. In some cases restrainers that would develop forces required to hold segments of a bridge together would pull the ends out of the spans or pull over the columns. When hinges are not restrained, segments of a bridge can act independently and forces in the columns can be significantly greater than if hinge movements are limited. Thus, retrofitting hinges with restrainers can significantly reduce the probability of column failures.

Although retrofitting columns may prevent some types of failures that were experienced in the San Fernando earthquake, the overall level of shaking required to collapse a bridge may not be increased significantly because other details may be equally critical.

PRIORITIZING RETROFITTING WORK

It was realized immediately after the 1971 earthquake that existing bridges should be retrofitted in order to increase their seismic resistance. A prioritizing system was devised that assigned weighted values to:

- Type of bearings.
- Width of hinge or bearing seat.
- Restraint of supports.
- Height of structure.
- Type of supports.
- Flexibility of supports.
- Curvature in alignment.
- Probable earthquake intensity.
- Hazard to public on and under structure.
- Disruption to traffic and utilities.
- Danger to buildings or facilities under the structure.

This system worked well for identifying candidate structures for immediate retrofitting. However, the prioritizing numbers obtained did not always reflect the true relative importance of some structures. The input is largely a matter of judgment, but under certain circumstances a single factor might be important enough to justify a high priority regardless of all other factors. A less important structure could rate lower in a number of less important categories but get a higher overall rating. The results from any prioritizing system should be subject to adjustment by good judgment.

There are also practical considerations that can, to some extent, override the strict adherence to a prioritizing system. If a large number of bridges spread over a very large area are identified for retrofitting, there are considerations in contracting and inspection that should not be overlooked. Although there are not any definitive rules that can be followed, there are general quidelines which should be considered. A greater degree of efficiency can be achieved if a number of bridges in one area can be included in a single contract. It is more efficient to prepare plans and let contracts for a few large jobs than a great number of single bridge contracts. A contractor's mobilization costs can be spread out and personnel can be trained and used efficiently on a job with a number of bridges, but a single inspector on too small a job will have time to waste unless he can be given other work to do. For efficiency, it is obvious that bridges in a contract should be located reasonably close together. It is generally true that groups of bridges in different contracts should not ordinarily overlap. If individual structures are prioritized by an inflexible system, it is highly unlikely that structures with nearly equal priorities will be geographically located to form logical contracts.

HINGE AND BEARING RESTRAINERS

An arbitrary decision was made in 1971 that restrainers should be capable of resisting a minimum force equal to 25 percent of the weight of the lighter segment of superstructure connected. Bridge designs at that time were based on working stresses. Column shears were deducted from that 25% on a number of structures and many restrainers had the appearance of being under designed. It was decided to ignore the reduction due to column shears. Dynamic analyses made for many structures since that time have reinforced the opinion that assuming the 25% force and ignoring column shears give reasonable minimum requirements when using working stresses.

When Load Factor Design methods are used, almost identical results are obtained by using 33% of the dead load, yield strength of the restrainers, and ignoring column shears.

In either case, larger restrainer capabilities should be provided whenever required to satisfy dynamic analysis.

A minimum of two restrainers are used at each bent or hinge - one as close as possible to each edge of the superstructure. Restrainers are adjusted to permit normal movements of the joint and to start acting as soon as maximum normal open joint width is exceeded.

All of California's dynamic analyses are based on load factor methods and a ductility factor of one is assumed for cable and bar restrainers.

Slightly different assumptions for restrainer arrangements, foundation conditions, column stiffnesses, abutment restraints, linear or non-linear action of the restrainers and columns, etc., can make drastic differences in the results of a dynamic analysis. In some cases, the computer has given forces in restrainers that were so low, or movements of joints so little, that they did not appear to be consistent with observed actions of structures.

Piers may move with respect to other piers, columns may tilt or move up and down, piers may be accelerated at different times. Basic information is lacking on how modern type bridges react to devastating earthquakes. Too few have been subjected to the severe shaking of actual earthquakes and no strong motion records have been obtained. Physical characteristics of some bridge members are not known and have to be assumed.

Considering all of these uncertainties will lead a practical designer to the conclusion that the seismic analysis of a bridge is a developing art rather than an exact science. A number of analyses should be made and the results tempered with judgment.

The main factors considered in designing retrofitting devices are:

- Adequacy
- Adaptability for attaching to existing structures
- Economy

The ideal restrainer should absorb and dissipate energy, keep joint movements within a safe range, and force the structure back to its pre-earthquake position.

For practical reasons, the most suitable devices for new construction are not necessarily the best for retrofitting existing bridges. We have considered using springs, neoprene pads, bellvue washers, steel bars and rolled sections in tension, bending and torsion, friction devices, hydraulic type dashpots or shock absorbers, steel bars and cables.

Most of our restrainers to date have used steel cables or bars which act as tension members only. These devices may not be ideal from a strictly theoretical point of view and they may not prevent as much damage as other types of restrainers that have been considered but, reviewing all of the factors involved, they are hard to beat. They will raise the level of seismic resistance of a bridge, they are relatively easy to install and they are economical.

It is very unlikely that a great number of existing bridges in California will be subjected to a major destructive earthquake. It is less likely that more than a few, if any, will ever be subjected to more than one major earthquake during their lifetime. Thus it is likely that only a small percentage of restrainers will ever be required to function as planned.

Developing and installing better restrainers will not necessarily keep an older bridge from collapsing completely if subjected to severe seismic shaking. Older bridges often have other features that may lead to premature collapse of the structure even if its hinges and bearings are retrofitted with theoretically perfect devices.

California has used 3/4" pre-formed 6 x 19 galvanized cables (Federal Spec RR-W-410C) with a minimum breaking strength of 205kN(23 tons) as the basic unit for its restraining devices. Swaged end fittings are used that are required to develop the minimum breaking strength of the cable. This type of cable and end anchorage have been used in highway barrier systems for many years. They are being tested on a regular basis and have an excellent performance record.

1-1/4" diameter galvanized steel bars (ASTM Designation A-722 with supplementary requirements) that have a specified minimum elongation of seven percent measured in 10-bar diameters are also being used.

The steel cables and rods can store energy, but transfer it back into the structure as they pull the segments of superstructure back together. Much of the energy is probably dissipated by the pounding of the superstructure elements when they come back together. The damage caused by this action should be repairable and not cause the bridge to collapse.

When the restrainer retrofitting program was started, most bridges were designed by working strength methods. A working load of 50% of the ultimate strength for galvanized cables plus an overstress of 33% permitted for seismic conditions gives a total allowable load of 136kN(30.6 kips) per 3/4" cable. For load factor design methods a yield strength of 85% of ultimate load of 174kN(39.1 kips) per cable is used.

The design yield stress for high strength steel bars is 827kPa(120 ksi) or 667kN(150 kips) per 1-1/4" diameter bar. These bars are particularly useful in cases where it is impractical or undesirable to use the number of 3/4" cables required to obtain the necessary resisting force. Many older bridges that are being retrofitted have shear keys that are inadequate for keeping the two sides of the hinge aligned longitudinally if the structure is subjected to seismic shaking. Since a transverse shearing action at the hinge could cause the rods to fail and become ineffective in tension, supplemental concrete filled steel pipes are installed through the hinges in order to provide additional shear resistance.

There aren't any rules of thumb for predicting whether either cables or bars are better than the other for any particular installation. Dynamic analyses of long and short cables and bars will give the designer information for selecting the proper number and length of either type to keep joint movements within a tolerable range.

California has conducted a number of tests of 3/4" cables and 1-1/4" Ø bars to compare their qualities as restrainers. Figure 1 shows the stress-strain relationship of specimens tensioned from near zero stress to specified minimum yield stress (assumed to be 0.85 fy for cables) for 14 cycles and then to failure.

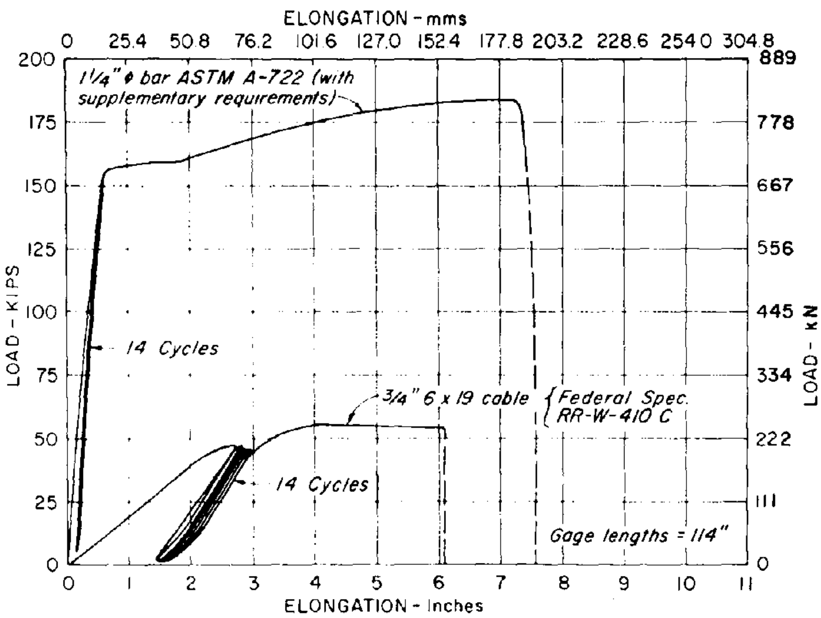


Figure 1

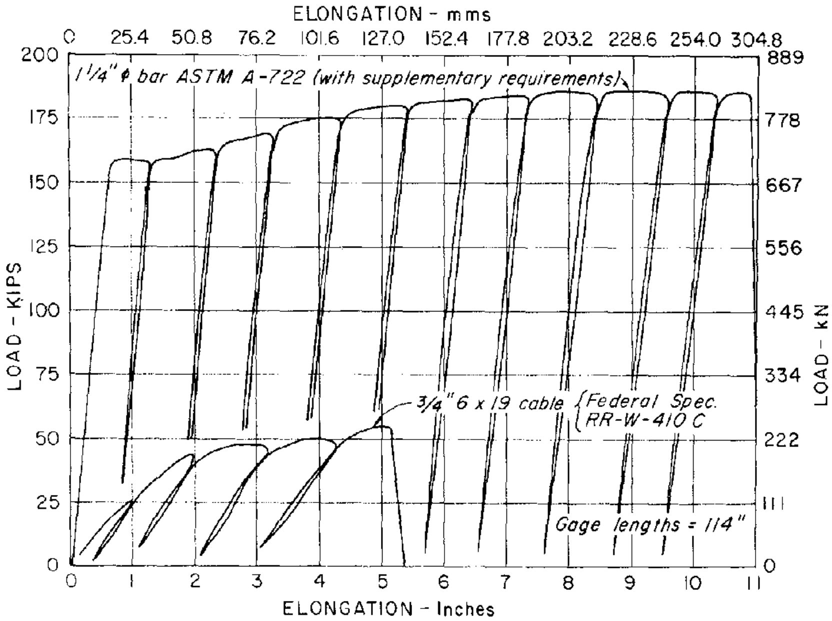


Figure 2

Figure 2 shows stress-strain relationships for cables and bars tensioned to failure but releasing the load to nearly zero at approximately one inch increments of stretching.

Cycling 3/4" cables within the elastic range required more than twice the amount of energy than cycling an equivalent number of 1-1/4" Ø bars of the same length for the same number of cycles. This is due to the fact that bars have a greater modulus of elasticity and the elongation within the elastic limit is less than for cables. Within this range the cables and bars store energy but do not dissipate any significant amount.

The bars stretched and cycled beyond the elastic limit dissipated approximately 3 times as much energy as the equivalent number of the same length cables.

If restrainers are permitted to yield, greater joint openings and column deflections will be realized. Once either type of restrainer is stretched beyond its elastic limit it obviously will not assist in closing the joint to its normal position. Although bars will dissipate more energy than cables when failure occurs, the elongation will also be much greater. This could be an extra factor of safety in some structures but could be disastrous in structures with relatively short, stiff columns. When a restrainer is stretched to its ultimate limit, the structure is vulnerable to any additional shocks.

Considering the impreciseness of predicting a bridge's response to a possible future earthquake, it is generally not prudemt to depend on restrainers acting beyond their elastic limit.

RESTRAINER DETAILS

Figure 3 shows the most commonly used detail for retrofitting hinges of existing concrete box girder bridges. The concrete bolsters are generally required to spread out the concentrated forces of the restrainers so that they don't destroy the hinge diaphragms. A minimum of one 7-cable (1900kN, 428 kip) unit placed in each exterior cell at each hinge is generally considered to be a minimum requirement in order to provide maximum resistance to transverse bending of the entire superstructure.

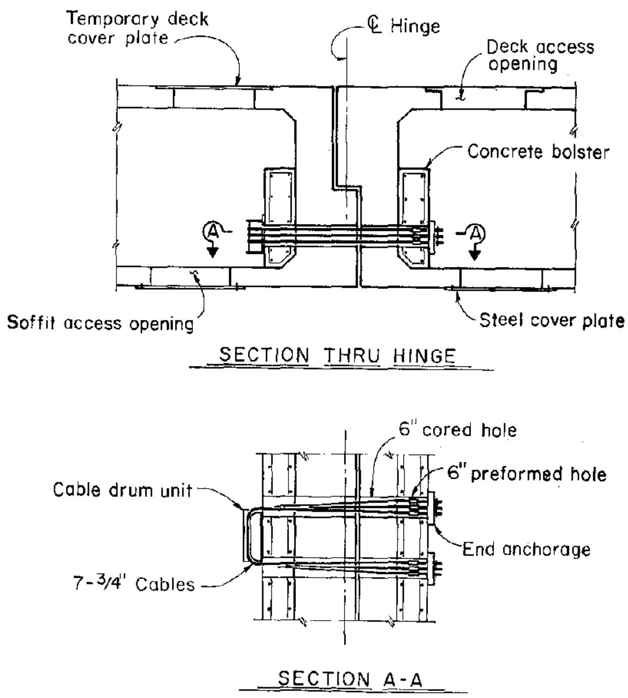


Figure 3

Access to the cells is made through the soffit whenever possible in order to avoid interfering with traffic on the bridge. If access through the soffit is not possible or desirable due to conflicts with traffic under the structure, or other reasons, work is done through deck openings. In this case, traffic handling may become critical and work limited to off-peak hours. Steel plates set flush with the roadway surface are used to carry traffic across the access holes between working periods. Deck access holes must be permanently closed when work is completed. Holes in the soffit are covered with galvanized steel plates that can be readily removed for future inspections.

Figure 4 is a modification of the concept shown in Figure 3. It is generally restricted to hinges and end supports of shorter span T-beam bridges where the restraining force requirements are considerably lower.

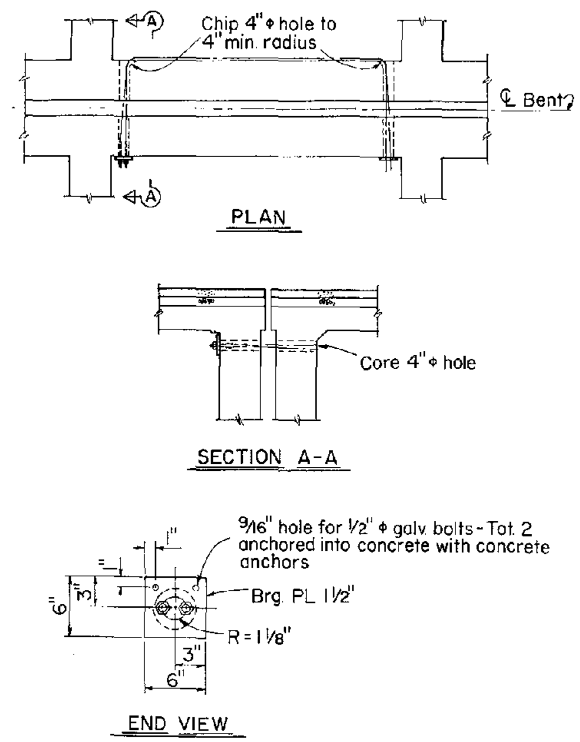


Figure 4

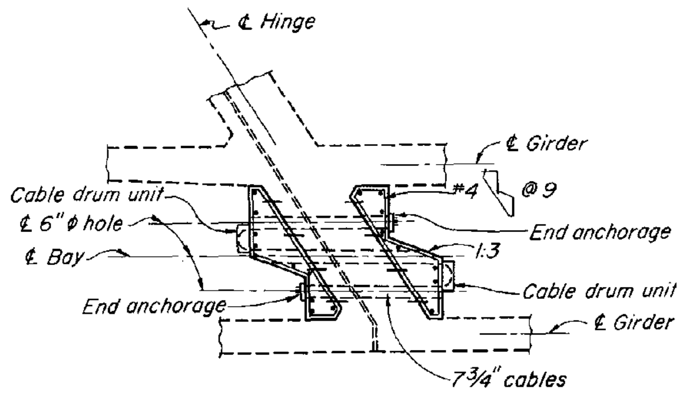


Figure 5

Figure 5 is another modification of Figure 3 and has been used in a few situations where the existing diaphragms are capable of resisting the greater force provided by the seven cables which pass through the joint three times.

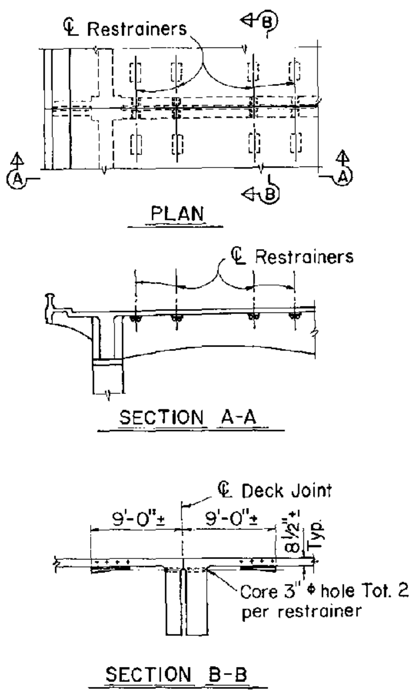


Figure 6

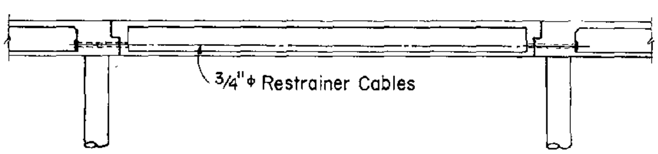


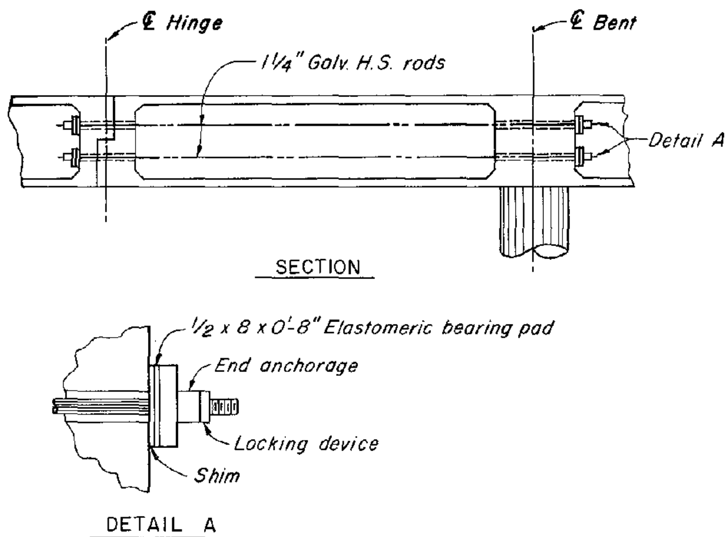
Figure 7

The detail shown in Figure 6 has been used on a limited basis where the diaphragms are not capable of being adequately strengthened and it would have been less desirable to attach restrainers directly to the girder stems. In this particular case it was necessary to place the cable anchorages far enough from the ends of the deck slab so that they would not pull the ends out of the spans.

Variations of Figure 7 have been used in a number of instances where drop-in spans could be expected to fall if the structure were shaken in an earthquake. If the hinge seats are very narrow and the cables very long, additional cables might be required to order to limit the amount of stretching under seismic loading. This method is uneconomical in very long spans.

An installation using high strength rods is illustrated in Figure 8. Cables could also be used in this scheme.

Figure 9 shows a commonly used detail for restraining steel girders which are in line with each other. When girders in adjacent spans are offset, transverse beams are attached to the bottom girder flanges which are used for anchoring the restrainer cables, as shown in Figure 10.



HIGH STRENGTH ROD RESTRAINER

Figure 8

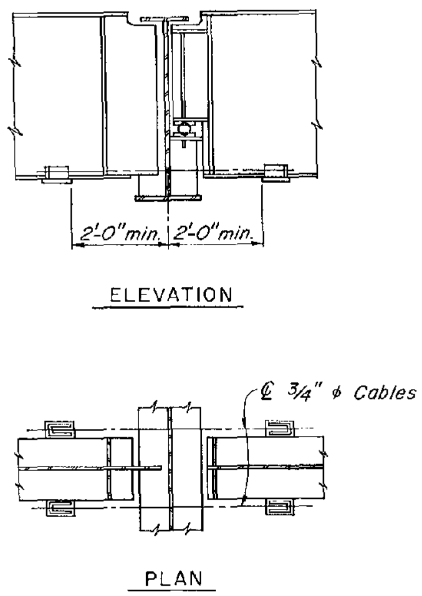


Figure 9

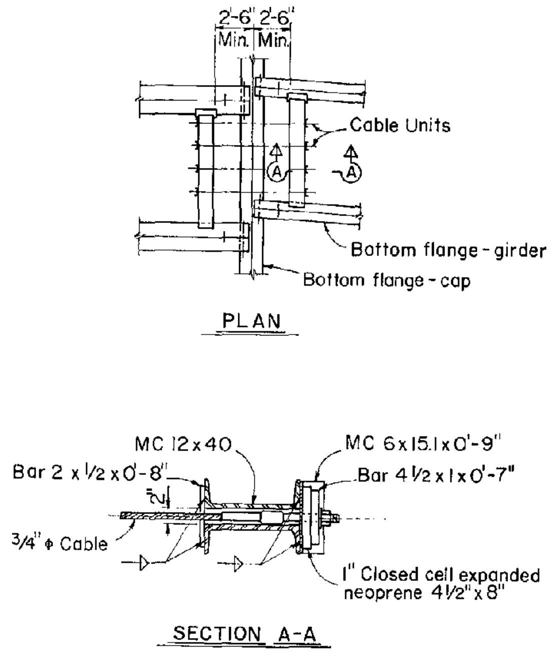


Figure 10

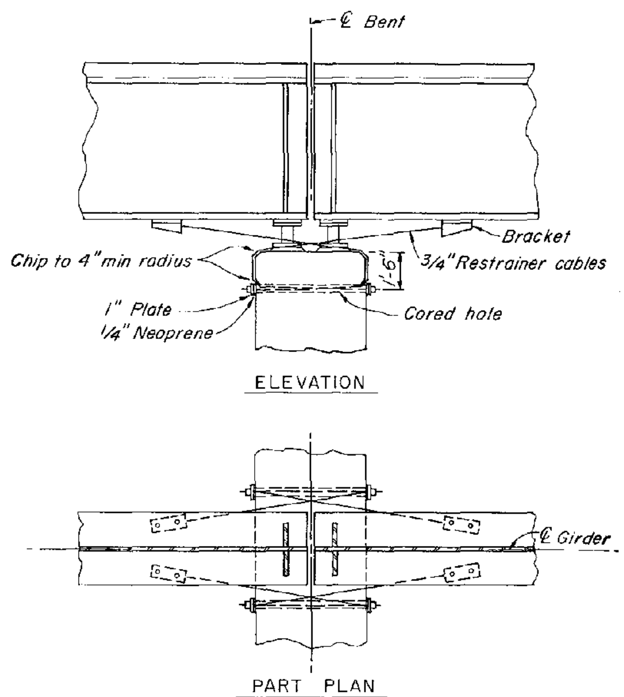


Figure 11

Figure 11 illustrates a method of attaching the ends of steel girders directly to the supporting concrete bents.

The restrainers illustrated above are only a few of the many types we have used to date. Each bridge has its own peculiarities and requires special attention and details.

Costs

The following contract unit prices are taken from a large number of recent contracts that were bid competitively:

	Low	Avg.	High	
Deck access openings	$200.	$230.	$300.	/each
Soffit " "	200.	228.	300.	/each
Miscellaneous metal (cables, fittings, brackets, etc.)	1.50	1.75	5.00	/pound
Core 6" holes	38.	42.	62.	/lin.ft.
Core 4" holes	26.	33.	55.	/lin.ft.
Core 2" holes	18.	23.	30.	/lin.ft.
Diaphragm bolsters	200.	253.	300.	/each
Close deck access oepnings	200.	251.	350.	/each

INSTALLATION OF RESTRAINERS

One of the main problems in connection with retrofitting existing bridges is minimizing interference with existing traffic. It is frequently necessary to limit work to off-peak hours. When retrofitting box girder bridges, the designer is given the option of specifying access to the girders through either the deck or soffit. Deck and soffit openings are generally made quite close to the hinges where tensile stresses in the girder reinforcement and compressive stresses in the concrete are relatively low, but far enough away so that the openings are not an inconvenience to the workmen.

Steel cover plates are generally required over the deck openings to provide for traffic during non-working hours. The 5/8" thick cover plates were placed on top of the deck in earlier contracts but were found to be hazardous to certain vehicles. Plates are now required to be recessed into the deck so they provide a flush riding surface. After work inside the girder cells is completed, extensions are welded to the ends of the cut reinforcing steel in the deck, to provide lap splices, and the opening is filled with concrete.

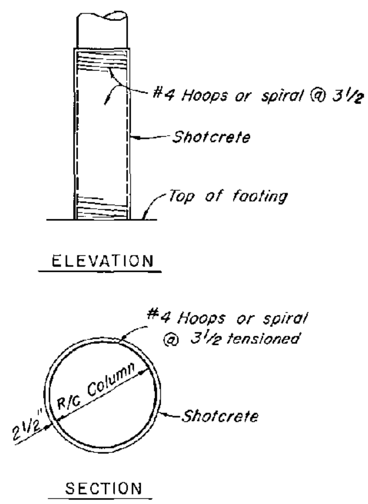
It is not considered necessary to replace reinforcement and concrete in soffit openings. Exposed ends of the reinforcing steel are painted with zinc-rich paint and a galvanized steel plate bolted over the opening.

Some contractors have expressed a preference for doing all of their work through the soffits whenever possible, in order to avoid conflicts with traffic on the bridge deck. Present equipment allows them to work as much as 100 feet from ground underneath a structure. A preference has also been expressed for gaining access to a temporary platform suspended underneath narrower structures from the bridge deck.

RETROFITTING COLUMNS

The second greatest weakness of Pre-1971 structures pointed out by the San Fernando earthquake was that the reinforcing steel ties in columns did not provide adequate confinement of the concrete. Bridges with single column bents are particularly vulnerable. Since the restraining of the superstructure at hinges and bearings was judged to be a more serious problem; and providing that restraint alleviated the seriousness of the column deficiency, more can be obtained for the money by retrofitting the hinges and bearings first. Methods of retrofitting columns to make them more earthquake resistant are being investigated and a developmental contract will be let for trying out these schemes.

All bridges that might require column retrofitting are currently being identified. When the developmental contract is completed a program to retrofit the columns of some of the State's more critical structures will be considered.



COLUMN RETROFITTING

Figure 12

Figure 12 illustrates reinforcing steel hoops that are prestressed on the outer face of the column which is then covered with shotcrete. The device shown in Figure 13 was especially designed for this purpose. It is basically a turnbuckle that develops the strength of the reinforcing steel and places an initial pre-stress in the hoop.

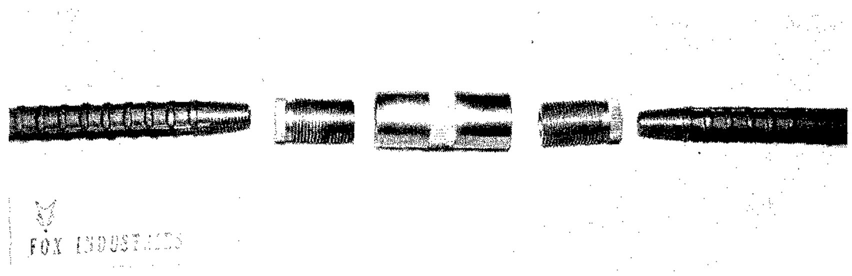


Figure 13

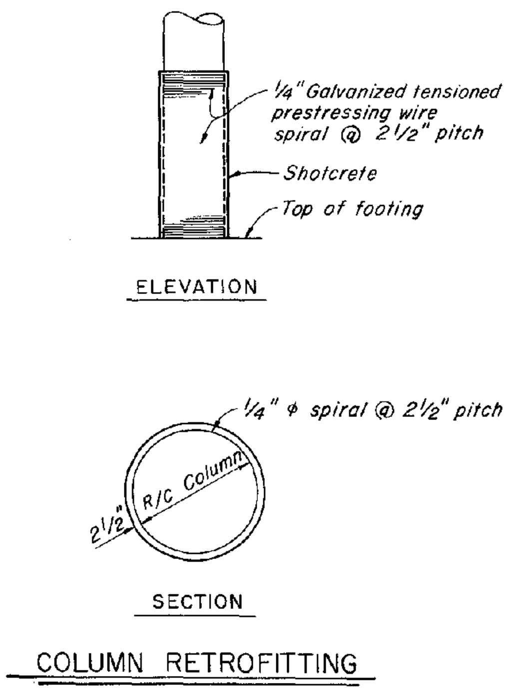


Figure 14

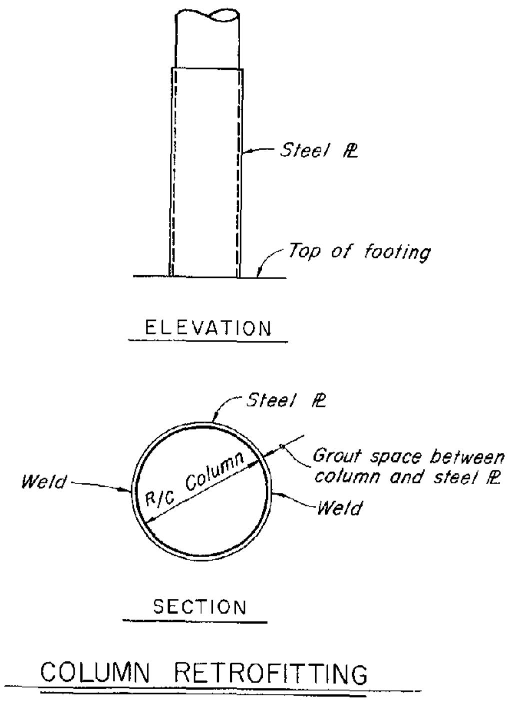
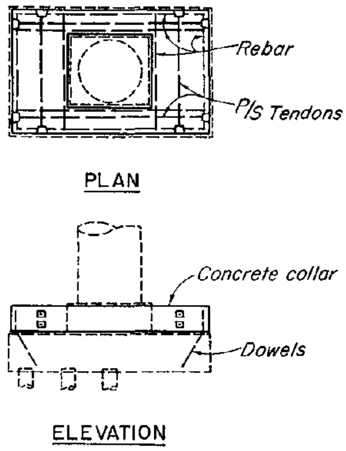


Figure 15

The column retrofitting method shown in Figure 14 consists of wrapping a column with tensioned prestressing wire and applying a protective coat of shotcrete.

Figure 15 illustrates a method that consists of welding a steel shell around an existing column and filling the space between the shell and column with grout. "Weathered" steel can be used for achieving an architectural effect, if desired, or ordinary steel can be used and painted.



COLUMN-BASE RETROFITTING

Figure 16

The structure selected for developing methods for retrofitting columns also has the main column reinforcement lap-spliced in the footing bolster. The detail illustrated in Figure 16 shows one proposal which is being considered to confine the concrete and strengthen the splices.

RETROFITTING OF EXISTING HIGHWAY BRIDGES SUBJECT TO SEISMIC LOADING ANALYTICAL CONSIDERATIONS

by

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ABSTRACT

This narrative summarizes the results of two studies supported by the Federal Highway Administration. The objective of the first was to identify practical techniques and criteria for retrofitting existing bridges so as to increase their resistance to seismic forces. This was done on the basis of analyses of actual bridges located in different parts of this country. The second study had two main objectives: (1) to prepare design details and installation specifications for retrofit measures that are to be employed on existing highway bridges to minimize earthquake damage; (2) to demonstrate an approach in the application of the seismic analysis technique which can be used by the practicing bridge engineers to decide whether a bridge needs retrofitting and if it does, what type of retrofit measure(s) to employ. To meet these objectives the study produced:

- a simplified structural analysis method for deciding if a given bridge is expected to survive a postulated earthquake,
- a procedure for deciding if a potentially weak bridge in a given road network warrants being retrofitted,
- examples of bridge retrofit measures on the basis of which actual retrofit concepts may be developed to preclude catastrophic collapse of bridges.

Corresponding results are summarized with conclusions and recommendations for further research.

RETROFITTING OF EXISTING HIGHWAY BRIDGES SUBJECT TO SEISMIC LOADING ANALYTICAL CONSIDERATIONS

by

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INTRODUCTION

The work described in this narrative was at least in part motivated by the damage that was sustained by highway bridges during the February 9, 1971 San Fernando earthquake. This earthquake clearly pointed out a number of deficiencies in bridge design specifications existing at that time. It also focused attention on the fact that numerous existing bridges may be expected to fail in some major way during their remaining life if subjected to strong motion seismic loads. Bridge failures are clearly undesirable since a bridge may be a vital link in a road network. When a portion of the road network is disrupted by the collapse of a bridge, vital services to the surrounding communities are impaired for the time required to repair or replace the bridge or to find an alternate route. Depending on the extent of other physical damage and casualties produced by the earthquake, the loss of vital bridges, i.e., those that provide access to hospitals for example, can magnify the effects of a disaster.

Although a great deal of research work has been, and continues to be done in this area of design of structures subjected to seismic motions, surprisingly little of this has been directed to the evaluation of existing bridges located in high risk seismic areas. The task of deciding if a given bridge, designed using currently outdated criteria, requires retrofitting is generally more complex than the one which deals with the design of a new bridge. In such an effort the engineer first needs to determine: (1) the physical state of the bridge based on engineering drawings and field inspection; and (2) the response of the bridge when subjected to the probable earthquake motions occurring at the bridge site. On the basis of this information he then decides if the bridge is structurally sound and what retrofit measures (if any) to employ, and in what manner. When a decision to retrofit is made and a preliminary retrofit measure has been selected, there follows a structural reevaluation to determine the influence of the candidate retrofit on the response of other portions of the bridge. The procedure is therefore iterative. When a satisfactory retrofit is found, a design effort, concerned with the sizing of new structural members, preparation of drawings and implementation specifications follows.

It is in the design stage that AASHTO specifications mostly apply. AASHTO specifications do not contain specific provisions for bridge retrofitting. For example, there are no criteria for computing bridge motions produced by an earthquake nor provisions to limit such motions to a given maximum value. Criteria for computing uplift forces are also not provided.

Following the February 9, 1971 San Fernando earthquake, the Federal Highway Administration (FHWA) launched a study [1] whose objective was to identify and define practical techniques and criteria for retrofitting existing highway bridges so as to increase their resistance to seismic forces. This was followed by another effort which produced a design reference manual [2,3]. The objective was to illustrate retrofit concepts that can be applied to existing bridges, which will enhance the probability of survival of the structure when it is subjected to postulated seismic motions. This narrative is based on the results of these two efforts.

BRIDGE RETROFIT DECISION PROCESS

In deciding if a given bridge should be retrofitted three steps should be considered:

1. Will the bridge suffer a critical failure (i.e., so extensive that the bridge could not remain in even emergency use) if subjected to the probable earthquake ground motions for the bridge site? If the structural analysis produces a negative answer to this question one need go no further. If the answer is affirmative the second step is:

2. Determine the level of importance of the bridge to the given locality by considering the type of highway, traffic volume, accessibility of other crossings, etc. If it is determined that the bridge is unimportant to the locality, it may be decided that retrofitting is not feasible even though the answer to step 1 was affirmative. If however, it is decided that the bridge is important to the area, the third step is:

3. Determine the type or types of retrofit measure(s) to employ.

The manner in which these steps can be accomplished is described in subsequent paragraphs.

ANALYSIS OF EXISTING BRIDGES SUBJECTED TO SEISMIC LOADS

Preliminary Screening

Earthquakes generally affect relatively large land areas and therefore a large number of highway bridges may be involved in any one case. When dealing with large numbers of bridges, the evaluation of their structural integrity for the purpose of deciding if retrofits are warranted, can be a time-consuming and costly task. It is therefore desirable to have a ranking procedure that allows for the classification of bridges based on a simplified screening process involving an examination of bridge plans, design specifications and other relevant data, visit to the bridge site, etc. When the ranking is complete it would then be possible to place each bridge into one of four categories:

Structural Category	Structural Factor
Unsound, i.e., certain failure	0
Probably unsound	1
Probably sound	2
Sound, i.e., certain survival	3

Such a procedure was developed and is facilitated by the use of Table I which relies heavily on engineering judgment and is based solely on observed bridge damage caused by past earthquakes. An inherent assumption in using this procedure is that the structure is in fact sited in a region that is capable of generating potentially damaging ground motions during the life of the bridge.

Table I

STRUCTURAL FACTORS FOR EXISTING BRIDGES*

Structural Factor (SF)	Bridge Type and Details
0	Multiple span bridges with simple spans. Bridges with two hinges in one span or hinges in adjacent spans.
1	Continuous span bridges with one hinge or with hinges separated by at least one span.
2	Simply supported spans with continuous, composite deck slabs. Continuous reinforced concrete slab bridges that have spans that will not fall down due to dead load only if hinges become unseated. One, two or three simple-span bridges with high backfilled or bin type abutments.
3	Rigid frame bridges. Single span bridges. Continuous, multiple span bridges without expansion joints.
Note:	Bridges with skewed hinges or bearings or other features that increase the vulnerability to shaking should be given a more critical SF.

*Provided by O. H. Degenkolb, Senior Bridge Engineer, California Department of Transportation, Sacramento, California.

Table I should be considered as preliminary as it considers general bridge types and is intended as a general guide only. It may be advantageous in a future effort to incorporate additional structural conditions in this classification such as foundation type, structure alignment, column or pier flexibility, skew, etc. The use of Table I should be supplemented with additional structural analyses whenever any doubt exists.

Bridges falling in SF categories 0 and 3 pose no serious problem for the analyst or the planner since a definite statement has been made about them. However, in some cases, depending on the importance of the bridge, further analysis may be required. Bridges falling in SF categories 1 and 2 should be further analyzed to determine if a more definite statement as to their structural integrity could be made. To this end the analyst may need a simplified method of analysis to further refine the previous classification. Should the results from the simplified analysis prove marginal, i.e., not capable of providing a definite answer, then more detailed analyses should be performed.

Simplified Analysis Methods

IITRI simplified analysis method--As part of the study reported [1] IITRI developed a simplified method of bridge analysis. It was subsequently modified as part of a validation effort reported in [2]. A detailed description of this method is given in [2].

In this analysis method, the predominant mode for a given bridge is assumed to be horizontal and can be resolved into two orthogonal directions, longitudinal and lateral. If the lateral restoring force or resistance of the structure is considerably larger than that in the longitudinal direction, then the lateral response does not govern and can be ignored. Vertical vibrations are also important for some bridges and for piers rigidly connected to the girders and are considered. These four assumptions are made concerning structural idealization:

1. Rollers and expansion joints are considered to be frictionless.

2. Expansion joints are assumed capable of transmitting longitudinal forces only if longitudinal ties are provided through the joint.

3. Skewed bridges are analyzed as if they were unskewed (i.e., the longitudinal stiffness is assumed to be perpendicular to the skewed piers).

4. Horizontally curved bridges are analyzed by converting the structural properties into the chord line direction (which is nominally referred to as the longitudinal direction) and perpendicular to the chord (lateral) direction.

Each bridge is idealized separately in the longitudinal and lateral directions as single degree of freedom systems. This is done by determining an equivalent mass and spring stiffness for the bridge in each of the two directions by combining the individual stiffness from the various contributing bridge components. Procedures for determining equivalent masses and individual stiffnesses and for combining them are provided in [2].

Seismic loading is specified by selecting appropriate base ground accelerations at the bridge site. For a given geographic region, base ground accelerations may be selected using seismic risk maps such as that developed by Algermissen and Perkins [4].

The response spectrum used is the Newmark-Blume spectrum [5] for $\mu=1$ and an appropriate value of damping modified for the particular base ground acceleration representing the given bridge site.

From the equivalent lateral and longitudinal stiffnesses, an artificial bridge frequency is computed by hand. Using this frequency, a horizontal displacement and acceleration are obtained from the response spectrum. These values are then modified to the maximum horizontal elastic acceleration or displacement of the bridge site by multiplying these values by the peak acceleration at the site. Seismic forces are then obtained by multiplying the longitudinal (lateral) acceleration (as obtained above) by the equivalent mass.

It is assumed that the bridge will fail catastrophically if the analysis indicates that any of these four conditions will occur:

1. The anchor bolts of fixed bearings fail by shear.

2. A plastic hinge is formed at the bottom of a pier that is hinged at the top if no additional lateral stability is provided by the adjacent piers or abutment.

3. Piles are subjected to excessive lateral forces which create large horizontal displacements of the structure. When vertical piles in good soil are subjected to lateral forces of more than 15 kips (each) such lateral force magnitudes are considered to be excessive. Clay is considered to be a poor soil for providing lateral resistance for piles.

4. Slipout of pins in hinge connections or bearings due to excessive horizontal or vertical relative motion.

It is assumed that catastrophic failure will not occur if the analysis indicates that the following overloading and/or yielding conditions will occur:

1. A plastic hinge occurring at the bottom of a pier that is fixed at the top as long as the pier is not an isolated one between expansion joints.

2. The bending moment for a reinforced concrete pier (based on the elastic response spectrum seismic loading) does not exceed three times the ultimate moment of the section. This is based on the fact that the seismic acceleration will be reduced by a factor of $1/\sqrt{2\mu-1}$ where μ is the ductility factor. For reinforced concrete, a value of $\mu=5$ is commonly used.

3. If, due to vertical vibrations, a plastic hinge forms at the top of a pier which is monolithic with the superstructure and the pier is framed into a transverse superstructure diaphragm. Failure will not occur as long as no plastic hinge develops at the bottom of the pier.

This method of analysis was the subject of a validation effort involving bridge structures previously analyzed and reported in [6]. This reference document contains 15 cases of seismic analysis of bridges. Using the same physical data, the IITRI simplified analysis method was used to analyze these structures subject to the same loading conditions as [6] and was then compared to results given in [6]. The subject bridges are actual structures currently located in the State of California. Analysis performed on them in [6] all involved using the local response spectra relative to the fundamental periods of the bridges. Three methods were recommended for computing the fundamental period. These are referred to as the formula method, the uniform load method and the dynamic analysis method. They are jointly referred to here as the California Highway Department (CHD) methods and are briefly described.

CHD analysis method--The formula and uniform load methods are essentially static analysis methods which involve four steps:

1. Calculate the period of the first mode of vibration in the direction under consideration.

2. From the CHD response spectra obtain the corresponding response coefficient "C" and compute the equivalent earthquake force $EQ = C \cdot F \cdot W$ [9] (where F = framing factor, W = deadweight of the structure).

3. Distribute the resulting earthquake force to structural elements.

4. Perform the design/analysis.

In the formula method each bent of the bridge is taken as carrying the weight of half of the span on each side and thus has its own frequency. This results in a concentrated force applied at the top of each bent. Transverse bending and torsional stiffnesses of the superstructure are assumed not to contribute to the stiffness of the system.

In the uniform load method the continuity of the superstructure is maintained and the natural period of the system is computed on the basis of an assumed mode shape. This results in a uniform static load applied along the superstructure. For complex structures using this method to find the fundamental frequency is no better than modeling the structure for computer analysis. The dynamic analysis method is a standard, computerized structural analysis method which determines modes and frequencies.

Summary and conclusions of the validation--The 15 case studies included in [6] are a reprsentative mix of the following bridge parameters, i.e., number of spans, span length, number of columns per bent, column length and fixity, curvature, skew, structure width.

Using the information provided in [6], the IITRI simplified method of analysis was applied to each of the 15 cases to compute the period of the structure, and shear and bending moments at the pier columns. The input motion used for the comparisons was represented by a 0.7 g maximum bedrock acceleration. A damping factor of 5 percent of critical was used and a reduction factor of 8 due to ductility and risk was applied to the values obtained from the Newmark-Blume response spectrum. A comparison of results is given in [2]. Based on this comparison it is felt that the CHD method has the advantage of providing reasonable design forces based on the fundamental period calculated by means of computer modeling and the CHD response spectra. However, it has the disadvantage that computer modeling needs to be used and only the design force in lateral vibration is provided. Longitudinal and vertical vibrations are not considered and displacements of the structure are not provided. The IITRI method has the disadvantage that the period obtained by hand computation is artificial. However, it has the advantage of giving reasonable forces and displacements for vibration in any of the three principal directions.

It is felt that this method of analysis is a potentially adequate tool for deciding if a retrofit is required in any one instance and what force magnitudes it is to resist. For cases where the results as to the need for retrofitting are not conclusive, other more detailed methods should be used.

Detailed Analysis Methods

The unique aspect of earthquakes is the fact that motions generally persist for a "long time" relative to the natural period of bridge components. A component can therefore experience many load cycles of varying magnitude during the passage of the earthquake. Since the component may be forced to respond in the nonlinear range of its material or experience nonlinear displacements, analysis methods for the response of bridges subjected to earthquakes should have the capability of considering both linear and nonlinear behavior.

Nonlinear numerical methods of analysis have received a good deal of attention in recent years but the majority of the researchers have restricted their studies to the investigation of certain types of structures or nonlinearities. The structural failure of a bridge due to earthquake loading is an important nonlinear problem that has not been adequately studied for a number of reasons. First, a comprehensive and accurate solution scheme that properly treats the material and geometric nonlinearities of the problem is not readily available or it has not been developed with the design professional in mind. Second, any such computer code requires that the user possess a good deal of background knowledge in the complex subject matter of nonlinearities. Third, and probably most important, the cost can be prohibitively high for obtaining solutions to even seemingly simple problems involving relatively few (less than 100) nonlinear finite elements to model a bridge subjected to relatively long (20 to 100 sec) seismic loading duration. These barriers are usually sufficient to discourage the desired investigation of bridge structures subjected to seismic loads. For this reason, relatively few comprehensive nonlinear studies of bridge response to seismic loading have been undertaken to date. Undoubtedly this shortcoming will be eliminated some time in the future. Unfortunately, a realistic appraisal of the failure phenomenon for bridge structures subjected to the complex loading associated with seismic environments is not currently known.

At the outset of the research reported in [1] a review of the existing general purpose computer codes that were available in the public domain indicated that they could not be readily applied to the above problem area. For this reason, a special purpose computer code was developed which was addressed to the nonlinear dynamic response of a space frame subjected to arbitrary transient motion loading of the support points. With this code, the user can comprehensively model reinforced concrete beam elements that are subjected to strains beyond the yield value and even up to failure without having to resort to the yield surface concepts employed by plasticians. Furthermore, the complex problem associated with the analysis of expansion joints that undergo relative motion large enough to cause impact is treated by the user by merely specifying the characteristics of the expansion joint such as joint gap, impact stiffness, longitudinal restrainer parameters (if restrainers are provided), etc. Since expansion joint damage can be massive during a severe earthquake loading and precipitate major failure modes, the importance of an adequate treatment of this essential component of a bridge cannot be overemphasized. The general characteristics of this computer code are described in Appendix A. This also includes its application to the analysis of a two-span continuous reinforced concrete box girder bridge.

SELECTION OF CRITICAL BRIDGES

This section summarizes a method [2,7] which was developed to decide whether or not a given bridge warrants being retrofitted. The method is based on a concept in which the criticality (i.e., worth) of a given bridge is compared to its structural integrity in resisting a stipulated earthquake motion environment. The criticality of the bridge is evaluated by considering the effects associated with the loss of the bridge with regard to the highway system and the local community, ability to provide emergency services, the national security/defense network, and the recovery of the area following the earthquake. In this method, the worth of a bridge is evaluated in the following terms.

Administration/transportation system effects--The City, County, State and Federal Highway Departments classify roads and streets. These classifications result from plans which relate the importance of roads and streets to the normal and emergency transportation needs of the community and the nation.

Social/survival effects--These effects involve the ability of the community to meet its short-term emergency needs following a disaster. Normally the social/survival effects are considered in the administration/transportation system plans for roads and streets. There are instances, however, in which those plans have been dated by changes within, or near the community (e.g., a subdivision was added, a hospital was built); or the effect on the community was not considered in the original plan.

Security/defense effects--These effects concern the importance of the bridge in regard to its ability to move toops and equipment to, and within, an area to maintain law and order or to meet a threat to the security of the area, region or nation.

Economic/personal effects--After the emergency has passed, the recovery process begins. The economic/personal effects relate the need for the bridge to the ability of the community to return to its predisaster social and business status.

Each measure of worth (social/survival effects) has measures of effectiveness (medical support) which relate to the need for the bridge. (As an illustration: if a bridge is out of service in a community where the only ambulance route may be blocked such that disaster victims may be unable to reach the hospital.) These measures of effectiveness can be grouped, quantified or qualified to allow a relative assessment as to the worth of the bridge (an absolute assessment would require all of the measures of effectiveness be identified and quantified).

The four measures of worth can be interrelated to allow an overall assessment to be made of the need for the bridge. After all of the bridges within an area of interest are thus evaluated they can be segregated into two categories: a) retrofit is required (to enable the bridge to withstand the earthquake); and b) retrofit is not required (either the bridge can withstand the earthquake or the loss of the bridge does not justify the retrofit expense).

The task/decision flow for deciding if a given bridge warrants being considered for retrofit is illustrated in Figure 1. In the procedural flow diagram, the work elements are rectangular and the decision elements are diamonds.

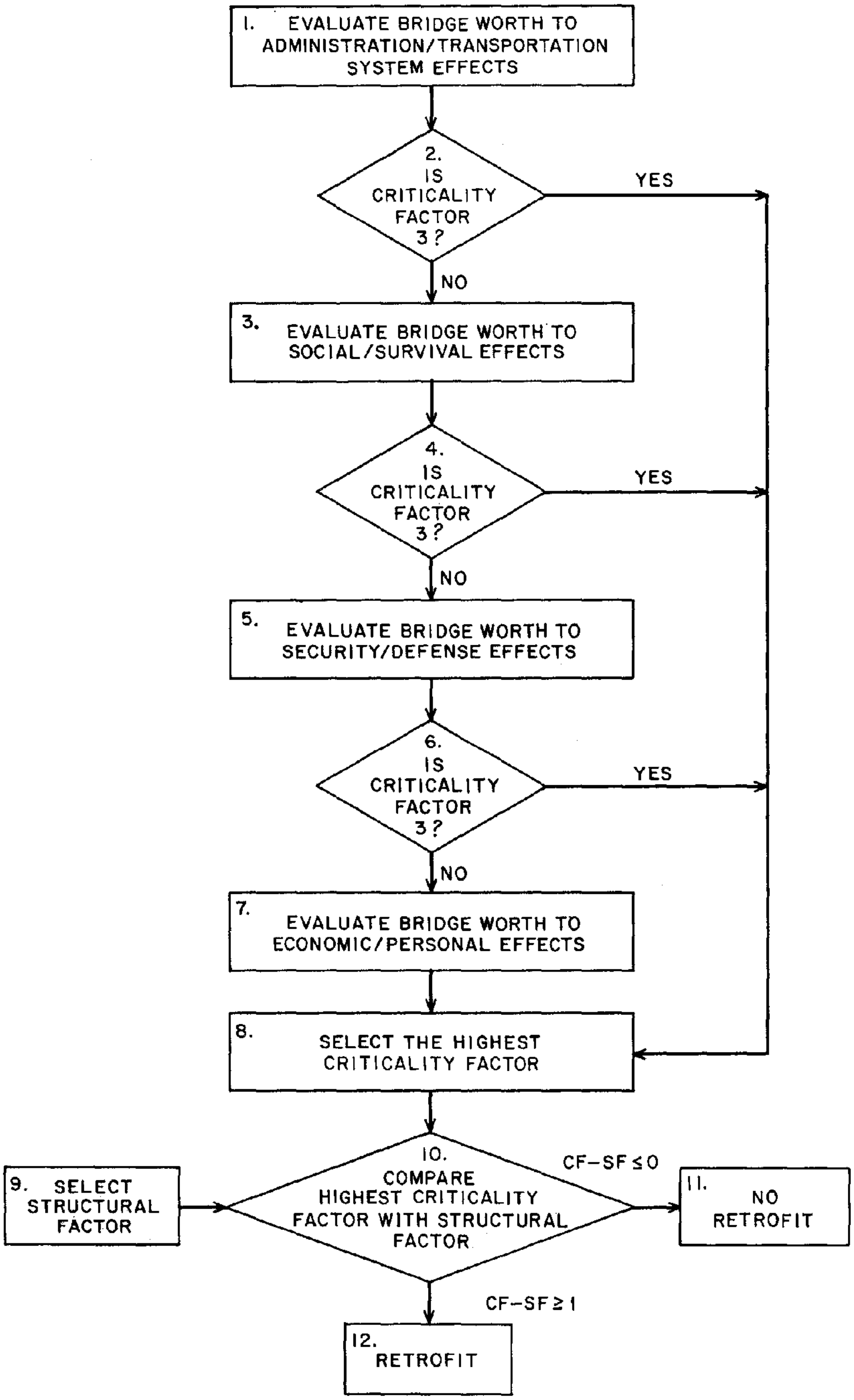


Figure 1. Bridge Retrofit Warrant Procedural Flow Diagram

The measures of worth are evaluated in sequence (elements 1, 3, 5, and 7). The output of each work element is a criticality factor (CF), rated from 0 to 3, which relates the worth of the bridge to the particular measure of worth. The higher the CF the more the bridge is worth to that particular element. As an illustration, the bridge may be evaluated in terms of its worth to the social/ survival effects (element 3). A CF of 3 will indicate that the bridge is critical (e.g., it is the only ambulance route), a factor of 2 indicates that the bridge is desired, CF=1 indicates that it is convenient and CF=0 indicates that it is expendable.

Decision points (elements 2, 4, 5, and 10) are inserted between work elements. These decision points act as filters to reduce the effort needed to make a warrant decision. At each of the first three decision points (elements 2, 4, and 6) a CF of 3 (CF=3) indicates that the bridge is of sufficient worth to merit an evaluation. If a CF of less than 3, (2, 1, 0) is determined the evaluation process is continued until work element 8 is reached.

In work element 8 the highest CF of the four measures of worth (elements 1, 3, 5, and 7) is selected. Similarly, a SF is selected (element 9) which is related to the ability of the bridge to survive damage from the earthquake. The SF are graded over the same range as the CF. Thus SF=3 indicates that the bridge is sound, SF=2 indicates the bridge is probably sound, SF=1 indicates the bridge is probably unsound and SF=0 indicates that the bridge is unsound.

The CF and SF are then compared (CF minus SF). If the result of this subtraction is less than or equal to zero, the retrofit of the bridge is not warranted. If the difference is greater than or equal to one, then retrofit should be considered.

BRIDGE RETROFIT MEASURES

Retrofit measures were selected [3,8] on the basis of the type of failure modes and damage experienced by highway bridges in previous earthquakes.

Observed failure modes can be grouped into two categories, i.e., substructure (pier or abutment) failures and hence loss of superstructure support capacity, and superstructure collapse due to excessive relative motion at the support bearings. Both of these types of failure occurred during the San Fernando earthquake.

Structural failures and damage to bridges are also caused by inadequate foundation strength or load-bearing degradation during the course of seismic loading. Soil liquefaction is an example of this failure mode.

Severe motion of the soil supporting the foundation can cause large horizontal and vertical deformations of the support point. These transient motions create relative movement between the support points which can lead to failures described above. The types of damage that most often occur to bridge components during strong motion earthquakes are:

- displacement and tilting of piers
- displacement, cracking and dislodging of superstructure girders
- displacement, settlement and tilting of abutments

- concrete crushing or reinforcement failure at supports
- bearing anchor bolt pullout or shearing deformation
- settlement, sliding and tilting of wingwalls
- bearing instability and failure
- expansion joint damage
- settlement of approach slabs

Based on highway bridge damage observed in previous earthquakes, eight retrofit measures were identified:

1. Concrete box girder hinge longitudinal restrainer
2. Girder longitudinal displacement stopper at abutment
3. Steel girder vertical displacement restrainer
4. Steel girder hinge expansion joint longitudinal restrainer
5. Girder bearing area widening
6. Pier footing strengthening
7. Reinforced concrete bent column strengthening
8. Steel girder pin bearing vertical and lateral displacement restrainer.

Each of these retrofit measures is addressed to increasing either the rigid body stability of the superstructure or the strength of the substructure. Thus the retrofit measures, if appropriately designed, will enhance bridge resistance to the dominant failure modes that have been observed in previous earthquakes.

Since the emphasis of the study reported in [3] was on demonstrating the design of retrofit measures rather than on the analysis of bridges, the seismic forces that the various retrofit measures were designed to resist were not determined by analyzing each bridge subjected to a site dependent probable earthquake. Instead, the seismic forces were determined in a very simple manner.

Horizontal restrainers were designed for a force of 0.25 times contributing dead load. This method is somewhat more conservative than the AASHTO Interim Specification since it does not include a reduction in the design force for the column shear due to the earthquake load. For a simply supported span fixed at one end and free to translate at the other, the contributing dead load is the total superstructure weight at the fixed end for the longitudinal seismic loading and one-half of the superstructure weight at each end for transverse loading. Other examples of contributing dead load are given in [9].

Vertical motion restrainers connected between the superstructure and the substructure across the bearings were designed to withstand a separation force equal to 0.10 times the bearing dead load reaction. This criterion has been obtained by interpreting article 5.3 of the English version of the Japan Road Association Specifications [10] which state:

> "The vertical design seismic coefficient for the design of connections between superstructures and substructures shall be assumed as 0.10. When the vertical design seismic coefficient applies upward, only

seismic forces shall be considered, neglecting the effects of the dead loads. The same value of the vertical design seismic coefficient shall be employed for the design of any connections similar to the above."

In an actual bridge retrofit effort it is expected that the bridge engineer will obtain seismic forces from a seismic analysis of the bridge.

To keep the illustrations simple, only one component of earthquake motion was considered with each retrofit concept. Obviously in the actual case of bridge retrofit analysis, all three components must be considered. Some representative bridge retrofit measures are included in Appendix B of this narrative.

APPENDIX A: NONLINEAR BRIDGE ANALYSIS CODE FOR SEISMIC LOADING

The IITRI bridge analysis computer program was developed for the purpose of analyzing the dynamic structural response of highway bridges when subjected to ground motions produced by an arbitrary earthquake. Its specific purpose is to evaluate the merits of various retrofit measures that would be employed in a given bridge for the purpose of eliminating or reducing a structural damage. The program is based on the finite element method of structural analysis and models a given bridge as a three-dimensional (space) frame. As such, six degrees of freedom are allowed per node.

The method employs a nonlinear dynamic response analysis of the structure. An implicit integration solution scheme employing equilibrium checks and an optional iteration procedure is used to solve the incremental form of the equations of motion. A tangent stiffness matrix for the complete structure is reassembled at user defined arbitrary increments of the integration step. This feature, coupled with the equilibrium checks and the stable implicit integration technique, permits one to feasibly obtain realistic solutions to intermediate size problems (up to 1000 degrees of freedom) subjected to long duration loading (up to 50 sec). The most outstanding feature of the computer code is its ability to model the reduction in load carrying capacity of a member subjected to relatively small overstressing for cyclic loading. This is a most important behavior characteristic which is believed to be largely responsible for structural component failures, such as reinforced concrete columns. The elastic-plastic yield surface technique usually employed to model beams subjected to overstressing does not readily lend itself to conveniently modeling this behavior especially for members of complex cross section subjected to combined thrust and bending loading about two axes.

A soil-structure interaction finite element is provided in the program to model the connection of the bridge structure to the ground. This element simulates the connection by employing three translational and three rotational springs and corresponding viscous dampers. The prescribed seismic motion is imposed at the ground node point of the soil-structure interaction finite element. At the users option, different spring stiffnesses can be employed for the compression and tension translational degrees of freedom. This feature is quite useful for seismic loading, due to the cyclic nature of the imposed motion, since it permits one to use a high stiffness for compression interaction and a low or zero stiffness for tension interaction for stimulating a footing on piles or a spread footing.

A nonlinear expansion joint finite element (Figure A.1) patterned after that in [11] is included to provide the analyst with a means of accurately modeling connections between different spans of the superstructure, superstructure-abutment (or pier) interactions, and hinges. It models the expansion joint gap, includes Coulomb friction and variable spring rates to realistically simulate differences in compression and tension resistances either horizontally or vertically. Longitudinal restrainers or tie bars with a gap can also be modeled with the expansion joint element.

A three-dimensional elastic beam finite element is also provided in the computer code to model those components of the structure that will not be subjected to severe overstressing and yielding due to the dynamic loading. It is generally employed to model the majority of the superstructure. Shear deformations are optionally included in the elastic beam element stiffness and stress-displacement matrices.

The computer code treats the nonlinear response of overstressed members by employing a special beam element that realistically models the behavior in a simple to use automatic manner that is particularly adapted to numerical analysis. Recalling the formal development of a finite element beam model, the constitutive equations of the material enter the analysis through integrals over the beam cross section which have the form

$$\int_A \sigma(\varepsilon)\ dA, \qquad \int_A \sigma(\varepsilon)\ y\ dA, \qquad \int_A \sigma(\varepsilon)\ z\ dA$$

where $\sigma(\varepsilon)$ is the stress at any fiber location (y,z) and ε is the associated strain which in turn can be obtained in terms of extension, ε_o, and the curvatures k_{oy}, k_{oz}. The method employed in the computer code evaluates these integrals by numerical integration over the cross-sectional area at each stage in the deformation. The user subdivides the cross section for each of these beam elements into a finite number of incremental areas (Figure A.2). Each of the subdivided areas is specified as a particular type of material (such as concrete or steel) and as such has a certain nonlinear stress-strain behavior which is also specified by the user. A knowledge of the current strain and the previous stress-strain path for each subdivided area provides the information necessary to determine the current stress and hence evaluate the integrals to determine the current force resultant. This concept is employed to derive the incremental force-deformation relationships at each of the beam's two node points (six degrees of freedom per node) which permits one to derive the time varying tangent stiffness matrix for these elements.

The matrix equations of motion can be written as

$$M\ddot{D}_{i+1} + C\dot{D}_{i+1} + F_i^{int} + K_T\ \Delta D = F_{i+1}^{ext} \qquad (A.1)$$

where

M = mass matrix

C = damping matrix

K_T = tangent stiffness matrix

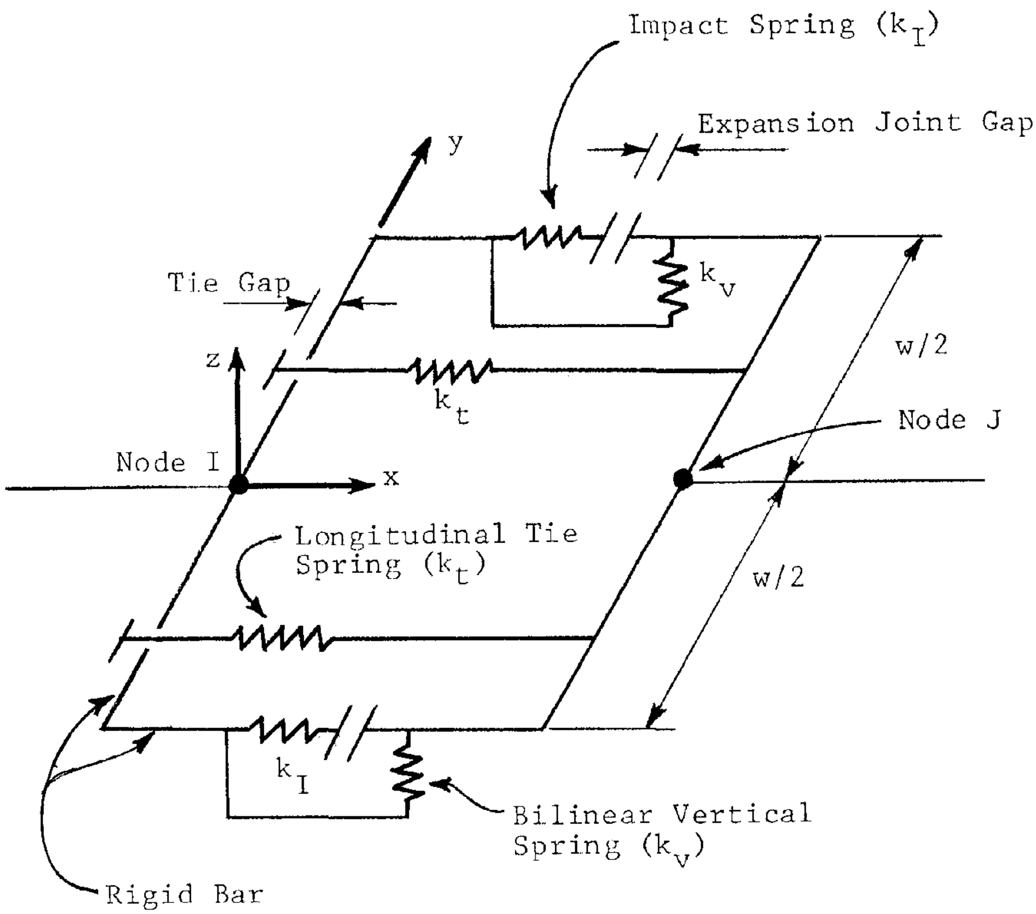
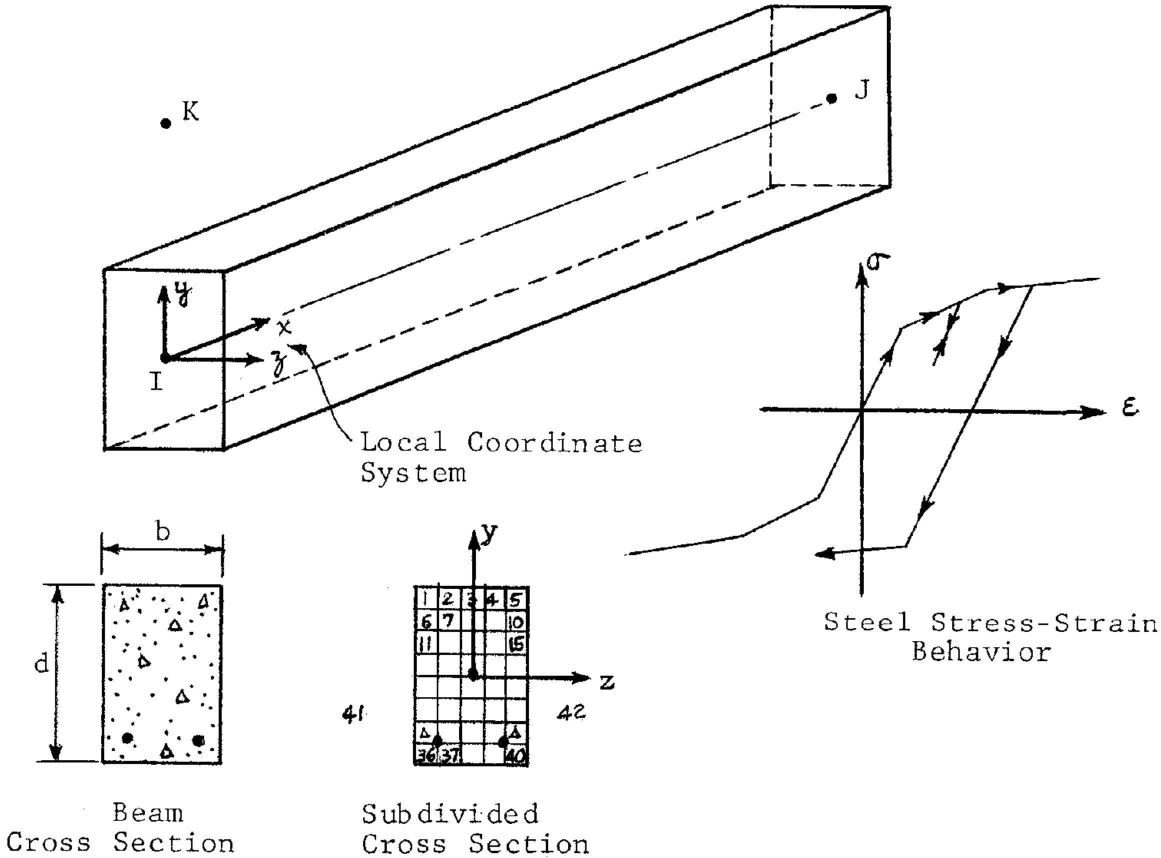


Figure A.1 Expansion Joint Finite Element Model



Subdivision i	Area A_i	Location y_i	z_i	Material Type
1	bd/40	$3.5\frac{d}{8}$	$-2\frac{b}{5}$	Concrete
↓	↓	–	–	↓
40	bd/40	–	–	Concrete
41				Steel
42				Steel

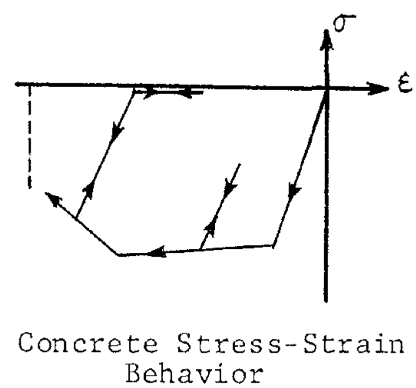


Figure A.2 Nonlinear, Composite Material Three-Dimensional Beam Finite Element (Used for Reinforced Concrete Beams)

F^{ext}_{i+1} = vector of external forces at integration time step i+1

F^{int}_{i} = vector of internal forces at the beginning of the integration time step (time t_i)

$\dot{D}_{i+1}, \ddot{D}_{i+1}$ = unknown vectors of velocity and acceleration end of time step (time t_{i+1})

ΔD = unknown vector of the increment of displacement over the integration time step

$= D_{i+1} - D_i$

The implicit method of solution employed by the computer code uses Newmark's β method of numerical integration to solve for the velocity and acceleration vectors at the end of the time step ($\ddot{D}_{i+1}$, $\dot{D}_{i+1}$) in terms of the known acceleration, velocity and displacement vectors at the beginning of the step ($\ddot{D}_i$, $\dot{D}_i$, D_i) and the unknown ΔD. Newmark's β integration relations can be written as

$$\ddot{D}_{i+1} = a_o \Delta D + a_1 \dot{D}_i + a_2 \ddot{D}_i \tag{A.2}$$

where

$$a_o = 1/(\beta\Delta t^2)$$

$$a_1 = - \Delta t \; a_o$$

$$a_2 = 1 - 0.5/\beta$$

$$\dot{D}_{i+1} = b_o \Delta D + b_1 \dot{D}_i + b_2 \ddot{D}_i \tag{A.3}$$

where

$$b_o = 1/(2\beta\Delta t)$$

$$b_1 = a_2$$

$$b_2 = \Delta t \; (1 - 0.25/\beta)$$

Substituting equations (A.2) and (A.3) into (A.1) after rearranging terms

$$\begin{aligned}(a_o M + b_o C + K_T) \Delta D = {} & F^{ext}_{i+1} - F^{int}_{i} \\ & - (a_1 M + b_1 C) \dot{D}_i \\ & - (a_2 M + b_2 C) \ddot{D}_i\end{aligned} \tag{A.4a}$$

or

$$K^{eff} \Delta D = \Delta F^{eff} \tag{A.4b}$$

This system of linear equations can be solved for the unknown increment in displacement, vector ΔD, since all of the other terms are known. Note that as the structure deforms nonlinearly, the tangent stiffness matrix (K_T) changes and hence the effective stiffness matrix must be changed to reflect this structural modification. It is also possible to obtain accurate solutions without modifying K_T and the effective stiffness matrix K^{eff} at each time increment. A modified Newton-Raphson iteration procedure is employed in the computer code to obtain acceptable results. This concept is illustrated in Figure A.3 for the static problem. Referring to this figure, the displacement and internal force is known at solution step i

$$(D_i, F_i^{int})$$

The tangent stiffness at an earlier point in the solution process is also known and denoted by K. It is desired to find the displacement at the end of the time step such that the error in the equilibrium equations is within a certain tolerance. This notation is employed:

$D_i^{(j)}, F_i^{int(j)}$ = displacement and internal force at the ith solution step, jth iteration cycle

It is desired that $F_{i+1}^{ext} - F_{i+1}^{int} = 0$. With an assumed K, compute $\Delta D^{(o)}$ from

$$K\ \Delta D^{(o)} = F_{i+1}^{ext} - F_i^{int} \tag{A.5}$$

$$D_{i+1}^{(o)} = D_i + \Delta D^{(o)} \tag{A.6}$$

The internal forces are computed from the 0th iteration of the i+1 step displacements, i.e.,

$$F_{i+1}^{int(o)} = F^{int}\{D_{i+1}^{(o)}\} \tag{A.7}$$

The error in equilibrium is

$$F_{i+1}^{err(o)} = F_{i+1}^{ext} - F_{i+1}^{int(o)} \tag{A.8}$$

The improvement to the end of the step displacement is

$$\Delta D^{(1)} = K^{-1}\ F_{i+1}^{err(o)} \tag{A.9}$$

The displacement after the first iteration cycle is

$$D_{i+1}^{(1)} = D_{i+1}^{(o)} + \Delta D^{(1)} \tag{A.10}$$

This process is repeated until the error at the end of the jth iteration cycle

$$F_{i+1}^{err(j)} = F_{i+1}^{ext} - F_{i+1}^{int(j)} \tag{A.11}$$

is within the permissible value or the number of iteration cycles becomes excessive.

For the general case of seismic loading, the vector of external forces is not an appropriate designation of the loading phenomenon since the loads are in

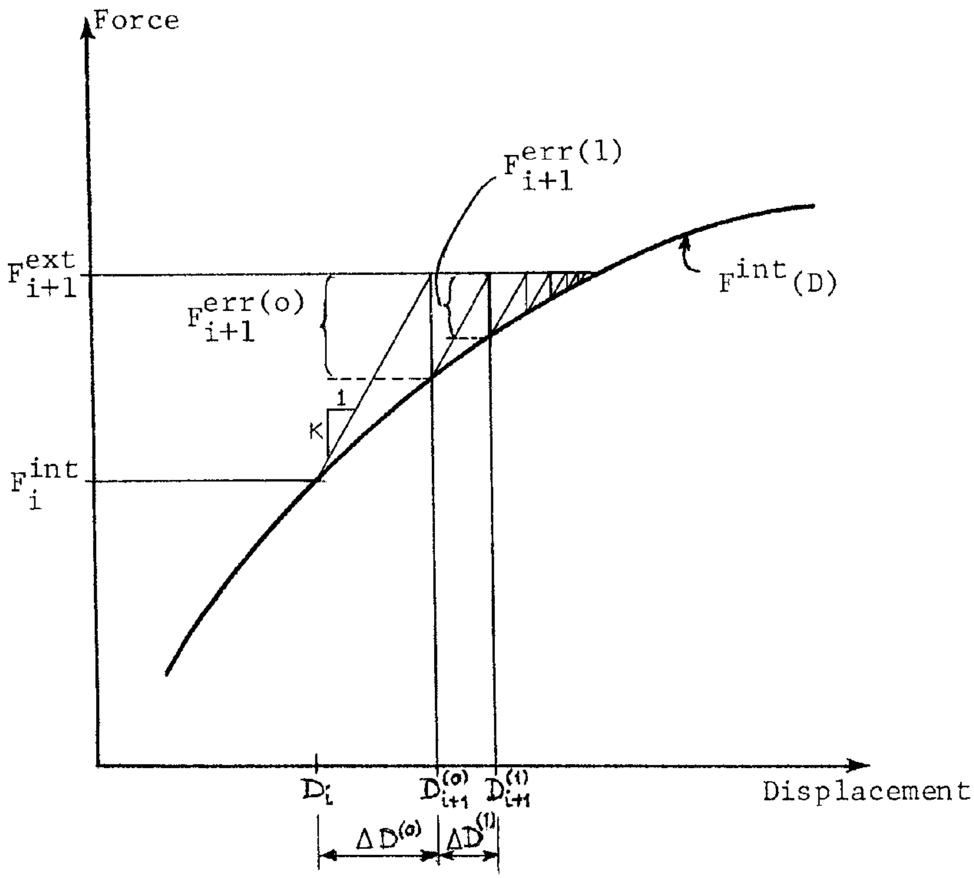


Figure A.3 Modified Newton-Raphson Iteration Procedure for Static Loading the form of prescribed motion variations at the support points. The treatment of this type of loading, together with any external loads which for this case are merely the gravity dead loads, is discussed by again considering the static solution case.

If there are prescribed motions at some of the degrees of freedom (say M of the DOF) and known external forces (or loads) at the remaining degrees of freedom (say L of the DOF) where the total number of DOF = M + L then the L and M degrees of freedom can be grouped and the incremental equilibrium equations can be rewritten as

$$\begin{bmatrix} K^{LL} & K^{LM} \\ K^{ML} & K^{MM} \end{bmatrix} \begin{bmatrix} \Delta D^{L} \\ \Delta D^{M} \end{bmatrix}^{(o)} = \begin{bmatrix} F^{ext} & L \\ F^{ext} & M \end{bmatrix}_{i+1} - \begin{bmatrix} F^{int} & L \\ F^{int} & M \end{bmatrix}_{i} \tag{A.12}$$

or

$$K\Delta D^{(o)} = F^{ext}_{i+1} - F^{int}_{i}$$

where ΔD^{M} are the known prescribed incremental motions and

$$F^{ext\ L}_{i+1}$$

are the known external loads at the remaining degrees of freedom. The unknowns are

$$\Delta D^{L} \text{ and } F^{ext\ M}_{i+1}$$

where the latter are the reactions which are of little interest since they can be readily obtained from the internal forces

$$F^{int\ M}$$

at the M degrees of freedom.

To solve the equations for ΔD^{L} and also the known ΔD^{M}, the equations are recast as

$$\begin{bmatrix} K^{LL} & 0 & \Delta D^{L} \\ 0 & I & \Delta D^{M} \end{bmatrix}^{(o)} = \begin{bmatrix} F^{ext\ L}_{i+1} \\ \Delta D^{M} \end{bmatrix} - \begin{bmatrix} F^{int\ L}_{i} + K^{LM}\,\Delta D^{M} \\ 0 \end{bmatrix} \tag{A.13}$$

or

$$K^{*}\,\Delta D^{(o)} = F^{*}$$

Thus the forces obtained from the $K^{LM}\,\Delta D^{M}$ operation are the assumed[1] incremental internal forces due to the known prescribed incremental displacements ΔD^{M}.

[1]They are assumed (or approximate) because K^{LM} is based on some assumed relationship (tangent stiffness) between internal forces and displacements for the increment.

The total displacements at the end of the step (after the 0th iteration cycle) are

$$D_{i+1}^{(o)} = \begin{bmatrix} D_{i+1}^{L} \\ D_{i+1}^{M} \end{bmatrix}^{(o)} = \begin{bmatrix} D_{i}^{L} \\ D_{i}^{M} \end{bmatrix} + \begin{bmatrix} \Delta D^{L} \\ \Delta D^{M} \end{bmatrix}^{(o)} \tag{A.14}$$

Since the displacements $D^{(o)}$ at the M degrees of freedom are exact, one does not want to compute any additional displacement for these degrees of freedom during the iteration cycles. Hence, the error force for iteration is based only on the error for the L degrees of freedom

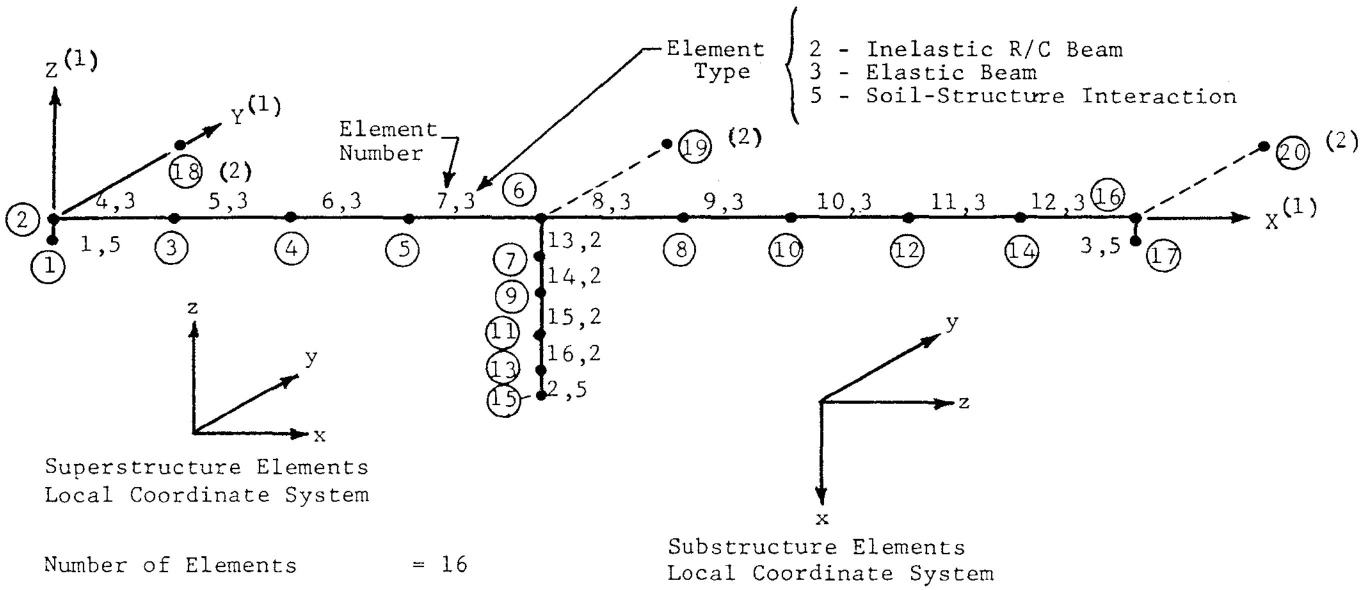
$$F_{i+1}^{err(o)} = \begin{bmatrix} F_{i+1}^{ext\ L} \\ 0 \end{bmatrix} - \begin{bmatrix} F_{i+1}^{int\ L(o)} \\ 0 \end{bmatrix} \tag{A.15}$$

The iteration solution scheme for prescribed motion then follows that defined by equations (A.5) through (A.10) with $K \leftarrow K^*$ and $F_{i+1}^{err(j)}$ determined as indicated in equation (A.15).

The application of the computer code to the response of a two-span continuous reinforced concrete bridge structure shown in Figure B.4 of Appendix B is briefly described. The finite element model of the structure (Figure A.4) that was used in the analysis consists of 17 node points and 16 elements. The superstructure was modeled with nine elastic beam elements. Three soil-structure interaction elements were used at the foundation attachment points corresponding to the abutments and the central pier. The pier column, which is monolithic with the superstructure, was modeled with four nonlinear reinforced concrete (composite) beam elements.

Node points 1, 15, and 17 represent the soil connection points which were subjected to the vertical and horizontal ground motion variation (see Figures A.5, A.6) that was determined for the site. These prescribed motions were applied to each of the soil nodes with a phase delay time. The delay times were based on the assumption that the seismic disturbance is traveling in the longitudinal direction at a velocity of 1000 ft/sec. The horizontal motions were applied in the longitudinal direction. The most severe response of the structure is the bending moment at the top of the pier as shown in Figure A.7. For comparison, the ultimate moment of the pier (without axial loading) is shown on the figure. Even though this ultimate moment is exceeded several times during the solution, the column does not fail. This is due to the increase in the bending resistance from the axial compression load since the column would fail by the reinforcement exceeding the tension ultimate stress.

If a retrofit procedure is deemed necessary to eliminate the damage that will occur to the pier column, a reasonable structural modification would be to strengthen the column. The technique illustrated in Figure B.4 of Appendix B can be used to add 16 additional number 11 rebars to the pier column (eight bars each on the wide faces) and provide 76.2 mm (3 inches) of additional concrete on the wide faces. The computed results for the bending moment at the top of the pier for the modified structure are shown in Figure A.8. It is observed that the addition of the longitudinal reinforcing bars and concrete to the pier causes



Number of Elements = 16

Number of Structure Nodes = 17

Number of Element Types = 4

(1) Global Coordinate System of Structure

(2) Nodes 18,19 and 20 Determine the Local Coordinate Systems of the Elements

Figure A.4 Finite Element Model for Nonlinear Analysis of Reinforced Concrete Bridge

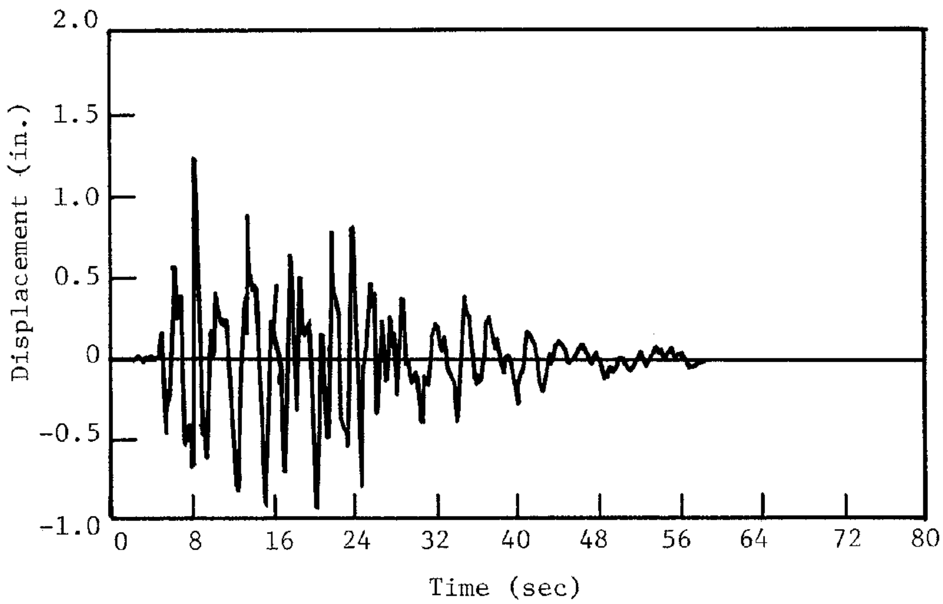


Figure A.5 Simulated Ground Motion - Vertical

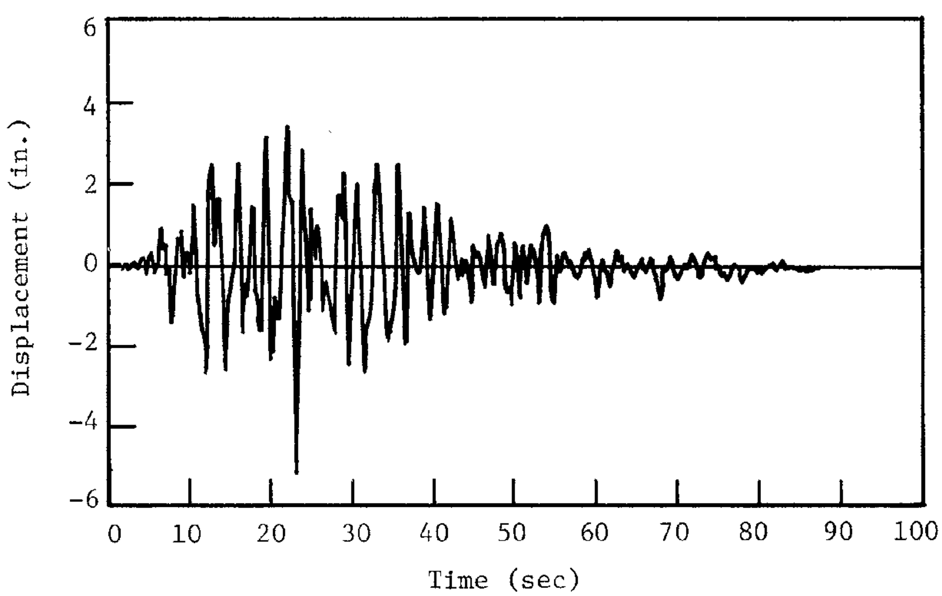


Figure A.6 Simulated Ground Motion - Horizontal

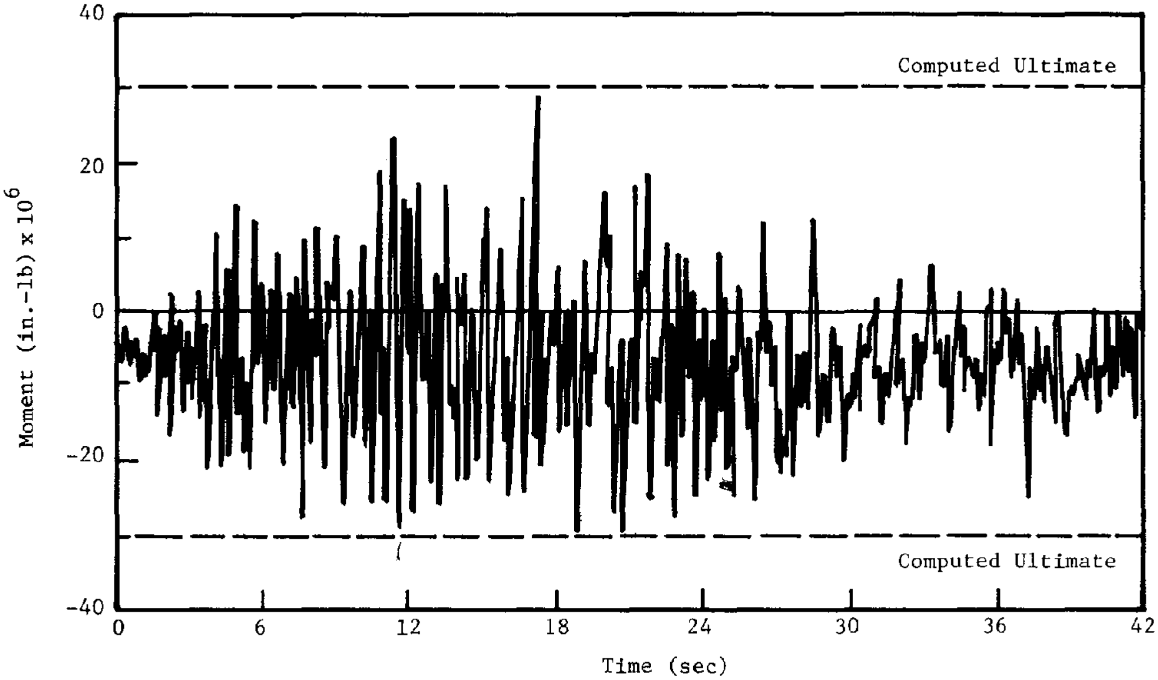


Figure A.7 Bending Moment about the Weak Axis (Lateral) at the Top of Pier (Element 13)

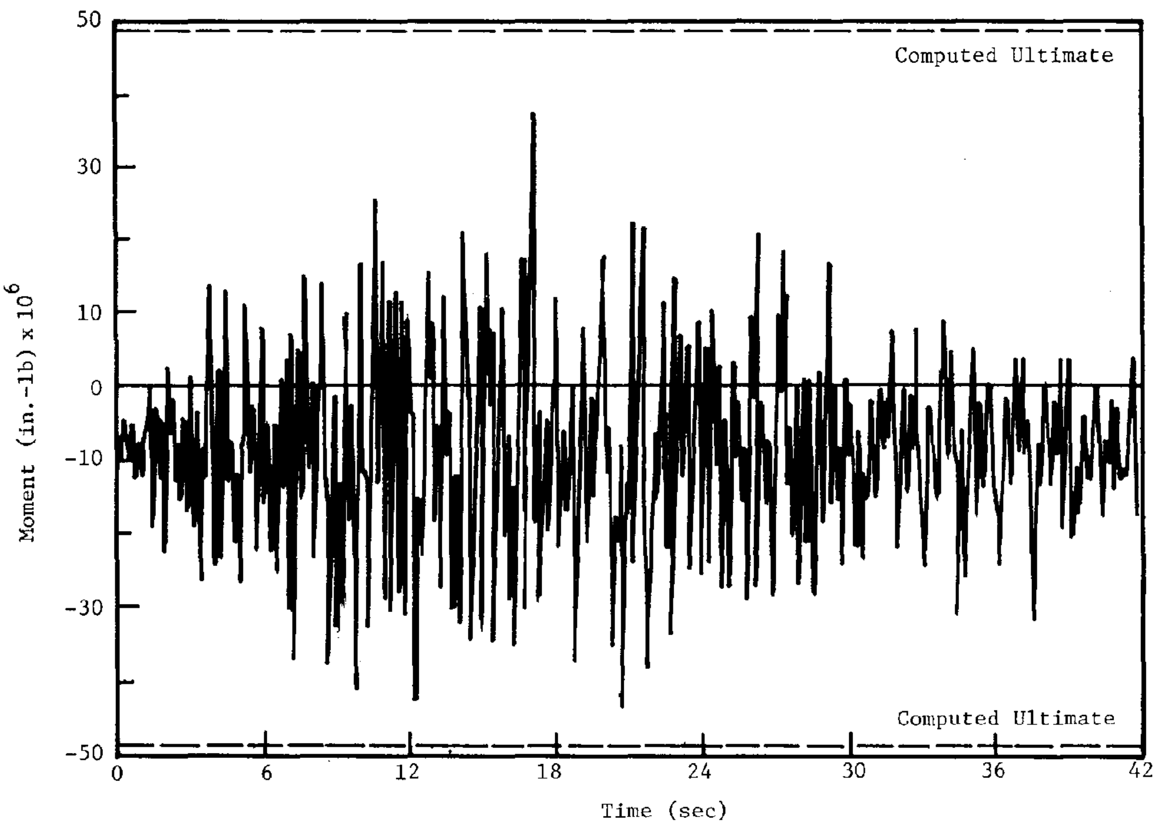


Figure A.8 Bending Moment about the Weak Axis at Top of Pier for Retrofitted Structure an increase in the maximum bending moment of the column during the seismic loading. The ratio of the moment for the retrofitted case to the as-built case is 1.19. At the same time, the retrofit measure produces a 61 percent increase in the computed ultimate bending moment. There were essentially no increases in the internal forces for the unretrofitted portions of the structure, i.e., the superstructure.

APPENDIX B: SELECTED BRIDGE RETROFIT MEASURES

This appendix contains descriptions and illustrations of four retrofit measures developed in connection with the study reported in [3]. This reference document contains design details, design procedures, and considerations of retrofit materials, construction equipment and maintenance.

Steel Girder Vertical Displacement Restrainer

The objective of the vertical displacement restrainer is to restrict the relative vertical motion between the superstructure and the pier or abutment seat during an earthquake with a strong vertical component. The use of this retrofit will limit the vertical separation that can occur at the support bearings and eliminate bearing instability and hence loss of superstructure support.

To illustrate the design of this concept, a bridge was considered with longitudinal girders supported by bearings which do not provide a positive restraint to uplift forces. The piers are reinforced concrete frames with sufficient open space under the cap beam to accommodate the restrainer details. Figure B.1 illustrates the resulting concept.

Steel Girder HingeExpansion Joint Longitudinal Restrainer

The purpose of the expansion joint longitudinal restrainer is to restrict the relative longitudinal motion across the expansion joint during an earthquake. Using this retrofit concept, excessive separation displacements across the hinge are reduced and hinge failures created by this effect are thus eliminated. A certain amount of free thermal expansion is permitted at the hinge before resistance is encountered.

A typical four-span grade separation of cantilever and suspended span construction illustrates the use of this retrofit concept. The original design of the expansion joint is such that no longitudinal force can be transmitted across the joint.

The retrofit concept makes use of existing headers (see Figure B.2) located at either side of the expansion joints. Restrainer rods located close to the bridge girders are used to tie the bridge together. Since in this particular case the headers are not by themselves sufficiently strong, steel channels (see Figure B.3) provide a diagonal brace to transfer the design load from the restrainer rods to the girder web.

Reinforced Concrete Bent Column Strengthening

The objective of this retrofit is to increase the flexural capacity of a bent column so that bent failure will not occur during a strong motion earthquake. The method used provided additional longitudinal reinforcement to the exterior

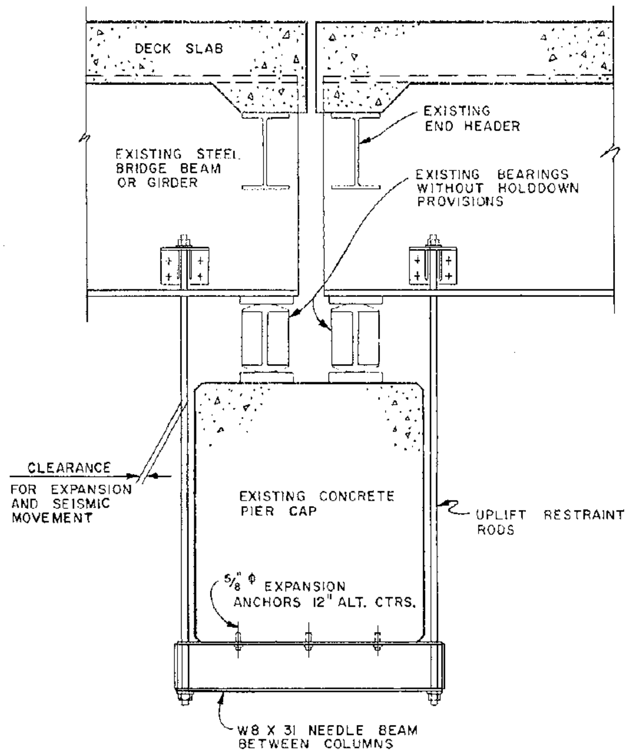
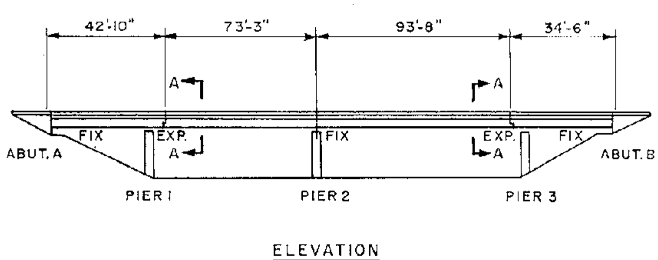
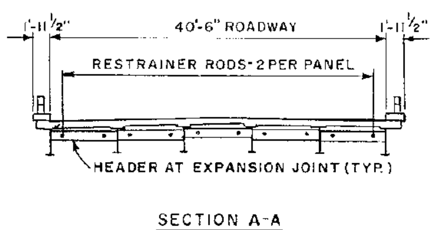


Figure B.1 Steel Girder Vertical Restrainer Retrofit Concept





1 ft = 0.305 m
1 in.= 2.54 cm

Figure B.2 Steel Girder Hinge Expansion Joint Longitudinal Restrainer Retrofit Concept

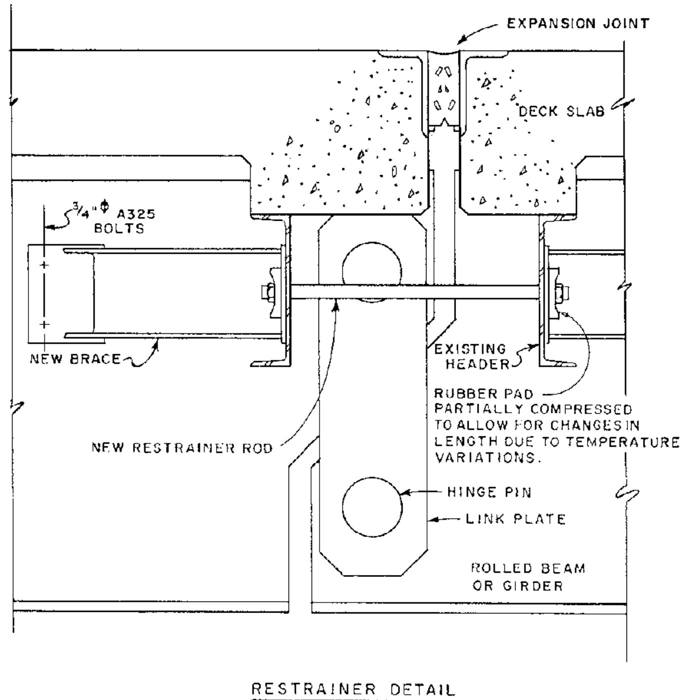
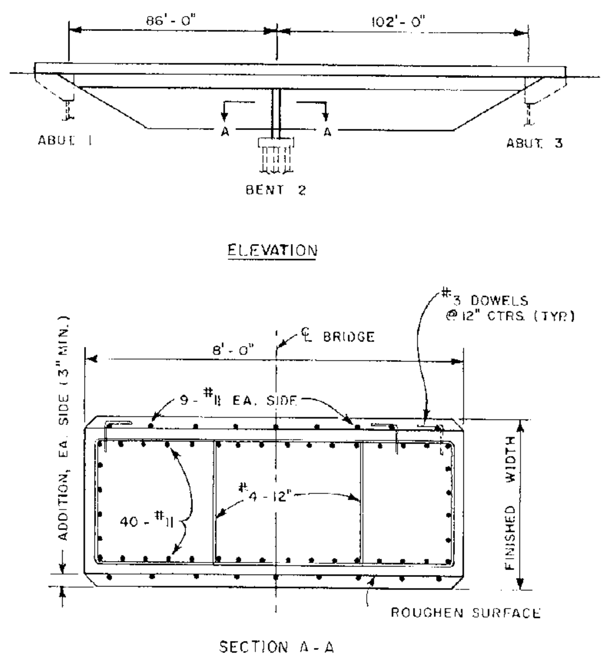


Figure B.3 Hinge Expansion Joint Longitudinal Restrainer Details



1 ft = 0.305 m
1 in.= 2.54 cm

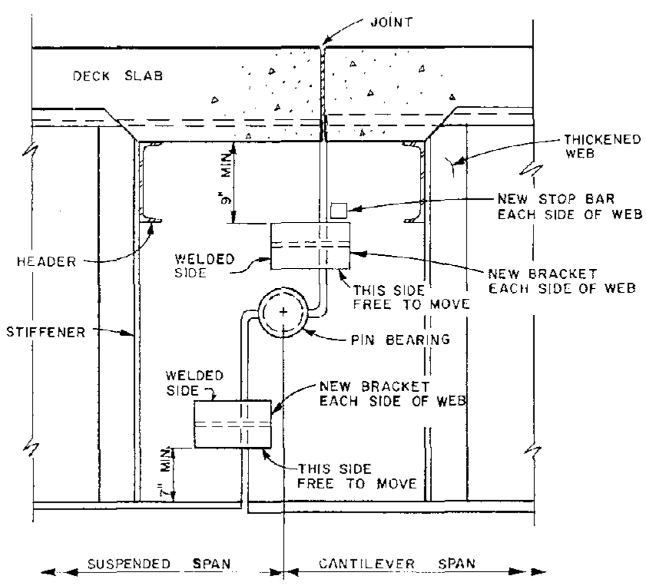
Figure B.4 Reinforced Concrete Bent Column Strengthening Retrofit Concept of the column which is connected to the bent cap and footing by grouting the new bars in drilled holes. Lateral dowels are also introduced to enhance the monolithic behavior of the new addition to the parent column.

A representative two-span reinforced concrete box girder bridge is used to demonstrate the retrofit measure. The original design employed a single column pier with the cap monolithic with the superstructure and a pile spread footing. The structural characteristics of the retrofit are shown in Figure B.4.

Steel Girder Pin Bearing Vertical and Lateral Displacement Restrainer

The objective of the pin bearing displacement restrainer is to inhibit essentially all of the relative vertical and lateral motions across the bearing that could take place during an earthquake. With this retrofit, potential vertical and lateral motions during an earthquake are arrested by the addition of a bracket and stopper bar arrangement welded to the webs of the girders. The joint is also effectively restrained against relative longitudinal motion during an earthquake by the new vertical restraint which prevents the suspended span from rolling over the pin.

The retrofit method is applicable to any bridge with longitudinal girders supported by hinged bearings which do not provide positive restraint against uplift or lateral motion. This concept is illustrated in Figure B.5.



1 ft = 0.305 m
1 in.= 2.54 cm

Figure B.5 Steel Girder Pin-Type Bearing Vertical and Lateral Displacement Restrainer

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INSPECTION AND RETROFITTING OF EARTHQUAKE RESISTANCE VULNERABILITY OF HIGHWAY BRIDGES ---- JAPANESE APPROACH

by

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Tadayuki Tazaki ; Senior Research Engineer, do

ABSTRACTS

This paper introduces the inspection and retrofitting of earthquake resistance vulnerability of highway bridges in Japan.

Firstly the basic idea of retrofitting design is presented. The probabilistic approach of retrofitting and the residual life to be considered are proposed.

Next the inspections and retrofitting of highway bridges conducted or scheduled by Ministry of Construction in 1971, 1976 and 1979 are introduced. The results of the inspection in 1976 are concentratively discussed.

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INTRODUCTION

This paper introduces the inspection and retrofitting of earthquake resistance vulnerability of highway bridges in Japan.

Firstly the basic idea of retrofitting design is presented. Secondly the inspections and retrofittings of highway bridges conducted or scheduled by Ministry of Construction in 1971, 1976 and 1979 are discussed.

BASIC IDEA OF RETROFITTING DESIGN

Selection of Vulnerables

Criteria to select the structures retrofitted consist of items of physical and socio-economic conditions. The items to examine in selection of the structures are shown in Table-1.

Probabilistic Analysis of Retrofitting

Each vulnerable structure should be retrofitted to have the reasonably little probability of failure and not to require the excessive expense. This criterion is written as the following formulae.

Let R and S the resistance and the intensity of loading respectively. The probability p that structures are becoming limit states is given by Eq. (1).

$$p = \Pr [R < S] = \int_0^\infty F_R(X)\ f_s(x)dx \qquad (1)$$

where $\Pr [R < S]$: probability of $R < S$

$F_R(x)$: cumulative distribution function of R

$f_s(x)$: random variable of S

In case of earthquake, supposing $f_s(x)$ the random variable of maximum earthquake intensity in a year and $F_R(x)$ the cumulative distribution function of mean quake-resistance in a year, cumulative probability of failure of a structure during its life is given by Eq.(2).

$$U_D = 1 - \prod_{t=1}^{T_D} (1 - p) \qquad (2)$$

U_D : cumulative probabulity of failure

T_D : durable life

If p is constant during its durable life, Eq. (2) can be written as,

$$U_D = 1 - (1 - p)^{T_D} \qquad (3)$$

U^*, the desirable value of U_D, should be given by social needs of safety in earthquakes.

$$U_D \leqq U^* \qquad (4)$$

The order of U^* is empirically estimated as $10^{-4} \sim 10^{-6}$ for steel structures and $10^{-5} \sim 10^{-7}$ for concrete structures when they are designed following the current specifications. [7]

In case of retrofitting it is relatively practical to consider the residual life after the retrofitting rather than the total durable life. Because the utility of the bridge to be retrofitted has been partially redeemed during the bygone service-life. In the case, the durable life T_D in Eq.(2) should be substituted for the residual life τ_R (i.e. the difference between the durable life T_D and the service-life T).

$$U_R = 1 - \prod_{t=T+1}^{T_D} (1 - p) \qquad (5)$$

The desirable value U_R^* for U_R can be reduced by the service-life, because of the redeemed utility.

$$U_R \leqq U_R^* \qquad (6)$$

$$U_R^* = \frac{\tau_R}{T_D} U^* \qquad (7)$$

Examples

In the examples the value of U^* is assumed as 2.1×10^{-3} using the probability of failure in the past earthquakes. [7] It is also assumed that the R in Eq.(1) has no deviation for the distribution, but has a certain value. The probability of failure p only depends on the seismic loading S. Generally the relationship between the earthquake intensity I_{JMA} and the probability p, which means the probability of the earthquake occurence exceeding I_{JMA} in a year, is well known as follows.

$$\log p = \theta - \mu I_{JMA} \qquad (8)$$

θ , μ : constants

I_{JMA} : earthquake intensity of Japan Meteorogical Agency

Appling the report of Katayama [6] $\theta=3.24$ and $\mu=0.87$ in Tokyo are determined.

Quake-resistance of bridge at time t, R(t), is assumed as shown in Fig.1 considering the deterioration of materials.

Figs. 2 and 3 show examples for retrofitting of bridges after T years of completion whose cumulative probability of failure is higher than the desirable value due to insuffficent resistance at their completion.

These are examples of which initial earthquake resistance is 94% and 96% of R_0 (sufficient resistance), respectively, In these figures, curve A indicates the earthquake resistance of structures without retrofitting, curves B and C indicate those which are retrofitted of ΔR satisfying Eq. (4) and of $\Delta R'$ satisfyung Eq. (6), respectively. The curve B does not exist in Fig. 2 after 30 years of completion. It means there is no way to retrofit the structures so as to satisfy Eq. (4). And also in Fig. 3, the amount of retrofitting $\Delta R'$ in curve C moderately increases during the whole life in both figures. Therefore it is condidered that the criterion represented by curve C is applicable one from the practical point of view.

Fig. 4 shows an example when the assumed U^* of 2.1×10^{-3} at first is reduced to 1/2 of the original U^*. The meanings of curves A, B and C are the same mentioned in Figs. 2 and 3. The amount of retrofitting ΔR increases rapidly in accordance with time in the case of curve B. On the other hand it increases slowly in the case of curve C.

Based on the above mentioned numerical examples, it is not practical to enforce the same probability of failure for both existing structures and new structures when the earthquake resistances of such structures decrease in accordance with time since their completion. It is considered more practical to retrofit the structures by the probability of failure based on the residual life.

INSPECTION AND RETROFITTING OF HIGHWAY BRIDGES

Circumstances

All highway bridges are supervised technologically through the anthorized specifications by the Ministry of Construction. The ministry has conducted or plans to conduct the inspection of highway bridges three times (i.e. 1971, 1976 and 1979). The one in 1971 was to point out the deteriorated bridges liable to collapse in earthquakes. The second inspection in 1976 was to check the items being closely related with the possibility of collapse. The scheduled inspection in 1979 is to classify bridges according to their earthquake resistances.

Inspection in 1976

In 1976 the bridges, tunnels and pedestrian crossing bridges on the routes shown below, which constitute the major highway network, were inspected.

All routes of National Expressway,
Urban Expressway,
Designated National Highway (directly administrated by the ministry)
Toll Road of Japan Highway Public Corporation
The route within the DID of
Non-designated National Highway

Prefectural Road
Municipal Road
Toll Road (excluding above mentioned)

The items of inspection are shown in Table 2.

The results of the inspection are summarized in Figs. 5 ∿ 10. The structures which are identified vulnerable are to be concentratively retrofitted within the eighth five-year-plan of 1978-1982.

Inspection Scheduled in 1979

The ministry is planning to conduct more detailed inspection in 1979. The inspection consists of two steps. The first step is to extract the possibly vulnerable bridges in superstructure, substructure and subground conditions.

The bridges extracted by the first step are to be inspected in the second step. It is to classify bridges into three groups (i.e. highly resistant, fairly resistant and doubtful).

The retrofitting method for each type of vulnerability identified are proposed. The priority of retrofitting works is to be determined by the availability of substituteve routes and the easiness of traffic resumption in emergency.

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[1], [2], [3], [4]. [5], [6] and [9] are written in Japanese.

Table 1 Items to Examine for Earthquake Disaster Mitigation

CLASSIFI-CATIONS	FUNCTIONAL IMPORTANCE			DURABILITY	REPAIR AND RECONST-UCTION
	CONTRIBUTION	NUISANCE	DUMMY		
VICINITY	O	O	O		
STRUCTURE	O	O	O	O	O
SCALE	O	O	O	O	O

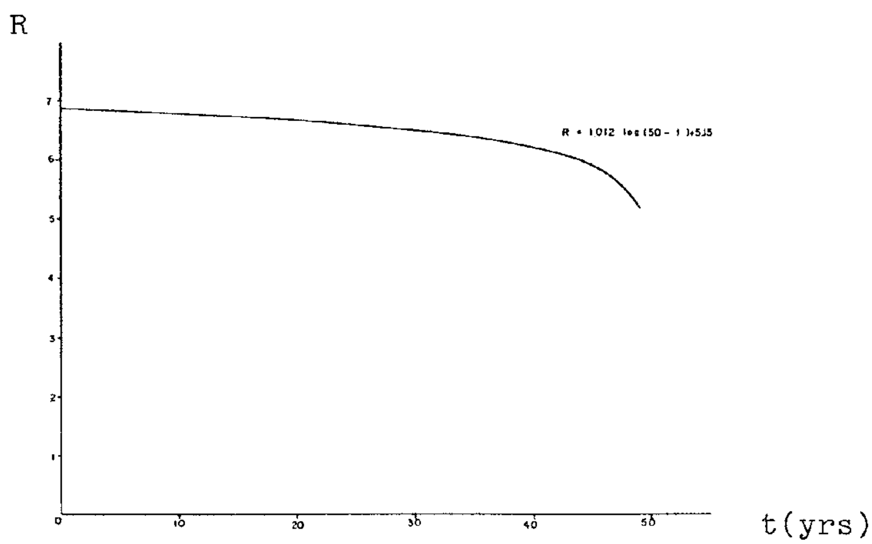

Fig. 1 Assumed Transition of Earthquake Resistance

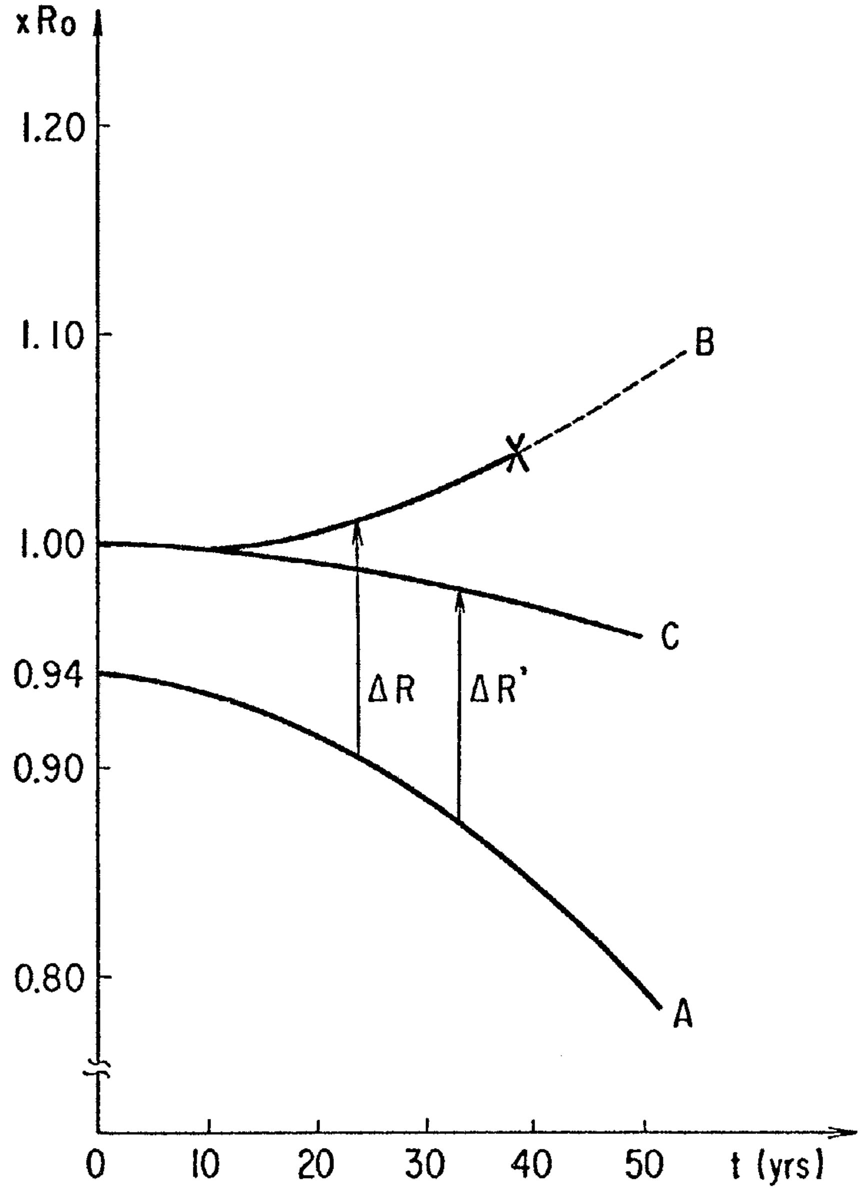

Fig. 2 Example of Retrofitting (Case 1)

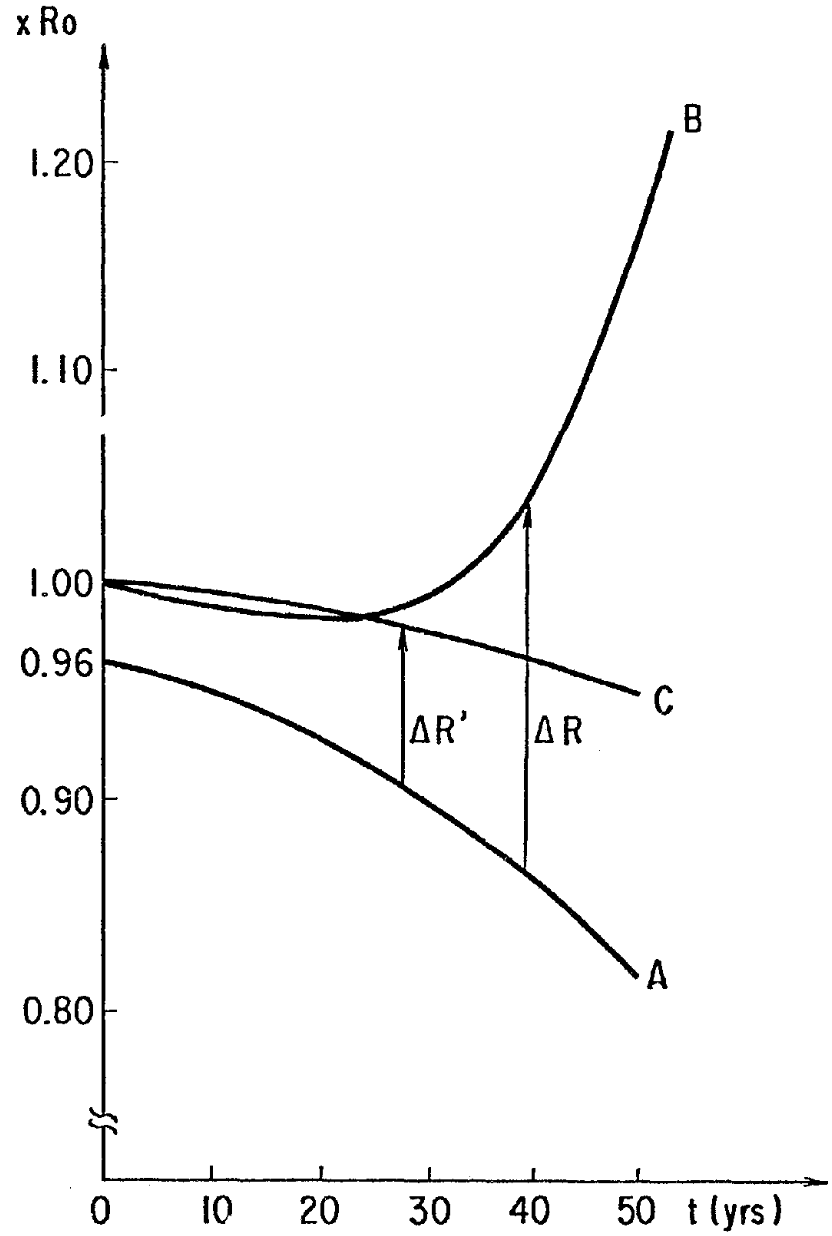

Fig. 3 Example of Retrofitting (Case 2)

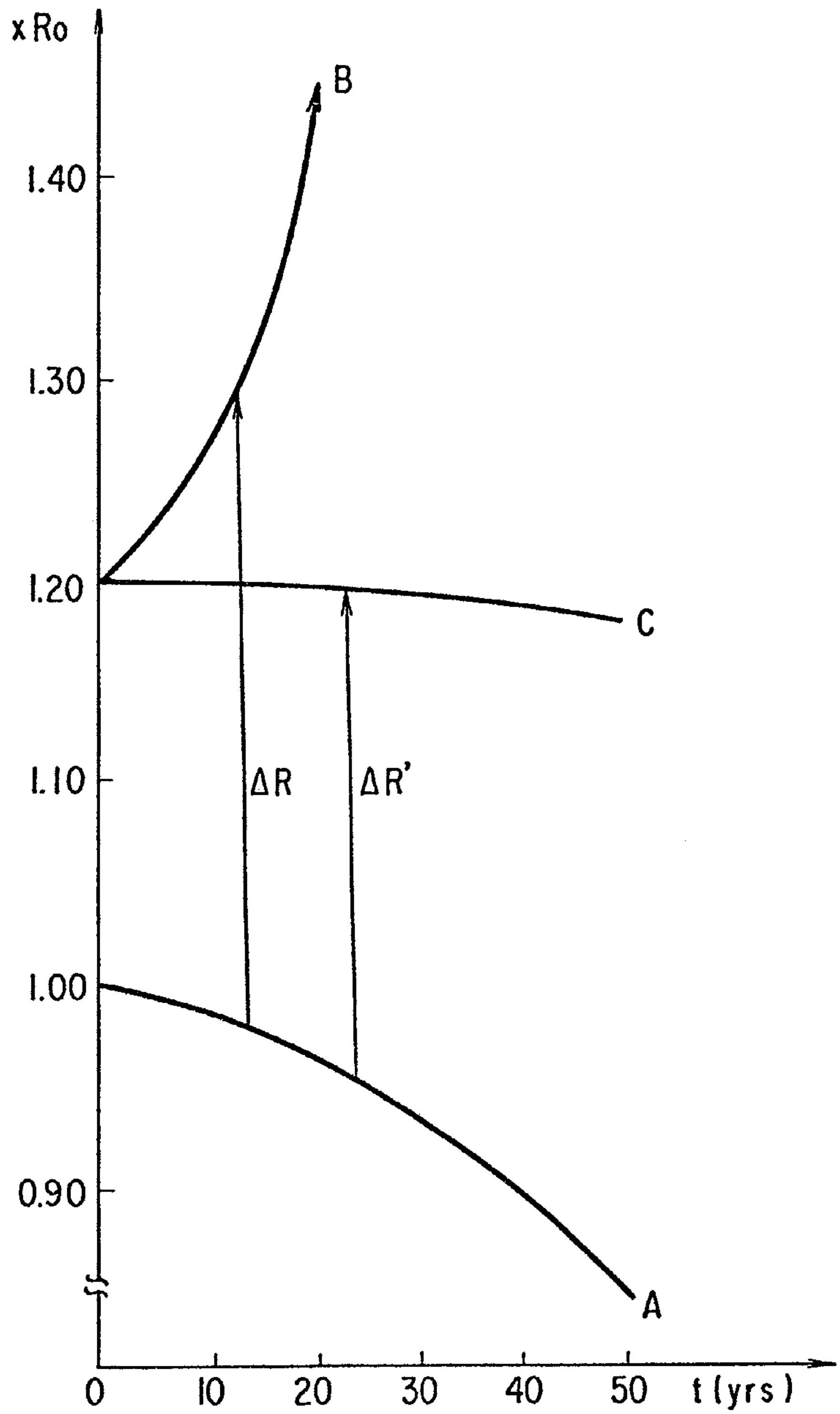

Fig. 4 Example of Retrofitting (Case 3)

Table 2 Items of Inspection in 1976

Bridge	Materials of Substructure Fall of Concrete of Substructure Scour Deviation of Support Width of Coping, Device to Prevent Dislodgement Deterioration of Concrete of Coping Anchor Bolt of Support Settlement, Slide and Inclination of Substructure Crack of Concrete of Superstructure Damage of Steel Girder Wooden Girder Based Specification
Tunnel	Deformation of Natural Ground Damage of Lining Landslide near Portal Settlement and Inclination of Portal Deterioration of Portal
Pedestrian Crossing Bridge	Devices to Prevent Dislodgement Deterioration of Superstructure and Substructure Settlement and Inclination of Substructure

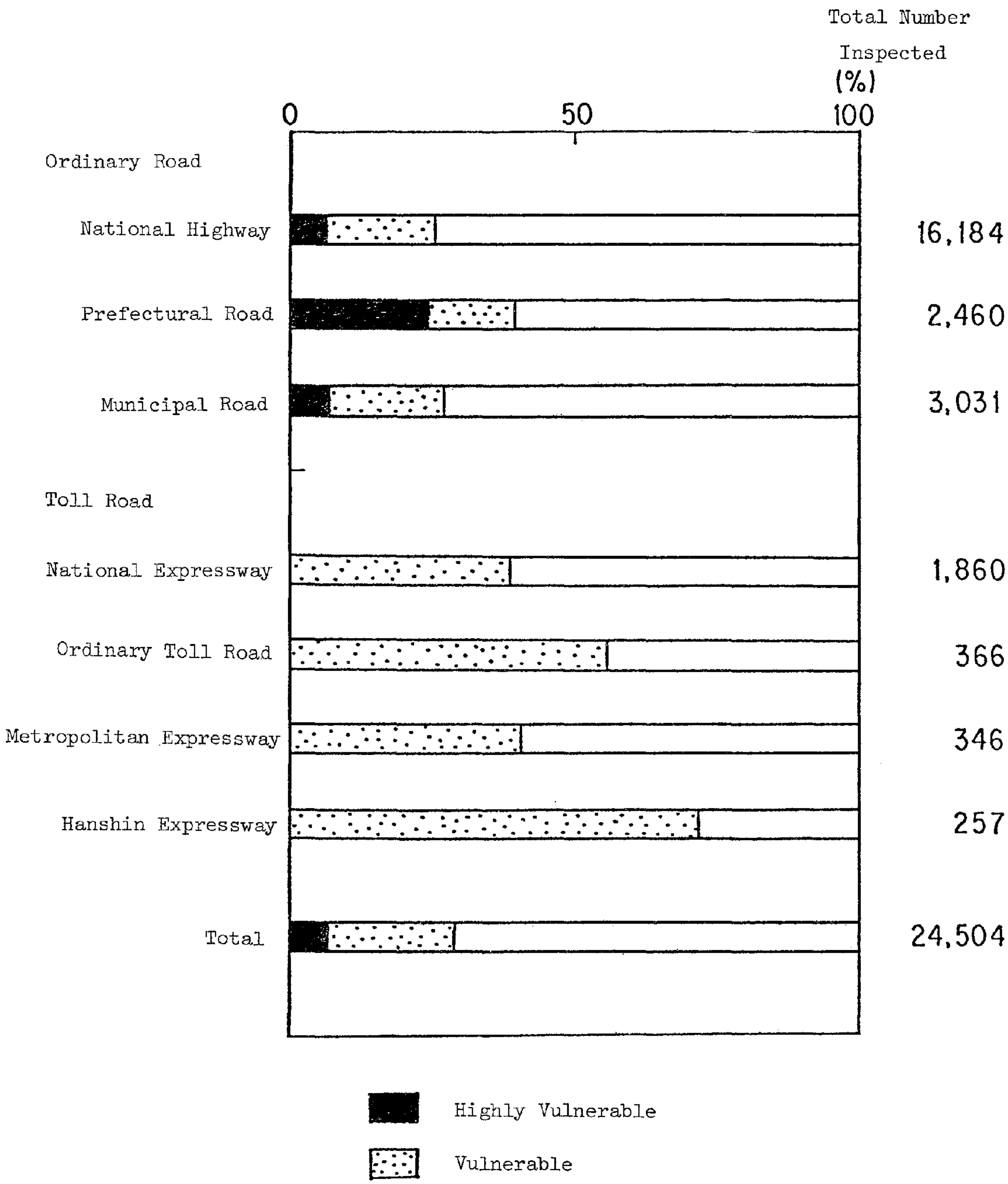

Fig.5 Inspection of Bridges in 1976

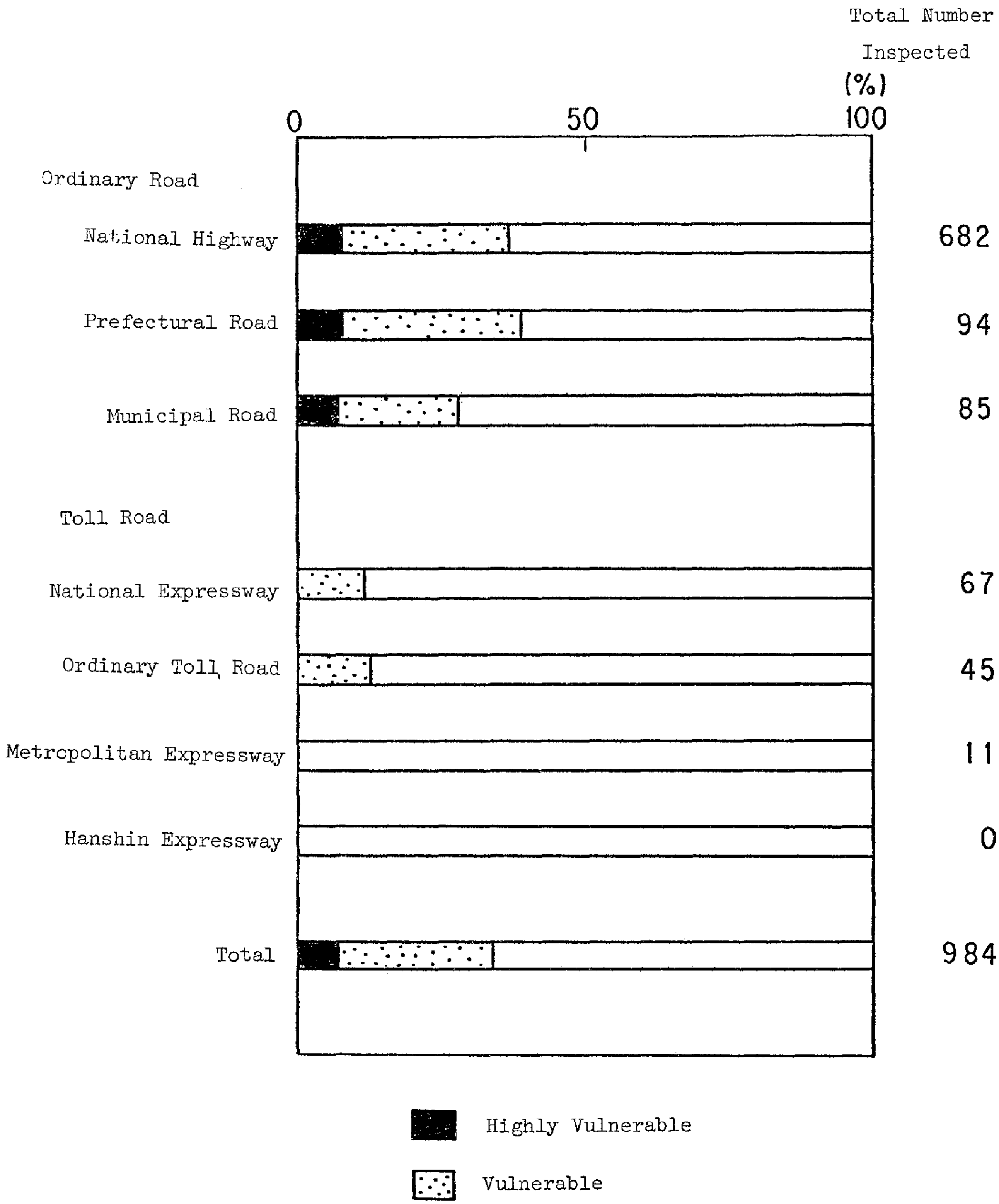

Fig.6 Inspection of Tunnels in 1976

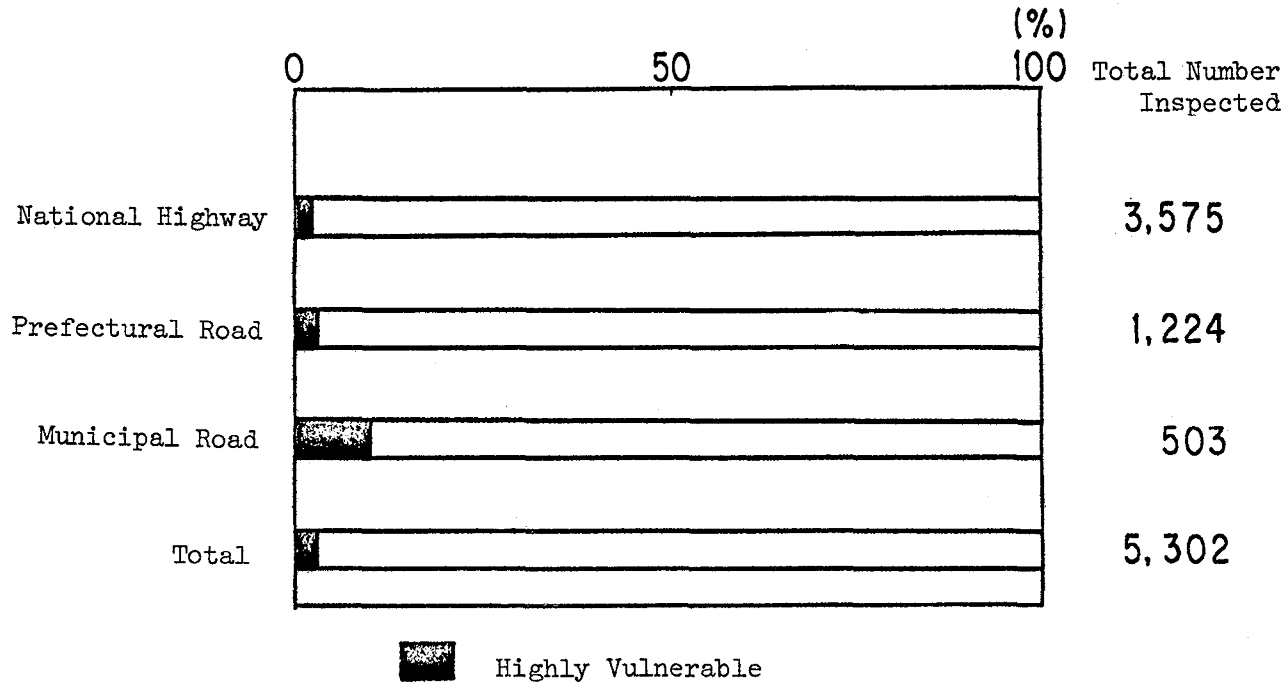

Fig.7 Inspection of Pedestrian Crossing Brides in 1976

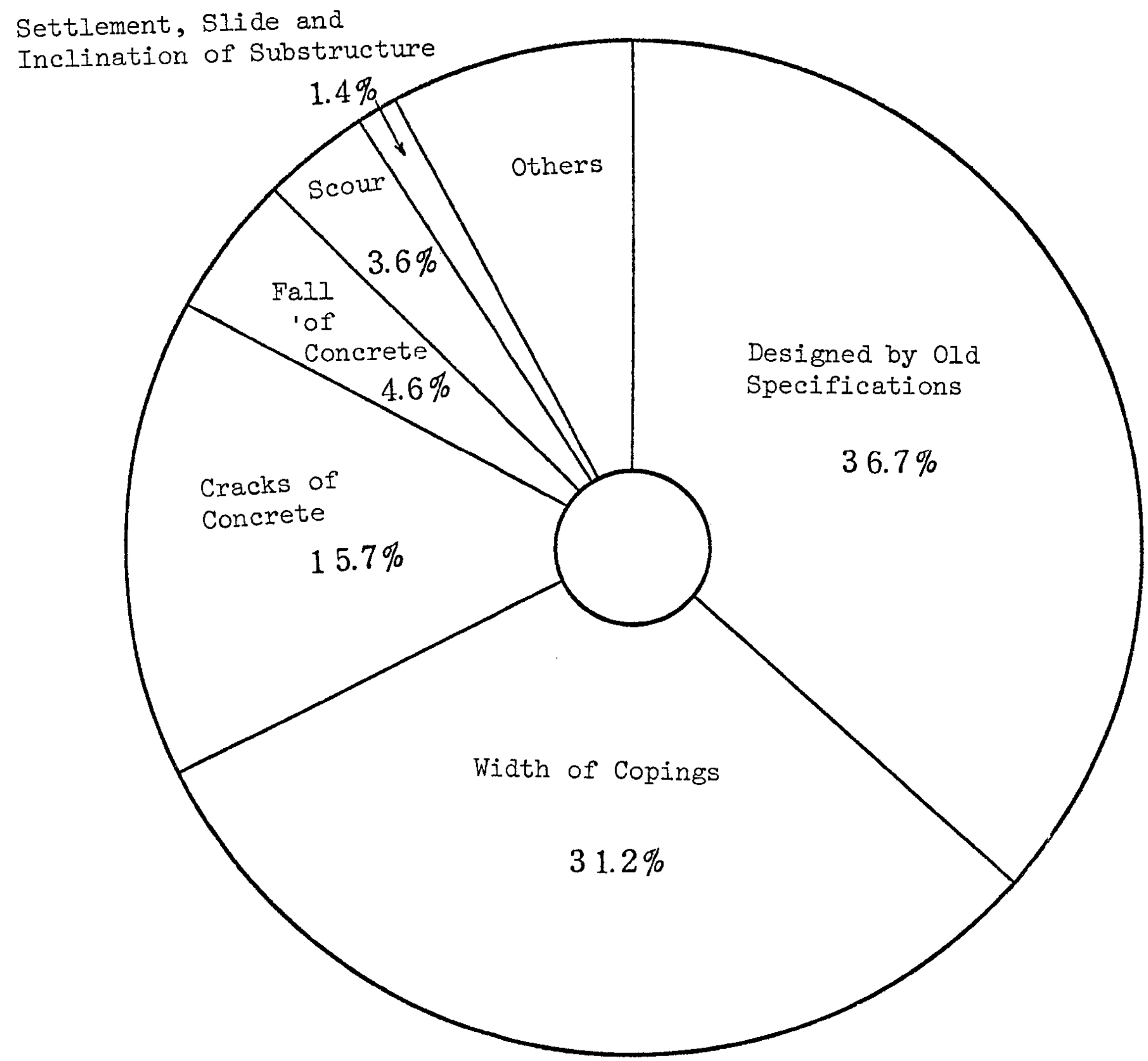

Fig. 8 Classification of Vulnerables of Bridges in 1976

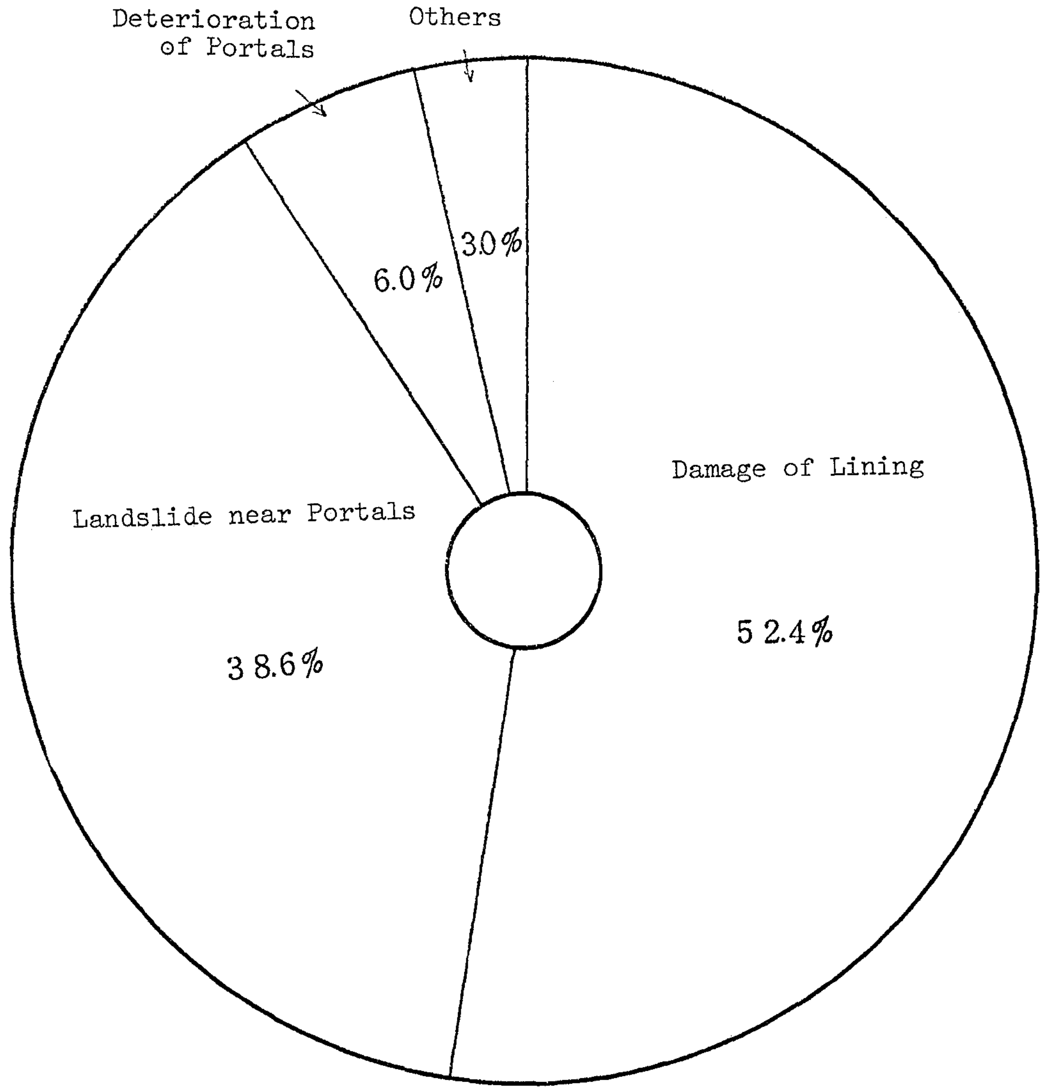

Fig. 9 Classification of Vulnerables of Tunnels in 1976

Fig. 10 Classification of Vulnerables of Crossing Pedestrian Bridges in 1976

SEISMIC MODEL STUDIES OF LONG-SPAN CURVED BRIDGES

by

William G. Godden
Professor of Civil Engineering
University of California, Berkeley

ABSTRACT

This paper describes two recent shaking table studies on the seismic behavior of long-span curved highway bridges. The first study deals with a multi-span concrete bridge supported on a single line of columns. The second study deals with a proposed single-span curved cable-stayed steel girder bridge. In both cases experimental data is presented to show the principal dynamic characteristics, and these are discussed in relation to analytical procedures. The potential benefits of horizontal curvature are indicated as well as certain problems that this generates.

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INTRODUCTION

A recent study of seismic damage has shown the vulnerability of both short- and long-span bridges to strong ground motion [1]. The horizontally curved bridge has certain unique characteristics, and with the damage and collapse of structures of this type in the San Fernando 1971 earthquake, a comprehensive analytical and experimental study was initiated at the University of California, Berkeley, to review current design practice, to study dynamic characteristics, and to develop improved analytical and design procedures where necessary. Much of this work has been discussed in the literature [1-10]. This paper deals partly with the experimental studies developed in that project and partly with model studies conducted on a proposed new highway bridge in California.

The partial collapse of the long multi-span 5/14 South Connector overcrossing in the San Fernando earthquake showed both the susceptibility of this type of structure to strong seismic ground motion and also the need for experimental studies to show overall dynamic behavior and collapse mechanisms as well as to provide response data for checking proposed analytical procedures.

The problems of modeling long-span structures for seismic testing are many: available facilities generally require reproducing the structure at very small scale, and this results in problems of material characteristics, self-weight effects, and a time-scale that may conflict with the performance specifications of the shaking table. These considerations led to different types of modeling in the two studies under discussion: in the first study involving fracture and ductility, the scale used was 1/30; in the second study which included the nonlinear vibrational effects of long cables but did not include material damage, the scale adopted was 1/200. Some of the considerations in modeling as well as the results of the shaking table tests are discussed for both studies.

CURVED MULTI-SPAN CONCRETE BRIDGE

The 5/14 South Connector overcrossing at San Fernando had a total span of 411 m (1349 ft) and was supported on eight columns between abutments. The longest spans were 58 m (191 ft) and the longest column 43 m (140 ft). The overall size of the structure made it impractical to model the complete system for study on the 6.1 m x 6.1 m (20 ft x 20 ft) shaking table at Berkeley. As it was required in this study to investigate the collapse mechanisms of the system, and as a preliminary dynamic analysis of the prototype structure indicated that the principal dynamic characteristics of the bridge could be reproduced on half of its total length, the east half which included the collapsed spans was taken as a basis for the 1/30th scale simplified symmetrical model shown in Fig. 1. This small structure included the expansion joints that contributed to failure, and its natural frequencies and modes were similar to the equivalent values for the actual bridge.

The component/system model was made of microconcrete and consisted of a girder made in three sections, two expansion joints bolted to the girder, and three columns (in later tests four columns) also bolted to the girder. The model was designed to study linear and nonlinear effects in the complete system, material damage being restricted to the columns and to the girder section adjacent to the expansion joints. The design was such that the major part of the girder would remain elastic even under shaking intense enough to cause expansion joint or column failure, and material damage when it occurred would not propagate to the bolted connections [7]. The model was tested separately for horizontal shaking in both the symmetric Y and antisymmetric X directions, and the added effect of the vertical component was also studied with the X and Y tests. The model was tested with different designs of expansion joint restrainer, and in later tests the single central column was replaced with two columns, as this reflects current design practice in California. The seismic input was an accelerogram developed for an adjacent site [12], and this was applied in increasing intensity to failure. The measured dynamic response included the X and Y displacements of the girder at the columns and the movement across the expansion joints measured on the inside and outside edges of the girder. Typical response time-histories of the model are shown in Figs. 4 and 5.

Fig. 4 shows the X displacement of the girder at midspan column 2 and the relative movement across expansion joint 1 on the inside edge due to horizontal shaking in the X (antisymmetric) direction with a peak acceleration of 0.6 g. The primary response was due to vibration in the first antisymmetric mode, with the two expansion joints vibrating out-of-phase in the radial direction. It is evident, particularly from the expansion joint gap movement, that the behavior is quite nonlinear due to impact at the joint which makes the displacements unsymmetrical about the zero position. Also, there is a small residual displacement in the system due to friction and to the yielding of the expansion joint restrainers. Clearly, to analyze such a system it is necessary to use a nonlinear analysis of the type proposed by Penzien [11]. The 5.6 mm (0.22 in) peak relative displacement across the joint represents an opening of 17 cm (6.6 in) in the prototype, and there appeared to be little danger of the center span falling off the 3 cm (15 in.) ledge for this type of motion. Damage to the columns was minor and restricted to small flexural cracking at the bases.

The most serious effect on the bridge of this type of motion was the damage inflicted on the expansion joints. Due to the large impact, shear, and torsion at this discontinuity in the girder, the joint was damaged in three separate ways -- vertical cracking of the ledge, impact spalling of adjacent surfaces (Fig. 2), and fracture of the horizontal shear key. In spite of this damage, however, the structure did not seem to be in imminent danger of total collapse.

Fig. 5 shows typical response curves for horizontal shaking in the Y (symmetric) direction, and it is evident that although the intensity of table motion was less than in Fig. 4 the effects were more severe. This motion caused the bridge to respond primarily in the first symmetric mode, with in-phase opening and closing of the two joints. Yielding of the restrainers caused significant joint opening, and the peak value of 10 mm (0.4 in) indicated that the center spans were in imminent danger of falling off their supports. This motion was accompanied by the same kind of joint fracture caused by horizontal shaking in the X direction. Although subsequent testing with larger capacity restrainers did reduce the joint movement to some extent, it was still excessive; and this design modification did nothing to reduce the failure of the joint itself.

From the tests done on this structure [5,6,7], the following general deductions can be made:

1. The critical locations of the structure where damage occurs, and hence where ductility is required, are at the expansion joints and at the column bases.

2. Compared with a long straight bridge, the curved bridge is potentially much more efficient in an earthquake, provided the girder is continuous. It is the existence of the discontinuities in the deck caused by the expansion joints that greatly reduces this efficiency. If joints are required, then care should be taken both in their location and design, as the deck forces at these locations are potentially very large.

3. The design which incorporated two columns supporting the central girder was much more efficient in suppressing hinge failure. This supports recent changes in bridge design practice.

4. Consideration should be given to eliminating expansion joints altogether in a long curved bridge. This opinion was adopted in the design of the cable-stayed bridge discussed below.

CURVED CABLE-STAYED GIRDER BRIDGE

T. Y. Lin International of San Francisco has recently proposed an interesting concept for a bridge across a river in California where the sides of the canyon are steep and cause problems in the siting of the approach roads [13,14]. The solution involves a 396 m (1300 ft) span box girder with a 458 m (1500 ft) horizontal radius of curvature supported on 48 cables, each of which is individually anchored to the canyon wall (Fig. 6). The girder is a continuous box section. Model and analytical studies were done concurrently so that the correlation between computed and measured values could be studied.

As the bridge was designed to remain essentially elastic under seismic conditions, the model was designed to study all effects, with the exception of material damage or local buckling. A distorted model was used in which the cross section properties, mass, and mass distribution of the girder were reproduced to scale, together with the flexibility and mass distribution of the cables. The resulting model (Fig. 7) was comparatively simple and capable of reproducing dead load effects, static live loads, natural modes and frequencies, and seismic responses for prescribed table motions. All potentially nonlinear effects from such sources as cable sag and local cable vibrations were correctly reproduced. After the shaking tests of the continuous bridge were completed, an expansion joint was introduced at midspan so that comparative response data could be studied. Typical response data for the continuous bridge, in terms of prototype quantities, are shown in Figs. 8 and 9.

The response of the bridge to horizontal shaking in the symmetric Y direction was virtually zero and is not shown. The horizontal curvature and resulting arching action of the girder effectively resulted in it moving as a rigid body without significant vibration in any of the symmetric modes. Cable vibrations were evident but had no measurable effect on the total response of the system. The response to shaking in the antisymmetric X direction was greater and resulted in vibration primarily in the first antisymmetric mode of the girder, as in the previous study. Fig. 8 shows the gross horizontal bending moment in the girder at midspan, 1/4 span, and at the abutment. The midspan response was virtually zero, as this was an inflexion point. The response at the other two sections was out-of-phase, with the maximum values occurring at the abutment. Although the dynamic response was measurable for this component of prescribed ground motion, the resulting values were small and well within the design stresses for the girder. Again, the cable vibrations had only a very minor effect on the overall response of the system.

The largest dynamic response of the bridge was due to the vertical component of ground motion, and this was primarily in the first vertical mode for the prescribed table motion. This can be seen in Fig. 9, which shows the table accelerations and the resulting forces in three cables. It will be noted that the response is almost sinusoidal in the first mode and that the decay of motion is slow, indicating a very small value of system damping. This damping, measured in preliminary dynamic tests at approximately 0.1%, is apparently not affected in this design either by the existence of a set of cables of different natural frequencies or by the visible local cable vibrations produced during the seismic event. This result differs from previous general observations made in connection with the dynamic response of cable-stayed girder bridges [15,16,17].

The model was used to study the validity of using a linear dynamic analysis on such a system and the possible bounds of linearity. By increasing the intensity of shaking, the response of the model was taken to a point where the vertical dynamic displacements of the girder at midspan reached a maximum of 1.2 m (4 ft) prototype; and even at such large displacements in the fundamental mode, the overall response was still essentially linear.

The correlation between analytical and measured values was very close at all stages. Static deflection values were within 6%, significant natural frequencies within 3%, and dynamic response to seismic motions approximately within 10%. Subsequent tests on the bridge with a midspan expansion joint showed an increased horizontal response of the system due to shaking in the symmetric Y direction and impact forces in the deck. The presence of the joint did not, however, increase the vertical response of the bridge due to vertical ground motion, though it did make it nonlinear.

The overall test program on this structure led to the following general observations:

1. This design is very effective in resisting all components of horizontal ground motion due to the curvature and continuity of the girder.

2. The primary response of the bridge is in the vertical direction due to the vertical component of ground motion, and the effects of vertical and horizontal shaking are effectively uncoupled.

3. Cable vibrations have very little influence on the overall response of the system, either in increasing the system damping or in producing nonlinear effects in the seismic response.

4. A linear dynamic analysis which neglects cable vibration gives results accurate enough for design purposes, and the limits of linear response are large.

5. The presence of a midspan expansion joint in the deck may change the dynamic response, depending on the value of prestress across the joint. For the design prestress given, it increased the transverse girder moments under severe ground motion in the symmetric direction and caused impacting. Under strong vertical excitation, the maximum vertical response of the girder was increased very little.

CONCLUDING REMARKS

These two studies indicate the value of structural models in investigating the dynamic behavior of large scale bridge systems under seismic excitation. The overall dynamic characteristics, such as the effects of horizontal curvature, become evident. The usefulness and limitations of linear and nonlinear theory can be determined. Specific problems, such as designing for the large dynamic forces that occur at expansion joints in a curved structure, can be studied. Finally, it should be pointed out that the shaking table tests discussed give the response of the system to rigid-body ground motions. Differential ground motions may also have to be considered in the design of a long structure, particularly when it is continuous.

ACKNOWLEDGEMENTS

The first study was sponsored by the U.S. Department of Transportation, Federal Highway Administration, and the author wishes to express his sincere thanks to Dr. David Williams, who designed and tested the model, and to Professor Joseph Penzien, who supervised the associated analysis. The second study was sponsored by the U.S. Department of the Interior, Bureau of Reclamation, and the author wishes to express his appreciation to Professor T. Y. Lin and the staff of T. Y. Lin International, who designed the bridge, and to Dr. Z-A Lu, who conducted the analytical studies.

DISCLAIMER

The contents of this paper reflect the views of the author, who is responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the opinions or policy of the sponsors or the designers.

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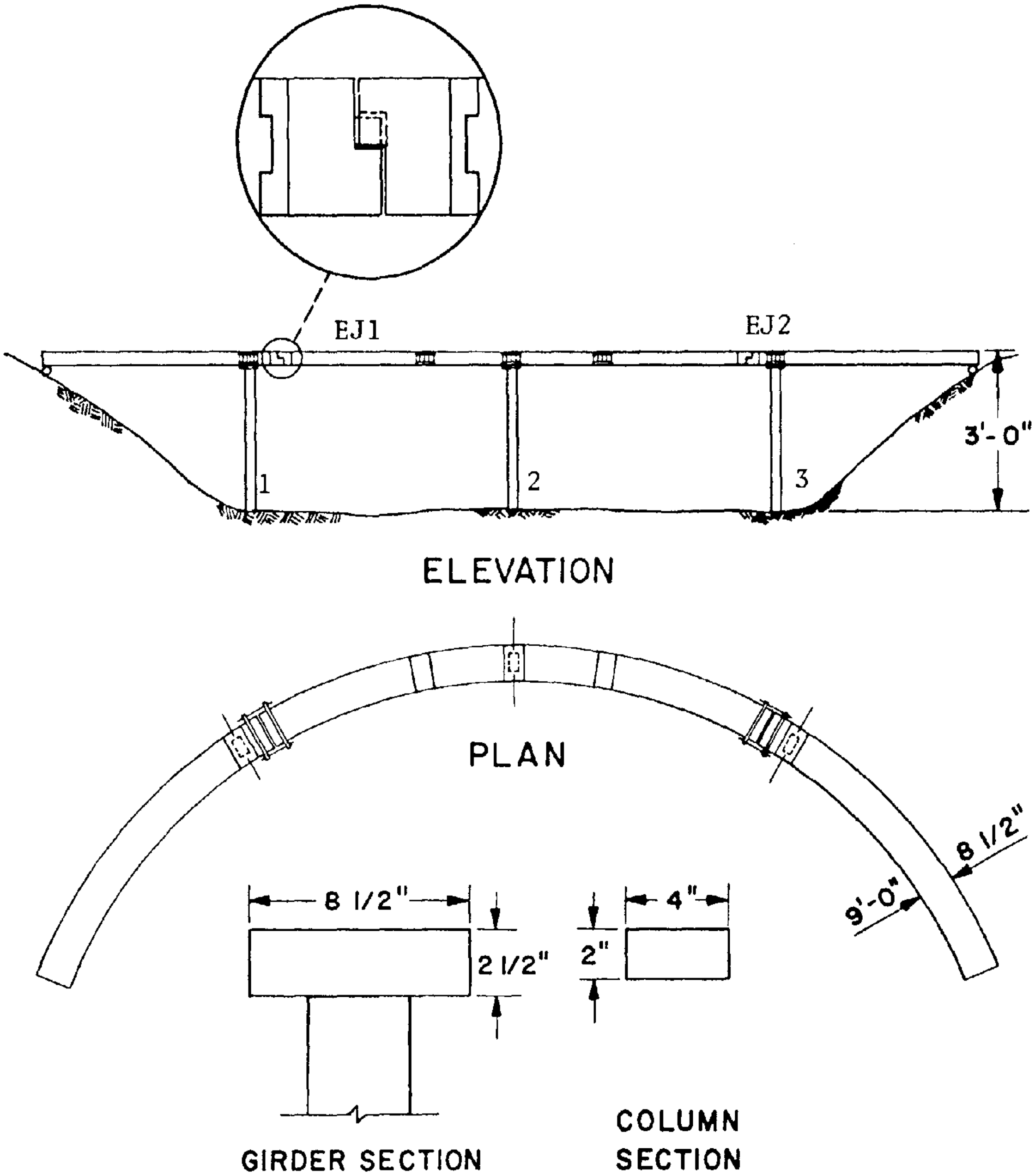

Fig. 1   Schematic of Test Model

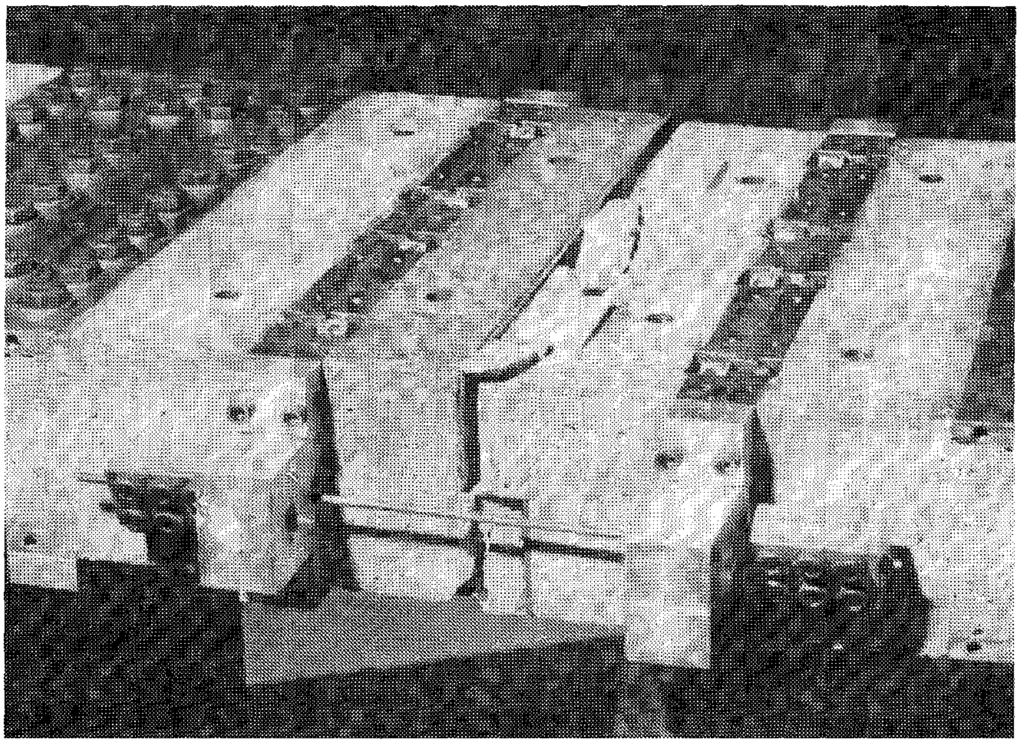

Fig. 2 Damaged Expansion Joint

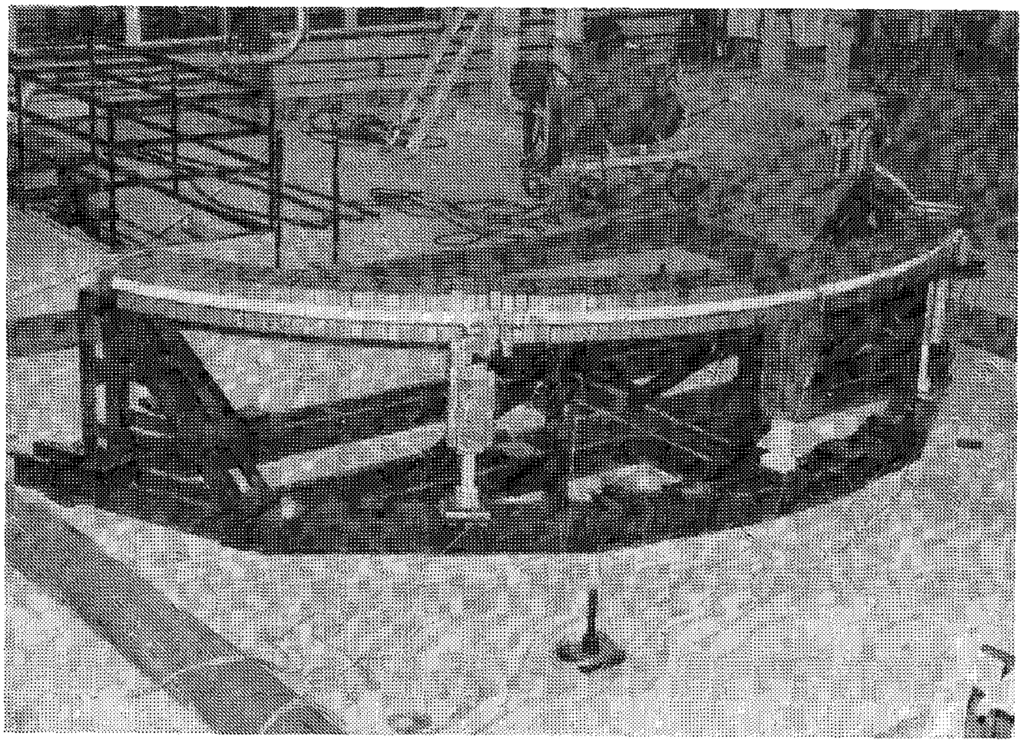

Fig. 3 Model on Shaking Table

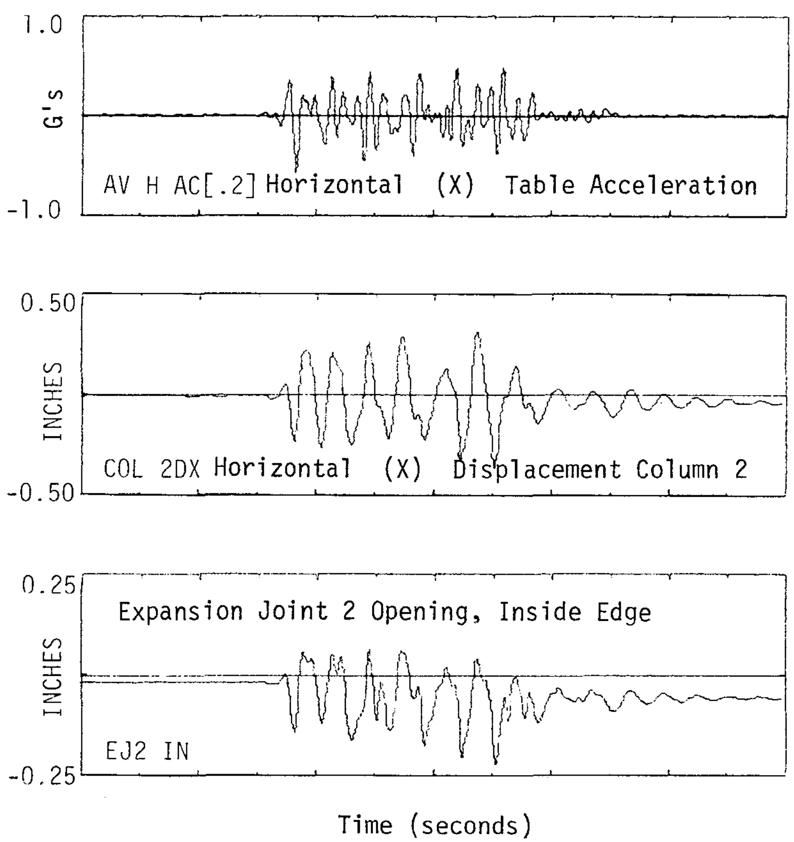

Fig. 4 Response to Horizontal X Table Motion

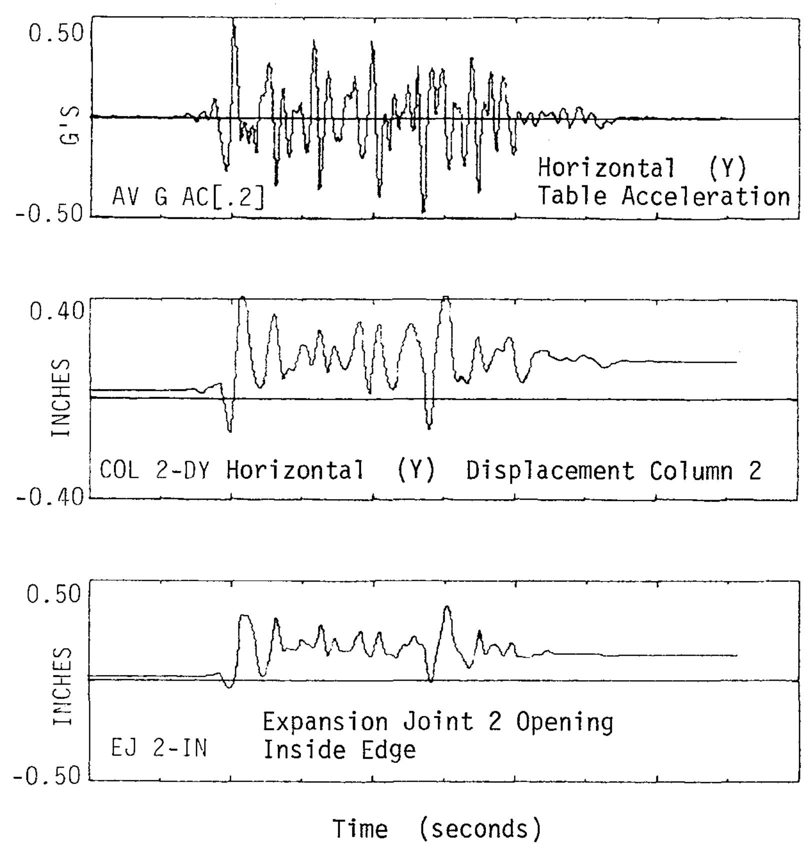

Fig. 5 Response to Horizontal Y Table Motion

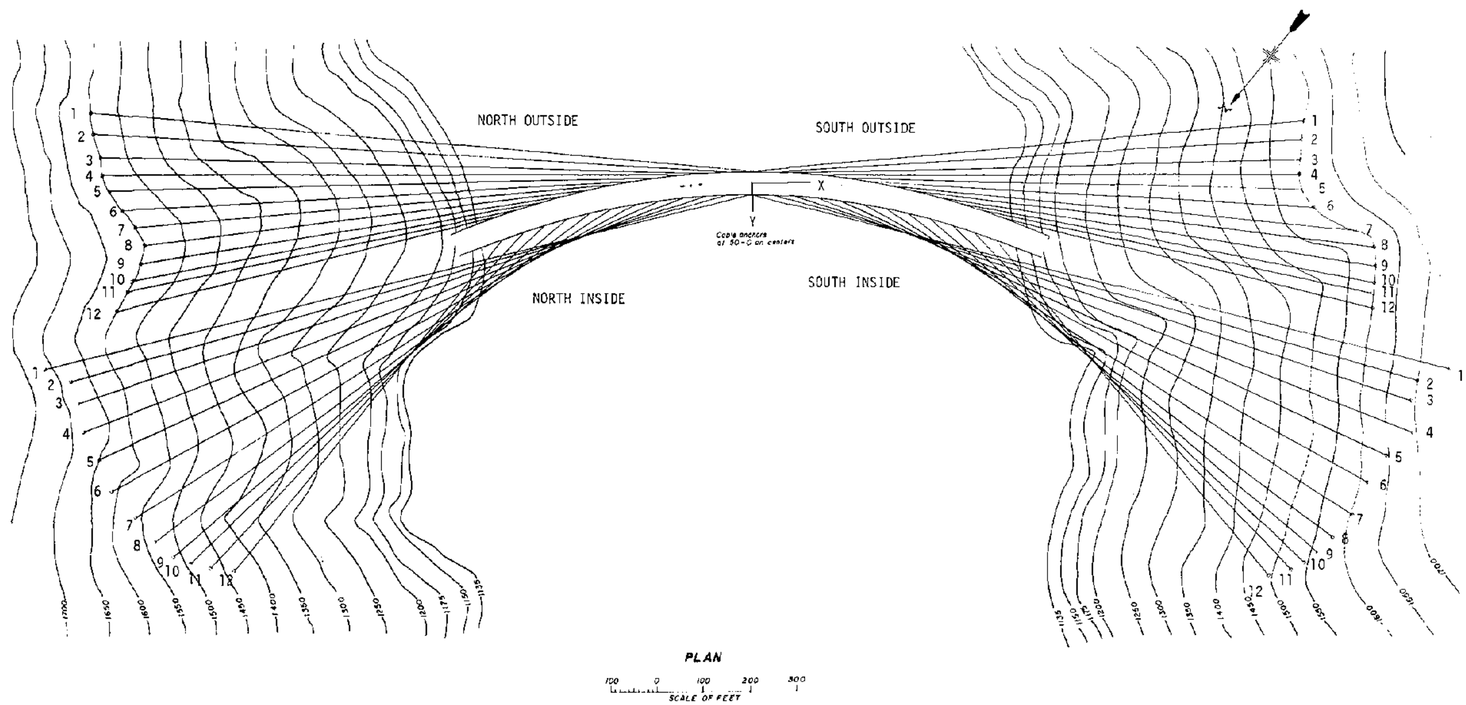

Fig. 6 Layout of Prototype Bridge

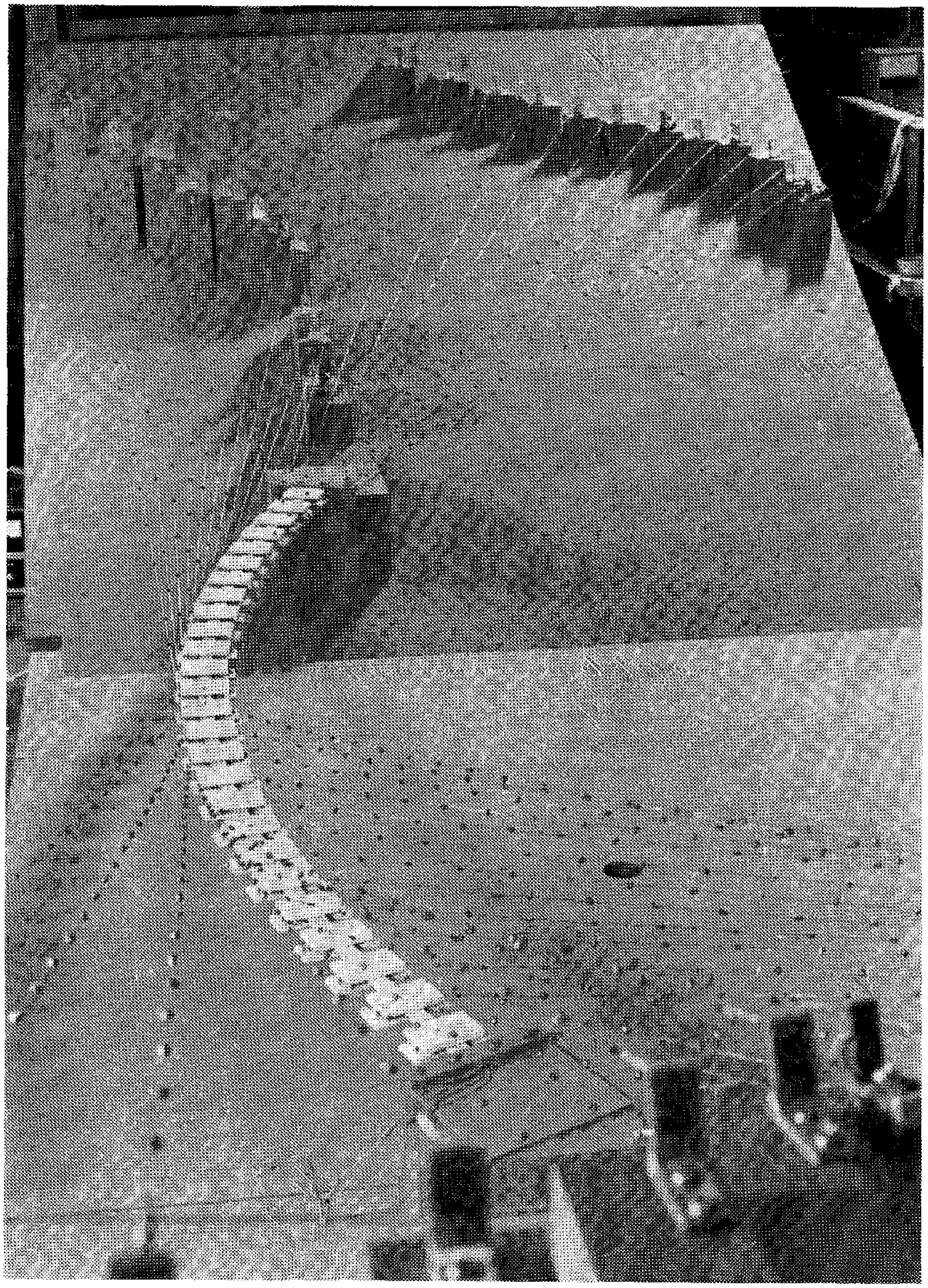

Fig. 7 1/200th Scale Bridge Model

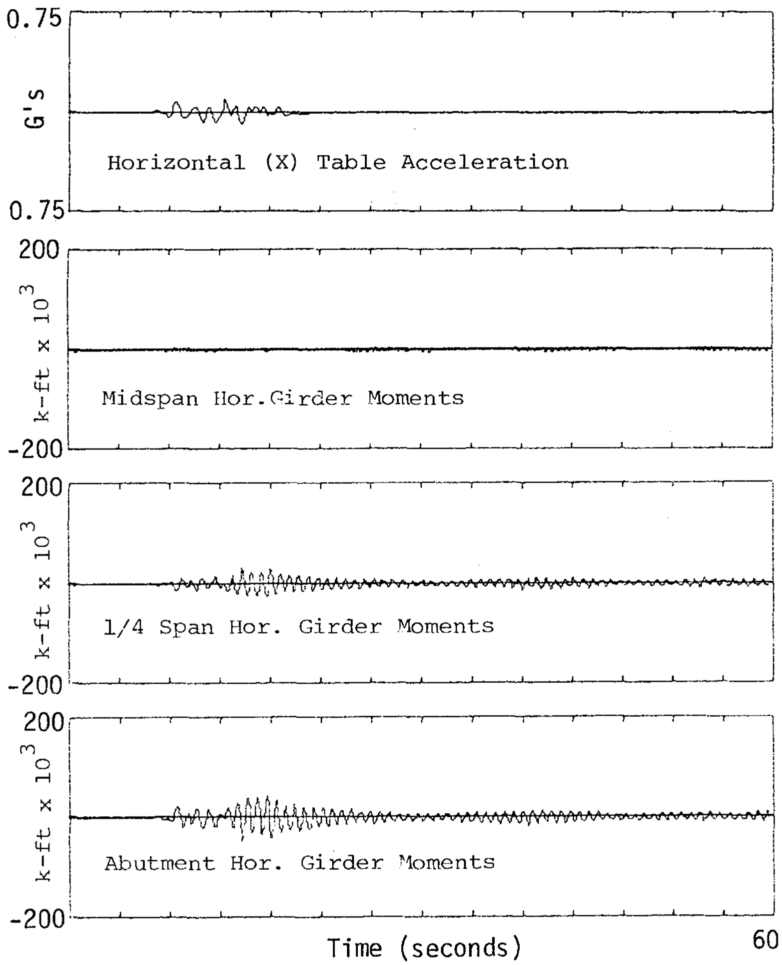

Fig. 8 Response to Horizontal X Table Motion

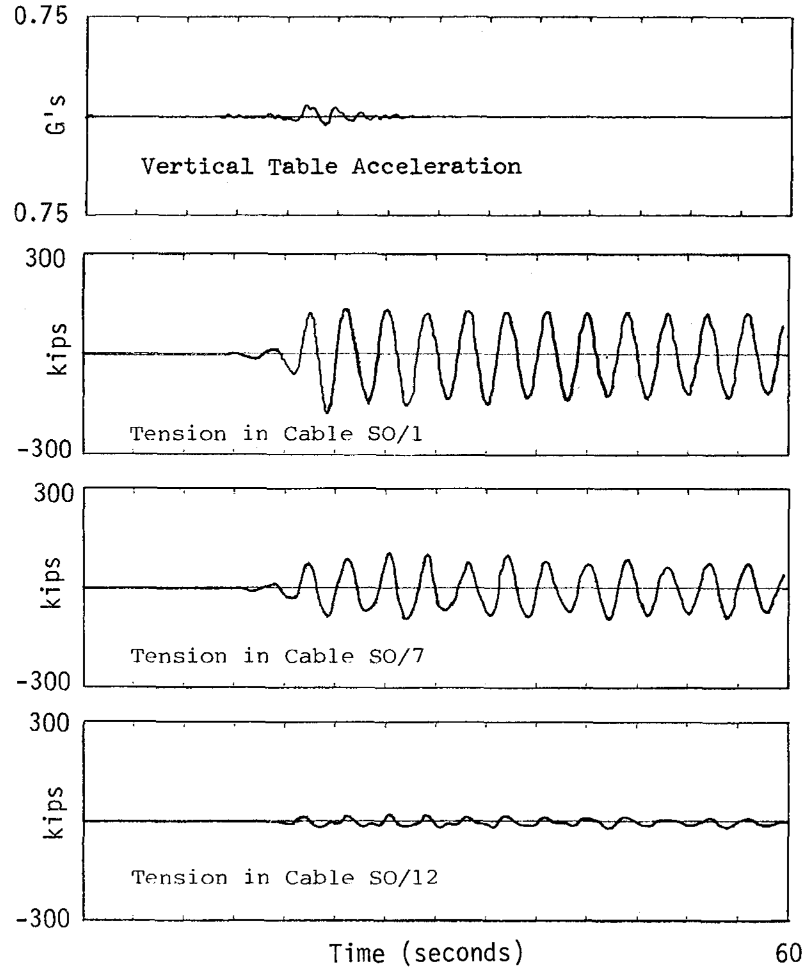

Fig. 9 Response to Vertical Table Motion

SEISMIC ANALYSIS OF MULTI-SPAN REINFORCED CONCRETE BRIDGES

by

Joseph Penzien

SUMMARY

Presented is a brief description of nonlinear mathematical modelling and analysis procedures for predicting the response of multi-span reinforced concrete bridges to strong seismic ground motions. The validity of these procedures is verified through a correlation of predicted response with the measured response of a model bridge structure when subjected to simulated seismic motions using the University of California, Berkeley, two-component shaking table. It is concluded that these theoretical procedures can be used effectively in developing improved seismic design criteria and that they can be of great assistance in developing the design of an unusual structure.

INTRODUCTION

The susceptibility of modern multi-span reinforced concrete bridge structures to damage from the vibratory effect of strong seismic ground motions became evident during the San Fernando, California, earthquake of February 9, 1971. Because many bridges of this type suffered severe damages, it was clear that the seismic design criteria in effect at that time were inadequate. The State of California Department of Transportation immediately revised its seismic design requirements with the intent of providing for satisfactory seismic performance of new bridges and it initiated a retro-fit program to upgrade certain existing bridges. At about the same time, the Federal Highway Administration of the U.S. Department of Transportation, initiated a research program at the University of California, Berkeley, to critically examine current seismic design methodologies for bridges and to develop improved procedures which could be used in the design process. This program has continued to the present time resulting in numerous publications on the various phases of the overall program[1-10].

The purpose of this paper is to present a brief description of the mathematical modelling and analysis procedures[2,3,4,8] developed during the program for predicting the nonlinear seismic response of curved (or straight) multi-span reinforced concrete bridges and to verify the validity of these procedures through a correlation of theoretical response with the measured response of a model bridge structure when subjected to simulated seismic motions using the University of California, Berkeley, two-component shaking table[7,8]. The details of the experimental investigation are presented in a companion paper entitled "Seismic Model Studies of Long Span Curved Bridges" by W. G. Godden.

ANALYTICAL PROCEDURES

The type of bridge structure considered herein normally consists of multiple spans having columns of various lengths supporting a cellular deck which may be straight or curved in its plan view. Expansion joints which greatly affect the dynamic response characteristics of the overall system are usually present in the deck. The 5/14 South Connector Overcrossing shown schematically in Fig. 1 is representative of this type of structure. This particular overcrossing experienced total collapse of spans 3-4 and 4-5 during the San Fernando earthquake.

Mathematical modelling of the overall structure is accomplished by a discrete parameter system as indicated in Fig. 2. Since the deck is sufficiently strong to remain elastic during severe earthquakes, it is modelled using linear curved beam elements having six degrees of freedom at each end (3 translational and 3 rotational). The columns are modelled using elasto-plastic straight beam elements having a similar number of degrees of freedom at each end. Yielding of these elements is assumed to occur only due to the combined action of axial force and biaxial bending, i.e. deformations due to transverse shear and torsion are assumed to remain elastic. The elasto-plastic behavior of the column elements is therefore completely characterized by the yield surface defined in Fig. 3 which shows the axial force P and each bending component M_y and M_z in their non-dimensional forms P/P_o, M_y/M_{yo}, and M_z/M_{zo}, respectively, where P_o, M_{yo}, and M_{zo} represent yield values. Foundation conditions at the base of each column are modelled by discrete springs and dashpots as shown in Fig. 2.

Analyses have shown that the characteristics of the expansion joints greatly influence the nonlinear seismic response of certain bridges, particularly those which are curved in plan view. As shown in Fig. 4, these joints are usually provided with shear keys to prevent transverse shear displacements, vertical restrainers to prevent vertical uplift, and longitudinal tie bars (or cables) to resist longitudinal separations in the joint. Modelling is accomplished as shown in Fig. 5 where the end diaphragms of the deck at the joint are treated as rigid bars interconnected by a transverse elastic spring representing the shear keys, two vertical elastic springs representing the vertical restrainers, two very stiff elastic impact springs to model impact behavior upon closure of the joint, tension elasto-plastic tie bar elements with gaps at one end, and longitudinal elasto-plastic shear elements to model the behavior of the elastomeric pad placed within the support interface of the joint (not shown in Fig. 5), including coulomb friction which is present upon sliding[2,8].

The mass of the bridge structure is lumped at discrete nodal points as shown in Fig. 6 for the 5/14 South Connector Overcrossing with six degrees of freedom permitted at each nodal point. Earthquake excitations are prescribed at the boundary nodal points. The dynamic equilibrium equations of motion are formed to include each degree of freedom permitted in the system with viscous damping introduced to represent energy dissipation in the linear elastic range of response. Because of the various nonlinear elements introduced into the overall model, these equations of motion are solved numerically using step-by-step integration procedures[2,8].

CORRELATION STUDIES

As part of the overall investigation, an experimental program was conducted by subjecting a model bridge structure to simulated seismic motions using a two-component shaking table[7]. This model structure, shown schematically in Fig. 7, was constructed to have features similar to those of the 5/14 South Connector Overcrossing shown in Fig. 1, i.e., a curved deck with expansion joints modelled after the prototype and with supporting columns which would yield at scaled force levels. The details of this investigation are described in the companion paper "Seismic Model Studies of Long Span Curved Bridges" by W. G. Godden which was mentioned earlier.

To verify the validity of the mathematical modelling and analysis procedures previously described, correlation studies were carried out on selected tests conducted during the experimental investigation mentioned above. Due to the general nature of this paper, the correlation study for only one test will be discussed herein, namely, a test during which the model structure was subjected simultaneously to the transverse horizontal excitation shown in Fig. 8 (0.5 g peak acceleration) and the vertical excitation shown in Fig. 9 (0.25 g peak acceleration). This input produced the time-history of transverse deck displacement at mid span of the center girder (see Fig. 7) shown in Fig. 10. The solid curve in this figure is the theoretically predicted time-history of response using a linear elastic model for the complete structure while the dashed curve is the experimentally measured response. Because of the nonlinearities produced by slippages and impacts which occurred in the two expansion joints and the yielding which developed in the tension tie bars, large discrepancies are apparent between the measured response and the predicted response. On the other hand, if nonlinear modelling of the complete structure is used, the predicted response is as shown by the solid curve in Fig. 11. Note that this response agrees well with the measured response, thus indicating the validity of the nonlinear mathematical modelling and analysis procedures previously described.

To further illustrate the correlation of predicted and measured response during the test, the predicted and measured time-histories of average separation of one expansion joint (No. 2) are shown in Fig. 12 for the linear model and in Fig. 13 for the nonlinear model. Again, it is very apparent that linear modelling leads to large errors in the predicted response while nonlinear modelling gives very satisfactory results. Clearly, if one is to predict certain important nonlinear behavior within an expansion joint, nonlinear modelling is essential. For example, Fig. 14 shows the time-histories of force and yield elongation in one tension tie bar at expansion joint No. 2 using the nonlinear modelling and analysis procedures. Obviously, this important realistic behavior could not possibly be predicted through linear modelling.

CONCLUDING REMARKS

Based on the good correlations found between predicted and measured response of the model bridge structure, it is concluded that the nonlinear mathematical modelling and analysis procedures described herein for predicting the nonlinear seismic response of multi-span reinforced concrete bridge structures are reasonably valid. Therefore, they can be used effectively in developing improved seismic design criteria and can be of great assistance in developing the design of unusual bridge structures.

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DISCLAIMER

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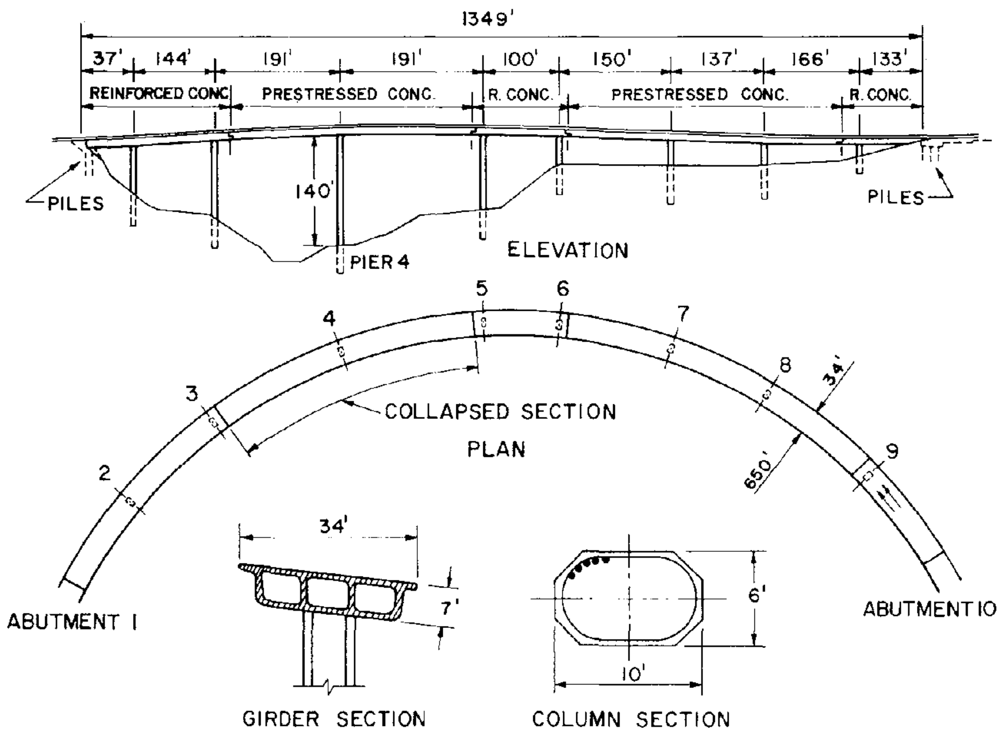

Fig. 1 5/14 South Connector Overcrossing

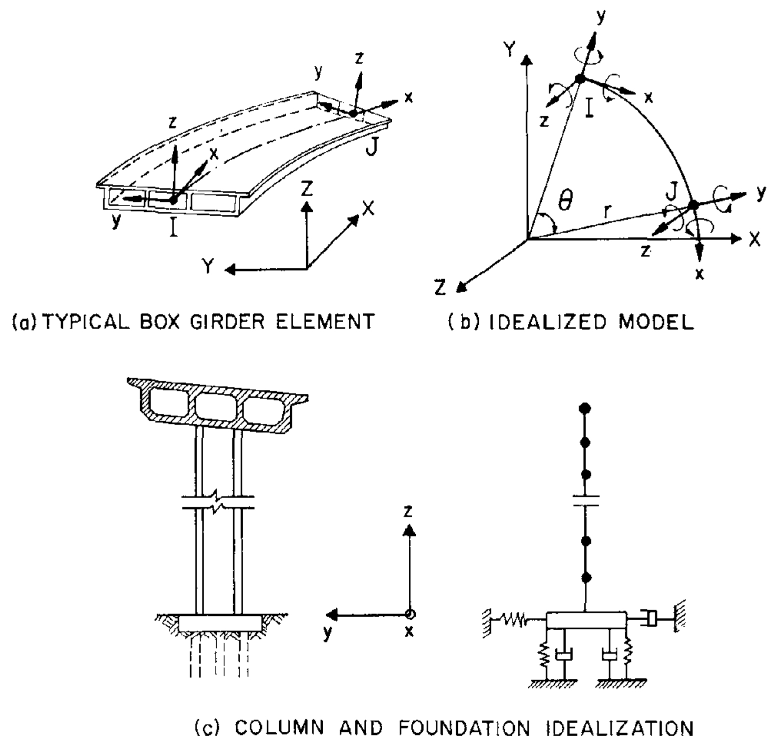

Fig. 2 Mathematical modelling of bridge structural system

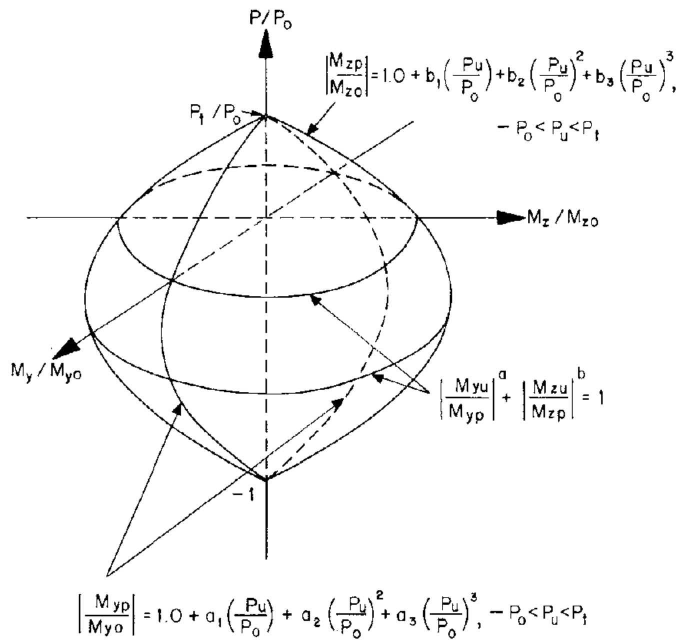

Fig. 3 Yield surface of typical bridge column

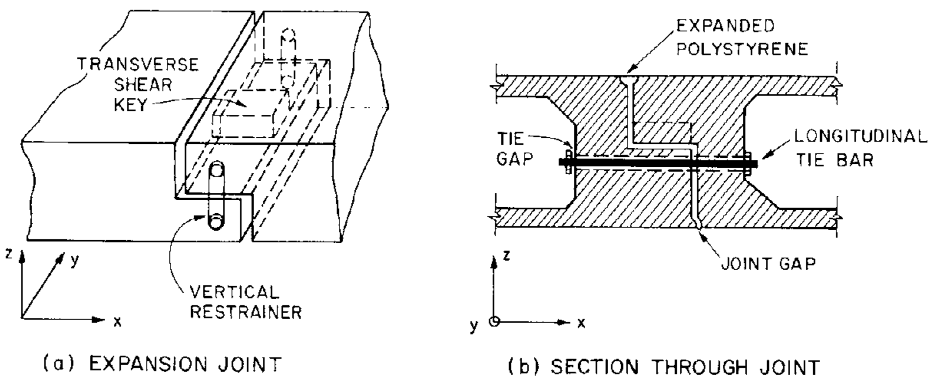

Fig. 4 Bridge expansion joint

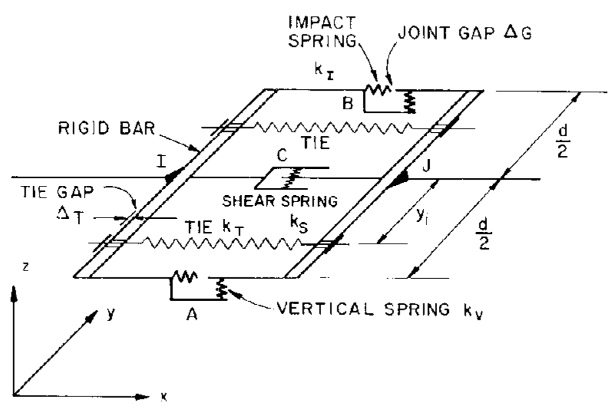

Fig. 5 Idealized expansion joint

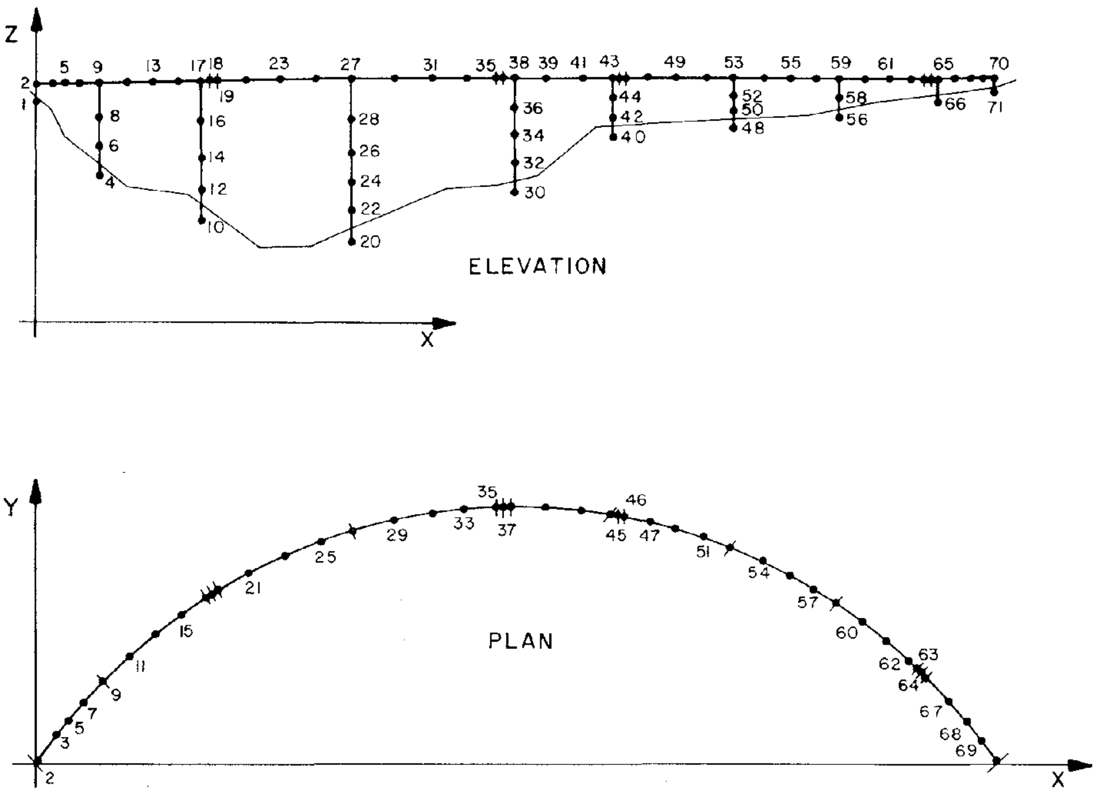

Fig. 6 Lumped mass model - 5/14 South Connector Overcrossing

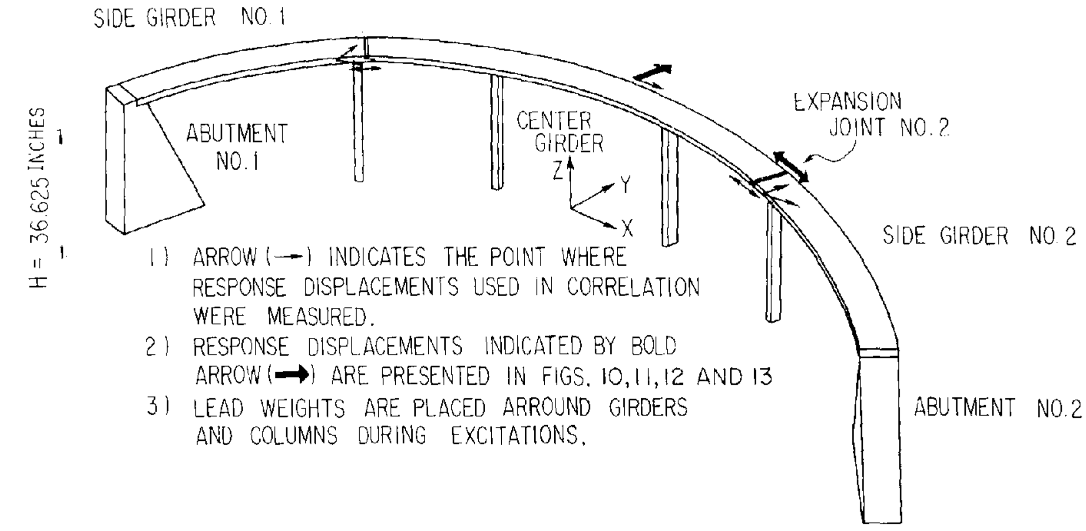

Fig. 7 Model bridge structure

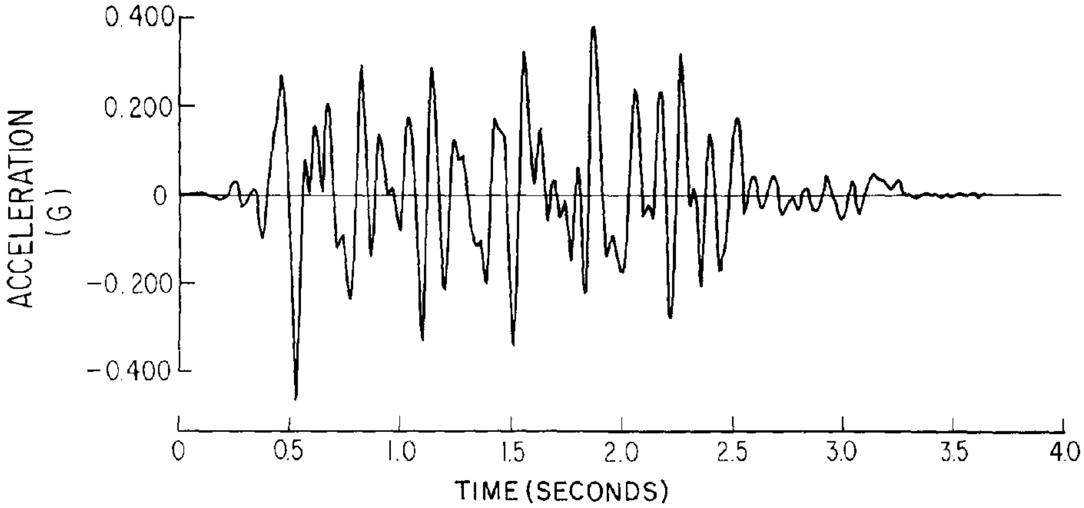

Fig. 8 Transverse support acceleration

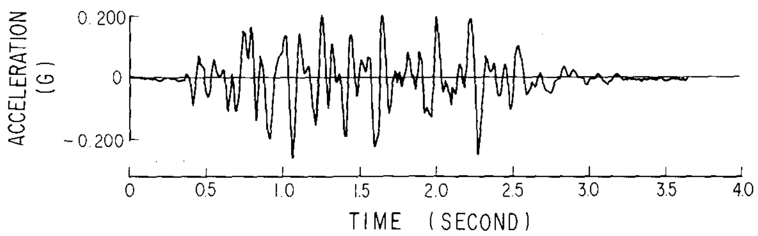

Fig. 9 Vertical support acceleration

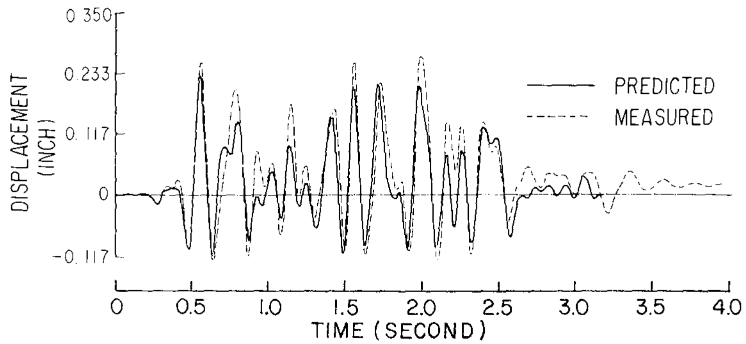

Fig. 11. Transverse deck displacement at location of center column - Nonlinear Model

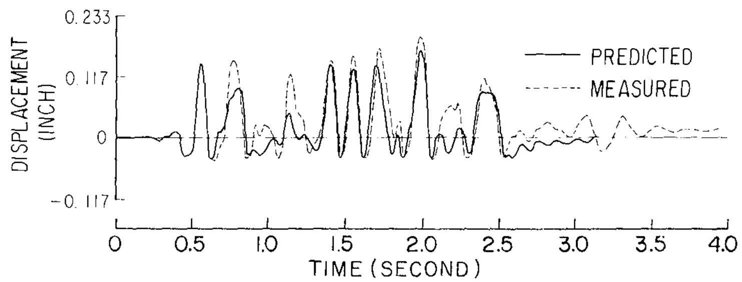

Fig. 13 Expansion joint separation - Nonlinear Model

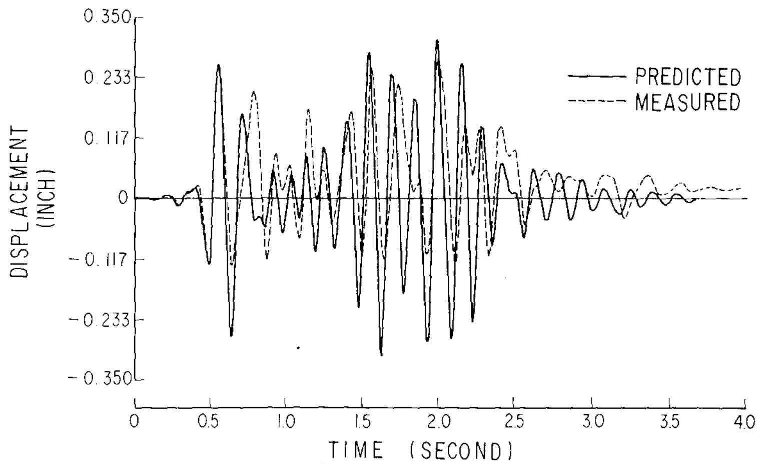

Fig. 10. Transverse deck displacement at location of center column - Linear Model

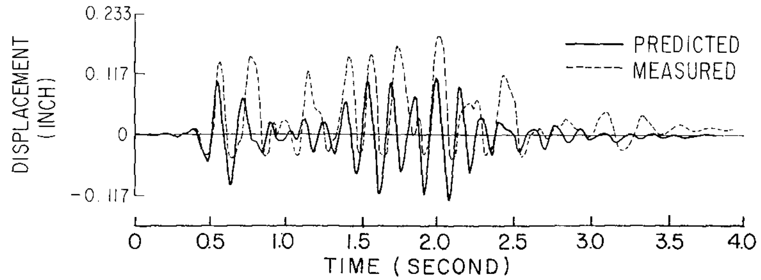

Fig. 12 Expansion joint separation - Linear Model

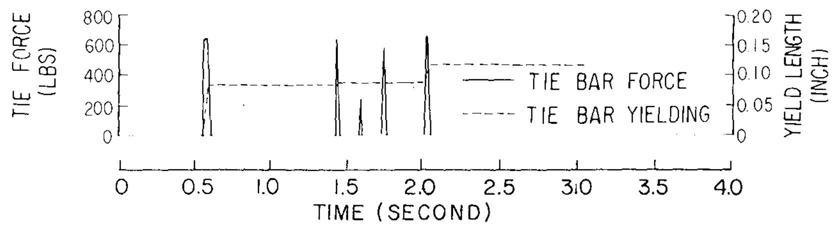

Fig. 14 Tensile force and yield extension of tie bar - Nonlinear Model

SOIL-STRUCTURE INTERACTION OF SHORT HIGHWAY BRIDGES

by

MA-CHI CHEN
Project Engineer
Engineering Data Analysis Company

JOSEPH PENZIEN
Professor of Structural Engineering
University of California

ABSTRACT

One of the objectives of the investigation is to develop suitable mathematical models for earthquake response analysis of short highway bridges of the type where soil-structure interaction effects are important. A three-span bridge was investigated with various degrees of complexity in the analytical model. Parameters affecting the soil-structure interaction are identified through numerical examples.

Design approaches specified by CALTRANS design code and other simple design methods are compared to the analytical results for the seismic design force. The Mononobe-Okabe method is examined for the dynamic lateral pressure on the abutment wall.

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INTRODUCTION

The seismic response of short highway bridges are greatly affected by the phenomena of soil-structure interaction. The dynamic forces exerted by backfill soils on the abutments influence the distribution of seismic forces developed in the overall structural system. Short span bridges usually have relatively short and stiff columns which interact strongly with their supporting foundations. Skewness of a bridge also influences the distribution of seismic forces due to the torsional response of its interaction with the backfill soils.

To study the effects of soil-structure interaction and skewness on bridge response, two computer programs were developed. The first, a program for analyzing two-dimensional mathematical model, considers nonlinearities of the soils, yielding of concrete columns under combined axial forces and bending moment, and separations and impacts between abutments and backfills. The second, a program for analyzing three-dimensional mathematical model, limits the nonlinearities considered to only separation and impact between abutments and backfills.

TWO-DIMENSIONAL MODEL

Four basic elements are used in modelling two-dimensional bridge and soil systems [1].

1. Soil Finite Element

The soils adjacent to the abutment are modelled by two-dimensional finite elements which may have an arbitrary quadrilateral or triangular shape. Elastic-perfectly plastic material properties are assumed using the Mohr-Coulomb yield criterion shown in Fig. 1.

2. Prismatic Beam Element

Prismatic beam elements are used to model the bridge deck, columns, and equivalent columns representing the soil foundation. The force components acting on each beam element are axial force, shearing force, and bending moment.

The first step in finding the equivalent column for the foundation is to determine the lateral, vertical, and rotational stiffnesses of the foundation at the footing (or pile cap) level. Once these three stiffnesses have been determined, the foundation is replaced by a column of length L, flexural stiffness EI, and axial stiffness AE which, when fixed at its base, provides the equivalent lateral, vertical, and rotational stiffnesses to the footing. The foundation stiffnesses can be obtained by either the numerical procedure outlined by Penzien or by a closed form approach reported by Gerrand and Harrison [2,3].

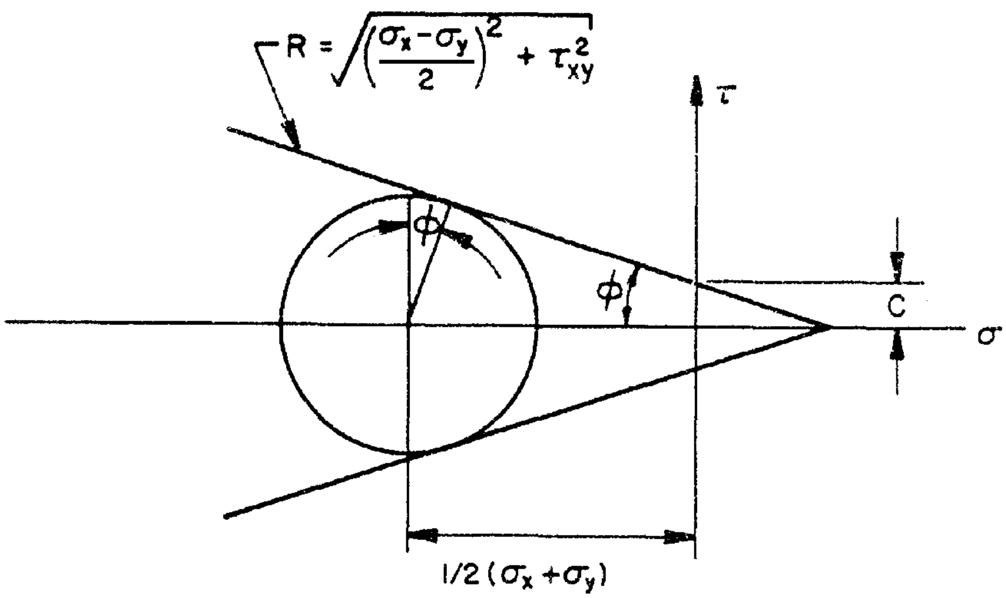


FIG. 1 MOHR-COULOMB YIELD FUNCTION

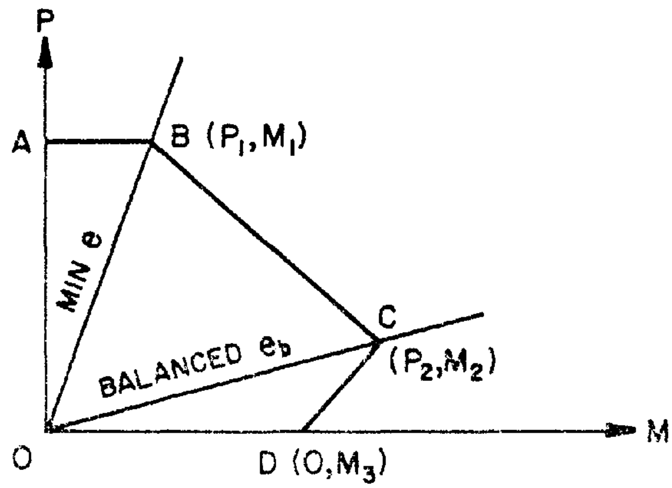


FIG. 2 COLUMN INTERACTION CURVE

3. Soil Boundary Element

Linear springs are used as soil boundary elements to account for the elastic action which occurs at the vertical boundaries of the soils being considered.

4. Frictional Element

A so-called frictional element is used to model the frictional action, separation, and impact which take place at the interfaces of soil backfills, and abutment walls. This element has the following characteristics (1) the frictional force per unit area is proportional to the normal interface pressure and a coefficient of friction; thus, slippage occurs when the direction angle of the resultant of pressure and friction exceeds the angle of friction of wall and soil, (2) impact occurs at the interface upon closure of any gap which may have earlier developed, and (3) no frictional resistance can develop at the interface when wall and soil surfaces have separated. Discontinuous elements similar to this have been developed by Ghaboussi and Wilson, Scholes and Strover, White and Enderly, and Tseng and Penzien [4,5,6,7]; however, the element developed by Goodman and Taylor has been adopted [8].

THREE-DIMENSIONAL MODEL

The three-dimensional mathematical model consists of four elements [9].

1. Solid Finite Element

An eight-node isoparametric hexahedron is used to model the abutment walls and backfills. Linear elastic isotropic material properties are specified for each element.

2. Beam Element

The three-dimensional prismatic beam element used to represent the bridge deck, pier columns, and pier caps, was assumed to be linear elastic. The deformations considered in the element were those caused by torsion, bending about the two principal axes of the cross-section, axial force, and the two-components of transverse shear.

3. Frictional Element

The frictional element representing separation, impact, and slippage at the interfaces of abutments and backfills uses relative displacements as independent degrees-of-freedom [10,11].

4. Boundary Element

A boundary element is used for modelling foundation flexibility at the base of columns supported on either piles or mat footings and soil flexibility at both horizontal and vertical boundaries of the backfill models, when necessary. The element consists of 3 translational and 3 rotational degrees of freedom. The individual stiffness in each degree of freedom can be approximated using either numerical or closed form solutions [1,3,12].

INPUT EARTHQUAKE

In the numerical results being presented, the horizontal ground motion was prescribed in accordance with the acceleration time-history shown in Fig. 3. This artificial accelerogram was generated by A.K. Chopra to simulate the ground motions produced by the San Fernando earthquake at the site of the Olive View Hospital located about 6 miles southwest of the epicenter [17]. It has a peak acceleration of 0.5 g and a uniform phase of high intensity shaking for 8 seconds.

The vertical ground motions were assumed zero for the present study, but the computer program has the option to permit input of vertical ground motions.

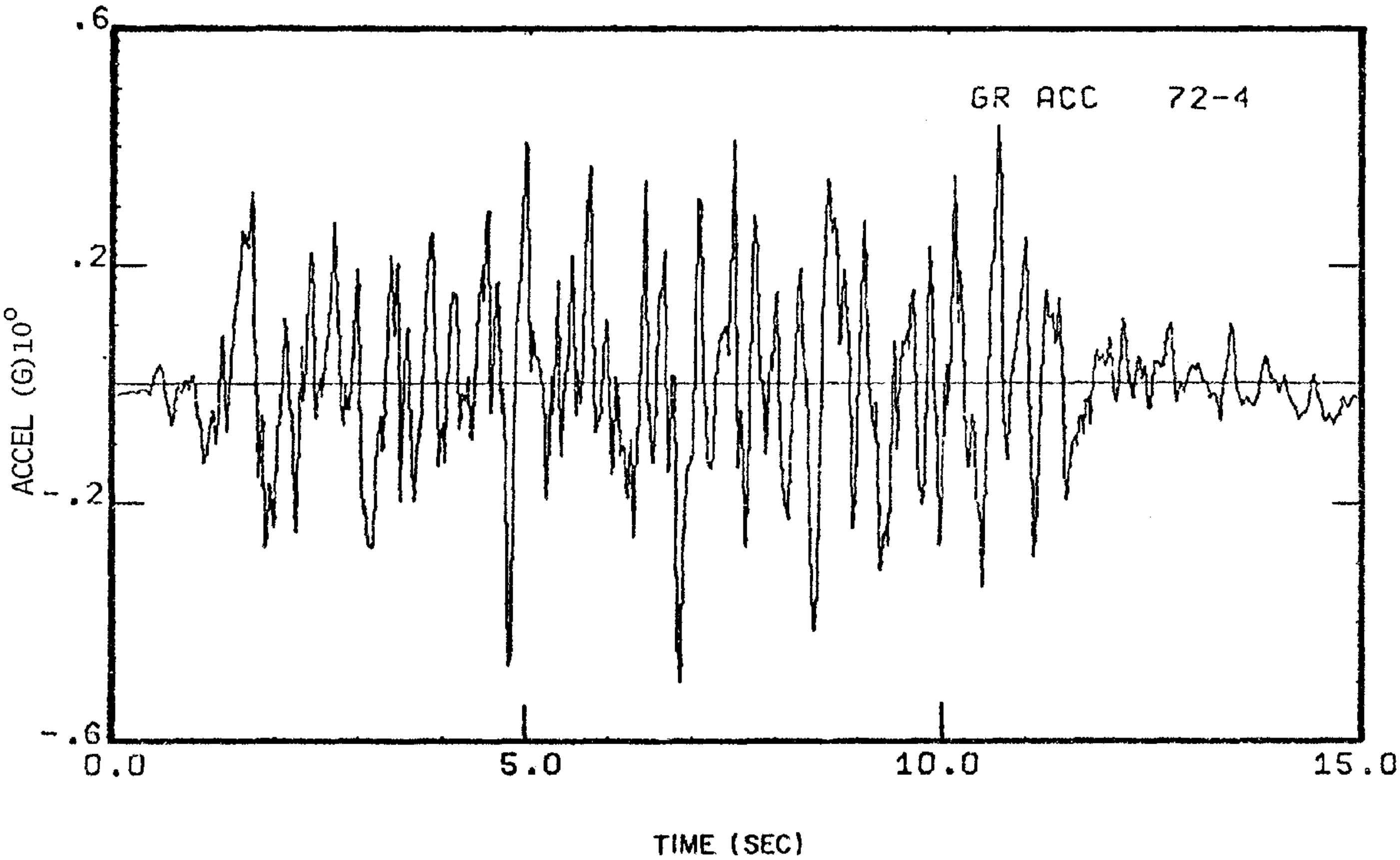


Fig. 3 Simulated ground acceleration record of the San Fernando Earthquake at the Olive View Hospital site

NUMERICAL RESULTS

The previously defined mathematical modelling procedures have been applied to a bridge similar to the Nother Connector Undercrossing located approximately 800 feet northerly of Route 5--San Fernando Road Interchange in the city and county of Los Angeles. Plan and evaluation views of this bridge are shown in Fig. 4.

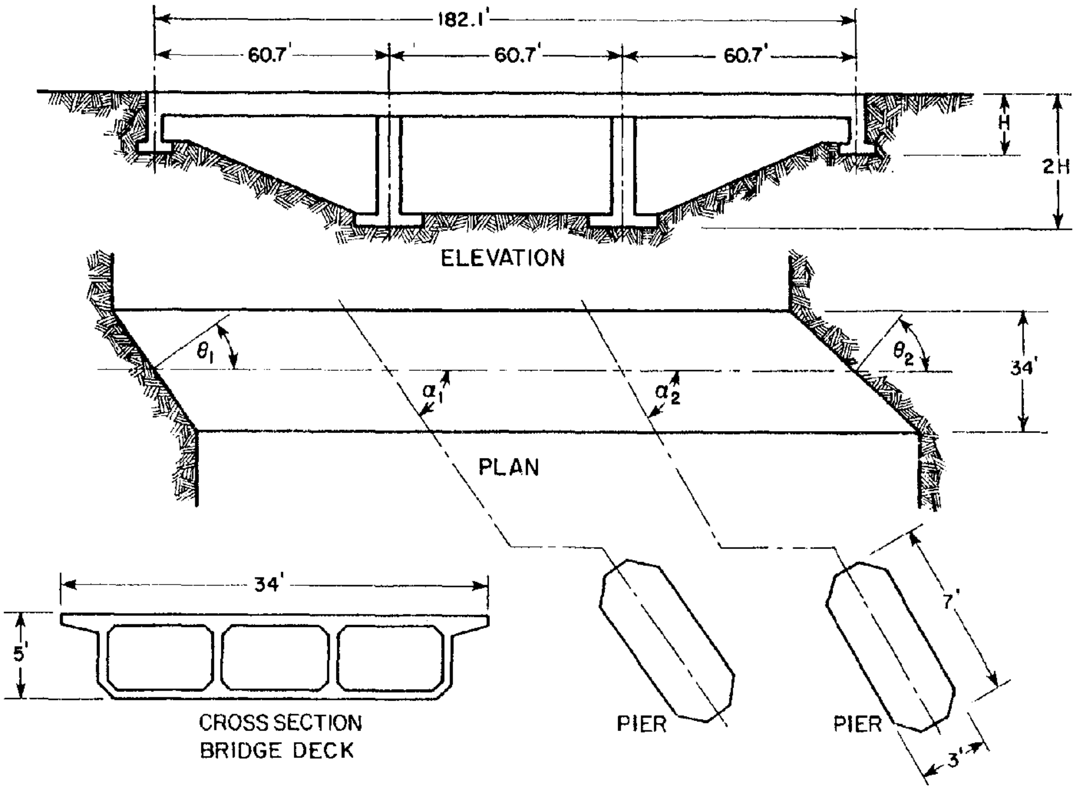
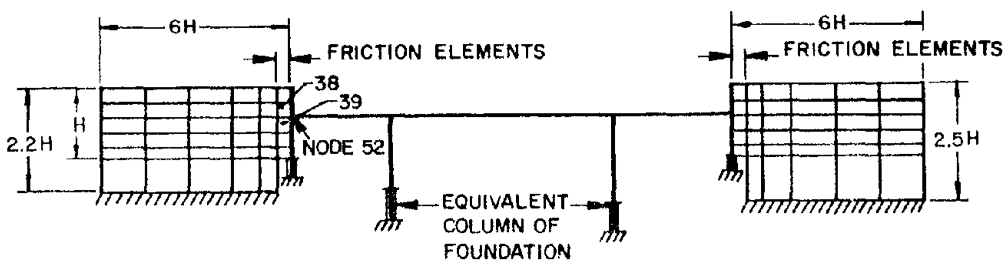


Fig. 4 General plan of model bridge

1. Dynamic Soil Pressure Distribution on Abutments

Considering a straight version of the bridge in two dimensional form, an appropriate mathematical model of the entire bridge-soil system was established as shown in Fig. 5. The backfills have fixed boundary conditions at depth 2.2 and 2.5 times the height of abutment, H, which correspond to base elevations of columns. Longitudinally, the backfills extend a distance 6H in the model as shown. Friction elements are placed between the backfills and the abutments. The bases of the columns and the abutments are attached to equivalent columns representing their corresponding foundation flexibility. The maximum dynamic soil pressure distribution on one of the abutments is shown in Fig. 6. The resultant of this distribution is located 0.54H from the base of abutment.



SUBSTRUCTURES ON EQUIVALENT COLUMNS

FIG. 5 NORTH CONNECTOR UNDERCROSSING, BRIDGE-SOIL SYSTEMS

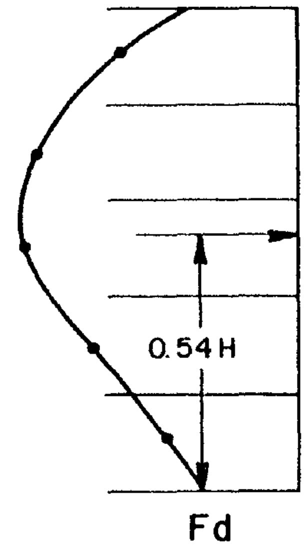


FIG. 6 MAXIMUM DYNAMICAL PRESSURE DISTRIBUTION

2. Effects of Skew on Bridge Response

To study the effects of skew on dynamic response, the same bridge previously described is modelled with three different degrees of skewness as shown in Fig. 7. Model A has no skew and the backfills extend laterally only over the width of the bridge deck. Model B is identical to Model A except the deck is skewed 37.5°. Model C has one abutment and its backfill similar to Model A while the other abutment and its backfill are similar to Model B. The elevation views of Models A, B, and C are identical as shown in Fig. 7d. The backfills in each case extend longitudinally a distance 1.5 times their depth H. All of these three models have identical abutment and columns which are assumed to be fixed at their bases.

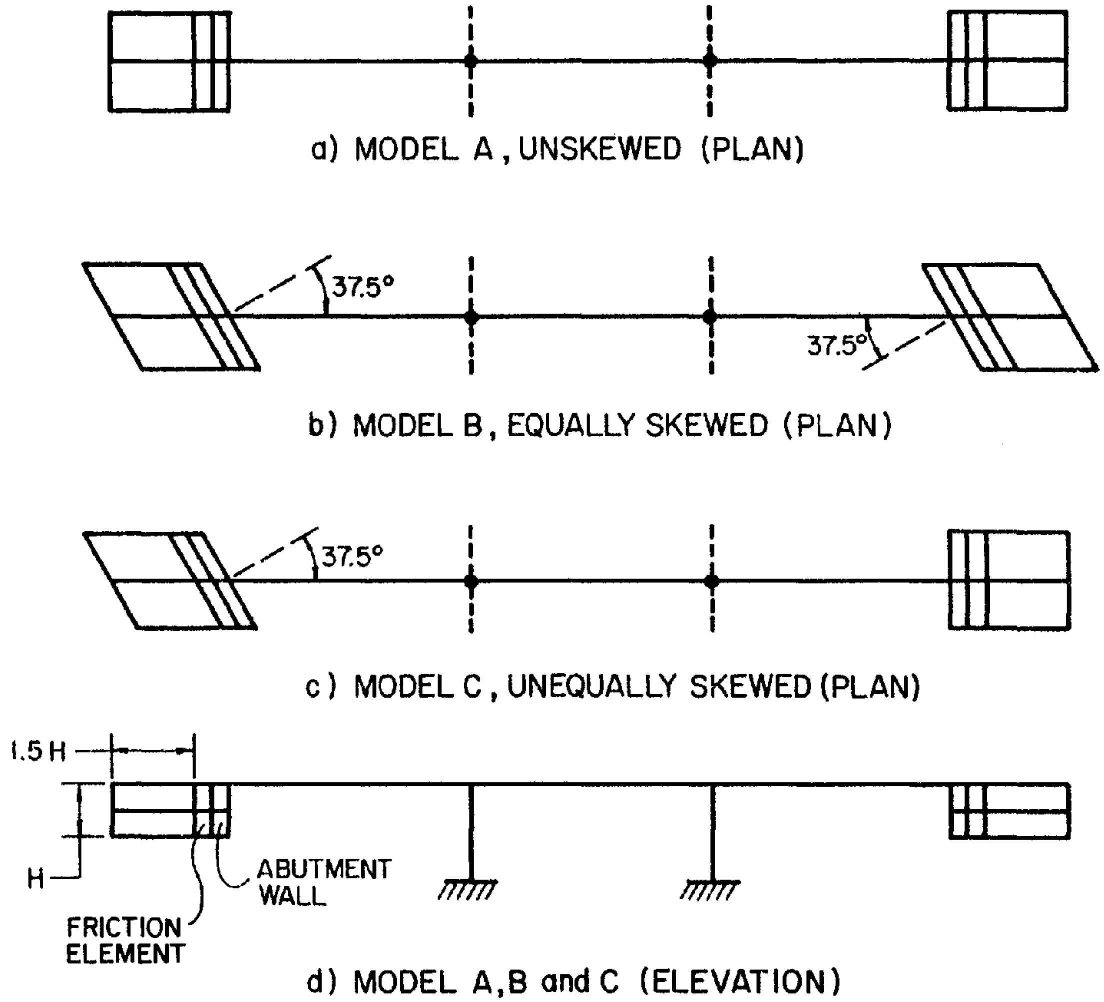


FIG. 7 MATHEMATICAL MODELS WITH VARIOUS SKEWNESS

The longitudinal displacement time-histories for the top of the right bridge column are shown for Models A, B, and C in Figs. 8a, 8b, and 8c, respectively. The dissimilarities in amplitudes and shapes noted in these wave forms are due to differences in amplitudes and phasing of the backfill forces on the two abutments.

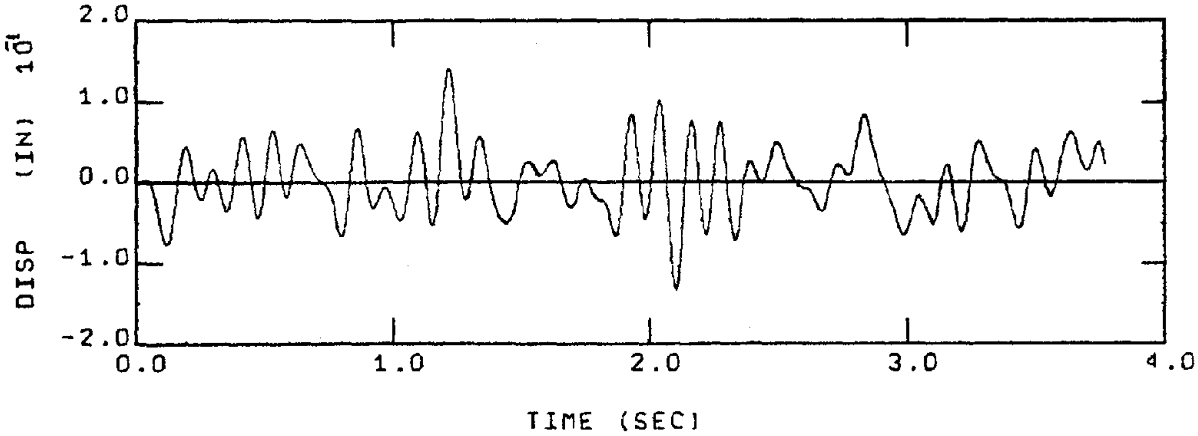
Figures 9a and 9b show the time-histories of the transverse shear component in the left and right columns of Model B. The relatively low values of shear and the similarity in time-histories indicate that the dynamic backfill forces at the two abutments were nearly in-phase resulting in low torsional response of the bridge. Figures 10a and 10b show the time histories of the transverse shear component in the same two columns for Model C. The relatively large values of shear produced and the dissimilarities noted for the two columns in this case indicate that large torsional response developed due to the presence of skew at only one abutment. The backfill forces at the two abutments had large out-of-phase components.

Figures 11, 12, and 13 show time-histories of backfill force on the left and right abutment walls for Models A, B, and C, respectively. It is noted that the dynamic pressures on both walls for Models A and B are nearly in-phase, i.e., when the pressure is positive on one abutment, it is negative on the other, and vice versa. However, for Model C as shown in Fig. 13, these dynamic pressures on the two abutments differ considerably in amplitude and in their phasing. These results again provide evidence that unequal skewness of the two abutments produce large torsional response.

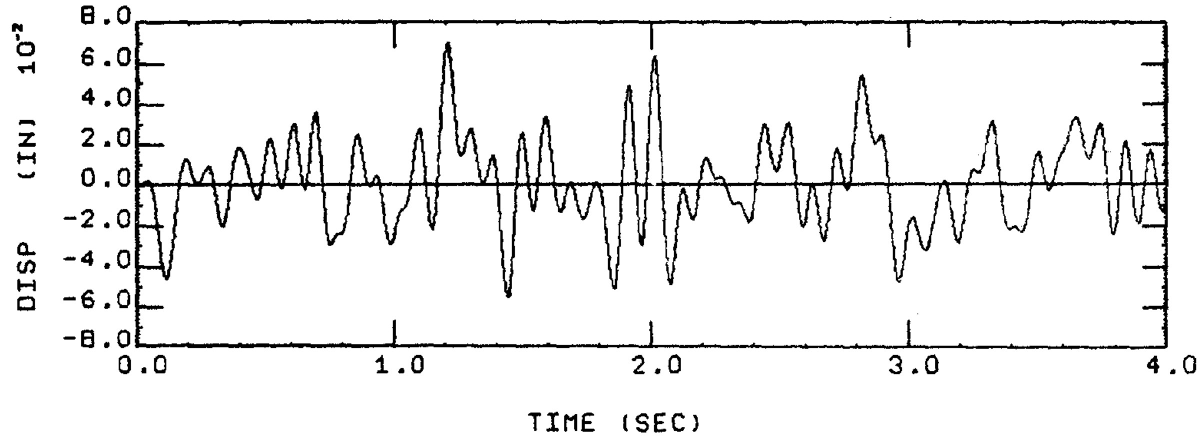
To provide further comparisons of the results for Models A-C, maximum dynamic amplitudes of displacement, shear, and wall force are presented in Table 1. As indicated by the values in rows (1) and (2), the maximum amplitudes of longitudinal displacement and longitudinal shear in the right column are greatly reduced by the presence of skewed abutments. Rows (3) and (4) in this table give maximum values of lateral shear in the left and right columns, respectively. Row (5) gives the ratio of maximum lateral shear to maximum longitudinal shear produced in the right column. The increase in this ratio with skewness indicates the corresponding increase in torsional response which induces a differential shear force between the two columns as shown at the top of Table 1. Half the difference in the shear forces of these two columns is the shear produced by torsional response. The maximum values of these torsional shears are 0.13 and 6.03 kips for Models B and C, respectively, as shown in row (6). Although the magnitude of maximum torsional shear is negligible for Model B, it is large for Model C. The maximum amplitudes of the dynamic wall force are shown in rows (7) through (10). The ratios of maximum positive pressure on the left abutment to maximum negative pressure on the right abutment and maximum negative pressure on the left abutment to maximum positive pressure on the right abutment for both Models A and B are all equal to 1.0 which indicates the two wall pressures are in-phase with each other. Finally, as indicated in row (11), the time history of the resultant of both backfill forces $p(t)$ acts longitudinally along the axis of symmetry in the case of Model A but acts at angle $\theta(t)$ to the longitudinal axis in the case of Model B; causing no torsion in each case. However, in the case of Model C, the resultant force $p(t)$ acting at an angle $\theta(t)$ has an eccentricity about the elastic center of the bridge. This is equivalent to its acting through the elastic center but with a torque $T(t)$ applied as shown in the table.

3. Effect of Foundation Flexibility

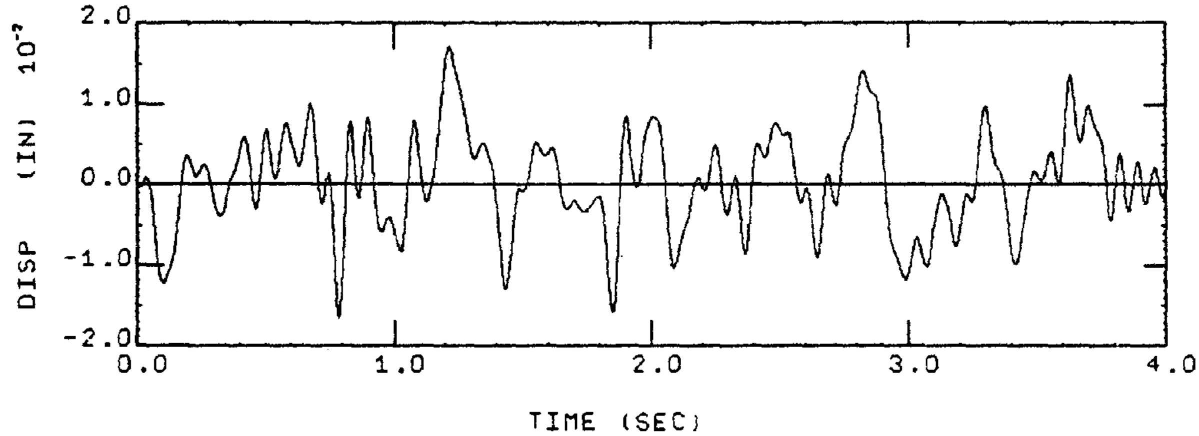
To study the effects of foundation flexibility on dynamic response, models D and E shown in Fig. 14 were analyzed. Model D assumes the backfill behind each abutment extends a distance 7H in the longitudinal direction and a distance 6H beyond the deck in the transverse direction. Each backfill in this case is modelled using 4 equal layers in depth and 3 different widths in the longitudinal direction as shown in Figs. 14a and 14b. Model E is identical to Model D except that the abutments and backfills are of depth 2H and the bases of the columns are provided with linear translational and rotational springs representing foundation flexibility. The backfill soils are modelled with three layers of depth H/3 and one layer of depth H as shown in Fig. 14c.



a) MODEL A, UNSKEWED

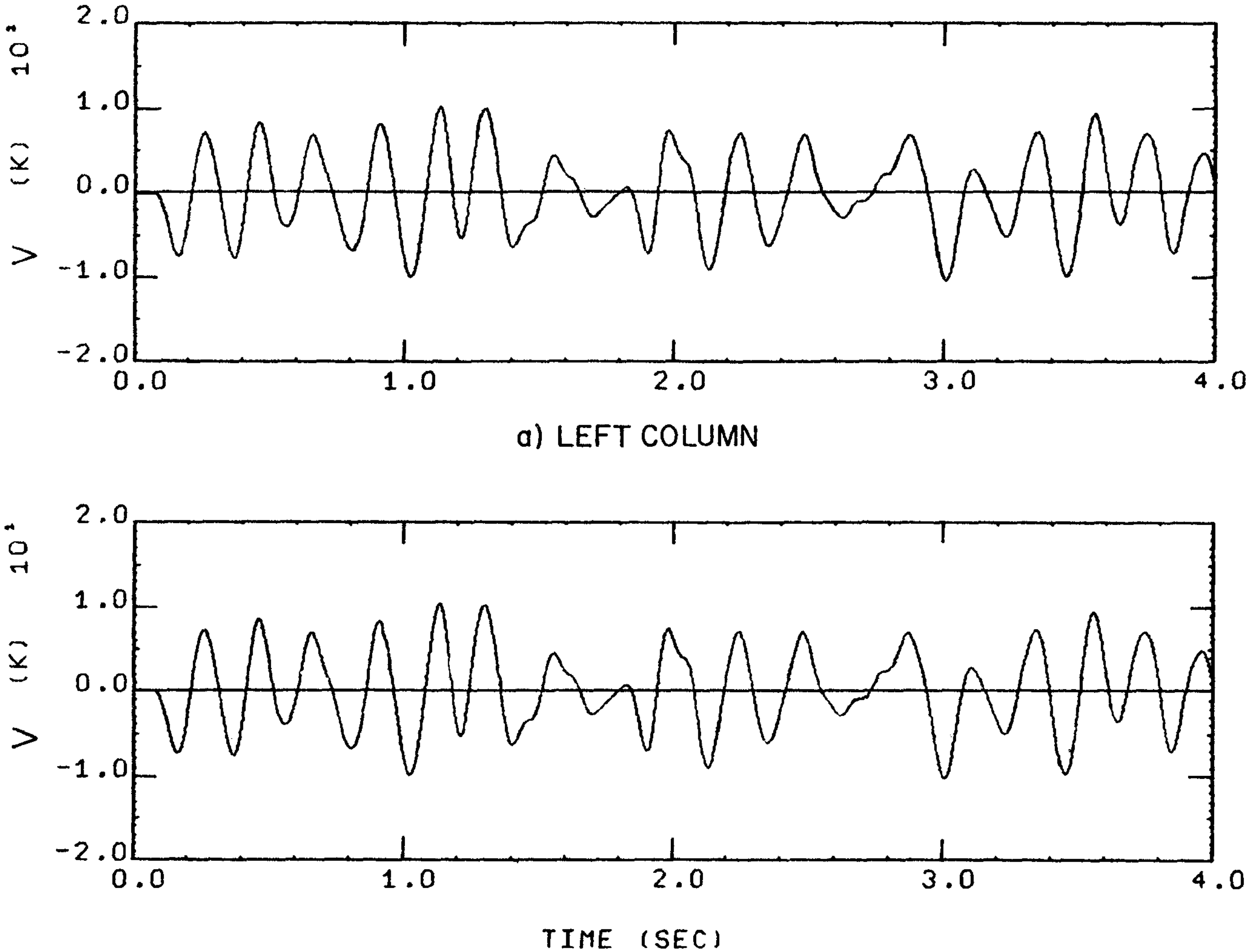


b) MODEL B, EQUALLY SKEWED



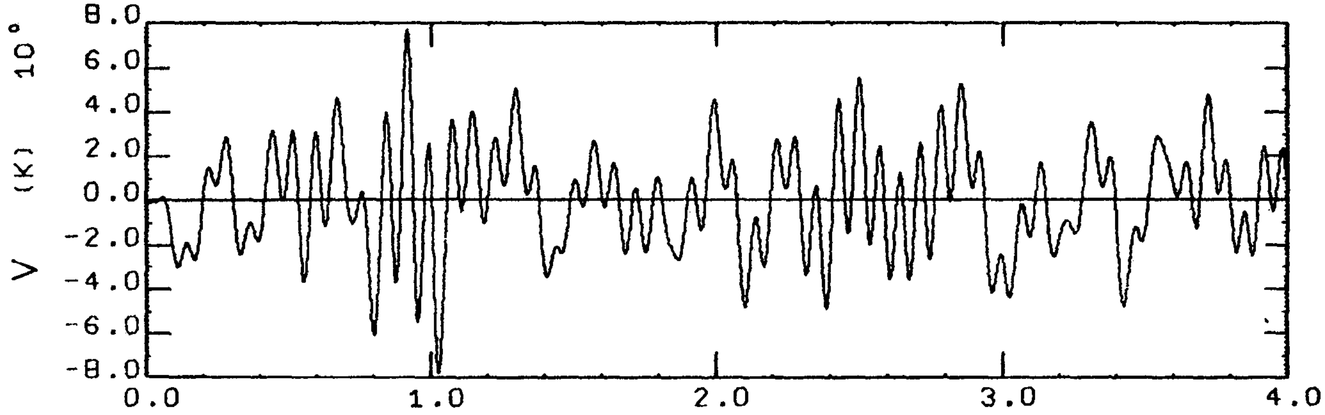
c) MODEL C, UNEQUALLY SKEWED

Fig. 8 Longitudinal displacement at top of right column

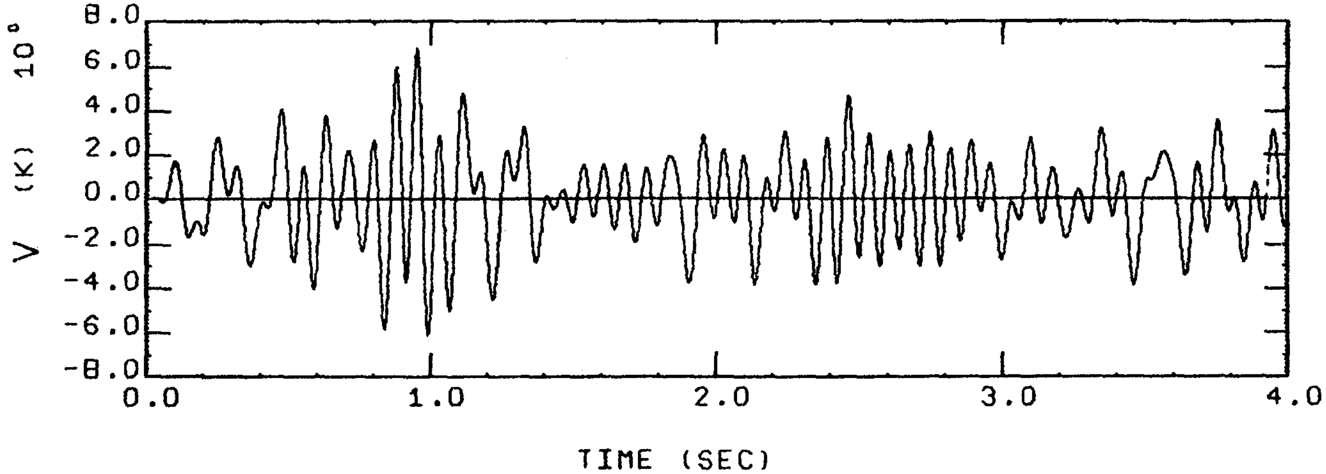


b) RIGHT COLUMN

Fig. 9 Lateral shear - equally skewed - Model B

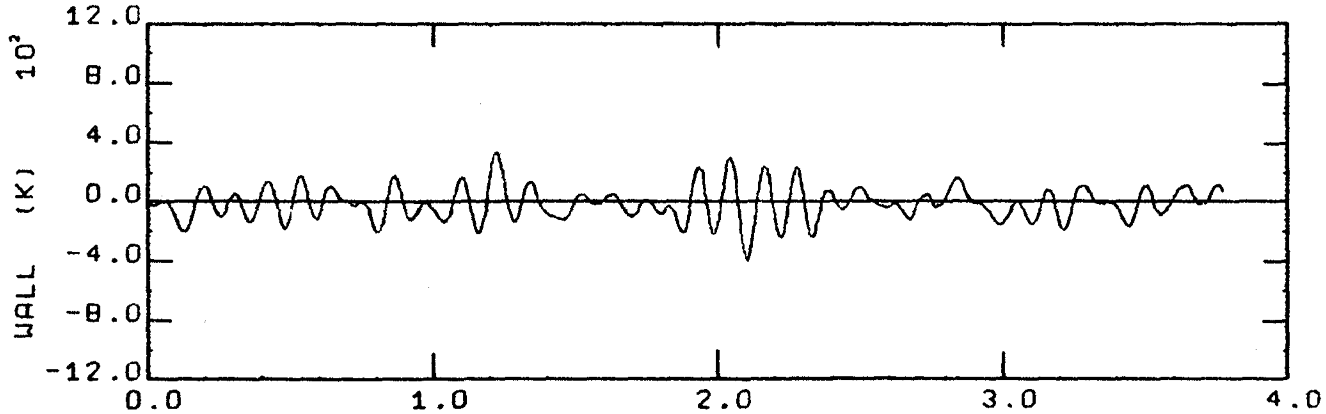


a) LEFT COLUMN

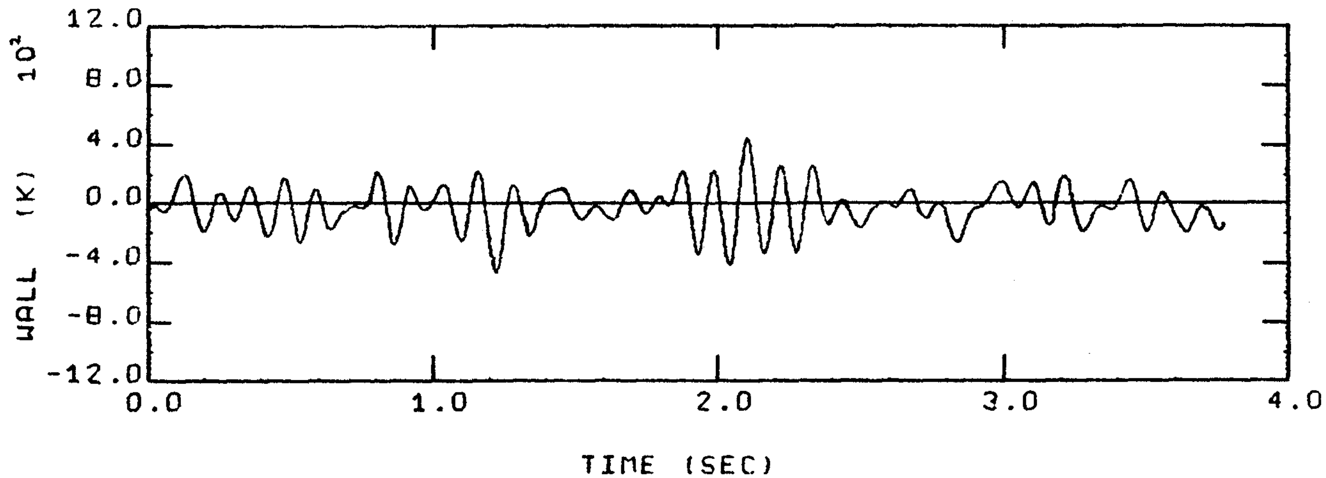


b) RIGHT COLUMN

Fig. 10 Lateral shear - unequally skewed - Model C



a) LEFT WALL



b) RIGHT WALL

Fig. 11 Wall pressure - unskewed - Model A

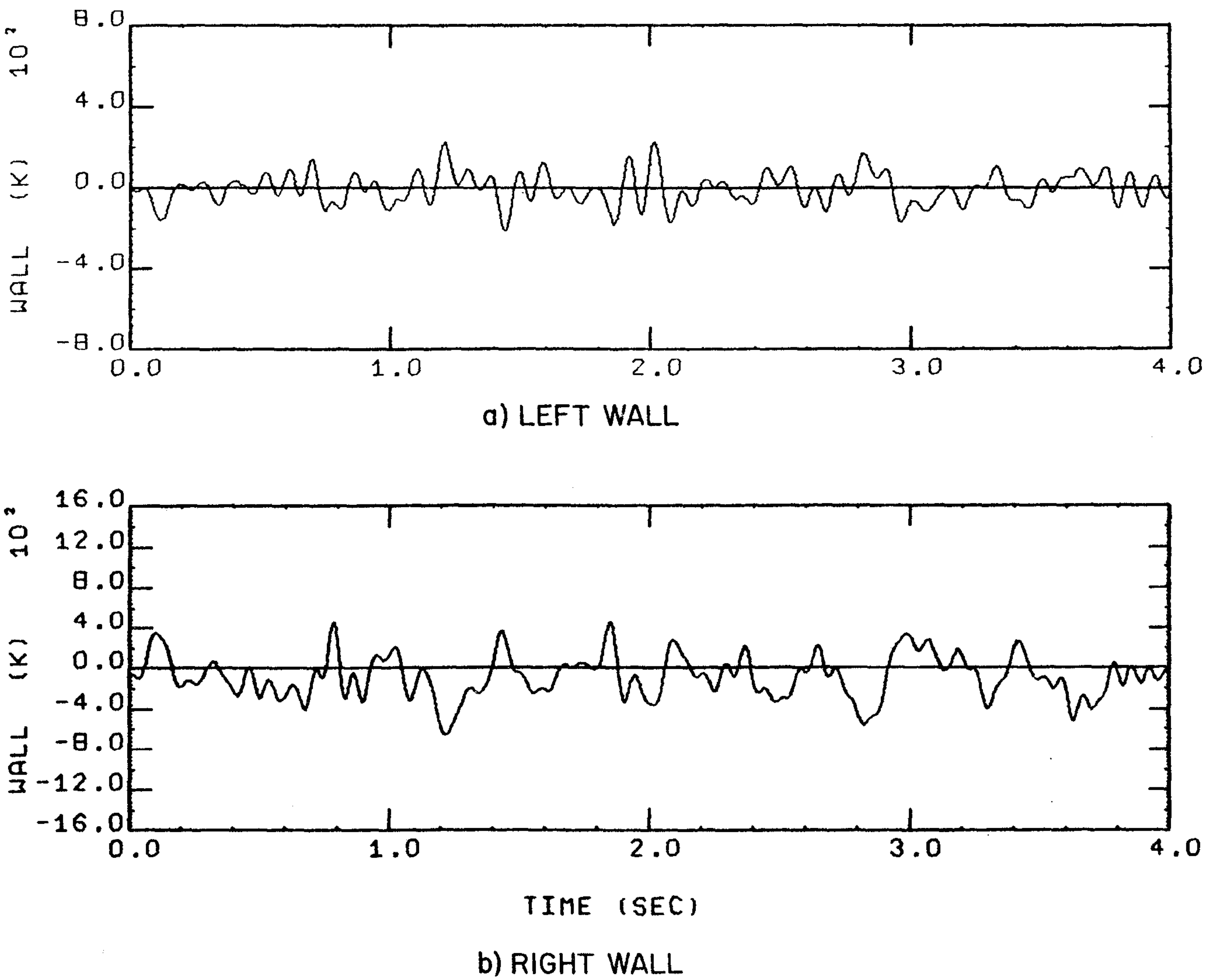
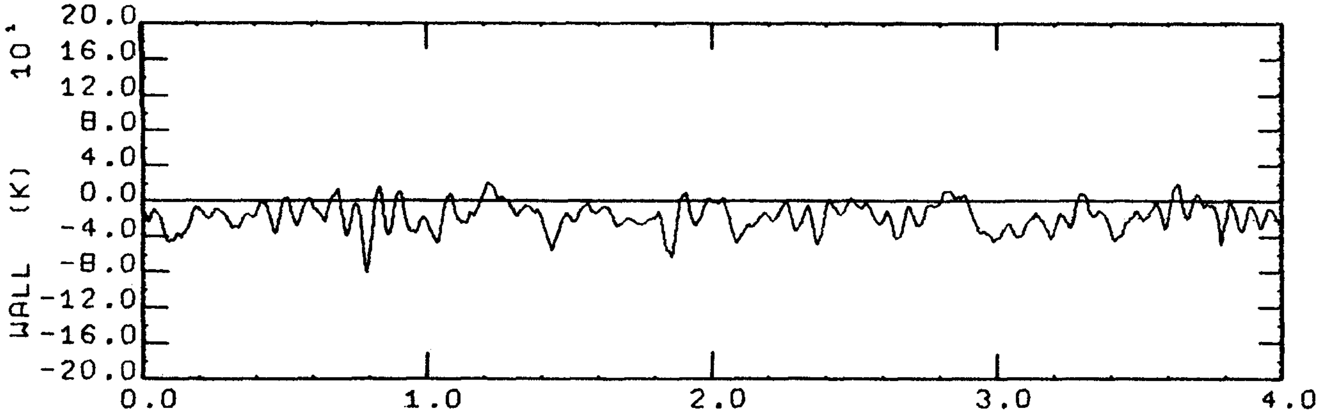
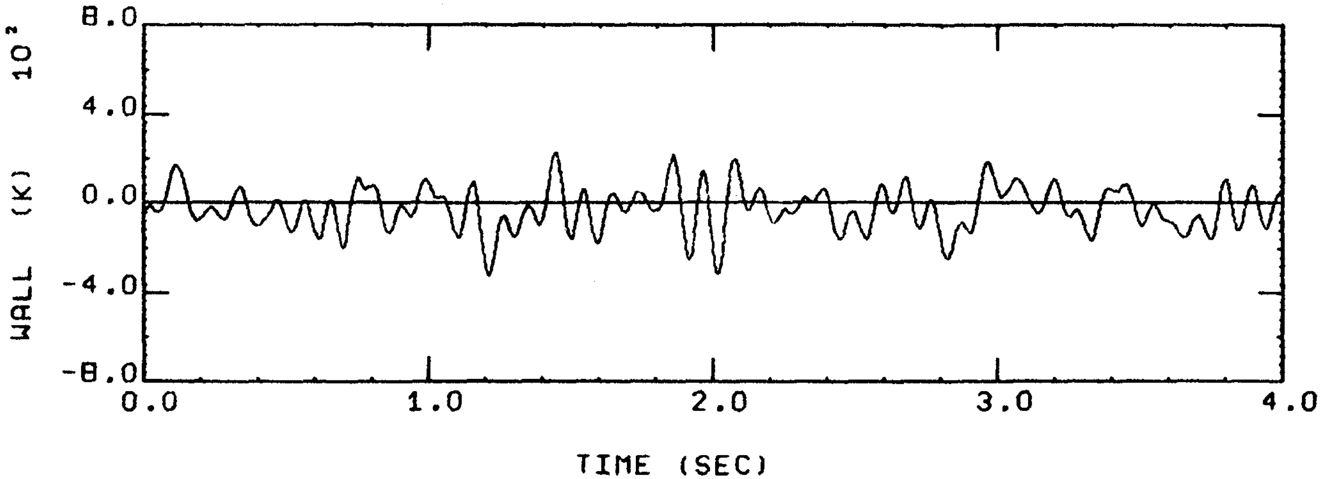


Fig. 12 Wall pressure - equally skewed - Model B



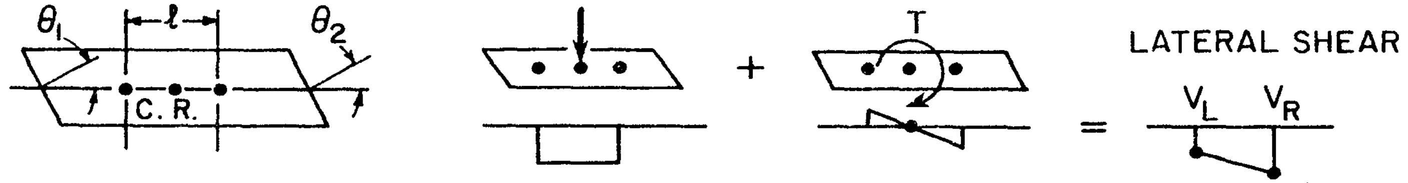
a) LEFT WALL

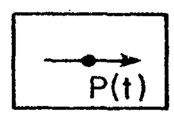
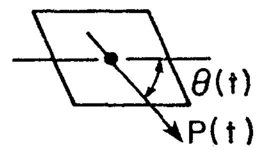
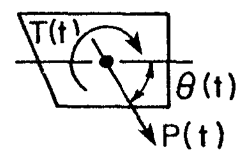


b) RIGHT WALL

Fig. 13 Wall pressure - unequally skewed - Model C

TABLE 1. EFFECTS OF SKEWNESS-MAXIMUM DYNAMIC AMPLITUDE



Model			A $\theta_1 = \theta_2 = 0$	B $\theta_1 = \theta_2 = 37.5°$	C $\theta_1 = 37.5°$ $\theta_2 = 0$
Long. Disp. @ Top of Rt. Col.		(1)	0.14	0.07	0.17
Long. Shear, Rt. Col.		(2)	47.37	23.70	5.64
Lateral Shear	Lt. Col.	(3)	0.	10.47	7.80
	Rt. Col.	(4)	0.	10.49	6.85
Ratio of Shear (4)/(2)		(5)	0.	0.44	1.22
Tors. Shear $T/\ell = \lvert V_L - V_R \rvert /2$		(6)	0.	0.13	6.03
Left Wall Force per ft	Tension	(7)	9.27	6.45	10.20
	Comp.	(8)	9.96	5.30	16.40
Rt. Wall Force per ft	Tension	(9)	9.96	5.30	11.20
	Comp.	(10)	9.27	6.45	12.30
Resulting Model		(11)			

Note: All displacements in inches; All forces in kips.

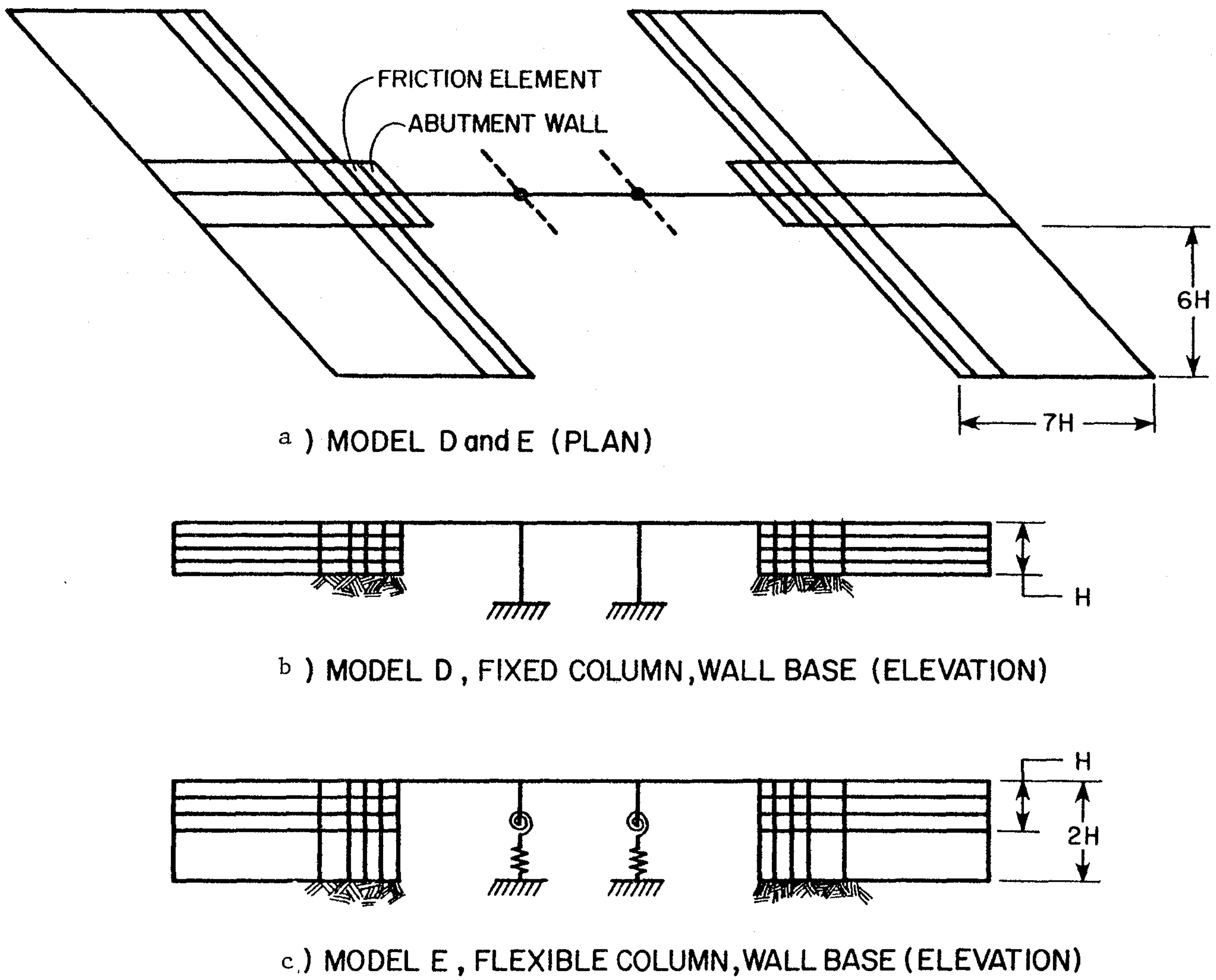


Fig. 14 Mathematical models with different foundation flexibilities

The time-histories of longitudinal displacement at the top of the left column for Models D and E are shown in Figs. 15a and 15b, respectively. Noting the different displacement scales used, these two wave forms differ considerably in form and in their peak amplitudes.

To provide further comparative data, the maximum dynamic amplitudes of displacement and acceleration of the bridge deck, column shear forces, and forces on the abutment walls are listed in Table 2. Based on the ratios of corresponding responses for Models E and D, given in rows (1) through (4) of the last column of this table, it is quite clear that the overall response of Model E having foundation flexibility is considerably greater than that for Model D. All of these ratios simply indicate that Model E has less constraint provided by its backfills than does Model D; thus, the bridge structural response is higher for Model E.

Rows (9) and (10) in Table 2 show ratios of maximum column shears to maximum total backfill force on one abutment wall. Comparing the magnitudes of these ratios confirms the above statement explaining the reason for higher overall structural response in the case of Model E.

4. Effect of Impact Between Abutment and Backfill

To study the effects of impact and separation on seismic response, results obtained by linear and nonlinear analyses for Models A, B, and E in Figs. 7 and 14 are compared. The nonlinear modelling differs from the linear modelling by allowing separation and impact between abutments and their corresponding backfills.

The most distinctive difference between the results obtained by linear and nonlinear analyses is the high acceleration produced at the point of impact in the nonlinear case. A typical acceleration time-history response for Model A is shown in Fig. 16. The high peaks of acceleration in this wave form are produced at moments of impact. While these acceleration peaks are high near the point of impact, the influence is very localized, i.e., the amplitudes of the peaks produced by impact decay rapidly with distance from the point of impact. Acceleration time-histories at the top of the left column as produced without and with impacts are shown in Figs. 17a and 17b for Model A. While the general features of the two wave forms are essentially the same, localized differences in the form of high frequency noise caused by impact are noted. This feature is better observed in Fig. 18 which shows an expanded-scale view of the first second of time-history shown in Fig. 17b.

5. Effects of Separation Between Abutment and Backfill

A characteristic feature of allowing separation between wall and backfill soil to occur is that only positive pressure is permitted at the interface. Therefore, the backfill soils at the interfaces of both end abutments can have phase differences in their responses. Figures 19a and 19b show the time-histories of soil force on the left and right abutment walls, respectively, as determined by the nonlinear analysis for skewed Model B. Clearly there are

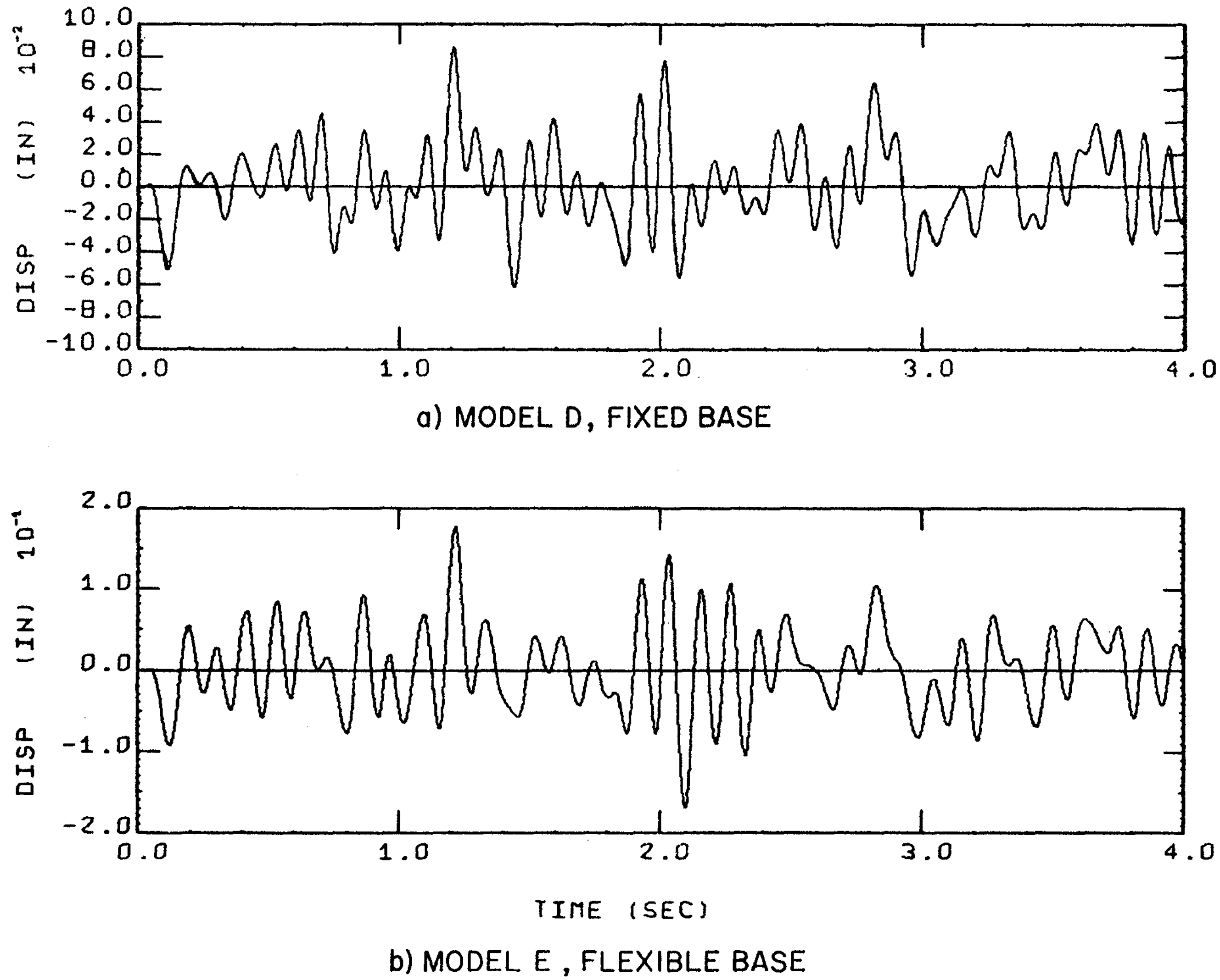
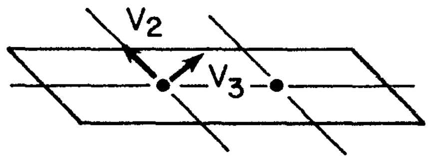
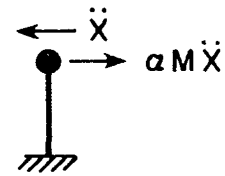


Fig. 15 Longitudinal displacement at top of left column - Model D and E

TABLE 2. EFFECTS OF FLEXIBILITY AT BASE

(a) 

(b) 

Maximum Response Values			Model D Fixed Base	Model E Flex. Base	Ratio E/D
Long. Disp. @ Top of Lt. Col.		(1)	0.087	0.178	2.05
Long. Accel. (g)		(2)	0.98	1.36	1.39
Shear @ Lt. Col.	V_2	(3)	31.35	55.70	1.78
	V_3	(4)	42.59	70.60	1.66
Lt. Wall Force per ft	Tension	(5)	11.06	9.98	0.91
	Comp.	(6)	13.31	8.95	0.67
Rt. Wall Force per ft	Tension	(7)	13.31	8.95	0.67
	Comp.	(8)	11.06	9.98	0.91
V_2/Wall Force,	$\frac{(3)}{(5)+(6)}$	(9)	1.29	2.95	2.29
V_3/Wall Force,	$\frac{(4)}{(5)+(6)}$	(10)	1.75	3.74	2.14

Note: All displacements in inches; All forces in kips.

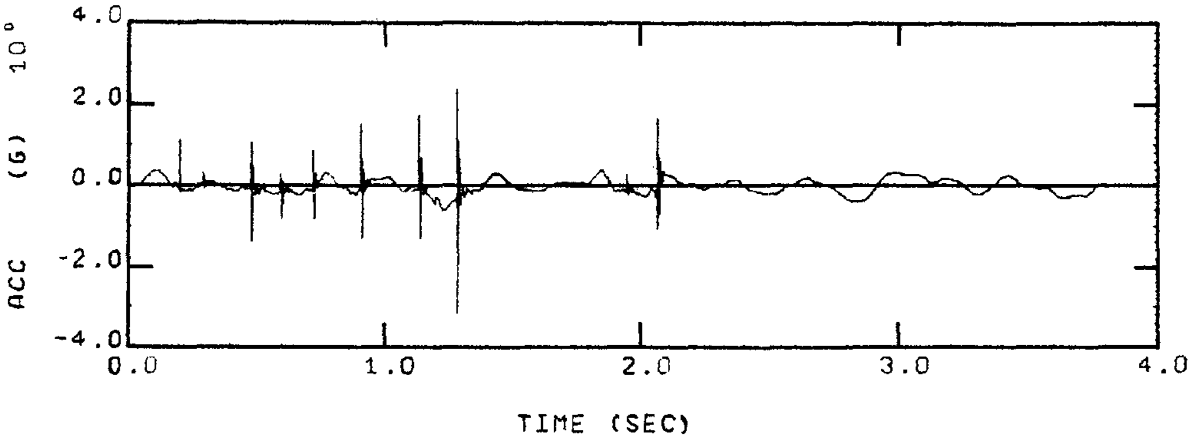
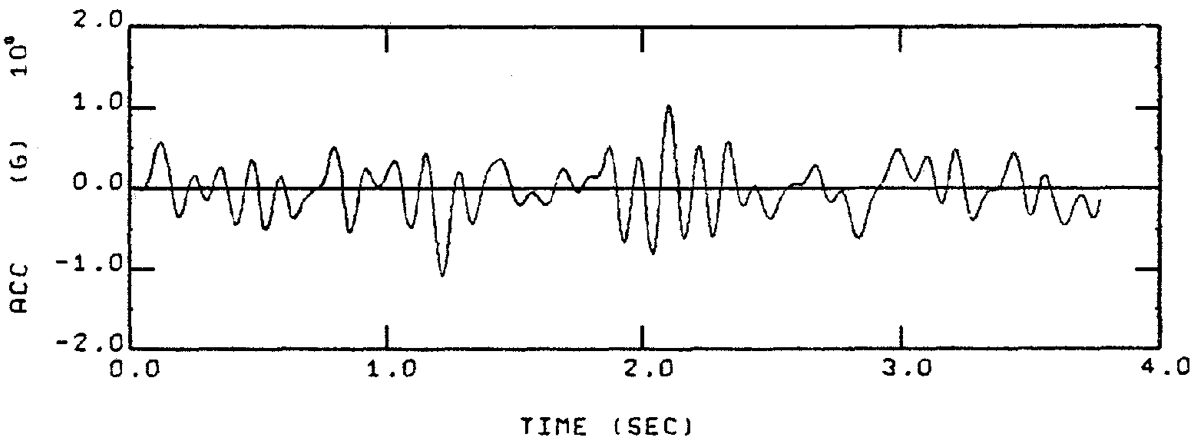
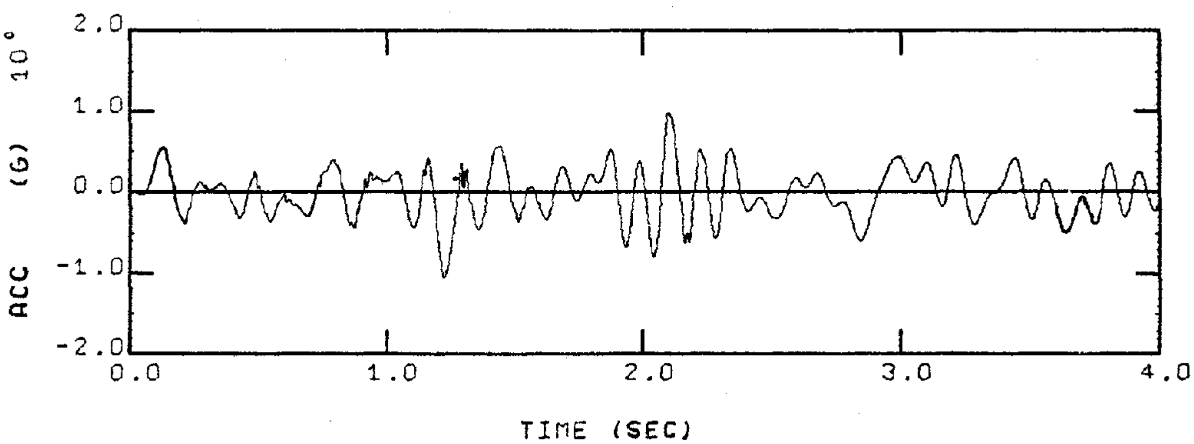


Fig. 16 Acceleration time history at contact point with impact - Model A



a) WITHOUT IMPACT EFFECT



b) WITH IMPACT EFFECT

Fig. 17 Comparison of time histories at top of left column without and with impact - Model A

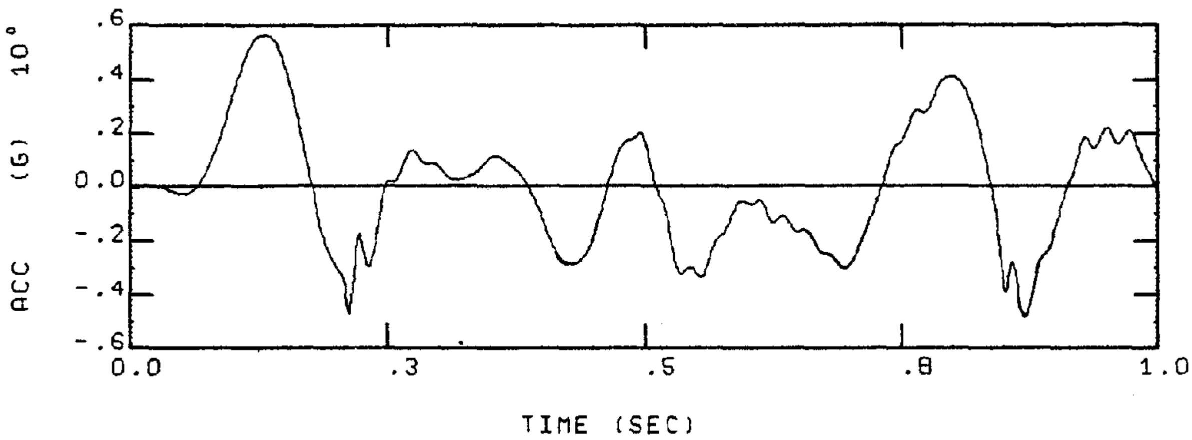
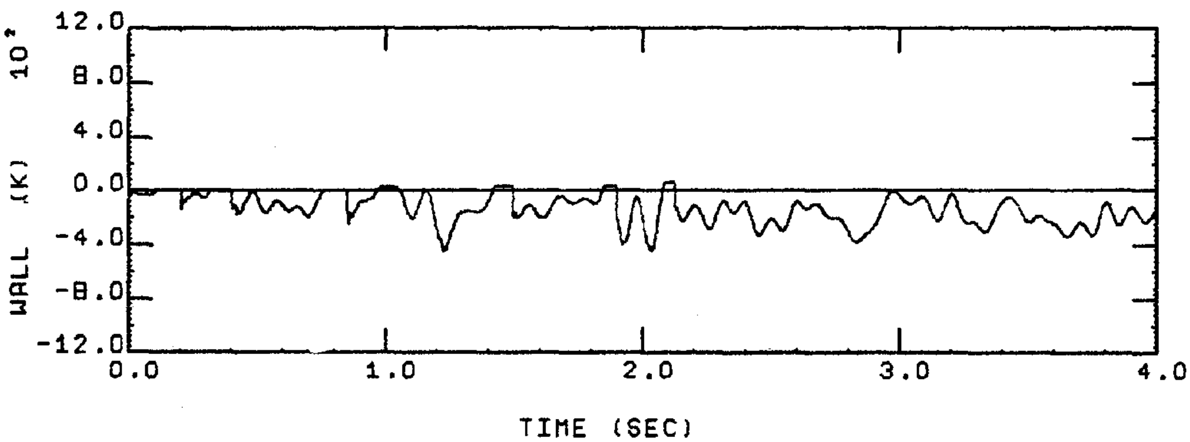
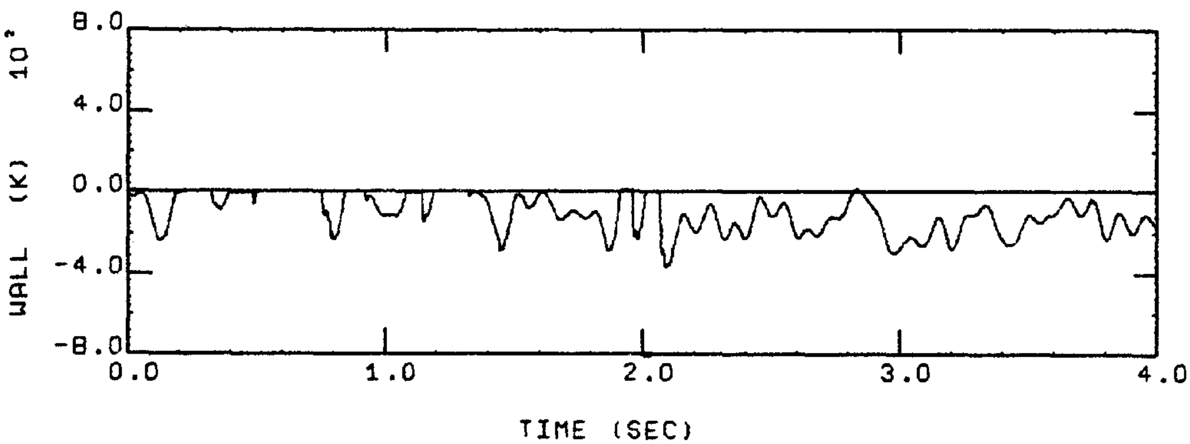


Fig. 18 Expanded scale view of effect of impact on acceleration - Model A



a) LEFT WALL



b) RIGHT WALL

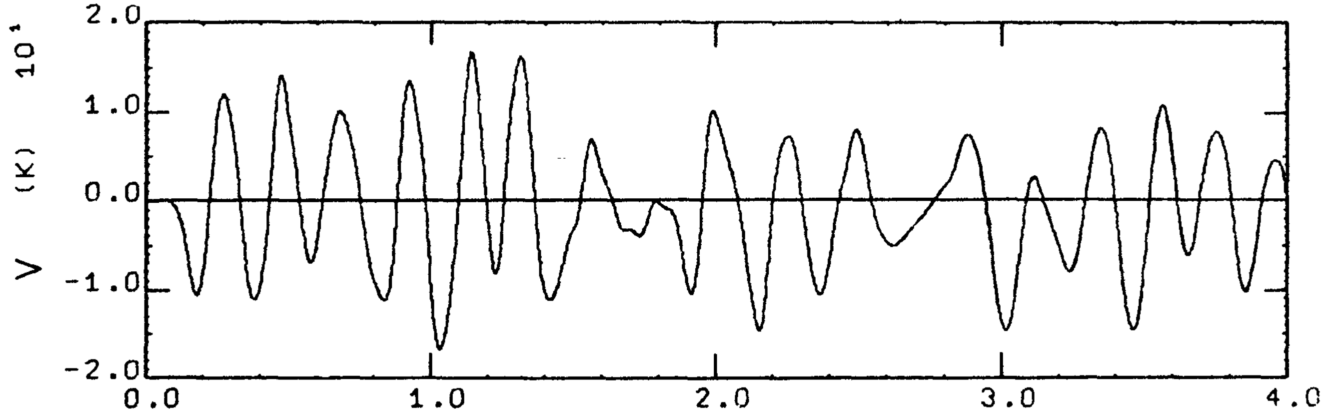
Fig. 19 Non-linear response of wall pressure - Model B significant out-of-phase components of response between the two abutments. Note that a small overshoot error is present during certain moments of the time-history. This overshoot error can be controlled by reducing the integration time-step and by introducing a variable time-step procedure. Both of these procedures have been incorporated into the computer program.

The out-of-phase components of soil force on the end abutments produces a torsional response of the bridge structure. This effect is quite apparent when observing the unequal lateral shears produced in the two columns. This comparison can be made in Fig. 20 which shows the transverse shear time-histories for the two columns of Model B. While the frequency content of the two wave forms in this figure are similar, significant differences are present in the amplitudes. The maximum transverse shear produced in the left column is 16.81 kips while the maximum transverse shear in the right column is 18.95 kips. The maximum difference in the entire time history of the two shears is 3.72 kips.

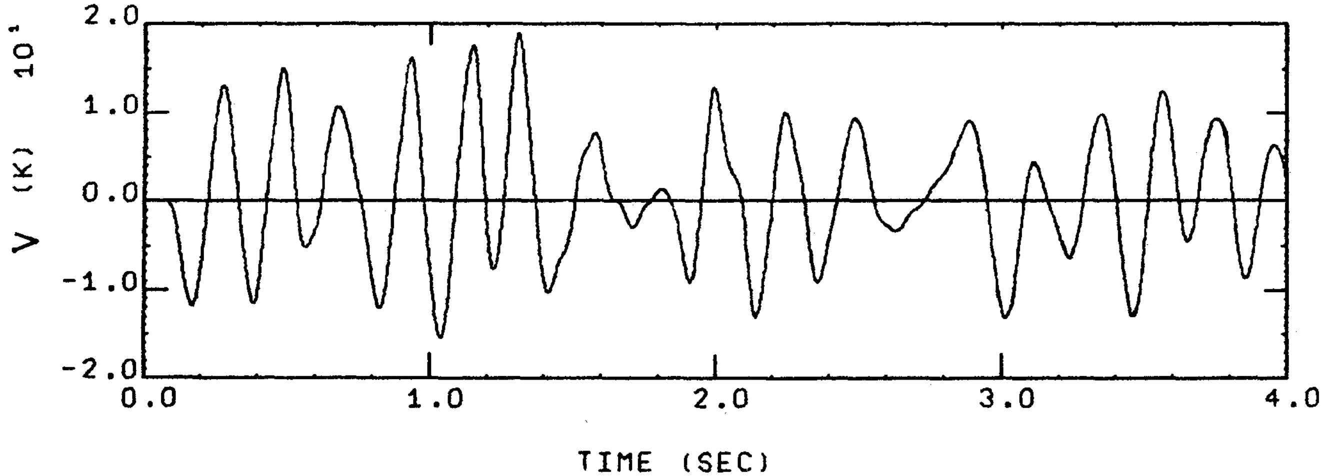
6. Maximum Dynamic Response of Linear vs. Nonlinear Model

For further comparison, maximum amplitudes of response obtained by linear and nonlinear analyses for Models A, B and E are shown in Table 3. The particular responses presented are longitudinal displacement and acceleration at the top of the left column and the shears in both principal directions of the left column. In Models A and B, principal shears V_2 and V_3 are the lateral and longitudinal shears, respectively, as the column is oriented with one principal axis coinciding with the longitudinal axis of the bridge. In Model E, the column is placed so that one principal axis is oriented 52.5° from the longitudinal bridge axis.

The maximum amplitudes of dynamic response are listed for both linear and nonlinear response and for comparison purposes the ratios of linear to nonlinear response amplitudes are shown. From the results shown in Row 1 of Table 3, it is quite apparent that the displacements of Models A and B produced by nonlinear response are larger than the corresponding displacements produced by linear response. However, the reverse comparison is seen for Model E. From the results in Row 2 it is seen that the accelerations produced by the linear response are larger than the corresponding accelerations produced by nonlinear response. The differences in the amplitudes for both types of response are relatively small, however. Row 3 shows a large difference in the transverse lateral shears produced in Model B. This large difference results from the torsional response produced in the nonlinear case. Row 4 shows only small differences in the longitudinal shears produced by the two types of response.



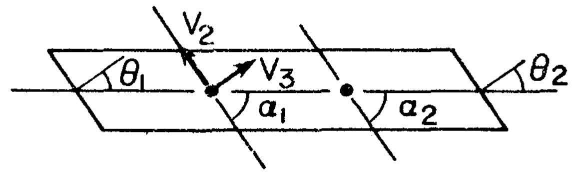
a) LEFT COLUMN



b) RIGHT COLUMN

Fig. 20 Lateral column shears - Model B (non-linear)

TABLE 3. LINEAR Vs. NONLINEAR - MAXIMUM DYNAMIC RESPONSES



Model	A $\theta_1 = 0°$ $\theta_2 = 0°$ $\alpha_1 = 90.0$ $\alpha_2 = 90.0$			B $\theta_1 = 37.5°$ $\theta_2 = 37.5°$ $\alpha_1 = 90.0$ $\alpha_2 = 90.0$			E $\theta_1 = 37.5°$ $\theta_2 = 37.5°$ $\alpha_1 = 52.5$ $\alpha_2 = 52.5$		
Type of Analysis / Responses	Linear	Non-linear	Ratio Linear/Non-linear	Linear	Non-linear	Ratio Linear/Non-linear	Linear	Non-linear	Ratio Linear/Non-linear
Long. Disp. (1)	0.14	0.15	0.93	0.071	0.088	0.81	0.18	0.17	1.06
Long. Accel. (2)	1.09	1.05	1.04	0.80	0.75	1.07	1.36	1.31	1.04
Shear V_2 (3)				10.47	16.81	0.62	57.27	56.89	1.01
Shear V_3 (4)	47.6	51.5	0.91	29.96	28.63	1.04	70.60	68.01	1.01

Note: All displacements in inches; All accelerations in g's; All shears in kips.

7. Seismic Load Transfer to Column and Abutments

It is of particular importance to know the division of the total longitudinal seismic deck force between the supporting columns and the abutments. To check this behavior characteristic, consider the unskewed Model A which experienced a maximum longitudinal deck acceleration of 1.09g as shown in Table 3. The tributary bridge weight for each column (center to center of spans of deck plus one-half of columns) in this case is 340 kips; thus, the estimated maximum column shear based on this tributary weight is 371 kips (340 x 1.09 = 371). Since the maximum calculated column shear as shown in Table 3 is only 47.6 kips, it is clear that most of the tributary seismic deck force (87%) is transferred to the foundation through the abutment walls. To further check this transfer characteristic, let us consider the total deck seismic force plus the seismic forced produced in the upper-half portions of both columns. The maximum combined seismic force in this case amounts to 1078 kips (989 x 1.09 = 1078) which occurs at about 2.1 seconds. The algebraic sum of the two abutment wall forces at this same instant of time is 855 kips (404 + 451 = 855; see Figs. 11a and 11b). Considering the bridge as a whole, this information indicates that about 79% of the maximum seismic force in the total deck is transferred to the foundation through the interaction of abutment walls with the backfills. Further, calculations show the maximum combined longitudinal shear in the two columns which occurs at the critical time of 2.1 seconds is approximately 94 kips. Therefore, about 9% (94/1078) of the maximum seismic force is transferred to the foundation through the columns. The remaining 12% of the maximum seismic force is transferred to the foundation by shear in the abutment walls. Making comparisons as shown above for the other bridge models gives similar results. Also an investigation of short bridges damaged during a particular earthquake indicates that abutment walls resist most of the total seismic force acting on such bridges [18].

It is also clear from the above analysis that the inertia force does mainly come from the bridge itself instead of the soil. If the bridge-soil system is simplified using the mass of bridge deck Mb and the mass of soil Ms, as shown in Fig. 21, it is proper to conclude that the mass of soil can be neglected in evaluating the total dynamic force of the bridge-soil system. The reason is not because of the lack of inertia forces participating from the soil, but due to the fact that the inertia forces of bridge deck and soil are usually out of phase with each other.

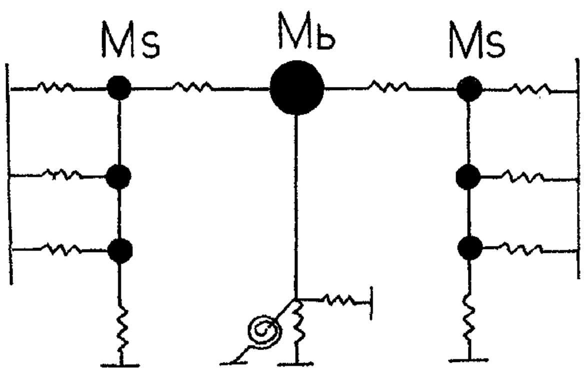


Fig. 21 Lumped mass model of bridge-soil system

8. Comparison of Resultant Backfill Force on Abutment Obtained by Analysis and the Mononobe-Okabe Method [14, 15, 16]

One commonly used formula in calculating the resultant dynamic lateral force on the abutment wall is the Mononobe-Okabe formula [14, 15, 16]. This formula has the following form:

$$E_{AE} = \frac{1}{2}\gamma H^2 (1 - k_v)\cdot K_{AE}$$

where
$$K_{AE} = \frac{\cos^2(\Phi - \theta - \beta)}{\cos\theta \cos^2\beta\cos(\delta+\beta+\theta)\left[1 + \sqrt{\frac{\sin(\Phi+\delta)\sin(\Phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^2}$$

E_{AE} = the total active force

$\theta = \tan^{-1}\frac{k_h}{1 - k_v}$

γ = unit weight of soil

H = height of wall

Φ = angle of friction of soil

δ = angle of friction of wall and soil

i = slope of ground surface behind wall

β = slope of the wall to the vertical

k_h = horizontal acceleration/g

k_v = vertical acceleration/g

Using $\Phi = 30°$; $\theta = \tan^{-1}\frac{0.5}{1-0} = 26.6°$; $\beta = 0°$; $\delta = \frac{\Phi}{2} = 15°$; $i = 0°$;

$\gamma = 110\ \text{lb/ft}^3$, $H = 13.5'$

The active static earth pressure

$$Pa = \frac{1}{2}\gamma H^2 \times \frac{1-\text{sn}\Phi}{1+\text{sn}\Phi} = 10. \times 0.33 = 3.3^k/\text{ft.}$$

$K_{AE} = 1.17$

Total active pressure

$E_{AE} = 10 \times 11.7 = 11.7^k/\text{ft.}$

$E_{AC}/Pa = 11.7/3.3 = 3.5$

The total seismic active pressure from M-O method is 3.5 times the active static pressure.

The total seismic pressure (dynamic + static) for various linear models from Table 2 and 3 ranging from 12.6 kip (9.3 + 3.3 = 12.6, line 10, Table 2) to 16.6 kip (13.3 + 3.3 = 16.6, line 6, Table 3) is about 3.8 to 5.0 times the static pressure.

The analytical results are 8% to 42% higher than the M-O method. It should be noted that there are several different assumptions between the M-O method and the analysis method:

(1) The M-O method assumes a gravity wall with no deformation of the wall allowed, while the analytical method assumes a flexible wall.

(2) The inertia of wall is neglected in the M-O method, while the analytical method considers it.

(3) The dynamic response of the bridge structure is neglected in the M-O method.

9. Analytical Results Compared with Design Values

It is of interest to compare the analytical results obtained with corresponding design values specified by the CALTRANS and other design method [13]. The acceleration response spectrum curve (5% damping) for the input earthquake motion has a peak spectral value of 1.6g at periods of about 0.3 and 0.4 seconds as shown in Fig. 22. The closest CALTRANS A·R·S spectrum is the one with 0.5g peak rock acceleration and 1.6g peak surface spectral acceleration at period of 0.3 second as shown in Fig. 22.

where A = Maximum expected acceleration at bedrock of the site
R = Normalized rock response
S = Soil amplification spectral ratio.

Depending on the facilities and experiences that the designer may have, there are various design approaches as summarized in Table 4. The first approach is as specified by the CALTRANS design code. The estimation of natural period of the bridge and the distribution of resulting earthquake forces to individual members include the stiffness of the superstructure, supporting piers and restraint of the abutments. Due to the lack of a unified method in evaluating the restraint of the abutment backfill, the analytical results of Model D of Fig. 14 are used and they are simplified as shown in column (1).

Comparing the results shown in Table 4, shows that the abutment force obtained by design approach No. 1 is 25 percent higher than that obtained by the finite element method; however, it shows that the column shear obtained by design approach No. 1 is 50 percent lower than that value obtained by the finite element method.

The second design approach which is a simple and common one is to calculate the natural period and seismic force of the structure including the

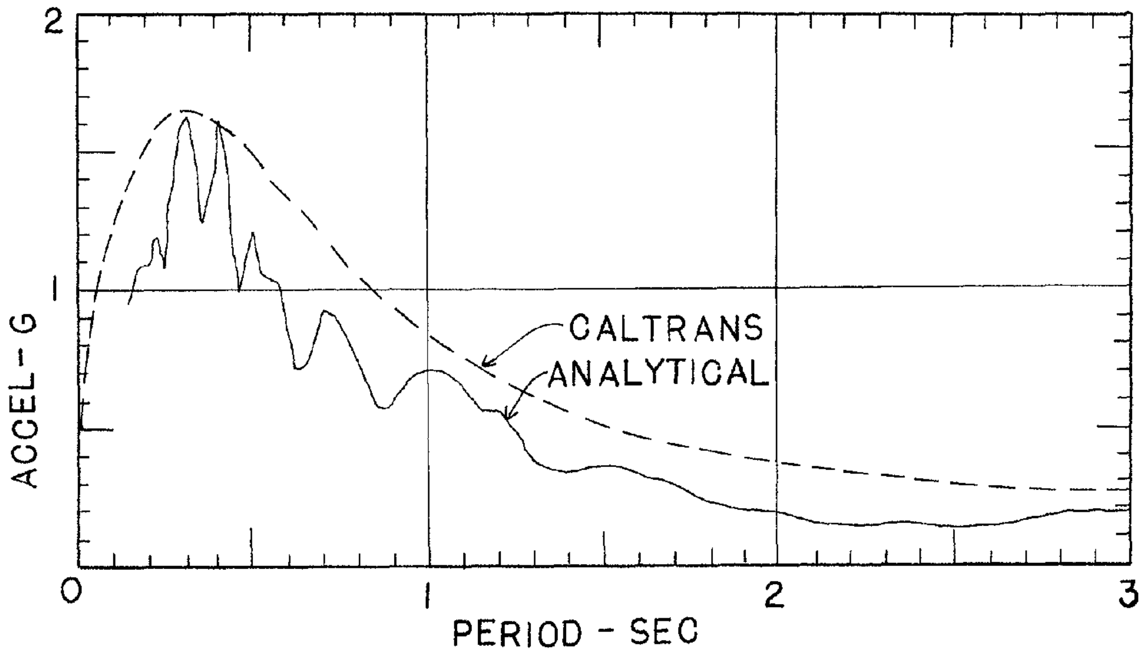
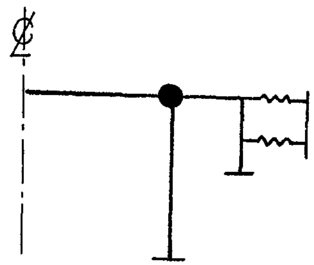
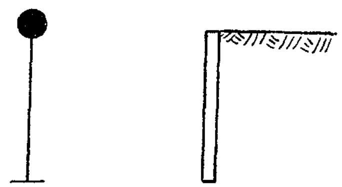
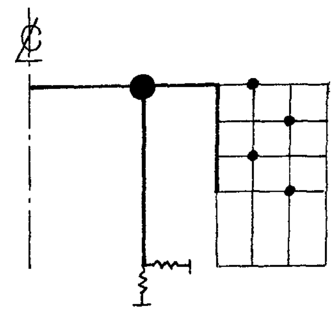


FIG. 22 COMPARISON OF RESPONSE SPECTRA USED IN ANALYTICAL METHOD AND CALTRANS

weight of one span and half the pier and neglecting the influence of abutment fill and foundation flexibility as shown in column (2) of Table 4. In calculating the abutment force, the Mononobe-Okabe method is used which only considers the inertia force from backfill while neglecting the weight of the structure.

In comparing the results obtained by the second design approach to the finite element results, it can be seen that the abutment forces obtained by both methods are close; however, the column shear is 5 times higher when using the second design approach.

TABLE 4. EARTHQUAKE FORCE ON BRIDGE

Approaches	(1) Design Approach No. 1	(2) Design Approach No. 2	(3) Finite Element Method
Model			
1st Natural Period, Sec.	0.11	0.28	0.13
Elastic Column Shear	52.8*(a)	544	88.7
Design Column Shear*(b)	8.8/50%	90.6/508%	17.8/100%
Abutment Force/ft.	13.3 + 3.3*(c) = 16.6/125%	11.7/88%	10 + 3.3*(c) = 13.3/100%

Note: (a) All forces in Kips

(b) The design shear is calculated by dividing the elastic column shear with adjustment factor Z=6 for ductility and risk assessment

(c) The total force at abutment = dynamic force + static active earth pressure

CONCLUSIONS

Based on the studies contained herein for short bridges, conclusions may be deduced as follows:

(1) The total seismic load of the bridge deck is transmitted to the foundation primarily through the abutments with the columns carrying only a small percentage.

(2) Backfill and soil forces on the two end abutments remain essentially in-phase under linear conditions but can develop significant out-of-phase components under nonlinear conditions.

(3) Skewness of a bridge tends to reduce maximum longitudinal response but it causes coupled lateral response to develop.

(4) Unequally skewed end abutments can cause both lateral and large torsional responses to develop.

(5) Foundation flexibilities at the bases of columns and abutments have significant influence on overall bridge response.

(6) Impacts at the interfaces of abutment walls and the backfill soils cause very large local transient accelerations but they have little effect on the average deck acceleration.

(7) Separations which occur between abutments and backfill soils cause significant out-of-phase components to develop in the backfill forces.

(8) The position of dynamic resultant force on the abutment wall tends to be much higher than the static resultant of equivalent hydraulic pressure, i.e., it is usually located at about mid-height rather than at the one-third point from the bottom. The total seismic pressure on abutment wall ranges from 3.8 to 5 times of static active earth pressure.

(9) Evaluating the seismic shear forces in columns without considering the restraints of abutments tends to overestimate these forces.

(10) Total bridge seismic force can be estimated with reasonable accuracy without considering the inertia forces of the backfills.

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EVALUATION OF ANALYTICAL PROCEDURES USED IN BRIDGE SEISMIC DESIGN PRACTICE

by

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ABSTRACT

Presented are the descriptions and findings of case studies on several bridges using currently available methods of dynamic analysis. The equivalent static force methods are described and evaluated using elastic dynamic analysis techniques. The linear response spectrum and time history analysis techniques are evaluated using a recently developed nonlinear dynamic analysis program for bridges. Conclusions and recommendations based on the results of the studies are presented in terms of current design practices and code provisions.

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INTRODUCTION

The accurate prediction of stresses and displacements induced in the various components of a structure during a strong motion earthquake is the key to improved earthquake resistant design. Predicting these stresses and displacements in bridge structures may be divided into the following two general tasks:

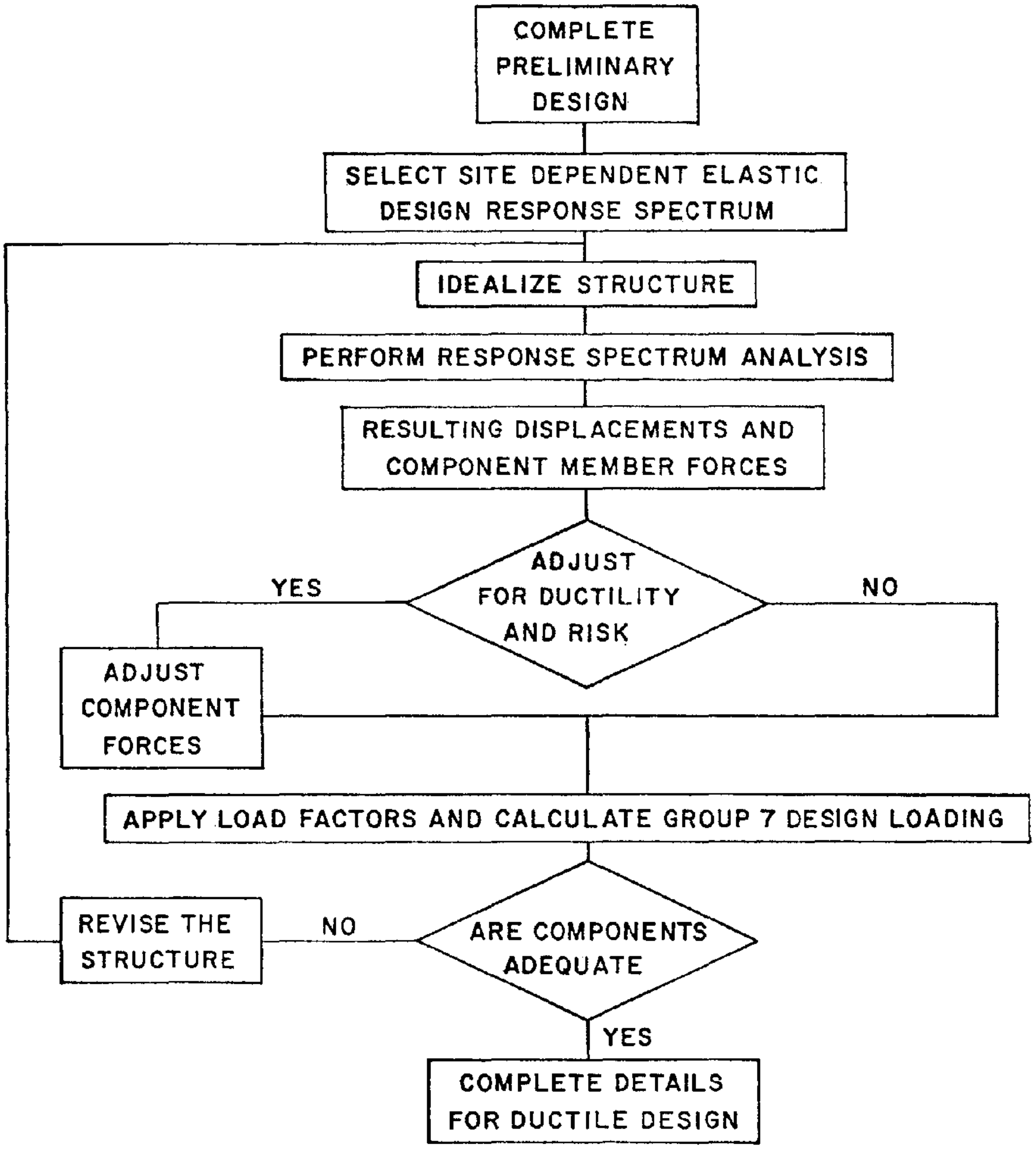
(1) Determination of the seismic load.
(2) Determination of the effect of this load on the structure.

These two tasks are typically reflected in current seismic design processes such as the one used at the Office of Structures, California Department of Transportation (CALTRANS). This process is depicted in Figure 1. The seismic load to which a structure will be subjected is determined by selecting the appropriate site dependent design response spectrum. The effect of this loading on the bridge structure is then determined by predicting the elastic response of the structure by any one of several methods, and reducing the elastically determined forces to account for the effects of structure yielding. Elastic displacements are generally considered to be equal to the actual displacements.

With the revolution in structural analysis brought on by the advent of modern digital computers, it may appear to the casual observer unfamiliar with structural dynamics, that the second task (i.e., predicting the effect of a given seismic loading) has evolved to a state which approaches an exact science. However, this is not the case. One of the primary reasons for this is the lack of field data on the actual magnitude of stresses and displacements occurring in bridges during a major earthquake.

In an effort to overcome, at least partially, this absence of data, a model structure was subjected to simulated earthquake loading on the shaking table at the University of California Richmond Field Station. Data gained from this experiment was correlated with results from a sophisticated research oriented computer program developed specifically to predict seismic response of bridge structures. This correlation study resulted in a substantial improvement in the algorithms used to calculate nonlinear response.

Many bridge designers do not have access to computer facilities and those that do must use programs that are less sophisticated than the one mentioned above. In practice, therefore, stresses and displacements are determined by more approximate means which employ several simplifying



CALTRANS
SEISMIC DESIGN PROCESS

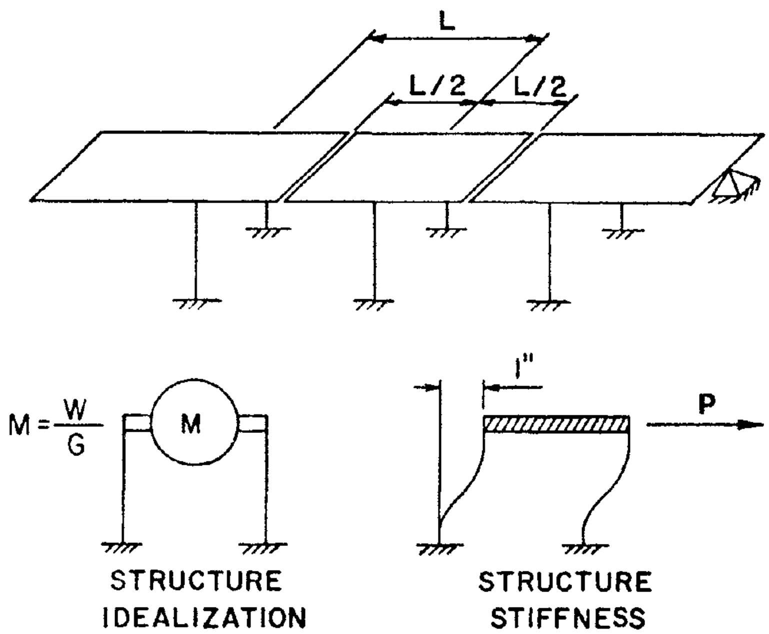
FIGURE 1

assumptions. With the present absence of field data, evaluation of these means can only be done by comparison with more sophisticated analytical approaches which are known to better model reality.

This paper deals with an evaluation of the currently used methods for predicting the response of bridge structures to a given seismic loading. An evaluation of both the equivalent static load and the response spectrum techniques for determining seismic effects on bridge structures is included. The experiences of the authors in their association with the University of California at Berkeley and the California Department of Transportation were drawn on to make this evaluation.

BACKGROUND

Prior to the San Fernando earthquake of 1971, bridges were generally designed for earthquake forces using an equivalent static force approach known as the Lollipop Method. In other words, the bridge bents were assumed to act independent of one another as single-degree-of-freedom oscillators with a lumped mass equivalent to the tributary deck mass as shown in Figure 2. Both structure period and load distribution were determined using this method.



"Lollipop" Idealization
Figure 2

Immediately following the earthquake, CALTRANS recognized the need to develop a more rational earthquake design procedure for bridges. Efforts were initiated to develop new earthquake design guidelines that would consider seismicity and the vibrational properties of both the bridge and the underlying soil. There were two basic approaches that evolved regarding the method that should be used to perform the seismic analysis for bridge design. Proponents of the first approach proposed that a simplified technique for applying equivalent static force be devised that would allow the designer to use his present knowledge of the static behavior of structures to design the bridge. Those who favored the second approach, felt it was more desirable to train the bridge designer to perform more sophisticated analyses which more realistically considered the dynamic behavior of the structure.

The first approach required the development of an improved equivalent static force approach. It became evident to the CALTRANS engineer that the previously used Lollipop Method was not a realistic method of analysis. Efforts to find a simple but realistic method of applying an equivalent static force to a wide range of bridges resulted in the formulation of a uniform lateral load technique, known as the Uniform Load Method. This technique, which was the first attempt to revise the equivalent static force method, is still not totally satisfactory, however, in that it produces accurate results for only a limited number of bridge types.

At CALTRANS there were several factors that have made the second approach involving more sophisticated analysis the most desirable. Some of these factors are as follows:

(1) The unusual geometric alignments, support conditions, and restraints of many bridge structures on a modern highway system required more sophisticated three-dimensional mathematical idealizations to obtain realistic results.

(2) Sophisticated in-house computer capabilities were available with the required mathematical idealizations to perform a dynamic analysis.

(3) It was necessary to use the same computer program to perform a space frame analysis to effectively apply the Uniform Load Method as was required to perform a dynamic analysis. Thus with modest additional training, a more sophisticated analysis was possible at a relatively small additional effort and cost.

(4) There was a combination of: 1) willingness of management, 2) ability of bridge designers to learn new techniques, and 3) an availability of qualified personnel who were assigned to provide technical support on an ongoing basis.

This approach, which has proved successful at CALTRANS, resulted in the implementation of three-dimensional response spectrum modal analysis to determine design seismic forces for bridges on a routine basis.

The AASHTO Specification [1] for Bridges (1977) reflects the two approaches by specifying that the effect of seismic forces on bridges shall be evaluated by considering the dynamic response characteristics of the total bridge using one of the following methods:

(1) Equivalent static force
(2) Response spectrum dynamic analysis

For "special cases," the specifications recommended the use of dynamic analysis techniques. Special cases are considered to be structures with one or more of the following characteristics:

(1) Located adjacent to active fault(s)
(2) Located in area with unusual geologic conditions
(3) Unusual geometry, cost, importance, etc.
(4) Structure period greater than 3 seconds

These specifications were written following the San Fernando earthquake of 1971. They are to a very large degree the reaction of CALTRANS bridge design and research engineers to the failures that occurred during that earthquake.

The San Fernando earthquake also stimulated a renewed enthusiasm for additional theoretical and experimental studies into the seismic behavior of bridges. One of these studies, conducted at the University of California at Berkeley, was designed to investigate the effectiveness of existing bridge design methodology in providing adequate structural resistance to seismic disturbances. This project extended over approximately six years and included the following six phases:

(1) A review of the world's literature relating to seismic effects on highway bridges [2]

(2) An analytical investigation of the dynamic response of long, multiple span highway overcrossings [3]

(3) An analytical investigation of the dynamic response of short, single and multiple span highway overcrossings [4,5]

(4) Detailed model experiments on a shaking table to provide dynamic response data which could be used to verify theoretical response predictions [6]

(5) Correlation of experimental and theoretical response, and modification of analytical procedures as necessary [7]

(6) Preparation of recommendations for changes in seismic design specifications and methodology [8,9]

This project made substantial contributions to the advancement of the state of knowledge regarding the dynamic response analysis of bridge structures subjected to seismic loadings. As part of Phase 6 of this project, case studies were performed to evaluate the accuracy of results obtained from currently available computer analysis techniques. Of primary concern was the response spectrum technique that has gained wide use in bridge design. The results of these case studies provided the basis for the evaluation of response spectrum analysis presented in this paper.

EQUIVALENT STATIC FORCE METHODS

Introduction

The development of a realistic simplified equivalent static load approach for the dynamic analysis of bridges that would suffice for the final design of simple bridges and could even be used for preliminary design on the more complex bridges, is desirable for the following reasons:

(1) Simple extensions of what is currently used and would be easy to implement

(2) Does not require a computer

(3) Quick and easy to apply

The determination of seismic response by the equivalent static force method basically involves three steps:

(1) Calculating the period of the first mode of vibration in the direction under consideration.

(2) Obtaining the corresponding response coefficient "C".

(3) Distributing the resulting equivalent static earthquake force to the substructure elements.

Lollipop Method

In the past, the determination of the period and distribution of the earthquake force was accomplished by simply applying the formulas in the code. The idealization for the Lollipop Method implied the following simplifying assumptions about the dynamic behavior of a bridge:

(1) Each bent vibrates in its own natural period, independent of the other bents.

(2) The transverse bending and torsional stiffness of the superstructure do not contribute to the stiffness of the system.

There are several obvious over-simplified assumptions in this approach. Even for bridges of simple geometry, the assumptions were somewhat in error. The inaccuracies that occurred in the calculation of structural period resulted in unrealistic values for the equivalent static earthquake force. In addition, the distribution of this force was in error. The main advantage of this technique was that it was simple and easy to apply.

Uniform Load Method

To overcome the deficiencies in the Lollipop Method, an empirical approach, called the Uniform Load Method, was devised with the following objectives:

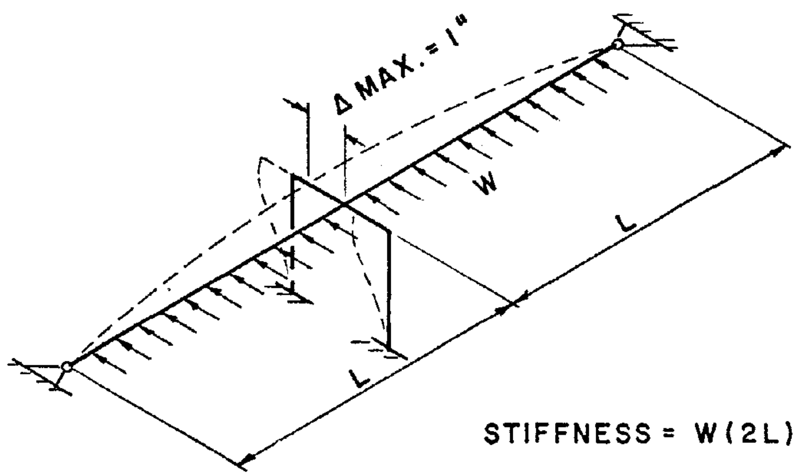
(1) Maintain continuity of the superstructure in determining the natural period of the system.

(2) Distribute the earthquake force to all of the participating elements of the bridge.

(3) Allow for ease of application using seismic design coefficients and static analysis techniques.

The steps in the Uniform Load Method approach can be summarized as follows:

(1) Apply a uniform horizontal load (usually taken as unity) to the structure in the direction of vibration as shown in Figure 3.



Uniform Load Idealization
Figure 3

(2) Perform a static analysis on the structure to determine the resulting displacements and member forces due to the applied uniform load.

(3) Adjust the maximum displacement to 1 inch. Using this adjustment factor, adjust the uniform load to correspond to a maximum displacement of 1 inch.

(4) Multiply the adjusted uniform load by the length of the structure. This is the value for stiffness which, along with the total dead load of the structure, can be used to compute the fundamental transverse period of the structure.

(5) Having obtained the period, determine the response coefficient "C" from the response curves.

(6) Determine the total earthquake force acting on the structure by combining the response coefficient with the framing factor and the total dead load.

(7) Convert the total earthquake force into an equivalent uniform load.

(8) To determine forces in the members due to this uniform earthquake loading, prorate the forces in the members from the original uniform loading applied to the structure.

The desirability of using a simple approach employing a seismic coefficient in a static analysis, rather than a complex dynamic analysis, has provided the impetus for implementing the Uniform Load Method. Recent experience has shown that this empirical approach gives accurate results for certain types of simple bridges, but it can require more effort than a response spectrum dynamic analysis. This is because the Uniform Load Method requires a space frame analysis for all but very simple structures to properly analyze the transverse stiffness of the columns interacting with the superstructure.

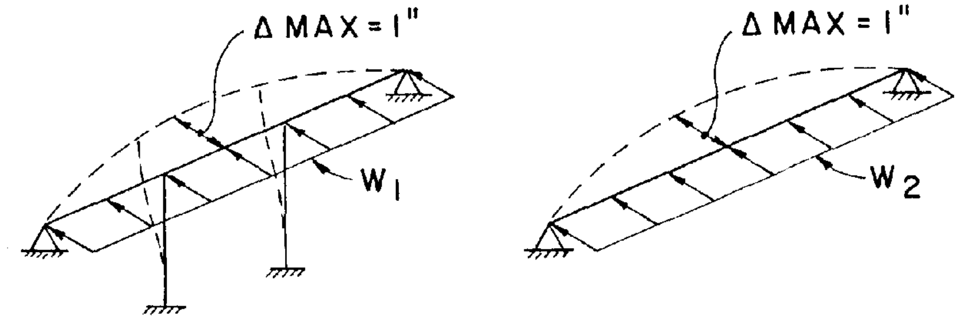
Several case studies [10] were performed to evaluate the accuracy and limitations of the Uniform Load Method as compared to a response spectrum dynamic analysis. For comparison, the Lollipop Method was also included in these case studies. In selecting bridges for these case studies, different structural and geometric characteristics were considered in order to evaluate the effect of the following parameters:

(1) Number of spans
(2) Ratio of span lengths
(3) Number of columns per bent
(4) Curvature
(5) Skew
(6) Structure width
(7) Column length and fixity

An attempt was made to categorize the types of structures which could be accurately analyzed by the Uniform Load Method. It was found that the single most important criterion for categorizing the structure was the relative stiffness between the superstructure and substructure. In order to quantify this criterion, a stiffness index was established.

The Stiffness Index relates the relative contribution of the columns to the transverse stiffness of the entire structure. As illustrated in Figure 4, the Index is found by taking the ratio of the transverse stiffness of the entire structure, including the columns, to the stiffness of the superstructure alone, acting as a simple beam.

Based on the cases considered, it was observed that the Uniform Load Method can yield accurate results for structures with certain characteristics. Continuous structures on a straight, non-skewed alignment could generally



$$\text{STIFFNESS INDEX} = \frac{W_1}{W_2}$$

Stiffness Index Definition
Figure 4

be analyzed using this approach provided the stiffness index was 2 or less. However, for structures with a stiffness index greater than 2, only those with balanced span lengths and equal column stiffnesses could be accurately analyzed. This method was not satisfactory for structures with skewed supports, intermediate hinges, or curved alignments.

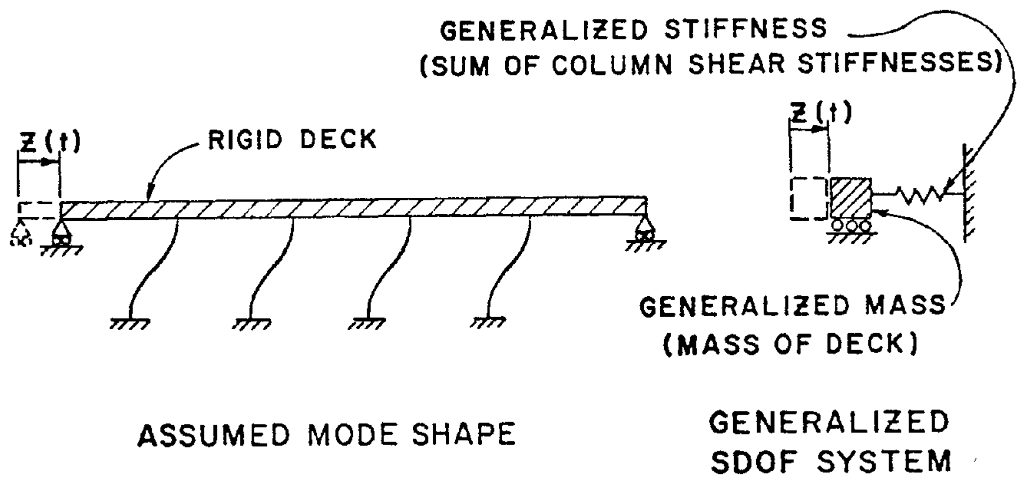
Since there are several limitations to the Uniform Load Method and since it generally requires a space frame analysis, there is a need to develop a simple but effective means for applying the equivalent static force approach to bridge structures.

In the development of an equivalent lateral force analysis procedure, it is necessary to determine the period of a structure and the distribution of the resulting lateral force. A reliable method for calculating the period must include the effective stiffness of the deck, restraining devices and soil springs, and the discontinuity of expansion joints, in addition to the individual column stiffnesses. In short, the true dynamic behavior of the bridge should be considered. The period should, if estimated, be an underestimated value to provide a conservative estimate of the equivalent lateral force. It is unlikely all bridge types will lend themselves to simplified techniques, but a large percentage of common types of bridges should be covered. Both longitudinal and transverse modes should be considered. Above all, the method should not require the use of a computer.

Generalized Coordinate Method

Another equivalent static force approach, that shows promise, can also be used to determine the period and earthquake response of certain types of bridges by applying energy principles to a generalized single-degree-of-freedom system. This method is based on the premise that the shape of the vibrating structure can be assumed and expressed mathematically in terms of a single generalized coordinate. The longitudinal and transverse modes of vibration can be separated into two classes of generalized single-degree-of-freedom systems.

For the longitudinal mode of vibration the structural displacement is characterized by the behavior of a rigid deck, limiting all the columns to equal longitudinal displacements as shown in Figure 5. This is the classical approach which has been used in the past to determine the longitudinal earthquake force for design.



Generalized Coordinate Approach
Longitudinal Mode
Figure 5

The transverse mode of vibration is more complex in that the transverse displacement of the columns are not all equal but rather are functions of their position along the superstructure as shown in Figures 6 and 7. In addition to this, the continuous superstructure will undergo bending and will thus make a contribution to the potential energy of the system.

The reliability of this method depends on the ability to predict and define the structure's mode shape. The effective application of this technique also requires that one mode dominate in each direction. Fortunately, many of the simpler bridges being designed today satisfy both of these requirements.

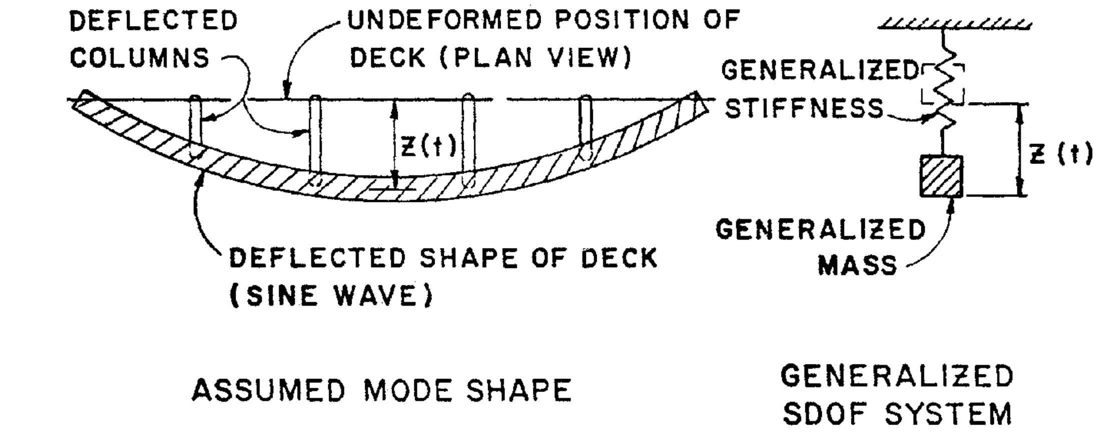
The method may be applied to girder deck bridge with no more than one intermediate hinge and having the following characteristics:

(1) Tangent or nearly tangent alignment

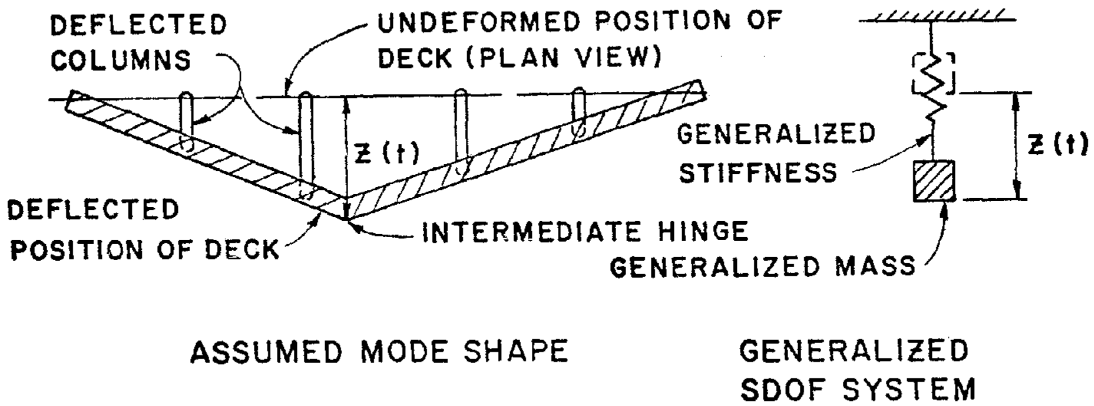
(2) Deck length to width ratio less than 15

(3) Skew angles of the abutments and supports less than twenty degrees

(4) Approximately uniform span lengths and column stiffness



Generalized Coordinate Approach
Transverse Mode (Continuous Deck)
Figure 6



Generalized Coordinate Approach
Transverse Mode (Intermediate Hinge)
Figure 7

The basic approach of the method is outlined in the following steps:

(1) Assume the predominate mode of vibration and define a generalized coordinate at the location of maximum displacement in the direction under consideration.

(2) Calculate virtual work done by external forces and internal member forces as the structure vibrates through a unit virtual displacement at the assumed generalized coordinate.

(3) Equate work to zero and solve for the structure period of the predominate mode in terms of the "Generalized Mass" and the "Generalized Stiffness".

(4) Determine the seismic coefficient from the appropriate response spectrum chart.

(5) Determine the earthquake excitation factor and scale the seismic coefficient.

(6) Determine the maximum generalized displacement.

(7) Determine the individual column forces using the generalized displacement calculated.

(8) Calculate member forces, apply ductility factors and design the member.

It should be noted that the first three steps given above are used only in the development of the formulas. The designer need not repeat these steps for each design since they are implied in the use of the formulas.

This approach was tested on several bridges which had previously been analyzed by the response spectrum technique. In most cases where this approach could be applied, the results compared well with those from the response spectrum analysis. In almost all cases, the comparison was better than was obtained using either the Uniform Load Method or the Lollipop Method.

Although the generalized coordinate approach to the equivalent static force method is not widely used, it appears to be a definite improvement over the other two methods.

THE RESPONSE SPECTRUM TECHNIQUE

Introduction

The response spectrum dynamic analysis procedure is indeed an improvement over the equivalent static force method. There are limits to its applicability, however.

The first shortcoming of the response spectrum approach is that the time domain has been removed. Since maximum modal responses do not occur simultaneously, it is necessary to use a statistical combination of modal responses such as root mean square in order to obtain realistic design loads. The actual combination of modal response depends on several factors related to the type of structure and the nature of the actual ground motion. Therefore, the use of a statistical approach to replace the effects of the removed time domain may not yield realistic results in certain cases.

Another deficiency in the response spectrum is that the duration of shaking is not accounted for by the spectrum. The major effect of duration will be on stiffness degradation and strength loss once the member begins yielding.

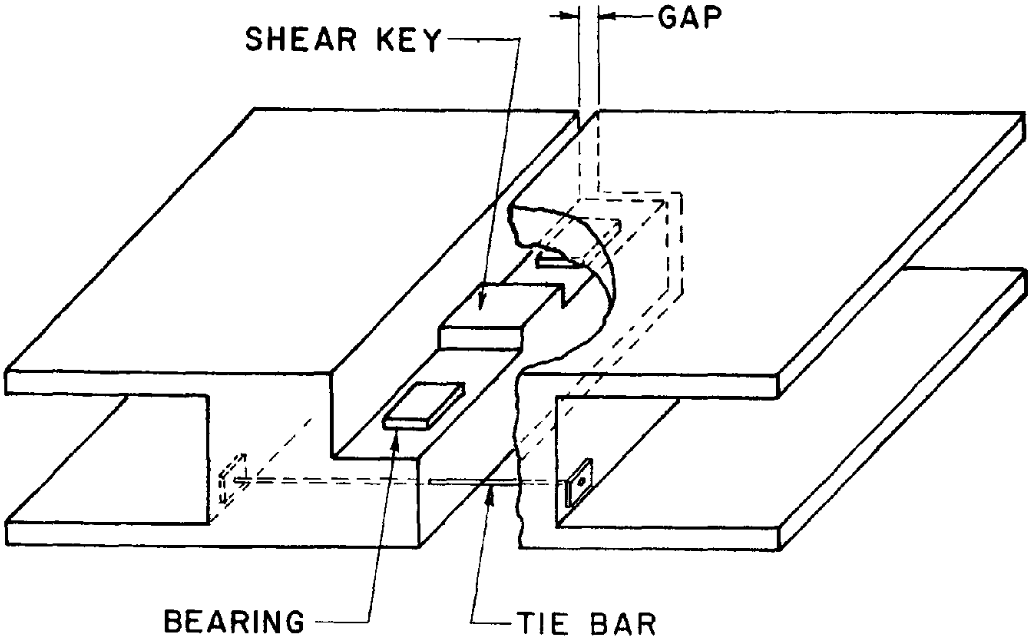
Since postelastic behavior is not specifically accounted for in the overall response analysis, a ductility factor or reduction factor is applied to reduce the forces obtained from a linear response spectrum analysis. This factor is applied either directly to the response spectrum or to the forces obtained from an unreduced spectrum. Because little is known about ductile behavior of bridges, the ductility factors used to determine the magnitude of reduction in bridge design have been extrapolated from research on building structures. Furthermore, the linear analysis does not account accurately for nonlinear behavior at expansion joint hinges, nor does it provide a means for assessing the redistribution of stress as yielding occurs in the ductile members. The analytical capabilities which evolved through the various phase of the University of California research project made it possible to evaluate the nonlinear behavior in the columns and expansion joint hinges. Recognizing both the limitations inherent in using elastic analysis techniques and the availability of improved analytical capabilities developed and refined during this research effort, case studies were conducted on three bridges to evaluate the analytical approaches currently used for seismic design of highway bridges.

The purpose of these case studies were to compare the results of a time history analysis that considers nonlinear behavior with results from both a linear time history and response spectrum analysis. Based on this comparison, the effectiveness of the current response spectrum approach as shown in Figure 1 can be evaluated.

Properties of the Bridges

Three bridges which were designed by the California Department of Transportation were selected for this study. All three structures consist of curved concrete box girder decks cast monolithically with single column bents. Because of the length of the bridges, each structure has one or more intermediate expansion joints to accommodate temperature movement.

This type of structure is common in California and is typically used in freeway interchanges. During the San Fernando earthquake of 1971, some of the most spectacular failures involved this type of bridge [2,11]. One of the primary cause of failure appeared to be the separation of expansion joint hinges. As a result, all structures of this type designed since the earthquake, including the three used in this study, have been fitted with restrainers designed to prevent separation. These restrainers must be gapped to allow freedom of movement for temperature, etc. A typical expansion joint hinge of this type is shown in Figure 8.



Typical Bridge Expansion Joint
Figure 8

In order to obtain a better understanding of the behavior of this type of bridge, each of the structures selected had a different fundamental period of vibration. A summary of some of the important properties of these bridges is shown in Table 1. These bridges are shown in Figure 9, 10, and 11.

Bridge No.	Spans		Curve Radius (ft)	Column Lengths (ft)		Hinges		Periods of the First 20 Modes (Sec)	
	Length (ft)	No.		Min.	Max.	No.	Span Location	Max.	Min.
1	694	6	600	24.3	26.3	1	3	.40	.07
2	1138	8	1075	25.1	49.4	1	5	1.11	.15
3	1410	9	1050	60.7	85.6	2	3,7	1.94	.21

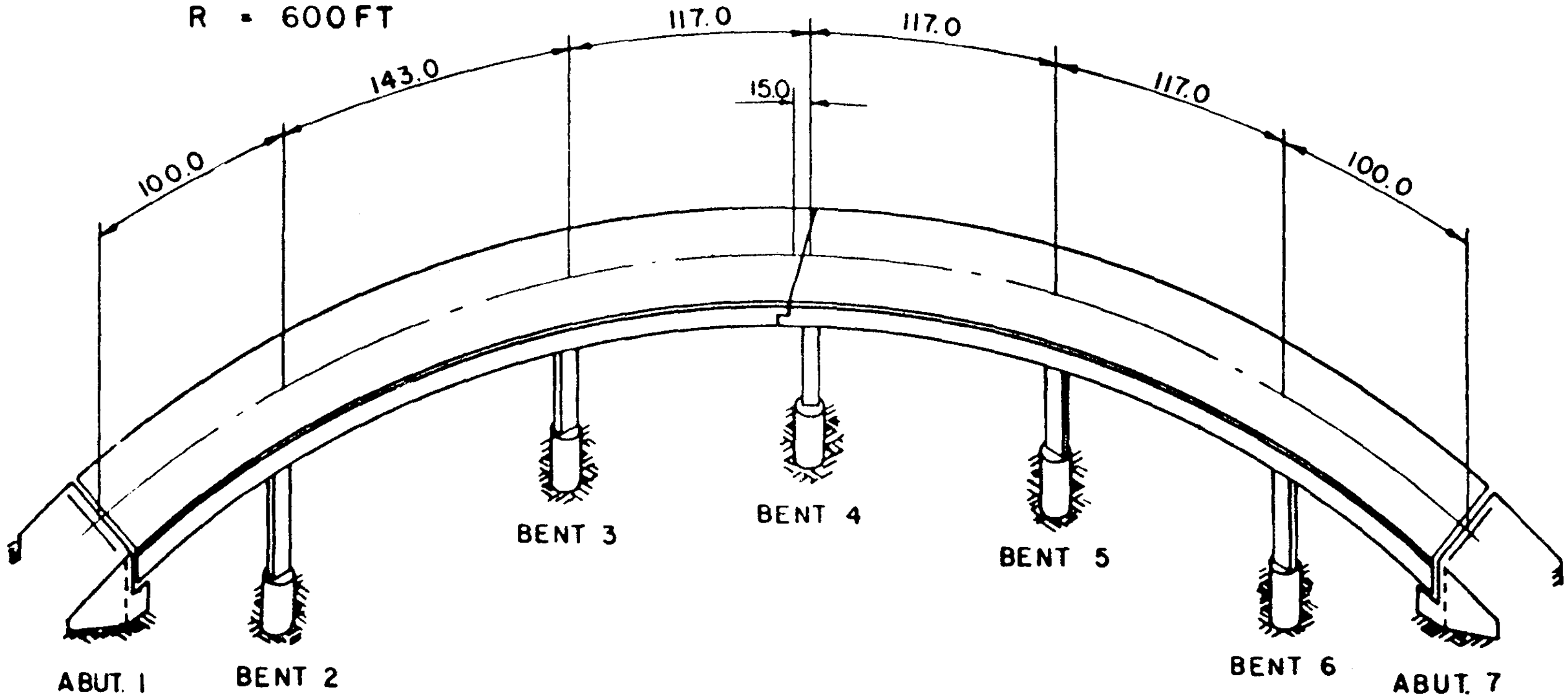
Basic Characteristics of Bridges Selected
for Case Studies
Table 1

Methods of Analysis

The following three types of analyses were performed on each of the three bridges selected.

SUPERSTRUCTURE PROPERTIES

L = 694.0 FT
A = 83.7 FT^2
I_x = 814.1 FT^4
I_y = 353.7 FT^4
I_z = 12868.8 FT^4
DL = 12.56 $^{K/FT}$
E = 3000 ^{KSI}
R = 600 FT



BRIDGE I

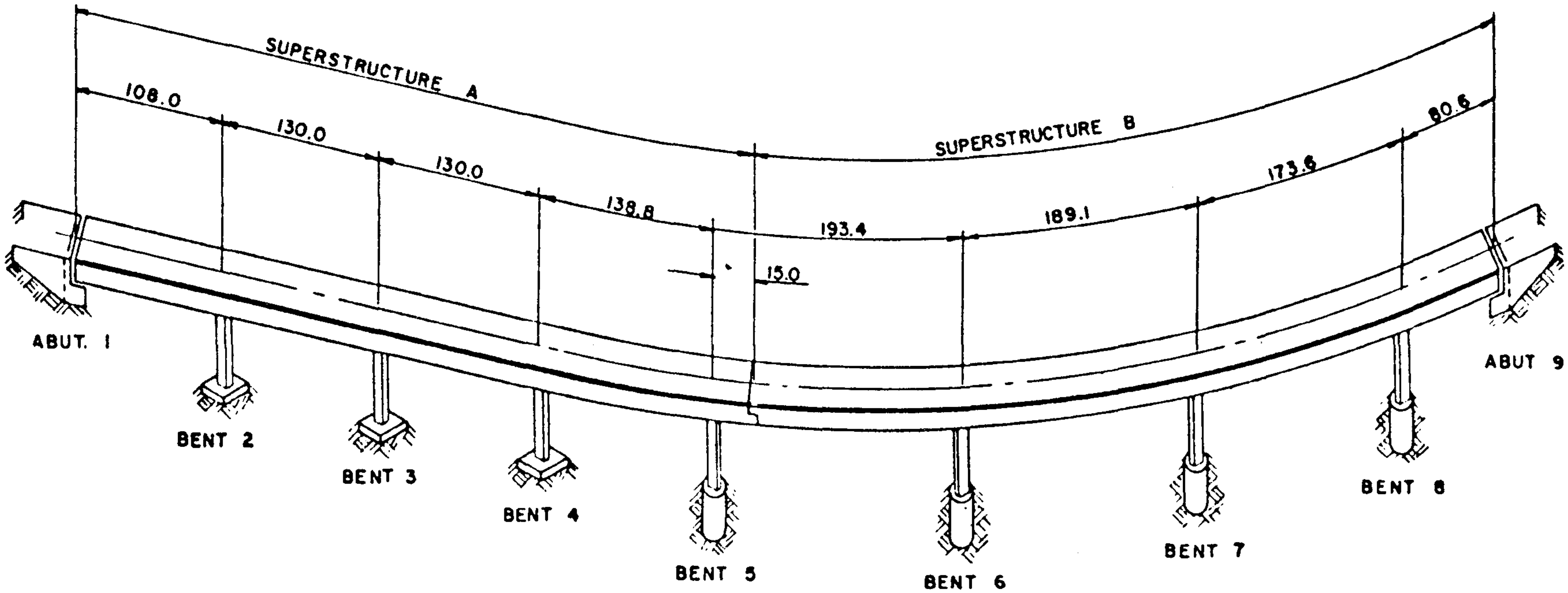
ROUTE 80 ON-RAMP OVERCROSSING

SUPERSTRUCTURE A

$L = 521.8^{FT}$
$A = 48.0^{FT2}$
$I_x = 850.0^{FT2}$
$I_y = 3130.5^{FT^4}$
$I_z = 381.4^{FT^4}$
$DL = 7.2^{K/FT}$
$E = 3000^{KSI}$

SUPERSTRUCTURE B

$L = 621.7^{FT}$
$A = 54.5^{FT2}$
$I_x = 850.0^{FT2}$
$I_y = 3528.3^{FT^4}$
$I_z = 406.5^{FT^4}$
$DL = 8.175^{K/FT}$
$E = 3000^{KSI}$
$R = 1083^{FT}$



BRIDGE 2

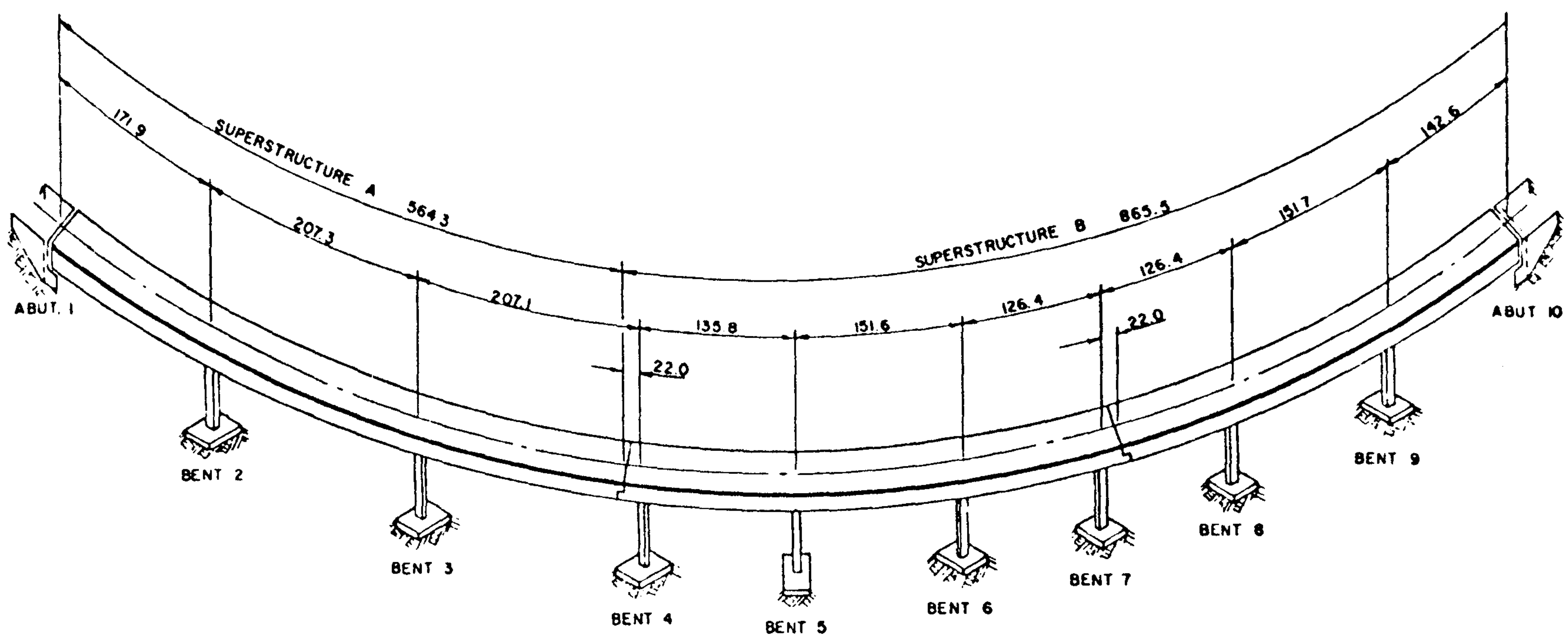
NORTHEAST CONNECTOR OVERCROSSING

SUPERSTRUCTURE A

A = 81.3 FT^2
I_x = 1000 FT^4
I_y = 9713 5 FT^4
I_z = 664.6 FT^4
DL = 12.195 K/FT
E = 3000 KSI
R = 1062 FT

SUPERSTRUCTURE B

A = 75.5 FT^2
I_x = 980 FT^4
I_y = 9152.9 FT^4
I_z = 640.0 FT^4
DL = 11.325 K/FT
E = 3000 KSI
R = 1062 FT



BRIDGE 3
SOUTHWEST CONNECTOR OVERCROSSING

(1) A response spectrum modal analysis, which is the approach that was used at CALTRANS, and appeared to be the most desirable for general use in bridge design.

(2) A linear time history modal analysis, which includes consideration of the time domain but not the effects of nonlinear behavior.

(3) A nonlinear dynamic analysis, which employed a step-by-step integration technique and included the effects of both expansion joint and column nonlinearity.

The linear analysis capabilities of STRUDL (STRUctural Design Language) were used to perform the response spectrum and linear time history analyses [12]. STRUDL is a well-known general purpose computer program for static and dynamic analysis of linear structural systems. The MCAUTO proprietory version was used [13].

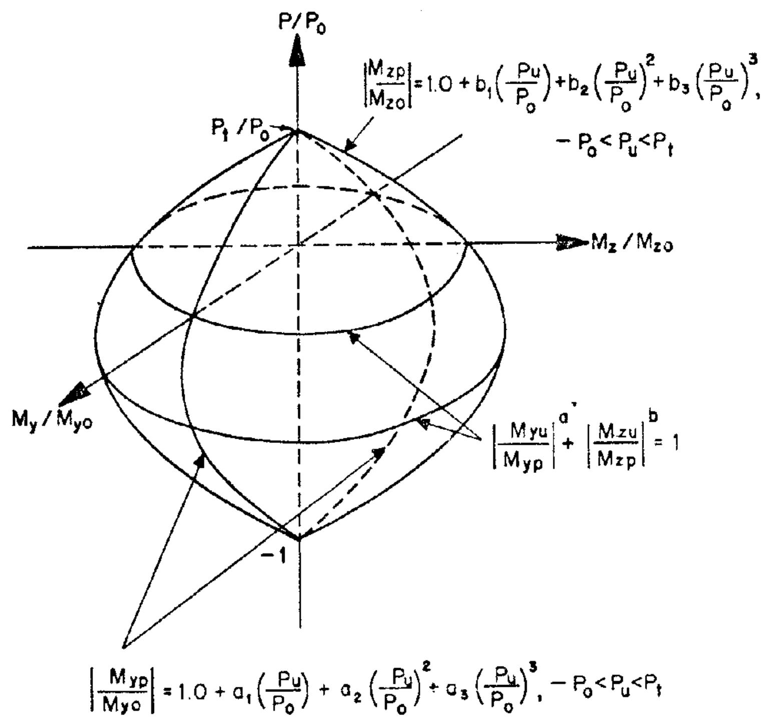
The nonlinear analysis was performed by the NEABS (Nonlinear Earthquake Analysis of Bridge Systems) program [3,7]. This computer program uses a step-by-step integration procedure which assumes piecewise linear behavior over each increment of time. The linear acceleration method was used for this study. Loading was input as rigid support accelerations. The program element library has the conventional linear elements plus the following nonlinear element types:

(1) Elasto-plastic straight beam elements
(2) Bi-linear boundary spring elements
(3) Nonlinear expansion joint elements

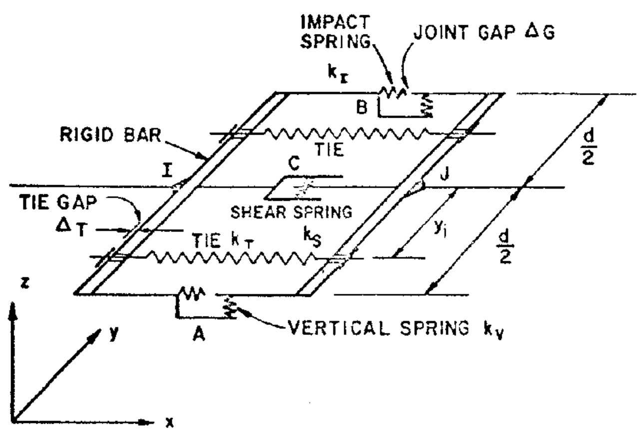
The two nonlinear parameters considered for this study were the yielding of the single column bents, and the nonlinearity of the expansion joint hinges.

The yielding of columns was limited to axial and flexural yielding along an interaction yield surface. The yield surface for a typical bridge column is shown in Figure 12. The ultimate capacity of the column in shear was considered to be infinite.

The nonlinear behavior of the expansion joint hinges were modeled using the expansion joint element shown in Figure 13. In this expansion joint hinge idealization, the restrainers were assumed inactive until movement at the joint was sufficient to take up the gaps which are normally placed in the restrainer anchorages to allow for normal movements of the joint. When the restrainers were active, they behaved in an ideally elasto-plastic manner. Relative movement at the hinge was limited by stiff impact springs which were activated upon closure of a seat gap. This represented banging of the two adjacent superstructure sections. The vertical and shear stiffnesses of the bearing pads were also included in the expansion joint element. Relative movement of the pads at the pad-concrete interface when the Coulomb friction force is overcome was also considered.



Yield Surface Description
Figure 12



Expansion Joint Idealization
Figure 13

Rigid support motion was used for all of the bridges. The SI 8+ time history ground motion developed by Seed and Idress [14] for a simulated 8+ Richter magnitude earthquake was used. The response spectrum for this motion, shown in Figure 14, was generated for 5 percent damping. This ground motion was applied to the bridges in the two orthogonal directions. The longitudinal and transverse motions were directed parallel and perpendicular to a line between the abutments.

With three types of analysis for each of the three bridges studied and ground motion in two directions, the total number of cases examined amounted to 18.

The bridge decks and columns were modeled with space frame members. Masses in the deck were lumped at the quarter points. Column masses were lumped at the third points. For simplicity, the base of each column was assumed fixed at the footing. The abutments were assumed to be free to move in the longitudinal direction. A typical structure idealization showing the location of lumped masses is shown for each bridge in Figures 15, 16 and 17.

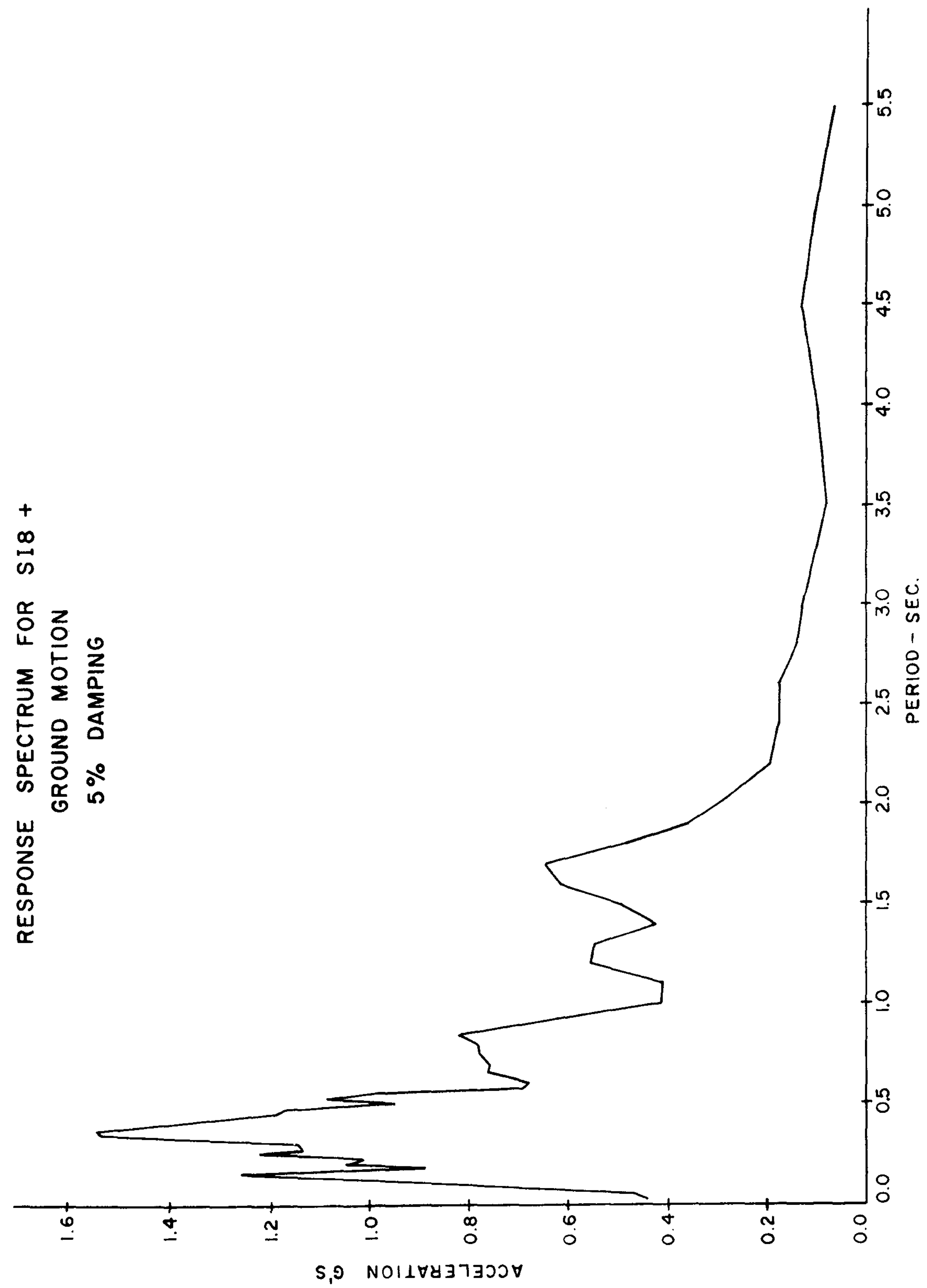
The hinge idealization for the elastic analyses was modeled by releasing main girder member axial forces, and superimposing transversely eccentric space frame members between both sections of the superstructure to account for the restrainers. This idealization assumes no gap and both tension and compression at the restrainers.

The expansion joint element used in the nonlinear analysis includes several parameters which more realistically describes the boundary conditions at the hinge. Design values shown on the plan drawings for tie and seat gaps were used. In actuality, these values will vary depending on such factors as temperature and shrinkage. Cable restrainer stiffnesses were calculated assuming an effective Young's modulus of 13,800 kips per square inch. The yield force in a typical 3/4 inch restrainer was taken as 30.6 kips. The shear stiffness of elastomeric bearing pads was calculated based on an assumed shear modulus of 135 psi. The coefficient of sliding friction for elastomeric pads on concrete was assumed to be 0.4. For lubricated sliding steel plates, the shear stiffness was assumed to be very high and the friction very low. For the purposes of modeling impacting of the superstructure, the impact spring was assumed to have the axial stiffness of the shortest adjacent section of superstructure.

Nonlinear column elements were used at locations where column yielding might be expected. Nonlinear columns were modeled on NEABS by mathematically describing the yield surface as shown in Figure 12.

Results

Modal participation factors indicated that all three structures had a tendency to respond in more than one mode. Also, because of the curved alignments, each of the bridges had some modes which included high participation in more than one global direction. This makes it likely that similar internal resisting forces will result due to seismic excitation in either global direction.


RESPONSE SPECTRUM FOR SI8 +
GROUND MOTION
5% DAMPING
ACCELERATION G'S
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
0.0
0.5
1.0
1.5
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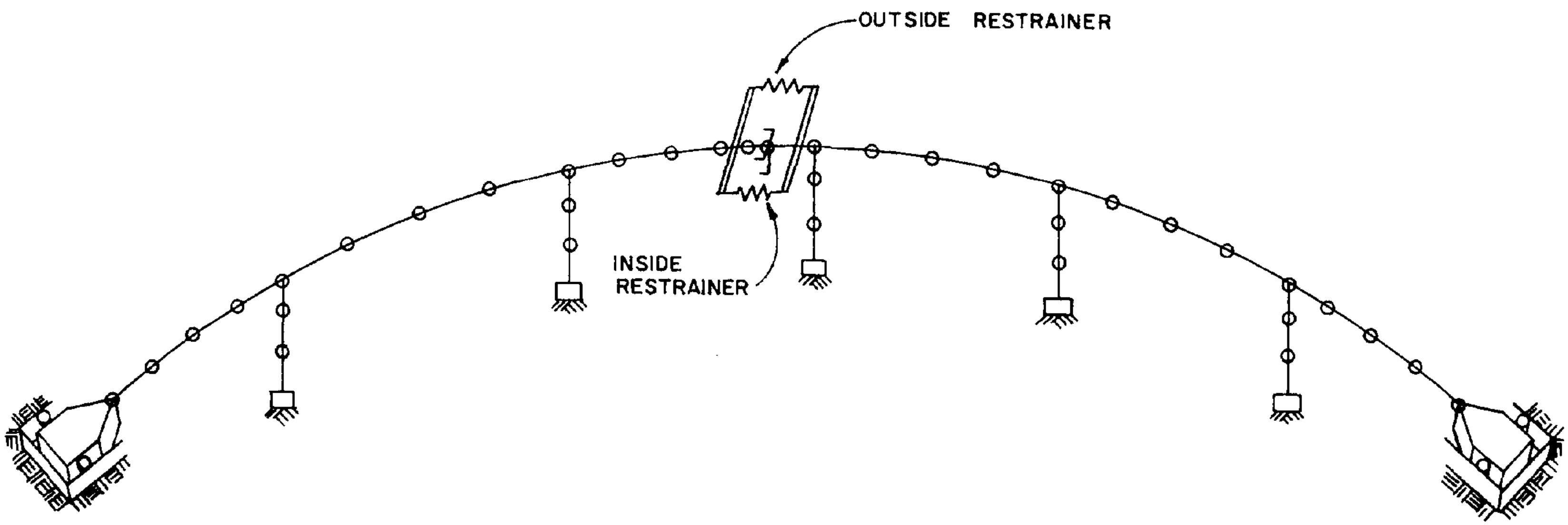
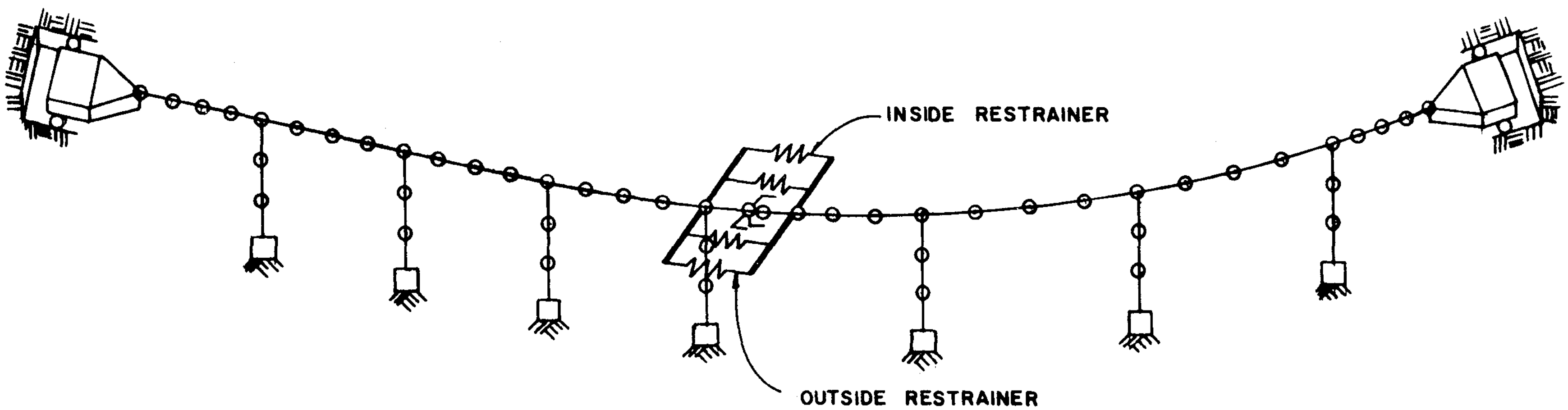
5.5
PERIOD - SEC.



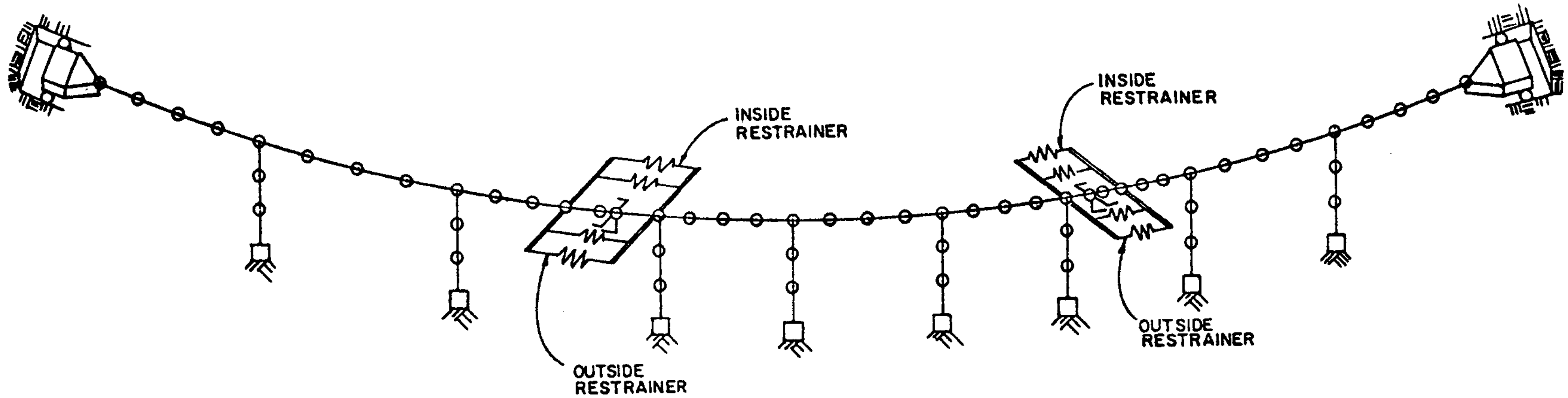
BRIDGE I

ROUTE 80 ON-RAMP



BRIDGE 2

NORTHEAST CONNECTOR OVERCROSSING



BRIDGE 3

SOUTHWEST CONNECTOR OVERCROSSING

Current bridge design practice is to consider seismic excitation in each of the global directions separately. However, because of the possibility of simultaneous excitation in more than one global direction, and the sensitivity of certain internal force components to excitation from different directions, it would appear that earthquake resistant design would be improved by considering some simultaneous contribution from seismic loading in each of the global directions.

In the case of Bridge 1, the modal periods of the first few modes were very close, and occurred near the peak on the response spectrum for the ground motion used. This resulted in the in-phase modal contributions in the direction of ground motion. In the horizontal direction perpendicular to the ground motion, however, the tendency was for the modes to respond almost exactly out of phase. This was accounted for in both the linear and nonlinear time history analysis. The response spectrum analysis, however, which was based on a root-mean-square combination of modal response, yielded results that did not agree well with the time histories. This was more pronounced as indicated by forces resulting in the direction perpendicular to the ground motion.

Because of the high response of several modes in each of the bridges studied, it was found that a combination of modes that included the peak response plus the RMS of the remaining responses yielded results more in agreement with the linear time history in most cases.

The nonlinear time history analysis results indicated that significant column yielding could be expected in Bridges 1 and 2 while Bridge 3 would have experienced very little yielding. Since these bridges were designed to resist different intensity loadings, this was not considered to be significant.

Bridge 1, because of its lower fundamental period, was subjected to a considerable number of stress reversals that resulted in substantial yielding of the columns. Intuitively, from observing the time history of yielding for these columns, it would appear that a great deal of column degradation would have occurred. Yet the ductility demands, which were based on the maximum nonlinear column deformations, were well below the values considered to be available based on monotonic loading experiments. This points up an interesting deficiency in the current method of designing bridge columns. Based on the above observation, it would appear that short period structures would have a reduced available ductility in the columns due to the increased column degradation that would occur during the larger number of excursions into the nonlinear range. Not only is this not considered in applying a ductility reduction factor to column forces derived from an elastic analysis, but it is common practice to further reduce the forces in short period structures by a risk factor of 2. It would appear that this is just opposite to what should be done.

The nonlinear results for Bridge 2 yielded the highest single maximum column ductility demand of all three structures. The ductility demands in the remaining columns were not as high. It was interesting to note that the elastic moments from this earthquake were approximately double the yield moments. Therefore, had the normal ductility reduction factor been used to design the column for this seismic loading, the ductility demands would have been even higher. The reason for the high ductility demands in this single column, was the nonuniformity of column stiffness and yield moments which resulted in nonuniform yielding. The current practice of approximating nonlinear behavior by applying a constant ductility reduction factor to an elastic analysis cannot predict this type of behavior.

The effect of large deadload moments was demonstrated in the nonlinear results for Bridge 2. Column yielding was more pronounced in the direction of high deadload moments. This resulted in a biased response that resulted in a tendency to relieve the deadload moments due to yielding. Since this would effect the distribution of normal service load moments and shears following an earthquake, this should be considered during design.

In all the transverse loading cases where column yielding occurred, the nonlinear analysis yielded seismic shear forces at the abutments that were greater than the linear time history analysis results. This is because the columns were incapable of carrying all the shear forces determined in the elastic analysis, and the excess was transferred through the deck to the abutments. This same phenominon was observed at the hinge in Bridge 2. This particular hinge was located near a stiff column that behaved similar to an abutment during an earthquake. In general, however, hinge shear key forces were slightly less in the nonlinear analysis.

The maximum deck displacements from the nonlinear analysis were almost always less than those from the elastic time history analysis. The exceptions to this were when localized maximum yielding occurred early in the earthquake, and when the deadload moments caused biased yielding as mentioned earlier. Classical methods of predicting nonlinear displacements based on equating strain energy from an elastic analysis to the sum of strain energy and energy dissipated in a yielded structure did not apply for these bridges.

It was obvious that because of reduced deck displacements and the normal gaps that are placed at the hinges to allow for free movement, that hinge restrainers were not stressed in the single hinge bridges. Stresses were developed in the restrainers in the two hinge bridge. The banging action that occurred between the adjacent sections of superstructure caused these forces to vary considerably from the elastic analysis, however. Currently, there appears to be no way of accurately predicting restrainer forces from an elastic analysis. The methods currently used seem to, at least for these bridges, yield conservative results.

CONCLUSIONS AND RECOMMENDATIONS

Based on the evaluation of the current methods for determining dynamic response to seismic loading, the following general recommendations can be made relative to the improvement of seismic design methodology for bridges:

(1) The Uniform Load Method for applying the equivalent static force approach to seismic design of bridges is not totally satisfactory. An improved method using energy principles should be further developed and implemented into the bridge design process.

(2) The response spectra currently used in the AASHTO specifications should be revised so as not to include the reduction for ductility. Ductility reductions should be made on an individual component basis.

(3) Seismic design provisions should consider the simultaneous application of earthquake motion in the three component directions since there is in many types of bridges coupling between the component directions within each mode of vibration.

(4) The PRMS (i.e., peak plus RMS of the remaining) combination of modal contributions resulting from a response spectrum analysis is an improvement for certain bridges analyzed by the response spectrum technique and may potentially be used for bridges having two modes of vibration with approximately equal periods.

(5) Seismic design provisions should establish some threshold of yielding for moderate earthquakes expected to occur several times during the expected life of the bridge. The need for this aspect of seismic design becomes more prevalent when consideration is given to the unequal distribution of ductility demands in a structure having non-uniform column stiffnesses.

(6) The number and levels of inelastic excursions which take place in reinforced concrete columns during a maximum credible earthquake should be such that stiffness and strength degradations are minimal. This control is accomplished by proper design and detailing of reinforcement.

(7) The seismic design should provide for an increase of approximately 1.5 to 2 in the forces at the abutments derived from an elastic analysis if yielding in the columns is anticipated.

(8) Design provision for combining girder moment due to dead and live-loads should include the effects of deadload moment redistribution due to possible relief of deadload moments at the location of a plastic hinge in the column during an earthquake.

(9) The use of intermediate hinges should be avoided if possible in bridges located in areas of high seismicity.

(10) Nonlinear computer capabilities should be made more user oriented for the practicing engineer and should be disseminated to the engineering profession so that they can be used to:

   (a) Make parameter studies of the seismic nonlinear behavior of bridges

   (b) Develop more realistic seismic design code provisions

   (c) Apply nonlinear analysis as a design tool for complex bridges

The questions raised during the course of this evaluation indicate the need for future studies to perfect analytical capabilities for predicting seismic response. Some of the areas that need particular attention are as follows:

(1) Stiffness and Strength Degradation - The possibility of occurrence and the effects of stiffness and strength degradations in reinforced concrete columns on nonlinear dynamic response should be considered.

(2) Energy Absorption - The important role of inelastic energy absorption in the columns and expansion joint restrainers should be studied further. Special attention should be given to developing a clearer understanding of the concept of ductility and how it relates to bridge design so that elastic analysis techniques may be used with a greater degree of confidence by the bridge engineer.

(3) Restrainer Units - Non-uniform yielding and ductility demands in columns result in larger forces at the restrainer units for bridges with more than one intermediate hinge. These effects should be studied further to investigate the current minimum specification in the code and to determine if elastic analysis techniques currently used can predict these restrainer forces.

(4) Response Spectrum Analysis - Special studies to improve the results gained from a response spectrum analysis are needed. The determination of the most effective means of combining modal results for a particular bridge is especially needed.

(5) Equivalent Static Force - Additional studies should be made to better define the degree of applicability of the generalized coordinate approach to the simplified equivalent static force method for the seismic analysis of bridges.

A computer capability such as NEABS represents a powerful research tool. It may be effectively used for studying special problems related to bridge design and analysis, and for analyzing bridge response due to past and future earthquakes. Because of its potential for advancing the state of knowledge, these computer capabilities should be made more user oriented to provide researchers and engineers with effective means for analytically studying bridge seismic behavior.

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EXPERIMENTAL DYNAMIC RESPONSE
INVESTIGATIONS OF EXISTING HIGHWAY BRIDGES

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ABSTRACT

A six span four hundred and fifty foot long continuous composite girder bridge has been subjected to extensive dynamic testing to identify its structural dynamic properties. Transverse motions were induced by quick release pull-back testing and vertical motions were induced by normal vehicular traffic. Analysis of the data obtained indicates that this type of dynamic testing is very effective for bridge structures of this type.

After obtaining the experimental dynamic properties, a linear analytical model was used for purposes of comparison with the experimental results and for estimating the seismic forces induces by small to moderate earthquakes having recurrence times on the order of 30 to 40 years in Western Nevada. Results of this analysis indicate that the earthquake forces prescribed by the AASHO specifications (1961), under which this bridge was designed, are too low for seismic regions of the Western United States. The 300 pound per linear foot minimum transverse wind load controlled in the design, but dynamic seismic analysis indicates that lateral forces in excess of four or five times those caused by the wind loads would be expected to occur every 30 to 40 years.

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INTRODUCTION

The state-of-the-art regarding the earthquake resistive structural dynamic performance of buildings is relatively advanced when compared to that of highway bridge structures. On the experimental side, a number of actual building structures have been extensively tested by subjecting them to sinusoidal forced vibrations and their associated dynamic properties obtained. Two examples of such studies may be found in references 24 and 26. Many building have been studied for their dynamic properties by means of ambient vibrations [3, 19, 31, 32, 34] where the excitation may be wind gusts, microtremors, or other random sources. Other buildings have also been appropriately instrumented during nuclear tests in order to recover the resulting building vibrations caused by the ground shaking induced by underground nuclear explosions [4, 5, 9, 10]. Moreover, during the 1971 San Fernando, California earthquake, eleven major tall buildings which were equipped with suitable strong motion instrumentation have been studied in detail for their structural dynamic properties and seismic response characteristics during that earthquake [17]. The value of the experimental data obtained was enhanced by comparing it to theoretical calculations made for these eleven buildings using the measured input ground motion obtained from each of the buildings during that earthquake. In addition, there is a very extensive literature on the subject of theoretical structural dynamics and seismic response studies conducted on a wide variety of structural systems.

Stimulated by the substantial damage caused to a large number of bridges [13, 15, 22] during the moderate (magnitude 6.4) 1971 San Fernando earthquake, a good deal of progress has been made in upgrading our knowledge regarding the seismic resistance and structural dynamic performance of bridge structures.

The research group at the University of California at Berkeley has been very active in this work. A survey of their project in this area has been prepared by Penzien and others [27]. At the outset, a compilation of the worldwide literature on the seismic effects on highway bridges was developed by Iwasaki and others [21]. Subsequently Tseng and Penzien [35, 36, 37] and Chen and Penzien [7] have analytically examined the response of some bridges representative of those damaged during the San Fernando earthquake. They have included linear and nonlinear behavior in their analytical models as well as soil-structure interaction effects. Godden [18], and Williams and Godden [38] have been conducting sophisticated physical model studies on the University of California shake table for one of the long-span bridges which totally collapsed during the San Fernando event. Their physical model may be used to examine both the linear and nonlinear behavior of principal features of this particular bridge structure. Comprehensive analytical studies have also been conducted for the same structure, and studies [23] correlating the measured and theoretical responses have also been made.

The availability of dynamic response measurements suitable for seismic studies obtained from full scale bridge structures is much more limited than for buildings. The Japanese have done much of the previous work in this area, and Iwasaki and Penzien summarize these experiments in their literature survey [21]. Shepherd and Charleson [29] and Shepherd and Sidwell [30] have performed response tests using steady state sinusoidal excitation to obtain the transverse dynamic properties of several reinforced concrete bridges in New Zealand. Abdel-Ghaffar [1] has conducted experimental and analytical work on a suspension bridge.

The lack of strong motion earthquake instrumentation on bridge structures in the United States prior to 1971 has contributed to the fact that little dynamic response data for bridge structures exists for bridges. This problem has been discussed by Matthiesen [25]. The San Fernando bridge damages and recent research has stimulated changes in earthquake resistant design procedures which have been reported by Gates [16] and Cassano[6]. Also motivated by the San Fernando bridge damages and current research, Cooper and others [8] and Privitzer and others [28] have made recommendations regarding the seismic retrofitting of existing bridges to improve their performance during earthquakes.

In order to continue to advance the state-of-the-art of seismic resistant highway bridge design and construction, the research efforts in this area need to be continued and expanded. This is particularly true for <u>experimental studies on full scale bridges</u>. The benefits to be derived from such an effort will be the development of a safer more reliable highway system in earthquake prone areas of the United States, as well as a reduction in the earthquake caused financial losses associated with highway bridges in future earthquakes. During the 1971 San Fernando earthquake, about $10,000,000 worth [13] of damage was done to highway bridges alone.

One of the experimental methods which can be used to identify the structural dynamic properties of bridge structures is quick release pull-back testing. The remainder of this paper summarizes the results obtained from applying the method of quick release pull-back testing to a six span composite girder highway bridge [12].

EXPERIMENTAL ANALYSIS

<u>Data Acquisition</u>. The purpose of the experimental phase of this research was to determine whether the dynamic properties (natural frequencies, mode shapes, and modal damping ratios) of bridge structures could be effectively identified by using quick release pull testing for transverse motions and vehicular induced vertical motions. The structure studied was the six span continuous composite girder ramp 13 access bridge to US 395 south from I-80 east in Reno, Nevada, shown in Fig. 1. Four Kistler force balance (305A-515) accelerometers were used to measure the bridge vibrations which were recorded on a four channel FM Ampex (SP700) tape recorder.

To produce the transverse vibrations a D-8 Caterpillar tractor was used to pull a one inch wire rope cable attached to one of the bridge piers. A special purpose quick release hook was developed to safely quick release the cable, thus setting the structure into motion. A typical transverse accelerogram obtained in this manner is shown at the top of Fig. 4. Cable tensions were measured with a calibrated load cell and were in the range of 5 kips to 12 kips. Peak accelerations produced were in the range of 0.5 to 1.0 percent gravity. Vertical motions used in this experiment were induced by vehicular traffic, and analysis of the vertical motion data indicates that the presence of the traffic live load on the bridge does not affect the results.

All acceleration measurements were made at stations spaced 20 feet apart on the concrete barrier rail. Several sets of transverse measurements were taken with one of the accelerometers left in a fixed position at the centerline of pier 3. (The piers are numbered consecutively from the south in Fig. 1.) At the same time the remaining three accelerometers were set at stations 20 feet apart starting at one end of the bridge. After measuring the motions produced by a single release of the cable, the three moving accelerometers were then set up at new stations until the motions of all stations were measured relative to the fixed station at pier 3.

Experimental Mode Shapes and Natural Frequencies. The Fourier analysis techniques used in the study of ambient vibrations of buildings were applied in this study to determine the natural frequencies of the structure and their associated mode shapes. In the plan of ramp 13 (Fig. 1) a median strip between ramp 13 and the US 395 overcrossing structure can be noticed to extend to the north 130 feet from the south abutment. This median slab is cast monolitically with the US 395 structure and rests on the ramp 13 deck separated by tar paper. Analysis of the transverse data indicates that this structure did not behave as a dynamically unique structure at the amplitudes of the test motions due to the presence of this median strip. This means that multiple frequencies of vibration exist for each mode of vibration and that each frequency has its own mode shape. The lowest and second lowest natural frequency observed within each mode are listed in the first two columns of Table 1. The two experimentally determined fundamental mode shapes at 2.0 hz and 2.3 hz are shown in Fig. 2. It is apparent from Fig. 2 that these mode shapes are quite well defined by the data. Multiple second transverse modes and a transverse fifth mode were equally well defined experimentally. The first four vertical mode shapes obtained from the vehicular data while the traffic is still on the bridge, are shown by the data points in Fig. 7 and can be seen to be very well determined.

Mode No.	Lowest Exp. Value (HZ)	Second Lowest Exp. (HZ)	Analytical Model (HZ)
1	2.0	2.3	1.75
2	3.4	4.0	2.64
3	6.1	6.8	4.58
4	8.2	9.2	7.17
5	10.9	12.0	9.32

Table 1. Transverse frequencies of vibration through first five modes

Experimental Modal Damping Ratios. Damping estimates from the quick release data were made by use of a moving window Fourier amplitude spectrum.

An alternate method which has been used for estimating modal damping ratios from pull-back data is the application of a band pass filter to the time series [20]. For a given well separated mode of vibration, the theoretical decay of the peak amplitude of the spectrum versus initial time of the window can be estimated by using a damped single degree of freedom oscillator. This theoretical decay of the peak spectral amplitude versus initial time of the moving window for various percentages of critical damping is shown in Fig. 3. Experimental damping estimates were made by comparing these theoretical decay curves with decay curves obtained from the data. Two such experimental decay curves and the assoicated theoretical curves are shown in Fig. 4 for the 2 hz fundamental mode and the 11 hz fifth mode. In excess of 10 well defined damping estimates were made in the first, second and fifth modes, and all the modal damping ratios were found to range between two percent and three percent of critical for peak transverse accelerations in the range of 0.5 to 1.0 percent of gravity, and peak transverse displacements in the range of five to ten thousandths of an inch.

THEORETICAL ANALYSIS

Analytical Model. After determining the system's dynamic properties experimentally, a linear lumped mass model was developed to predict these properties theoretically. The SAP IV structural analysis program developed by the University of California at Berkeley was used for this purpose [2].

The composite reinforced concrete and steel grider deck is supported at the abutments and at piers 1, 2 and 5 by 12 inch high rocker bearings. At piers 3 and 4, the deck is attached by fixed bearings that prevent longitudinal movement relative to the pier caps. This allows the longitudinal thermal displacements to take place relative to piers 3 and 4. The clearance between the abutment walls and the nearest girder steel is 4½ inches, and the clearance to the deck concrete is 3 inches. All longitudinal loads are resisted by piers 3 and 4 in vertical cantilever action relative to the 12 foot by 12 foot spread footings at the base of each of the 3½ foot diameter pier columns. For these reasons a single degree of freedom dynamic model was used for the longitudinal motion and had a fundamental period of 1.3 seconds.

In the vertical sense, the composite girder was modeled by the lumped mass technique as a roller supported continuous beam with a nodal spacing of 4 feet. A detailed accounting of all section geometry for the 3 steel girders and the composite concrete deck was made. All changes of section stiffness caused by differences in flange steel were included in the analysis.

In the transverse sense the system was treated as a lumped mass continuous beam restrained by transverse elastic supports which model the pier and diaphram frame elasticity. A four foot nodal increment was used. The deck was modeled both as a continuous beam using the gross section of the deck concrete, and as a completely composite concrete and steel member using all girder steel. Differences in mode shapes and modal frequencies for these two models were negligible out through the fifth mode, and in all subsequent analyses the deck was modeled by using the gross section of the concrete only. As a final refinement, the effect of the 130 foot median strip alluded to previously, was modeled as a continuous variable stiffness elastic support with a total stiffness measured in multiples of the total transverse stiffness of piers 1 and 2.

Analytical Mode Shapes and Natural Frequencies. It was noted under the experimental section that the structure did not behave as a unique dynamic system, but had multiple transverse mode shapes and associated natural frequencies. At the top of Fig. 5 the two observed fundamental mode shapes at 2 hz and 2.3 hz are shown as curves (b) and (e) respectively. Also shown are the theoretical fundamental mode shapes (a), (c) and (d) which were calculated for a median strip stiffness of 0, 5 and 100 respectively (herein median stiffness is measured in multiples of the total transverse stiffness of piers 1 and 2). The first five transverse natural frequencies plotted as a function transverse median strip stiffness are shown in Fig. 6. From Figures 5 and 6 it can be readily seen that the restraining effect of the median strip accounts for the existence of multiple mode shapes for a given mode at the smaller amplitudes of vibration achieved during the tests. In Fig. 6 the black dot on each of the frequency curves indicates the lowest observed frequency in a mode. Frequencies at the left represent the analytical natural frequencies for zero median elasticity and the assymptote for the frequency curves f_1 through f_5. In Fig. 5 the observed 2 hz fundamental mode (b) can be seen to lie between curves (a) and (c) with median stiffness of zero and five. The lowest experimental frequency is also observed to lie in this stiffness range in Fig. 6. The 2.3 hz observed fundamental mode is approximated by the analytical curve (d) with a median stiffness of 100. Both the f_1 frequency curve in Fig. 6 and curve (d) in Fig. 5 indicate that the median restraint in this mode is greater than 100. The lowest frequency experimental second mode is shown in the lower curve (b) in Fig. 5. The lower curves (a) and (c) are for median stiffness of 5 and 20 respectively. From Fig. 6 the associated second natural frequency can be seen to be in this stiffness range.

Even though the elastic modeling of the median stiffness is a very gross approximation of the complex way in which the presence of the median strip affects the transverse motions of ramp 13, it is clear that the median strip is the explanation for the multiple mode shapes and frequencies observed. Fig. 6 shows that a wide range of possible frequencies that could exist within the various modes and is the probable explanation for not being able to experimentally isolate mode shapes 3 and 4.

The data points in Fig. 7 experimentally define the first four mode shapes for the vertical sense of motion. The envelope curves in Fig. 7 represent the envelopes of all computed vertical mode shapes. The models used in this envelope computation include treating the deck as a composite section with the deck regarded as acting fully composite with the three girders over the full length of the bridge. A second model treats the deck plus concrete rails acting fully composite with the girders. Also included are the individual girders acting separately with the tributary deck concrete acting composite with them. In addition, a model was used in which the concrete in the negative moment regions over the piers where the shear connectors are spaced at two foot intervals was removed entirely. In this case, the modulus of elasticity of 4000 psi deck concrete in the center of the spans, where full composite action is expected, was taken at its full instantaneous load value. The envelope mode shapes in Fig. 7 computed from these models can be seen to be in excellent agreement with the experimental mode shapes.

Fig. 8 shows a plot of the natural frequencies in these four modes versus the modular ratio, where modular ratio is defined as the ratio of the modulus of elasticity of the girder steel to the modulus of elasticity of the deck concrete. For 4000 psi concrete, the modular ratio for full composite action for instantaneous loading is 8. In Fig. 8 the upper solid curves represent plots of natural frequency versus modular ratio for the case where the deck and rails were regarded as acting as composite with the girders. The lower dashed curves are for the case using the deck without rails. The horizontal lines at 2.56 hz, 3.42 hz, 4.52 hz, and 5.13 hz are the observed vertical natural frequencies. It is clear from Fig. 8 that the effective modular ratio is in the range of 21 to 26 for the no rail case and 30 to 33 for the case where the rails are considered. These high values occur because of the incomplete composite action over the piers.

EARTHQUAKE RESPONSE

Analytical Model. To estimate the seismic response of structure the analytical model without median strip was used. Since the test transverse displacements were small, the median strip was capable of restraining the bridge during the tests, but for the larger motions expected during earthquakes, it would not be expected to restrain the structure. The natural frequencies of the system used are listed in Table 1. The experimental damping ratios obtained were in the range of two to three percent of critical. To account for the possibility of higher damping ratios associated with larger motions, all seismic response calculations were made for modal damping ratios of two percent and five percent of critical.

Static Response. For purposes of comparison with the earthquake response results, the static lateral forces delivered to the piers and the abutments by the 0.3 K/ft. wind load, which controlled in the design, were computed. These wind lateral forces are listed in column 6 of Table 2. The associated bending moments, reactions, displacements and center line geometry of the pier frames are also listed in this table. The total dead load reaction delivered to each pier by the deck superstructure is shown in column 7. The longitudinal static response of piers 3 and 4, (the only piers resisting longitudinal load) due to the four percent g earthquake load required by the 1961 AASHO specifications for this bridge is also shown.

Input Ground Motions and Recurrence Times. Since the analytical model used is linear and appropriate only for relatively small motions, five accelerograms recorded primarily on soft sites were selected which could be expected to occur at least several times in the lifetime of the structure. The bridge is founded on a relatively soft site. The five accelerograms used in this study are shown in Fig. 9, and their properties including estimates of recurrence times in Western Nevada[11] are indicated at the top of Table 3. In each case, the lower recurrence time is appropriate for dynamic analyses in which the peak response occurs early in the time history and the duration of shaking from the onset of the record is short. The longer recurrence times are appropriate for the full duration accelerograms.

Fig. 10 shows the curve from which recurrence times were estimated, and is appropriate for the average seismicity of Western Nevada. One of the principal assumptions that entered into the development of these curves was that it is equally likely that earthquakes can occur anywhere in the region, which is reasonable for the more or less homogeneous distribution of faulting and earthquake activity in Western Nevada. Seven hundred earthquakes having a magnitude greater than 4 occurring in the 38 year period in a 33,000 square mile zone were used for developing the earthquake recurrence rates used in developing this result. In Fig. 10, $\hat{a}$ is a peak rock acceleration expected at the site, T is recurrence time in years for the peak acceleration to equal or exceed $\hat{a}$. M is Richter magnitude, and D is duration of shaking in seconds. The recurrence time for peak acceleration to exceed $\hat{a}$ caused by all earthquakes having a magnitude of 5 or greater is read off the left hand scale for M = 5. If it is desired to find the recurrence time for peak acceleration to exceed $\hat{a}$ caused by all earthquakes having a magnitude greater than M = 6 then the recurrence times are read from the scale where $\hat{a}$ intersects M = 6. For example, peak accelerations exceeding 0.15 g caused by all earthquakes greater than M = 5 occur about every 20 or 30 years, while longer duration strong ground shaking having peak accelerations greater than ½ g caused by all earthquakes having magnitudes greater than 6½ would be on the order of one thousand years.

Response Results. In Table 4, the average response of the bridge to all five earthquakes is listed for damping ratios of two and five percent. In the table of transverse results, column 5 lists the peak transverse dynamic displacement of the piers. Column 6 gives the time at which the peak response occurs. Column 4 gives the ratio of the dynamic pier forces and displacement to the design wind load levels as indicated in Table 2. The M_b ratio in column 3, is the ratio of the elastic dynamic moment at the base of the pier column to the ultimate moment capacity of the column. Column 2 lists the ratio of the dynamic pier force transmitted to each diaphram frame compared to the allowable force on each frame as dictated by the non-moment resisting bolted joints of the frame. It should be recalled that the safety factors in bolted joint design are on the order of three.

In the table of longitudinal results, Δ is the displacement in inches, column 2 gives the ratio of the peak longitudinal pier force to the static force caused by the four percent g design earthquake force (Table 2). The M_b ratio is ratio of the peak longitudinal dynamic moment at the base of the pier column to the ultimate moment capacity.

Table 5 gives the peak dynamic transverse responses when transverse restraints at the abutments are assumed to be nonexistent.

DISCUSSION

In column 2 in Table 4 it is apparent that the dynamic forces on the diaphram frames are substantially in excess of the allowable levels, particularly at the central piers and at the abutments. In addition, the detail designed to prevent the abutment rocker bearing from falling out should uplift occur is inadequate. Earthquake motions of this amplitude will cause these small keepers to be sheared off. Transverse displacements at the end of the deck at the abutments in excess of the available 1/8 inch of transverse clearance will initiate this failure. From Table 5 can be seen that these earthquakes could induce transverse displacements at the abutments between 0.7 inches and 1.7 inches if the keepers failed.

From Table 4 it can also be seen that the linear theory indicates that the pier frame bending moments at the middle piers will be at or above ultimate moment levels in the transverse sense. In the longitudinal sense, the moments are on the order of twice the ultimate level. These moments should be added vectorially at the base of the columns because transverse and longitudinal motions occur simultaneously. The larger longitudinal moments occur because all the longitudinal loads on structure are carried by piers 3 and 4 only.

It should also be noted that because of the near balance of moments at the top and bottom of the pier columns (Table 2), the ultimate moments will be reached at about the same time. This means that pier frames in this condition are in a plastic collapse mode, and provides no further resistance to lateral forces.

A few comments are in order regarding the relatively moderate levels of ground motions used in this study. The average peak acceleration was 0.17 g, the peak velocity was 6.5 in/sec. and peak displacement was 2.8 inches. Because the peak dynamic responses occurred within the first few seconds of shaking the appropriate recurrence time is on the order of 30 to 40 years.

CONCLUSIONS

1. Quick release pullback testing is an effective method of identifying the structural dynamic characteristics (natural frequencies mode shapes, and modal damping ratios) of bridge structures. The method is particularly attractive for experimental testing of bridge structures because data can be obtained during lulls in traffic. Even in the case where the traffic on the structure is heavy, the quick release pull-back testing approach would mean that traffic would only have to be stopped for very brief periodic intervals to obtain the necessary data. Resonance testing would involve the closure of the structure to traffic for more sustained periods of time. In addition, the cost of acquiring field data would be low compared to resonance testing.

2. More physical testing of various classes of bridge structures should be undertaken to identify their dynamic characteristics in the field. Vibration amplitudes should be varied in order to determine the effect of vibration amplitude on these properties. Achieving the desired experimental amplitudes with the quick release method is relatively simple, and by carefully studying the effects of low amplitude vibrations, safe larger vibration amplitudes could be selected. The ideal time to select a bridge for this form of physical testing would be in the design phase. The quick release dynamic loads are well defined so it would be a relatively simple matter for the designer to estimate their effect upon the structure at any desired amplitude level. In addition, simple pick up points for attaching the loading cable could be provided.

3. The soil structure interaction problem should be considered at least in an approximate manner if a detailed soil structure interaction dynamic analysis is not performed. In the case of ramp 13, the analytical results were sensitive to the modulus of subgrade reaction used to approximate the soil structure interaction effects.

4. In composite girder construction, the details used to transmit the lateral loads from the deck superstructure to piers and abutments need a complete review. In particular, the diaphram frames should be moment resisting of adequate capacity. Also the rocker bearing details need adequate restrainers to prevent excessive displacements and their use eliminated wherever possible.

5. Relatively too much emphasis has been placed on vertical and thermal

loads compared to the seismic forces. Care should be exercised to insure that the capacity of bridge structures to resist seismic forces should be more balanced in the transverse and longitudinal directions.

6. The earthquake forces specified in the 1961 AASHO code under which this structure was designed are too low for Western Nevada and other areas of comparable seismicity. Bridges similar to the one studied herein, can be expected to sustain some seismic structural damage every 30 to 40 years in areas with seismicity comparable to Western Nevada, and would probably collapse in the event of a maximum credible earthquake.

7. In seismic areas comparable to Western Nevada, a review of all important bridges designed under the old AASHO earthquake regulation should be undertaken for purposes of making possible retrofit recommendations regarding their seismic performance.

8. Vehicular traffic induced motions can be used to effectively identify the vertical mode shapes and natural frequencies. Analysis of vertical motion, data obtained from ramp 13 strongly suggests that the vertical stiffness of composite girder bridges can be estimated by treating them as continuously composite with an increased effective modular ratio applied uniformly over the whole bridge deck. No special account need be taken of the fact that little composite action occurs over the piers in the negative moment regions other than using this increased effective modular ratio.

ACKNOWLEDGEMENTS

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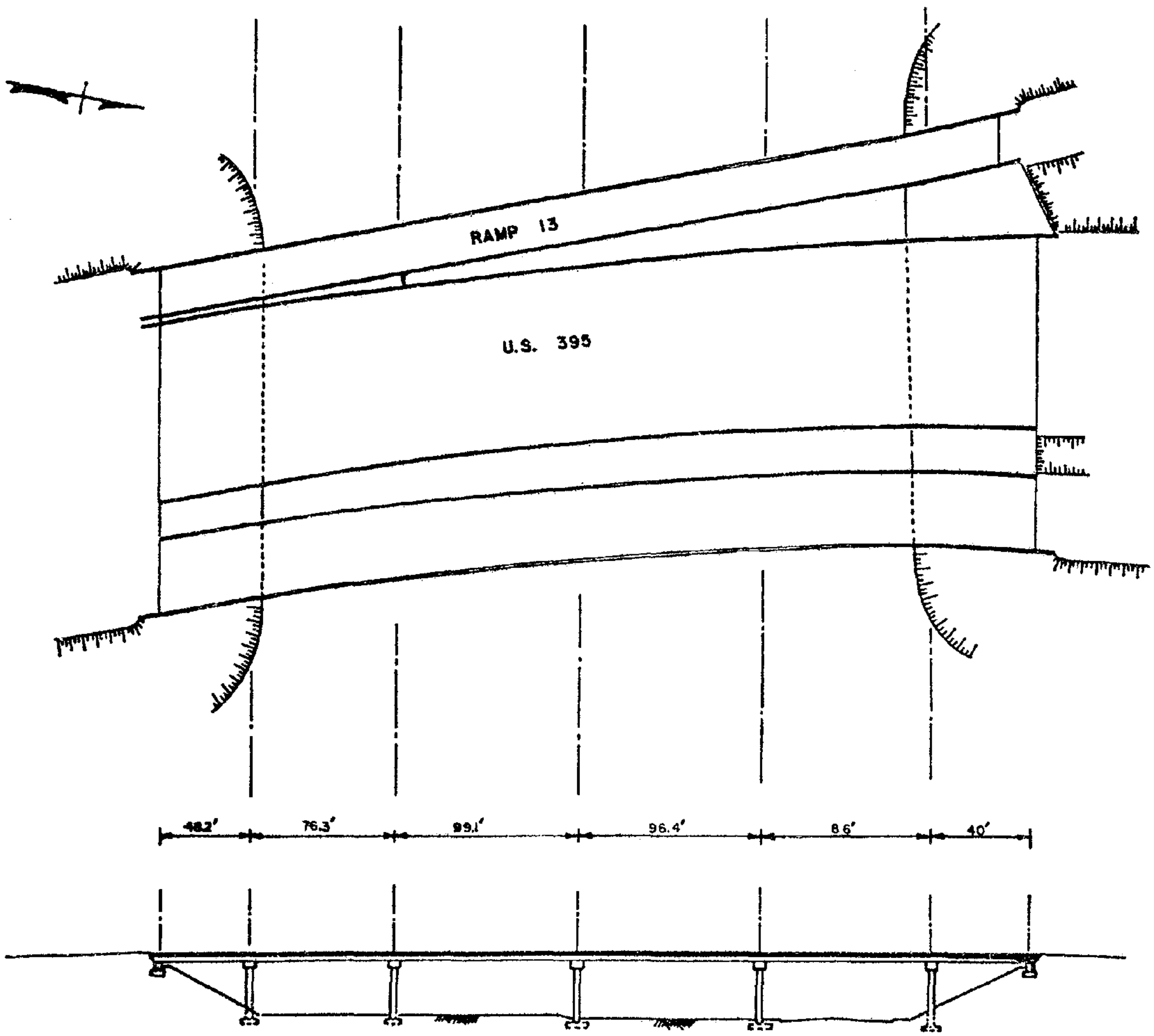

Fig. 1. Plan and Elevation
(Piers are numbered 1 to 5 consecutively from left to right)

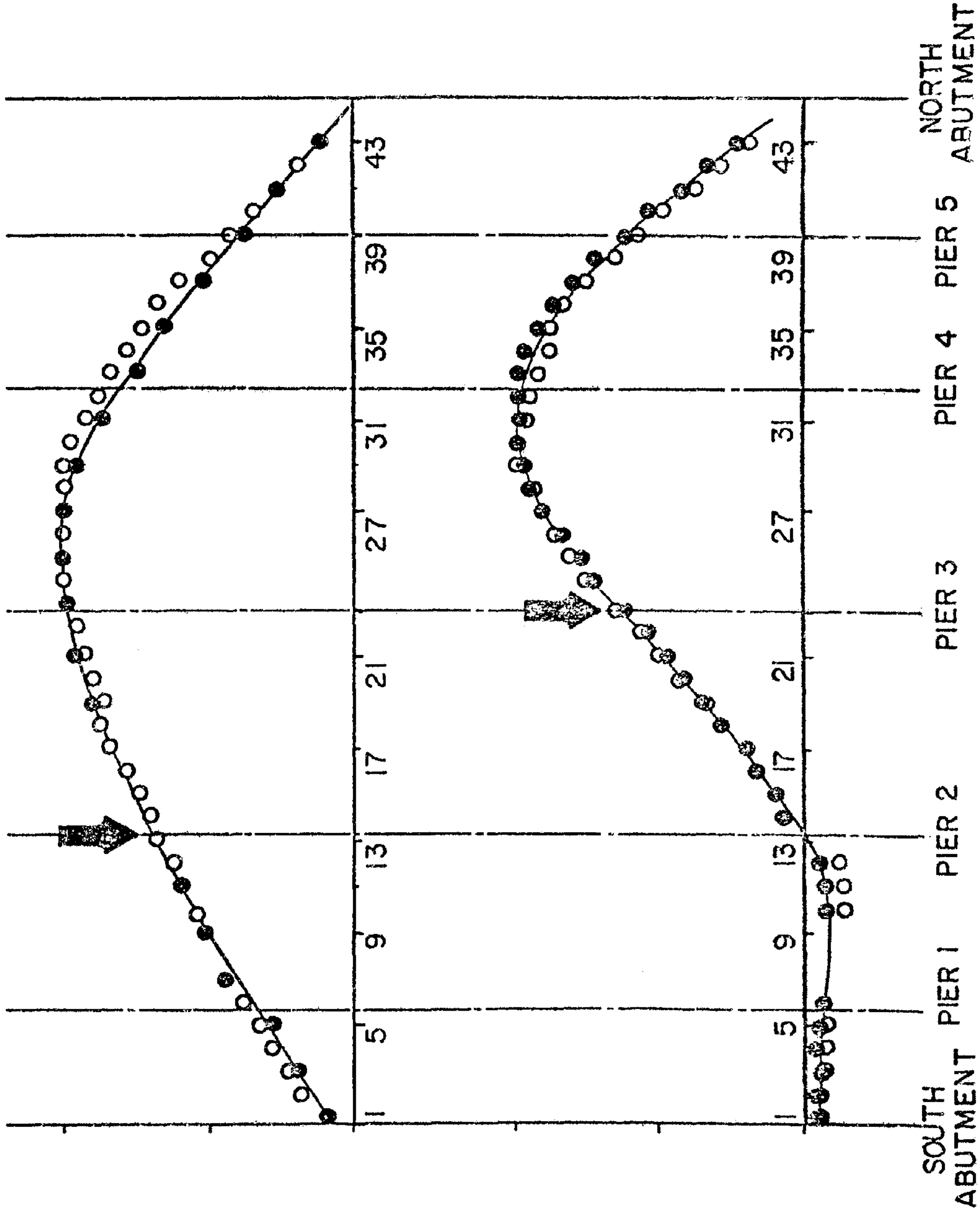

Fig. 2. Fundamental Mode Shapes - The Upper Curve Is The 2.08 hz Mode and the Lower Curve is the 2.32 hz Mode.

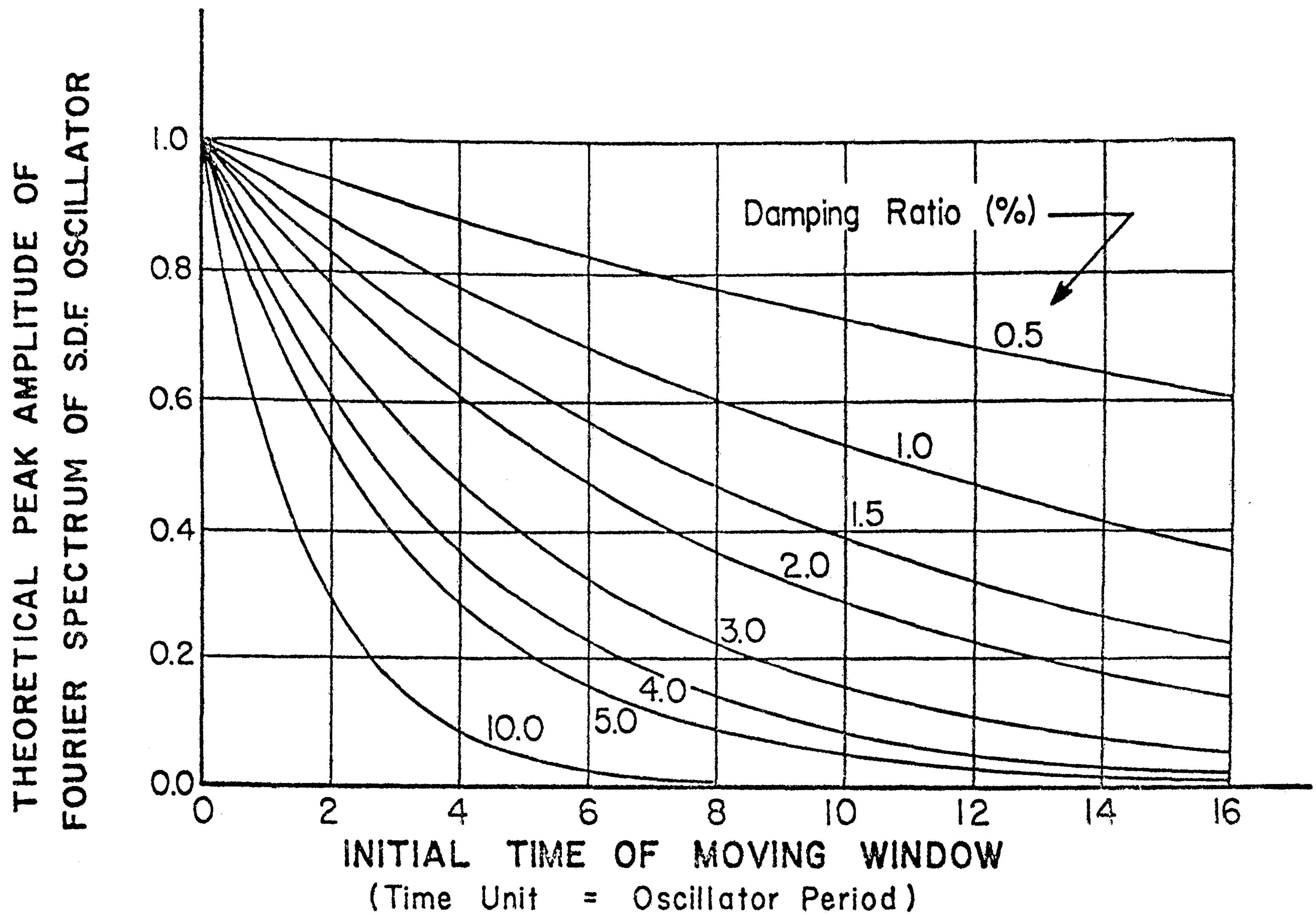

Fig. 3. Theoretical Moving Window Spectral Decay Curves.

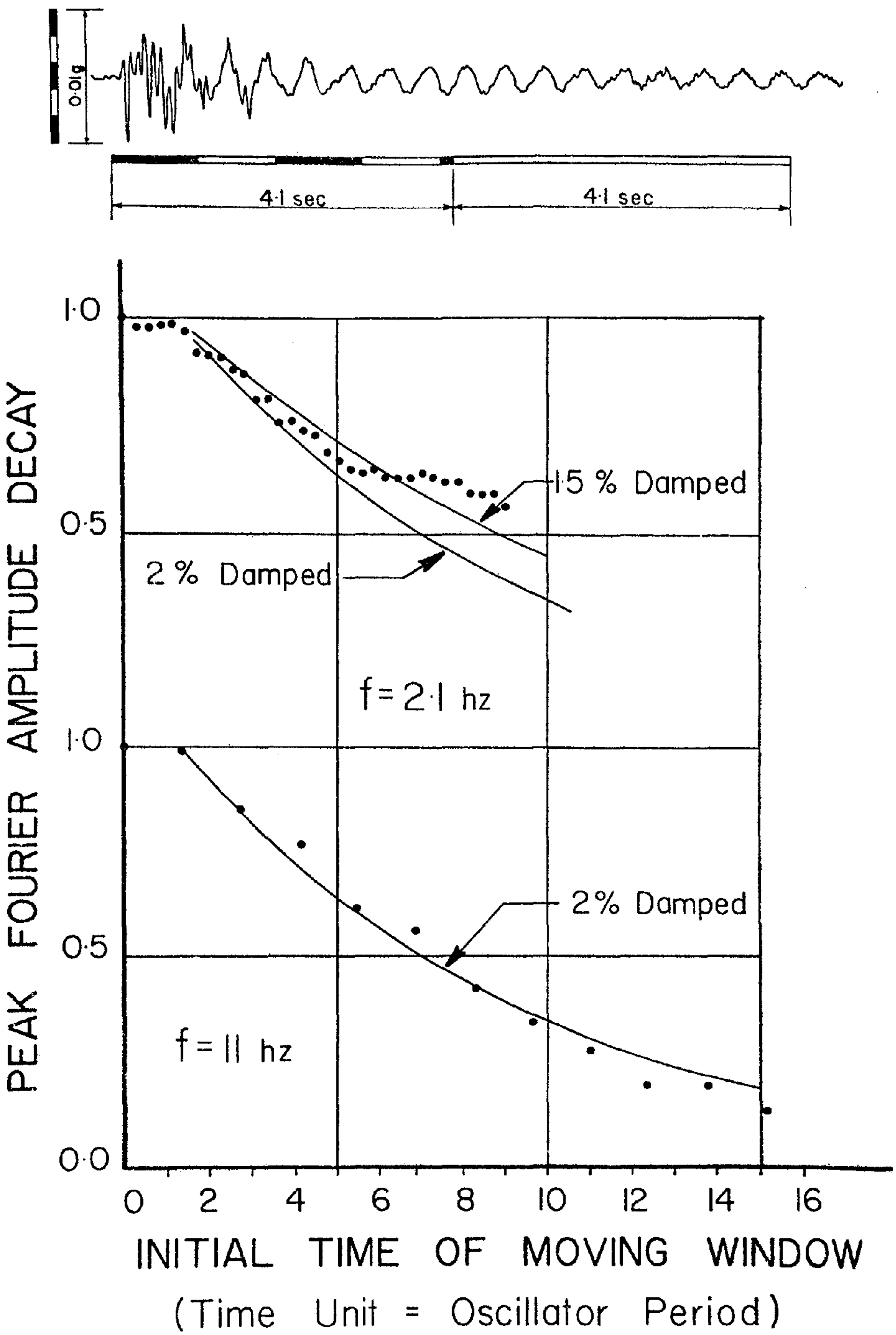

Fig. 4. Typical Quick Release Accelerogram and Associated Spectral Decay Curves.

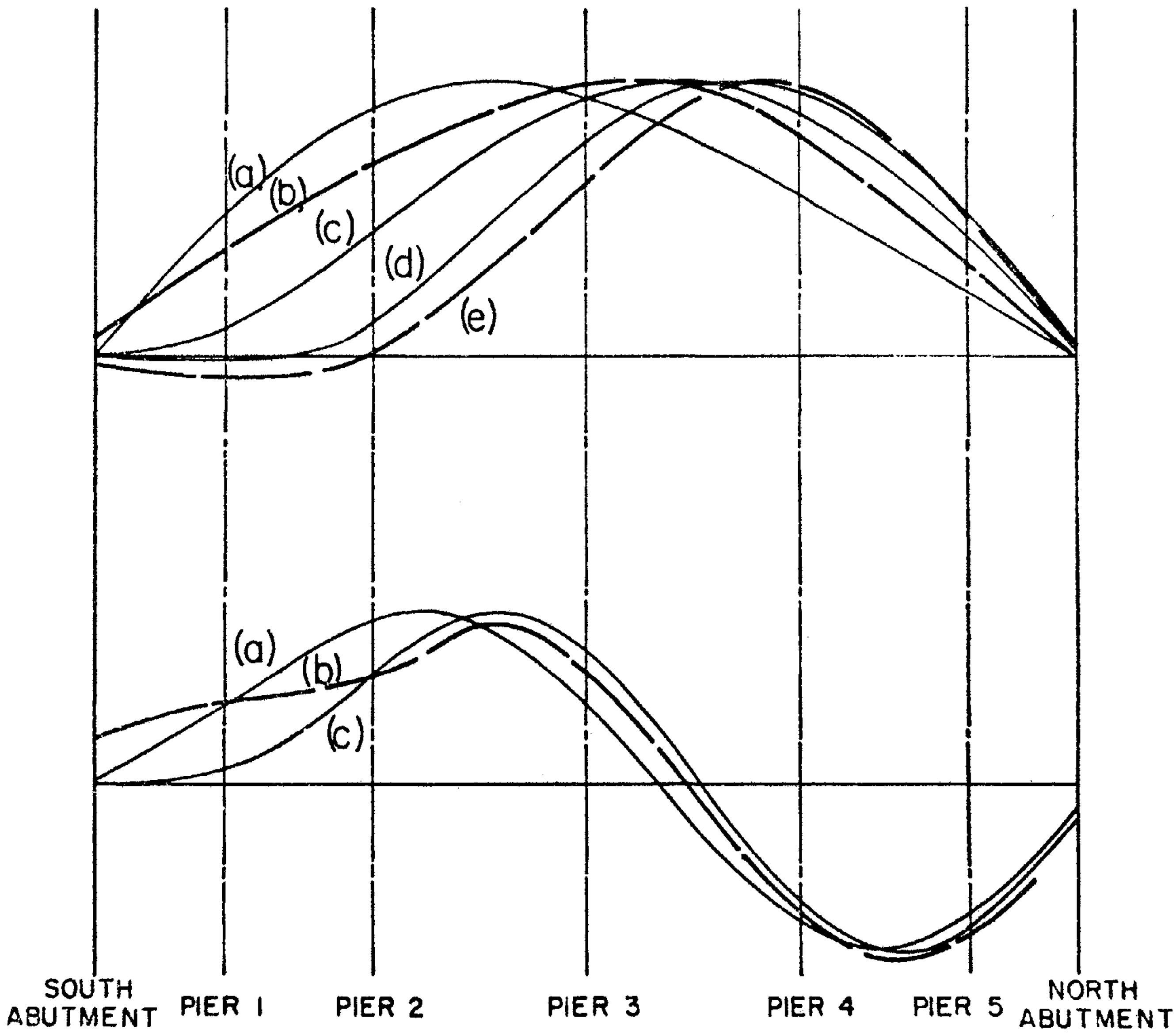

Fig. 5. Theoretical Modes Compared to Experimental Modes.

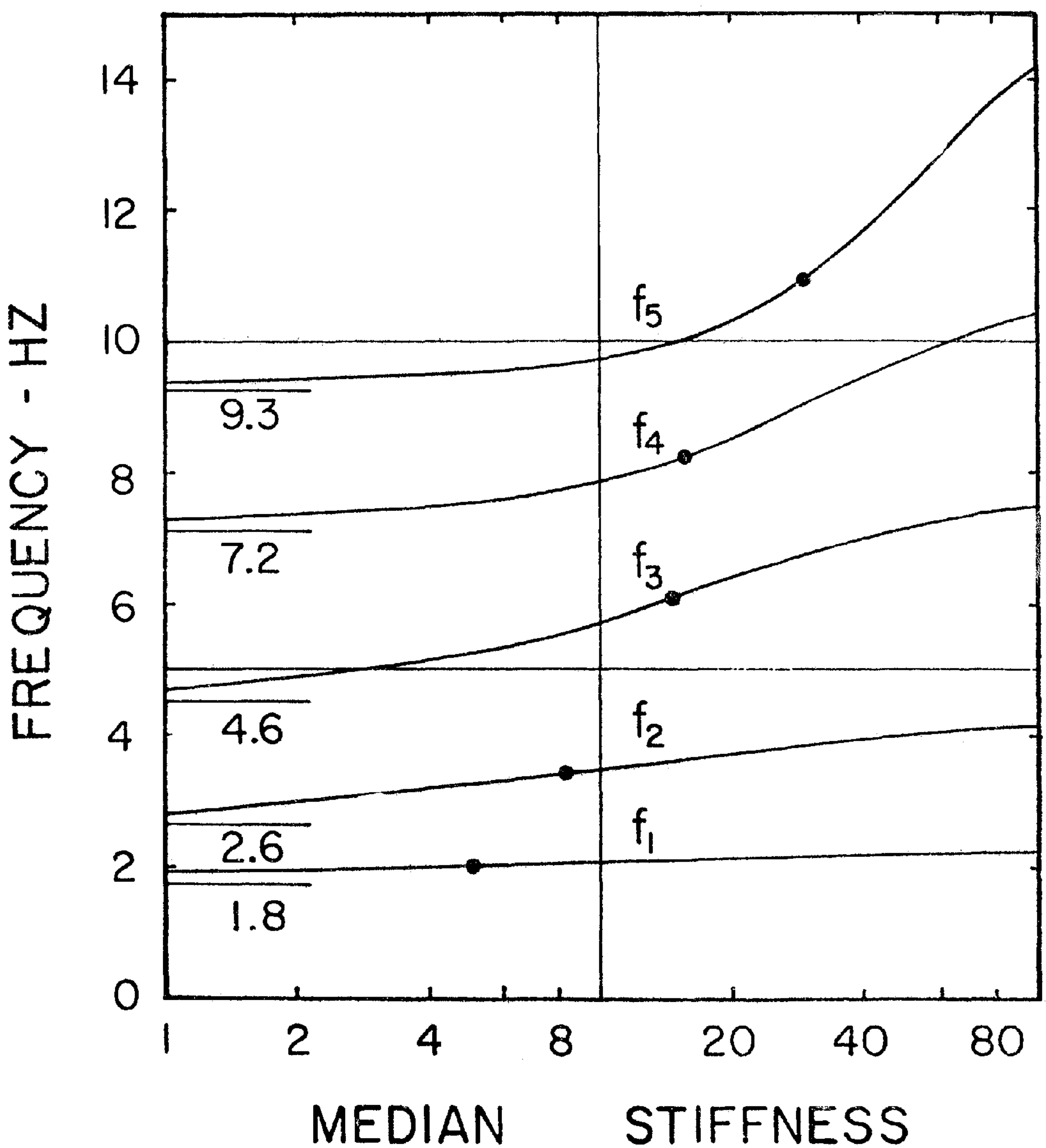

Fig. 6. Natural Frequencies as a Function of Median Restraint

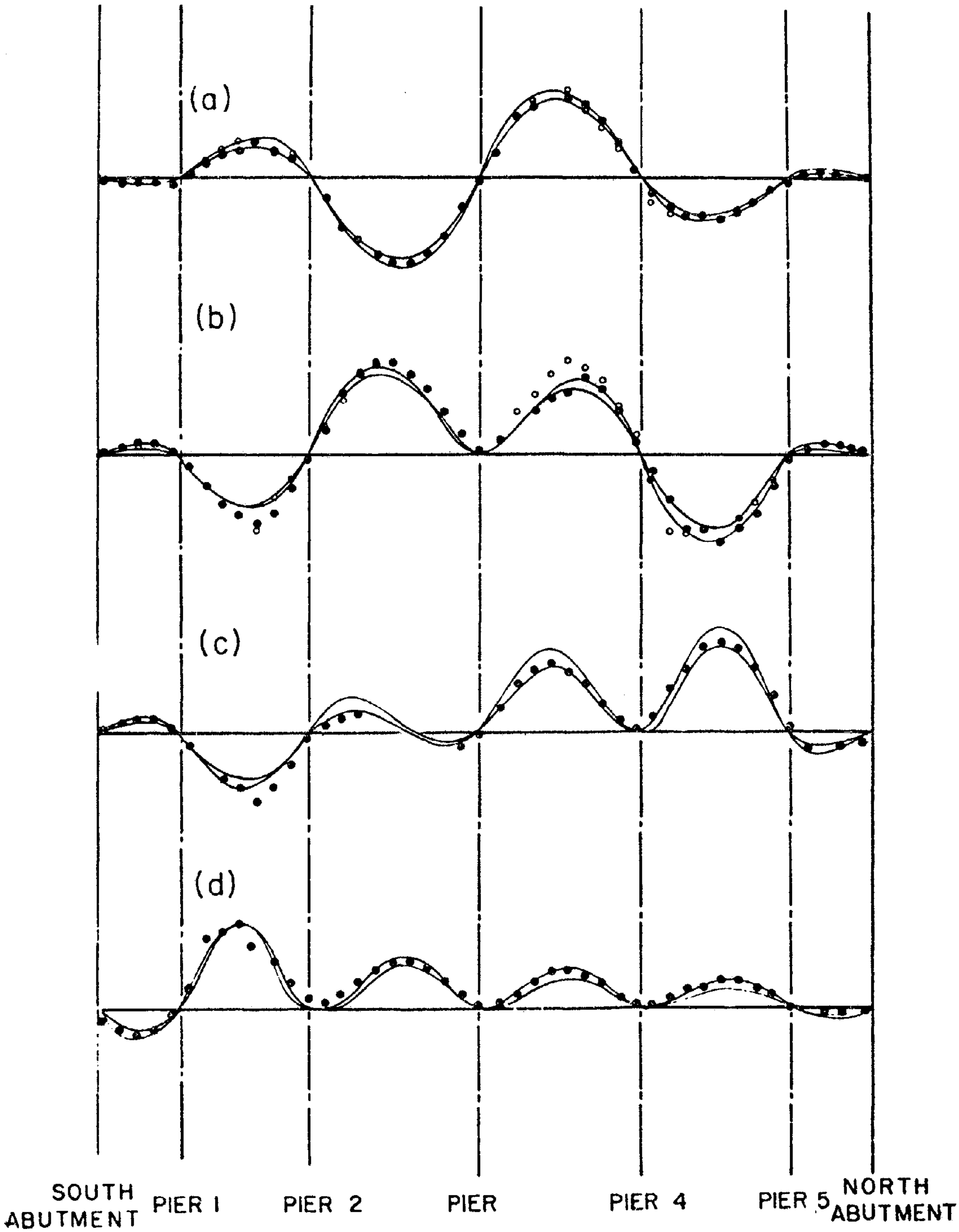

Fig. 7. Vertical Mode Shapes

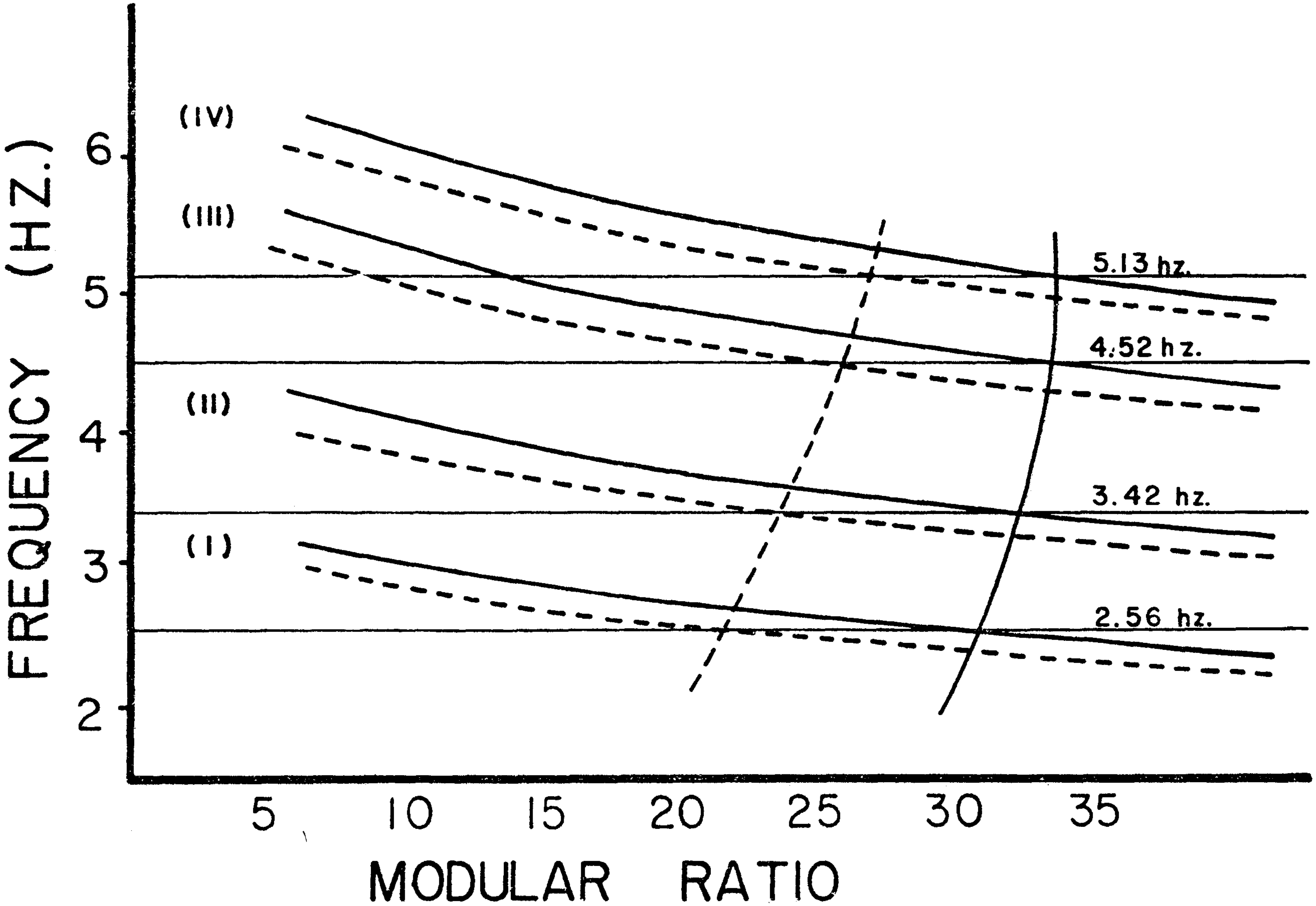

Fig. 8. Vertical Natural Frequencies Versus Modular Ratio

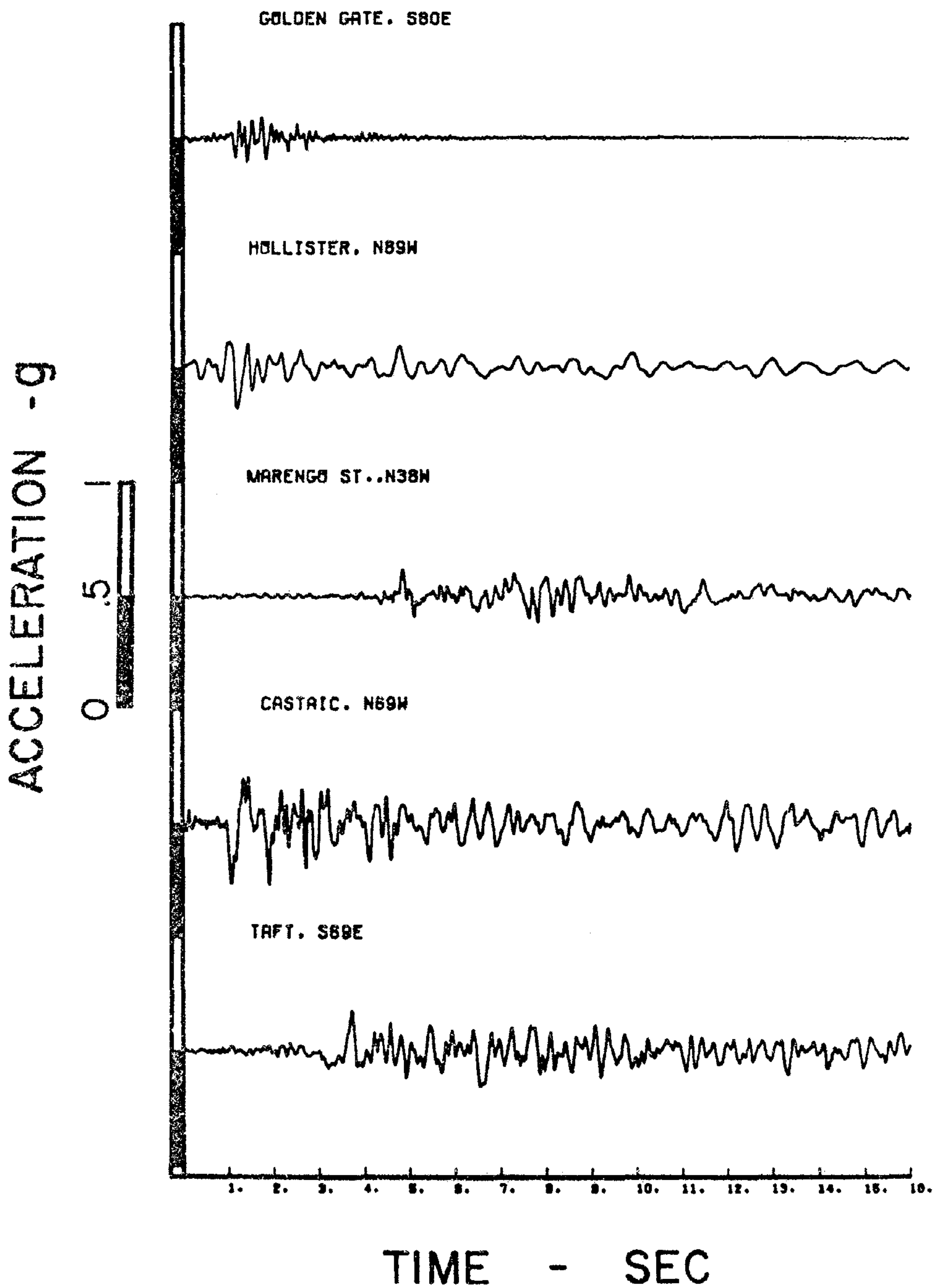

Fig. 9. Accelerograms Used for Earthquake Response Calculations

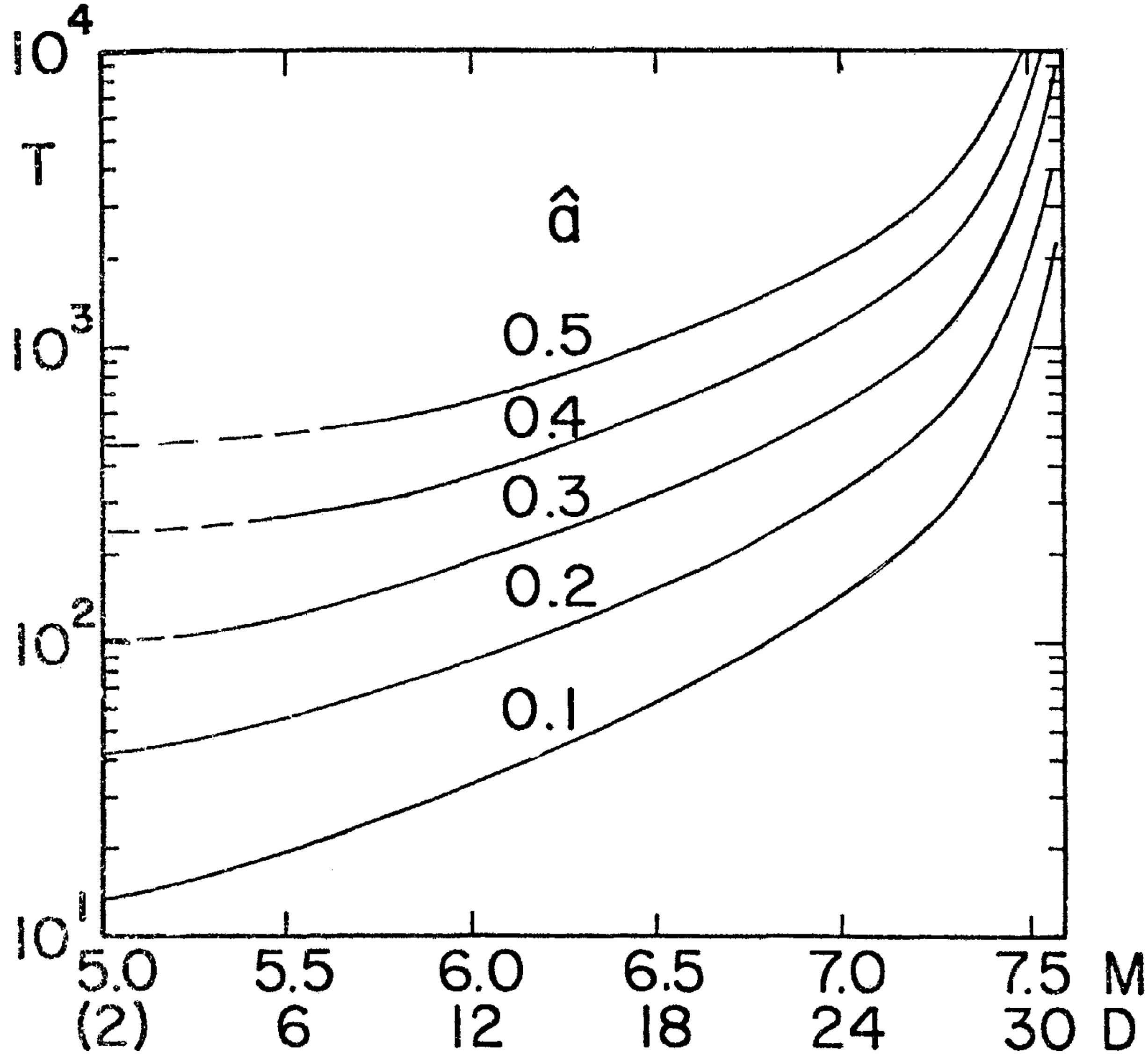

Fig. 10. Return Period For Peak Acceleration Versus Causal Earthquake Magnitude From Reference 11.

TRANSVERSE STATIC RESPONSE (300#/ft. Wind)

Pier No.	H (in)	W (in)	M_B (Kip·in)	M_T (Kip·in)	F (Kip)	R_{DL} (Kip)	R (Kip)	Δ (in)
S. Abut.					18.6	71.3		.0093
Pier 1	362	168	875.0	891.0	9.76	256.0	10.6	.0976
Pier 2	373	168	1636.0	1665.0	17.7	406.0	19.8	.1915
Pier 3	360	168	3235.0	3119.0	35.3	456.0	37.1	.1960
Pier 4	341	168	2596.0	2502.0	29.9	413.0	29.8	.1439
Pier 5	297	168	748.0	767.0	10.2	308.0	9.13	.0599
N.Abut.					13.0	29.5		.0065

1940 ≡ Total DL Superstructure

LONGITUDINAL STATIC RESPONSE (4% EQ)

Pier No.	M_B (Kip·in)	F (Kip)	Δ (inches)
Pier 3	6624.0	36.8	.652
Pier 4	7351.0	43.24	.652

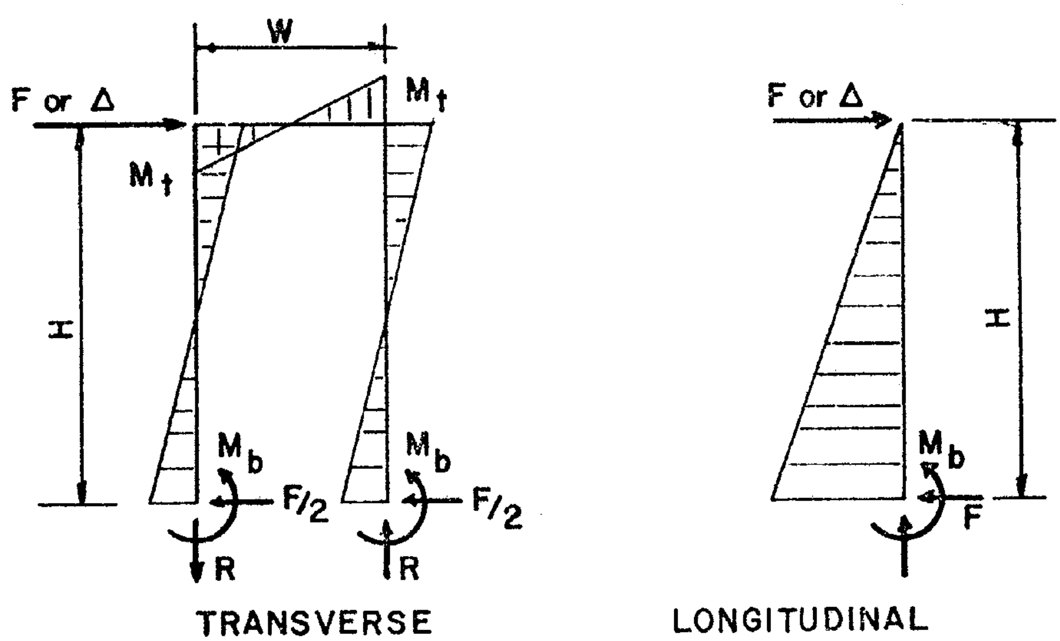

TABLE 2. Static Response Results

GROUND MOTION DATA

1. Description: 1957 San Francsico Earthquake (Golden Gate S80E)

   Magnitude = 5.3 — A_{max} = .105 g
   Distance to Fault = 7 mi. — V_{max} = 1.81 in/sec.
   Soil Type = (1) Hard — D_{max} = .31 in.
   Return Period 14 to 16 years.

2. Description: 1961 Hollister Earthquake (City Hall N89W)

   Magnitude = 5.6 — A_{max} = .18 g
   Distance to Fault = 9 mi. — V_{max} = 6.73 in/sec.
   Soil Type = (0) Soft — D_{max} = 1.49 in.
   Return Period 35 to 50 years.

3. Description: 1971 San Fernando Earthquake (First Floor 1640 Marengo Street, N38W)

   Magnitude = 6.6 — A_{max} = .12 g
   Distance to Fault = 20 mi. — V_{max} = 6.34 in/sec.
   Soil Type = (0) Soft — D_{max} = 4.72 in.
   Return Period 16 to 87 years.

4. Description: 1971 San Fernando Earthquake (Castaic N69W)

   Magnitude = 6.6 — A_{max} = .27 g
   Distance to Fault = 18 mi. — V_{max} = 10.7 in/sec.
   Soil Type = (1) Hard — D_{max} = 3.66 in.
   Return Period 80 to 280 years.

5. Description: 1952 Kern County Earthquake (Taft S69E)

   Magnitude = 7.7 — A_{max} = .18 g
   Distance to Fault = 30 mi. — V_{max} = 6.96 in/sec.
   Soil Type = (0) Soft — D_{max} = 3.62 in.
   Return Period 34 to 400 years.

TABLE 3

GROUND MOTION DATA

Description:

Magnitude = 6.4 A_{max} = .17 g

Distance to Fault = 17 mi. V_{max} = 6.5 in/sec.

Soil Type = (0) Soft D_{max} = 2.8 in

Return Period 32 to 100 years

In Western Nevada

TRANSVERSE RESPONSE PARAMETERS

Pier	F_d Ratio		M_b Ratio		Δ & F Ratio		Δ (inches)		Time
	2%	5%	2%	5%	2%	5%	2%	5%	
S.Abut.	7.1	4.9			5.8	4.1	0.54	.038	3.2
Pier 1	2.2	1.5	.69	.47	7.5	5.1	.72	.50	3.6
Pier 2	4.0	2.8	1.3	.91	7.6	5.3	1.5	1.0	3.6
Pier 3	7.8	5.4	1.9	1.3	7.5	5.2	1.5	1.0	3.5
Pier 4	5.6	3.9	1.3	.91	6.4	4.4	.91	.63	2.7
Pier 5	1.8	1.2	.46	.30	5.8	3.9	.35	.23	2.8
N.Abut.	4.0	3.1			4.8	3.7	.031	.024	3.1

LONGITUDINAL RESPONSE PARAMETERS

Pier	F Ratio		M_b Ratio		Δ (in)	
	2%	5%	2%	5%	2%	5%
Pier 3	4.2	3.4	2.3	1.8	2.8	2.2
Pier 4	4.2	3.4	2.5	2.0	2.8	2.2

Table 4. Average Response Results

EFFECTS OF TRAVELING SEISMIC WAVES ON THE RESPONSE OF BRIDGES

by

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L.C. Lee, Member of Technical Staff
Agbabian Associates, El Segundo, California

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ABSTRACT

A new methodology for analyzing the three-dimensional response of soil/structure systems to traveling seismic waves is described and used to analyze a single-span bridge subjected to incident plane SH-waves. The analysis results demonstrate the importance of traveling wave effects and show how the excitation frequency and direction of incidence of the seismic waves influence the three-dimensional response characteristics of this bridge/soil system.

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INTRODUCTION

Prior experience from earthquakes in the United States and throughout the world shows numerous instances of bridge damage that have resulted in significant hazards, disruptions, and costs to the surrounding population. This underscores the need for improved bridge design procedures and for further research directed toward increasing the earthquake resistance of bridges.

One aspect of the earthquake problem that can be of particular importance for bridges is the nature of the seismic excitations applied to the bridge along its length. Typically, the seismic design of bridges is based on the assumption that these excitations are identical at all bridge foundations. However, this assumption is only approximate, since it does not account for the spatial variations of the incident seismic waves. These spatial variations cause the various foundations along the length of the bridge to be subjected to excitations that differ in both amplitude and phase. Such excitations can have an important effect on the bridge response.

The influence of traveling seismic waves has been studied not only for bridges [1,5,9], but for several other types of structures as well, such as rigid footings [12,15], conventional buildings [14,17], buried pipelines and tunnels [8,11], earth dams [6,10], and nuclear power plants [13,19]. These investigations have provided insights into traveling wave effects and, in addition, have pointed out where deficiencies in the understanding of such effects still exist. The main insights are that (a) traveling wave effects become pronounced when the wavelengths of the incident waves are comparable to or less than a characteristic length of the structure or foundation; (b) the net translational excitation of shallow foundation elements caused by nonvertically incident P- and S-waves can be reduced relative to the excitations caused by vertically incident waves; (c) nonvertically incident SH-waves lead to torsional excitation of the structure and nonvertically incident P- and SV-waves lead to rocking excitation; and (d) traveling Rayleigh waves may excite all six components of response, depending on their direction of incidence relative to the structure. The primary deficiencies that still exist are related to (a) the lack of suitable recorded strong-motion data necessary to guide the specification of spatially varying input motions for seismic response analyses; and (b) the lack of available engineering guidelines for assessing the behavior of structures subjected to traveling seismic waves.

This paper presents results from the first phase of an ongoing research program directed toward the second deficiency noted above--i.e., the development of engineering guidelines for assessing the effects of traveling waves on the response of structures. The paper features an analysis of the three-dimensional dynamic response of a single-span bridge structure subjected to incident plane SH-waves and provides insights into the fundamental traveling wave effects that result for this simple bridge configuration and seismic excitation. In addition, the paper summarizes a new methodology that has been developed for carrying out such analyses for any aboveground elastic structure configuration. Detailed descriptions of this bridge analysis and the methodology itself are contained in a report [18] from which this paper is extracted.

METHODOLOGY

The methodology presented here has been named CAST1* and has the following features (see Fig. 1).

a. It computes the three-dimensional dynamic response of an arbitrarily configured, elastic, aboveground structure. It can also consider two or more closely spaced structures.

b. It assumes each structure to be supported on any number of rigid foundations of arbitrary shape that are bonded to the surface of an elastic half-space.

c. It represents input motions as any desired combination of harmonic body and/or surface waves with arbitrary excitation frequencies, amplitudes, and angles of incidence.

CAST1 is a first step in the development of a more general analysis procedure that will involve viscoelastic and horizontally layered soil media, embedded foundations, and arbitrary transient input motions. The manner in which this methodology represents the superstructure and the foundation/soil system and then performs the overall system response analysis is summarized below.

Representation of Superstructure

CAST1 uses a three-dimensional finite element model to represent the superstructure. The model can comprise any combination of the various element types shown in Figure 2 [4], and is used to define the stiffness matrix, mass matrix, and fixed-base mode shapes and frequencies of the superstructure. Either a

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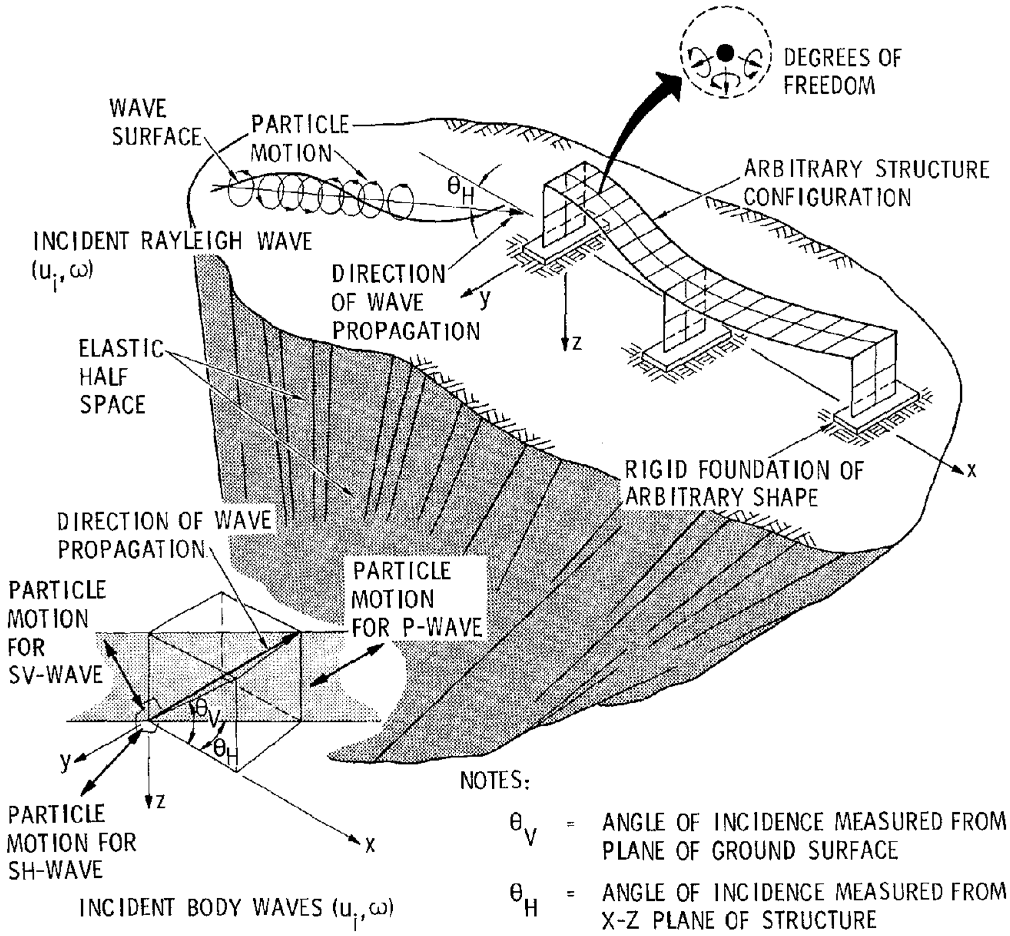
*The acronym CAST1 stands for Continuum and Arbitrary Structure Subjected to Traveling Waves, Version 1.



NOTES:

θ_V = ANGLE OF INCIDENCE MEASURED FROM PLANE OF GROUND SURFACE

θ_H = ANGLE OF INCIDENCE MEASURED FROM X-Z PLANE OF STRUCTURE

u_i, ω = AMPLITUDE AND CIRCULAR FREQUENCY OF INCIDENT WAVE

x, y, z = SYSTEM COORDINATE AXES

FIGURE 1. SOIL/STRUCTURE SYSTEM CONSIDERED BY CAST1

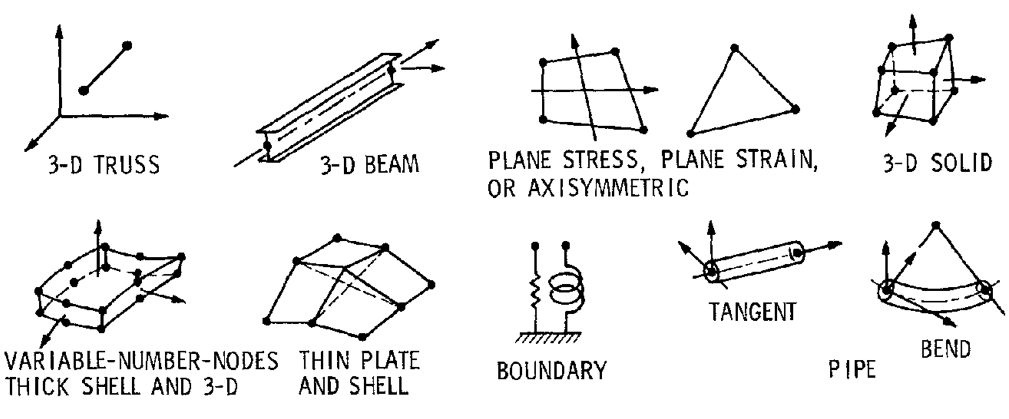


FIGURE 2. SUPERSTRUCTURE ELEMENT TYPES [4]

determinant-search method or a subspace-iteration approach can be used to carry out the mode shape and frequency calculations [2,3]. Damping in the superstructure is represented by a Rayleigh damping matrix.

Representation of Foundation Soil/System

Foundation/soil interaction effects under the action of the incident-wave motions are represented by using a continuum solution based on the work of Wong [20]. This solution characterizes the foundation/soil system in terms of complex, frequency-dependent driving force vectors and impedance matrices. The driving forces correspond to the reaction forces that result when each foundation is fixed and subjected to the incident waves transmitted through the soil medium (Fig. 3a). The elements of the jth column of the impedance matrix are computed as foundation reaction forces caused by a unit harmonic displacement of the jth foundation degree of freedom when all other foundation degrees of freedom are fixed (Fig. 3b). These quantities are derived for one or more foundations of arbitrary shape by first using Green's functions for an elastic half-space to define stress/displacement relationships for various subregions of each foundation, and then by imposing rigid-body displacement boundary conditions and equilibrium requirements [18].

System Response Analysis

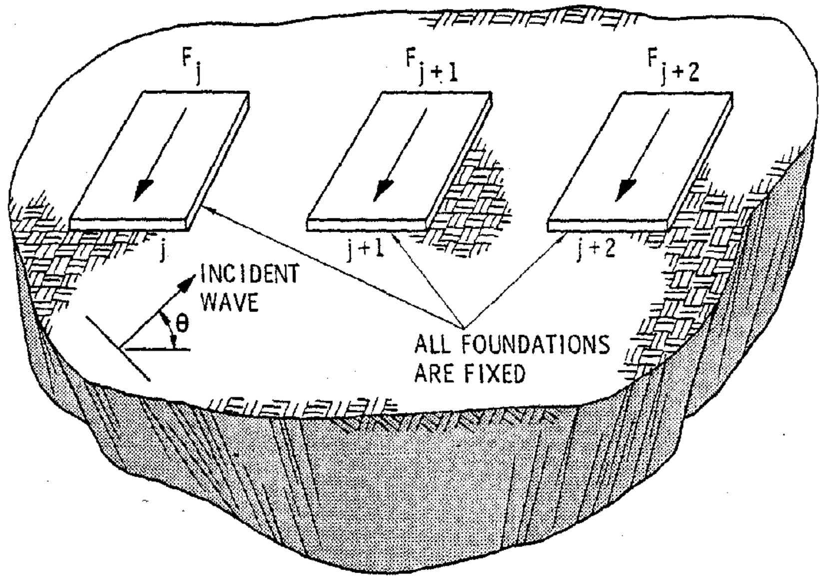
Once the superstructure and foundation/soil systems are characterized as described above, compatibility and equilibrium requirements at the superstructure/foundation interface are used to couple these two sets of results and to thereby represent the complete soil/foundation/superstructure system [16]. The steady-state response of this system is then computed using an extension of a procedure described by Clough and Penzien [7]. A formulation of this analysis procedure is provided in [18].

RESPONSE OF A SINGLE-SPAN BRIDGE TO INCIDENT SH-WAVES

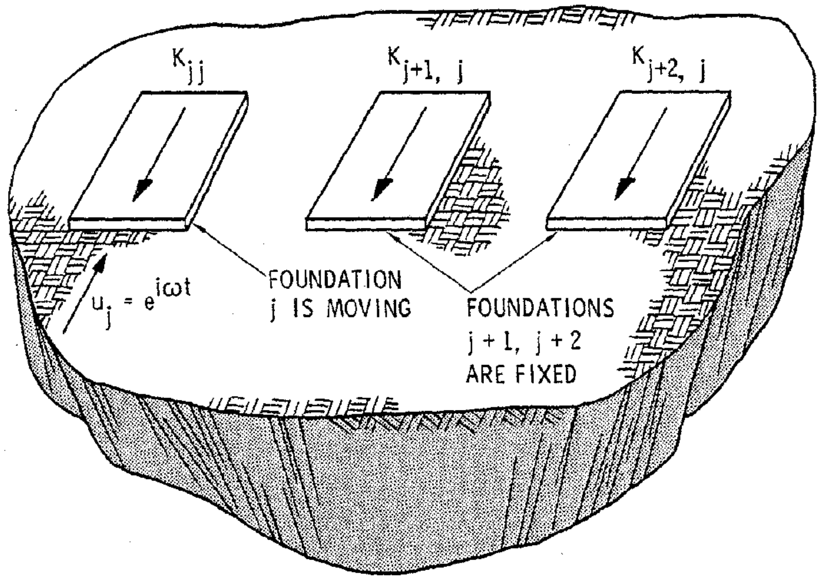
The foregoing methodology is used to compute the three-dimensional response of a single-span bridge resting on the surface of an elastic half-space and subjected to incident plane SH-waves. Parametric analyses are carried out to show how the bridge response is influenced by the excitation frequency and the angles of incidence of the incident wave.

Bridge Model and Excitation

The bridge configuration for this example is shown in Figure 4a to be 120 ft (36.5 m) long, 70 ft (21.5 m) wide, and 20 ft (6.1 m) high. The bridge is modeled using the system of undamped beam elements shown in Figure 4b. Table 1 furnishes section and material properties for the bridge and material properties for the soil medium.

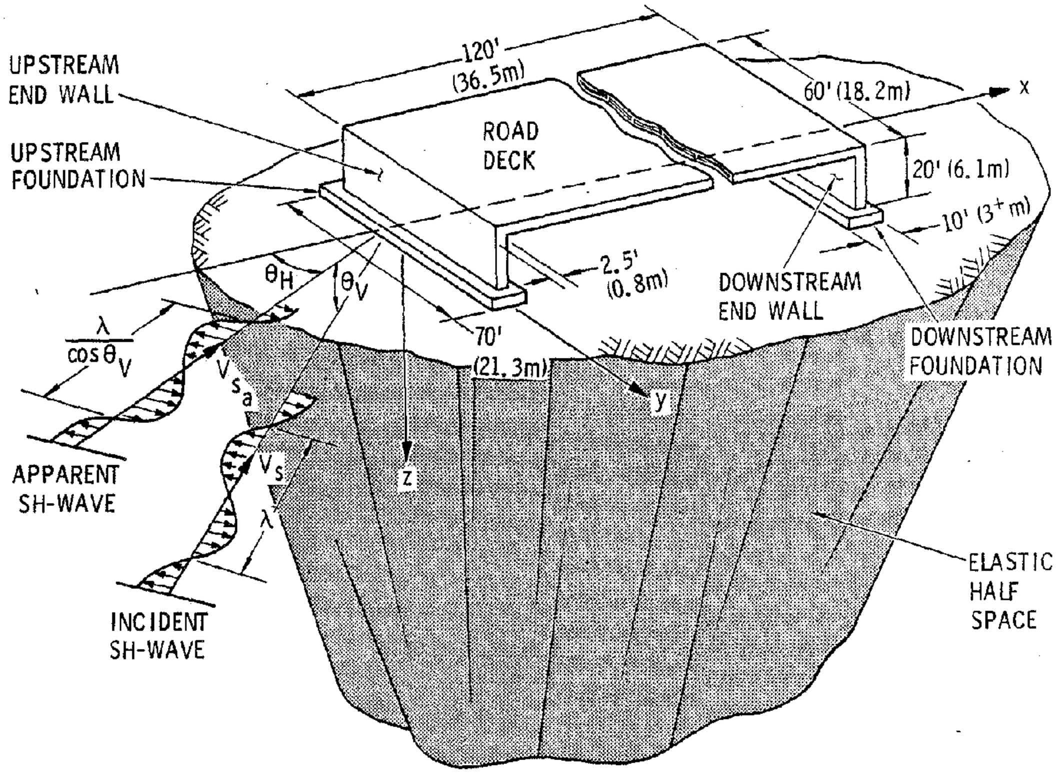


(a) Development of foundation driving forces
(only longitudinal forces shown)

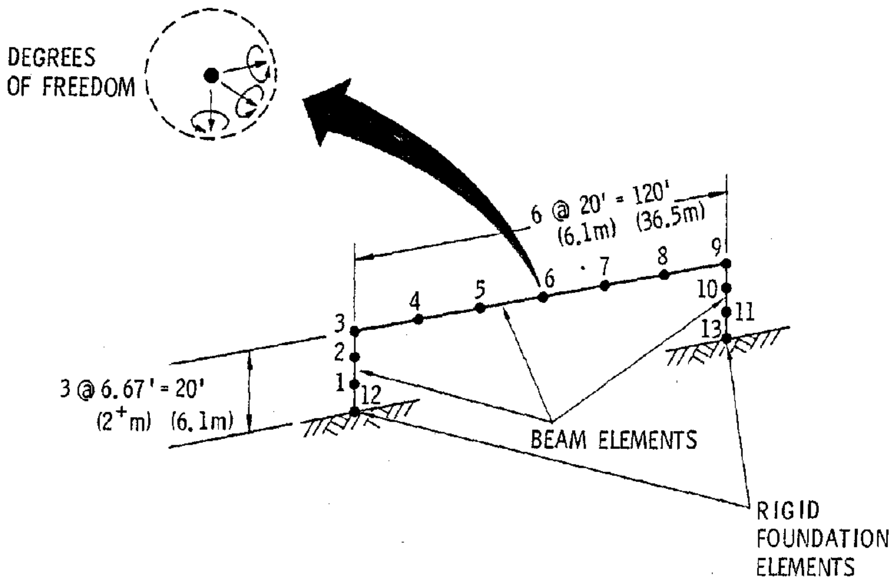


(b) Development of jth column of soil/foundation impedance matrix (only longitudinal forces shown)

FIGURE 3. DEVELOPMENT OF FOUNDATION/SOIL DRIVING FORCES AND IMPEDANCE MATRIX [20]



(a) System configuration



(b) Bridge model

FIGURE 4. BRIDGE CONFIGURATION AND MODEL USED IN EXAMPLE ANALYSIS

TABLE 1. SECTION PROPERTIES AND MATERIAL PROPERTIES CONSIDERED IN BRIDGE RESPONSE ANALYSIS

(a) Superstructure section properties

Element	Cross-Sectional Area, in feet^2 (meters^2)	Moment of Inertia, in feet^4 (meters^4)		
		About Strong Axis	About Weak Axis	Torsion
Road Deck	0.98×10^2 (9.1)	3.56×10^4 (306.9)	3.29×10^2 (2.8)	1.01×10^3 (8.7)
End Walls	1.48×10^2 (13.7)	4.28×10^4 (369.4)	0.77×10^2 (0.7)	0.31×10^3 (2.7)

(b) Material properties

Element	Shear Wave Velocity, in feet (meters) per second	Unit Weight, in pounds per foot^3 (kilograms per meter^3)	Poisson's Ratio
Elastic Half-Space	500 (150)	110 (1760)	0.33
Superstructure	6900 (2100)	150 (2400)	0.15

The free-field excitations from the incident SH-waves have a surface amplitude of 2.0, an arbitrary excitation frequency (up to 25 Hz maximum), and a zero phase angle at the upstream foundation, which is the origin of the coordinate system for these analyses. The orientation of these excitations and the direction of wave propagation are represented by the two angles of incidence, θ_H and θ_V, that are shown in Figure 4a. Five different combinations of these angles are used to define five different excitation cases for which the bridge response is analyzed. These cases, listed in Table 2, were selected from a more extensive set described in [18] because they illustrate some of the more interesting features of the three-dimensional bridge response.

For each excitation case, bridge motions are presented in the form of response amplitudes vs. dimensionless frequency plots and as three-dimensional plots of the deformed shape of the bridge at times of peak cyclic response. The dimensionless frequencies used in presenting these results are defined as

$$R_L = \frac{\ell}{\lambda} = \frac{\ell \omega}{2 \pi V_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where λ is the wavelength of the incident wave along its propagation path; V_S is the shear wave velocity of elastic half-space; ω is the circular frequency of the excitation; and ℓ is a characteristic structural dimension that is selected according to the orientation of the propagation path of the incident wave, as defined by θ_H. The selection of ℓ for the three values of θ_H considered in these example analyses is shown in Table 3.

TABLE 2. EXCITATION CASES FOR BRIDGE RESPONSE ANALYSIS

Case No.	θ_H, in degrees (radians)	θ_V, in degrees (radians)	Description of Incident SH-Wave Motions
1	90 (1.57)	90 (1.57)	Vertically incident waves with particle motions directed along span of bridge.
2	90 (1.57)	0 (0)	Horizontally incident waves propagating in y-direction, with particle motions directed along span of bridge.
3	0 (0)	90 (1.57)	Vertically incident waves with particle motions perpendicular to span of bridge.
4	0 (0)	0 (0)	Horizontally incident waves propagating in x-direction, with particle motions perpendicular to span of bridge.
5	45 (0.79)	45 (0.79)	Waves traveling at angle of 45 deg to ground surface and in plane oriented at 45 deg to span of bridge. Particle motion directed at angle of 45 deg relative to x- and y-axes.

NOTE: For each case, excitations have a surface amplitude of 2.0, a variable frequency (up to 25 Hz maximum), and a zero phase angle at the upstream foundation.

TABLE 3. DIMENSIONLESS FREQUENCY DEFINITIONS

Angle of Incidence, θ_H, in degrees (radians)	Orientation of Incident Wave Propagation Path	Definition of ℓ (Eq. 1)		Dimensionless Frequency, R_L (Eq. 1)
		Description	Numerical Valve (Fig. 4a)	
0 (0)	Within vertical plane parallel to x-z plane of bridge	Distance between the two bridge foundations	120 ft (36.6 m)	$R_{L_x} = \frac{120 \text{ ft} \cdot \omega}{2\pi V_s}$
90 (1.57)	Within vertical plane normal to x-z plane of bridge (or parallel to y-z plane)	Length of foundations along y-axis	70 ft (21.3 m)	$R_{L_y} = \frac{70 \text{ ft} \cdot \omega}{2\pi V_s}$
45 (0.79)	Within vertical plane oriented at 45 deg (0.79 rad) to x-z plane	Same as for θ_H = 0 deg and 90 deg (0 rad and 1.57 rad)		R_{L_x} and R_{L_y}

where λ is the wavelength of the incident wave along its propagation path; V_S is the shear wave velocity of elastic half-space; ω is the circular frequency of the excitation; and ℓ is a characteristic structural dimension that is selected according to the orientation of the propagation path of the incident wave, as defined by θ_H. The selection of ℓ for the three values of θ_H considered in these example analyses is shown in Table 3.

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1	90 (1.57)	90 (1.57)	Vertically incident waves with particle motions directed along span of bridge.
2	90 (1.57)	0 (0)	Horizontally incident waves propagating in y-direction, with particle motions directed along span of bridge.
3	0 (0)	90 (1.57)	Vertically incident waves with particle motions perpendicular to span of bridge.
4	0 (0)	0 (0)	Horizontally incident waves propagating in x-direction, with particle motions perpendicular to span of bridge.
5	45 (0.79)	45 (0.79)	Waves traveling at angle of 45 deg to ground surface and in plane oriented at 45 deg to span of bridge. Particle motion directed at angle of 45 deg relative to x- and y-axes.

NOTE: For each case, excitations have a surface amplitude of 2.0, a variable frequency (up to 25 Hz maximum), and a zero phase angle at the upstream foundation.

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90 (1.57)	Within vertical plane normal to x-z plane of bridge (or parallel to y-z plane)	Length of foundations along y-axis	70 ft (21.3 m)	$R_{L_y} = \frac{70\text{ ft} \cdot \omega}{2\pi V_s}$
45 (0.79)	Within vertical plane oriented at 45 deg (0.79 rad) to x-z plane	Same as for θ_H = 0 deg and 90 deg (0 rad and 1.57 rad)		R_{L_x} and R_{L_y}

Fixed-Base Mode Shapes and Frequencies

The fixed-base mode shapes and frequencies for the bridge are shown in Figure 5. This figure shows that the out-of-plane modes have significantly higher frequencies than do the corresponding in-plane modes, a direct result of the greater stiffness of the bridge in the y-direction. A total of 29 modes was used to characterize the superstructure to provide adequate convergence of the response computations within the range of excitation frequencies considered in this analysis (up to 25 Hz).

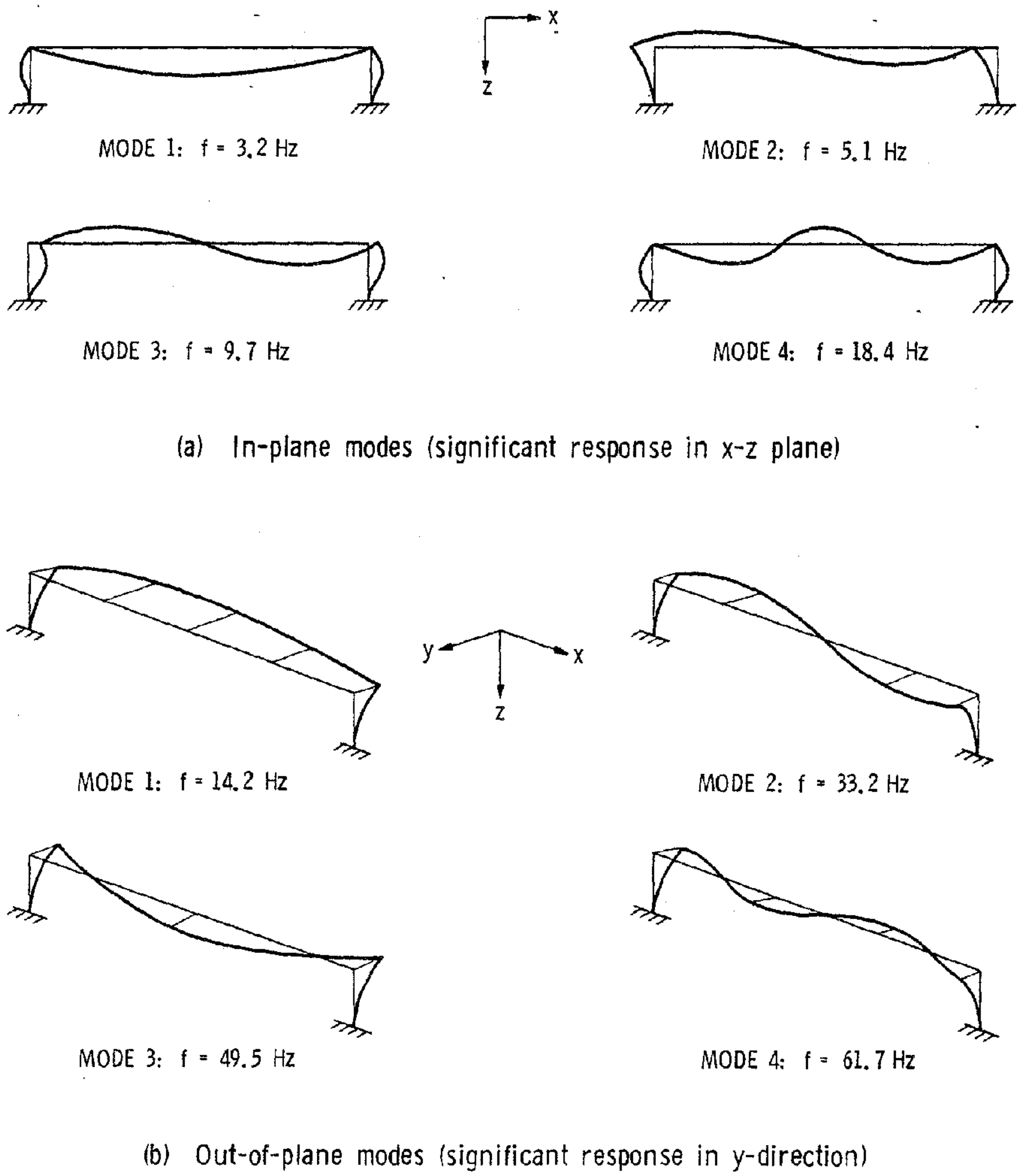
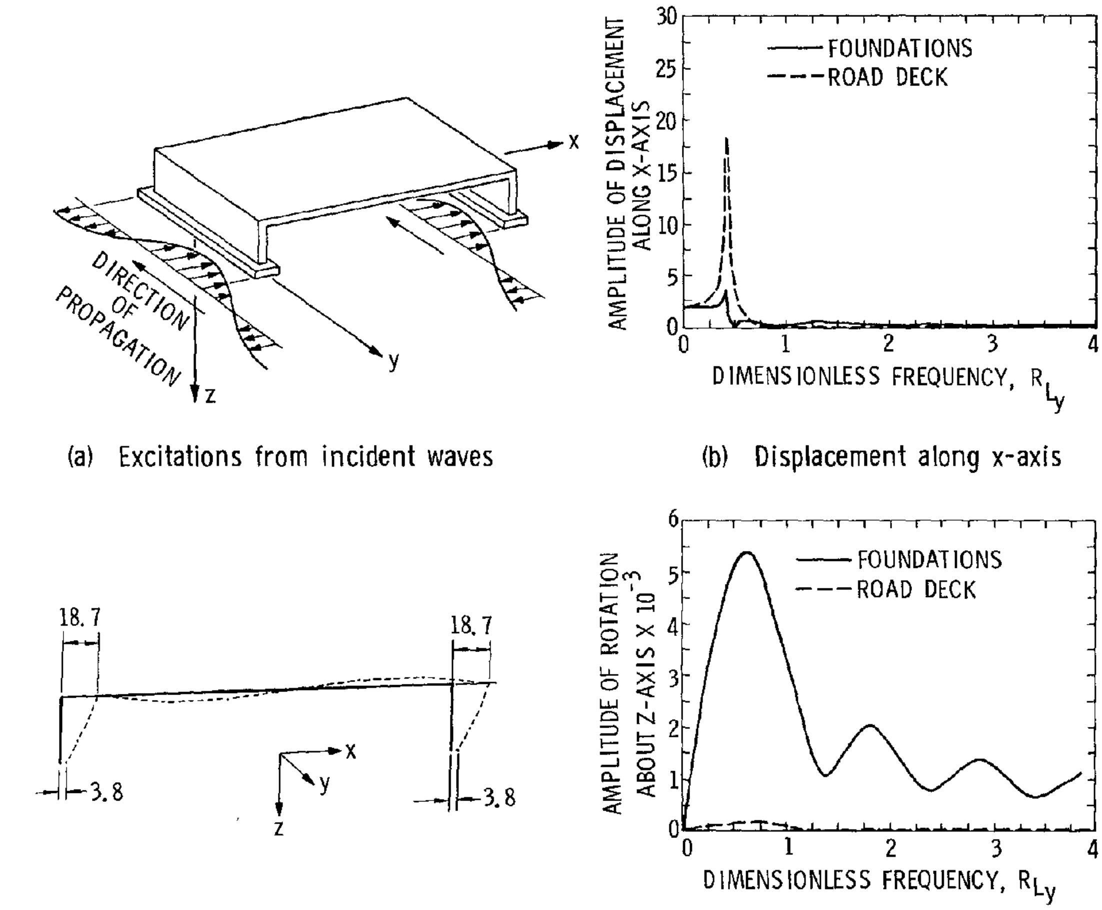


FIGURE 5. FIXED-BASE MODE SHAPES AND FREQUENCIES OF BRIDGE

very small displacements of the road deck in the x-direction and by displacements of the foundations that are similar in amplitude and phase to those of the free field [18]. No foundation rotations about the z-axis are generated by this excitation because of the complete symmetry of the bridge and the excitation with respect to the x-axis. This response, which is the simplest of the various cases considered, is analogous to that of a simple single-degree-of-freedom system whose natural frequency corresponds to this resonant frequency of the bridge/soil system.

Case 2: θ_H = 90 deg (1.57 rad), θ_V = 0 deg (0 rad)--The second case considers SH-waves with particle motions oriented in the same direction as for Case 1 (in the x-direction, or along the bridge span); however, the waves are now horizontally incident, rather than vertically incident, with a propagation path in the y-direction (i.e., normal to the span of the bridge). As a result, the excitations are no longer uniform over the entire length of the two foundations; instead, they exhibit spatial variations as the wave propagates in the y-direction (Fig. 7a).



(a) Excitations from incident waves

(b) Displacement along x-axis

(c) Deformed shape at time of peak response (R_{L_y} = 0.412)

(d) Rotation about z-axis

FIGURE 7. CASE 2: BRIDGE RESPONSE TO INCIDENT SH-WAVES WITH θ_H = 90 DEG (1.57 RAD), θ_V = 0 DEG (0 RAD)

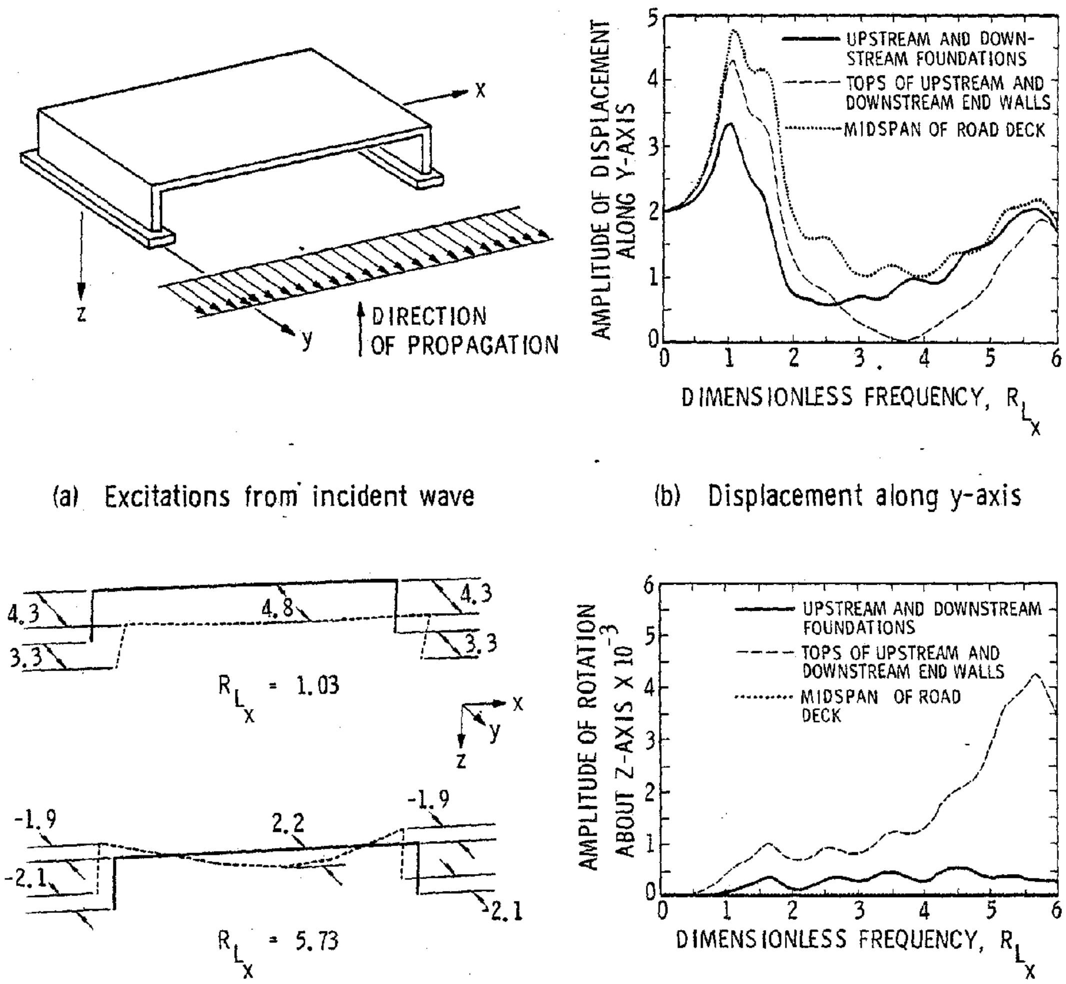
The bridge response for this case exhibits significant sidesway displacements, which occur over the same narrow resonant frequency range as that for the vertically incident waves of Case 1 (Figs. 7b, 7c). However, the amplitudes of these displacements are reduced from those of Case 1 because, as shown in [18], only part of the energy from the spatially varying excitation--i.e., that corresponding to excitation components symmetric about the x-axis--is now available to drive the structure in the x-direction. The remaining energy--which corresponds to antisymmetric excitation components--causes rotations of the foundation about the z-axis (Fig. 7d). Since these foundation rotations are large relative to those of the road deck, they correspond to significant torsional deformations in the end walls. They are seen to be largest in the frequency range of about $0.25 \leq R_{L_y} \leq 1$, but are still prominent at higher excitation frequencies. Such rotations are not induced by the vertically incident waves of Case 1.

Still another important feature of the bridge response for this case is the nature of the foundation displacements in the x-direction that result from incident waves whose wavelengths are short relative to the foundation length (i.e., for high excitation frequencies that are represented by large values of R_{L_y}). In contrast to Case 1, these foundation displacements are now substantially smaller in amplitude than the corresponding displacements of the free field (Fig. 7b). This is because, for incident waves with short wavelengths, the net loadings applied along the length of the rigid foundation by the symmetric components of the spatially varying excitations are reduced; therefore, they are less effective in driving the foundations in the x-direction [18].

<u>Case 3: θ_H = 0 deg (0 rad) and θ_V = 90 deg (1.57 rad)</u>--The third case considers vertically incident SH-waves that differ from Case 1 in that they propagate in a plane parallel to (rather than normal to) the bridge span. For this case, the corresponding input motions applied to each foundation are identical in amplitude and phase and are directed along the y-axis (Fig. 8a).

The resulting bridge response consists of displacements in the y-direction and rotations about the z-axis that are symmetric about the midspan of the bridge (Figs. 8b to 8d). Peaks in these response amplitudes occur at frequencies of R_{L_x} = 1.03 (4.3 Hz), where the response primarily comprises rigid-body motions, and at R_{L_x} = 5.73 (23.9 Hz), where the response features prominent bending deformations in the road deck and torsional deformations in the end walls (Figs. 8c and 8d). Because of the underlying soil medium, these frequencies are much lower than those of the corresponding out-of-plane fixed-base modes shown in Figure 5b (i.e., Modes 1 and 3, which have frequencies of 14.2 Hz and 49.5 Hz, respectively).

<u>Case 4: $\theta_H = \theta_V$ = 0 deg (0 rad)</u>--The fourth case considers horizontally incident SH-waves that, like the waves considered in Case 3, propagate in a plane parallel to the bridge span and apply excitations to the bridge that are directed



(a) Excitations from incident wave

(b) Displacement along y-axis

(c) Deformed shapes at time of peak response

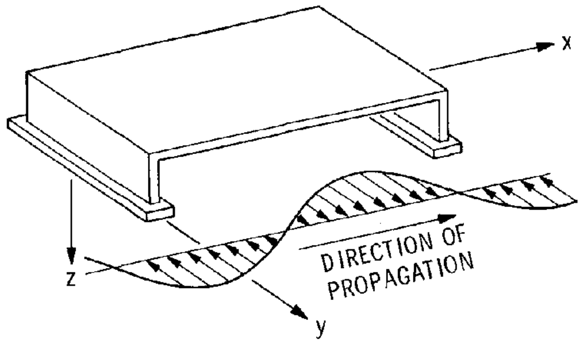
(d) Rotations about z-axis

FIGURE 8. CASE 3: BRIDGE RESPONSE TO INCIDENT SH-WAVES WITH θ_H = 0 DEG (0 RAD), θ_V = 90 DEG (1.57 RAD)

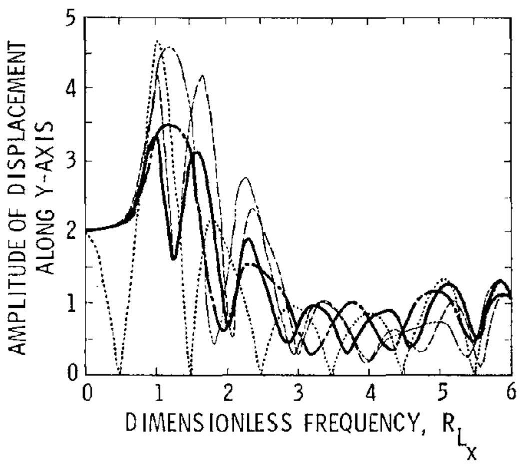
along the y-axis (Fig. 9a). However, since the waves are not vertically incident, there is now a phase difference between the excitations applied to each foundation.

As for Case 3, the principal bridge response consists of displacements along the y-axis and rotations about the z-axis. Response amplitudes are given in Figures 9b and 9c and show two important trends. First, the bridge response is clearly more complex than that resulting from the vertically incident SH-waves of Case 3. Second, close examination of response amplitudes given in Figures 9b and 9c and phase angles given in [18] shows the existence of distinct patterns of bridge response--symmetric, antisymmetric, and whipping--that occur at particular sets of wavelengths of the incident wave.

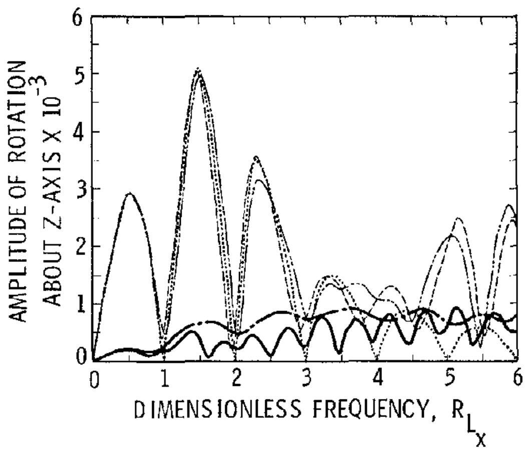
Bridge responses that are symmetric about the midspan occur when the wavelength of the incident wave is such that the excitations applied to each foundation



(a) Excitations from incident wave



(b) Displacements along y-axis



(c) Rotations about z-axis

UPSTREAM FOUNDATION
DOWNSTREAM FOUNDATION
TOP OF UPSTREAM END WALL
TOP OF DOWNSTREAM END WALL
MIDSPAN OF ROAD DECK

FIGURE 9. CASE 4: BRIDGE RESPONSE TO INCIDENT SH-WAVES WITH $\theta_H = \theta_V = 0$.DEG (0 RAD)

are identical in amplitude and phase; this occurs at dimensionless frequencies of $R_{L_x} = 1.0$, 2.0, 3.0, etc. (Fig. 10a). Deformed shapes of the bridge for this case are shown in Fig. 11a and, from comparisons with Figure 8c, are seen to be similar to those resulting from vertically incident waves.

Responses of the bridge that are antisymmetric about its midspan occur when the wavelength of the incident wave is such that the excitations applied to each foundation are of equal amplitude and opposite phase, as shown in Figure 10b. This occurs at dimensionless frequencies of $R_{L_x} = 0.5$, 1.5, 2.5, etc. The antisymmetric responses at lower frequencies in this type of excitation (e.g., $R_{L_x} = 0.5$ and 1.5) feature rigid-body rotations of the road deck about the z-axis that are much larger than the corresponding rotations of the foundations (Fig. 9c). Therefore, the end walls are undergoing torsional deformations. At higher frequencies (e.g., $R_{L_x} > 3.5$), these torsional deformations are reduced. Antisymmetric deformed shapes of the bridge are shown in Figure 11b.

A third type of response (whipping) occurs when the wavelengths of the incident waves are such that the excitations applied to each foundation are 90 deg (1.57 rad) out of phase; this occurs at dimensionless frequencies of $R_{L_x} = 0.25$, 0.75, 1.25, 1.75, etc. (Fig. 10c). The resulting bridge response features large displacements in the y-direction at one end while, at the same time, the other end is experiencing relatively small displacements; i.e., the bridge is "whipping" about a center of rotation near the end with the small displacements. At low excitation frequencies ($R_{L_x} = 0.75$), this response comprises displacements at the

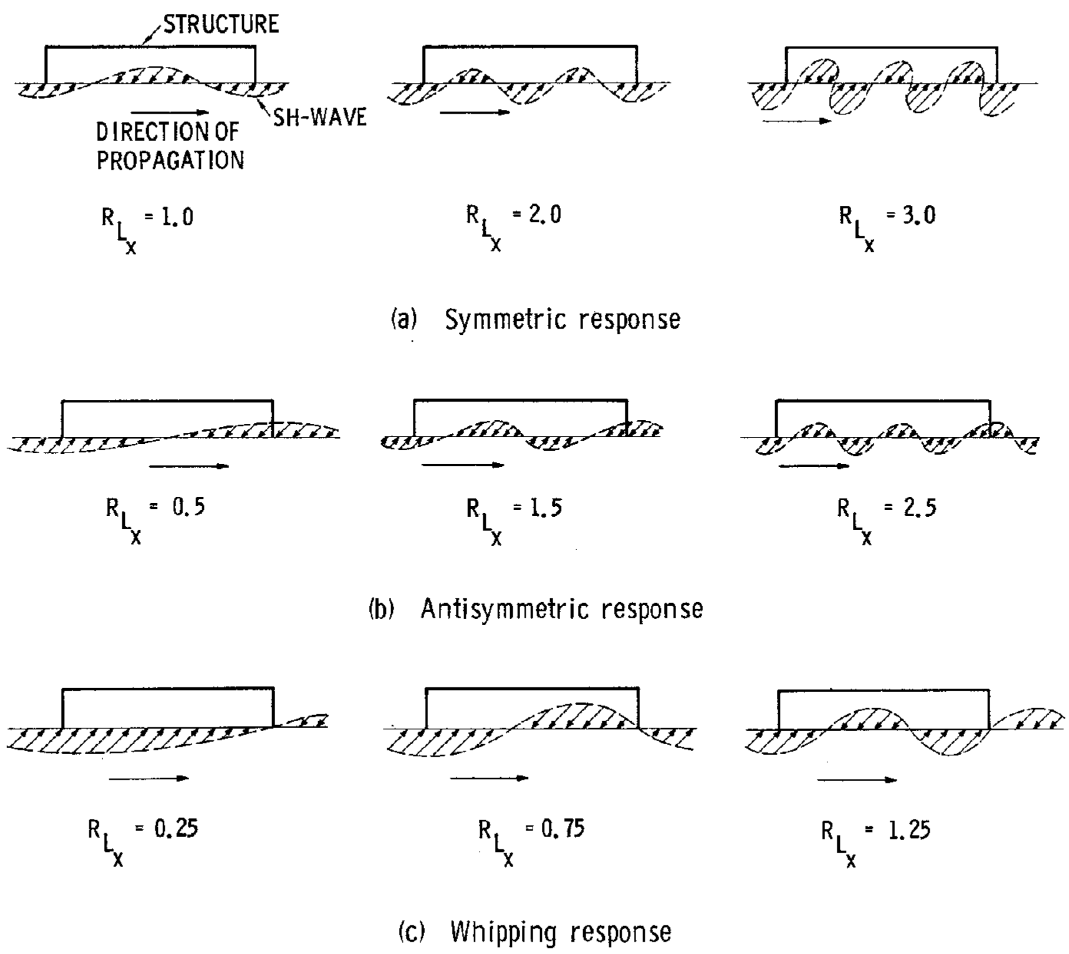


FIGURE 10. CASE 4: RELATIONSHIP BETWEEN WAVELENGTH OF INCIDENT SH-WAVES AND BRIDGE RESPONSE CHARACTERISTICS, $\theta_H = \theta_V = 0$ DEG (0 RAD)

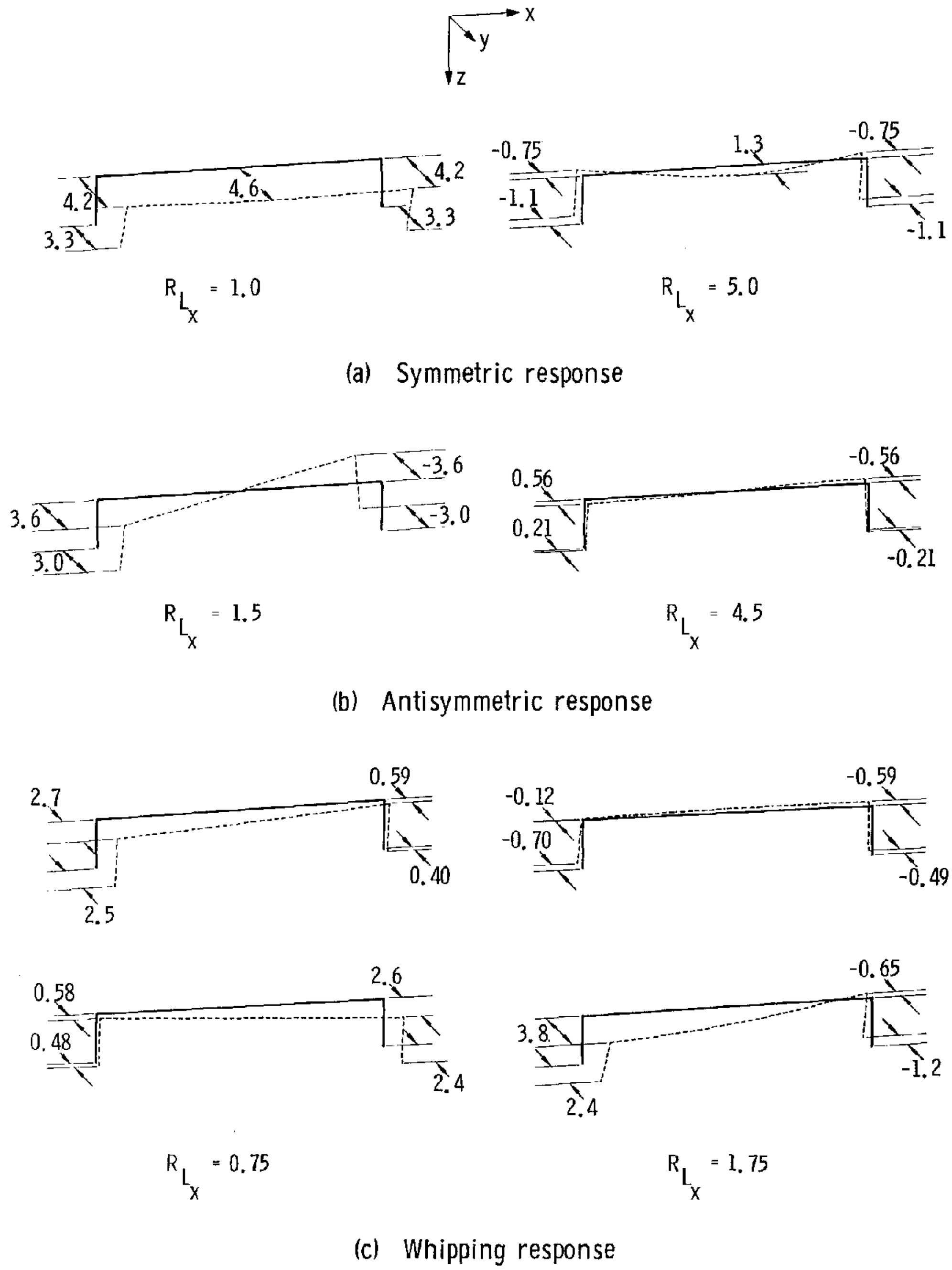


FIGURE 11. CASE 4: DEFORMED SHAPES OF BRIDGE AT TIMES OF PEAK RESPONSE TO INCIDENT SH-WAVES WITH $\theta_H = \theta_V$ = 0 DEG (0 RAD)

two ends of the bridge that are of nearly equal amplitude and are nearly in phase with the free-field excitations (i.e., they are about 90 deg (1.57 rad) out of phase with each other). This low-frequency response can therefore be envisioned as whipping about first one end of the bridge and then the other, and is seen to consist primarily of rigid-body displacements (Fig. 11c). At higher excitation frequencies (R_{L_x} = 1.75), the whipping response is more complex because of the increased effects of wave diffraction about the upstream foundation and wave scattering from the downstream foundation. For this case, the displacements of the two end walls now exhibit sizable differences in amplitude, and the bridge is whipping about its downstream end only (Fig. 11c). In addition, the bridge response now features prominent bending deformations; and as shown in [18], the two ends of the bridge are no longer responding in phase with the free-field excitations.

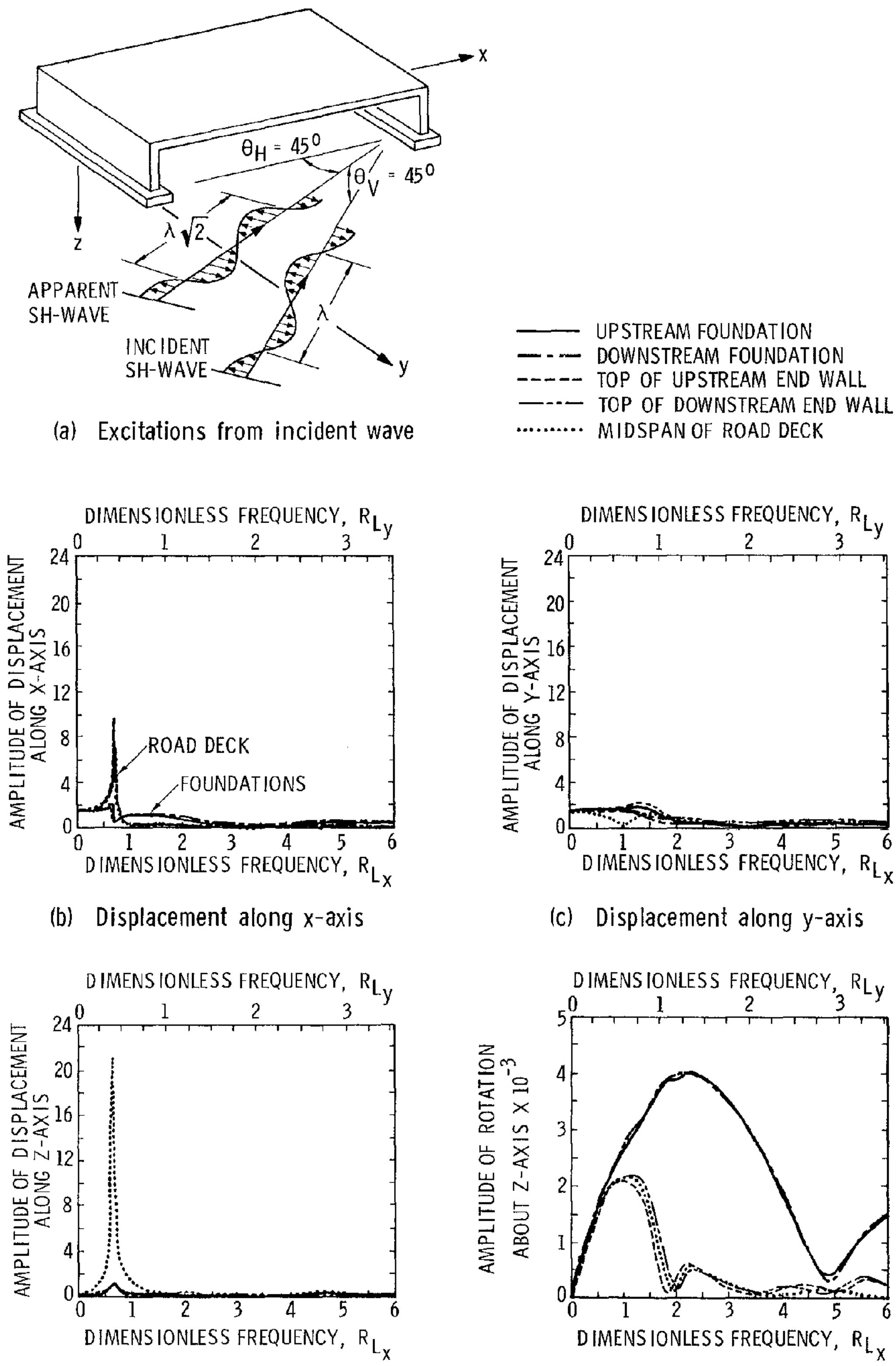
Case 5: $\theta_H = \theta_V = 45$ deg (0.79 rad)--The final case considers waves that propagate at an angle of 45 deg relative to the ground surface and in a plane at 45 deg (0.79 rad) relative to the x-z plane of the bridge (Fig. 12a). The resulting bridge response is fully three dimensional, as shown by the response amplitudes for all three displacement components (Figs. 12b through 12d) and for the rotations about the z-axis (Fig. 12e). These figures show the following trends:

a. All three displacement components are coupled at excitation frequencies below $R_{L_y} = 1$, with the most significant coupling occurring over a narrow frequency range ($R_{L_y} = 0.4$ to about 0.5) and involving large amplitudes of displacement along the x- and z-axes (Figs. 12b to 12d and 13). This can be contrasted with Cases 1 to 4, where no such coupling effects were induced. At higher excitation frequencies, the displacement amplitudes are small.

b. The largest displacements are those of the road deck in the x-direction and the z-direction. The peak values of these displacement components occur at nearly the same frequency ($R_{L_y} = 0.423$ and 0.401, respectively) and result from two resonance phenomena that are coupled for these particular angles of incidence (Fig. 13). The displacements of the road deck in the x-direction are analogous to the sidesway vibrations already discussed for Cases 1 and 2, and occur at nearly the same frequency (i.e., $R_{L_y} = 0.412$ for Cases 1 and 2 vs. 0.423 for this case). The displacements of the road deck in the z-direction correspond [18] to a resonance with the fundamental in-plane mode of the bridge/soil system (the associated fixed-base mode is shown as Mode 1 in Fig. 5a). These large vertical displacements result from the horizontal incident wave motions because of the off-diagonal coupling terms in the foundation/soil impedance matrix and from the different phases of the wave motion at the two foundations for this particular excitation frequency and these angles of incidence [18].

c. The displacements along the y-axis do not exhibit any prominent amplifications of the incident wave motions (Fig. 12c). The amplitudes of these displacement components fall below those of Case 4 (Fig. 9b).

d. At lower excitation frequencies ($R_{L_y} < 0.5$), significant rotations about the z-axis are induced in both the foundations and the road deck (Fig. 12e). This differs from Case 2, where such rotations were only generated in the foundations, and from Case 4, where much larger rotations occurred in the road deck (Figs. 7d and 9c). At higher excitation frequencies, there is a marked increase in the rotations of the foundations relative to those of the road deck. This is



(a) Excitations from incident wave

(b) Displacement along x-axis

(c) Displacement along y-axis

(d) Displacement along z-axis

(e) Rotation about z-axis

FIGURE 12. CASE 5: BRIDGE RESPONSE TO INCIDENT SH-WAVES WITH $\theta_H = \theta_V$ = 45 DEG (0.79 RAD)

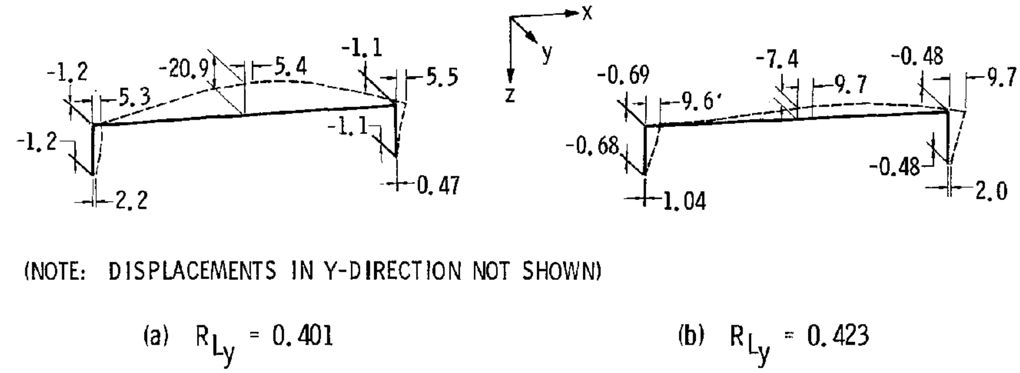


FIGURE 13. CASE 5: DEFORMED SHAPES AT TIMES OF PEAK DISPLACEMENT IN X- AND Z-DIRECTION - INCIDENT SH-WAVES WITH $\theta_H = \theta_V = 45$ DEG (0.79 RAD)

similar to the foundation rotations observed in Case 2 (Fig. 7d) except for a reduction in the peak amplitudes of these rotations and an expansion of their frequency scale because of apparent wavelength effects [18].

CONCLUSIONS

A new methodology has been developed for analyzing the three-dimensional response of soil/structure systems excited by traveling seismic waves. To provide insights into the response of such systems, the methodology has been used to analyze a single-span bridge supported on an elastic half-space and subjected to incident SH-waves. The results of this analysis lead to two main conclusions. First, phase differences in the input ground motions applied to the bridge foundations can have significant effects on the bridge response. Therefore, it is important to consider such traveling wave effects when designing earthquake-resistant structures of this type. Second, the nature of the bridge response to these traveling seismic waves is strongly dependent on the direction of incidence as well as on the excitation frequency of the seismic waves. Therefore, it is not sufficient to consider only a single direction of propagation when evaluating the effects of traveling waves on the three-dimensional response of a bridge structure.

ACKNOWLEDGMENTS

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APPENDIX I. – REFERENCES

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APPENDIX II. — NOTATION

The following symbols are used in this paper:

R_L, R_{L_x}, R_{L_y} = dimensionless frequencies;

V_s = shear wave velocity of soil medium;

ℓ = structure dimension used in defining dimensionless frequencies;

u_i = amplitude of incident wave motion;

θ_H, θ_V = angles of incidence; and

ω = circular frequency.

APPLICABILITY TO BRIDGES OF EXPERIMENTAL SEISMIC TEST RESULTS PERFORMED ON SUBASSEMBLAGES OF BUILDINGS

by

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The University of Texas at Austin

ABSTRACT

One dominant problem occurs in all types of reinforced concrete construction in seismic areas--detailing the members or elements and joints between the elements to achieve a satisfactory level of performance when the seismic event occurs. In this regard, bridge structures are no different than buildings. The engineer must consider details which will ensure that the ductility, strength, and integrity of the structure are acceptable.

Strength in flexure and shear is essential. Ideally the proportions and reinforcement are selected to ensure that the shear strength equals or exceeds the flexural strength. The mode of failure can be easily controlled under monotonic loading; however, under reversed cyclic loading, a member responding in a flexural mode can fail in shear if sufficient cycles and levels of deformation are applied. Therefore, the design criteria and the detailing required are a function of the loading history imposed.

During the past 10 to 20 years, a great deal of work has been done on the response of building subassemblages to cyclic load histories. Selected studies are discussed to give an indication of the work that has been done and the applicability of that work to bridge behavior is discussed.

APPLICABILITY TO BRIDGES OF EXPERIMENTAL SEISMIC TEST RESULTS PERFORMED ON SUBASSEMBLAGES OF BUILDINGS

by

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INTRODUCTION

One dominant problem occurs in all types of reinforced concrete construction in seismic areas--detailing the members or elements and joints between the elements to achieve a satisfactory level of performance when the seismic event occurs. In this regard, bridge structures are no different than buildings. The engineer must consider details which will ensure that the ductility, strength, and integrity of the structure are acceptable.

Ductility can be broadly categorized as the ability of the structure to deform inelastically without substantial loss of strength and without sudden brittle failure occurring before a desired level of deformation is reached. In frame building structures this is achieved by permitting certain regions of the structure to form hinges. Furthermore, proportions generally are controlled so that the hinges are forced into the beams--"the weak beam-strong column" philosophy. In a bridge structure this may not be possible. Inelastic deformations may occur only in the columns or piers because of the relative proportions of the elements. Therefore, if inelastic flexural or shear deformations are to occur, the amount of reinforcement must be carefully considered and it must be properly anchored and perhaps confined in some cases.

Strength in flexure and shear is essential. Ideally the proportions and reinforcement are selected to ensure that the shear strength equals or exceeds the flexural strength. The mode of failure can be easily controlled under monotonic loading; however, under reversed cyclic loading, a member responding in a flexural mode can fail in shear if sufficient cycles and levels of deformation are applied. Therefore, the design criteria and the detailing required are a function of the loading history imposed.

Integrity of the structure must be maintained. Even though individual members may be heavily damaged in a severe earthquake, the structure should remain standing and be able to support some reduced level of service. Proportioning the structure for service loads may not lead to the most efficient structure for seismic overloads; however, the integrity of the structure in both cases must be considered. In terms of design requirements, bridges are not substantially different from buildings in that deformation limits must be established and the condition of the structure at the deflection limits considered.

While the preceding discussion has focused on the similarities between bridges and buildings, some major differences are apparent. Most buildings are multi-bay, multi-story three-dimensional structures. In an earthquake some elements of the structure will experience inelastic, possibly severe, deformations. However, it is unlikely that all elements will be forced into the inelastic range simultaneously or that enough "hinges" will form to initiate an unstable mechanism. On the other hand, a bridge is often a single or multiple bent. Many bridges could be considered planar structures. As such, the redundancy is much less than in a building and if one element becomes highly nonlinear or if a "hinge" forms, the result may be catastrophic. Therefore, design and detailing of each element in the bridge framework must be considered more carefully than in a building.

Another, perhaps more significant, difference is one of scale. The size of members and the reinforcement used is generally larger in bridges than in buildings. While #14 and #18 bars would rarely be used in conventional frame building construction, such bars are commonly used in bridge columns and piers. Similarly, column cross sections of 36 in. would be large in a building; such dimensions would be common in a bridge and columns of 6 ft. diameter might be used. In the following discussion of tests of building subassemblages, special mention will be made of the size of test specimens so that the applicability of the work to bridges can be judged.

During the past 10 to 20 years, a great deal of work has been done on the response of building subassemblages to cyclic load histories. The number and scope of the investigations does not permit an account of all of them here. Selected studies will be discussed to give a "flavor" of the work that has been done. A workshop held in July 1977 at the University of California-Berkeley on Earthquake Resistant Reinforced Concrete Building Construction [1] surveyed a great deal of the work and is recommended for further study.

LOADING HISTORY

Before any discussion of experimental work can proceed, it is necessary to examine briefly the influence of loading history. Since different investigators use different loading patterns, the results of the studies may not always be directly comparable. It is quite possible to produce different answers to questions of structural behavior by varying the loading history. Analytical work is needed to determine loading histories which might be expected for elements in bridge structures.

Types of Loading

Figure 1 shows some of the loading histories used in experimental investigations. The differences are readily apparent. In some cases the load is cycled between prescribed deflection limits until failure or severe distress is observed. In other cases, the deformation limit increases after the application of a number of load cycles at a given limit. A further complication arises when deformations used to control the loading history are not directly comparable, such as deflection in one case and member rotation or curvature in another. It may not be necessary to have "standard" loading routines used by all investigators, but it is important that users of experimental results are cognizant of the influence of loading history on response in evaluating experimental data. In fact, there may be merit in diversity because there will be a greater need for more thoughtful comparison of test results.

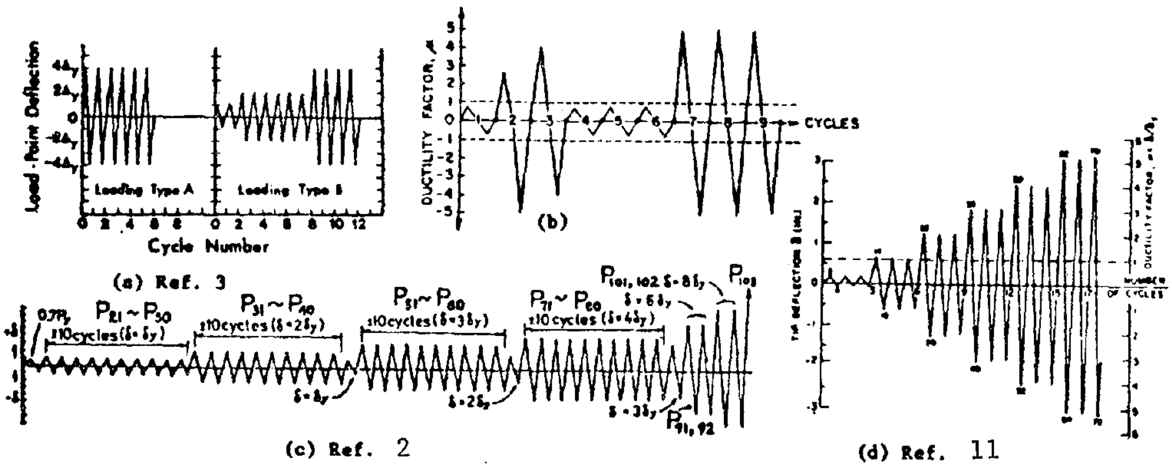


Fig. 1 Various Types of Loading History

Load-deflection curves are shown in Fig. 2 for two specimens subjected to the loading history shown in Fig. 1c. The performance can be compared visually by noting the "pinching" of the curves toward the origin indicating reduced energy-absorbing characteristics.

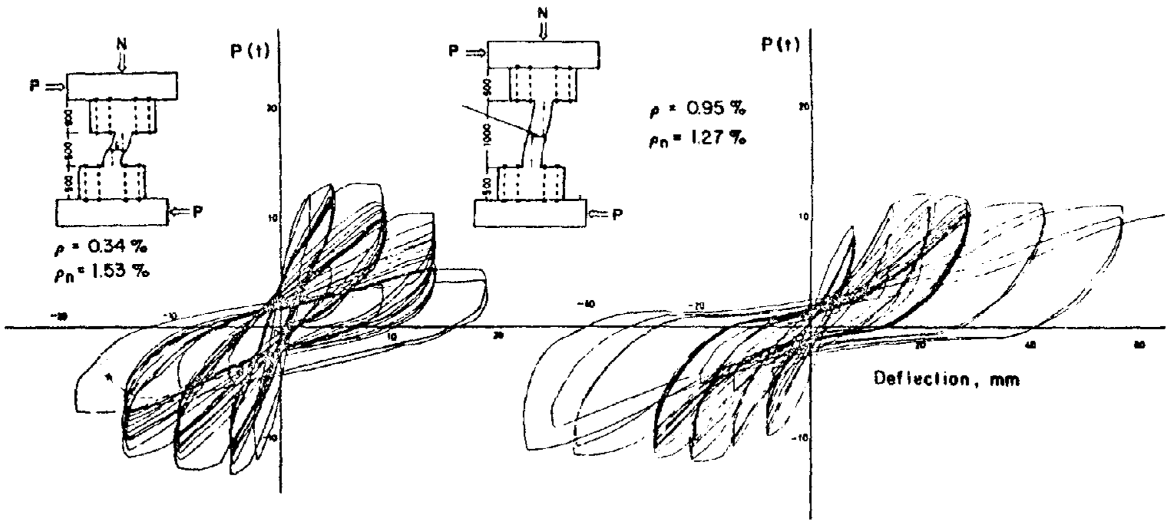


Fig. 2 Lateral Shear-Deformation Curves [Ref. 2]

From the response shown for the specimen in Fig. 3 (load history in Fig. 1a), it is evident after the second cycle that the performance of the specimen is unsatisfactory [3]. Poor performance will generally be evident after a relatively few load applications and subsequent loading only confirms the trends evident initially. Higashi, Ohkubo, and Ohtsuka [4] conducted a series of tests using loading patterns, shown in Fig. 1c, on companion specimens subjected to either three or ten load reversals at each

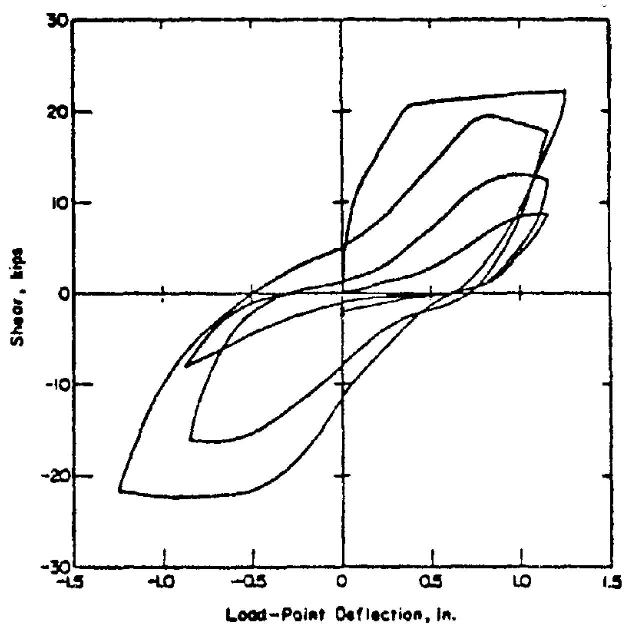


Fig. 3 Shear-Deflection Curves [Ref. 3]

deflection level. The results indicate that increasing the number of cycles did not alter the response substantially. Of greater significance was the severity of the reversal. Where specimens were subjected to equal deformation levels in each direction, the strength degradation was more rapid and severe than when the deflection in one direction was limited. Both types of loading caused a more rapid decay of strength than for monotonic loading.

Biaxial Loading

Frames are generally designed considering that each principal direction resists lateral forces independently of the orthogonal direction. As a result, research has been limited to tests of members or subassemblages of planar frames. Analytical studies indicate that two-dimensional response characteristics may be considerably more severe than when only one direction of motion is considered. Aktan, Pecknold, and Sozen [5] compared 1D and 2D response of a reinforced concrete column subjected to various ground acceleration records. A comparison of the relative displacement of the column for one record (Taft 1952) is shown in Fig. 4. From this study it was concluded that calculations based on one horizontal component of ground motion were unconservative compared with the displacement obtained from a consideration of both components. Others [6,7] have undertaken similar students to determine the influence of biaxial deformations on reinforced concrete column response. However, there is very little experimental work available to complement the analytical studies. Extensive research, both experimental and analytical, is needed to ascertain the importance of bidirectional loadings and to evaluate the strength of structures under such loadings.

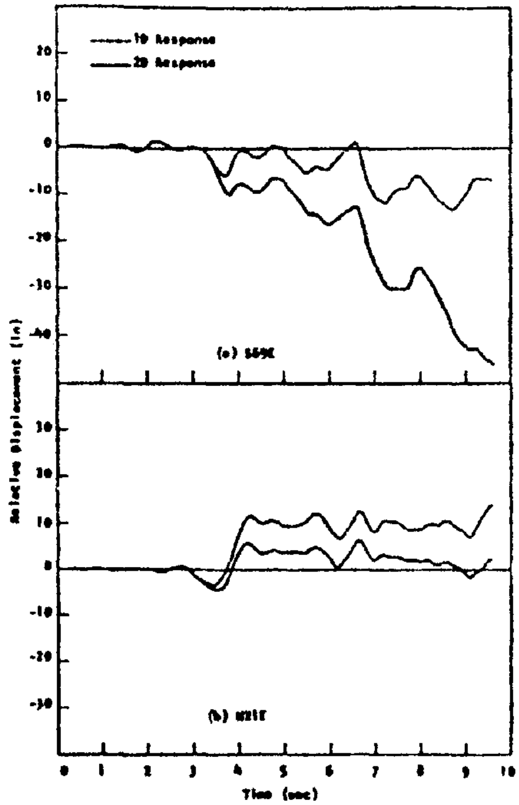


Fig. 4 Comparison of Computed 1D and 2D Response [Ref. 5]

BEHAVIOR OF COLUMNS IN SHEAR AND FLEXURE

The performance of members failing in shear under planar lateral forces and axial compression has been studied fairly extensively. Some of the types of test specimens utilized are shown in Fig. 5. An extensive test program

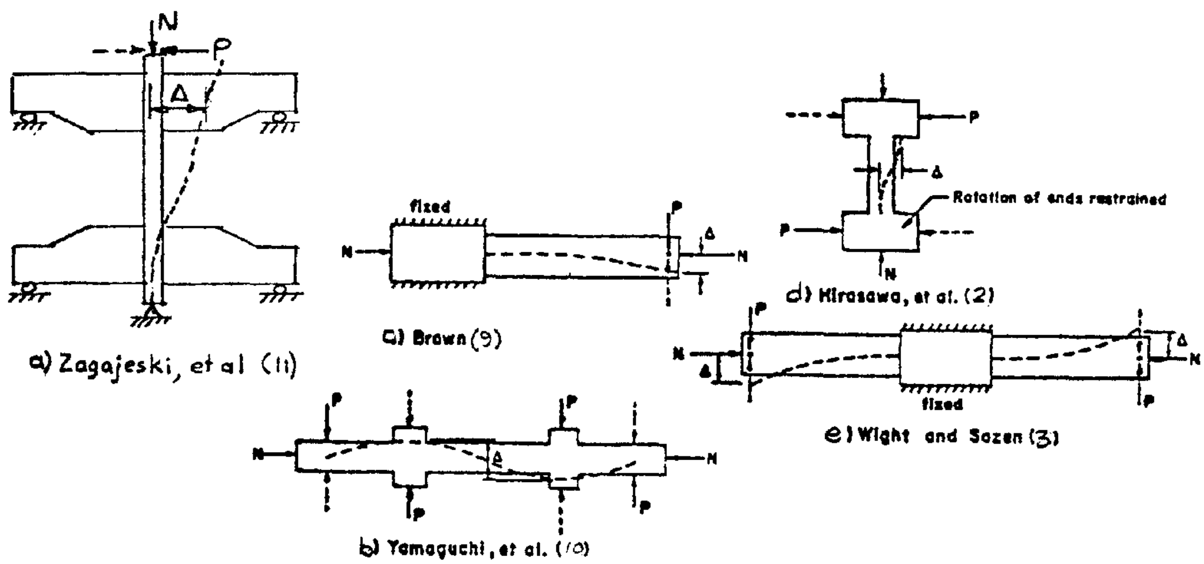


Fig. 5 Type of Test Specimens

conducted in Japan, summarized by Higashi and Hirosawa [2], provides a summary of the types of behavior or mode of failure to be expected in members subjected to load reversals. Using load histories, as shown in Fig. 1c, the characteristics of the member performance were summarized as follows:

A. Very ductile members failing in shear or buckling of compression bars at large lateral deformations.
B. Ductile members failing in shear, bond deterioration, or bar buckling at ductility ratios (Δ/Δ_y) between 4 and 6.
C. Members reaching yield but failing in shear, bond or bar buckling in early stages of loading.
D. Members failing in shear or bond before flexural yielding is reached.

A recent report by Kubota and Sozen [8] summarizes virtually all of the test data available in Japan (791 tests). All but 62 of the specimens had cross sections of 25 x 25 cm or less. The remaining 62 had cross sections as large as 40 x 60 cm. Other tests [3,9,10] utilized specimens with cross sections of 12 x 12 in. or less. These are considerably smaller than typical bridge columns.

Hinging Regions Failing in Shear

Members failing in Cases A, B, and C reach flexural yield strength and then, depending on the severity of loading, fail in shear, bond, or bar buckling. Generally, such members would perform satisfactorily under unidirectional loading. Gosain, et al. [12], proposed an approach for estimating the relationship between severity of loading and shear resistance in hinging regions using the area under the load-deflection curves. Rather than compute the actual area under the curves, a simplified procedure was used. First, the load-deflection curves were normalized with respect to yield values (P/P_y and Δ/Δ_y). From the normalized load-deflection curves, the maximum deflection ratio was determined for each cycle. In the tests considered, the deflection ratio in each direction from the origin was about equal. The area under the normalized load-deflection curves was termed the work index.

Examination of the load-deflection curves indicated that the work index was sensitive to the shear span-to-depth ratio (a/d_c) and to the level of axial load (N/A_c) on the member. Only core dimensions were used because the outer shell tends to spall away at early stages of loading, leaving the core to carry all forces. Figure 6a shows load-deflection curves with low axial load. Figure 6b and Fig. 2 show curves with shear span-to-depth ratios varying from 6.2 to 1.4. Note the severe pinching of the curves toward the origin as the axial load and shear span ratio are reduced.

Maximum Allowable Shear Stress --A relationship between the work index and the measured ultimate shear stress was developed. Depending on the performance required, the maximum allowable shear stress on the core can be estimated. Assuming that a performance equivalent to 5 cycles at 5 times the yield deflection with no more than a 25 percent reduction in shear capacity is required, shear stresses on the core should not exceed about $6\sqrt{f'_c}$. In addition, transverse steel should provide a capacity approximately equal to that provided by the concrete so that

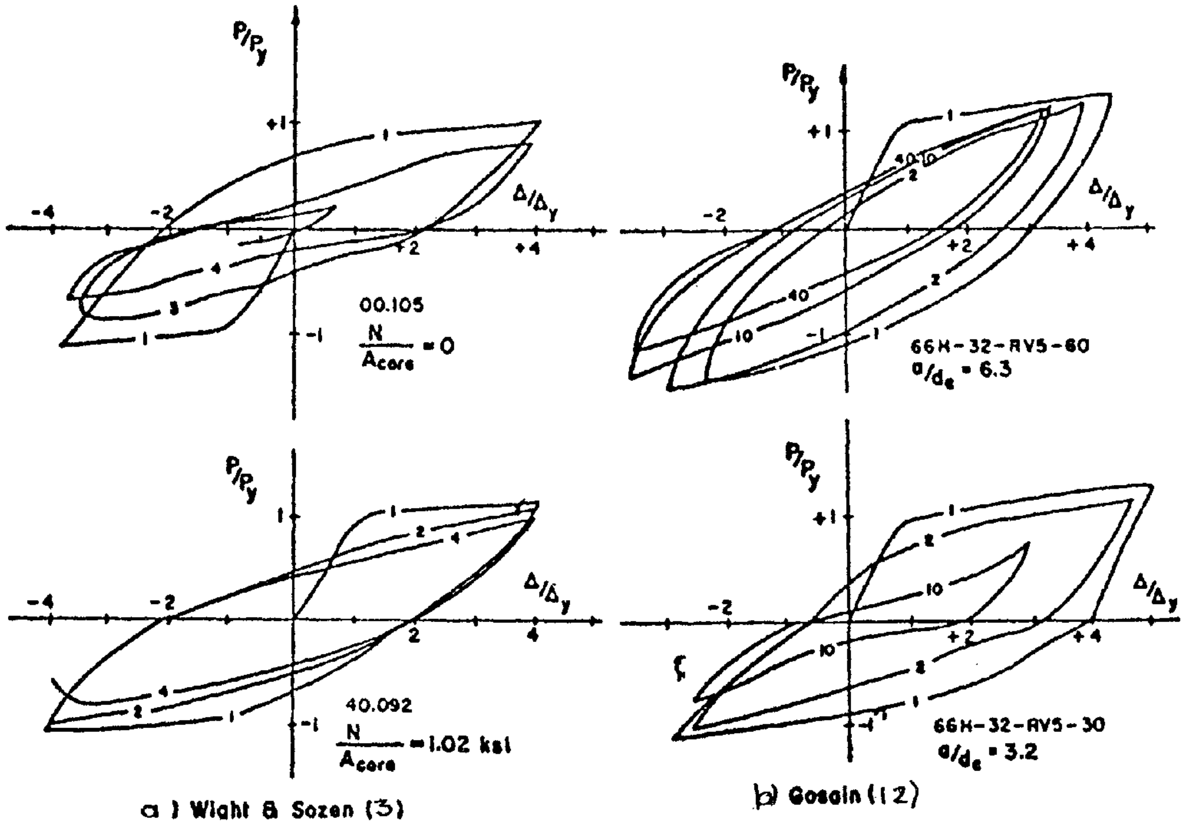


Fig. 6 Influence of Axial Load (N/A_{core}) and Shear Span (a/d_c) on Response

$$\rho_w f_y = \frac{A_v}{sb_c} f_y \geq 6\sqrt{f'_c} \tag{1}$$

where s is the spacing of stirrups, b_c is the core width, and A_v is the area of transverse steel. These values are similar to those recommended by the other investigators [13].

It should be noted that in all the tests considered, zero or compression axial loads were imposed. The behavior of hinging regions in the presence of tension has not been studied.

Buckling of Longitudinal Reinforcement --Failure of hinging regions is often a complex interaction between shear deformation, concrete crushing, bond deterioration, and longitudinal bar stability. Because the shear deformation is primarily across flexural cracks almost normal with the direction of bending [9], the reinforcement serves more as confinement for the core than as a shear-carrying element. In addition to confining the core, it binds the longitudinal steel to the core and reduces the unsupported length of compression bars. To provide adequate lateral support, Gosain, et al. [12] recommended spacings not exceeding 6 longitudinal bar diameters, Bertero and Popov [13] recommend 6 to 8 bar diameters, and Higashi and Hirosawa [2] recommend 8 bar diameters. Additional work is needed on large bar sizes to determine whether the relationships developed from tests with small bars are valid for #14 and #18 bars.

Bond Failures --The influence of bond and anchorage will be discussed later, but brief mention is made here of problems associated with such failures. Higashi and Hirosawa [2] indicate that using the concept of bond stress was not adequate to explain the failures observed. Rectangular hoops were not as effective as spiral hoops in improving bond characteristics. One approach to improvement of bond in the hinging region is to limit the flexural capacity of the section to some fraction of the shear capacity at the end of the hinge (a distance from the face of the support equal to the effective depth) where bond failures were observed to start. It should also be noted that bond deterioration within the joint aggravates distress in the hinging region. As the bars slip within the joint, flexural cracks in the member widen and reduce the effectiveness of shear transfer across the crack.

It should be noted that where the section is subjected to tension, similar failures may be produced but very little experimental work has been done regarding sections in tension.

Columns Failing in Shear

Where failure occurs prior to the achievement of flexural yielding, the failure is generally of a brittle nature and, under reversed loading, is characterized by a rather rapid degradation of shear strength. This can be seen in Fig. 2 for the specimen with a shear span-to-depth ratio of 1.4. Where flexural yielding occurs, adequate ductility is generally obtained; however, hinging in columns is to be avoided in most designs. Therefore, it is essential that the designer also prevent shear failure in the column from occurring before flexural hinging in the beam occurs. Where this cannot be done, the frame will have to be designed for lateral loads based on the maximum shear strength of the columns. Unfortunately, the M-P-V interaction for columns has not been adequately described and estimates of shear strength under various combinations of M and P cannot be made at present. Wakabayashi [14] has discussed this problem in some detail.

Two investigations in which shear failure were studied include Yamada and Yagi [15] and Zagajeski, et al. [11]. Yamada and Yagi tested specimens in double curvature using specimens similar to the type shown in Fig. 5b. It was concluded that the transition from flexural yielding to shear failure is a critical a/d ratio which is a function of axial load, material properties, and amount of longitudinal reinforcement. In general terms, shear failure is likely where the shear span-to-depth ratio is less than 2. To provide deformation capacity where shear failure is likely, Yamada and Yagi recommend that the transverse reinforcement ratio be at least 1 percent.

Zagajeski et al. [11] tested specimens of the type shown in Fig. 1a. The shear span-to-depth ratio was 1.5. Both spiral and tied columns were tested. The spirals were more effective in preventing shear failures but produced conditions which led to bond problems because the spirals were closely spaced and virtually all cover concrete spalled away. The shear behavior of the columns was found to be heavily dependent on the loading history. Addition of transverse reinforcement significantly improved the lateral deformation capacity of the assemblage.

A special problem with regard to shear strength is the scale of the specimen. In the tests mentioned above the column sections were 12 x 12 in. or less. Large-scale tests are needed but the difficulties in applying large axial loads have prevented investigations of such magnitude. Virtually all existing tests on the shear strength of columns have been conducted on relatively small columns with axial compressive loadings. Because frame structures may be subjected to overturning effects, or vertical accelerations, tensile loads may be produced which will likely reduce the shear resistance of the column.

Columns under Bidirectional Loading

As mentioned previously, the behavior and analysis of structures under biaxial loading is receiving attention; however, the experimental work available is very limited.

Flexural Behavior --Several investigators [16,17,18,19] have examined the behavior of columns under 2D lateral loading. The results indicate that for structures failing in a flexural mode (ductile moment-resisting frames), 2D motions become critical when deflections exceed about twice the yield deflection under 1D loading. Although the computations are lengthy, models can be developed to estimate 2D behavior which compare closely with the measured results. An example of this is shown in Fig. 7 from work by Takiguchi [16], which compares measured and computed results for biaxial bending.

Shear Behavior --To investigate shear behavior of columns under 3D loading histories, a test program has been undertaken at The University of Texas at Austin [20]. The test specimen is similar to that shown in Fig. 1d. The primary variable is the loading history. The column is a stiff element framing into a stiff floor system (column section 12 x 12 in.).

Figure 8 shows some results of a recent study by Okada, et al. [18], on reinforced concrete members under biaxial load reversals and constant axial load. The specimens developed flexural hinges. Under biaxial loading the specimen deteriorated more rapidly, as indicated by the restoring force history.

Figure 9 shows the lateral force-deformation curves for three tests in which the lateral load history was varied (no axial load). The force-deformation relationships are shown for a principal axis of the column. Such a comparison indicates a severe reduction of capacity due to prior or simultaneous loading in the orthagonal direction. However, it is interesting to note that if under 2D loading the resultant force is plotted against the resultant deformation or the radial deformation from the original position, differences between 1D and 2D response are not as large. While a great deal of additional testing will be needed to quantify the response, the results to date indicate that 2D response may not be dependent on deformation path but on the resultant force and deformation on the section resisting shear.

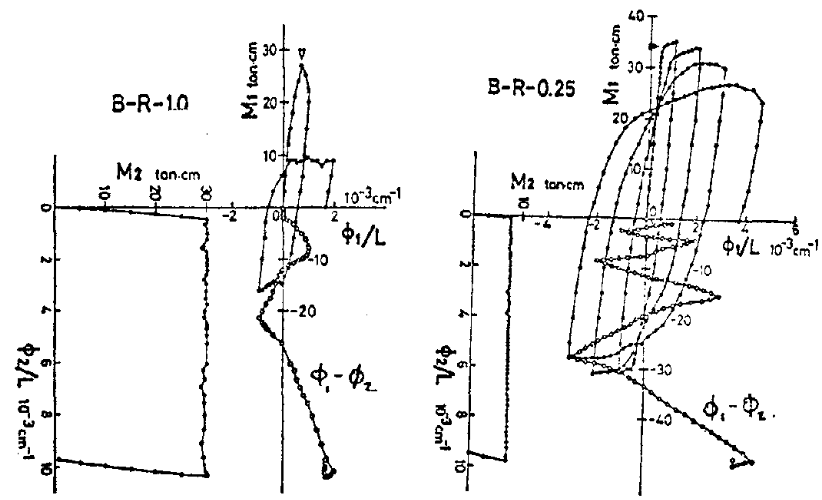
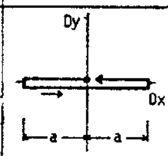
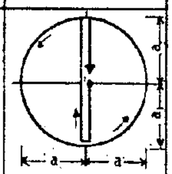
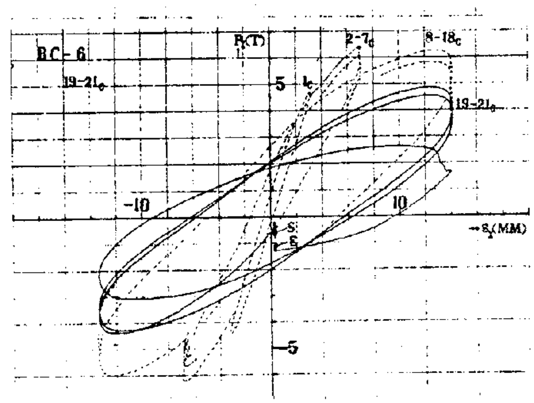
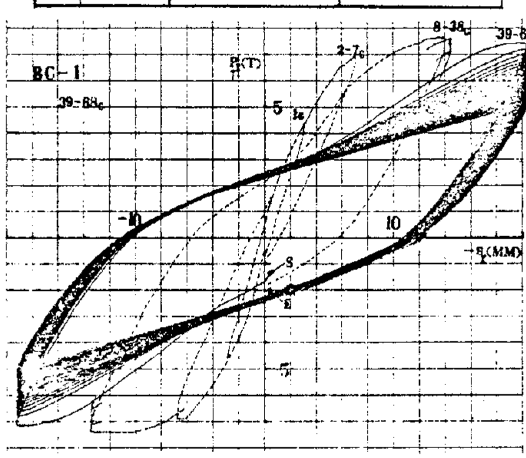


Fig. 7 2D Moment-Curvature Response [Ref. 16]

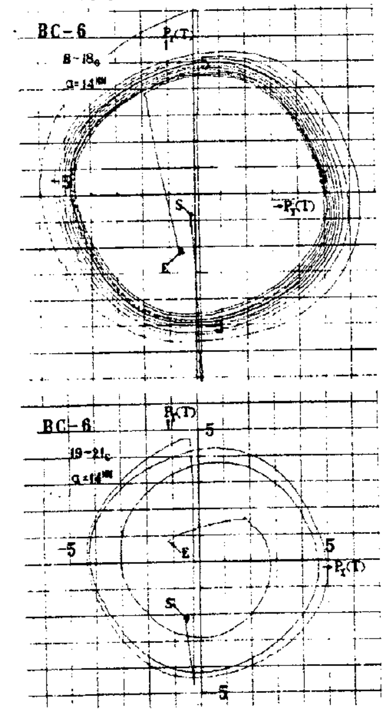
TEST		BC - 1		BC - 6	
DISPLACEMENT HISTORY					
A		a		a	
B		0		a	
φ		-		90°	
RUN	a (RADIAN)	CYCLES NUMBER	(SUM)	CYCLES NUMBER	(SUM)
I	3 mm. (1/300)	1		1	
II	7 mm. (1/125)	6	(7)	6	(7)
III	14 mm. (1/60)	31	(38)	14	(21) COLLAPSE
IV	20 mm. (1/45)	30	(68)		



FORCE VS. DISPLACEMENT RELATIONSHIP

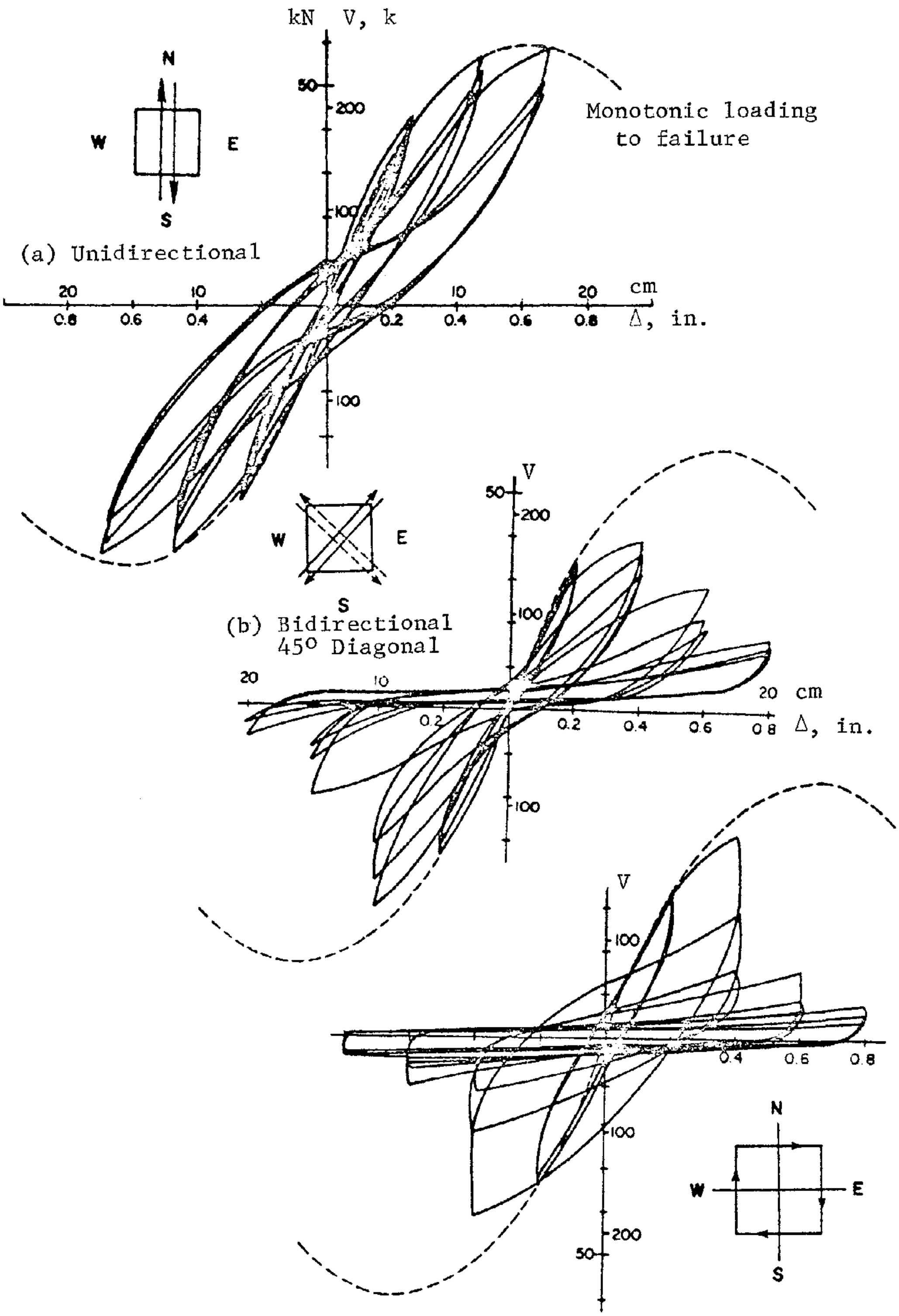


FORCE VS. DISPLACEMENT RELATIONSHIP



RESTORING FORCE HISTORY

Fig. 8 Biaxial Lateral Loading of Columns [Ref. 18]



(c) Bidirectional - square path

Fig. 9 Bidirectional Column Shear Tests--No Axial Load

Because columns subjected to 2D lateral loadings are also subjected to axial loads which may vary, a series of columns has been tested to examine the influence of axial load on specimens failing in a shear mode. Compressive axial loads had little influence on the response as compared with zero load. The shear capacity increased slightly. Tension substantially reduced the shear capacity and the stiffness near the origin. Under cyclic loading the shear capacity did not deteriorate even under large lateral deformation. Additional tests were conducted with 2D lateral loadings and axial load variation (tension and compression). Axial loads appear to have an influence on response only while the load is on the structure and do not influence subsequent response. This is quite different from lateral loadings where loads in one direction influence subsequent response in an orthogonal direction. It should be noted that since the columns were short, lateral displacements were not large enough to cause an increase in moment due to eccentricity of column load.

When scale effects, tensile loadings, and bidirectional lateral loadings are all considered, it is clear that this area needs to be studied in depth for an understanding of shear strength which will lead to the development of design recommendations covering a variety of load cases.

ANCHORAGE AND DEVELOPMENT OF REINFORCEMENT

Two problems are of concern--(1) anchorage failure and (2) slip of the bar relative to the concrete reducing the stiffness of the structure. Anchorage failures are often difficult to identify in building structures following earthquakes; however, in relativity simple column support structures anchorage failures have been noted. Especially notable in this regard is the failure of a pier on a highway overpass during the San Fernando earthquake (Fig. 10). The contribution of bond slip to overall stiffness degradation has been well-documented [13,9]. Slip has been shown to be responsible for as much as 50 percent of the total deformation in some test specimens.

Anchorage to Footings

At the base of the column, longitudinal enforcement is extended into the footing or cast-in-place shaft. Following the San Fernando earthquake, Ikeda, et al. [21] investigated anchorage failure in piers. Pull-out tests of large bars were conducted and the results indicated that the greater the cover and the more transverse reinforcement surrounding the bar, the better the anchorage. A test was devised to simulate a column framing into a large footing. Although the anchorage was sufficient, it was noted that the footing or shaft must have sufficient moment capacity also if satisfactory behavior is expected. Large elongations of the anchored bars were observed. Such elongations would produce large deformations of the super structure.

It should also be noted that the failure in Fig. 10 indicates that the entire column pulled out almost as a "plug" from the footing. It may not be realistic to determine anchorage characteristics of individual bars in cases where a number of bars are closely spaced and failure may occur when the group fails rather than when individual bars pull out of the concrete.

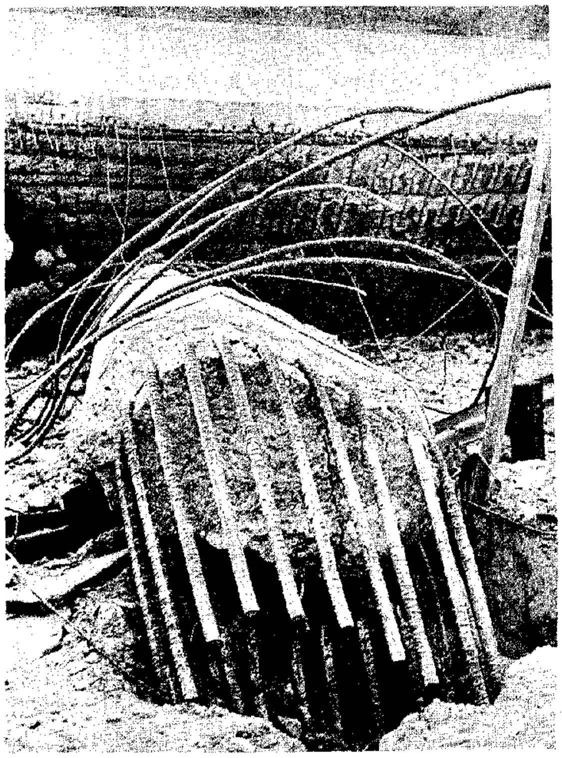


Fig. 10 Failure at Base of Column

Anchorage to Bent Caps or Superstructure

At the upper end of the column, moment capacity is needed between the column and the bent cap or bridge superstructure. The problem is illustrated in Fig. 11. The California Department of Transportation [22] initiated studies following the San Fernando earthquake regarding end anchorage of large bars in bridge decks or bent caps. An alternative to hooked bars was desired to reduce construction congestion problems. The results indicated that bars with anchorage devices (plates welded to splice sleeves) proved satisfactory. The bars with anchorage devices could be shortened with respect to required straight or hooked bar embedments. One significant feature of this work is the inferior performance of groups of anchored bars when compared with that of an individual bar.

Within the bent cap or the bridge superstructure the reinforcement details must also be evaluated to ensure continuity across the support and to provide moment capacity consistent with the design concept. If under large lateral deformation, hinging or inelastic behavior is desired in the column, the moment capacity of the bent cap must be greater than the column. If hinging in the column is not to be permitted, it is likely that substantial hinging in the bent cap will occur or the superstructure will be subjected to large bending and twisting forces and the moment capacities must be consistent with this behavioral concept.

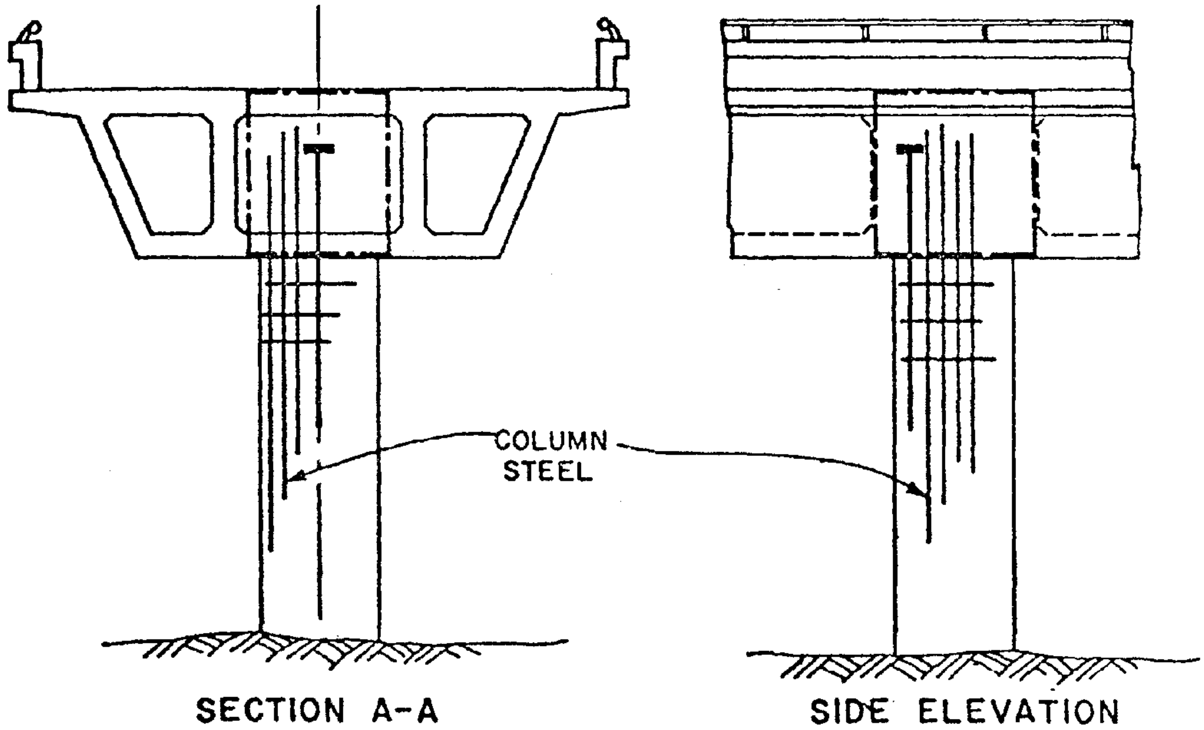


Fig. 11 Anchorage of Column to Superstructure

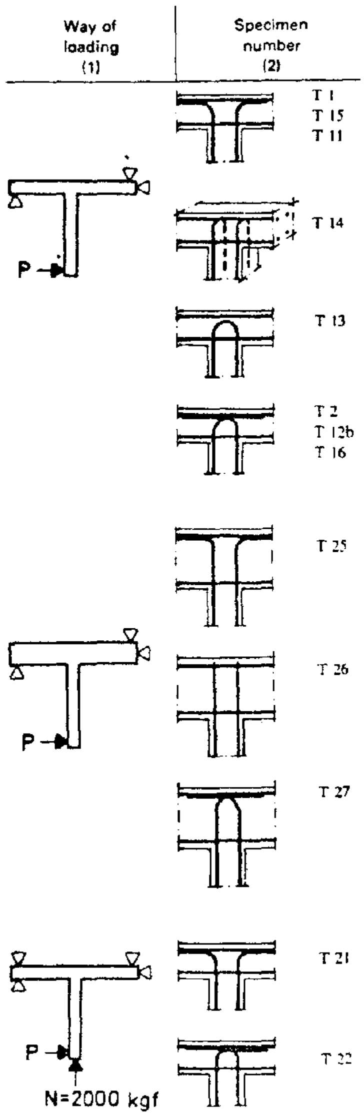
The joint detail in this case is a T-connection. Very little work has been done in this area for buildings because such joints occur only in the top story. Nilsson and Losberg [23] have tested joints of this type and have made recommendations regarding the reinforcement details which give the highest joint efficiency. The T specimens tested are shown in Fig. 12 and the details which proved most satisfactory were T2, T22, and T27. It should be noted that the specimen size was quite small with member dimensions not exceeding 30 cm.

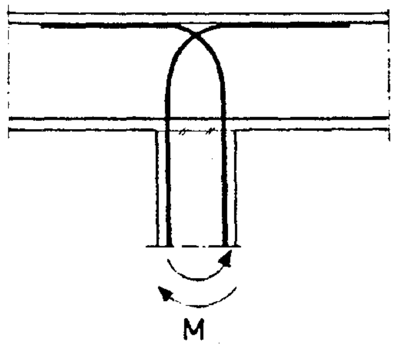
Splices

A review of the literature regarding anchorage and development of reinforcement indicates that while much work has been done that is applicable to buildings, the special joint and connection details make it difficult to apply the results directly to bridges. Additionally, the size of specimen and reinforcement used do not reflect typical bridge construction practice.

CONCLUDING REMARKS

Before any application of building subassemblage tests to bridge structures can be made, the type of loading and specimen used must be carefully evaluated. Considerable information is available on column behavior and much of it will be applicable to bridge structures; however, some tests must be run to ensure that the results of small scale tests can be extrapolated to large-sized elements. Because a bridge structure is not as redundant as most buildings, attention to details of connections between elements in a bridge





Layout of Reinforcement in T-Joint

Fig. 12 Outline of T Joints Tested [Ref. 23]

is critical. The types of connections common in bridges do not have direct counterparts in buildings and much larger bars are used in bridges so that research will be needed before design recommendations can be developed.

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SUMMARY OF EXPERIMENTAL AND ANALYTICAL SEISMIC RESEARCH RECENTLY PERFORMED ON HIGHWAY BRIDGES

by

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INTRODUCTION

A number of strong earthquakes hit Japan during these decades and caused extensive damages to modern engineering structures including bridges. Highway bridges sustained considerable damages during the Kanto Earthquake of 1923, the Nankai Earthquake of 1946, the Fukui Earthquake of 1948, the Niigata Earthquake of 1964, and also the Miyagi-ken-oki Earthquake of 1978. These experiences have encouraged bridge engineers to perform research works in the field of earthquake engineering associated with bridge designing.

This paper briefly summarizes recent research activities in Japan which are related to seismic effects on bridges, and derives further research subjects necessiated for pursueing rational seismic design for bridge structures.

INVESTIGATION OF DAMAGE TO BRIDGES

For getting better understanding of seismic effects on bridges and for pursueing a reasonable design method against seismic forces, it seems very important to investigate seismic damage to existing bridge structures due to earthquakes previously experienced.

Fig. 1 and Table 1 provide the brief information of eleven major earthquakes which caused comparatively severe damage to bridge structures since 1923. Details of seismic damage to bridge structures observed during nine earthquakes (except two recent ones in Table 1) are already summarized in the previous papers.[1),2)] In the following, therefore, damage characteristics caused by the two recent earthquakes (Izu Ohshima Kinkai Earthquake of January 14, 1978 and Miyagi-ken-oki Earthquake of June 12, 1978) will be summarized, and also general features of seismic damage to bridges will be briefly discussed.

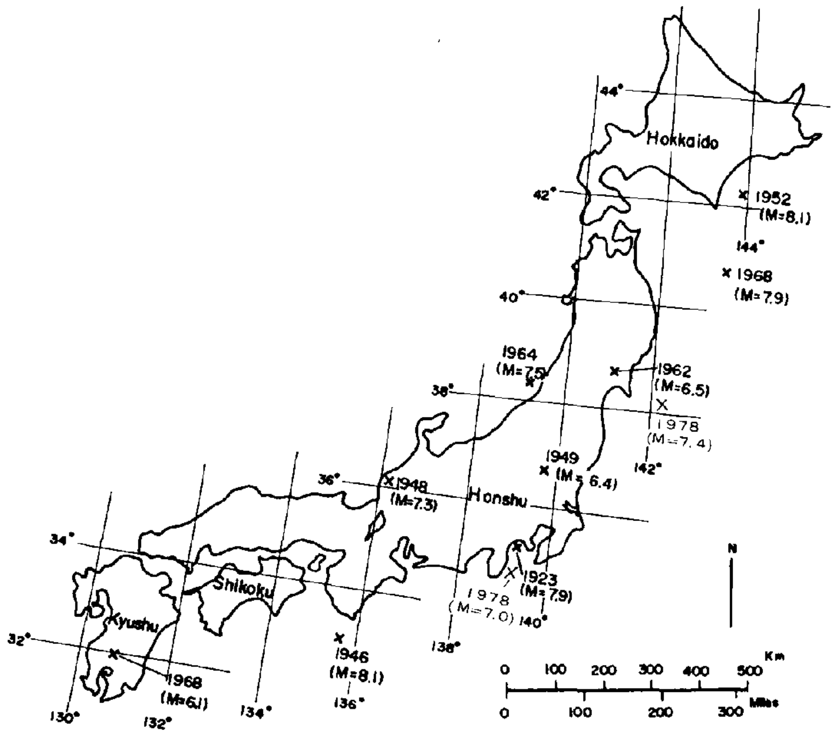


Fig.1 Epicenters of Eleven Earthquakes which Caused Comparatively Severe Damage to Bridge Structures in Japan (See Table 1)

Table 1. List of Eleven Major Earthquakes Causing Damages to Highway Bridges

No.	Date	Name	M*1	No. of Damaged Bridges	No. of Fallen Bridges	Amount of Loss of Bridges*3	Remarks
1	Sept.1,1923	Kanto	7.9	1,785*2	17*4	Unknown	
2	Dec.21,1946	Nankai	8.1	346	1	95,605 Thousand Yen	
3	Jun.28,1948	Fukui	7.3	243	7	207,651 "	
4	Dec.26,1949	Imaichi	6.4	1	0	minor	
5	Mar. 4,1952	Tokachi-oki	8.1	128	0	200,000 "	
6	Apr.30,1962	Northern Miyagi	6.5	187	0	43,000 "	
7	Jun.16,1964	Niigata	7.5	98	3	1,470,000 "	
8	Feb.21,1968	Ebino	6.1	10	0	50,000 "	
9	May 16,1968	Tokachi-oki	7.9	101	0	421,046 "	
10	Jan.14,1978	Izu Ohshima Kinkai	7.0	7	0	39,000 "	
11	Jun.12,1978	Miyagi-ken-oki	7.4	108	1	4,000,000 "	As of Dec, 1978
Total				3,014	29*4	-	

*1 Magnitudes are on the Richter scale, after either Annual Report of Science or Japan Metorological Agency.
*2 The number includes bridges damaged by fires (roughly 400 bridges).
*3 Amounts of loss are estimated at the time of each earthquake occurrence.
*4 The numbers include 9 bridges fallen due to fires.

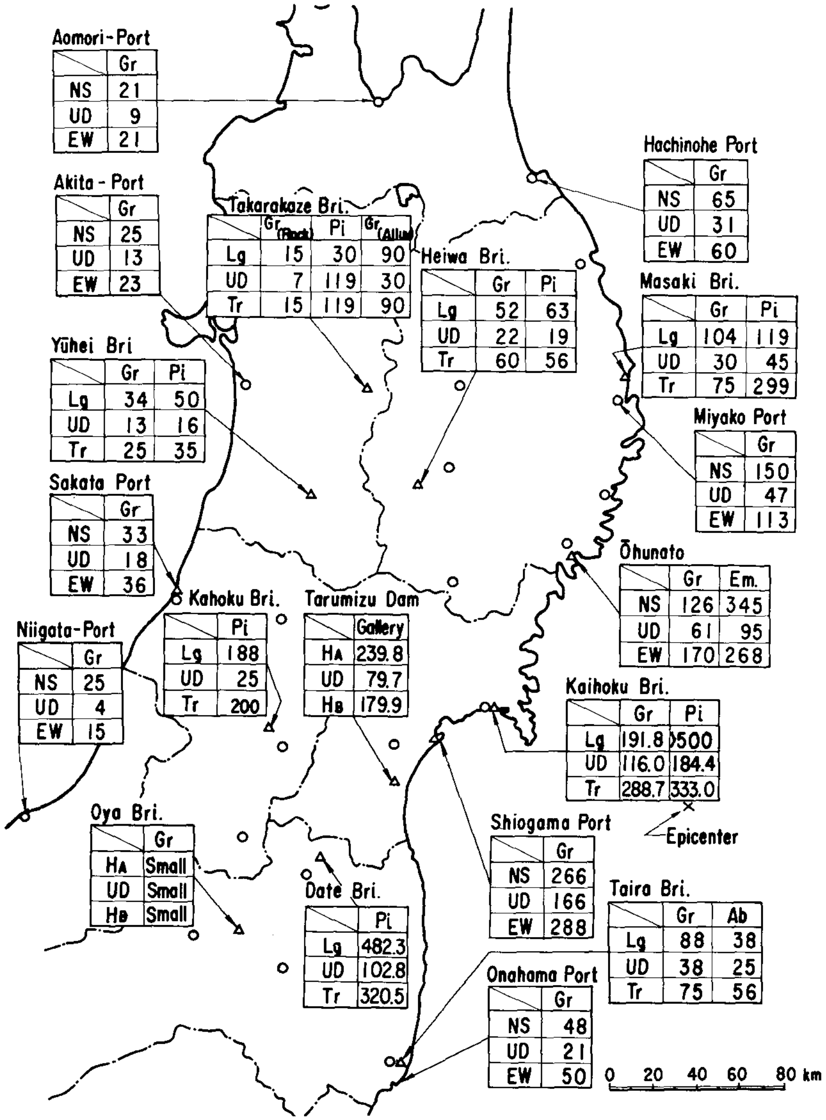


Fig.4 Distribution of Maximum Accelerations (in gals)

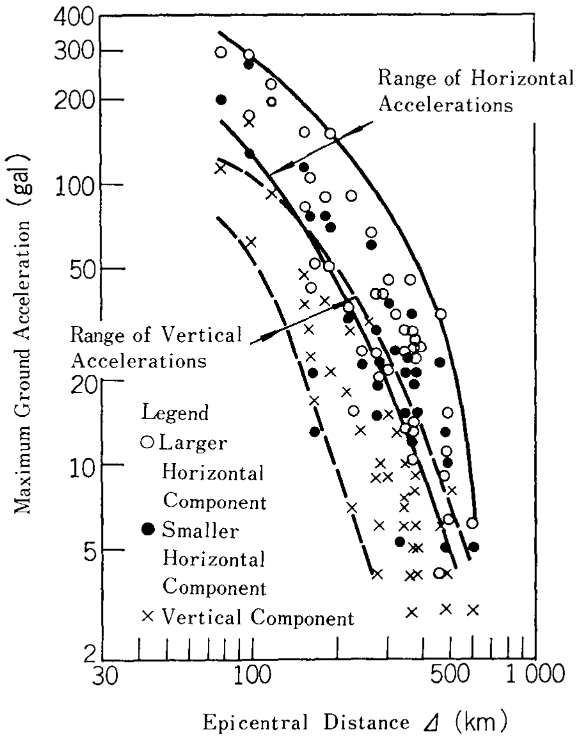


Fig.5 Relation between Epicentral Distance and Maximum Ground Acceleration

IZU OHSHIMA KINKAI EARTHQUAKE OF JANUARY 14, 1978

A severe earthquake hit the eastern part of Izu Peninsula, Shizuoka Prefecture on January 14, 1978, registering a Richter magnitude of 7.0. Although highways in the middle and eastern parts of the Peninsula were severly damaged due to a lot of large landslides, highway bridges sustained rather minor damage.[3)]

In the Shin-Shimoda Bridge (Shimoda City - fault distance of 13 km), it was observed that a free end of prestressed concrete girder translated horizontally with an amount of 7 cm toward the downstream and that due to this lateral movement concrete handrails were crushed at both sides since they were rigidly connected between abutments and the girders in order to place statues of mermaids (see Fig. 2). It was also observed that several concrete pieces at abutment parapets and near pier caps fell down at Shirata Bridge (Higashi Izu Town - fault distance of 5 km) and that a pier of Minato Bridge in Shimoda City settled approximately 0.5 m (see Fig. 2).

It should be also noted that two pedestrian overcrossing bridges did survive safely with minor cracks at parapets, although one of them was located only 20 m apart from the fault line (right lateral dislocation of 18 cm is seen on the highway pavement) in front of Inatori Junior High School (see Fig. 3). Ground motion at the site was estimated very severe (more than 400 gals) from the facts that several wooden residential houses were severely damaged and that most of tombstones were overturned near the site.

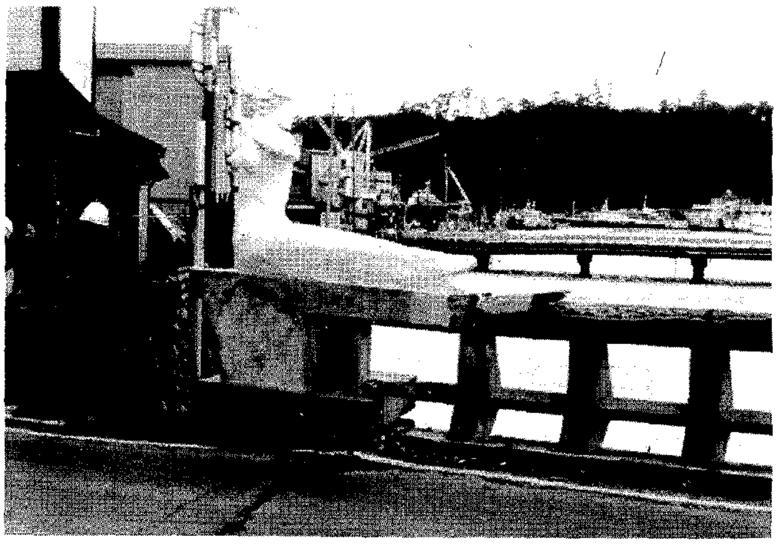


Fig.2 Failure of Concrete Handrail and Lateral Movement of Concrete Grider at Shin-Shimoda Bridge, Shimoda City. (Minato Bridge Settled is Seen in the Downstream)

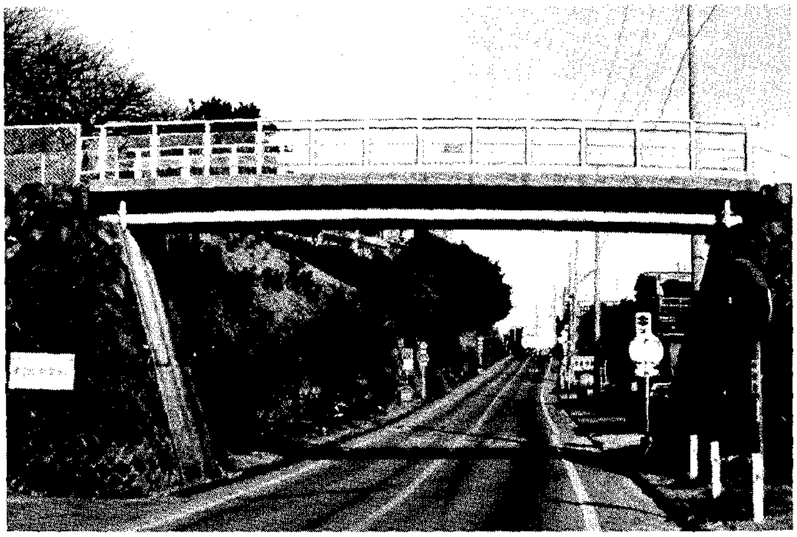


Fig.3 Overcrossing for Pedestrians Located Only 20m Apart from Fault near Inatori Junior Highschool at Inatori Area in Higashi-Izu Town, which Survived Safely against the Earthquake with Minor Cracks at Parapets.

MIYAGI-KEN-OKI EARTHQUAKE OF JUNE 12, 1978

Outline of the Earthquake[4),5)] — On June 12, 1978 a strong earthquake took place under the sea bottom approximately 120 km east of Sendai City, Miyagi Prefecture. The Japan Meteorological Agency has reported the magnitude of 7.4 on the Richter scale and the focal depth of 30 km. The earthquake caused very severe damages to various engineering structures including bridge structures. Due to the earthquake more than 100 highway bridges sustained structural damages. It should be also noted that another earthquake ($M = 6.7$) whose epicenter was approximately 60 km north from the June earthquake had hit almost the same area on February 20, 1978, and caused some minor damages to engineering structures.

Strong-Motion Records[4),5)] — During the Miyagi-ken-oki Earthquake of June 12, 1978 a number of strong-motion seismographs traced acceleration records at various stations in Hokkaido and northern Honshu. Fig. 4 indicates a distribution of maximum accelerations in Tohoku region. Fig. 5 shows a relationship between epicental distance and maximum acceleration. These acceleration records were obtained at structures such as highway bridges and port structures and on ground surfaces near the structures. Fig. 6 shows typical strong-motion records (by SMAC-B2 type accelerographs) which were obtained at Kaihoku Bridge located 80 km west from the epicenter. Response spectrum curves (amplification factor β = ratio of maximum absolute response acceleration to maximum ground acceleration) for two horizontal components of Kaihoku Bridge ground record, were computed as Fig. 7. Although very high accelerations (more than 500 gals) were measured on the pier top, Kaihoku Bridge did not sustain any structural damage. Only fixed bearing supports and oil dampers were slightly damaged to their anchor bolts.

Next, Fig. 8 shows a record at a pier cap of Date Bridge located near Fukushima City (epicentral distance of 160 km). The maximum accelerations on the pier cap were 480 gals in the longitudinal direction, and 320 gals in the transverse direction. This bridge suffered moderate damages to bearing supports and to a truss member above the fixed bearing.

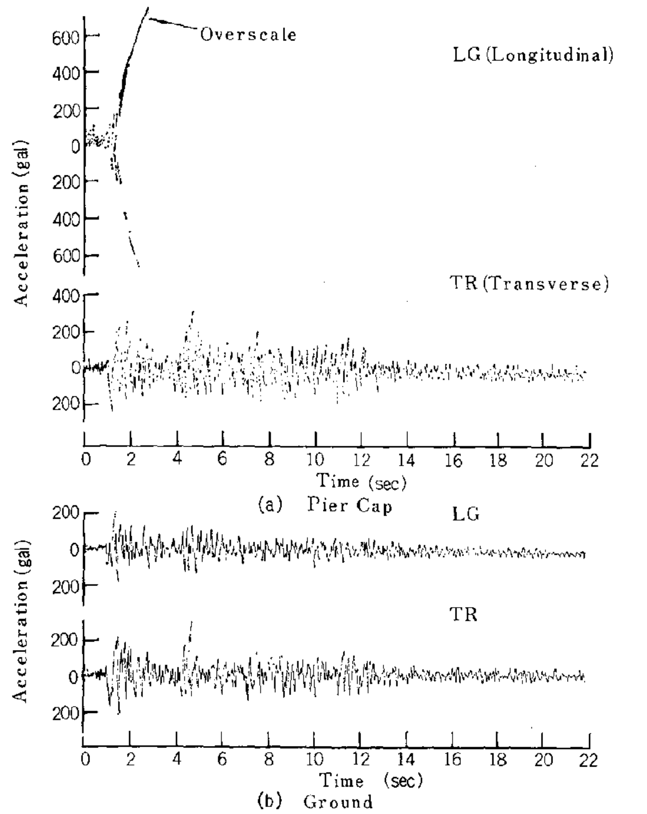


Fig.6 Strong-Motion Records at Kaihoku Bridge and Ground Nearby

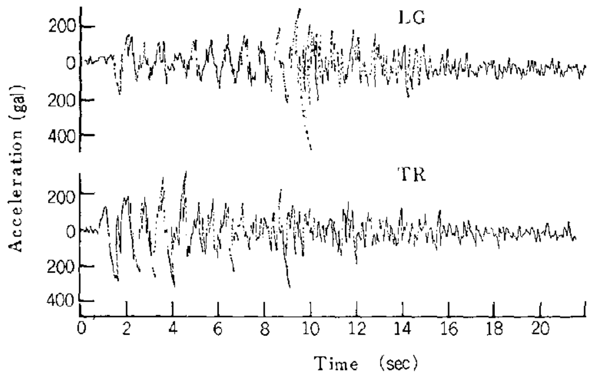


Fig.8 Strong-Motion Record at Cap of Pier 2, Date Bridge

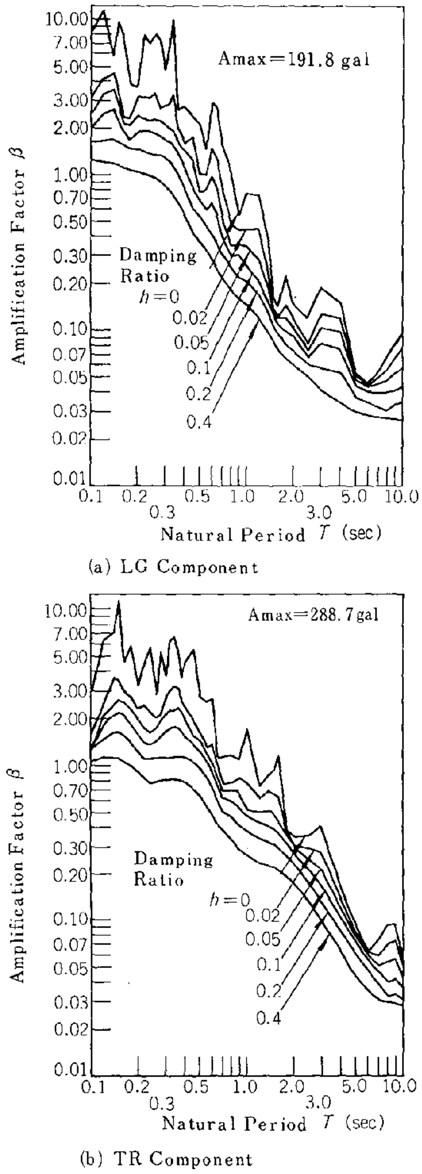


Fig.7 Response Spectrum Curves of Kaihoku Bridge Ground Record

Features of Bridge Damage[5] — Fig. 9 shows locations of severely damaged highway bridges (black circles), places where liquefaction was observed (white circles), and geological conditions in Miyagi Prefecture.

Numerals near black circles coincide with the number of bridges listed in Table 2, which describes briefly features of damages to highway bridges. It is seen from Fig. 9 that most of major bridge damages and liquefaction took place in alluvial lands along large rivers such as Kitakami, Naruse, Yoshida, Natori, and Abukuma.

Sendai Bridge[6] — Sendai Bridge, completed in 1965, is located in south part of Sendai City, and is crossing over Hirose River as a part of the National Highway No. 4. The general side view is shown in Fig. 10. Superstructures are 9-span simply supported composite steel-plate girders, with span length of 9 x 33.840 m, total length of 310 m, and width of 19 m. Substructures are T-shape columns (6.1 m high) founded on rigid well foundations (9 to 18 m deep) embedded into rather stiff sands. Bearing supports are of type of line bearings. Since this highway connecting Kanto and Tohoku regions is an important one, Sendai Bridge carries very heavy traffic (54,000 cars daily). As shown in Fig. 10, the lower half of the column height is embedded into higher river bed for three piers (P6 to P8). Column bases are above the surface of the lower river bed for other piers (P1 to P5).

Due to the earthquake (the epicentral distance to the bridge is $\Delta = 120$ km), all of the nine pier columns sustained damages. Piers 1 through 4 cracked horizontally at the column bases and surface concrete pieces separated heavily from the columns near the bases. Piers 5 through 8 had similar damages near the haunches which connect columns and beams. Pier 6 which has the lowest free height sustained the severest cracking at both sides (see Fig. 11). Concrete pieces separated at the haunch and reinforcing bars buckled. Near the haunch volume of reinforcing bars as well as concrete sectional area change rapidly. It is estimated that relative displacements between adjacent girders were 1 to 2.5 cm on the pier caps and that displacements at the pier caps of Piers 1, 2 and 6 were 11 to 18 cm.

Fig. 12 shows temporary frame works supporting the girders near Pier 6. Since the bridge is very important, damaged piers were repaired without stopping traffic even for a short time. Fig. 13 illustrates an example of permanent repairing work at Pier 6. Fig. 14 is a picture of Pier 6 after the repairing work was finished. The thickness of added concrete was 50 to 70 cm, and vertical reinforcing bars were fixed by epoxy adhesive into the well foundation and lateral bars were fixed to the columns. Moreover, chemical resin was placed into small cracks. It took only one month to completely repair the all damages to this bridge. An analysis of the causes of the failure of the pier columns are now undertaken at the Ministry of Construction.

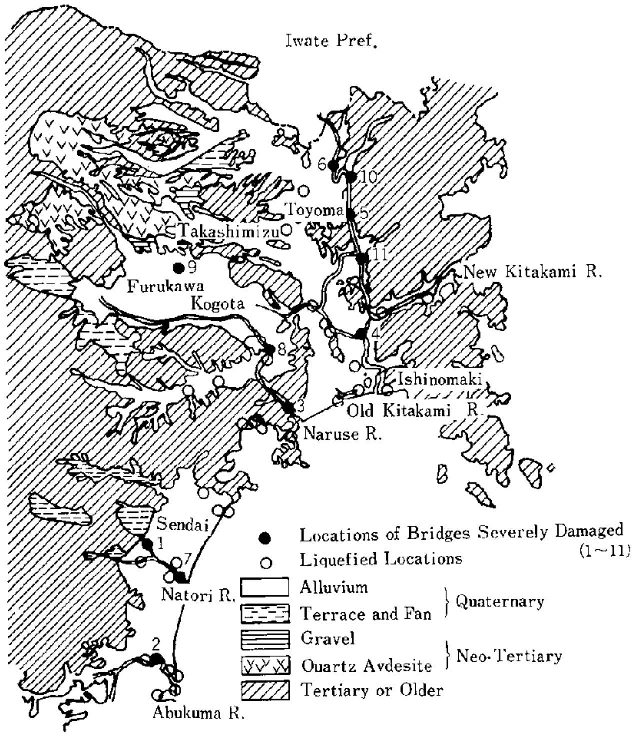


Fig.9 Geological Features and Locations of Major Bridge Damages and Liquefaction Sites

Fig.11 Failure of Pier 6, Sendai Bridge

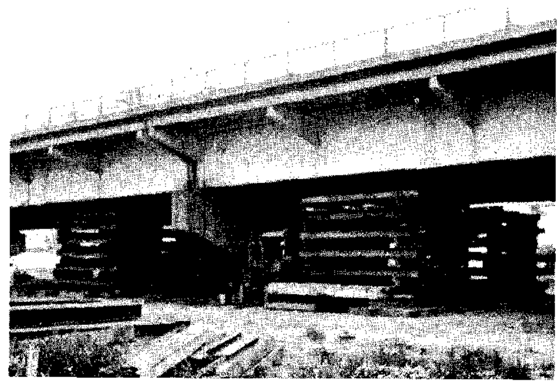


Fig.12 Temporary Frames Supporting Griders near Pier 6, Sendai Bridge

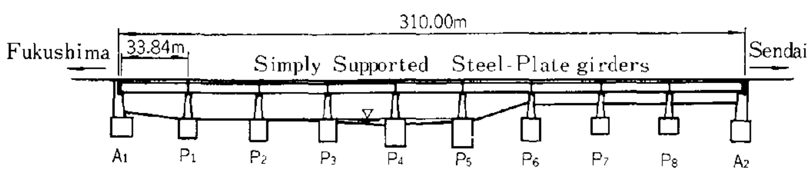


Fig.10 General View of Sendai Bridge

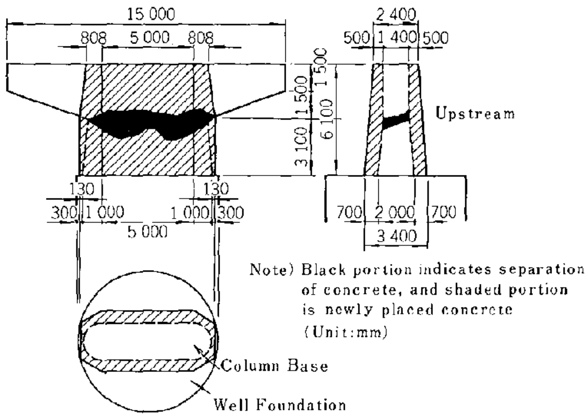


Fig.13 Damage and Repair Work of Pier 6, Sendai Bridge

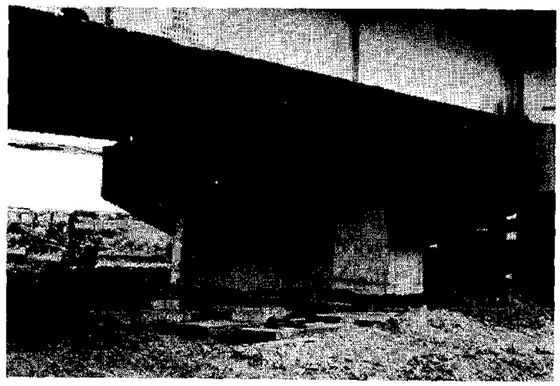


Fig.14 Pier of Sendai Bridge after Repair Work Completed

Table 2. List of Highway Bridges Severely Damaged in Miyagi and Fukushima Prefectures

No.	Bridge	Route	Characteristics of Bridges: Total Length (m)	Width (m)	Superstructure	Year Completed	Outline of Damages
1	Sendai	National Highway No.4	310.0	19.0	Composit Steel Plate Girder	1965	Horizontal Cracks at All Pier Columns
2	Abukuma	National Highway No.6	571.6	6.0	Steel Warren Truss, Simple Steel Pl. Girder	1932	Horizontal Cracks at Pier Columns
3	Ono	National Highway No.45	247.3	5.5	Simple Steel Pl. Girder	1936	Movement of All Girders, Failure of Bearing Supports, Failure of Slabs
4	Ten-noh	National Highway No.45	367.7	6.0	Steel Gerber Pl. Girder, Steel Langer Truss	1959	Vertical Crack at a Pier Column
5	Toyama	National Highway No.342	306.0	5.3	RC T-beam	1945	Cracks at Beams and Pier Columns
6	Kin-noh	National Highway No.346	575.5	6.0	1-Simple Steel Pl. Girder, 5-Steel Truss, Steel Gerber Pl. Girder	1956	Fall of a Suspended Girder, Failure of Bearing Supports
7	Yuriage	Miyagi Prefectural Highway	541.7	8.0	3-Hinged PC Girder, 7-Simple PC Post-Tension T-beam	1972	Cracks at Pier Columns, Liquefaction
8	Kimazuka	Miyagi Prefectural Highway	236.0	4.5	19-Simple Steel Pl. Girder	1931	Failure of Bearing Supports, Failure of Expansion, Liquefaction
9	Eai	Miyagi Prefectural Highway	155.0	7.5	9-Simple Steel Pl. Girder	1932	Cracks at Pier Columns
10	Maiya	Miyagi Prefectural Highway	181.4	5.5	Gerber Truss Girder	1928	Break of Upper Chord, Movement of Truss Girder
11	Yanaizu	Miyagi Prefectural Highway	450.0	8.5	1-Simple Steel Truss 2-Cont. Steel Truss 3-Cont. Steel Truss	1974	Failure of Bearing Supports, Crack and Buckling at Lower Truss Chord
12	Date	Fukushima Prefectural Highway	288.0	7.0	4-Cont. Steel Truss	1963	Failure of Bearing Supports, Buckling at Lower Truss Chords, Crack of Pavement

Kin-noh Bridge[7] —Kin-noh Bridge, completed in 1956, is on National Highway No.346, and crossing over Kitakami River. As shown in Fig. 15, the superstructures of the bridge are single-span steel plate girder, 5-span simply supported steel trusses, and 9-span Gerber-type steel plate girders from the left to right. The total length and the width are 575.5 m and 6.0 m, respectively. Substructures are RC columns on caisson foundations for the truss span, and RC columns on footing foundations with RC piles for the Gerber plate girder span. Soils are of soft silts and sands, and a firm sand layer exists approximately 30 m below the ground surface. During the June earthquake one girder of this bridge fell down (see Fig. 16). This Kin-noh Bridge is only one bridge which completely fell down during the June Earthquake.

This bridge was damaged three times by three different earthquakes, namely Northern Miyagi-ken Earthquake of 1962 ($M = 6.5$, $\Delta \fallingdotseq 15$ km), two Miyagi-ken-oki Earthquakes of February 20, 1978 ($M = 6.7$, $\Delta \fallingdotseq 80$ km) and of June 12, 1978 ($M = 7.4$, $\Delta \fallingdotseq 110$ km).

Due to the 1962 Earthquake[8] side blocks of bearing supports (oval line bearing) of Gerber girders failed, and concrete near the fixed bearing supports on the right-bank abutment cracked. After the 1962 Earthquake, a repairing work to add stiffening plates was undertaken at three piers (P8, P9, and P10) as shown in Fig. 17.

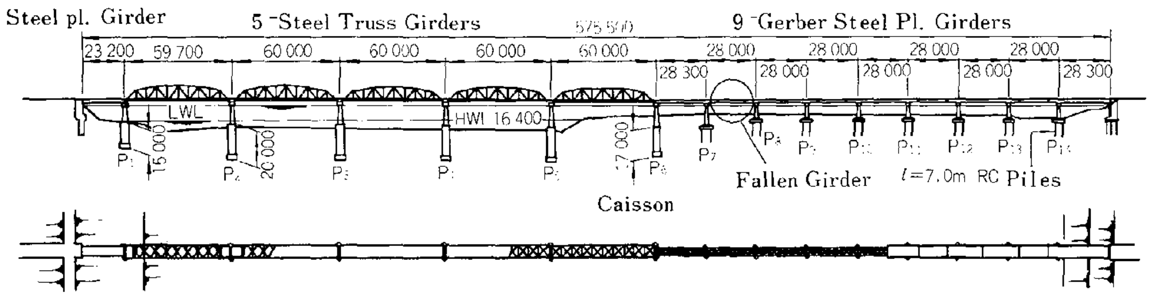


Fig. 15 General View of Kin-noh Bridge

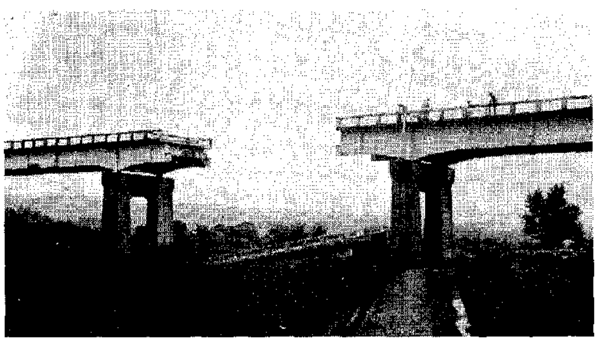


Fig. 16 Fall of a Suspended Girder, Kin-noh Bridge (During the Earthquake of June 12, 1978)

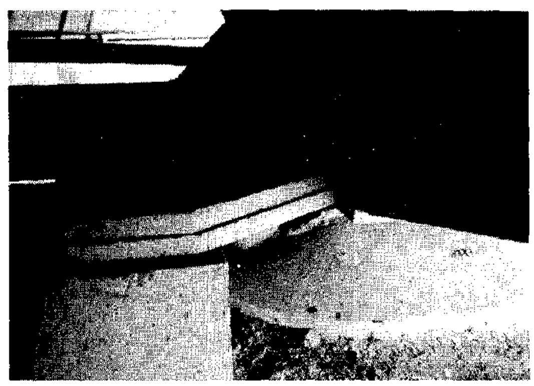


Fig. 17 Bearing Stiffening Plates to Resist Transverse Movement, Kin-noh Bridge (Added after 1962 Earthquake)

Anchor bolts of the bearing stiffening plates were cut off during the Earthquake of February 20, 1978. Side blocks of bearing supports, which did not sustain damages during 1962 Earthquake and therefore were not repaired, were also failed during the February Earthquake (see Fig. 18).

During the Earthquake of February 20, 1978, most of bearing supports at the truss girders also failed, in addition to the failure of bearing supports at the Gerber girders. As for the truss span, fixed bearing supports of pin-type were most severely damaged on Pier 6 (see Figs. 19 and 20). Fig. 19 shows the upstream support in which four anchor bolts were pulled out and bent unclockwise. It is supposed from Fig. 19 that the bearing would have rocked severely, rotated, and translated. Fig. 20 is the downstream support in which set bolts were sheared off. Most of anchor bolts of fixed bearing supports on other piers were also pulled out. Movable bearing supports of pin-roller-type were also failed. Fig. 21 shows protrusion of all rollers at the upstream movable bearing on Pier 5.

Since there were only four months after February 20 Earthquake, repairing works of these bearings were still undertaken at the time of June 12 Earthquake. Accordingly, all the girders were possible to move freely without any restraint by bearing supports.

A suspended girder between Piers 7 and 8 fell down on the river bed, as shown in Figs. 15 and 16. The superstructure moved toward the right-bank side by 55 cm on the top of Pier 8 (see Fig. 22). Since the upper support dislodged from the bearing and lower flange supported the dead weight of the girder, a local buckling took place at the web of the girder. All the Gerber span between Pier 8 and Right Abutment moved toward the right-bank. The girder moved 10 cm toward the right on the right-bank abutment, and the end of girder collided into the parapet of the abutment (see Fig. 23). The asphalt pavement of the backfill heaved due to the collision (see Fig. 24).

On the other hand, truss girders were also heavily damaged during the June Earthquake. Figs. 25 and 26 are pictures taken after the June Earthquake at the same places as Figs. 19 and 20, respectively. Anchor bolts of the upstream fixed bearing at Pier 6 were much severely pulled out by about 20 cm at

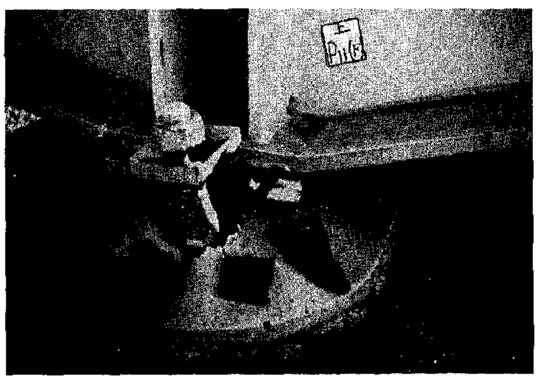


Fig. 18 Failure of Side Block of Oval Line Bearing on Pier 11, Kin-noh Bridge (Just after February 20, 1978)

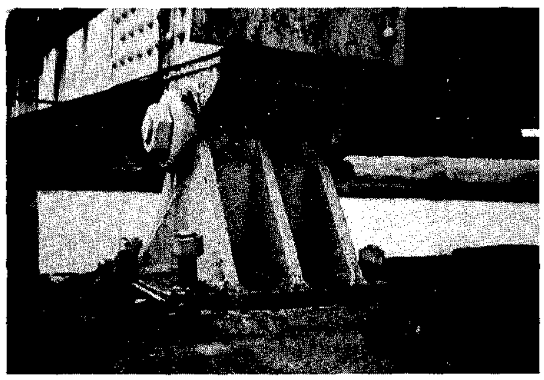


Fig. 19 Pull-out of Anchor Bolts at Upstream Fixed Pin-type Bearing on Pier 6, Kin-noh Bridge (Just After February 20, 1978)

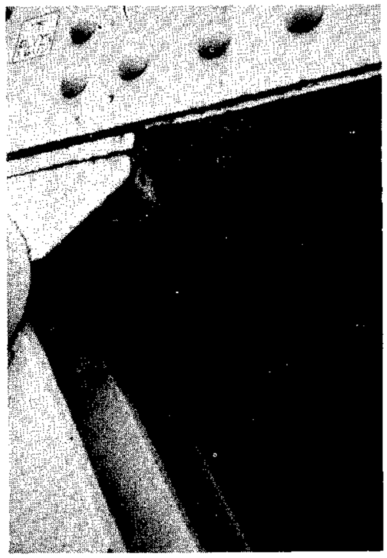


Fig. 20 Failure of Set Bolts at down-stream Fixed Pin-type Bearing on Pier 6, Kin-noh Bridge (June After February 20, 1978)

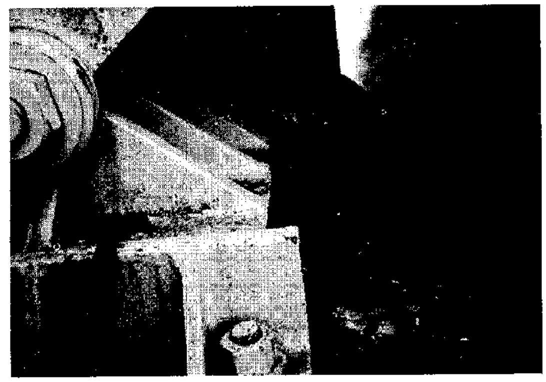


Fig. 21 Protrusion of Rollers at the Movable Pin-roller-type Bearing on Pier 5, Kin-noh Bridge (Just after February 20, 1978)

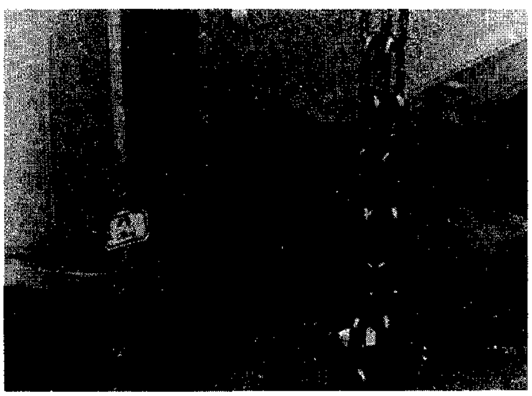


Fig. 22 Movement (55 cm) of Plate Girder at the Downstream Support on Pier 8, Kin-noh Bridge (After June 12, 1978)

Fig. 23 Failure of the Right-Bank Abutment Girder Moved Toward Abutment Kin-noh Bridge by 10 cm, Temporary Support is seen. (After June 12, 1978)

Fig. 24 Heaving of Asphalt Pavement at the Backfill of the Right Abutment, Kin-noh Bridge (After June 12, 1978)

Fig.25 Pull-out of Anchor Bolts and Settlement of the Shoe at the Upstream Fixed Bearing on Pier 6, Kin-noh Bridge. (After June 12, 1978)

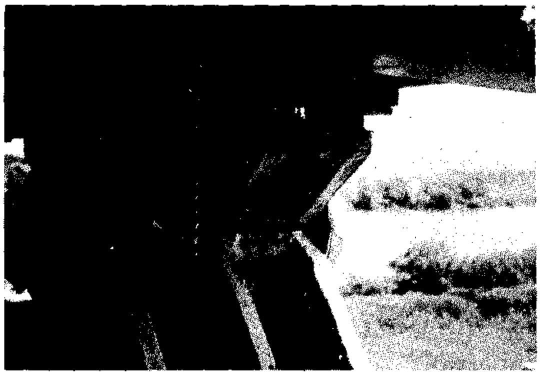


Fig.26 Failure at Downstream Fixed Bearing on Pier 6, Kin-noh Bridge (After June 12, 1978)

Fig.27 Failure of Upstream Movable Bearing on Pier 5, Kin-noh Bridge (After June 12, 1978)

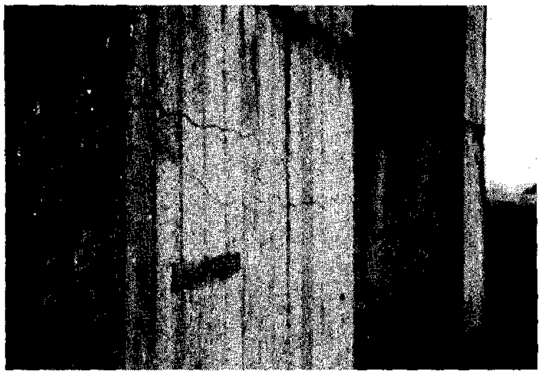


Fig.28 Cracks of Sides of Columns of Pier 8, Kin-noh Bridge (After June 12, 1978)

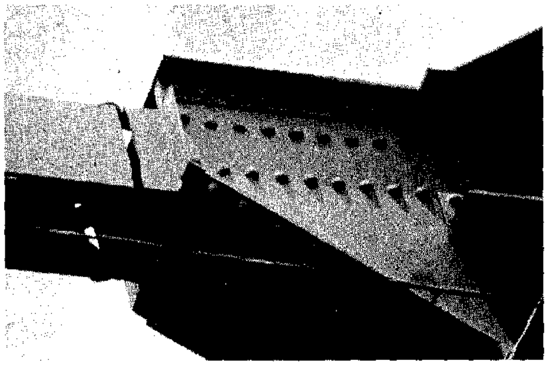


Fig.29 Breakage at Upper Truss Chord, Maiya Bridge

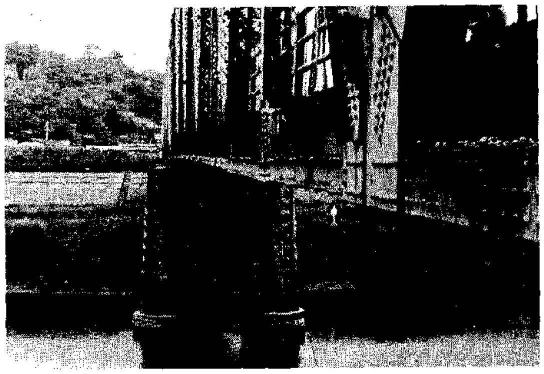


Fig.30 Drooping of the Lower Truss Chord, Maiya Bridge

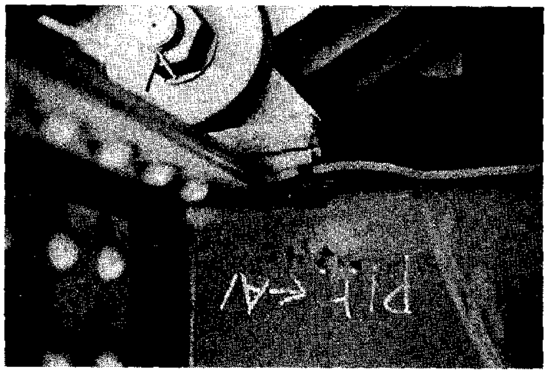


Fig.31 Damage to Lower Flange of Truss Chord near a Fixed Pin-type Bearing, Yanaizu Bridge

most (compare Figs. 25 and 19) presumably due to rocking and translation motions of the bearing, and some concrete underneath the lower bearing plate was taken out and the bearing sunk by 2.5 cm. As for the downstream fixed bearing on Pier 6 (Fig.26), a deformed bar which had been used as a temporary set bolt after the February Earthquake was sheared off again. The key of the upper shoe dislodged from the sole plate, and the sole plate deformed. As for pin-roller-type movable bearings, rollers had rolled out of the shoes during the February Earthquake. Fig. 27 shows the state after the June Earthquake at the upstream movable bearing on Pier 5 whose rollers completely had rolled out.

As for pier columns, only the right-bank side of Pier 8 sustained heavy cracks (see Fig. 28). It is estimated that these cracks would have taken place when the superstructure collided with the right abutment and the reaction toward the left bank applied to the pier.

To grasp the causes of the damage to Kin-noh Bridge, the Miyagi Prefecture is now conducting comprehensive studies including field surveys and dynamic analyses, with a cooperation of the Public Works Research Institute.

Maiya Bridge[9] — Maiya Bridge, completed in 1928, is crossing Kitakami River 4 km downstream from Kin-noh Bridge. Superstructures are of 3-span Gerber-type steel truss girder with the total length of 181.4 m and the width of 5.3 m. Two abutments have footing foundations, and two piers have well foundations. Due to the June Earthquake a upper chord member was broken off as shown in Fig. 29. Due to the breakage the lower chord drooped considerably (see Fig. 30). The chord member was made of steel channels and broken near the sudden sectional change as shown in Fig. 29. At this bridge a pin came out of the fixed pin-type bearing.

Toyoma Bridge[7] — Toyoma Bridge, completed in 1945, is also crossing Kitakami River 6 km downstream from Maiya Bridge. The superstructures are Gerber-type RC T-shape girders, with the total length of 306 m. The pier columns are on well foundations. Due to the June Earthquake many heavy cracks occurred at the mid-point of several girders, and at webs above the bearing supports. A heavy crack also broke out at the base of the pier column closest to the right bank.

Yanaizu Bridge[7] — Yanaizu Bridge, completed recently in 1974, is crossing Kitakami River 6 km downstream from Toyama Bridge. The superstructures are 6-span steel truss girders, with the total length of 450 m and the width of 8.5 m. The substructures are of RC columns on steel-pipe pile foundations. A lower chord was damaged at the fixed pin-type bearing, as shown in Fig. 31. Due to the seismic forces set bolts between the upper shoe and the lower chord were cut off, the welding between the uppershoe and sole plate detached, the web plate deformed, and painting came off. Substructures and bearing supports of this bridge did not sustain any damage.

Kimazuka Bridge[7] — Kimazuka Bridge, completed in 1931, is crossing Naruse River in Kashimadai Town. The superstructures are 19-span simple supported steel plate girders, with the total length of 236 m, and the width of 4.5 m. Two abutments are on footing foundations, 18 piers are on well foundations. Due to the June Earthquake, one pier cap was severely damaged near the bearing (see Fig. 32), and one girder was almost dislodging from the pier cap (see Fig. 33). The girder moved also in the transverse direction.

Eai Bridge[7] —Eai Bridge, completed in 1932, is crossing Eai River in Furukawa City. The superstructures are 9-span simply supported steel plate girders, with the total length of 155 m and the width of 7.5 m. Two abutments are on pile foundations, and each of 8 piers is on two separate well foundations with diameter of 2.5 m and depth of 7 m. At the time of this earthquake, the embedment of well foundations was almost half of the initial depth due to scoring effects. Locating close to the epicenter of the Northern Miyagi Earthquake of 1962 (epicentral distance was approximately 15 km), this bridge had been damaged to bearings in 1962. Due to the June, 1978 Earthquake lower beams of the eight pier columns severely cracked, and concrete pieces separated from the beams (see Fig. 34). The largest opening of the cracks was 20 mm, and reinforcing bars appeared.

Yuriage Bridge[6] —Yuriage Bridge, completed rather recently in 1972, is crossing over Natori River near its mouth. The superstructures are of 3-span continuous PC box girders with a center hinge (cantilever erection) and of 7-span simply supported post-tension PC beams (T-shape) with the total length of 541.7 m and the width of 8 m. Two abutments are on steel pipe pile foundations, two piers in the lower river bed are on pneumatic caisson foundations, and 7 piers are on well foundations. Due to the June Earthquake the nine pier columns sustained many cracks (almost all around) mostly at the level of the ground surface. Especially Pier 1 (first pier from the left-bank) sustained numerous heavy cracks (see Fig. 35), and concrete pieces separated from the column. Stoppers of single-roller-type movable bearing on Pier 1 were damaged, guide pieces of the bearing failed, and the roller was almost rolling down from the lower shoe (see Fig. 36).

A simply supported PC beam on Pier 6 moved 6 cm toward downstream. The ends of one handrail were inserted into the ends of another handrail just above a pier. The length of insertion had been 8 cm. During the Earthquake the handril ends completely came out of the adjacent handrail ends. It is understood from this that the two adjoining beams vibrated relatively at least 8 cm in the longitudinal direction.

A number of ground cracks and sand boils were observed on the higher river bed near the right bank. The subsoils made of mostly sands are loose near the surface (about 5 m deep) and medium to dense underneath. A hard layer exists approximately 70 m below the surface.

The Public Works Research Institute, the Ministry of Construction is investigating the causes of the damage to Yuriage Bridge with emphasis on the effects of liquefaction on the behavior of the bridge.

Date Bridge[4),5] —Date Bridge, completed in 1963, is crossing over Abukuma River near Fukushima City, with a epicentral distance of approximately 160 km. Superstructures are 4-span continuous steel truss girders, with the total length of 288.0 m and the width of 7.0 m. The two abutments are on steel-pipe pile foundations, and the three piers are of tall RC columns on caisson foundations embedded into gravel and sand layers.

Due to the June Earthquake a lower chord member buckled just at the fixed bearing on Pier 2 (see Fig. 37). Several pins at the fixed bearing and one of the movable bearings were sheared off and came out of the shoes. The substructures did not sustain any damage.

Fig.32 Failure of Concrete near a Bearing, Kimazuka Bridge

Fig.33 A Girder almost Dislodging from a Pier Cap, Kimazuka Bridge

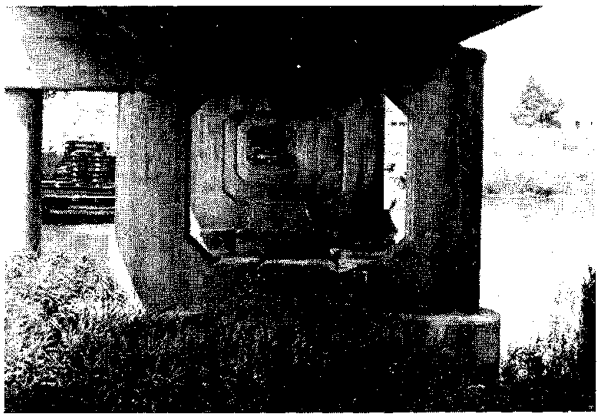


Fig.34 Cracks at Pier Columns, Eai Bridge

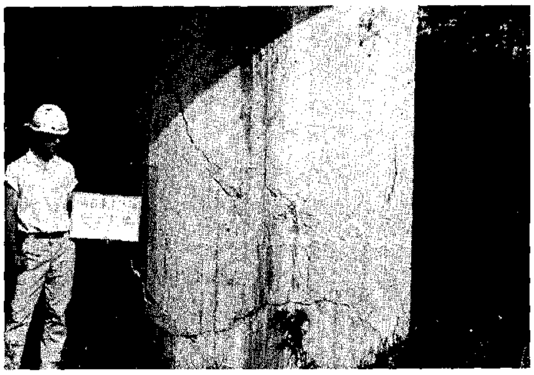


Fig.35 Cracks at Pier 1, Yuriage Bridge

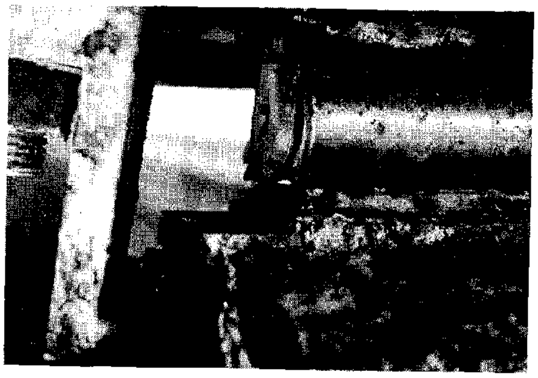


Fig.36 Failure of Guide Piece at Movable Bearing on Pier 1, Yuriage Bridge

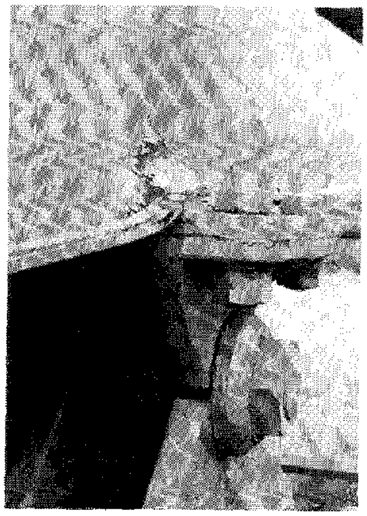


Fig.37 Buckling of Lower Chord Member Above the Fixed Pin-Bearing on Pier 2, Date Bridge

A strong-motion accelerograph is set up on the cap of Pier 2, and triggered a complete time history of the acceleration at the pier cap (see Fig. 8). It is regretable that another accelerograph on the ground surface nearby did not get a strong-motion record, because of lack of paper. The Public Works Research Institute is analyzing dynamic behavior of Date Bridge with use of the record.

Lessons from Bridge Damage due to the Miyagi-ken-oki Earthquake[5),7)]

In view of the damages to bridge structure during the Miyagi-ken-oki Earthquake of June 14, 1978, the following lessons can be derived,

1) Damages to superstructures concentrated on bearing supports and adjoining portions. On the other hand, most damages to substructures were cracks and separations at of concrete at pier columns and abutments.

2) Damages to bearing supports were most frequently observed. It is advisable to carefully investigate design practices of bearing supports and to develop better bearings which are properly strong against seismic disturbances. It seems, however, that breakage of bearing supports have reduced failure of bridge girders and also failure of substructures. Therefore, it is not always advised to design too strong bearings.

3) Because of causing the extensive damages to the whole bridge structure, fall of bridge girders should be avoided.

4) A number of older bridges such as Kin-noh, Maiya, Toyoma, Kimazuka, Eai Bridges sustained relatively severe damages. In most of these bridges, either Gerber-type or simply supported type is used, the width of pier caps is narrower, and no special consideration to prevent from falling girders is introduced. It seems very important to retrofit these older bridges by widening pier caps, installing devices to prevent girders from falling, etc.[9)]

5) In view of the damages to pier columns at Sendai and Yuriage Bridges which were recently constructed according to the current specifications, it is recommended to consider ductility of pier columns when designing short reinforced concrete columns. In this respect further experimental and analytical researches are necessitated, and future seismic specifications should include an appropriate regulation on ductility of pier columns.

GENERAL FEATURES OF SEISMIC DAMAGE TO BRIDGES

Seismic damage to bridge structures generally observed at supports, abutments, piers, and girders which are consisting of a complete bridge structure (Fig.38). The mechanisms of the bridge damage can be classified into three categories.[1),2)]

1) Due to the Weakness of Supports — Any portions of a bridge structure will be exerted to move in any directions during earthquakes. If the supports are not sufficient to undergo the differential movements between the superstructures and the substructures, they may fail. If the failure of a support occurs, the superstructures will move extensively relative to the caps of the substructures. Occasionally the superstructures may dislodge from the substructures and fall down into the river or on the underpass. Both the superstructures and the substructures will suffer considerable damage by the strong shock of the fall of the superstructures.

In view of the experiences of seismic damage due to the recent earthquakes, it seems reasonable that bridge structures which were properly designed and constructed in accordance with the bridge specifications (Specifications for Earthquake-Resistant Design of Highway Bridges, by Japan Road Association published in 1971) can resist major earthquakes without sustaining fatal damage, provided that they were designed with a special attention to (1) geological consideration to evade catastrophic ground disasters, (2) decrease in bearing capacities of the subsoils during earthquakes (such as liquefaction of sandy soils), (3) the design details for preventing the fall of girders and for avoiding severe damage caused by the failure at supports, and (4) the sufficient ductility for preventing cracking at pier columns especially at short rigid reinforced concrete columns.

On the basis of the viewpoint, new specifications[10] for seismic design of highway bridges are now under preparation by the JRA, with a special request of the Ministry of Construction.

CONSIDERATION ON SOIL LIQUEFACTION

Any bridge superstructures are supported by the substructures with the foundations which rest on the subsoils. As described in the previous papers,[1), 2)] soil liquefaction often cause severe damages to bridge structures. It is very important, therefore, to assess the possibility of occurrence of liquefaction and also to evaluate the effects of liquefaction on dynamic behavior of bridge structures. The following briefly introduces a practical method for assessing soil liquefaction potential and also a laboratory test regarding the effects of liquefaction on a bridge foundation.

PRACTICAL METHOD FOR ASSESSING SOIL LIQUEFACTION

The Public Works Research Institute, Ministry of Construction has proposed a simplified method for assessing soil liquefaction potential of sandy soils[11)].

Outline of the Method — In this method an ability to resist the occurrence of liquefaction of a soil element at an arbitary depth can be expressed by the liquefaction resistance factor (F_L)

$$F_L = \frac{R}{L} \qquad (1) \qquad (1)$$

In eq.(1) R is the in situ resistance (or dynamic strength) of the element (see Figs. 39 and 40), and can be simply estimated by

$$R \fallingdotseq R_\ell \fallingdotseq \begin{cases} 0.0882\sqrt{\dfrac{N}{\sigma_v'+0.7}} + 0.225\,\ell og_{10}\left(\dfrac{0.75}{D_{50}}\right) & \text{for } 0.02 \leqq D_{50} \leqq 0.6\text{ mm} \\ 0.0882\sqrt{\dfrac{N}{\sigma_v'+0.7}} - 0.05 & \text{for } 0.6 < D_{50} \leqq 2\text{ mm} \end{cases} \qquad (2)$$

where N = number of blows by the standard penetration test in situ, σ_v' = effective overburden pressure (in kgf/cm^2), and D_{50} = mean particle diameter (in mm). In eq.(1) L is the dynamic load due to an earthquake motion (see Fig. 39) and can be estimated by

$$L = \frac{\tau_{max}}{\sigma_v'} \div \frac{(\alpha_s)_{max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \qquad (3)$$

where τ_{max} = dynamic shear stress induced by the earthquake motion, $(\alpha_s)_{max}$ = maximum acceleration on ground surface (in gals), g = acceleration of gravity (= 980 gals), σ_v = total overburden pressure, and r_d = reduction factor for dynamic shear stress accounting for deformation characteristics of the ground. As for r_d, an average relation

$$r_d = 1 - 0.015Z \quad (Z : \text{depth in m}) \qquad (4)$$

can be assumed for alluvial deposits (see Fig. 41). From a spacial distribution of F_L-value obtained by eq.(1) for the site concerned, liquefaction potential can be evaluated. Fig. 42 shows an example of F_L-values with depth at a site in Niigata City where liquefaction was not observed during the Niigata Earthquake of 1964. It is seen from the figure that F_L-values are mostly larger than 1.0. On the other hand, Fig. 43 indicates F_L-distribution at another site in Niigata City where extensive liquefaction took place during the same Earthquake. It can be seen that F_L is generally less than 1.0 for the liquefied zone estimated. Fig. 44 summarizes the result (F_L-Z relationship) of the similar analyses for various sites in Niigata. Small black dots denote the depths where liquefaction took place, and open circles show the depths where liquefaction did not take place. The ranges of liquefied depths were estimated either by behavior of foundations or comparison of properties of boiled sands and those of individual layers. It can be seen from Fig. 43 that most points having F_L-values less than 1.0 liquefied, and most points with F_L-values greater than 1.0 did not liquefy. This tendency is especially noted for the depth shallower than 10 m. It can be concluded from this fact that the simplified method can be used for evaluating approximate liquefaction occurrence, although future modification may be necessitated.

It is obvious that damages to structures caused by soil liquefaction are considerably affected by the degree of liquefaction at each site and by the depth of liquefied zones. Damages to bridge structures were studied in comparison with the distribution of F_L-values for the Niigata Earthquake. It is found from this study that bridge foundations were severely damaged only where the thickness of layer whose F_L-values are much less than 1.0 (say less than 0.6) is thicker than 5m.

The concept briefly described in the above is proposed in the previous paper[11] in details, and will be introduced in the JRA's new specifications for earthquake-resistant design of highway bridges[10].

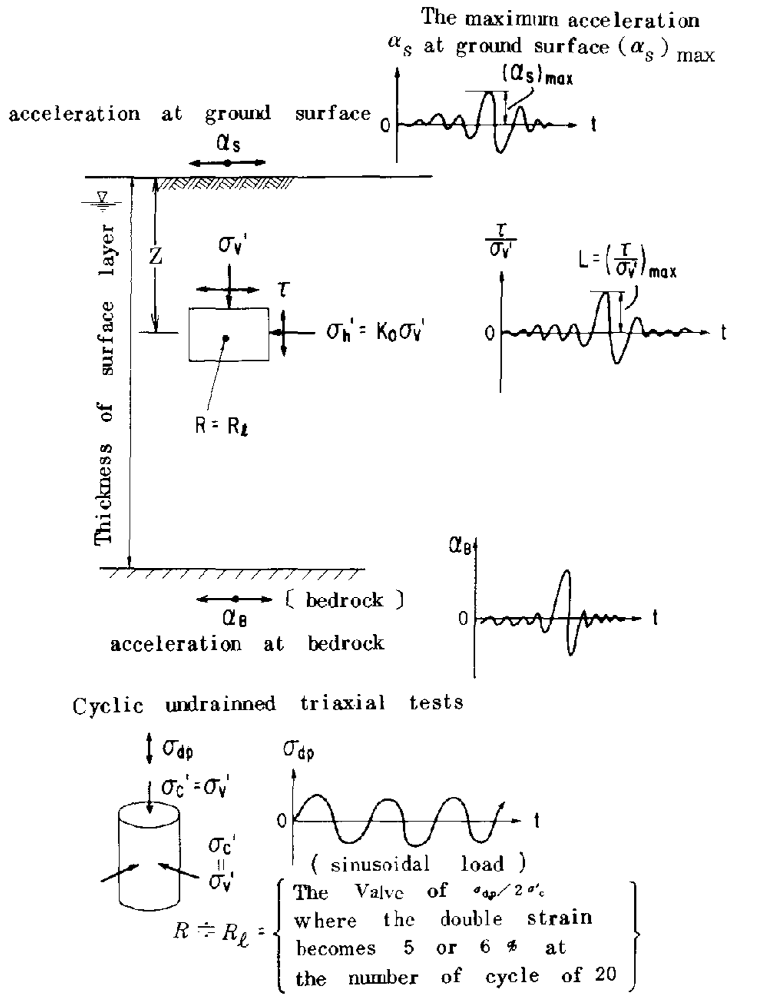


Fig.39 Definitions of Earthquake Induced Stress L and Dynamic Strength R

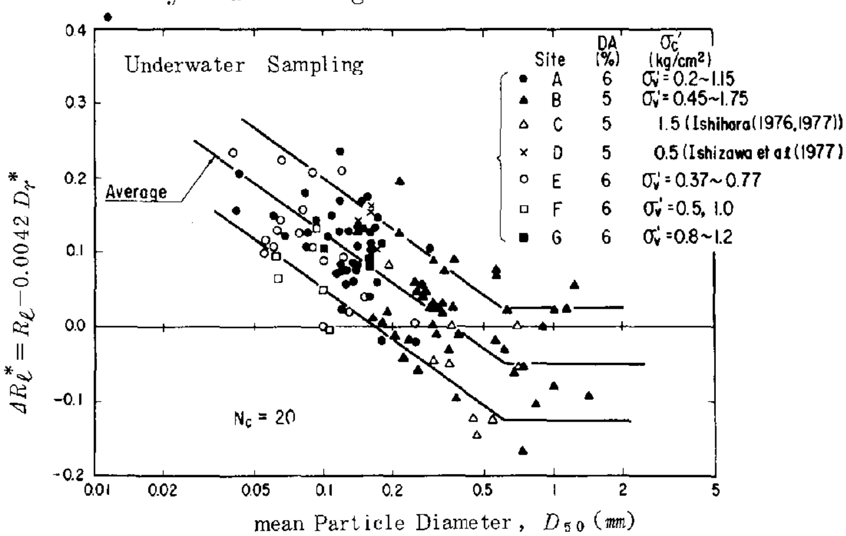


Fig.40 ΔR_ℓ^* and D_{50} Relation

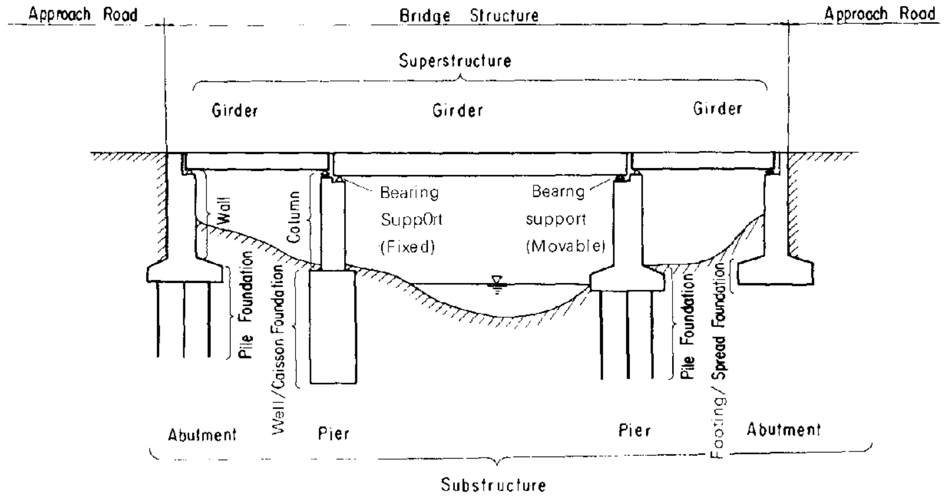


Fig.38 Schematic Sketch of a Typical Highway Bridge

2) Due to the Weakness of Substructures —If a substructure is not sufficient to resist its own inertia force and seismic forces of the girders transmitted through the supports, it may crack, deform, and sometimes fail or even overturn completely. The superstructures supported by the substructure will sustain considerable damage caused by the lack of the resistance of the substructure.

3) Due to the Weakness of Surrounding Soils —If the soils surrounding a substructure are vulnerable to earthquake excitations, the substructure may settle vertically or move horizontally during earthquakes. An extensive decrease in the bearing capacities of the surrounding soils is often observed at loose saturated sandy soils due to liquefaction during earthquakes. If a drastic movement of substructure occurs due to the weakness of soils, the superstructures supported by the substructure can not keep their initial positions and may sustain considerable damage, and sometimes even fall down.

As a result of the behavior described in the above the following failures are often observed at individual portions of a bridge structure.

Substructures: Tilting, settlement, sliding, cracks, overturning.

Superstructures: Movement, buckling, crack or failure, fall of girders.

Supports: Failure of bearings, cut-off or pull-out of anchor bolts.

Appurtenant Structures: Settlement of approach roads (especially in case of banking), settlement and sliding of wing walls, separation of wing walls from abutments, and failure of parapet walls.

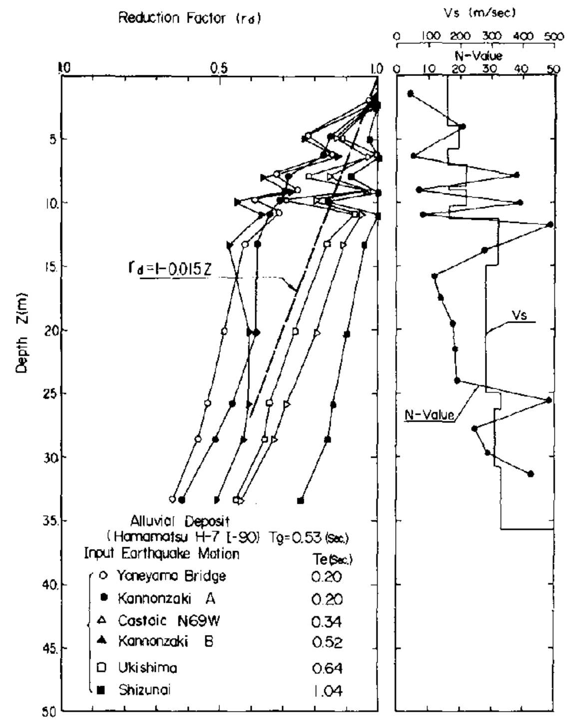


Fig.41 Computed Reduction Factor for an Alluvial Deposit

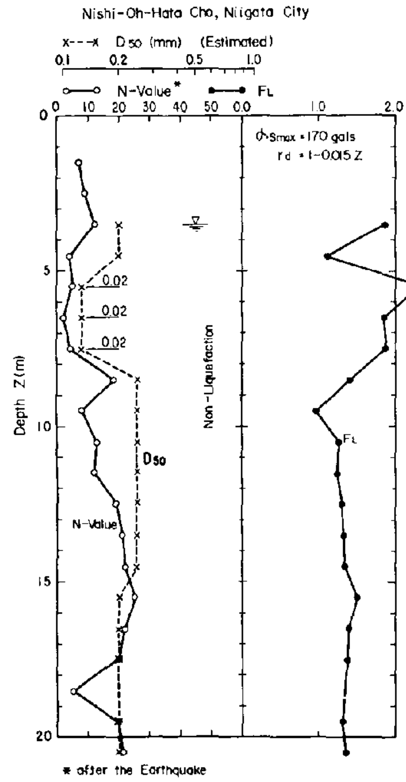


Fig.42 F_L at a Non-Liquefied Site

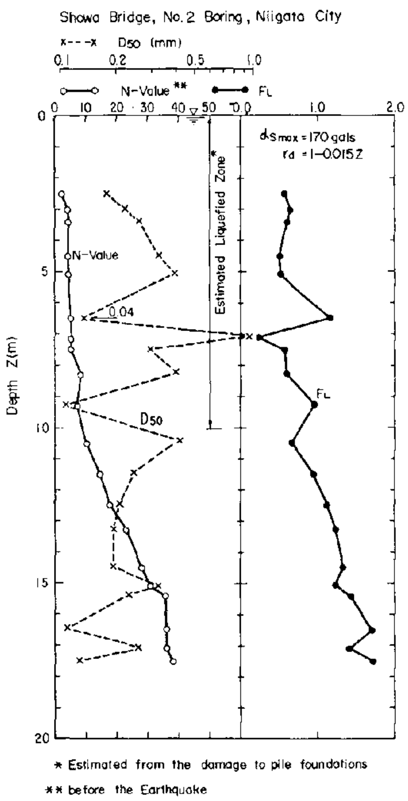


Fig.43 F_L at a Liquefied Site

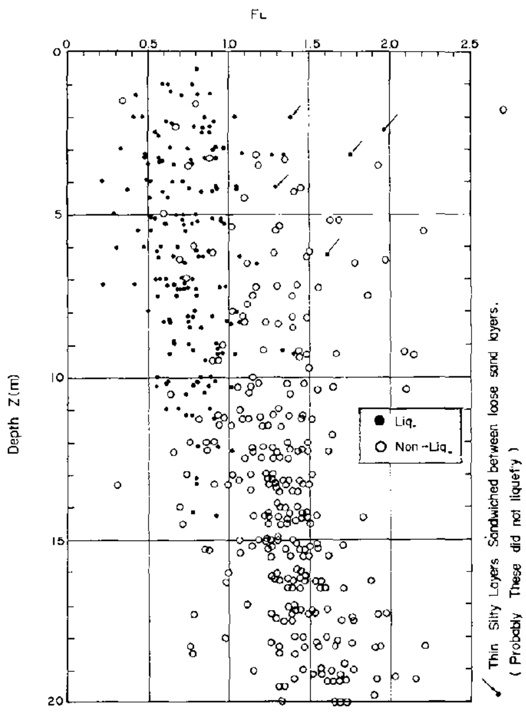


Fig.44 F_L at Several Sites in Niigata City during the Niigata Earthquake

EFFECTS OF LIQUEFACTION ON BRIDGE FOUNDATIONS

The Public Works Research Institute, Ministry of Construction has been conducting laboratory shaking table tests on dynamic behavior of pile foundations constructed in loose saturated sand deposits [11),12),13)]. Fig. 45 shows the test setup, and Table 3 indicates the outline of five cases conducted in the first stage. Fig. 46 illustrates an example of time histories of various pickups installed in the model and the sand. Fig. 47 is the summary of responses of pile foundation in liquefying sand layer, when subjected to sinusoidal input motions. In the later tests dynamic behavior of pile foundations subjected to random input motions have been studied. From the both tests the following remarks are derived tentatively.

Table 3. List of Test Cases

TEST CASE	SAND	Dr 3) (%)	Number 4) of Piles	Head Weight (kg)	5) fp (Hz)	6) fi (Hz)	7) α_{A1} (gal)	Shaking Duration (sec.)	8) fg (Hz)
T1	Iruma 1)	40	9	44	3.8	12	40 ~ 100	60	18
T2	Toyoura 2)	27	4	53	4	10	60 ~ 80	30	24
T3		34	6	18	10	20	80 ~ 130	15	
T4		24	6	18	10	10	60 ~ 80	30	
T5		40	9	13	14	10	60 ~ 120	30	

1) Iruma Sand (G_s=2.88, D_{50}=0.52mm, U_c=1.52, e_{max}=0.93, e_{min}=0.66)
2) Toyoura Sand (G_s=2.64, D_{50}=0.16mm, U_c=1.46, e_{max}=0.96, e_{min}=0.64)
3) Relative Density
4) Each Pile is made of aluminum bars with 70cm in length and 2cm in diameter.
5) Natural Frequency of Pile Foundation in Water
6) Frequency of Input Motion
7) Amplitude of Base Acceleration before Perfect Liquefaction
8) Estimated Natural Frequency of Non-liquefied Sand Layer (Shear Strain of 10^{-4}).

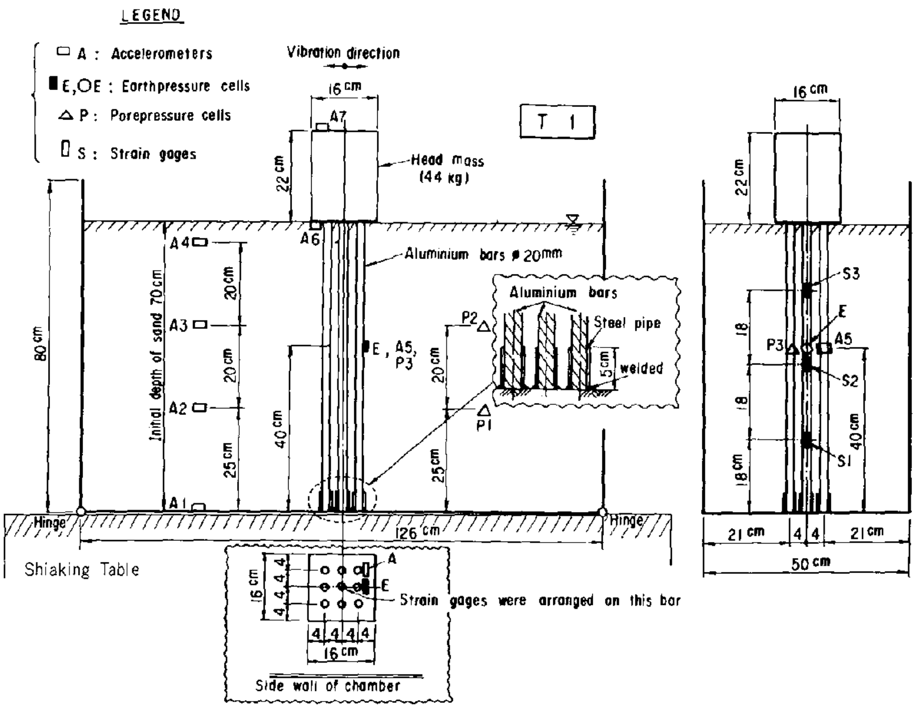


Fig.45 Test Arrangement in the Case of Test 1

1) Liquefaction phenomena expand with respect to time, and with respect to space. Therefore, the effects of liquefaction of sands on foundations vary with time and space. It can be generally recognized that the acceleration response of a pile foundation is small prior to the initiation of liquefaction, becomes considerably large in the course of occurrence of liquefaction, and finally decreases after complete liquefaction takes place. During the complete liquefaction sands behave as heavy liquid, and the foundation is subjected to hydrodynamic pressures due to the liquefied sands.

2) In estimating dynamic behavior of a pile foundation embedded into loose saturated sands, the relationship among the natural frequency of the pile foundation prior to the initiation of liquefaction, the natural frequency of the pile foundation in completely liquefied sands, and the predominant frequency of input motion seems the most important factor. Dynamic behavior of the pile foundation in the course of liquefaction of sands will be considerably affected by this relationship.

3) While the complete liquefaction succeeds, the soil-structure system will behave as a structure submerged into heavy water. Since its natural frequency becomes longer, it may resonate when subjected to seismic motions with longer periods during the latter time of an large earthquake. Accordingly, it becomes important to assess the components of longer periods included in induced seismic motions.

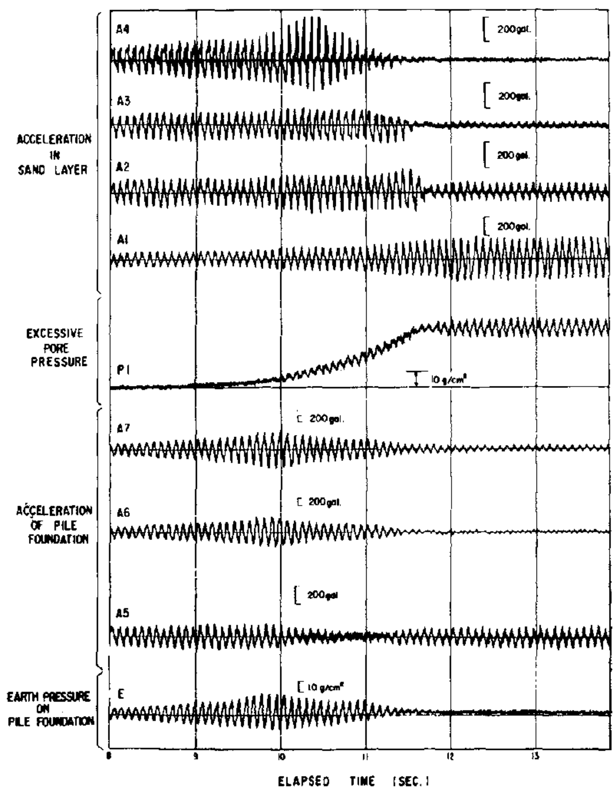


Fig.46 Typical Test Records (Test 1)

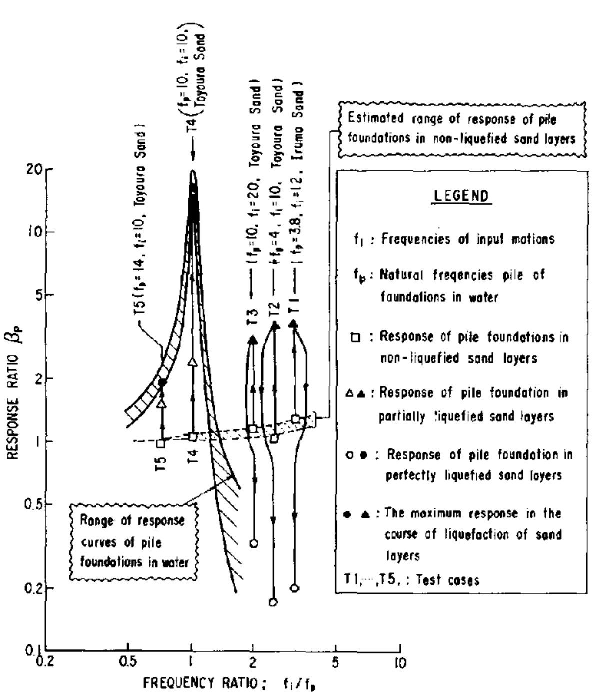


Fig.47 Response of Pile Foundation Models Embedded in Liquefying Sand Layer (Tests 1 to 5)

EARTHQUAKE MEASUREMENT AND DYNAMIC ANALYSIS OF BRIDGES

NETWORK OF STRONG-MOTION EARTHQUAKE MEASUREMENT AT BRIDGES

In Japan the observation of strong-motion earthquakes for engineering structures was initiated in 1953. As of March, 1978, the number of strong-motion accelerographs installed on engineering structures (such as buildings, bridges, railways, harbors, oil power plants, nucler power plants, dams, river structures, tunnels, etc.) is 1,094 totally. Among them, 194 SMAC-type accelerographs are equipped on 91 highway bridge structures (102 accelerographs) and on ground surfaces (92 accelerographs) near those bridges. Installation of the instruments on highway bridges is generally conducted by public agencies such as the Ministry of Construction, Hokkaido Development Agency, Prefectural Governments, and four Highway Public Corporations. These agencies are in charge of construction and maintenance of respective highways.

In addition to the 194 SMAC-type accelerographs on 91 highway bridges, twelve bridges equip electro-magnetic-type seismographs. Moreover, there are nine stations where dynamic behavior of surface soils during earthquakes are being measured using downhole seismometers (deeper ones are more than 100 m below the surface), in connection with the proposed large bridge projects such as Honshu-Shikoku Bridges, Tokyo Bay Loop Highways, etc. The underground seismic measurements are carried out to provide ample information for the earthquake-resistant design of large bridges under consideration.

Strong-motion records from the above bridge stations have been published periodically by the Public Works Research Institute[14], together with those from other public works such as highway tunnels, dam structures, river and coastal structures, sewage facilities, etc.

DYNAMIC ANALYSIS OF A HIGHWAY BRIDGE[15]

Itajima Bridge, completed in 1966 and located in Uwajima City, Ehime Prefecture in Shikoku Island, is a five-span simply-supported plate girder bridge (see Fig. 48). Measurements of strong-motion accelerations have been performed at a pier cap and on the free-field ground surface located about 200 m apart from that pier. Two sets of SMAC-B2-type accelerographs are set up at the both sites to measure strong accelerations in the longitudinal, transverse, and vertical directions to the bridge axis.

Four simultaneous acceleration records were obtained at the both sites as shown in Table 4. Figs. 49 and 50 show the acceleration records at the two sites in the longitudinal and transverse directions. It should be noted that although the response accelerations at the pier cap were rather high (200 to 300 gals), the bridge suffered no damage during the four strong earthquakes.

An discrete analytical model shown in Fig. 51 was formulated to compute seismic responses of the bridge. In modelling the bridge and surrounding soils, the results of detailed field soil survey(including shear wave velocity measurement) were employed. Fig. 52 compares computed responses with the measured ones.

From the analyses briefly described in the above, the followings are deduced.

1) Seismic responses of a bridge with a deeply embedded foundation are significantly influenced by the effects of the surrounding subsoils. In case that the lowest natural frequency of the foundation is smaller than the lowest natural frequency of the surrounding subsoil, the foundation motions will be considerably affected by the motions of the surrounding subsoil.

2) Seismic responses of the foundation can be computed with a fairly good accuracy, by an analysis utilizing strong-motion records obtained at the near free-field ground accelerations.

Table 4. Strong - Motion Acceleration Records at Itajima Bridge

Earthquake No.	Earthquake	Date	Richter Magnitude	Epicentral Distance (km)	Maximum Accelerations (Gal)			
					Pier Motion		Ground Surface Motion	
					Longitudinal*	Transverse*	Longitudinal*	Transverse*
A	The Hyuganada Earthquake	April 1,1978	7.5	100	219	310	170	186
B	The Hyuganada Earthquake (Aftershock)	April 1,1978	6.3	100	39	66	35	42
C	The Bungosuido Earthquake	August 6,1978	6.6	11	198	230	438	365
D	The Bungosuido Earthquake (Aftershock)	August 6,1978	5.3	11	100	63	220	165

(Note) Longitudinal and transverse directions to the bridge axis.

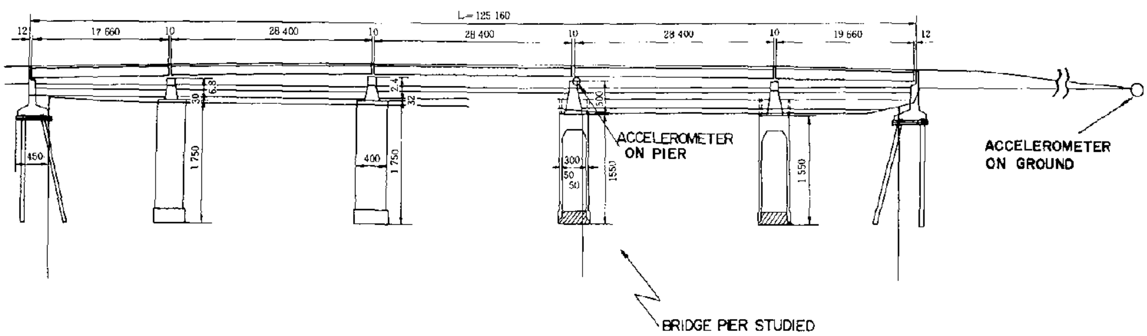


Fig.48 General View of the Itajima Bridge

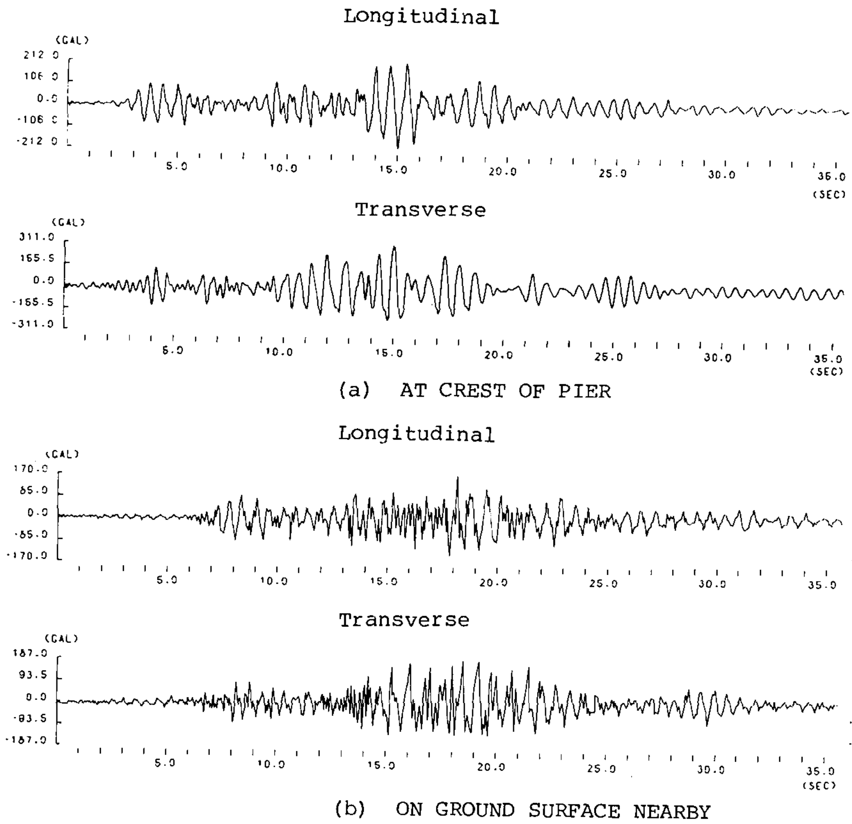


Fig.49 Acceleration Records at the Itajima Bridge during the Hyuganada Earthquake of April 1, 1968 (A-Earthquake)

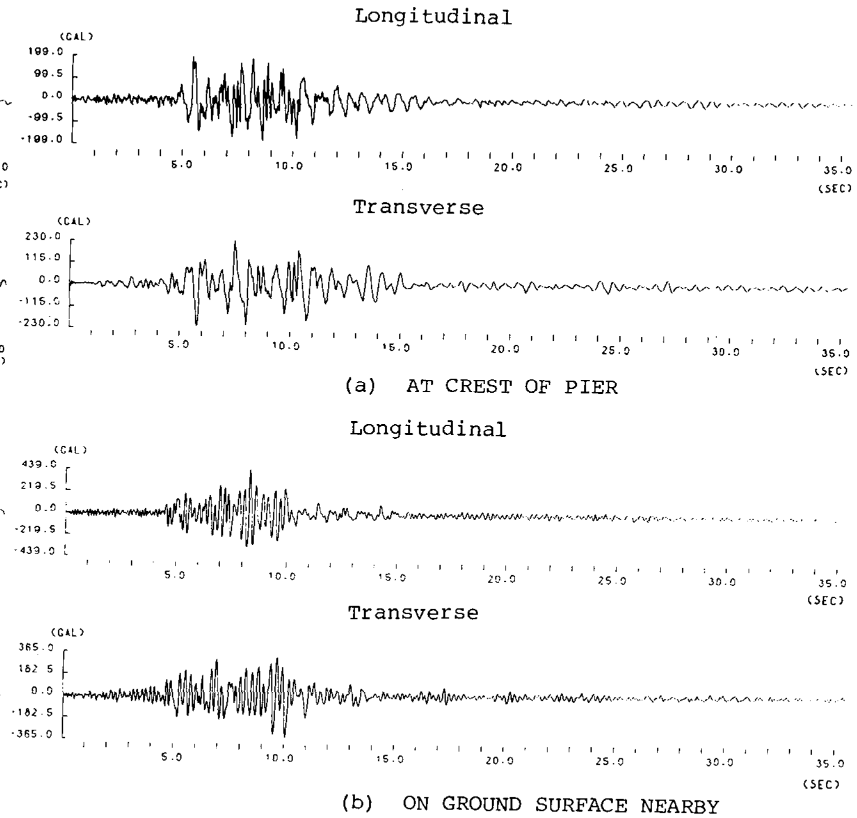
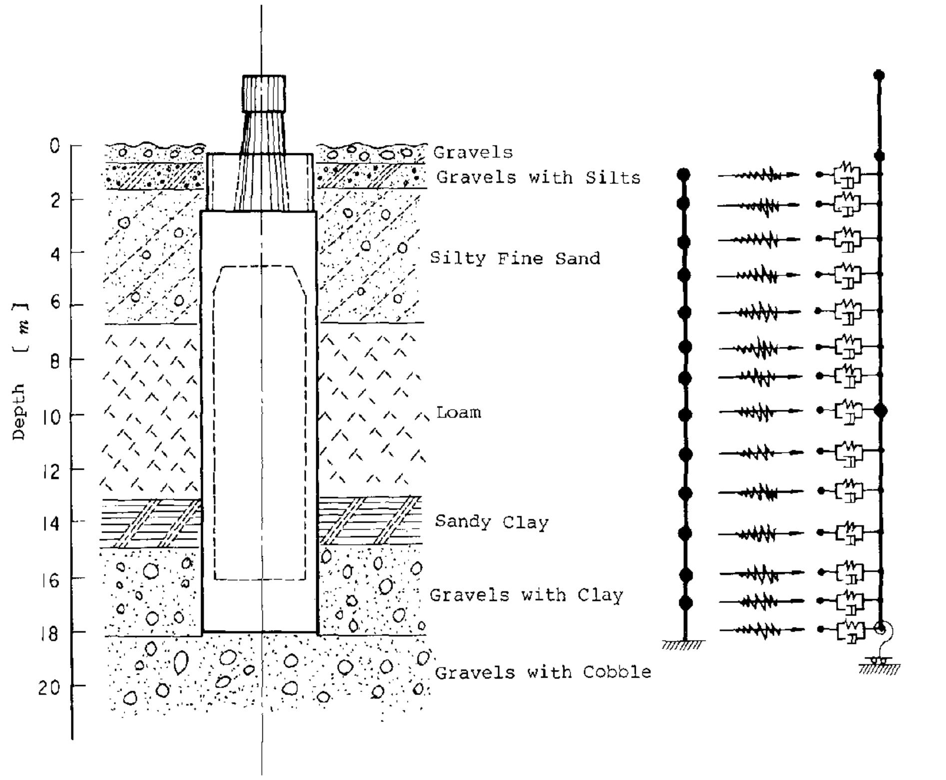
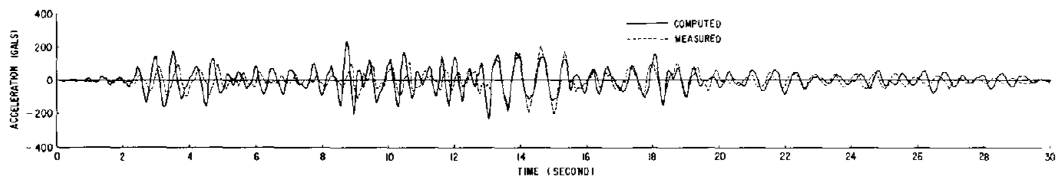


Fig.50 Acceleration Records at the Itajima Bridge during the Bungosuido Earthquake of August 6, 1968 (C-Earthquake)

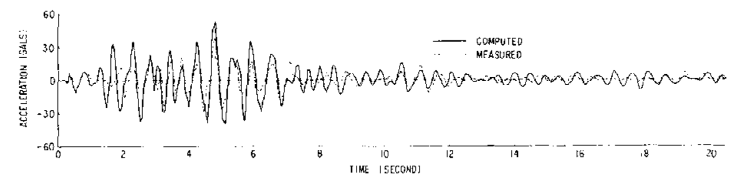


(a) Caisson-Pier and Surrounding Subsoils (b) Analytical Model

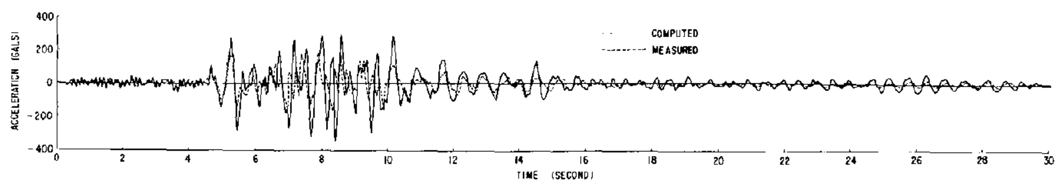
Fig.51 Analytical Model of Foundation of the Itajima Bridge



(a) Hyuganada Earthquake (A-Earthquake)



(b) Aftershock of the Hyuganada Earthquake (D-Earthquake)



(c) Bungosuido Earthquake (C-Earthquake)

Fig.52 Correlations of Seismic Response Acceleration at the Crest of Itajima Bridge (Longitudinal Accelerations)

LABORATORY EXPERIMENT ON DYNAMIC BEHAVIOR OF PIER COLUMNS

In view of damages to short bridge piers during the past earthquakes the Public Works Research Institute has been carrying out experimental studies on behavior of reinforced concrete columns subjected to alternatingly repeated loads. The following is a summary of a paper on this topic authored by Y. Ozaka and M. Ohta[16].

OBJECTIVES

Although experimental studies on dynamic behavior of reinforced concrete columns have been considerably performed for building structures, experiments of bridge pier columns are rare so far. As for building structures, columns with shear-span ratio of 1 to 3, longitudinal bar ratio (P_t) of 1 to 2%, hoop ratio (P_w) of 0.2 to 1.2%, and axial normal stress (σ_N) of 20 to 120 kgf/cm^2 have been generally experimented. Ordinary bridge pier columns, however, have different characteristics (see Table 5). Therefore, it has been necessitated to uniquely conduct experiments of pier columns.

SPECIMENS

Six specimens, shown in Fig. 53, have been experimented in the first stage of comprehensive test series. Six reinforced concrete columns with a shear-span ratio of 3.5 were tested to evaluate dynamic characteristics of short pier columns which have frequently sustained damages during recent earthquakes. Hoop ratios in the specimens were taken as $P_w = 0.04$ to 0.16%. The dimensions are commonly used in slab-type pier columns in Japan. Rigid-frame-type piers, however, have higher hoop ratios ($P_w = 0.16$ to 0.32%) for which the Metropolitan Expressway Public Corporation had conducted similar tests[17].

TEST FACILITY

A structure testing apparatus installed at the Public Works Research Institute in Tsukuba Science City, Ibaraki Prefecture, was employed for these tests. This is capable of applying cyclic loading up to 150 tf(see Fig. 54). In the tests various repeated lateral loads were applied to reinforced concrete columns which were subjected to constant vertical loads.

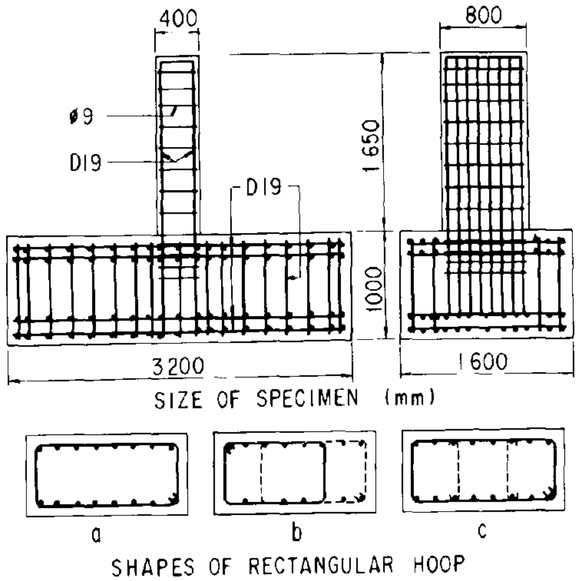
RESULTS OF EXPERIMENTS

Load-deflection relations obtained from the tests are shown in Fig. 55, in which A-specimen was subjected to one-direction repeated lateral loads and B-to F-specimens were subjected to alternatingly repeated lateral loads. Fig. 56 shows patterns of cracks after loading of 10 cycles at 3 δ_y (δ_y: yielding deflection).

From the laboratory experiments the following conclusions are deduced.

1) Lateral deformation of the columns subjected to repeated loads become considerable when longitudinal reinforcing bars tend to extend at the column bases.

2) Ductility factors (deflection normalized by the yielding deflection) of columns subjected to alternatingly repeated lateral loads are much smaller than those subjected to one-direction loads. Modes of failure in these two cases are



SPECIMEN	SHEAR SPAN RATIO	HOOP SPACING (cm)	HOOP RATIO P_w(%)	SHAPE OF HOOP	LONGITUDINAL BAR RATIO P_t (%)	HISTORY*	$\sigma_N = N/A_c$ (kgf/cm²)
A	3.5	20	0.08	a	0.82	I	10
B		20	0.08	a		II	
C		10	0.16	a			
D		20	0.16	b			
E		20	0.16	c			
F		40	0.04	a			

* LOADING I : ONE DIRECTION; 10 CYCLES AT $\delta_y, 2\delta_y, 3\delta_y, \cdots$
LOADING II : REVERSAL ; 10 CYCLES AT $\delta_y, 2\delta_y, 3\delta_y, \cdots$

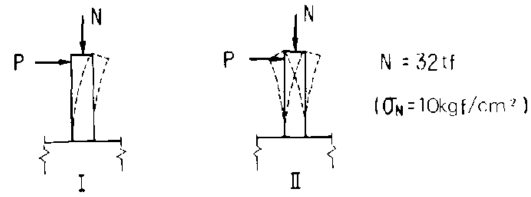


Fig.53 Six Test Specimens

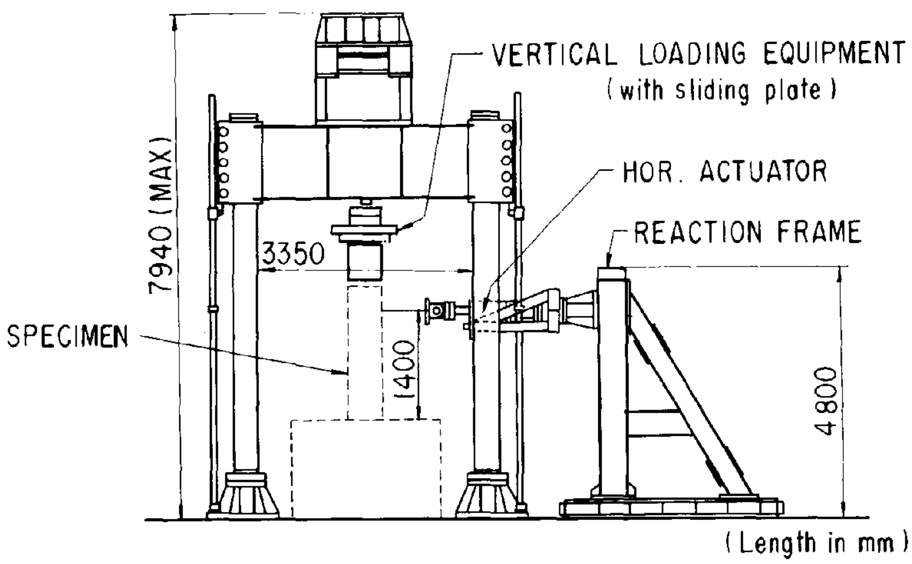


Fig.54 Test Set-up

Table 5. A Comparison between Building Columns and Bridge Pier Columns

Structures	Shear Span Ratio	Steel Ratio (%) Tension Bar, P_t	Steel Ratio (%) Hoop, P_w	Normal Stress $\sigma_N = \frac{N}{A_c}$(kgf/cm²)	Remarks
Buildings	1-3	1-2	0.2-1.2	20-120	Columns Experimented
Bridge Piers	> 3	< 1	< 0.1	10-20	Actual Pier Columns

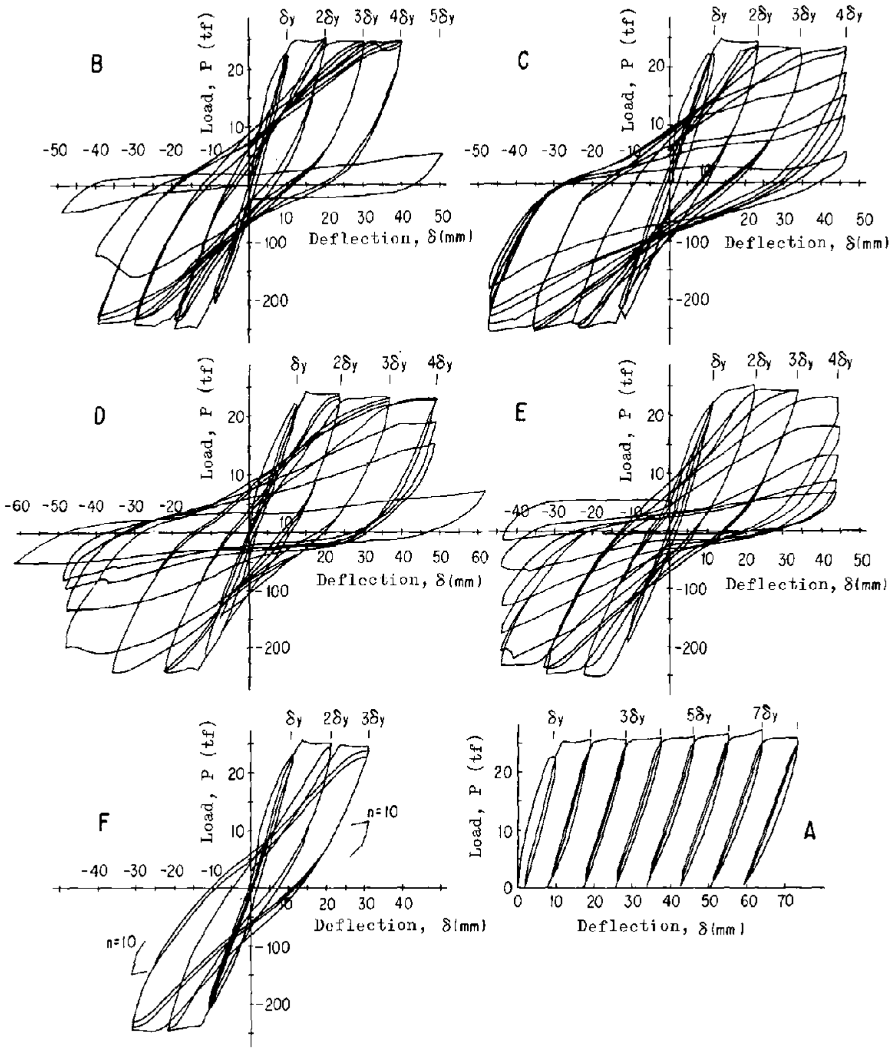


Fig.55 Load-Deflection Diagram

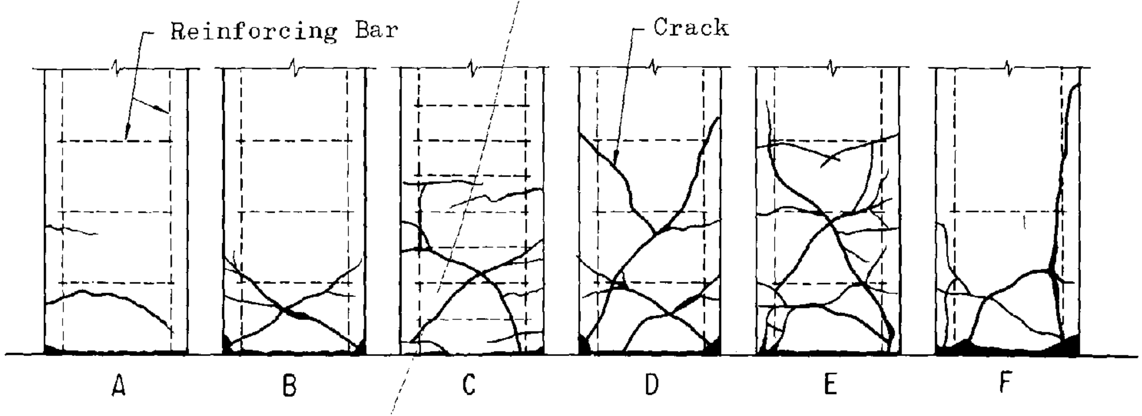


Fig.56 Cracks after 10 Cycles at 3 δ_y

different. In case of one-direction loadings only horizontal bending cracks generate and lead to the ultimate stage. In case of alternating loadings, however, diagonal cracks as well as horizontal cracks generate as the deformation increases, and lead to the ultimate stage. Accordingly, experimental behavior of pier columns subjected to alternating lateral loads should be introduced in conducting rigorous seismic analyses.

3) While the restoring force is not larger than the yielding force, energy absorbing capacities of columns increase as deflection becomes large.

4) Although criteria of the Japan Society of Civil Engineers specify that the hoop spacing shall be smaller than the minimum cross-sectional length, it is recommended from the tests that the hoop spacing should be smaller than the half of the minimum cross-sectional length. As for shapes of hoop, double-hoop (Specimen-D in Fig. 53) and single-hoop (Specimen-C) are better than single-hoop plus cross-tie (Specimen-E).

CONCLUSIONS

From a review of recent studies related to seismic effects on highway bridges, the followings may be concluded.

1) Seismic damage to bridge structures are generally caused by the lack of resistance at bearing supports, substructures, or surrounding soils. As the results of the weakness at these portions, substructures would tilt, settle, slide, cause cracks or failures, or sometimes overturn; superstructures would move, cause cracks or failures, or fall down; and bearing supports may cause failures. Moreover, it is often observed during earthquakes that appurtenant structures such as wing walls and approach banks settle, or separate from the main structures.

2) For provinding bridges with adequate resistance to seismic disturbances, the magnitudes of horizontal design seismic coefficients are most significant. In addition, it seems important to give special attentions to (a) topographical and geological consideration to avoid ground disasters, (b) soil dynamics consideration such as liquefaction of surrounding soils, (c) design details for protecting bridge girders from falling and for evading severe damage caused by the failures at structural joints, and (d) ductility of pier columns.

3) Further investigations are needed on the subjects shown below, in improving aseismic design of bridge structures.

(a) Evaluation of Seismicity and Ground Motions

For determining appropriate seismic forces to be considered in the design, it is important to study probabilities of occurrences of strong earthquakes either by a statistical way or by earthquake prediction and to investigate characteristics of strong ground motions in connection with various conditions concerned (such as seismicity and ground conditions). It seems also important to clarify the differences of ground motions between one location and another nearby, from the viewpoint of the propagation of seismic waves during strong earthquakes.

(b) Structural Planning

For assuring appropriate structural resistance to seismic disturbances, it seems very significant to properly select structural types in consideration of seismicity, topography, geological conditions, soils conditions, etc. at the site. It is recommended to investigate for setting up a standard for selecting types of bridges including superstructures, substructures, and foundations, with consideration of seismic effects.

(c) Effects of Subsoils on Bridge Structures during Earthquakes

It is necessary to evaluate dynamic earth pressures on substructures, bearing capacities of subsoils, effects of ground failures (such as faults, sliding, liquefaction, etc.) on bridge structures during earthquakes.

(d) Aseismic Design Method for Bridge Substructures

It is essential to establish a definite design calculation method for substructures consisting of piers and abutments, together with various foundations (such as spread foundations, caisson foundations, pile foundations, etc.). The magnitude of seismic coefficient should be reasonable in the aseismic design. It is noted that appropriate information on the properties of ground soils is necessary for properly conducting seismic design.

(e) Design Details of Superstructures and Bearing Supports

A specific attention shall be paid to design details of bearing supports and hinges, and also to devices to prevent the fall of girders from the crests of substructures. It seems reasonable that for superstructures design details are more significant than simply applying seismic forces on them.

(f) Measurement of Dynamic Properties and Seismic Response of Actual Bridges

It seems important to conduct field experiments for actual bridges in order to evaluate their dynamic properties such as natural periods, mode shapes, damping characters, etc. It is much more significant to measure dynamic behavior during actual earthquakes by installing strong-motion seismographs on bridges and on grounds nearby.

(g) Dynamic Analysis

It seems reasonable that an analytical approach is a better way to estimate dynamic behavior of structures subjected to seismic excitation. In dynamic analysis it is very important to adequately set up analytical models which represent probable behavior of bridges during strong-motion earthquakes. For this aim it is essential to compare the results of the analysis with the measurement of response of actual structures to forced and seismic excitation. It is advisable to reproduce structural failures or collapses in analysis, with respect to bridges severely suffered by the past earthquakes.

(h) Laboratory Experiment on Dynamic Behavior of Bridges

Dynamic behavior of materials, simplified structural systems, and composite structures are preferable to be examined through laboratory experiments, employing excitors, actuators, or shaking tables. To make model experiments more effective, it seems necessary to proceed to international cooperation, among researchers in various countries.

(i) Quantitative Evaluation of Ultimate Strength of Bridges through Seismic Damage Investigation

To obtain quantitative information on ultimate strength of bridge structures, it is effective to survey seismic damage to the similar structures, and to clarify the differences of characteristics between damaged structures and undamaged ones. In this respect international cooperations including mutual exchange of information and personnel are indispensable among various countries concerned. It is also important to establish standard ways to quantitatively clarify structural ultimate strength. For an example, dynamic experiment or dynamic analysis may be effective means to pursue structural behavior until suffering failures or collapses.

ACKNOWLEDGMENTS

In surveying structural damaged caused by the Miyagi-ken-oki Earthquakes of 1978, great cooperations were given by the Tohoku Regional Construction Bureau of the Ministry of Construction, and the Miyagi Prefecture. The authors express their sincere thanks to the staff members at the above organizations. The authors also express their cordial appreciation to Mr. Minoru Ohta, Head of Concrete Division, and Mr. Kazuhiko Kawashima, Research Engineer, Ground Vibration Division, the Public Works Research Institute for their invaluable supports in preparing this paper.

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APPENDIX A

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APPENDIX C

COMPILATION OF REFERENCES

The following list of references is compiled from information submitted by participants. It serves as a guide for reference material but should not be considered as a complete bibliography on the subject. The references are listed under the following categories: General; Abutments and Retaining Walls; Analytical Studies; Bearings and Energy Dissipating Devices; Codes; Columns and Piers; Damage Reports and Literature Surveys; Foundations and Retrofitting.

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