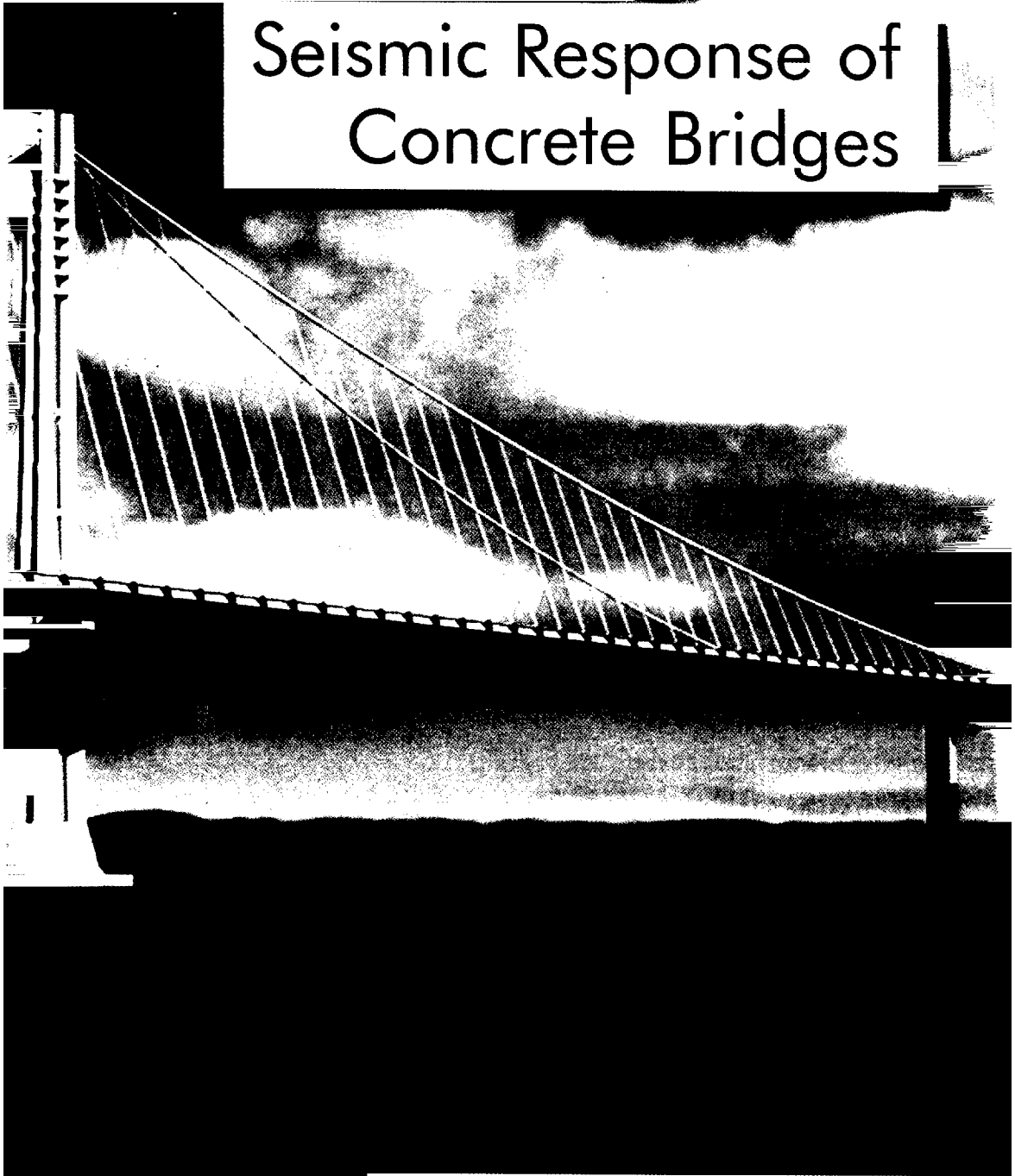


Seismic Response of Concrete Bridges



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Seismic Response of Concrete Bridges

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K. Krishnan



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PREFACE

This special publication contains papers presented during the following four ACI convention sessions sponsored by Committee 341, Earthquake-Resistant Concrete Bridges, during 1995 and 1996:

- *Effects of Recent Earthquakes on Concrete Bridges—Performance, Retrofit, and Design*
- *Response of Concrete Bridges during the Northridge Earthquake*
- *Seismic Retrofit and Design of Concrete Bridges*
- *State-of-Knowledge in Earthquake Response of Concrete Bridges Five Years after the Loma Prieta Earthquake*

A total of 24 papers were presented at the sessions, of which 17 papers were received for this publication. The papers were reviewed in accordance with the policies of the American Concrete Institute.

The papers presented cover a wide array of subjects related to seismic behavior of concrete bridge structures. Topics include the impact of recent earthquakes on the seismic design process, results of recent experimental research on behavior of bridge components during earthquakes, case studies of bridge seismic upgrades, and the use of composite materials and seismic isolation systems to enhance seismic performance.

This special publication has been made possible by contributions from many individuals. M. Saiid Saiidi, the founding chairman of this committee, gave his early leadership and support to this project. The editor is grateful to all authors who have contributed to the publication as well as the reviewers of the papers for their suggestions and comments. The cooperation of the authors in accepting reviewer's suggestions and revising their manuscripts accordingly is greatly appreciated. The editor acknowledges the support of Frederic R. Harris, Inc., without which this publication would not have been possible.

Editor
K. Krishnan

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Impact of Loma Prieta Earthquake on Seismic Design of Concrete Bridges— California Perspective

by J. E. Roberts

Synopsis: Almost nine years have passed since the disastrous Loma Prieta earthquake of October 17, 1989 and eight years have passed since the Governor's Board of Inquiry into the cause of highway structure failures during that earthquake issued its final report with the warning title "Competing Against Time". The California Department of Transportation has developed improved Seismic Performance Criteria, Seismic Design Specifications, seismic design procedures, and construction details based on lessons learned from the 1971 San Fernando earthquake and subsequent seismic events. The success of the Bridge Seismic Design and Retrofit program and the success of future seismic design for California bridges is based, to a large degree, on an unprecedented accelerated and "problem-focused" seismic research program. The Department has spent over \$40 million on this research and physical testing of details. This research has provided the bridge design community the assurance that the new specifications and design details perform reliably and meet the performance criteria. Caltrans staff engineers, consulting firms, independent Peer Review Teams, and university researchers have cooperated in this program of Bridge Seismic Design and Retrofit Strengthening to meet the challenge presented in the June, 1990 Board of Inquiry report. The eight year old Seismic Advisory Board has been an invaluable asset in reviewing the performance criteria, design specifications, design procedures, and construction details for both new design and retrofit strengthening of older, non-ductile bridges.

Keywords: confinement; displacements; ductility; nonlinear analysis; retrofit; seismic design

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INTRODUCTION

Prior to the 1933 Long Beach earthquake there was no special consideration for seismic design of buildings or bridges in California. The severe damage to schools that resulted from that seismic event resulted in creation of the Structural Engineer license and a requirement for special consideration of seismic forces in the design of public schools in California. After the 1940 El Centro earthquake the bridge design office of the California Division of Highways developed minimal seismic design factors for bridges. The 1940 El Centro record was digitized and used as the seismic design spectra for over 30 years before an earthquake of greater magnitude occurred in California. The 1971 San Fernando earthquake caused severe damage to hospitals, public utilities, and freeway bridges, recording a peak ground acceleration of 1.0g and large ground displacements. This earthquake caused both building and bridge designers to revise their design criteria and structural details to provide better resistance to the forces and displacements of major seismic events. The American Association of State Highway and Transportation Officials (AASHTO) is the agency responsible for development of bridge design specifications for nationwide use. AASHTO has typically adopted seismic design criteria modeled after those developed in California, and the initial adoption is only as a guide specification. Until the 1989 Loma Prieta earthquake most other states in the United States had not been concerned with seismic design for bridges, considering it a California or West Coast problem. For example, the 1940 California seismic design specifications were not adopted by AASHTO until 1961, and the 1973 California seismic design specifications were not adopted nationally until 1983. In May, 1990, responding to the disastrous 1989 Loma Prieta earthquake in California, AASHTO finally adopted the 1983 "Guide Specifications For Seismic Design of Highway Bridges" as a mandatory

requirement for those states which have a seismic hazard. Interestingly, some 37 states in the US have some level of seismic hazard. Understandably, there are hundreds of bridges in these other states which have been designed to seismic criteria that are not adequate for seismic forces and displacements that we know today. The seismic retrofit details designed by the California bridge engineers can be of great benefit to those states who are faced with seismic threats of lesser magnitude, and with little financial support for seismic retrofitting, and much less for research and seismic detail development.

The California State Department of Transportation (Caltrans) owns and maintains over 12,000 bridges (spans over 20 feet). There are an equal number on the City and County system. The bridge office maintains the condition data for all these and some 6,000 other highway structures such as culverts (spans under 20 feet), pumping plants, tunnels, tubes, Highway Patrol inspection facilities, maintenance stations, toll plazas and other transportation related structures. Structural details and the current condition data are maintained in the department's bridge maintenance files as part of the National Bridge Inventory System (NBIS) required by the US Congress and administered by the Federal Highway Administration (FHWA). This data is updated and submitted annually to the FHWA and is the basis upon which some of the Federal gas tax funds are allocated and returned to the states. The maintenance, rehabilitation and replacement needs for bridges are prorated against the total national needs. Seismic retrofit strengthening is eligible for use of those funds.

The two most significant earthquakes in California in recent history that produced the best information for bridge designers were the 1989 Loma Prieta and the 1994 Northridge events. While the experts consider these to be only moderate earthquakes, it is important to note the good performance of the many bridges that had been designed for the improved seismic criteria or retrofitted with the early era seismic retrofit details. This reasonable performance of properly designed newer and retrofitted older bridges in a moderate earthquake is significant for the rest of the United States because that knowledge can assist engineers in designing new bridges and in designing an appropriate seismic retrofit program for their older structures. Although there is a necessary concern for the "Big One" in California, especially for the performance of important structures, it must be noted that many structures which vehicle traffic can bypass need not be designed or retrofitted to the highest standards. It is also important to note that there will be many moderate earthquakes that will not produce the damage associated with a maximum event. These are the earthquake levels that should be addressed first in a multi phased retrofit strengthening program, given the limited resources that are available. Cost benefit analysis of retrofit details is essential to measure and insure the effectiveness of a program. It has been the California experience that a great deal

of insurance against collapse can be achieved for a reasonable cost, typically ten percent of replacement cost for normal highway bridges. It is also obvious that designing for a performance criteria that provides full service immediately after a major earthquake may not be economically feasible. The expected condition of the bridge approach roadways after a major seismic event must be evaluated before large investments are made in seismic retrofitting the bridges to a full service criteria. There is little value to the infrastructure in investing large sums to retrofit a bridge if the approaches are not functioning after a seismic event. Roadways in the soft muds around most harbors and rivers are potentially liquefiable and will require repair before the bridges can be used.

Approximately 2,200 of California's 12,000 bridges are located in the Los Angeles area so it is significant to examine the damage and performance of bridges in the Northridge earthquake of January 17, 1994. About 1,200 of these bridges were in an area that experienced ground accelerations greater than 0.25g and several hundred were in the area that experienced ground accelerations of 0.50g or greater. There were 132 bridges in this area with post San Fernando retrofit details completed (Hinge and Joint restrainer cables) and 63 with post Loma Prieta retrofit details completed (Additional joint restrainers and column strengthening). All of these retrofitted bridges performed extremely well (not closed to public traffic) and most of the other bridges performed well during the earthquake; newer bridges designed and constructed to Caltrans' current seismic specifications survived the earthquake with very little damage. Seven older bridges, designed for a smallerearthquake force or without the ductility of caltrans' current design, sustained severe damage during the earthquake. Another 230 bridges suffered some damage ranging from serious problems of column and hinge damage to cracks, bearing damage and approach settlements, but these bridges were not closed to traffic during repairs.

SEISMIC DESIGN PHILOSOPHY

Performance Criteria—An agency or designer must have a seismic performance criteria established. How do you want the structure to perform in an earthquake? How much damage can you accept? What are the reasonable alternate routes? How do you define various levels of damage? How long do you expect for repair of various levels of damage? A bridge seismic performance criteria was developed by Caltrans' Engineeringstaff for state owned bridges and was approved by the Seismic Advisory Board in September, 1992. That current Performance Criteria is **"No Collapse", "No Major Damage", "No Secondary Injuries or Fatalities Because Emergency Equipment Cannot Get Through", "Major Important Structures and Lifeline Routes Must**

Remain Operational". This criteria is generally attainable but the Lifeline Route structure performance criteria can be expensive if structures are expected to withstand severe earthquakes such as a 45 second duration shake due to large earthquakes on a major fault. That is what is expected of major important structures in California and the 45 second duration shake is what the experts predict when the "Big One" comes.

Once a seismic performance criteria is adopted the important issue then is to guarantee that the design criteria and construction details will provide a structure that meets that adopted performance criteria. In California a major seismic research program was financed to physically test large scale and full sized models of bridge components to provide reasonable assurance to the engineering community that those details will perform as expected in a major seismic event. The current phase of that testing program involves real time dynamic shaking on large shake tables. In addition the Caltrans bridge seismic design specifications have been thoroughly reviewed in the Applied Technology Council, ATC-32 project to insure that they are the most up to date with state of the art technology. On major projects a project specific design manual has been produced to provide guidance to the various design team members on what must be done to structural elements to insure the expected performance.

Seismic Design Principles--Continuity is extremely important and is the easiest and cheapest insurance to obtain. Well designed monolithic structures also have the added advantage of low maintenance. Joints and bearings are some of the major maintenance problems on bridges today. If structures are not continuous and monolithic, they must be tied together at deck joints, supports and abutments. This will prevent them from pulling apart and collapsing during an earthquake. **Ductility** in the substructure elements is the second key design consideration. It is important that when you design for ductility you must be willing to accept some damage during an earthquake. The secret to good seismic design is to balance acceptable damage levels with the economics of preventing or limiting the damage. Properly designed ductile structures will perform well during an earthquake as long as the designer has accounted for the displacements and controlled them or provided for them at discontinuities such as abutments and hinges. For a large majority of bridges displacement criteria controls overstrength criteria in the design for seismic resistance.

Nonlinear Analysis Procedures--Prior to the Loma Prieta event there was little use of nonlinear analysis in the design of bridges. In order to correctly analyze bridge performance in a major earthquake of long duration, the use of nonlinear analysis techniques is mandatory. Ample research has been completed in this area to give designers the necessary tools to conduct reasonable nonlinear analysis and design structures that will perform in a ductile manner during a

major earthquake with long duration. Additional work in this area will continue to improve the expertise of bridge designers and build confidence in non-linear analysis techniques.

Steel versus Concrete--While it can be argued that steel is inherently more ductile than concrete, it must also be noted that steel members have a high potential to buckle after reaching the yield point. Concrete members, being substantially larger in cross section, have a much better chance for survival without buckling, even after exceeding the yield point. This is due in part to the larger distance the concrete column must be displaced to cause buckling. Most damage in California during the Loma Prieta and Northridge earthquakes occurred on structures that had been designed 30 to 60 years ago to comply with codes that required seismic design forces of only 0.06g to 0.08 g. Today most of those structures would be designed for 8 to 10 times those seismic forces. The forces are derived from the acceleration coefficients multiplied by the weight of the contributing mass at any supporting member or joint. A quick survey of most bridges in this country will confirm that almost all have reinforced concrete substructures, regardless of the superstructure type. And the substructures are where most of the earthquake damage occurs. Since we rely on internal strain energy to dissipate the external work (energy) caused by the earthquake forces, concrete substructure elements are preferable. As long as they are properly confined they will remain intact and sustain the gravity loads after the event. Internal damage can be repaired after a seismic event.

Seismic Design Research Program--The Governor's Board of Inquiry recommended that Caltrans "Fund a continuing program of basic and problem-focused research on earthquake engineering issues pertinent to Caltrans responsibilities." The initial Legislative investment in bridge seismic research was \$8 million. Subsequently, the Department management has agreed to a problem-focused seismic research program at an annual expenditure level of \$5 million (approximately 1% of the Caltrans annual bridge capital expenditure program). It is this last recommendation that gave impetus to the major seismic research work which is being supported by the Department today and through which we are supporting the annual seismic research workshops. The workshops serve a major goal of technology transfer to the user community. All bridge seismic details for new design and retrofit have been proof tested in the university laboratories before being implemented in the final structures. Over \$40 million has been spent on bridge problem focused seismic research since the 1989 Loma Prieta earthquake.

SEISMIC DESIGN SPECIFICATIONS AND PROCEDURES FOR BRIDGES

A major element of the improved seismic design specifications and procedures was the adoption of a site specific seismic design philosophy shortly after the 1971 San Fernando earthquake. The California Division of Mines and Geology (CDMG) was engaged for the development of an earthquake ground fault map. The maximum credible events (MCE) on seismic faults throughout the state define peak bedrock acceleration levels. Average energy attenuation relationships were developed by various seismologists and contours of these decreasing levels of expected peak rock acceleration are included on the map. The original map was produced in 1973 and included 225 known faults. The current version is CDMG Map Sheet 45. This document maps the 275 currently known faults and is updated periodically as new information becomes available. This approach, using the MCE, has been criticized as too conservative but our cost analyses show that the additional cost for an average bridge is less than one percent for the maximum credible event versus design for a probabilistically determined event. Considering the limited ability to determine a probabilistic earthquake with a high degree of confidence and the high cost for a site specific study, we feel that the maximum credible approach for the smaller typical state highway bridges is a reasonable approach, given the current state of seismological event predictions. Since the 1989 Loma Prieta earthquake Caltrans has utilized a site specific probabilistic hazard analysis to determine the most probable design earthquake for major structures. This event is incorporated into the seismic design procedure along with the maximum credible event.

In 1973 Caltrans, working with Professor Harry Bolton Seed at UC Berkeley, then developed a series of Acceleration Response Spectra (ARS) for alluvium and harder soils of various depths above bedrock. These spectra have been used for determining the appropriate seismic design force for typical freeway bridges in California. After the 1989 Loma Prieta earthquake the staff and consultants began developing a similar set of response spectra for deep, soft soils. These new spectra have been included in the ATC-32 project which was an effort to upgrade the Caltrans bridge seismic design specifications. It is apparent, however, that more site specific soil studies must be used for major bridges being built or retrofitted over softer soil foundations.

Earthquake of October 1, 1987 reemphasized the inadequacies of pre-1971 column designs. Even though there was no collapse, the extensive damage resulted in plans for basic research into practical methods of retrofitting bridge columns on the existing pre-1971 non-ductile bridges. That research program had already been initiated in early 1987 at the University of California at San Diego and the Whittier earthquake merely speeded its approval and execution. The continuing bridge seismic retrofit research is currently being conducted at the University of California at San Diego, the University of California at Berkeley, and over six other University research facilities. Funding levels for seismic retrofit program implementation were increased four fold after the Whittier earthquake to an annual program of \$16 million. Even at that level it would have required nearly 300 years to complete the retrofitting program that was ultimately identified and implemented.

Most columns designed since 1971 contain a slight increase in the main column vertical reinforcing steel and a major increase in confinement and shear reinforcing steel over the pre-1971 designs. All new columns, regardless of geometric shape, are reinforced with one or a series of spiral wound interlocking circular cages. The typical transverse reinforcement detail now consists of #6 (3/4 inch, 18mm diam.) hoops or continuous spiral at approximately three inch (75mm) pitch over the full column height. This reinforcement detail provides approximately eight times the confinement and shear reinforcing steel in columns than what was used in the pre-1971 non-ductile designs. All main column reinforcing is continuous into the footings and superstructure. Splices are mostly welded or mechanical, both in the main and transverse reinforcing. Splices are not permitted in the plastic hinge zones. Transverse reinforcing steel is designed to produce a ductile column by confining the plastic hinge areas at the top and bottom of columns. The use of the more ductile and weldable grade 60, A 706 reinforcing steel in bridges has been specified on all new projects since 1992.

The Loma Prieta earthquake of October 17, 1989 again proved the reliability of hinge and joint restrainers, but the tragic loss of life at the Cypress Street Viaduct on I-880 in Oakland emphasized the necessity to immediately accelerate the column retrofit phase of the bridge seismic retrofit program with a higher funding level for both research and implementation. Other structures in the earthquake affected counties performed well, suffering the expected column damage without collapse. With the exception of a single outrigger column-cap joint confinement detail, those bridges using the post 1971 design specifications and confinement

details performed well. Damage to long, multiple level bridges showed the need to more carefully consider longitudinal resisting systems because earthquake forces cannot be carried into abutments and approach embankments as they can on short bridges.

After the Loma Prieta earthquake caused 44 fatalities on the State Highway System, capital funding for bridge seismic retrofiting was increased to \$300 million per year. At the same time bridge seismic research funding was increased from \$0.5 million annually to \$5.0 million annually with an initial \$8.0 million allocation from the special State Emergency Earthquake Recovery legislation of November, 1989 (Senate Bill 36X).

RESPONSE OF DEEP SOFT SOILS

The Acceleration Response Spectra (ARS) developed after the 1971 San Fernando earthquake were developed for harder soils and alluvium but were not accurate for prediction of the dynamic response of softer soils and bay muds. After the Loma Prieta earthquake Caltrans immediately engaged Professors John Lysmer and Raymond B. Seed at UC Berkeley to help develop a set of similar ARS curves for deep, soft soils and mud. Professors Lysmer & Seed and C.M. Mok & S.E. Dickenson of UC Berkeley have concluded that the deep, soft muds amplify the bedrock ground motions by factors of 2 to 3 and that amplification of the longer period components is especially pronounced, resulting in surface motions that are more damaging to the taller, longer period structures.

Seismic ground motions have been predicted in the deep, soft soils with an analytical procedure and those predictions have been confirmed with the actual recorded Loma Prieta motions at four sites around the San Francisco Bay. The Applied Technology Council project ATC-32 has provided a series of ARS curves for various soft soil conditions to augment those already in use.

Other geotechnical factors that contributed to structural damage in the Loma Prieta earthquake were non-uniform and out of phase response of the foundation materials and their affect on longer structures such as the 1.5 mile long viaducts and the San Francisco-Oakland Bay Bridge. The effect of non-coherent soil response is now being considered in the site specific response spectra that are developed for longer structures. While soil liquifaction was not a contributor to bridge damage, it was apparent near several major structures in the East Bay after the Loma Prieta earthquake and must be considered and dealt with in future seismic design for bridges. Results of studies by Lysmer, Seed,

Idriss and other investigators clearly indicated the shortcomings of the 1989 provisions for dealing with the influence of deep, soft foundations on structure response. In addition to developing the new set of ARS curves for deep, soft soils, Caltrans has also initiated a program to identify and map the soft soil sites in the state. Caltrans has developed a set of generic ARS curves for several representative deep, soft soil site conditions for use on the more standard and smaller bridges but, upon the recommendation of Doctor Seed and other advisors, we will concentrate on site specific response analyses for all major structures at soft soil sites. These researchers have shown that the analysis techniques and the computer programs currently available can reliably predict the response of deep, soft soils, and therefore, justify site specific analyses. For the major Bridges crossing the Bays from San Diego in the South to Antioch at the extreme Northeast end of the San Francisco Bay we have engaged consultants to conduct site specific complete hazard analyses using a probabilistic approach to provide the design earthquake for bridge seismic design purposes.

Working with in-house Engineering Geologists & Geotechnical Engineers and several Consultants, Caltrans is also identifying and mapping the sites which could potentially liquefy and working with Professor Geoffrey Martin at the University of Southern California and others to develop design mitigation procedures for those sites. For one site north of San Diego at Del Mar Heights, adjacent to the Pacific Ocean, Caltrans is using 16 inch diameter stone columns to stiffen the soil around the bridge piers. For the major Mission Valley interchange between Interstate Routes 8 and 805 in San Diego extensive substructure modifications are being incorporated to overcome the foundation strength loss due to liquefaction in a major seismic event. Jackura and Abghari of Caltrans have reported on the mitigation of liquefaction hazards at three California bridge sites, including the two in San Diego. On the relocation project for IS 880 in Oakland extensive foundation stiffening measures were incorporated to resist liquifaction impacts.

SEISMIC DESIGN PROCEDURES FOR BRIDGE STRUCTURE/ FOUNDATION INTERACTION

The two major considerations in seismic design of foundations are ground motion and foundation-substructure interaction. Caltrans adopted a site specific seismic design philosophy shortly after the 1971 San Fernando earthquake. For the average smaller freeway structures we use the Maximum Credible Earthquake (MCE) for determining the seismic design forces. For major structures we use a site specific probabilistic hazard analysis to determine the

most probable design earthquake spectra. These spectra are incorporated in the seismic design analysis along with the maximum credible event. This approach provides a good check of the two procedures and assures the use of the worst case seismic coefficients for lateral force design.

By 1973 Caltrans had developed a series of Acceleration Response Spectra for various depths of alluvium over the bedrock. These spectra have been used for determining the appropriate seismic design lateral force for typical freeway bridges in California. As discussed previously, we have developed a similar set of spectra for deep, soft soils. It is apparent, however, that more site specific studies must be used for bridges being built or retrofitted over softer soil foundations. For example, five significantly different soil conditions, ranging from deep, soft bay mud (over 100 feet thick) to bedrock at the surface exist along the length of the Terminal Separation Structure at the west end of the San Francisco-Oakland Bay Bridge, and five different foundation acceleration response spectra have been developed for the structural dynamic response analysis of various frames in this structure.

The design procedure involves modeling of the structure and foundation, including the foundation material stiffness and pile/footing stiffness using appropriate soil springs. Using this model and the appropriate design earthquake record a dynamic analysis is conducted to initially size the column dimensions and reinforcing steel patterns. The columns are sized to provide moment and shear ductility capacity greater than the joint ductility demands generated by the analysis program. This procedure is iterative, requiring several cycles to converge on the correct column sizes. After the initial column sizing, the foundations are designed to resist the plastic moment (M_p) and plastic shear (V_p) capacity at the base of the column (Plastic Hinge Zone), determined by the properties of the column concrete section and the reinforcing steel pattern. An allowable overstrength of 30% is typically applied to the column moments and shears to determine M_p and V_p for foundation design, to account for the ductile performance of the column in a seismic event. From this point forward foundations below the plastic hinge zone are designed without additional seismic input.

Caltrans had also developed a series of reduction factors for reducing the seismic design force based on the ductility inherent in the structural framing of the bridge, the degree of acceptable risk we were willing to take, and the amount of structural damage we would accept. These factors have been developed empirically, based on actual damage patterns recorded in California earthquakes since 1971. Seismic lateral forces are reduced using these factors but the joint displacements are not reduced from those actually calculated from the dynamic analysis. It is important to note that the reduction factors for shear keys and

hinge restrainers are 0.90 and 1.00, respectively. These factors actually result in conservative design for these details but it is felt that the relatively low cost of these elements and the difficulty of inspecting them after an event warrants the conservative design. This philosophy has been proven to be correct, based on damage to interior hinge details in the 1994 Northridge earthquake. The Ductility and Risk

Reduction Factors is one of the major elements of the current criteria that requiresome adjustment and it was studied as a first order of work in the ATC-32 project. Some experts feel that the current factors are not conservative enough in the short period range and too conservative in the long period range. The AASHTO Seismic Bridge Design Specifications which are used by other states utilize similar factors designated as "R" factors, which are generally more conservative than those used by California. It is anticipated that Caltrans will have new Risk and Ductility factors by the summer of 1998, and they will undoubtedly be more conservative than the current factors.

The proper modeling of bridge foundations is essential to achieving an accurate dynamic response of the foundations and their influence on the structural response of the bridge. Doctor I. M. Idriss has developed a procedure for predicting the ground response at soft soil sites using the computer program SHAKE and currently available ground and bedrock seismic response records. He has achieved reasonable agreement between recorded and calculated motions, so it is now possible to predict the ground motion characteristics of soft soil sites with a reasonable degree of confidence.

Doctors Po Lam, Roy Imbsen and Geoffrey Martin have discussed modeling of bridge foundations in a paper which was given at the Third Transportation Research Board (TRB) Bridge Engineering Conference at Denver, Colorado on March 10, 1991. Dr. Po Lam has found in his studies that the Caltrans recommendations to designers for abutment soil response given in Bridge Design Aids, Section 14, compare favorably with response characteristics calculated using Wilsons equation. With the techniques described in these papers and the work of other researchers it is not difficult to properly model the bridge structural elements and the bridge foundations.

The major factors to be considered in modeling bridge abutments are the stiffness of abutment systems, the soil pressure on abutment end walls and wingwalls, and the interaction between abutment inertia forces from the deck system versus the induced soil pressure. The UC Davis research project, headed by Doctor Karl Romstad deals with bridge abutment load/displacement characteristics. The work was conducted by Caltrans engineer Dr. Brian Maroney as part of his Doctoral Dissertation. This work will add to the limited data now available. Lam, Imbsen and Martin recommend an Iterative Design

Procedure for abutment design using the four steps below. Caltrans has been using this procedure since the Loma Prieta earthquake, and similar procedures since 1973.

- (1) Assume abutment stiffnesses based on abutment parameters and standard soil properties.
- (2) Conduct a Dynamic Response Analysis using the assumed abutment stiffnesses.
- (3) Check soil pressure versus soil capacity; if pressure exceeds capacity, repeat the analysis with reduced abutment stiffnesses. Repeat iterations until convergence is achieved.
- (4) Check abutment displacement and compare with original assumptions. Repeat analysis with revised stiffness if necessary to achieve a realistic displacement.

Footing design is handled in much the same manner, based on lateral load test data to determine lateral resistance of pile footings and pile groups as well as spread footings. Actual values can be used or appropriate soil spring constants can be used in modeling to conduct a dynamic response analysis of the structure. The lateral load tests reported by Abcarian and discussed at length by Lam provide valuable data for a limited number of pile load tests. More lateral load tests must be conducted to increase the reliability of predictions. For many soft sites, site specific lateral load tests must be conducted until the bridge engineering profession has developed sufficient knowledge in this area to develop good soil/structure interaction data and soil spring constants. Imbsen, Lam and Martin discuss the proper procedures for substructure design in their work for input to the FHWA course on Seismic Design of Highway Bridges.

For many bridge supports the use of single column bents is popular in California and it is necessary to develop uplift capacity in the foundation piling due to overturning moments. James Mason of Caltrans conducted a series of pile tests for various proprietary piles and micro-piles in the deep mud site below Interstate Highway 280 in South San Francisco. Design loads for these Micro Piles are now available and can be specified by the designers. Load tests to confirm required length at each deep soft soil site are still required, but these proprietary systems are pre-approved for use.

DUCTILE DESIGN DETAILS

Caltrans has used ductile design philosophy since developing the current seismic design specifications in 1973. It is considered too expensive to design all bridge components for earthquake loading to remain within the elastic range. An acceptable approach in the design of earthquake resistant structures is to allow some local and controlled yielding. In bridge structures this commonly takes the form of designed fuses at the base or top of columns as potential plastic hinges. Effective fusing at the column-footing interface greatly reduces the moments and shears that transfer into the footings and makes footing analysis easier. Also, the ductility prevents the development of brittle failures. This is especially helpful in retrofitting older existing bridges.

Beam-Column Joint Confinement--Major advances have also been made in the area of beam-column joint confinement, based on the results of research at both UC Berkeley and UC San Diego. The performance and design criteria and structural details developed for the I-480 Terminal Separation Interchange and the I-880 replacement structures reflect the results of this research and was reported by Cooper of Caltrans. Research is continuing at both Institutions to further refine the design details to ensure ductile performance of these joints. Large one-half scale models of the most critical three-way longitudinal edge beam, transverse bent cap, column joint detail were built and tested at both UC San Diego and UC Berkeley as a major element of the joint and column testing program. Additional large scale model testing programs are planned for the more complex multi-level beam-column details commonly used in major highway interchange structures. Mahin and Moehle of UC Berkeley, and Seible of UC San Diego have published reports of their research in this area.

Edge Beam-Column-Bent Cap joint detail--The most elaborate and expensive tests conducted to date have been the half scale and third scale models of the proposed retrofit details for the double deck viaduct structures in San Francisco. Models, using two different seismic retrofit techniques, were tested at both UC San Diego (Half scale) and UC Berkeley (Third Scale) in order to obtain the performance characteristics of the two different details so Caltrans' designers and consultants could make the final design and construction decisions and get the construction contracts started in 1992. Both models were nearly 50 feet long by 20 feet wide and 20 feet high. They included one-half of each span adjacent to the joint and half the column above and below the joint. Half width of the superstructure was also modeled to recreate the actual contributing dead load and stiffness of the elements that frame into these columns. The difference in the concepts is in the connection to edge of deck of the original structure.

The UC Berkeley concept uses an integral edge beam detail which is necessary on curved alignments, such as the Central Viaduct (US 101) in Downtown San Francisco and the Alemany Interchange on US 101 in South San Francisco. This research and testing program was reported by Mahin.

The UC San Diego concept is an independent edge beam which will be used on straight alignments, primarily the I-280 Southern Freeway Viaduct north of the Alemany Interchange with US 101. This research was reported by Priestley. Thirteen dynamic actuators were required to dynamically excite the UC San Diego model in all three directions. The beginning of plastic hinge formation at the top of the lower column occurred at design ductility's, exactly where the design intended to force its location. After a ductility of 8 had been achieved, the plastic hinge zone began to lose its cover but the core concrete remained intact because of the heavy confinement reinforcement and therefore the structure should remain standing and in operational service. This level of damage is easily repaired after a seismic event without closure to public traffic. Each of these test series represents a cost of \$500,000 but they are used to proof test the intended retrofitting scheme for structures totaling more than one mile in length with a replacement cost of over \$200 million. Mahin and Moehle of UC Berkeley and Priestley and Seible of UC San Diego have published results of their work.

SUMMARY

At this point in time, Caltrans has identified approximately 2200 bridges on the state highway system that will require seismic retrofitting to meet the current seismic safety criteria. As of February 1, 1998 we have completed seismic retrofit upgrading on 2028 bridges, have another 130 under construction, and awarded construction contracts for another 10 bridges. Bridge plans are completed on 99% of the bridges and the remainder are in the design process. Although we are about 95% completed in numbers, we have the major Toll Bridges remaining to complete so we will be busy for the next four years completing the program. Special design teams have been established to complete the remaining bridge retrofit plans and consultants are being used for six of the remaining Toll Bridges. All the hazard and vulnerability analyses for the Toll Bridges have been completed and detailed design is virtually completed.

Completion of all normal bridge seismic retrofit in California is anticipated by late 1999, and for all Toll bridges by early 2003. All new bridges in the state are designed in accordance with the philosophy and details described in this paper. Performance of new bridges that had been designed for the latest seismic criteria

or retrofitted older bridges gives a high level of assurance that the newer criteria and procedures are effective.

The success of this program has been due to dedicated staff and consultants, both of whom have developed innovative retrofit details to accomplish the seismic safety that is required for these bridges. The comprehensive research program has been instrumental in developing and proof testing the details prior to field use. Support of the Legislature and Administration for funding has been a key to successful completion. Other capital improvement projects have been delayed until seismic safety has been accomplished on the California Highway System.

SP 187-2

Northridge Earthquake Influence on Bridge Design Code

by R. Zelinski

Synopsis: The Northridge Earthquake, January 17, 1994, caused partial collapse of seven freeway bridges, and damage to 230 others. Vertical accelerations, failed hinge restrainers, and column flare behavior were cited as causes of collapse and major damage. It is true that these factors, plus a few others, contributed to the collapse and damage. However, the primary cause of all collapses and major damage can be summarized as insufficient ductility in the bridge structural frames. Furthermore, the bridge elements which failed in a non-ductile fashion behaved predictably. The details which led to the failures are no longer used in new bridge designs. These brittle details are converted to ductile ones in retrofit designs. This paper will investigate the undesirable bridge behavior observed in the Northridge Earthquake, and discuss how most of these issues are already covered in new bridge design codes. Finally, procedures adopted or proposed to correct the remaining issues and improve bridge behavior in a more reliable manner, as compared to undesirable Northridge Earthquake bridge response, will be outlined.

Keywords: analysis; bridges; concretes; design; ductility; prestressed; reinforced; retrofit; seismic

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NORTHRIDGE EARTHQUAKE BRIDGE SUMMARY

Although the California Highway System suffered some severe damage, there were also examples of favorable bridge behavior. This paper summarizes the damage, but will emphasize the positive features of Caltrans' Bridge Seismic Resistance Program.

Approximately 237 bridges were damaged in the Northridge Earthquake. Seven bridges were severely damaged and suffered partial collapse. Damage was serious on some of the other (230) bridges, but was only superficial on many of the others.

Most of the damage occurred within a zone bounded by a minimum of 0.25g ground surface shaking (Fig. 1). A total of 1600 bridges are located within this zone. Therefore, 1363 bridges received no damage.

Sixty fully retrofitted bridges existed within the 0.25g zone. Most of the 60 bridges suffered no damage. The damage suffered by the other few was superficial and predictable. Only one bridge designed to current new bridge standards was located in the 0.25g zone. It survived unscathed.

DAMAGE ASSESSMENT

The collapsed and damaged bridges in this earthquake had the same brittle features as damaged bridges in past earthquakes. Unseating of hinges and fractured columns were the primary causes of collapse. The moderate horizontal ground motions were identified as the source of the damaging displacements. Vertical accelerations are suspected of contributing to the damage, but not the primary source.

Most of the collapsed bridges had expansion joints which had been retrofitted with hinge restrainers in a previous program. However, these early restrainer versions did not meet the design strength and ductility provided by current procedures. Existing restrainers were being replaced with new versions routinely in the current column retrofit program when the Northridge Earthquake occurred.

The two collapsed bridges on State Route 118 were constructed in the mid-70's. Limited ductility was built into the existing columns. However, the confining reinforcement details were not equivalent to current standards. Also, some of the other physical features of the bridge (column flares, abutment details, varying column stiffness, etc.) were not in accordance with current seismic design policies. The combination of undesirable seismic features in the SR 118 bridges left them too vulnerable.

In conclusion, none of the damage can be considered surprising or unexpected. More accurately, it can be stated that most (1363) of the old brittle bridges resisted the applied forces amazingly well.

EVOLUTION OF SEISMIC DESIGN CODE

To state that damage in the Northridge Earthquake was neither surprising nor unexpected, it is important to know what is in the current seismic design code and how it developed. The following text will provide a summary discussion of Caltrans seismic policies, procedures and code development chronology. Informed with that background, a discussion of potential code changes, influenced by the Northridge Earthquake, will then be provided.

CURRENT ANALYSIS PROCEDURE

Caltrans' analysis procedures are based on demands emanating from the maximum earthquake expected on the dominant adjacent fault. The potential earthquake energy for each fault is converted to rock response in the form of percent gravity accelerations. The accelerations attenuate away from the fault and are plotted in the form of acceleration contours.

The geotechnical engineers have provided ground motion (ARS) spectral curves for different depths of soil above the rock and related to rock accelerations (Fig. 2). It is those acceleration spectra which

are used to analytically excite the bridge and create maximum elastic forces and displacements for design purposes.

To place the new design forces in perspective with the actual forces measured near the site, a comparison of seismicity is shown for the I5/SR14 Interchange site where bridge collapses occurred during the Northridge Earthquake. The comparison is:

	<u>Northridge</u>	<u>Design Criteria</u>
Maximum Horizontal Ground Acceleration	0.91g	(T= 0.8 Seconds) 2.5g
Maximum Vertical Ground Acceleration	0.60g	1.6g (2/3H)
Duration of Strong Shaking	10 Seconds	40 to 60 Seconds (Expected)

As can be observed, the Northridge Earthquake was not insignificant, but the design criteria for the maximum credible event will pack a much larger punch.

NEW BRIDGE DESIGN PROCEDURES

Member ductility and reliable load paths are the key features of the Caltrans' seismic design code for both new and retrofit bridges. In the case of new bridges, the designer has total control of the design. Numbers of bents and columns, sizes and shapes of columns, types of abutments, and stiffness of members are all subject to the designer's control. Those features can be manipulated to produce a dependable structure response to expected ground motions. The foundations are designed to provide stiffness or flexibility depending on the soil type (i.e. dense, soft, granular, clay).

Once the framing details are selected, the elastic dynamic analysis is performed. The generated maximum elastic forces are reduced by a force coefficient (Fig. 3). The column is then designed to satisfy all load cases including the reduced seismic forces. The column displacement capacity, dependent upon the supplied volume of transverse reinforcement, is then calculated (Fig. 4).

In the displacement capacity calculation, various factors such

as a) the plastic hinge zone length (L_p), b) the safe concrete yield strain (ϵ_c) the safe ASTM A706 steel reinforcement yield strains (ϵ_{st}) for both longitudinal and transverse column bars, and d) the internal yielded column equivalent fluid pressure, usually 25kg/cm^2 (350 psi) are combined to calculate a safe elasto-plastic displacement.

This displacement capacity is then compared to the displacement demands from the elastic dynamic analysis and reinforcement is modified, if necessary, to assure that capacity exceeds demand. This "Push Analysis" supplies a monotonic nonlinear simulation of a seismic response of the structure as an alternative check of the usual linear solution.

After the column design is completed, the column plastic moment, or ultimate yield moment, is calculated. This moment is normally considered equal to 1.3 times the nominal column moment. The yield (plastic) moment and the associated plastic shears are applied to the footing and the superstructure.

The superstructure and the footing are then designed to withstand those plastic demands. Since the plastic demands cannot be exceeded, the superstructure and footings should remain elastic and be considered reliable load paths throughout the duration of the earthquake. Of course, abutments must be designed to take expected impacts, and intermediate thermal joints must be supplied with adequate seat travel and tension restraints.

New bridges designed to these ductility standards (Fig. 5) should withstand the maximum earthquake demands expected at the site without collapsing. New bridges should suffer only cosmetic damage in a Northridge-level earthquake, and should be repairable after the maximum event.

BRIDGE RETROFIT DESIGN PROCEDURES

Like new bridges, retrofit designs are developed in a manner which provides system ductility including a reliable load resisting path. Unlike new bridges, the designer is usually burdened with a skeleton which lacks ductility, has widely varying member stiffness, and has dozens of short circuits in lieu of a dependable load path. In addition, there often are many features which interfere with the most promising retrofit solutions such as: traffic lanes, insufficient clearances, buildings and other constructed facilities, utilities and

bodies of water. The designer must be very open minded and creative to solve most retrofit problems.

The seismic design usually begins with an elastic dynamic analysis. Existing column and abutment seismic demands are compared to calculated capacities. The designer develops a strategy whereby existing member properties are modified. A common strategy assumes some fixed column ends remain fixed and some are allowed to fuse to a pin. Also, abutment boundary conditions can be revised to represent retrofit concepts.

The analytical model is then modified to more closely represent probable retrofit conditions. The model is then dynamically analyzed a second time in the modified condition. Demands are once again compared to established standards. The modifications and comparison process is then repeated as often as required to satisfy the standards. In most cases, two or three repetitions are sufficient to capture structure stability for the maximum expected earthquake event. Occasionally the designer must add supports or outriggers to obtain closure to the problem, or abandon the strategy totally and begin with a fresh approach. Bridge removal and replacement is the ultimate retrofit, and could be the most economical solution.

Unlike new bridges, the damage prognosis for retrofit bridges is not good. Retrofit bridges should do well in a Northridge-level earthquake, suffering only superficial or repairable damage. However, for a maximum event, retrofit bridges are expected to resist collapse, but will probably be damaged beyond usefulness and need to be replaced. In cases where retrofits are mandated to be serviceable for the maximum event, a tremendous cost increase, as compared to a no-collapse design, will probably result.

DESIGN POLICIES, PROCEDURES, AND STANDARDS

Design codes usually evolve over a period of time. The Caltrans' Seismic Design code for new and retrofit bridges follows this evolution process. Most of the changes are associated with earthquake events and research. Following is a chronological summary of seismic design standards which have evolved at Caltrans since the 1971 San Fernando Earthquake produced a wake-up call for bridge designers.

A. Pre-1980.

Continuous main column bars, spiral column reinforcement, top mat footing reinforcement and hinge restrainers across joints were the immediate changes for new construction made in 1971. Hinge restrainers were also introduced as a retrofit for existing bridges in 1971. Design procedure for these seismic enhancements was mostly empirical. Elastic spectral dynamic analysis was introduced for new and retrofit bridges in the mid-1970's.

B. 1980.

By 1980, many of Caltrans' current design procedures and practices were in place. Elastic spectral dynamic analysis was the norm for both new and retrofit bridges. Column ductility principles were established for new bridges. Transverse column reinforcement volume was calculated to satisfy ductile confinement and plastic hinging shear controls. Foundations were designed for column plastic hinging demands. Abutments were designed to resist elastic seismic forces. Restrainers for both new and retrofit bridges were designed to meet elastic dynamic spectral demands.

C. During the 1980's.

Abutment design philosophy was revised. Key design strength was reduced to lower pile lateral load demands. Since lateral loads had controlled abutment foundation designs, the numbers of piles and cost were thus reduced. This change tends to place more demand on columns, but the increase can usually be absorbed in available column ductility capacity. Also, a rational force-displacement procedure was developed for restrainer design. This change was primarily for new construction because the restrainer retrofit program was nearly completed. Ductile reinforcement detailing, based on empirical methods, was developed for outrigger bents and knee joints.

The column retrofit program was initiated in 1986. Steel column casing and foundation modification details (Fig. 6) were developed by Caltrans. A demand/capacity analysis procedure for assessing existing bridges was created. A pipe seat-extender detail was developed.

Caltrans began identifying soft soil sites in California.

D. Post Loma Prieta (October 1989).

Localized strength deficiencies in superstructures were identified by a peer review panel. Caltrans developed empirical reinforcement standards to resist flexural and joint shear demands in new bridge superstructures. Caltrans began examining existing bridges for superstructure capacity as a load path requirement. A strut-tie design method was introduced for knee joints. Site specific soil spectral curves for soft soil bridge sites became the norm. Experimentation with displacement and rotation procedures for obtaining column capacities as an alternative to the elastic moment reduction procedure was begun. This procedure was applied to selected new and retrofit bridges. ASTM A706 steel reinforcement replaced A615 steel. The A706 steel has tighter production standards which complements engineering ductility applications.

The chronological events listed above are listed to inform readers that standards are really variables. A bridge meeting today's code will probably not meet tomorrow's code. Therefore, today's bridge could be seismically vulnerable by tomorrow's standards. As retrofit programs are initiated, screenings of existing bridges are based on current standards and knowledge. As time progresses, standards change and past screenings and retrofits may not be totally appropriate. Owner agencies are then placed in an awkward and liable position because of these circumstances. There usually are no available funds to retrofit new bridges and re-retrofit the retrofitted bridges. These situations require tough decisions based on seismic probability and risk and must be elevated to high level managers. This progression of events naturally leads us to design code action precipitated by the Northridge Earthquake.

NORTHRIDGE EARTHQUAKE INFLUENCE ON CODE

Although there was some serious damage in the Northridge Earthquake most of it, as stated earlier, was due to brittle details in older structures. It was especially upsetting knowing that the bridges were scheduled for retrofit, and believing that harm to the victims could have been prevented given enough time. Following these tragic events, engineers feel a sense of disappointment and frustration, but must move forward, analyze the failures, and propose code revisions as required. Following is a discussion of those proposals.

New Bridges.

An empirical method for determining a minimum reinforcement volume in superstructures to resist vertical acceleration demand was conceived. A detail for flared columns to ease stiffness and ductility problems was adopted. Experimentation with new details for reducing abutment damage, making superstructure hinges collapse resistant, and improving flexibility for short columns, and studies to eliminate plastic hinges (damage) in pile shafts are being conducted.

Retrofit Bridges.

For retrofit bridges, a larger gravity load factor was suggested for determining static vertical acceleration demands when examining damage patterns and load paths in superstructures.

In summary, it is apparent that the Northridge Earthquake had only minimal influence on the Caltrans seismic design code. We at Caltrans, are optimistic that this evolving seismic code is dependable, and that future seismic events will confirm that optimism.

CONCLUDING REMARKS

With each earthquake, non-ductile bridges fail, and a certain amount of media hysteria surfaces and criticism is aimed at the bridge engineering community. Unfortunately, catastrophes seem to be the catalyst which create problem awareness and a source of program funds. The intent of this paper is to make fellow engineers aware of the development of the bridge seismic code in California, and that the code has not been truly tested in a large earthquake.

I will not declare that the Caltrans program is perfect, and that the new and retrofit bridges are free of flaws. Mother Nature has a way of teaching us a new lesson occasionally. Also, research or testing can hint at areas to improve (for example, Caltrans is studying probable rebar yield strains under cyclic tension/compression column loadings as compared to specified yield strains) seismic policies and procedures. However, new and retrofit bridge behavior in recent earthquakes indicates that Caltrans' seismic design criteria is reliable. Hopefully, this paper will help fellow engineers recognize the strides made in Caltrans' seismic code and appreciate the successes demonstrated in recent seismic events.



Fig. 1—Northridge Earthquake damage area.

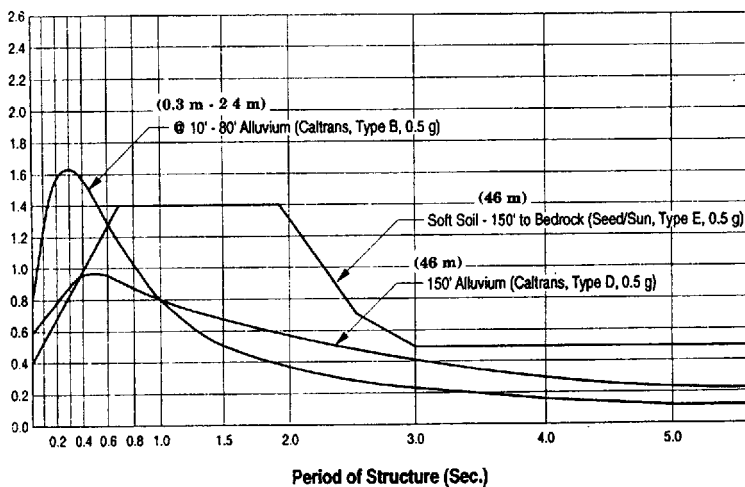
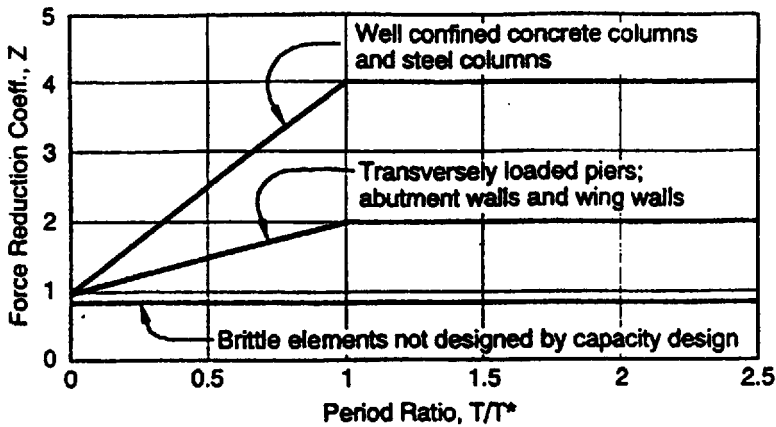


Fig. 2—Example ARS plots.

Full Ductility Structures



T = natural period of the structure

T^* = predominant period of ground motion

Fig. 3—Force reduction plot.

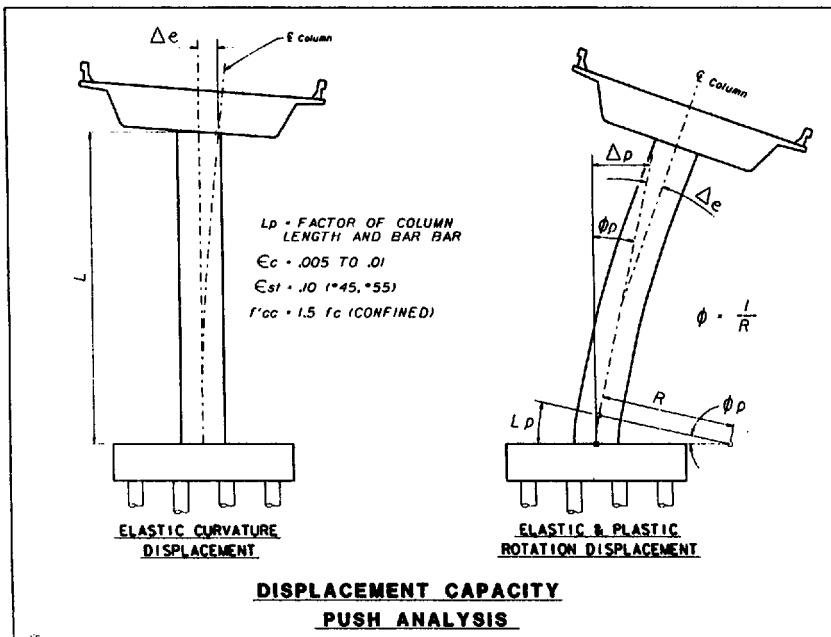


Fig. 4—Column displacement illustration.

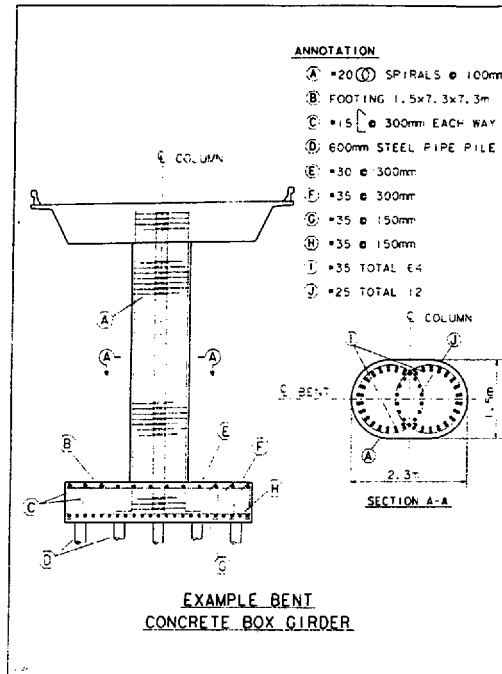


Fig. 5—Typical column/footing details.

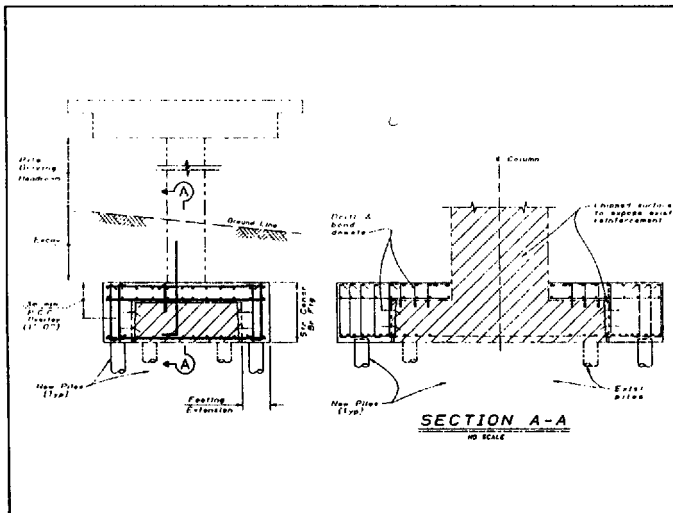


Fig. 6—Example retrofit footing.

SP 187-3

Lessons Learned from Bridge Performance during Northridge Earthquake

by F. Seible and M. J. N. Priestley

Synopsis: The January 17, 1994, Northridge earthquake taught the bridge engineering community invaluable lessons concerning vintage-based bridge damage, performance of retrofitted bridge structures, and design considerations for new bridges.

As a result of the Northridge earthquake, seven bridges collapsed or were damaged beyond repair; of the seven, three were designed and built prior to the San Fernando (1971) earthquake, two were designed before 1971 but construction completed after the 1971 event, and two bridges were designed and built a few years after the San Fernando earthquake. Many other bridges in the strongly shaken region sustained repairable damage. In all cases the damage could be explained based on simple rational assessment models. All but one of the seven bridges collapsed due to column failure, based on inadequate shear design, lack of flexural capacity and/or buckling of the compression reinforcement. One bridge failed by unseating of highly skew superstructure movement joints.

All bridges retrofitted since 1989 (when the Caltrans seismic retrofit program included substructures) performed without signs of damage. The analytical assessment of six of the seven collapsed structures showed that available column retrofit technology could have prevented the observed failures. However, the key lessons learned from the Northridge earthquake, over and above the proper member detailing, are in the systems response understanding. By taking movement joints out of the collapse path of the bridge by avoiding highly skew geometries, and by assigning equal demands to substructure assemblages to take advantage of ductility and redundancy, reliable seismic performance can be achieved. The encountered problems, lessons learned, and new bridge systems design approaches are discussed.

Keywords: bridges; design; earthquake; flexural capacity; retrofit; seismic

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M.J. Nigel Priestley, FACI, is Professor of Structural Engineering in the Division of Structural Engineering at the University of California, San Diego. He received ACI's Raymond C. Reese Award in 1984 and 1989, and ACI's Wason and Anderson Award in 1997. His research interests include reinforced and prestressed concrete structures and earthquake-resistant design.

1. INTRODUCTION

1.1 Overview of Bridge Damage

Following the Northridge earthquake of January 17, 1997, ($M_w = 6.7$), Caltrans reported [1] that seven bridges had partially collapsed, 39 bridges showed major yet repairable damage, 194 structures required minor repair, and 46 bridges required hinge and restrainer repair or replacement. The epicentral location and the locations of the seven collapsed highway bridges are depicted in Fig. 1. All seven bridge sites were located within 25 km of the epicenter with estimated ground accelerations between 0.3 and 0.8 g, as outlined in Table 1.

The design and construction completion dates for each of the seven bridges are also shown in Table 1, together with the probable cause of collapse observations as well as analysis [3] indicate that six of the seven bridge failures can be attributed to column deficiencies. In four of the seven bridges, failure resulted from inadequate shear strength of columns, generally in bents where the column or columns were stiffer and/or flexurally stronger than in other bents, resulting in higher shear forces. In these four bridges, the critical transverse reinforcement consisted of 12 mm diameter bars spaced 300 mm on centers regardless of column size or shape. One of the remaining bridges failed by unseating at highly skewed internal superstructure movement joints as a result of transverse response displacements. The final two bridges, designed more recently than the others, failed in plastic hinges at locations where the effective column length was shortened by either architectural flares or a channel wall built integral with the columns. In both cases it appears that the failure was the result of lack of flexural ductility capacity, possibly accelerated by buckling of compression reinforcement, though ductile shear failure is also a possible cause. The fact that the seven bridges were designed and built in three different vintage groups with respect to the 1971 San Fernando ($M_w = 6.6$) earthquake is of particular interest and will be discussed later since improvements in the seismic bridge design were expected from the lessons learned in 1971. Speculations that vertical accelerations played a major role in the collapse scenarios of the seven

bridges were not substantiated by the detailed post-earthquake response assessment analyses [3].

1.2 Performance of Retrofitted Bridges

Following the 1989 Loma Prieta earthquake, Caltrans initiated their Phase II retrofit program which, in addition to the hinge restrainers of Phase I also addressed substructure deficiencies, in particular deficiencies in single column bents. All bridge structures in the region of strong ground shaking that were retrofitted since the 1989 Loma Prieta ($M_w = 7.0$) earthquake performed adequately without damage requiring repairs, thus demonstrating the validity of the adopted Caltrans retrofit procedures. Based on Caltrans reports [1], there were 24 Phase II retrofitted bridges in the region where estimated strong shaking exceeded 0.5 g, and a total of 60 bridges with Phase II retrofits in regions with estimated peak ground accelerations of 0.25 g or greater. The Phase II retrofitted structures as a whole, resisted the earthquake motions much better than the unretrofitted structures. Assessment analyses based on simple rational assessment procedures [2], performed on the six bridges which collapsed due to column failure (see Table 1) showed that collapse of these bridges could have been prevented if existing Phase II column retrofit technology would have been implemented at the time of the Northridge earthquake.

Incidentally, four of the collapsed bridges were designated and scheduled for retrofit. The retrofit designs were completed for the I-10 structures (La Cienega & Venice and Fairfax & Washington) but had not been implemented at the time of the Northridge earthquake due to legal problems with leased-out air space. The SR-14/I-5 structures were scheduled for retrofit evaluation and design but delayed by the classification as structures in an Alquist-Priolo Special Study Zone which required appropriate seismological and geological studies on possible faulting and maximum expected vertical and horizontal fault rupture surface offsets prior to the vulnerability assessment and retrofit design.

2. REVIEW OF COLLAPSE SCENARIOS

These seven collapsed bridges can be classified by design and construction vintage into three groups:

1. Bridges designed and built before the 1971 San Fernando earthquake (Gavin Canyon Undercrossing on I-5, the Fairfax & Washington Undercrossing on I-10, and the La Cienega & Venice Undercrossing on the I-10 Santa Monica Freeway).
2. Bridges designed before 1971 but construction completed after 1971 (Antelope Valley Interchange with the SR-14/I-5 South and Northbound Connectors).
3. Bridges designed and built after 1971, but before basic design concepts were changed in 1974 (SR-118 Simi Valley-San Fernando Freeway at Mission & Gothic and Bull Creek Canyon).

This section summarizes results of inelastic pushover analyses [3] of the failed bridges, and compares predictions of the analyses with observed behavior.

A brief overview of the analytical process used for this assessment is provided as an Appendix to this paper. Theoretical background is available in [2].

2.1 Bridges Designed and Built Before the 1971 San Fernando Earthquake

About 75% of California's highway bridges were built before 1973, the date when modern seismic bridge design practices were introduced that incorporated the lessons of bridge performance in the San Fernando earthquake. Of the 12,176 state bridges, 9,206 or 76%, were designed before 1973.

Three bridge structures that were completed before, and survived the 1971 San Fernando earthquake were the Gavin Canyon Undercrossing, the Fairfax & Washington Undercrossing, and the La Cienega & Venice Undercrossing on the I-10 Santa Monica Freeway. In the 1960s, when these bridge structures were designed and built, earthquake engineering in general was not very advanced. This is reflected in seismic bridge design guidelines of that period that required elastic lateral load designs typically for only 6 percent of gravity without any provisions or considerations for inelastic structural response or ductile design detailing. Key characteristics inherent in these pre-1971 bridge structures are:

1. Low transverse reinforcement ratios in columns, typically 12 mm diameter reinforcing bars at 300 mm centers were provided nominally, without consideration of column size or strength.
2. Short seat widths at abutments and superstructure expansion joints due to underestimated displacement demands, as a result of the low lateral design loads.

The collapse at the I-5 Gavin Canyon Undercrossing (Fig. 2) can be attributed to geometric complexities arising from the 66 degree skew orientation of abutments and in-span expansion joints, as well as the 200 mm seat width. Even though retrofitted with Phase I first generation restrainers in 1974 (as a direct result of the 1971 San Fernando earthquake), restrainer design and detailing were not sufficient to prevent unseating and subsequent superstructure failure in the acute corners of the bridge at the in-span expansion joints. Cable restrainers were placed parallel to the girder webs and not perpendicular to the movement joint, see Fig. 3, thus allowing transverse superstructure displacements to occur.

Since the restrainer retrofit in 1974, continued research and development of restrainer units funded by Caltrans [4], as well as a better understanding of the skew geometry problems under seismic loads [2], would result now in significantly larger seat width extensions and restrainer capacities. However, quantitative failure assessments immediately following the Northridge earthquake [3] also indicated that shear failure in the shorter bents at the Gavin Canyon Undercrossing was imminent, had unseating and superstructure failure not occurred.

Pre-1971 transverse reinforcement detailing in columns was also a problem for the two bridges which collapsed at the I-10 Santa Monica Freeway. As stated above, the state-of-the-art in bridge design at the time, provided only nominal transverse column reinforcement, and not designed reinforcement as a result of current engineered capacity requirements that would today result in

transverse reinforcement ratios exceeding the nominally-provided ones by a factor of 8 to 10 or more. Typical column geometries and reinforcing details for the I-10 La Cienega-Venice Undercrossing are depicted in Fig. 4 and show eight different longitudinal column reinforcement details with two different footing connections, but all with the same nominal transverse reinforcement detailing. As a consequence of the low transverse reinforcement and underestimated flexural over-strength, the I-10 structures at Fairfax & Washington (Fig. 5), and at La Cienega and Venice (Fig. 6), collapsed with column shear failures either before or shortly after their initial flexural yielding. Quantitative failure assessments of these structures [3] showed that the failure modes encountered could have been prevented with available column retrofit jacketing technology.

2.2 Bridges Designed Before 1971 but Construction Completed After 1971

The second group of bridge structures, the I-5/SR-14 Antelope Valley Interchange, were designed to pre-1971 design standards, but were completed in 1974. This suggests that lessons learned from the 1971 San Fernando earthquake should have been implemented in the redesign.

A common misconception following the Northridge earthquake was that the same bridge structures that collapsed during the 1971 San Fernando earthquake collapsed again this time. Pre- and post-earthquake aerial photographs [5], clearly show that:

1. The 1971 bridge collapse was in a different separation structure, namely the I-5/SR-14 South Separation and Overhead.
2. During the 1971 San Fernando event, the two bridge sections that collapsed in the Northridge earthquake were under construction or completed (SR-14/I-5 South Connector Overcrossing spans 1 and 2 had the bottom soffit, webs, and cap beams cast, and the SR-14/I-5 North Connector Overcrossing was complete, with the exception of one column and the last two spans at the north end).
3. All columns in the Antelope Valley Interchange, except for the one mentioned above in 2, were completed and featured the nominal pre-1971 12 mm diameter @ 300 mm transverse column reinforcement.

Because unseating at expansion joints in tall bridges with single-column bents was identified as one of the primary reason for collapse of bridge structures in the 1971 San Fernando earthquake, the decision at the time must have been to complete the interchange with the already built substructures, but with added expansion joint restrainers to tie the superstructure together and prevent unseating at expansion joints.

During the Northridge earthquake, the two bridge failures at the I-5/SR-14 Interchange (Figs. 7 through 9) can be attributed to brittle shear failure of short or stiff columns. These columns, due to their larger stiffness, attracted more seismic force than their more flexible adjacent bents within the collapsed frames and did not have the necessary deformation capacity due to the pre-1971 transverse reinforcement detailing [3]. The south end (frame 1) of the SR-14/I5 Southbound Connector collapsed by shear failure of bent #2 in the longitudinal direction, see Figs. 9 and 10, with a subsequent superstructure failure at bent #3 by explosive rupture of the longitudinal post-tensioning tendons at the top of the

bent #3 pier diaphragm. Lack of sufficient mild reinforcement between the pier diaphragm and the multi-cell box girder superstructure resulted in complete superstructure failure around the bent cap as shown in Fig. 8. Even though bents #2 and #3 had almost the same clear height, the original grade profile depicted in Fig. 10 shows that bent #2 was founded in a deep cut whereas bent #3 was at the original grade, resulting in significantly stiffer soil reactions for bent #2 and with it reduced displacement capacities compared to bent #3. In a similar fashion, the shear failure of bent #2 of the north connector was aided by an effective shortening of the already short column in bent #1 through the construction of a truck ramp shoulder strip around the column with compacted aggregate and asphalt/concrete overlay, see Fig. 11. Thus, both structures at the I-5/SR-14 Interchange collapsed by brittle shear failure in the short columns next to the abutments with subsequent superstructure unseating at the abutment and superstructure flexural failure at the adjacent bent as a direct consequence of the short column collapse.

2.3 Bridges Designed and Built After 1971, but Before Basic Design Concepts were Changed in 1973

Finally, the two bridges that failed on the SR-118 Simi Valley Fernando Freeway at Mission & Gothic and Bull Creek Canyon were clearly post-1971 vintage in both design and construction. Both bridge failures during the Northridge earthquake can be attributed to:

1. Significantly increased stiffness due to decrease in length or effective column-shortening by heavy column flares at Mission & Gothic (Fig. 13), and by a channel wall at Bull Creek Canyon that was built integral with the columns along a bent line (Fig. 14);
2. Higher shear demands due to flexural over strength; and,
3. Degrading shear capacities in flexural plastic hinge regions under inelastic cyclic loading at high ductility demands.

At the Mission & Gothic Undercrossing, the column flares, extending over half the column height, were moderately reinforced along the flare. These moderately reinforced flares more than doubled the flexural capacity of the column top in the flared column direction. Transverse reinforcement, in the form of a smooth 16 mm spiral with 90 mm pitch, see Fig. 15, was provided along the entire circular column core, and flexural plastic hinging was forced to the bottom of the flare. The increased shear demand led to shear failure and vertical column bar buckling in the plastic hinge region (Fig. 13) at high local curvature ductilities. At Bull Creek Canyon, only the column ends over a distance of one column diameter were confined with tightly spaced spirals, while the column center portion featured again a 300 mm pitch. In the bents with the integral channel wall (Fig. 16), the channel wall top was clearly in the region of low transverse reinforcement ratio, see Fig. 14, that provided little or no ductility to the inelastic flexural hinge that formed on top of the channel wall and failed in shear at low flexural ductilities [3]. Both structures had heavy skew abutment and abutment movement joints which directed the superstructure movements with amplified displacement demands on specific columns.

3. LESSONS IMPLEMENTED IN THE DESIGN OF THE REPLACEMENT BRIDGES

Lessons learned from the observed damage and bridge failures were immediately incorporated by Caltrans in the design of the replacement bridge structures for all seven bridges. Design changes implemented by Caltrans addressed the system or global response of the bridge structures and consisted of (1) elimination of most in-span expansion joints through longer (up to 300 m) jointless superstructures and special two column hinge bents, (see Fig. 17), where even complete unseating does not result in partial superstructure collapse, (2) elimination of excessively skewed joints and abutments, (3) balancing the stiffness of bridge columns within individual frames through column extensions in sleeved holes, (see Fig. 18), and added abutment flexibility and controlled ductile abutment response through external CIDH-piles integrated with the superstructure and sliding support conditions at the abutment seat, (see Fig. 19). Other design improvements included the use of site specific geological and seismological data for ARS curves and substructure stiffness determinations and the consideration of potential vertical excitations for the superstructure design.

Caltrans executed the reconstruction of the collapsed bridges as well as the retrofit of one by December 1, 1994, well within one year following the earthquake and with most bridges completed within six months of the earthquake. This accelerated schedule provided not only challenges for demolition and construction, but also for design and seismic safety peer review. Demolition and construction contracts were given on an invited, pre-qualified, limited bid basis with heavy incentives and penalty clauses for early or late completion.

4. CONCLUSIONS

Lessons learned from the 1994 Northridge earthquake consisted of a reminder that still many pre-71 designed columns exist in the Caltrans highway network and that their vulnerabilities in terms of shear failure, limited ductility, and compression bar buckling are amplified by the global bridge response when (1) columns of different height/stiffness exist in a single frame, and (2) skew geometries at abutments and interior movement joints cause directed displacement amplifications. These lessons were incorporated in the post-earthquake replacement structure design by Caltrans by (1) eliminating highly skewed movement joints, (2) minimizing the number movement joints through longer frames, (3) providing similar stiffnesses to bents within a frame through column extension/casings, and (4) designing for controlled ductile abutment response.

All of the observed bridge failures during the Northridge earthquake correlated well with the assessment models and procedures outlined in the Appendix to this paper. The same assessment models also showed that all of the observed bridge failures could have been prevented if current Caltrans retrofit technology would have been implemented at the time of the earthquake. Finally, all bridge structure retrofitted under the Phase II Caltrans retrofit program since 1989 performed well without signs of significant damage.

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APPENDIX: ANALYTICAL PROCEDURES

Immediately after the earthquake, a decision was made to perform simple inelastic push-over analyses of the collapsed bridges rather than inelastic time-history analyses. This enabled analytical results for all seven bridges to be finalized by three weeks after the earthquake, and provided a valuable insight into the ability of simple pushover analyses, based on rational section analyses, to predict observed location and type of damage. The procedures adopted are summarized in the following, and are presented in more detail in [2] and [3].

Flexural Response

Of prime interest was the flexural strength, stiffness and displacement capacity of individual columns and multi-column bents based on moment-curvature analysis. The program adopted [2] includes effects of confinement in enhancing compression strength and ultimate compression strain, and effects of reinforcing steel strain hardening. An ultimate compression strain, ϵ_{cu} , is related to transverse reinforcement volumetric ratio ρ_h by

$$\epsilon_{cu} = 0.004 + \frac{1.4\rho_h f_{yh} \epsilon_{su}}{f'_{cc}} \geq 0.005 \quad (A-1)$$

where f_{yh} is the yield strength of the transverse reinforcement, which has a strain at peak stress of ϵ_{su} , and f'_{cc} is the strength of the confined concrete.

For members with transverse reinforcement provided by 13 mm dia hoops at 305 mm centers with lap splices within the cover concrete, it was conservatively assumed that the limit to flexural response occurs when a compression strain of 0.005 developed at the level of the hoop reinforcement at the compression side of the section.

Plastic rotations within plastic hinge regions were based on a bilinear approximation to the moment-curvature relationship, with ideal moment capacity occurring at five times the bilinear yield curvature, or maximum moment, whichever occurs first. The plastic rotation was given by

$$\theta_p = (\phi_u - \phi_y) L_p \quad (A-2)$$

where ϕ_u and ϕ_y are the ultimate and yield curvatures respectively, and the equivalent plastic hinge length was given by [3]

$$L_p = 0.08L + 0.022d_b \cdot f_{ye} \quad (A-3)$$

where L is the distance from the critical section to the point of contraflexure, and d_b is the nominal diameter of the longitudinal reinforcement of effective yield strength f_{ye} (MPa).

Many of the columns analyzed had nominally pinned base connections to their footings. Longitudinal reinforcement was stopped above the connection, and the core diameter of the column was reduced by placing an annular ring of expansion joint material around the circumference of the column. Hinge

reinforcement, typically consisting of four 36 mm dia. bars, was placed at the section center. In fact, these connections can transmit moments of considerable magnitude. Moment capacity was assessed using the reduced section diameter, with the dowel rebar acting as central tension reinforcement.

Member stiffness was calculated from the moment-curvature curve at first yield of longitudinal reinforcement. Thus

$$(EI)_{\text{eff}} = \frac{M_y}{\phi'_y} = \frac{M_N}{f_{ye}} \quad (\text{A-4})$$

where M_y and ϕ'_y are moment and curvature at first yield, M_N is the nominal strength, and ϕ_y is the equivalent bilinear yield curvature. Figure A1 clarifies the definition of these terms.

The value of $(EI)_{\text{eff}}$ from Eqn. (A-4) was used to calculate yield displacements of columns, on the assumption that EI is constant over the column height. Results of experiments have shown this to be a reasonable approximation.

Computed yield displacements included effects of cap beam, footing and superstructure flexibility where appropriate. Often these were assessed from simplified, approximate relationships, rather than from full 3-D frame analyses.

Effective material properties assumed for moment-curvature analyses were as follows:

Grade 40 rebar: $f_{ye} = 303 \text{ MPa}$

Grade 60 rebar: $f_{ye} = 454 \text{ MPa}$

3,000 psi or 3,250 psi concrete: $f'_{ce} = 34.5 \text{ MPa}$

For higher strength concrete: $f'_{ce} = 1.5 \times f'_c$ nominal.

Shear Response

The shear strength of column sections was compared with the shear V_{if} at maximum flexural response. Thus for a member of length ℓ developing moments M_t and M_b at top and bottom

$$V_{if} = (M_t + M_b)/\ell \quad (\text{A-5})$$

Note that this corresponds to nominal flexural strength at material strengths as noted above, and is thus not a maximum feasible, or "plastic" shear demand.

Shear strength was based on the model developed by Priestley et al [2]. Thus ideal shear strength is

$$V_i = V_c + V_s + V_p \quad (\text{A-6})$$

where
$$V_c = 0.8A_{\text{gross}} \times k \sqrt{f'_{ce}} \quad (\text{A-7})$$

$$V_s = \frac{A_v f_{yh} D'}{s} \cot 30^\circ \quad (\text{rectangular section}) \quad (\text{A-8a})$$

$$= \frac{\pi A_{sp} f_{yh} D'}{2s} \cot 30^\circ \quad (\text{circular sections}) \quad (\text{A-8b})$$

and
$$V_p = P \tan \alpha \quad (\text{A-9})$$

In Eqn. (A-6) V_c , V_s and V_p are the concrete contribution, transverse reinforcement contribution, and axial load contribution, respectively, to shear strength. Figure A2 describes the degradation of k in Eqn. (A-7) with increasing ductility, the definition of D' in Eqn. (A-8) and the meaning of $\tan \alpha$ in Eqn. (A-9). A full description of the model for shear strength is available in reference [2]. Two limit values, V_{ni} and V_{nd} corresponding to initial and final values for k result from Eqn. (A-10), and allow interpolation to find the member ductility at which shear failure occurs, if $V_{nd} < V_{if}$. Thus the following equations apply:

Equation (A-10): Ductility at Shear Failure

SHEAR	DUCTILITY UNIAXIAL RESPONSE	BIAXIAL RESPONSE
$V_{if} \geq V_{ni}$	$\mu_s \leq 1$	$\mu_s \leq 1$
$V_{if} \leq V_{nd}$	no shear failure, $\mu = \mu_\alpha$	no shear failure, $\mu = \mu_\alpha$
$V_{nd} \leq V_{if} \leq V_{ni}$	$\mu_s = 2 + 2 \frac{(V_{ni} - V_{if})}{(V_{ni} - V_{nd})}$	$\mu_s = 1 + 2 \frac{(V_{ni} - V_{if})}{(V_{ni} - V_{nd})}$

In the above table, μ_s and μ_α are the member displacement ductility at shear failure and flexural failure, respectively. Expected mode of failure and associated ductility were calculated for all columns. Where member and structural displacement ductility levels were expected to differ significantly, an equivalent approach based on curvature ductility was applied (2). The approach for shear strength outlined above has recently been slightly modified to provide improved representation of effects of axial load, column aspect ratio and longitudinal reinforcement ratio.

Lateral "Push" Analyses for Equivalent Elastic Strength

In each case the superstructure was considered to be a rigid diaphragm. The center of rigidity, total transverse stiffness and rotational stiffness were calculated, and hence an effective stiffness at the center of mass computed. The vector of bent displacements including both translational and rotational components was then compared with displacement capacities at predicted

ultimate capacity to identify the critical bent and mode of failure. Thus, using the generic frame of Fig. A3, the effective stiffness K at the center of mass is given by

$$\frac{1}{K} = \frac{1}{\sum K_i} + \frac{\bar{x}^2}{\sum K_i x_i^2} \quad (\text{A-11})$$

where $\bar{x}$ is the distance between center of mass and center of rigidity.

The natural period is then

$$T = 2\pi \sqrt{\frac{M}{K}} \quad (\text{A-12})$$

where M is the total mass of the frame, and the vector of bent displacements for unit lateral response force is then

$$\delta_i = \frac{1}{\sum K_i} + \frac{x_i^2}{\sum K_i x_i^2} \quad (\text{A-13})$$

If Δ_i is the vector of bent failure displacements, calculated as described above

$$V_E = \min. \left| \frac{\Delta_i}{\delta_i} \right|_i \quad (\text{A-14})$$

defines the equivalent elastic response shear force, based on an "equal displacement" approximation of response, and identifies the critical bent. The actual response force at failure will be less than V_E if inelastic response results. For simplicity, in these analyses, the inelastic response was approximately tracked by assuming the center of rigidity did not shift, but using a response force of $V_{Ei} \leq V_{Eui}$ where V_{Eui} is the computed ultimate lateral resistance. This results in a total response inertia force at failure of

$$V_u \leq V_E \quad (\text{A-15})$$

For long period structures, the equivalent elastic response force may be taken as that given by Eqn. (A15). However, for shorter period structures, the "equal displacement" approach is nonconservative, and an equivalent elastic response is calculated from the approximate relationship [2]:

$$\frac{V_E}{V_u} = 1 + (\mu - 1) \frac{T}{0.7} \quad (\text{A-16})$$

where μ is the assessed ductility based on a bilinear approximation of response. Equation (A-16) assumes average soil conditions appropriate for alluvial deposits.

The equivalent elastic spectral response acceleration at failure is then given by

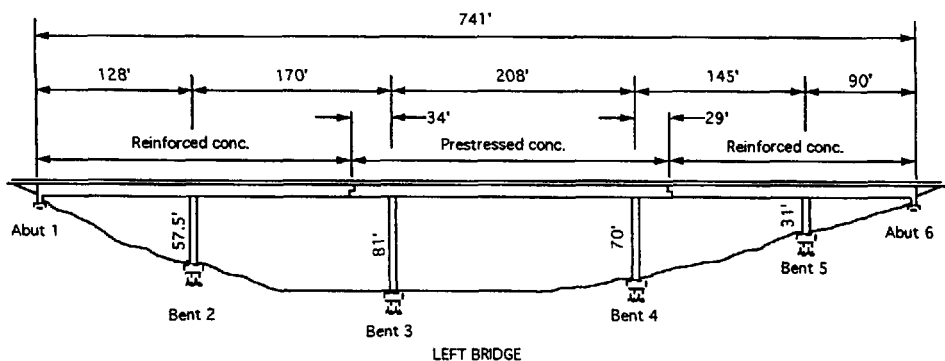
$$S_a(g) = \frac{V_E}{W} \quad (A-17)$$

where W is the weight of the bridge or frame considered. Since response spectra were not available for the sites at the time of performing the analyses the spectral shape of the Castaic record of the 1971 San Fernando earthquake was used to relate spectral accelerations to peak ground accelerations.. Since this was recorded on typical Los Angeles basin soils it is felt that it should reasonably represent the typical bridge sites. However, it must be recognized that considerable variations from this spectral shape are possible.

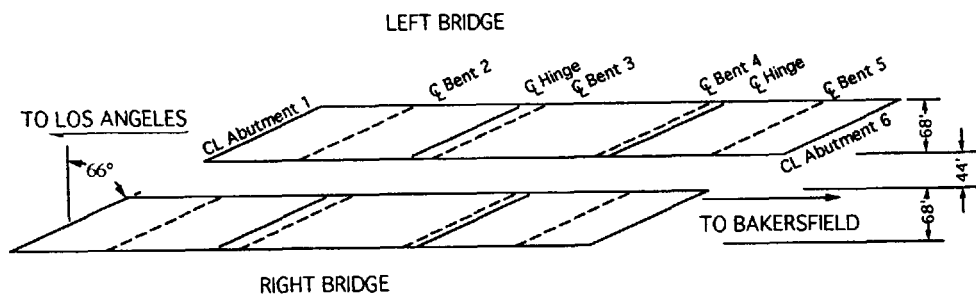
TABLE 1—SUMMARY OF HIGHWAY BRIDGE COLLAPSES IN NORTHRIDGE EARTHQUAKE.

Bridge	Route	Design	Construction Completion	Distance to Epicenter	Est. Peak Ground Acceleration	Probable Cause of Collapse
Gavin Canyon Undercrossing	I-5	1964	1965	14 km	0.4 g	Skew geometry and unseating of expansion joints
North Connector Overcrossing	SR-14/I-5	1968	1974	12 km	0.5 g	Short column brittle shear failure
South Connector Overcrossing	SR-14/I-5	1968	1974	12 km	0.5 g	Short column shear failure
Mission & Gothic Undercrossing	SR-118	1973	1976	6 km	0.8 g	Flexure/shear failure in architectural flared columns at bottom of flare
Bull Creek Canyon Channel Undercrossing	SR-118	1973	1976	6 km	0.8 g	Flexure/shear failure in shortened columns by channel wall & low transverse reinforcement ratio
Fairfax & Washington Undercrossing	I-10	1962	1964	24 km	0.3 g	Flexure/shear failure of short and stiff columns
La Cienega & Venice Undercrossing	I-10	1962	1964	24 km	0.3 g	Brittle shear failure of stiff columns

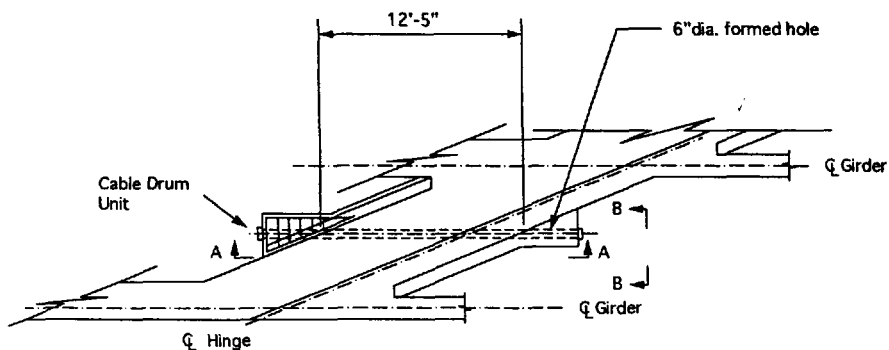




a) Elevation



b) Plan



c) Restrainer Detail

Fig. 3—Movement joint and restrainer geometry in I-5 Gavin Canyon [3]; (a) elevation; (b) plan; and (c) restrainer detail (1 in. = 25.4 mm; 1 ft = 0.305 m).

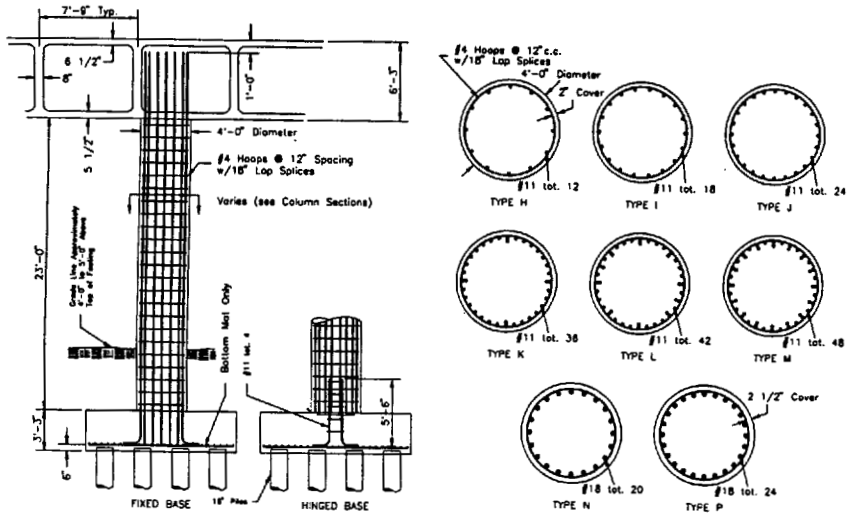


Fig. 4—Typical column geometries and reinforcement details in I-10 La Cienega-Venice undercrossing [3]; (a) column elevations; and (b) column sections (1 in. = 25.4 mm; 1 ft = 0.305 m).



Fig. 5—Shear distress pattern at I-10, Fairfax and Washington.