COMPETING AGAINST TIME

Report to Governor George Deukmejian
from
The Governor's Board of Inquiry
on the 1989 Loma Prieta Earthquake

George W. Housner, Chairman

May 1990

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Paul C. Jennings
Walter Podolny, Jr.
Alexander C. Scordelis
Robert E. Wallace

Charles C. Thiel Jr., Editor

May 31, 1990

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Acknowledgements

The Board of Inquiry appreciates the opportunity given to them by Governor George Deukmejian to examine the engineering implications of the Loma Prieta earthquake. He provided the environment in which our recommendations were developed solely on the basis of their merit.

The Board would like to express its appreciation to all those who directly and indirectly supported its efforts. It received the full cooperation and support of all the individuals and institutions that appeared before it. Individuals and organizations from State and local government, universities, professional associations, and the public unselfishly contributed their time and expertise to the deliberations of the Board. Without exception, those who appeared before the Board gave thoughtful and knowledgeable testimony. Officials and staff from all State, local, and independent agencies appearing before the Board were very cooperative, including: Department of Transportation; Department of Conservation, Division of Mines and Geology; Highway Patrol; San Francisco Bay Conservation and Development Commission; State Board of Registration for Professional Engineers; Department of Water Resources, Division of Dam Safety; Department of Finance; Seismic Safety Commission; Golden Gate Bridge District; Bay Area Rapid Transit District; and, the City of Los Angeles. Caltrans provided forthright testimony and expended considerable effort in providing information and data requested by the Board; the helpful cooperation of J.R. Roberts and J.H. Gates is acknowledged.

The efforts of many individuals contributed to the Board's work. Dr. John F. Hall, who served as Technical Secretary to the Board, and Mr. Ben Williams, who served as Administrative Officer, made important contributions to the completion of the report. Dr. Pat Reddy provided technical support to the Chairman and Mr. Peter Milne assisted the Board with meeting arrangements. The studies and testimony by the research teams from the University of California at Berkeley were particularly important to the Board's understanding of the Cypress Viaduct and Bay Bridge failures: S. A. Mahin, J.P. Moehle, A. Astaneh, D.K. Nims , E. Miranda, I.D. Aiken, A.S. Whittaker, and V.V. Bertero, investigators. Prof. Abolhassan Astaneh provided valuable insight on the performance of the Bay Bridge and Dr. Andrew S. Whittaker was particularly helpful in understanding what happened to the San Francisco Freeway Viaducts.

The editing of this report was done by Gail H. Shea, graphic design was done by Laura H. Moger, and line art was produced by Printz of San Francisco. The Board appreciates their efficient cooperation.

Finally, Dr. Charles C. Thiel Jr. undertook the technical task of organizing and editing the contributions of Board members into a coherent form, and also contributed portions of the text. His efforts are greatly appreciated.

George W. Housner
Chairman

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Preface

Governor George Deukmejian appointed an Independent Board of Inquiry to report on the October 17 , 1989 Loma Prieta earthquake. The formation of the Board was prompted by earthquake damage to bridges and freeway structures and the desire to know not only what happened, but how to prevent such destruction in future earthquakes. The Governor charged the Board with reporting on the causes of damage and what implications these findings have on the California highway system.

Future earthquakes in California are inevitable. Earthquakes larger than Loma Prieta with more intense ground shaking will occur in urban areas and have severe consequences—too large to continue "business as usual." The vast majority of structures that will fail in future earthquakes exist now—bridges, buildings, industrial facilities, and utilities. The Board of Inquiry has identified three essential challenges that must be addressed by the citizens of California, if they expect a future adequately safe from earthquakes:

- Ensure that earthquake risks posed by new construction are acceptable.
- Identify and correct unacceptable seismic safety conditions in existing structures.
- Develop and implement actions that foster the rapid, effective, and economic response to and recovery from damaging earthquakes

These challenges are addressed not only to the design and construction of bridges, whose failure prompted the Board's formation, but also to all the other constructed facilities upon which our modern economy and well-being depend.

The Loma Prieta earthquake should be considered a clear and powerful warning to the people of California. Although progress has been made during the past two decades in reducing earthquake risks, much more could have been done, and awaits doing. More aggressive efforts to mitigate the consequences of earthquakes are needed if their disastrous potential is to be minimized and one of the most fundamental of responsibilities of government is to be fulfilled—to provide for the public safety.

The State of California must not wait for the next great earthquake, and the likely tens of billions of dollars damage and thousands of casualties, to accelerate hazard mitigation measures. California must recognize that it has an earthquake problem that can be mitigated. It is hoped that the Board's findings and recommendations will provide a positive impetus toward actions that will provide adequate earthquake safety.

The Board of Inquiry urges the public and the State to take the actions necessary to implement its recommendations. Earthquakes will occur—whether they are catastrophes or not depends on our actions.

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Chapter 1

Overview

Future earthquakes in California are inevitable. They represent a clear and continuing danger to our population and economy. The consequences of severe earthquakes in urban areas will be extensive—too large for "business as usual." It is time for California to set priorities for seismic safety. Research and development must be supported to meet our needs for effective, economic strategies for earthquake hazards mitigation. We must act forcefully to reduce the risks, vulnerability, and exposure we all share.

While the principal responsibility for California's earthquake safety lies in the hands of individuals, government has an important role. The most fundamental responsibility of government is to provide for the public safety. Governor George Deukmejian appointed the Board of Inquiry on the Loma Prieta earthquake of October 17, 1989 to determine the cause of collapses of the Cypress Viaduct and Bay Bridge span and to recommend actions based on this experience that will limit the impacts of future earthquakes. This document is the response of the Board of Inquiry to the Governor's charge.

The vast majority of structures that will fail in future earthquakes exist now—bridges, buildings, industrial facilities, and utilities. The citizens of California are captives of these existing hazards *unless* new approaches for mitigating them are developed and applied, measures that design professions, businesses, individuals, and government officials can economically use.

Every Californian is affected by the occurrence of a major earthquake, whether expressed as direct damage, indirect loss of utilities, increased taxation, or reduced economic activity. Recent scientific research tells us that there is a high probability of a major earthquake in both southern and northern California within the next few decades. The Loma Prieta earthquake of October 17, 1989, a Magnitude 7.1 earthquake that occurred in a sparsely settled portion of the Santa Cruz mountains, caused damage approaching $6 billion and the collapse of buildings and structures as far away as 60 miles in San Francisco and Oakland.

The Loma Prieta earthquake was but one in a series of recent damaging earthquakes; others since 1970 include Whittier Narrows, San Fernando, Coalinga, Morgan Hill, Ferndale, Imperial County, Palm Springs, Livermore Valley, Mammoth Lakes, Santa Barbara, and Oroville. Each of these events has caused structures to fail. In many cases those that failed could have been identified as being at high risk in an earthquake. Each of these events was serious and calamitous to those affected, even though each was relatively minor compared to the earthquakes that are expected to occur near metropolitan areas along the northern and southern San Andreas, Hayward, or Newport-Inglewood faults, to name but a few active faults within the State.

In 1980 the National Security Council estimated that a single California earthquake has the potential to cause from 20 to 80 billion dollars in damage and kill tens of thousands. In terms of today's values, total damage impacts would be at least twice as large and would exceed $100 billion for several of their postulated earthquakes. These are only the direct physical damage costs, not the myriad other costs incurred as a result of the damage. Californians are justifiably concerned and should plan for these so-called "big ones." However, we must also recognize that small and moderate earthquakes, which are much more frequent, can cause severe, widespread damage and casualties.

Governor's Directives to the Board of Inquiry

The Governor directed the Board of Inquiry to address five specific issues arising from the Loma Prieta earthquake:

1. To determine why the Cypress Viaduct of Interstate 880 and the Bay Bridge span failed in the earthquake.
2. To determine whether these failures were or could have been foreseen.
3. To advise on how to accurately predict possible future bridge and structure failures.
4. To determine if the schedule for and manner of retrofitting these structures properly utilized the seismic and structural information that has been developed following other earthquakes in California.
5. To make recommendations as to whether the State should modify the existing construction or retrofit programs for freeway structures and bridges in light of new information gained from this earthquake.

To this group the Board has added the question that is on the minds of many Californians:

6. Are California's freeways earthquake-safe?

Summary responses to these issues, which have been recast as questions, are given below. Background information and more detailed discussion of these responses are to be found in the other chapters of this Report.

1 Why did the Cypress Viaduct of Interstate 880 and the Bay Bridge span fail in the earthquake?

The Cypress Viaduct and Bay Bridge appear to have no design or construction deficiencies as measured by bridge design practices in effect at the time they were built, nor is there evidence of subsequent maintenance deficiencies that contributed to their failure. However, the practice of earthquake engineering has improved substantially from that of the periods during which the Cypress Viaduct (1950s) and San Francisco-Oakland Bay Bridge (1930s) were designed and constructed.

The Cypress Viaduct was designed and constructed to Caltrans seismic requirements for reinforced concrete when it was built in the 1950s. Caltrans and the American Association of State Highway and Transportation Officials (AASHTO) seismic design codes had very low seismic requirements at the time of design of the Cypress Viaduct compared to those for buildings as specified by the Uniform Building Code of the same period and compared to current standards. The Cypress Viaduct was a brittle structure, possessing very little ductility, which was consistent with the practices of the period. It is generally agreed that soft ground, such as that beneath the collapsed sections of the Cypress Viaduct on the border of San Francisco Bay, amplified ground motions in this earthquake more than anticipated by current codes. However, these ground motions were not as high at Cypress as those expected in likely, large earthquakes. The combination of nonductile reinforced concrete, inopportune placement of hinges in the upper story columns to accommodate future proposed widening and post-tensioning required in some girders, and weak second story column pedestals coupled with the large mass of the decks led to severe overstressing of the pedestals and columns supporting the upper deck, failure of the pedestals below the column shear keys, and the collapse of the structure.

The Bay Bridge was designed for 10% g static equivalent loadings, comparable to the levels specified for earthquakes in the 1930 Uniform Building Code for buildings. The 50-foot-long upper and lower closure spans of the Bay Bridge over Pier E9 fell during the Loma Prieta earthquake. Ground shaking at the base of the bridge during the earthquake caused response motions of the bridge sufficiently large to shear the bolts that connected the pier and the 290' truss-span to the east. The 50' closure spans linking the two long truss-spans on the west and east sides of the pier were supported on six-inch-wide expansion-type bearing seats at the west end and bolted connections at the east end. When the truss-span to the east broke free from its supports, the closure spans were pulled with it, and the motions were large enough to slide them off their six-inch bearing seats. As a result, the spans hinged down under gravity, with the upper span coming down on the lower one. These then struck an electrical housing before coming to rest on the west truss-span connections to the pier. Another closure span at pier E23, close to the eastern edge of the bridge, was near failure of a comparable type.

2 Were these failures foreseen or could they have been foreseen?

No evidence was presented to the Board suggesting that Caltrans was specifically aware of the earthquake hazards that caused the failures of the Cypress Viaduct or the San Francisco-Oakland Bay Bridge. While there had been some seismic strengthening of both structures by the installation of cable restrainers, there is no other evidence that Caltrans had identified either as especially earthquake vulnerable.

Preparedness Plans developed by the State Office of Emergency Services and Division of Mines and Geology for Hayward and San Andreas earthquakes included assumptions that major bridges would be out of service, but not because of failure or collapse of the bridges themselves. Rather they assumed damage to approaches. It should be emphasized that these plans were formulated as planning assumptions for preparedness and were not based on any engineering assessment of the expected performance of the bridges themselves.

The issue of whether these failures could have been foreseen in the Loma Prieta earthquake is a difficult one because of the uncertainties and lack of previous studies. The fiscal environment at Caltrans in the last two decades seems to have inhibited giving the necessary attention to seismic problems. Many items ranging from research on earthquake engineering to seismic retrofitting were placed in low priority because of the limited possibility of funding due to budget constraints.

The Board of Inquiry concludes that an engineering seismic assessment of the Cypress Viaduct before the earthquake, performed after 1971 by a professional engineering organization in a manner consistent with the care and expertise usually exercised in evaluating such important structures, would have concluded that a collapse could be expected during a nearby major earthquake on the San Andreas or Hayward faults. Damage, but not extensive collapse, would have been expected for an earthquake with Loma Prieta's magnitude and location. Collapse would have been anticipated for the intensity of ground motion observed in the Loma Prieta earthquake; however, the extent of the collapse that actually occurred would probably not have been anticipated.

The Cypress Viaduct was a nonductile concrete structure. It has been common knowledge within the structural engineering community for the past ten years that nonductile reinforced concrete structures are particularly earthquake vulnerable. Most Caltrans concrete bridges constructed before 1971 have nonductile details. Caltrans instituted design changes in 1971, following the San Fernando earthquake, that required new construction to utilize ductile details. However, for the Cypress Viaduct retrofit of 1977 a prior decision in the retrofit program dictated that the limited available funds should be used to install longitudinal restraints at the transverse expansion joints in the box girder spans. This was done to prevent failures of the type experienced in the 1971 San Fernando earthquake. Unfortunately, no detailed comprehensive analysis of the entire structure-soil system was made until the time of the failure of the Viaduct on October 17, 1989 to determine if other weaknesses existed. If such an analysis had been made the Board believes the failure would have been predicted.

The Board thinks that a comprehensive seismic analysis of the Bay Bridge conducted before the Loma Prieta earthquake, performed after 1971 by a professional engineering organization in a manner consistent with the care and expertise usually exercised in evaluating such important structures, would have concluded that it had serious seismic deficiencies.

The Bay Bridge is a very large and complicated structure made of steel and concrete and has foundations extending to rock or stiff soils through very soft, water-saturated soils. The assessment of seismic performance and possible damage to such a complex structure requires an unusually thorough and detailed investigation. Had such a study been made it probably would have identified the possibility of collapse of the link span in addition to other hazards.

3 How may possible future bridge and structure failures be accurately predicted?

At present earthquakes cannot be scientifically predicted. Thus, predicting the possibility of failures is confined to determining whether the bridge or structure could fail under a given level of ground shaking. Predicting this failure potential requires: first, that the level of ground shaking be determined for the site; and second, that an engineering assessment of expected performance be made for this ground shaking. The ground shaking used for such a failure analysis should be the ground shaking expected for the site determined from a probabilistic risk assessment, with a sufficiently small probability of exceedance during the projected lifetime of the structure, consistent with its importance. Special seismic analyses must be made to determine a structure's potential for collapse under these motions.

Faced with an inventory of over 11,000 State-owned and a comparable number of locally-owned bridges, it is not expected all can or should be assessed with such rigor. Since design standards used by Caltrans after 1971 were better than those used before, it is reasonable to expect that the older ones pose a greater risk. Application of risk analysis procedures that consider the frequency of occurrence of different levels of ground motion and the characteristics of the structure (configuration, materials, foundations, soils, age and condition) could reduce this to a manageable list of potentially hazardous structures. Locations with a potential for ground failure, e.g. liquefaction and lateral spreading, deserve special attention. Engineering analyses would then determine the potential for failure and the engineering steps required to meet the performance goals recommended to the Governor by the Board for new and existing transportation structures.

The Recommendations of the Board in the section that follows, when fully implemented, are intended to give reasonable assurance that both existing and new bridges and transportation structures provide adequate seismic safety.

4 Did the schedule for and manner of retrofitting these structures properly utilize the seismic and structural information that has been developed following other earthquakes in California?

Several freeway bridges collapsed in the 1971 San Fernando earthquake when decks were pulled off of their supports at expansion joints. The decks fell, causing failure of the bridges. This happened even though no direct damage was necessarily done to the bridge elements themselves by the ground shaking. This type of hazard was the weak link on many bridges that existed in 1971. In response, Caltrans adopted a cable restrainer seismic retrofit program to tie decks together and to their abutments to prevent such failures. It took 17 years and expenditures of $54 million to complete this program. The Board believes this program significantly increased the seismic resistance of many structures. The Board's only major criticism is that 17 years was too long for completion of such an important program.

The Cypress Viaduct and the San Francisco-Oakland Bay Bridge had been retrofitted with cable restrainers to limit the relative motion between adjacent decks at expansion joints. No special seismic analyses were made of these structures. Following the 1971 earthquake, seismic design procedures for new structures were modified to include ductile detailing of concrete, but no special efforts were made to retrofit existing nonductile concrete bridge elements.

The near collapse of the I-5/I-605 overpass in the 1987 Whittier Narrows earthquake emphasized the need for strengthening nonductile concrete bridge columns. An ongoing research project was accelerated and an inventory was made of high-hazard-potential single column bridges. The Cypress Viaduct, being a multiple column bridge, was not identified as a high priority structure for attention.

Early in the retrofit program, Caltrans considered the performance of individual elements (restraining motion at expansion joints). Caltrans did not consider the response of the whole structure or the soil-structure system. This focus on elements, in hindsight, may have inhibited the likelihood of identifying problems in overall seismic behavior such as those uncovered in the failure of the Cypress Viaduct and the Bay Bridge.

The repair of the Bay Bridge appears to be appropriate for the short-term. The fact that the Bay Bridge was only slightly damaged during the Loma Prieta earthquake and the repair completed does not mean that the bridge may now be presumed to be adequately earthquake resistant. The expected performance of the bridge during major earthquake loadings should be assessed by comprehensive state-of-the-art methods in earthquake engineering analysis to determine what seismic upgrading should be completed to ensure adequate performance.

The San Francisco Freeway Viaducts are of substantially comparable design and construction to those of the Cypress Viaduct and could be expected to suffer severe damage and possibly collapse if they were subjected to more intense ground motions or ground motions of longer duration. The installation of cable restraints under the Caltrans seismic retrofit program appears to have improved their behavior, possibly saving some spans from collapse by limiting the relative displacements of the decks at the expansion joints.

The repair of some of the San Francisco Freeway Viaducts is already underway. The proposed retrofitting schemes are expected to substantially strengthen the columns, but the precise degree of improvement in seismic resistance of the structures will not be clear without detailed studies and analyses The Board is unable to evaluate the specific details of the retrofit designs and programs for the individual viaducts. It considers them to be only short-term approaches to their repair.

Substantially more engineering analysis and evaluation will be required to determine if additional seismic retrofitting may be appropriate for the Bay Bridge and San Francisco Freeway Viaducts to make them appropriately safe in the long-term.

5 Should the State modify the existing construction or retrofit programs for freeway structures and bridges in light of new information gained from this earthquake?

Yes.

This earthquake has demonstrated the fact that nonductile structures designed prior to 1971 can fail in a brittle manner with consequent collapse. It also emphasized that the intensity of ground shaking on soft soils may be greater than is anticipated by current seismic codes. This evidence therefore requires modifications to both the existing Caltrans retrofit program and to new design standards, as contained in the Recommendations and Findings of this Report. Caltrans appears to be vigorously modifying their technical approaches and standards for each.

The existing State-wide Caltrans seismic retrofit program should continue to consider the overall behavior of transportation structures and foundations and not be principally focused on the behavior of structural elements. It should be enhanced by the assignment of greater personnel and budgetary resources so that the retrofit program can be implemented and completed within this decade.

Most of California's reinforced concrete bridges were designed and built before the 1970s and many are deficient in their earthquake resistance. This was caused by the slow development through research of new knowledge in earthquake engineering for bridge design and the usual lag in putting research results into practice. The quality, effectiveness and economy of new construction and seismic upgrading will be enhanced substantially if a vigorous research program is undertaken on earthquake engineering, as opposed to the limited and occasional efforts of the past.

The fiscal environment at Caltrans in the past 20 years inhibited Caltrans from giving more attention to seismic problems. This environment has improved following the Loma Prieta earthquake. Increased attention, funds and personnel resources are being devoted to earthquake safety.

6 Are California's freeways earthquake-safe?

Most are, but some are not.

Among the more than 11,000 structures in the State highway system there are some that have the potential for severe damage and collapse in the event of a worst-case earthquake. These warrant prompt, systematic correction, but the Board of Inquiry is not aware of any that warrant closure, based on the Board's understanding of past Caltrans seismic design and construction practices. The occasional earthquake life-safety risk posed by highway structures is different than those continuously posed by traffic conditions. Earthquakes large enough to pose a threat have relatively low probability of occurring at a given location.

The Board thinks existing high hazard structures can and should be corrected in a planned and accelerated program. With the implementation of the Board's Recommendations, Caltrans can complete the identification of these structures and carry out the required seismic retrofitting. Then the freeway structures will be appropriately safe.

Three Challenges

The Board of Inquiry has identified three essential challenges that must be addressed by the citizens of California, if they expect a future adequately safe from earthquakes:

- Ensure that earthquake risks posed by new construction are acceptable.
- Identify and correct unacceptable seismic safety conditions in existing structures.
- Develop and implement actions that foster the rapid, effective, and economic response to and recovery from damaging earthquakes.

These challenges address not only the issue of bridges, whose failure prompted the Board's formation, but also all the other constructed facilities upon which our modern economy and well-being depend. The Board might have limited its recommended actions only to those it believes necessary to correct problems with State-owned bridges. But to do so would be to abdicate consideration of the most fundamental responsibility of government—to provide for the public safety. The Board has interpreted its Charter in a broader sense and has made recommendations that are directed both at seismic issues for bridges and some of the larger issues of seismic safety facing the State.

The Board has developed eight Recommendations for implementation.

Recommendations

For Action by the Governor:

1 Affirm the policy that seismic safety shall be a paramount concern in the design and construction of transportation structures. Specific goals of this policy shall be that all transportation structures be seismically safe and that important transportation structures maintain their function after earthquakes.

2 Establish that earthquake safety is a priority for all public and private buildings and facilities within the State by taking the following actions:

A. Propose legislation to ensure that every new facility in the State not otherwise subject to adequate seismic regulation and having the potential to cause substantial life loss during an earthquake be subject to compliance with adequate seismic safety standards for construction.

B. Require that seismic safety be a paramount concern in the design and construction of all State-owned structures. Specific goals of this policy shall be that all State-owned structures be seismically safe and that important State-owned structures maintain their function after earthquakes.

C. Initiate and fund a vigorous, comprehensive program of research to improve the capability in engineering and the physical and social sciences necessary to mitigate earthquake hazards and to implement the technology transfer and professional development necessary to hasten practical use of research results.

3 Direct the Seismic Safety Commission to review and advise the Governor and Legislature periodically on State agencies' actions in response to the Recommendations of this Board of Inquiry.

For Action by the Director of the Department of Transportation:

4 Prepare a plan, including schedule and resource requirements, to meet the transportation seismic performance policy and goals established by the Governor. The plan shall include the timely seismic retrofitting of existing transportation structures.

5 Form a permanent Earthquake Advisory Board of external experts to advise Caltrans on seismic safety policies, standards, and technical practices.

6 Ensure that Caltrans seismic design policies and construction practices meet the seismic safety policy and goals established by the Governor:

A. Review and revise standards, performance criteria, specifications, and practices to ensure that they meet the seismic safety goal established by the Governor and apply them to the design of new structures and rehabilitation of existing transportation structures. These standards, criteria, and specifications are to be updated and periodically revised with the assistance of external technical expertise.

B. Institute independent seismic safety reviews for important structures.

C. Conduct a vigorous program of professional development in earthquake engineering disciplines at all levels of the organization.

D. Fund a continuing program of basic and problem-focused research on earthquake engineering issues pertinent to Caltrans responsibilities.

7 Take the following actions for specific structures:

A. Continue to sponsor and utilize the Independent Review Committee's technical reviews of the engineering design and construction proposed for the short-term repair and strengthening of the San Francisco Freeway Viaducts.

B. Develop a long-term strategy and program for the seismic strengthening of existing substandard structures, including the San Francisco Freeway Viaducts, that considers their overall behavior, the degree of seismic risk, and the importance of the structure to the transportation system and to the community.

C. Perform comprehensive earthquake vulnerability analyses and evaluation of important transportation structures throughout the State, including bridges, viaducts, and interchanges, using state-of-the-art methods in earthquake engineering.

D. Implement a comprehensive program of seismic instrumentation to provide measurements of the excitation and response of transportation structures during earthquakes.

For Action by Transportation Agencies and Districts:

8 Agencies and independent districts that are responsible for transportation systems—rail systems, highway structures, airports, ports and harbors—should:

A. Adopt the same seismic policy and goals established by the Governor for State transportation structures and implement seismic practices to meet them.
B. Perform comprehensive earthquake vulnerability analyses and evaluations of important transportation structures—e.g., the BART Trans-bay Tube and Golden Gate Bridge—using state-of-the-art methods in earthquake engineering, and install seismic instrumentation.
C. Institute independent seismic safety reviews for important structures.
D. Conduct a vigorous program of professional development in earthquake engineering disciplines at all levels of their organizations.

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Chapter 2

Governor's Board of Inquiry

Following the October 17, 1989 Loma Prieta earthquake and the tragic loss of life caused by the collapse of the Cypress Viaduct and the failure of a span of the San Francisco-Oakland Bay Bridge, there was much public concern over how modern freeway structures relied upon by so many could suffer such devastating failures. In response, California Governor George Deukmejian immediately called for an independent examination of the collapse, stressing that the review "must be fair, objective and complete. The public will be satisfied with nothing less," he said. "I want the answers to the questions concerning the I-880 collapse so this tragic chapter in California's history will not repeat itself." The independent examination was to include input from the National Transportation Safety Board, Federal Highway Administration, research institutions, and private corporations.

On October 26, Governor Deukmejian appointed Dr. George W. Housner of the California Institute of Technology in Pasadena to head the independent examination. Dr. Housner is the CF Braun Professor of Engineering Emeritus for Caltech and is chairman of the Committee on Earthquake Engineering of the National Research Council. He is a member of the National Academy of Sciences and the National Academy of Engineering. He is a recipient of the prestigious National Medal of Science for his contributions to earthquake engineering.

Membership of the Board of Inquiry

In consultation with Professor Housner, the Governor selected ten other individuals to serve on the examining team, which was entitled "The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake." The Board of Inquiry was created by Executive Order D-83-89, which was signed by Governor Deukmejian on November 6, 1989 (Figure 2-1).

It is clear that certain considerations were paramount in selecting members for the Board of Inquiry. First, the members had to be well qualified to judge the highly technical information that would be presented to them. The composition of the Board supports this—six are members of the National Academy of Engineering, the most prestigious association of the profession; three members are past-presidents of the Earthquake Engineering Research Institute; and two members are medalists of the Seismological Society of America, the Society's highest award. Combined, the 11 members have over 350 years of experience in engineering, seismology, geology, architecture and design, and other earthquake sciences.

Second, the importance of California representation was apparent. All members are California scientists and engineers, except the two federal representatives from Washington, D.C. In fact, a third federal representative named to the Board from the United States Geological Survey, resides in the Bay Area.

EXECUTIVE DEPARTMENT
STATE OF CALIFORNIA

EXECUTIVE ORDER D-83-89

WHEREAS, Northern California was struck by a devastating earthquake on October 17, 1989, with a magnitude of 7.1 on the Richter scale; and

WHEREAS, this earthquake caused the Cypress Structure of Interstate 880 in Oakland and a span of the San Francisco-Oakland Bay Bridge to collapse, resulting in numerous deaths and injuries, as well as property damage; and

WHEREAS, it is necessary to ascertain the cause of these failures as expeditiously as possible so that additional actions may be taken to ensure the maximum possible safety of California's highways, bridges, and other structures in future earthquakes; and

WHEREAS, it is in the public interest that an investigation be conducted independent of any engineering studies that must be performed by Caltrans;

NOW, THEREFORE, I, GEORGE DEUKMEJIAN, Governor of the State of California, by virtue of the power and authority vested in me by the Constitution and Statutes of the State of California, do hereby issue this Order, to become effective immediately:

1. The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake is hereby created to investigate the collapse of the Cypress Structure of Interstate 880 and a span of the San Francisco-Oakland Bay Bridge.

2. The Board of Inquiry shall consist of 11 experts in fields related to civil, structural, and seismic earthquake engineering and design, and earthquake science. The Board shall be chaired by Dr. George W. Housner of the California Institute of Technology.

3. The Board of Inquiry shall have the following responsibilities:

   a. To determine why the Cypress Structure of Interstate 880 and the Bay Bridge span failed in the earthquake.

   b. To determine whether these failures were or could have been foreseen.

   c. To advise on how to accurately predict possible future bridge and structure failures.

   d. To determine if the schedule for and manner of retrofitting these structures properly utilized the seismic and structural information that has been developed following other earthquakes in California.

Figure 2-1. *Executive Order D-83-89*

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c. To make recommendations as to whether the state should modify the existing construction or retrofit program for freeway structures and bridges in light of new information gained from this earthquake.

4. The Board of Inquiry shall provide the Governor with a progress report by March 1, 1990, and a final report with its findings and recommendations by June 1, 1990.

5. The Office of Planning and Research shall provide administrative support to the Board of Inquiry and all state agencies, departments, boards, and commissions are directed to assist the Board in the implementation of this executive order.

IN WITNESS WHEREOF I have hereunto set my hand and caused the Great Seal of the State of California to be affixed this 6th day of November 1989.

George Deukmejian

Governor of California

ATTEST:

March Fong Eu

Secretary of State

Third, potential members had to indicate that they had no preconceived opinions and that they had no current contractual or other ties with Caltrans that might be perceived as a conflict of interest. Several qualified individuals were removed from consideration because of the possibility of perceived conflicts.

After a thorough review of qualifications and individual consultations, the following individuals were named to the Board of Inquiry on November 4, 1989:

Joseph Penzien, Professor Emeritus of Structural Engineering at the University of California, Berkeley, who was named vice-chairman of the Board of Inquiry. Dr. Penzien has conducted extensive research on the effects of earthquakes on bridges for the Federal Highway Administration.

Mihran Agbabian, Chairman of the Department of Civil Engineering at the University of Southern California. Dr. Agbabian specializes in earthquake engineering.

Christopher Arnold, President and cofounder of Building Systems Development, Inc. in San Mateo. Mr. Arnold is an architect who has been heavily involved in research and consultation on architectural aspects of earthquake-resistant building design.

Eric Elsesser, President of Forell/Elsesser Engineers in San Francisco. Mr. Elsesser is a civil and structural engineer who has extensive experience in designing earthquake-resistant buildings and structures.

Lemoine V. Dickinson, Jr., Member of the National Transportation Safety Board from June 1988 through February 1990. Dr. Dickinson is a civil engineer and has served in technical, research, and policy positions with the federal government and private corporations.

I. M. Idriss, Professor of Geotechnical Engineering at the University of California, Davis. Dr. Idriss specializes in geotechnical earthquake engineering, soil mechanics, and foundation engineering, and has worked both with academic institutions and for private consulting firms.

Paul C. Jennings, Vice President and Provost, California Institute of Technology. Dr. Jennings was formerly Chairman of the Division of Engineering and Applied Science at Caltech and specializes in earthquake engineering and earthquake-resistant structural design.

Walter Podolny, Jr., structural engineer with the Bridge Division of the Federal Highway Administration. Previously, Dr. Podolny has held positions in engineering management for several private firms.

Alexander C. Scordelis, Nishkian Professor of Structural Engineering at the University of California, Berkeley. Professor Scordelis specializes in research, analysis, and design of bridge and concrete structures.

Robert E. Wallace, Chief Scientist (retired), Office of Earthquakes, Volcanoes, and Engineering, U. S. Geological Survey, Menlo Park. Dr. Wallace specializes in seismic geology and California earthquakes.

Meetings Held and Testimony Recieved

The Board of Inquiry gathered its information through presentations from Caltrans and independent experts in seismology, structural engineering, geotechnical engineering, and other disciplines. Most of the information was presented at public hearings held in Sacramento, the Bay Area, and Southern California, at which times public testimony was also invited. Reports and written information were sent directly to Board members for their review.

The Board of Inquiry held seven public meetings between November 1989 and March 1990. Three of these were two-day meetings. A total of 70 individuals provided invited testimony at those meetings. The Appendix to this Report lists the presenters who appeared at each meeting.

Presentations to the Board by independent experts included:

- Accounts of the mode of failure of the Caltrans structures
- Geologic and seismic profiles of the Bay Area
- Effects of the earthquake on other structures, including office buildings, the Golden Gate Bridge, and the BART Trans-bay Tube
- Comments on all retrofit programs
- Estimates of future earthquake probabilities
- Recommendations for consideration by the Board

In addition, Caltrans presented approximately 21 hours of testimony regarding:

- State and local bridge retrofit programs
- Damage and repair plans pertaining to the Cypress Viaduct, Bay Bridge, and other State-maintained structures
- General soil and subsurface conditions in the Bay Area
- History and applicability of State and national seismic design criteria and standards
- Other issues of interest to the Board

In addition to these seven public meetings, Board members toured the Cypress test structure and several of the damaged San Francisco structures on December 13, 1989. At that time, Caltrans staff presented photographs and slides of the portions of the Cypress Viaduct that had already been demolished.

Other Information Considered by the Board of Inquiry

To supplement the oral testimony received at the public meetings, Board members also reviewed and considered an enormous amount of written material. Some of these reference documents were received from presenters to supplement or explain their verbal testimony. Other documents were sent to members at the direction of the Chairman or request of another member for use as general reference information. In addition, copies of many documents reviewed by Members of the Board during a visit to the Caltrans press room in Sacramento are maintained at Caltech, Pasadena.

The Annotated Bibliography to this Report contains a comprehensive listing of the material that was sent or made available to all of the Board members for their use. This material is maintained at the Office of Planning and Research in Sacramento and at the California Institute of Technology in Pasadena. Materials presented to the Board carry an Office of Planning and Research catalog number. The Annotated Bibliography also lists reference material consulted by Board Members in preparation of this Report.

Chapter 3

The Earthquake's Impact on Transportation Systems

The Earthquake

The Loma Prieta earthquake of October 17, 1989 (5:04 P.M., Pacific daylight time) occurred near three, large modern cities–San Jose, San Francisco and Oakland. It was named after the highest topographic point adjacent to the fault zone. The epicenter was in a sparsely populated, mountainous area. The fault rupture penetrated upward to within about 4 miles of the ground surface, but did not break the ground surface. This Magnitude 7.1 earthquake was felt from Los Angeles north to the Oregon State line, and east to western Nevada. It was the largest to occur in the San Francisco Bay area since the great San Francisco earthquake of 1906.

This was the nation's first prime time earthquake. It occurred during the opening ceremonies of the third game of the World Series, about to be played between the Oakland Athletics and the San Francisco Giants at Candlestick Park. Media coverage was intense from the very start, with live television focusing on the fires in the Marina District of San Francisco, on damage to bridges, and on search and rescue operations being broadcast to the nation well before local communities knew what had happened. Notwithstanding the emphasis of the news coverage on impacts 60 miles from the epicenter, the earthquake had major impacts on the counties to the south, as well as on Oakland and San Francisco. Two of the most dramatic impacts of the earthquake were the failures of the Cypress Viaduct and the link span of the San Francisco-Oakland Bay Bridge, both about 60 miles from the epicenter.

The Office of Emergency Services (OES) reports that the earthquake caused:

- 62 deaths
- 3,757 injuries
- Nearly 8,000 people homeless
- Property damage of $5.6 billion
- Widespread disruption of transportation, utilities, and communications

Over 1,300 buildings were destroyed and 20,000 buildings damaged. More than 3,500 business were damaged and about 400 destroyed. Thirteen State-owned and five locally-owned bridges were closed to traffic following the earthquake, a very small number considering that there are over 4,000 bridges in the area. Forty-one people died in the Cypress collapse, and one died on the Bay Bridge in a traffic accident moments after the earthquake. The cost of the earthquake to the transportation system was $1.8 billion, of which damage to State-owned viaducts totaled about $200 million and damage to other State-owned bridges was about $100 million. Fairly or not, the lasting legacy of the Loma Prieta earthquake probably will be the damage sustained by highway bridges.

The impacts of the earthquake were much more than the loss of life and direct damage. The Bay Bridge is the principal transportation link between San Francisco and the East Bay. It was out of service for a over a month and caused substantial hardship as individuals and businesses accommodated themselves to its loss.

Following the earthquake, ten counties were proclaimed State and Federal disaster

Table 3-1. *Earthquake effects for ten counties, preliminary*

County	Dead	Injured	People Displaced
Alameda	43	481	1,002
Contra Costa	0	22	0
Marin	0	0	5
Monterey	1	14	54
San Benito	0	110	412
San Francisco	13	700	0
San Mateo	0	451	0
Santa Clara	1	1,305	50
Santa Cruz	6	671	6,377
Solano	0	3	0
TOTAL	**64**	**3,757**	**7,900**

Source: Office of Emergency Services

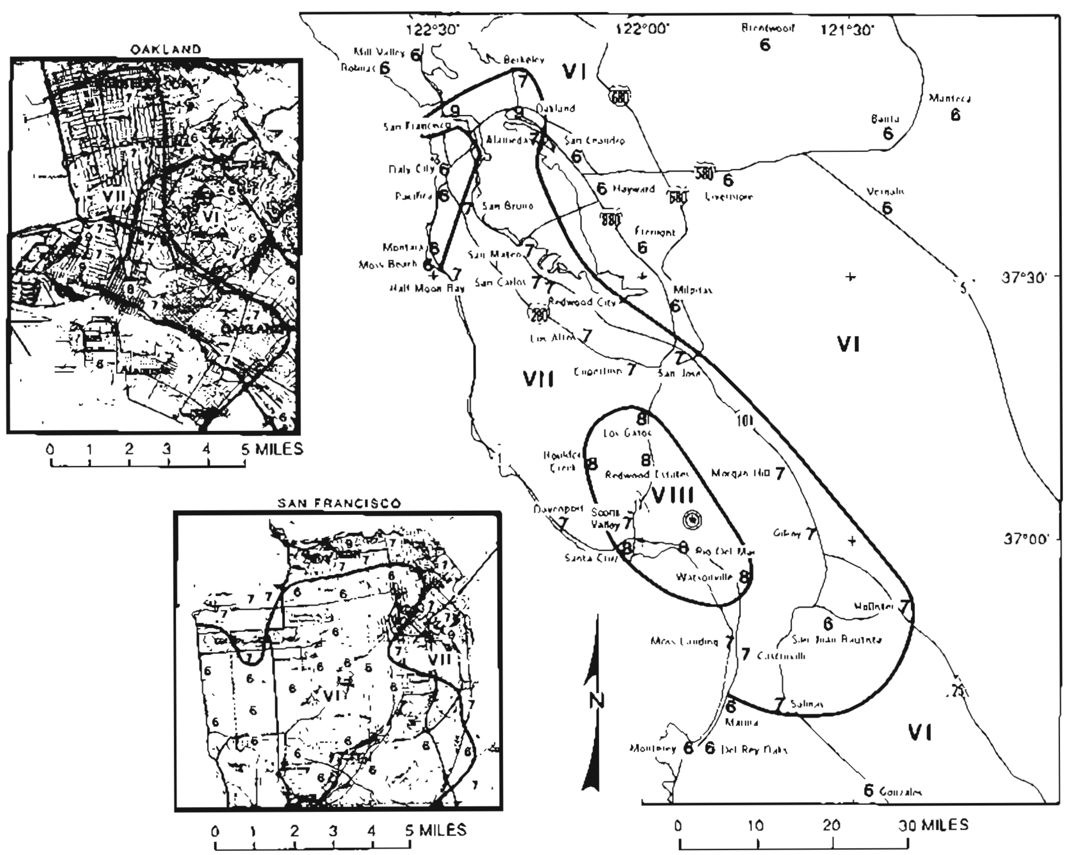

Figure 3-1. *Isoseismal map of the damage impacts of the Loma Prieta earthquake. MMI VII is termed strong and is described by the types of effects observed: weak unreinforced buildings damaged; unreinforced masonry chimneys broken at roof lines; disruption of building contents, plaster cracked. MMI VIII is termed very strong shaking: damage to nonearthquake-resistant structures can be significant, with some collapses, particularly those in poor condition; damage to nonstructural elements in modern, seismically resistant buildings, and substantial disruption of building contents and toppling of unanchored equipment*

areas—Alameda, Contra Costa, Marin, Monterey, San Benito, San Francisco, San Mateo, Santa Clara, Santa Cruz, and Solano—as were three cities—Tracy, Benicia, and Isleton. Table 3-1 shows the damage data for all the 10 counties affected as reported by the Office of Emergency Services (OES).

The Loma Prieta earthquake was not unexpected. It occurred along a segment of the San Andreas fault previously recognized as having a high potential for an earthquake of Magnitude 6.5 to 7.0, having been assigned a 30% probability of occurrence in the 30-year period beginning in 1988 [Working Group on California Earthquake Probabilities, 1988]. Other major earthquakes of equal or greater magnitude are also anticipated on the Hayward fault and other segments of the San Andreas fault — much closer to population concentrations than was the Loma Prieta earthquake.

The region affected by the strong, potentially damaging, ground motion extended from the Monterey Bay to the San Francisco-Oakland area (Figure 3-1) This area contains a wide range of modern engineered structures representing most forms of current construction: buildings, bridges, dams, tunnels, harbor works, pipelines, and manufacturing facilities. Shaking intensity was VIII on the Modified Mercalli Intensity scale (MMI) over an area of 30 miles long and 15 miles wide extending from Los Gatos to Watsonville and Santa Cruz. An outer zone of intensity VII extended 60+ miles northwest to San Francisco and Oakland, and 30 miles southeast to Salinas and Hollister. Within these regions, free-field, peak horizontal accelerations of ground motion exceeded 60% g close to the source and were as high as 26% g at a distance of 60 miles. Strong shaking lasted less than 10 seconds.

The regional damage distribution was unusual in several respects from what might have been expected. The duration of strong shaking was about half as long as is typical for a Magnitude 7.1 earthquake; ground motions were lower than would have been expected in San Jose, near the source, and higher than expected in the San Francisco-Oakland area, distant from the source. The nature of the soil conditions, both in the epicentral region and in the Bay Area, played a very strong role in the damage distribution in this as well as in most California earthquakes. The ground motions in the Bay Area at soft soil sites, where much of the damage to bridges and viaducts occurred, were significantly greater than the motions recorded at nearby rock and stiff soil sites. A soft soil site, as used in the balance of this Report, is defined as a site underlain by several feet to several tens of feet of young bay mud. Chapter 6 discusses the seismology and ground motion issues of this earthquake in more detail.

It is clear that this earthquake has provided a rich source of data upon which to base improvements in scientific and engineering understanding and to develop more effective approaches to prediction, zoning, engineering, construction, preparedness, response and recovery. There are many questions raised by these data that challenge

Figure 3-2. *Damage to unreinforced masonry buildings was extensive throughout the region, as is common in all damaging California earthquakes. These damaged buildings in the Pacific Garden Mall in Santa Cruz are typical and are virtually indistinguishable from those in photographs from the 1925 Santa Barbara, 1933 Long Beach, 1952 Kern County, 1971 San Fernando, 1984 Coalinga, and 1987 Whittier Narrows earthquakes.*

Figure 3-3. *Many older wood frame houses in the Watsonville and Santa Cruz areas were damaged when the unbraced cripple wall between the foundation and first floor failed. A weak cripple wall is unable to withstand the lateral earthquake forces between the stronger upper structure and the foundations, causing a building to "fall off" its foundation.*

current understanding and practice in the professions of engineering and the earth sciences.

Many reports have been written that characterize the overall impacts of this earthquake on buildings and lifelines [EERI, 1989; EERC, 1989]. Newer buildings generally performed well during and after the earthquake. Unreinforced masonry buildings experienced collapse and severe damage in the epicentral region (Figure 3-2) and in the regions with poor soils, and these resulted in loss of life. Older wood frame houses in the Santa Cruz and Watsonville areas suffered severe damage from collapse of unbraced cripple walls and thousands were made homeless by such failures (Figure 3-3). Older wood frame residential structures in the Marina District of San Francisco sustained major damage due to weak first stories caused by garage openings and due to poor soils (Figure 3-4). These same types of damage observations have been repeated in California earthquake after earthquake.

The Loma Prieta earthquake caused widespread disruption of utility services throughout the region. Water and sewage systems were severely damaged from the epicentral region to the San Francisco Bay area. Although electric power facilities and transmission routes in the earthquake-affected region suffered very little or no damage, initial power outages affected about 1.4 million users. Within 48 hours, service to most customers was restored. There was no reported damage to the major natural gas transmission and large distribution lines. Isolated failures in the local distribution

Figure 3-4. *Many of the buildings that failed in the Marina District of San Francisco were particularly vulnerable because of the lateral weakness of the first story caused by many openings, such as for garages. Many of the most severely damaged buildings in this area were outside the original shoreline on filled areas. These soft soils experienced more severe shaking than adjacent firm soil sites and in some cases liquefied.*

system occurred in the East Bay and in the epicentral regions. The telephone system fared well during the Loma Prieta earthquake, except for one major switching facility that suffered substantial damage. The loss of commercial power caused some temporary communication problems.

One of the major characteristics of the Loma Prieta earthquake was its effect on the transportation systems, mainly from damage to bridges and viaducts. Of the three major commercial airports—San Francisco, Oakland, and San Jose—in the affected area, only San Francisco International Airport closed for a period of time (about 12 hours) following the earthquake. At San Francisco Airport the control tower suffered minor damage; the runways suffered no damage, an air cargo building suffered major damage; and some passenger terminals suffered considerable nonstructural damage, including potentially serious firewater sprinkler failures. At Oakland Airport the northernmost 3,000 feet of the 10,000-foot main runway suffered extensive damage and was closed. The control tower at San Jose International Airport had minor nonstructural damage.

Overview of Damage to Bridges

Only a small percentage of the bridges in the earthquake area sustained any damage at all. Moreover, according to testimony received by the Board, most of the bridges damaged in this earthquake were constructed prior to 1971, before construction standards were stiffened to reflect lessons learned in the 1971 San Fernando earthquake. The greatest damage during the Loma Prieta earthquake occurred to older structures on soft ground.

Seven of the counties proclaimed as State and Federal disaster areas are within the jurisdiction of Caltrans District 4, which suffered nearly all the damage reported for Caltrans structures. These seven counties are: Alameda, Contra Costa, Marin, San Francisco, San Mateo, Santa Clara, and Santa Cruz. Six State bridges were damaged in other counties. Monterey and San Benito counties (District 5) sustained only minor damage to State bridges, while one State bridge in Solano County (District 10) sustained damage which will cost in excess of $100,000 to repair. That bridge, however, was never closed to traffic and did not require any temporary support bracing.

State-wide, Caltrans currently maintains 11,287 highway and pedestrian bridges with spans over 20 feet, a number almost identical to the 11,229 bridges maintained for California local governments in the State. District 4, whose jurisdiction approximates the area of greatest earthquake damage, is responsible for 1,896 State bridges, of which 91 (4.8 percent) incurred some degree of damage (mostly minor) during the earthquake. Structural damage or the potential threat to public safety was sufficiently serious in the case of 13 State bridges that they were closed to traffic for some period of time. Table 3-2 lists the Caltrans bridges that sustained major damage.

Table 3-2. *Caltrans bridges sustaining damage greater than $100,000 during the Loma Prieta earthquake. Conditions are as of May 1, 1990.*

Name of Bridge	Location	Description of Damage
Bridges Closed to Public Traffic after the Earthquake:		
San Francisco-Oakland Bay Bridge (I-80)	San Francisco Bay (Alameda Co.)	Upper and lower closure spans at Pier E9 fell; spans at Pier E23 were near failure; concrete pedestal base of Pier E17 cracked; connection bolts at Piers E-17 throughE-23 damaged; opened for traffic after one month; 1 death and 12 injuries.
Cypress Street Viaduct (I-880)	Oakland (Alameda Co.)	Collapse of 48 bents, causing the upper roadway to collapse onto the lower roadway; 41 deaths and 108 injuries with 1 subsequent death; demolished, reconstruction uncertain.
Struve Slough Bridge (SR 1)	Santa Cruz County	Extensive collapse of the "twin" bridges; opened on January 25, 1990 after reconstruction.
West Grand Avenue Viaduct (I-80)	Port of Oakland (Alameda Co.)	Damage to bents, columns and earthquake restrainers; open to traffic after several days.
Southbound connector over-crossing (I-980)	West Oakland (Alameda Co.)	Damage to two outrigger bents; opened on October 23, 1989.
Mora Drive over-crossing (I-280)	Santa Clara County	Damaged column requiring reconstruction; opened to traffic after a few hours.
Central Freeway Viaduct (US 101)	San Francisco	Damage to bents and columns; retrofit required; portions are still closed
Southern Freeway Viaduct (I-280)	San Francisco	Damage to bents; retrofit required; still closed to traffic.
China Basin Viaduct (I-280)	San Francisco	Damage to bents; retrofit required; opened to traffic after 6 weeks.
Terminal Separation Viaduct (I-480)	San Francisco	Damage to steel span bearings; retrofit required; still partially closed to traffic.
Embarcadero Viaduct (I-480)	San Francisco	Damage to bents and columns; retrofit required; still closed to traffic.
Route 92/101 interchange (US 101)	San Mateo County	Damage to bearings, expansion joints, footings, and columns; opened to traffic after 2 weeks.
San Mateo-Hayward Bridge (SR 92)	Between San Mateo and Alameda Counties	Failure of steel rocker bearings; opened to traffic after a few hours.
Other Bridges Requiring Major Repairs after the Earthquake:		
Temescal Creek (I-80)	Alameda County	Several large cracks in concrete box culvert walls and ceiling.
Distribution structure (I-580)	Alameda County	Damage to bent caps and columns.
Distribution structure (I-580)	Alameda County	Damage to bent caps and columns.
Fifth Avenue over-crossing (I-880)	Alameda County	Damage to columns, bent caps, bearings, and substructure.
Route 242/680 separation (SR 242)	Contra Costa County	Damage to bearing system at Bent 4.
West connector over-crossing (SR 242)	Contra Costa County	Cracks and spalls; damage to bearings and joint seals.
Benicia-Martinez Bridge (I-680)	Contra Costa County	Damage to open deck expansion joints.
Richardson Bay Bridge (US 101)	Marin County	Damage to bearings, caps, columns, and earthquake restrainers.
Pajaro River Bridge (US 101)	Santa Clara County	Anchor bolt and expansion joint damage; cracks and spalls.
Alemany Viaduct (I-280)	San Francisco	Spalling and column damage; retrofit required.
Napa River Bridge (SR 37)	Solano County	Superstructure shifted 4" longitudinally; earthquake restrainers damaged.

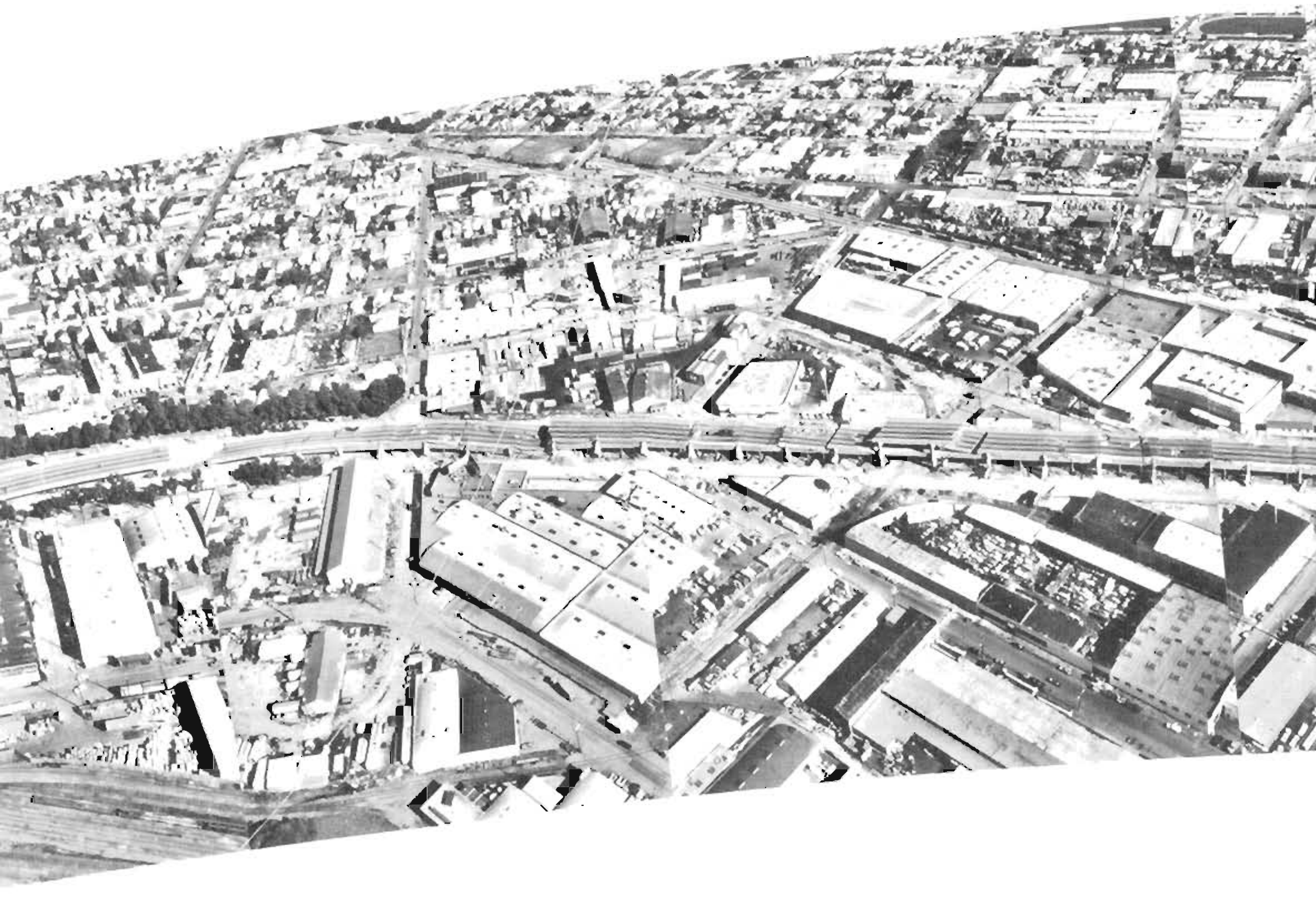

Figure 3-7. *Composite aerial photograph of the Cypress Viaduct showing the extent of the collapse*

Table 3-3. *Comparison of estimated costs of State and local bridge retrofit programs, from Caltrans.*

Retrofit Program	Number of Bridges	Cost (millions)
Local bridge program (SB36X)	1,500	$75-100
State bridge program		
Expansion joints and bearings (1971-1989)	1,261	54
Single column retrofit (1990-1994)	392	143
Remaining State bridges (1991-1994)	700	200-300

The most tragic impact of the earthquake was the life loss caused by the collapse of the Cypress Viaduct, while the most disruption was caused by the closure of the Bay Bridge for a month while it was repaired, leading to costly commute alternatives and probable economic losses. In addition, some of the steel rocker bearings supporting the Navigator Spans of the San Mateo-Hayward Bridge failed. This could have led to catastrophic damage if shaking had been longer or more intense.

On the other hand, the Board received reports of only very minor damage to the Golden Gate Bridge, which is founded on rock, and the BART Trans-bay Tube, which was specially engineered in the early 1960s to withstand earthquakes. A post-quake inspection of the Dumbarton Bridge, built during the 1970s with earthquake design criteria in mind, revealed no structural damage.

Caltrans has announced plans to conduct a complete seismic study of the Bay Bridge and the other Bay crossings. The Golden Gate Bridge District has already initiated a contract for a similar study. The San Francisco Bay Area Rapid Transit (BART) District is reviewing and further evaluating earthquake performance records of the Trans-bay Tube and will improve seismic monitoring, but presently plans no comprehensive studies.

Bridges maintained by local governments also incurred damage, though none as catastrophic as some of the Caltrans structures. A partial survey by Board of Inquiry staff found that at least 43 locally maintained structures in the earthquake area were damaged, of which at least 5 were closed to traffic for some period of time, but none collapsed. Reports from post-earthquake reconnaissance teams indicated that most local bridges performed remarkably well.

Following a Special Session of the State Legislature, Governor Deukmejian signed into law SB36X (Special Session Chapter 18) on November 6, 1989. This bill appropriated $20 million for a program to inspect, evaluate, and, if necessary, retrofit bridges maintained by local governments in the State. The review and evaluation is to be performed by Caltrans, except that the counties of Los Angeles and Santa Clara will review structures within their boundaries. The bill requires that contracts be initiated by December 31, 1991, though pending legislation would extend this date to December 31, 1993 for bridges with multiple-column bents.

Caltrans plans initially to allocate funds to retrofit deficient expansion joints with restrainers and to strengthen single columns in these locally maintained bridges. A subsequent phase will involve retrofit of other local bridges found to be vulnerable. Caltrans estimates that 1,500 local bridges will require retrofit at an estimated cost of $75 to $100 million, with work to be completed by July 1994. Table 3-3 compares the cost of this local retrofit program with the State bridge retrofit program.

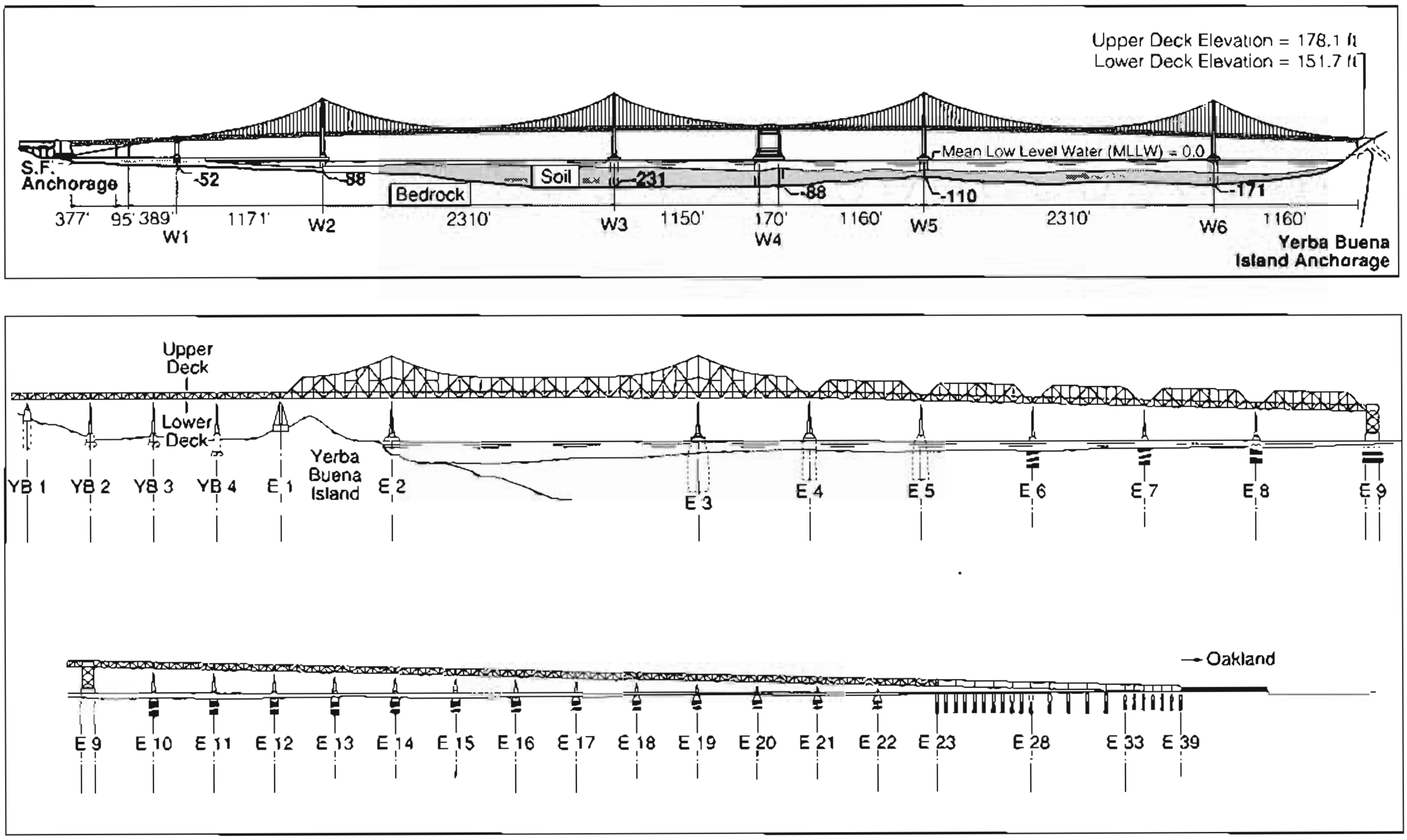

Figure 3-5. *The west and east sections of the San Francisco-Oakland Bay Bridge.*

Damage to the Bay Bridge

Design of the San Francisco-Oakland Bay Bridge was completed in 1933 and construction was finished in 1936. It consists of two sections—a West Bay Crossing from San Francisco to Yerba Buena Island and an East Bay Crossing from Yerba Buena Island to Oakland (Figure 3-5). The total distance along the alignment from the San Francisco anchor of the West Bay Crossing to Pier E39 of the East Bay Crossing is 4.35 miles. The bridge is of double-deck design, with the upper deck carrying five lanes of traffic in the westerly direction and the lower deck carrying five lanes of traffic in the easterly direction. The lower deck had been originally designed for trains.

The West Bay Crossing consists of a twin suspension structure. Its total length is 1.95 miles. Both anchorages and the main supporting piers are founded on rock. The East Bay Crossing consists of four shallow simple-span trusses on Yerba Buena Island, a long cantilever truss structure, five deep simple-span trusses, fourteen shallow simple-span trusses, and a number of simple-span deck systems that use steel and concrete stringers supported on transverse concrete bents. The total length of the East Bay Crossing is 2.14 miles.

The Bay Bridge was designed for 10% g earthquake accelerations, comparable to the levels specified in the 1930 Uniform Building Code for buildings. It should be noted that knowledge of damaging earthquake motions was very limited at this time; the first few measurements of strong ground motions were not made until the 1933 Long Beach earthquake. There had been some seismic strengthening of the Bay Bridge with the installation of cable restrainers in the 1970s

The principal earthquake damage to the bridge was the failure of the upper and lower closure spans at Pier E9. It was closed for one month for repair. These 50-foot-long upper and lower closure spans fell when the bolts failed that connected the pier and the 290' truss to the east (Figure 3-6). Another span at Pier E23, close to the eastern edge of the bridge, was near failure of a comparable type. Connections at Piers E18-E23 also failed. The concrete pedestal bases of Pier E17 cracked when the pier rocked back and forth and incurred some damage at the corners. Chapter 9 gives a detailed description of the Bay Bridge and the damage it sustained.

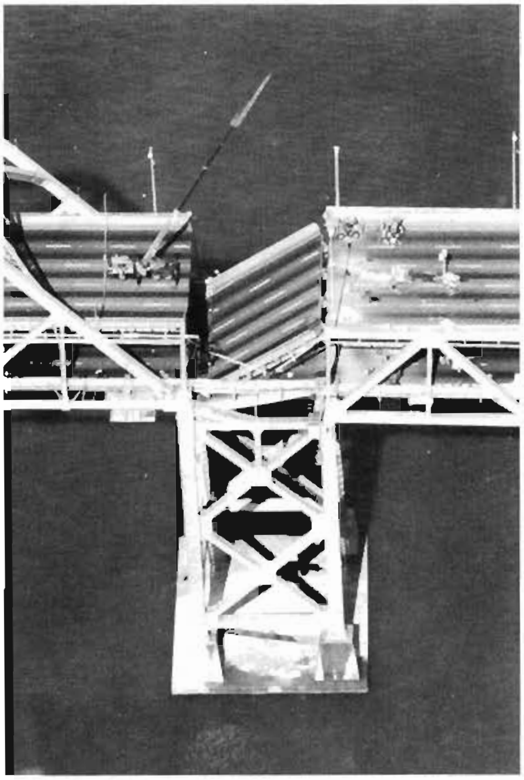

Figure 3-6. *The upper and lower closure spans fell when the bolts attaching the east (right) truss span were severed and the truss span moved to the east, pulling the link spans off their western supports.*

The closure spans linking the two long-span trusses on each side of pier E9 were supported on five inches of bearing on six-inch wide seat-type expansion joints at the west end and bolted connections at the east end. When the truss to the east broke free from its support, the closure spans were pulled with it, and the motions were large enough to slide them off the five-inch seats. As a result, the spans hinged down under gravity load, with the upper span coming down on the lower one. These hit an electrical housing before coming to rest on the west truss connection to the pier.

The approximate five-inch permanent displacement of the eastern span relative to Pier E9 was removed by jacking the trusses back into place. High strength bolts were used to attach the trusses to the pier, and larger seats for the replacement closure spans were installed. The trusses at Piers E18 through E22 were also jacked back into position and connected to the piers with high strength bolts. Replacement restrainers were installed at these locations. The seismic adequacy of these repairs, and of the bridge itself, in the event of a severe earthquake needs to be investigated and any necessary long-term seismic upgrading completed expeditiously.

The level of ground shaking in the Loma Prieta earthquake was smaller in both duration and intensity than that expected in larger and closer earthquakes. Moreover, the duration was not sufficient to excite all the different modes of the Bay Bridge's response that are likely in a longer duration event, nor was the level of shaking sufficiently close to that expected in major earthquakes to test the strength of bridge elements.

The Cypress Viaduct Collapse

The Cypress Viaduct was California's first continuous double-deck freeway structure, a design used again for the San Francisco Freeway Viaducts, but nowhere else in the State. Each deck of the Viaduct carried 4 lanes of traffic. During the Magnitude 7.1 Loma Prieta earthquake, a large portion of the Cypress Viaduct collapsed (Figure 3-7). This collapse was the most tragic consequence of the Loma Prieta earthquake. Forty-one people died. Search and rescue operations continued for a week. Fortunately, traffic conditions were very light compared to normal for the middle of rush hour, probably due to the start of third game of the World Series at Candlestick Park. Caltrans demolished and removed the standing portions of the structure and resurfaced the frontage roads by January. Figure 3-7 (foldout) shows an aerial view of the collapsed portion of the Cypress Viaduct.

Figure 3-8. *View of the failure of the west side of Bent 90 of the Cypress Viaduct*

Chapter 10 discusses the Cypress Viaduct design and damage in detail.

Caltrans began preliminary design of the Cypress Viaduct in 1949 and construction was undertaken between 1954 and 1957, during a period when little was known about seismic design of reinforced concrete structures. It was one of the earliest uses of prestressed concrete in U.S. bridges. More significantly, the Cypress Viaduct was designed before research had developed procedures for achieving ductility in overstressed structural members, and, therefore, the columns and joints failed in a brittle manner when overloaded. The Caltrans seismic design criteria in effect during 1949 to 1954 were introduced in 1943 and stipulated only a seismic strength coefficient of 0.06g.

The Cypress Viaduct was a reinforced concrete structure with some prestressing and two levels of elevated roadway. The box girder roadway was supported by a series of 83 two-story bents. Forty-eight bents collapsed in the October 17th earthquake, numbers 63 through 112, with the exception of Bents 96 and 97, which remained standing (the middle portion of Figure 3-7). A number of the bents had post-tensioned concrete transverse girders at the top level. The Cypress design did not incorporate ductility, since this was not common until the 1970s. Longitudinal cable restrainers were installed in 1977 at all transverse joints in the box girder bridge superstructure to provide continuity. The northerly two-thirds of the Cypress Viaduct, the major portion that collapsed, was founded on about 7' of dense-to-stiff artificial fill overlying a pre-existing triangular-shaped tidal marsh composed of soft bay mud and old slough channels that parallel the west side of the viaduct structure.

Failure occurred in the lower girder-to-

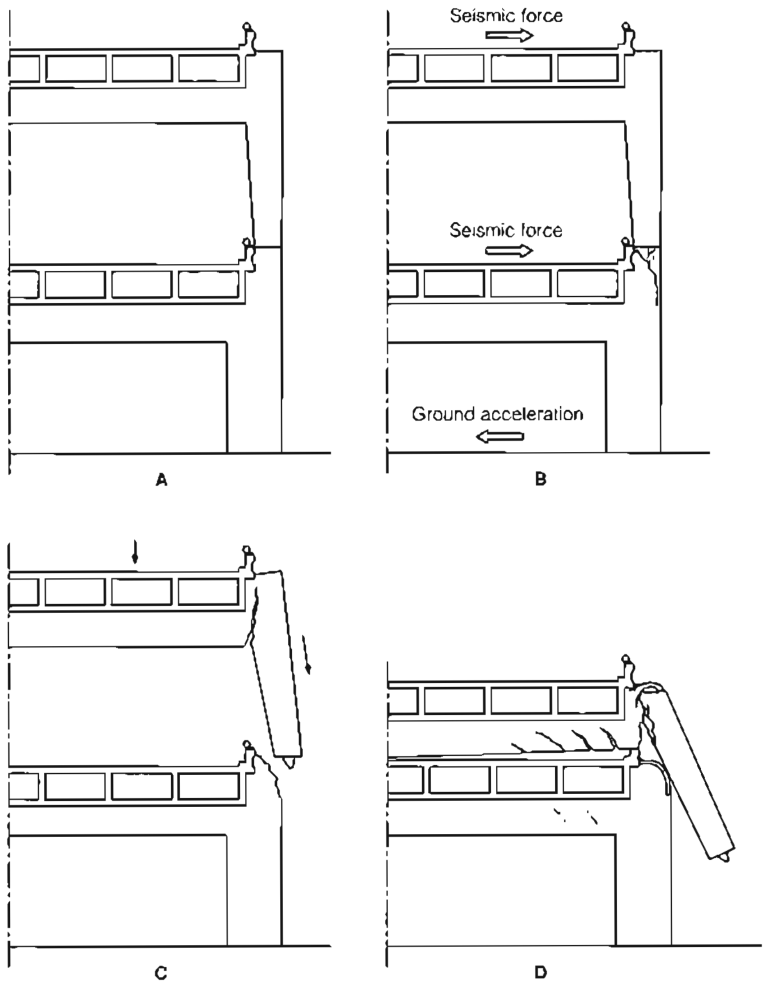

Figure 3-9. *Typical failure sequence of a B1 bent. All but seven of the 48 collapsed bents failed in this way For further discussion see Chapter 10 and Nims et al, 1989.*

column joints on both sides of a bent, with initial failure in the short column pedestals above the top of the lower deck and below the shear key. The failure surface was defined by the plane of the curved negative moment reinforcement bent down into the joint (Figure 3-8). The shear key did not fail in pure shear, as evidenced by the cone of concrete still attached to the four #10 bars that extended through the key into the lower column . The upper girder-to-column joint sometimes failed completely, but in other cases was just severely cracked. Almost all the damage in this upper joint seems to have been produced as a result of the collapse of the upper deck onto the lower deck. The #4 transverse ties in the joint region, as well as those in the column region, either failed or were severely damaged. Examples of this type of bent failure are Bents 63-69, Bents 81-94, and Bents 99-103.

The common failure mode is illustrated by the sequence shown in Figure 3-9. A diagonal crack formed in the pedestals and lower girder-to-column joint regions. This crack followed a plane of weakness in the joint region defined by the plane of the bent-down negative moment girder reinforcement. Then the gravity and seismic forces pushed the upper column down and away from the joint, resulting in the collapse of the upper deck. Much of the damage to the upper columns, roadway, and girders was subsequently caused by the impact of the upper deck on the lower deck. Experiments performed on standing portions of the Cypress Viaduct, as well as static and dynamic analyses, indicate that the calculated seismic demands during the Loma Prieta earthquake required to initiate failure in this nonductile structure were greater than the available structural capacities.

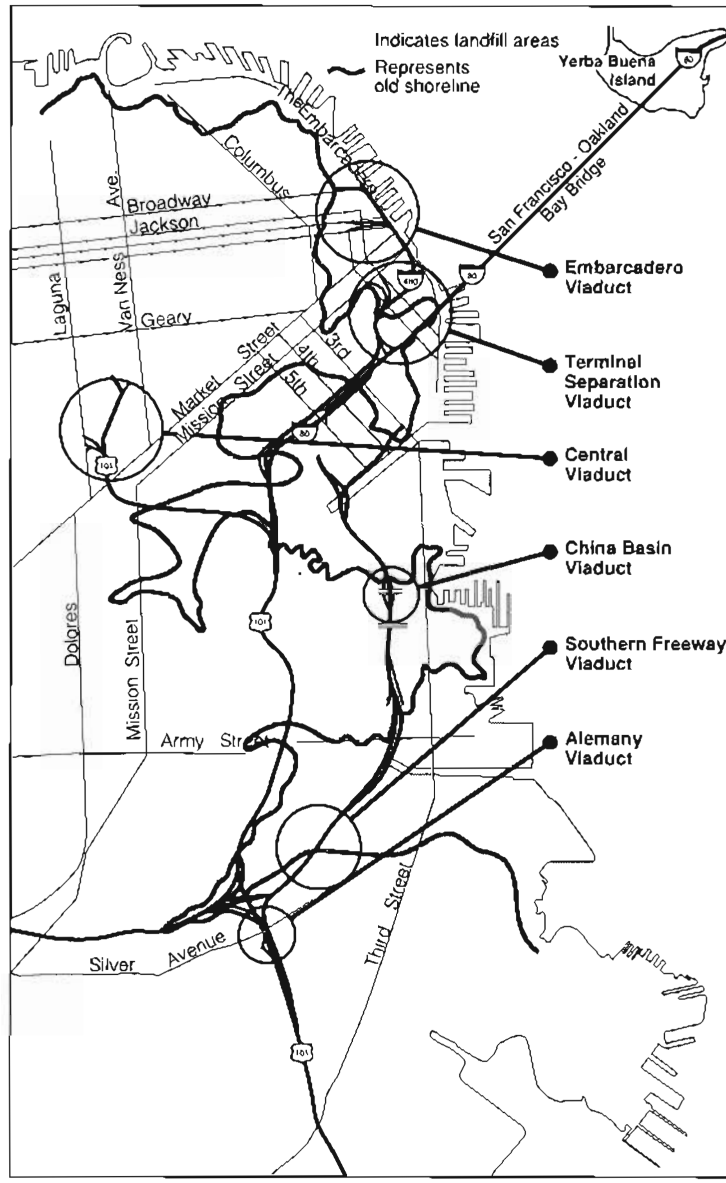

Figure 3-10. *Location of the damaged San Francisco Freeway Viaducts*

San Francisco Freeway Viaducts

The Loma Prieta earthquake was, for the San Francisco Freeway Viaducts, a minor-to-moderate earthquake. These viaducts (Embarcadero Viaduct, Terminal Separation Viaduct, Central Viaduct, China Basin Viaduct, Southern Freeway Viaduct and Alemany Viaduct) in San Francisco (Figure 3-10) were all built with the same technology used for the Cypress Viaduct and are the only structures in the State of this design. All of the freeway structures, with the exception of the Alemany Viaduct, were damaged during the earthquake and subsequently closed to traffic. Damage to the San Francisco Freeway Viaducts is discussed in detail in Chapter 11 and their repair is discussed in Chapter 12.

The San Francisco Freeway Viaducts are composed of single-column and multi-column bents, typically with two tiers, but with a maximum of three tiers of framing supporting two levels of roadway. The transverse lateral load resisting system in the multi-column bents typically consist of pinned-base single-story portal frames with one or more columns cantilevering to the upper level bent cap (girder). The reinforcement in the columns and girders was generally poorly detailed by current standards and reflects the engineering profession's lack of understanding regarding the inelastic response of reinforced concrete members at the time when these structures were de-

Figure 3-11.
Damage to the Central Freeway; typical of damage to the San Francisco Freeway Viaducts. See Chapter 11 for discussion.

signed. In the longitudinal direction, none of the six freeway structures have a planar lateral load resisting system. A lack of redundancy and the inadequate reinforcement detailing are two of the major seismic deficiencies in these freeway structures.

Damage to the individual viaducts varied and included shear cracking in columns, girders, and joints; torsional cracking in outrigger bents; anchorage failure of the girder reinforcement; and shear key failure, among others (Figure 3-11). Many of the crack patterns are similar to those observed in the collapsed and damaged portions of the Cypress Viaduct.

After the earthquake, Caltrans retained six consultants to prepare contract documents for the upgrading of the six designated freeway structures. The criteria set by Caltrans for the development of the upgrading schemes were outlined in a series of letters and memoranda and included requirements for the analysis of the freeway structures and interpretation thereof, and design and detailing requirements. The damage criteria set by Caltrans accepts serious damage, but not collapse, during severe earthquake shaking. Criteria are discussed in Chapter 12.

Figure 3-12. *Punching of severed pile extension columns through the deck of the Struve Slough Bridge.*

Figure 3-13. *The piles supporting the Struve Slough Bridge displaced the ground surface in several directions as much as 18"*

Collapse of the Struve Slough Bridge

The Struve Slough Bridge is located on California Highway 1 between Watsonville and Santa Cruz and consists of side-by-side structures constructed in 1964. One structure carried northbound and the other southbound traffic. These structures are about 800' long and 34' in width with spans of 37'. Typical of structures built at this time, they were of a reinforced concrete T-beam construction supported on pile bents. There were three deck expansion joints in the length of each structure, effectively dividing the length of each structure into four segments. Seismic retrofitting, completed in 1984, consisted of the addition of cable restrainers at each expansion joint.

These structures were supported along their length by 22 pile bents and on monolithic diaphragm abutments at the ends. Each bent consisted of four driven piles, which were approximately 80' long and driven to full length. Each pile was then extended by reinforced concrete columns to the underside of the superstructure into transverse cap beams. The pile extensions were lightly confined with number 3 wire at about 12" pitch.

As a result of the earthquake, these structures experienced extremely strong shaking, which led to the collapse of the center two segments of each structure. Pile extension columns within these center two segments suffered severe cracking, buckling of longitudinal reinforcing, and fracture of the lateral confining reinforcement. Most of the columns sheared off at the interface with the underside of the transverse cap beams. Seven spans of the northbound structure collapsed, dropping approximately 5' onto the damaged pile extension columns. Ten spans of the southbound structure collapsed, dropping approximately 8' to 10' to the ground surface. A few pile extension columns, sheared at the transverse cap beams, displaced and punched through the deck slab as the structure fell to the ground (Figure 3-12). Although the collapse was generally in the downward direction, the southbound structure displaced transversely approximately 2'.

Some piles also apparently failed below the ground surface. Soil displacements at the ground surface around the piles in several directions were found, up to a maximum of 18" at several piles (Figure 3-13). The approach fills settled approximately 3".

The structure showed little evidence of significant seismic forces reaching the superstructure above the abutments. There was no indication of horizontal movement at the abutments or hammering at the expansion joints. Despite the displacements experienced by the collapsed structures, the cable restrainers performed well and held the structure together.

The primary cause of collapse is attributed to a lack of adequate concrete confinement and shear reinforcing at the top of the columns that caused a very weak connection to the deck. This was a design deficiency by standards that were current when the bridge was designed. Current practice and standards would require ductile detailing of these members, which would have led to substantially better seismic performance.

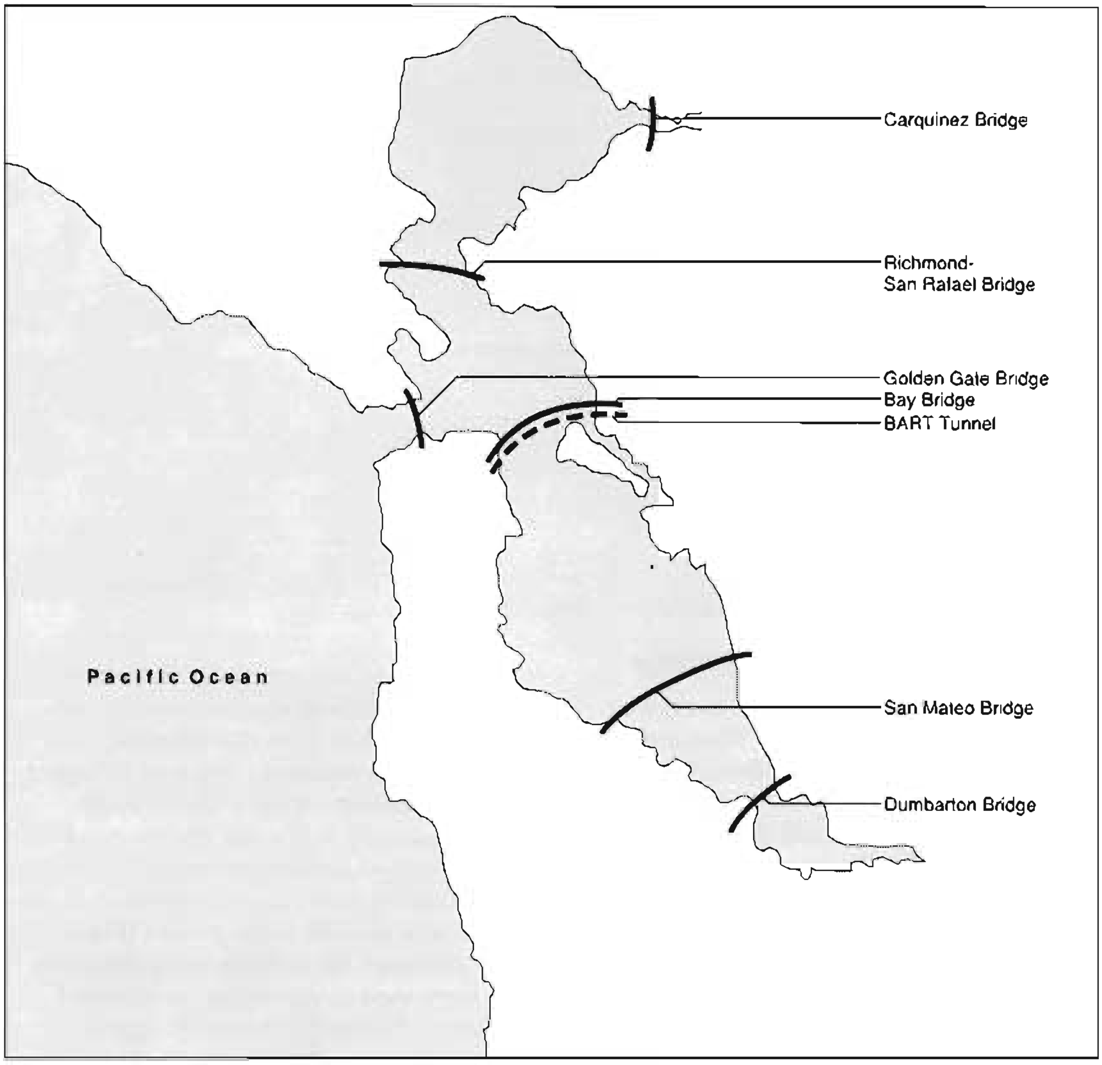

Figure. 3-14. *The San Francisco Bay transportation arteries*

Table 3-4. *Agencies responsible for cross-bay transportation arteries*

Trans-Bay Crossing	Responsible Agency
San Francisco-Oakland Bay Bridge	Caltrans
Golden Gate Bridge	Golden Gate Bridge District
BART Trans-bay Tube	BART
Richmond-San Rafael Bridge	Caltrans
Carquinez Bridge	Caltrans
Antioch Bridge	Caltrans
Benicia-Martinez Bridge	Caltrans
San Mateo-Hayward Bridge	Caltrans
Dumbarton Bridge	Caltrans

Impacts on Cross-Bay Transportation

San Francisco Bay stretches some 65 miles from Alviso in the south to its northern boundary at the Richmond-San Rafael bridge in the north. Beyond this bridge the stretch of water becomes San Pablo Bay, which ends at the Carquinez Bridge near Vallejo. Besides the Richmond-San Rafael Bridge, San Francisco Bay is crossed by three other bridges: the San Francisco-Oakland Bay Bridge, the San Mateo Bridge, and the Dumbarton Bridge. The Bay exits to the Pacific Ocean under a fifth bridge — the Golden Gate. In addition the bay is traversed by the Bay Area Rapid Transit (BART) Trans-bay Tube between Oakland and San Francisco (Figure 3-14). The Carquinez, Antioch, and Benicia-Martinez Bridges cross San Pablo Bay. All but two of these nine arteries that span San Francisco Bay are the responsibilities of Caltrans (Table 3-4). The critical connections between San Francisco and its eastern and northern neighbors are those of the Golden Gate and Bay Bridges, and the BART Trans-bay Tube.

The importance of integrating the Bay Area by bridging the water barrier between San Francisco and its neighbors was recognized early in the century. In 1912 the engineer John R. Freeman predicted that the population of the five Bay counties would reach 2 million within forty years, and that towards the end of the century a population of 3 million might be reached depending upon "the wisdom and vigor with which San Franciscans seize their opportunity" [Scott, 1985]. He was very close to correct in his first estimate, but by 1980 the five bordering San Francisco Bay counties had a population of 4.29 million, which is projected to rise to 5.24 million by the year 2005 [Diridon, 1988].

The Bay Bridge opened in November 1936, six months ahead of schedule. The lower deck was not yet equipped for trains, however, and construction of the Trans-bay Terminal Building did not begin until the summer of 1937. In 1958 the lower deck was modified to be used by automobile traffic. Meanwhile, a majority of commuters continued to use the ferries, although automobile patronage steadily rose, leaving no doubt that this was to become the transportation method of choice.

During the period following World War II, the explosive population rise in the Bay Area and housing policies that encouraged the development of housing subdivisions with single-family dwellings on large lots resulted in urban and suburban sprawl. The public came to depend almost entirely on the private automobile. By the late 1960s only 6% of all trips in the Bay Area used public transportation.

In 1951 the California Legislature created a special commission to study Bay Area transportation problems. In 1957 the nine-county Metropolitan Transportation Commission recommended the creation of a five-county Bay Area Rapid Transit District. In 1962 a three-county (San Francisco, Alameda and Contra Costa) rapid transit plan was adopted by the Board of Directors and in the same year a $792 million bond issue was approved for construction of the system. Construction began in June, 1964; by August, 1969 the Trans-bay Tube structure was complete. BART passenger service began on a limited basis in September 1972, and in September 1974 service began on the Trans-bay Tube. The system today encompasses 71.5 miles of track, of which approximately 20 miles are in subway, 24 miles elevated, and 27 miles at grade. BART is a heavy-rail transit system that operates on electric power. The Trans-bay Tube, which runs from Oakland to San Francisco along a route close to that of the Bay Bridge, is a very important addition to the Bay crossings.

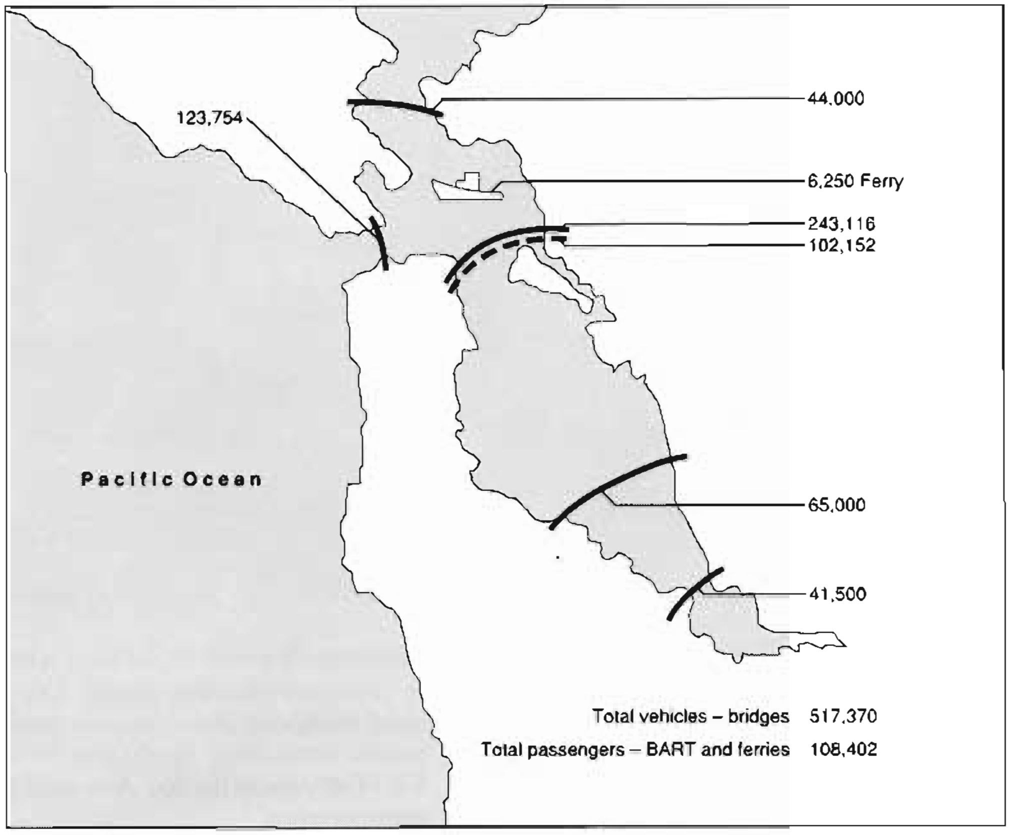

Figure 3-15. *Total daily vehicles and passengers, 2-way, before the earthquake*

Importance of the Bay Crossings

The San Francisco Bay crossings form a crucial part of the entire Bay Area transportation network. In the 9 Bay Area counties, about 20% of workers are employed outside their county of residence. By the year 2005 it is estimated that this percentage will grow to slightly under 25%. At the same time, automobile ownership in the area is expected to rise from today's 3.3 million autos to 5.1 million in the year 2005 [Diridon, 1988]. Hence the Bay Area highway network will continue to be crucial to the economy, but its maintenance as a swift and convenient means of travel will become increasingly difficult.

Before the Loma Prieta earthquake, the total number of vehicles on the five San Francisco Bay bridges was an average of 517,000 per weekday, together with an average of 108,000 passengers on the BART Trans-bay Tube and the ferries. The breakdown of this traffic is given in Table 3-5 and illustrated graphically in Figure 3-15. Two facts stand out: the importance of the Oakland-San Francisco link, and the volume of traffic borne by the San Francisco-Oakland Bay Bridge—approximately double that of the Golden Gate Bridge, and almost equal to the combined traffic carried by all four other bridges. For automobile traffic, the Golden Gate and Bay bridges are essentially nonredundant systems, with alternative routes via the other bridges being time consuming to a level that seriously impacts commercial and institutional productivity. In contrast to freeways, which are superimposed over an existing (if inadequate) road pattern that is still available if a section of freeway is knocked out, the bridges have no satisfactory alternative.

When the Loma Prieta earthquake damaged the Bay Bridge, causing immediate closure of the most widely used cross-bay route for an indeterminate period, Bay Area traffic patterns were forced to change, and rapid planning had to be done to accommodate the situation. During the period of closure of the Bay Bridge, traffic was redistributed, as detailed in Table 3-5 and illustrated in Figure 3-16. Part of the apparent change in cross-bay totals owes to the fact that BART and the ferries count individual fares, while the bridges count vehicles.

The critical role played by the BART Trans-bay Tube in cross-bay transportation is clear, as is the fact that the South Bay

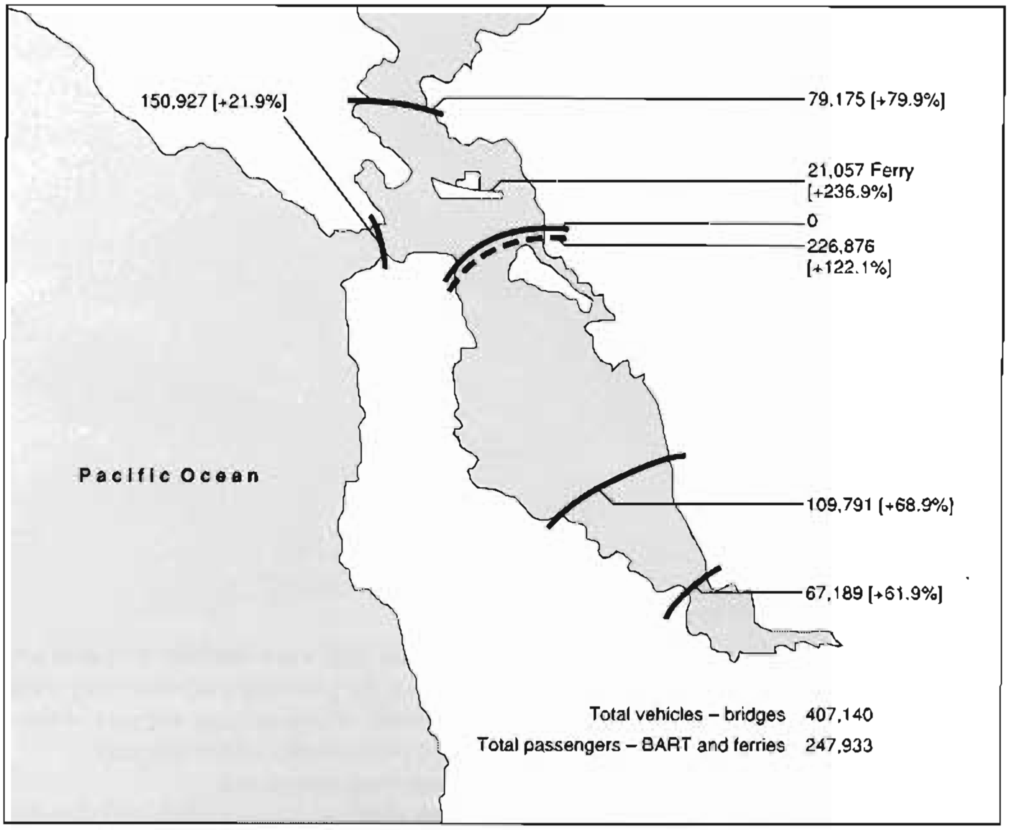

Figure 3-16. *Total daily vehicles and passengers, 2-way, after the earthquake, but before reopening the Bay Bridge.*

Table 3-5. *Traffic use of the San Francisco Bay transportation links before and after the Loma Prieta earthquake.*

Total Vehicles two-way, weekdays:	Before Earthquake	After Earthquake	Difference
San Rafael Bridge	44,000	79,173	+79.9%
Golden Gate	123,754	150,927	+21.9%
Oakland-Bay	243,116	0	-100.0%
San Mateo	65,000	109,791	+68.9%
Dumbarton	41,500	67,189	+61.9%
BART tube	102,152	226,876	+122.1%
All ferries	6,250	21,057	+236.9%

Source: Caltrans, Post-Earthquake Commute Summary-Daily Trips, December 19, 1989.

bridges (San Mateo and Dumbarton) accommodated most of the redistribution of vehicular traffic. While emergency ferry service more than tripled, from 6,250 to 21,057 passengers, the number of people carried was still well below that of automobiles and the BART system. Total vehicles over the crossings dropped from 517,370 to 407,140 during the closure period; a lot of people used BART or curtailed their travel. The effect of this pattern was noticeable in San Francisco—for example, restaurants in the city were virtually empty during this period, and suffered severe economic losses.

That the economic and personal losses of the Bay Bridge closure were considerable is not in doubt, although much detailed study would be necessary to define them fully. If the Bay Bridge had not opened within a month—a much shorter time interval than initially projected immediately after the collapse—the operational and economic consequences would have been much more severe. The equipment and maintenance facilities of BART, for example, were severely strained, and the post-earthquake increase in traffic could not have been sustained indefinitely. If the BART Transbay Tube or the Golden Gate Bridge had simultaneously been closed for a comparable time, the economic and social consequences could have been catastrophic.

This (fortunately) short closure of the Bay Bridge gave some indication of the disruption that could be caused by the loss of one of the essential cross-bay links. The Loma Prieta experience can be seen as a "live" exercise demonstrating the short-term closure of a single cross-bay link. A previous accident—the Trans-bay Tube fire in April, 1979—had resulted in a BART tube closure for two and one-half months. This experience also showed that failure of a single trans-bay crossing can be accommodated, albeit with some loss of convenience.

Contingency Earthquake Planning for the Bay

The consequences of earthquake damage to the Bay Area highway system (and a number of other urban "lifelines") were considered in two earthquake planning scenarios developed and published by the California Department of Mines and Geology for the Office of Emergency Services. The first, published in 1982, sketched the possible effects of a Magnitude 8.3 earthquake on the San Andreas fault [CDMG, 1982]. The second, published in 1987, considered the effects of a Magnitude 7.5 earthquake on the Hayward fault [CDMG, 1987]. Both these studies were intended not as predictions but as carefully considered pictures of what might happen if the scenario earthquakes occurred, with the primary objective of making emergency response and recovery authorities aware of the types and degree of situations that they might have to face. While they were carefully prepared and appropriate for planning purposes, they were not the result of engineering analyses of the expected performance of the physical structures they considered.

Both these scenarios anticipated that the bridges would withstand the shaking, but the approaches were deemed vulnerable to landslides and soil failures, leading to closure. For the San Andreas event the Golden Gate Bridge closure was estimated at 24 hours, and for the Bay Bridge, closure was estimated for "over 72 hours." For the Bay Bridge, the scenario states "total collapse can be discounted." There was no engineering basis for any of these assessments of the seismic performance of the bridges, but the assessment of the closure of the approaches to the Bay and Golden Gate Bridges was based on the nature of the soils there, and indeed the approaches at the east tollgate of the Bay Bridge were damaged and closed for repair following the Loma Prieta earthquake.

The Hayward fault event was seen as closing the Bay Bridge for 72 hours, but "an all-out effort could possibly open the bridge to limited traffic in about 36 hours. The bridge is available to emergency traffic between San Francisco and Yerba Buena." This scenario earthquake was not expected to affect the Golden Gate Bridge.

The San Andreas earthquake was expected to close the BART Trans-bay Tube, although the Tube was not expected to rupture; again, this was not based on any engineering evaluation of the expected performance of the Tube itself. It was thought highly probable, however, that at least a few of BART's elevated spans would fall somewhere along the system and result in closure of the system. Damage from the Hayward fault event was anticipated to be similar. The Trans-bay Tube would not rupture, but would be without power and would shut down. For scenario purposes, four elevated spans were expected to fail.

In many respects the scenarios quite accurately anticipated some of the experience of the Loma Prieta earthquake, particularly with respect to ground failures. However, the collapse of the Bay Bridge span was not anticipated, and the degree of damage probably would not have been estimated to be as high if a scenario had been prepared for a Magnitude 7.1 earthquake some 60 miles distant on the San Andreas.

The Loma Prieta earthquake showed the vulnerability of a Bay Crossing and other critical links in the freeway system to a relatively distant event. Future planning must recognize the likelihood and potential consequences of closer, and more powerful events on the San Andreas and Hayward faults. In particular, the possibility of a dual, or even triple failure, of Bay crossings would result in a situation for which there has been no precedent.

Moreover, even if additional retrofits are implemented for the Bay Bridge, and the Trans-bay Tube and the Golden Gate Bridge are deemed safe, the possibility of post-earthquake closure is always present, because no engineering measures can guarantee a damage-free response.

Conclusions

The Loma Prieta experience emphasized that the Bay transportation network must be maintained as a flexible and integrated system; it was the inherent flexibility of the system that enabled the redistribution of traffic among a number of complementary carriers after the earthquake.

The crucial importance of protecting the three key Bay crossings, and, by direct inference, other key structures throughout the State that perform important roles in transportation systems is clear. While protection of life obviously is the first concern, the devastating economic consequences of prolonged closure of one or more of the crossings can now be visualized clearly. To mitigate these possible effects, several steps must be taken:

- Engineering studies should be instigated of the Golden Gate and San Francisco-Oakland Bay Bridges, of the BART system, and of other important transportation structures throughout the State that are sufficiently detailed to reveal any possible weak links in their seismic resisting systems that could result in collapse or prolonged closure.
- Seismic standards for the design, construction, and retrofitting of important transportation structures must be selected so that the likelihood of their being out of service for an extended period of time is very low.

In view of their age, attention should focus initially on the Golden Gate and Bay Bridges: studies should be both analytical and experimental. Studies should be specifically related to future probable earthquakes exceeding Loma Prieta in magnitude and duration of ground shaking.

- Any retrofits deemed necessary as a result of analysis should be rapidly funded and implemented using methods that, as far as possible, do not impede normal traffic flow.
- A major contingency planning effort should be undertaken to develop alternative routes and procedures considering a number of alternative post-earthquake closure patterns.

The intent should be to develop readily available emergency plans and procedures that, as far as possible, anticipate a reasonably predictable set of events, such as the closure of two or more Bay crossings. The planning should also recognize the need to improvise to deal with unexpected events. This planning should be conducted on a sound theoretical basis in order to provide input to operational management and staff that must prepare the response plans.

The Bay crossings represent one component—perhaps that with the least redundancy—of the entire Bay Area highway transportation system. This, in turn, forms part of a State-wide, indeed national, system—rail, air, ship—that is critical for the economic transportation of people, materials, and products. The transportation network itself forms one subsystem of the entire urban and regional structure of the State. In terms of future earthquakes it is necessary not only to identify and strengthen the weak links in bridges, but to protect the buildings and urban utility systems that enable our society to function.

The Loma Prieta earthquake gave other indications of earthquake effects on our physical, social, economic and political structures that must be attended to. While newer, well engineered, buildings fared well, much of our business, commercial and institutional activity still takes place in older buildings that were constructed before seismic design and construction was well advanced. Only slowly, at the rate of about 2% a year, is our older building inventory being upgraded and/or replaced.

Many of our older governmental buildings—often of great architectural merit, such as the San Francisco and Oakland City Halls—suffered considerable structural and nonstructural damage. Six months after the earthquake, Oakland City Hall is still unusable, and San Francisco is evaluating alternative schemes for extensive repairs to its administrative center and other important institutional buildings. The damage to these buildings, just below the threshold of life-threatening structural failures and collapses, must be understood as far less than that which would be incurred during future high probability earthquakes of the same or greater magnitude on closer sections of the San Andreas or Hayward faults.

Only aggressive programs of identification and strengthening will prevent the possibility of perhaps scores of tragic collapses similar to that of the Cypress Viaduct in the event of a future closer earthquake of long duration. In addition, the economic and administrative effects would be deep and long-enduring, and might threaten the viability of California as an attractive environment in which to live and work.

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Chapter 4

Findings

Central to the Board's process has been the determination of what occurred during the earthquake and why. These findings form the basis for the recommendations made by the Board of Inquiry in Chapter 5. A great deal of information has been considered, some provided through public presentations to the Board, other through study of documents and files. Many individuals and organizations made presentations to the Board; those who appeared are listed in the Appendix. The technical materials, historical documents, design drawings and other materials made available to the Board are listed in the Annotated Bibliography. The Board of Inquiry has reached a number of conclusions based on the materials available to it. The fifty-two specific findings of this chapter are organized in the following general areas:

1. Findings on seismology and ground motion.
2. General findings on transportation structures.
3. Findings on Caltrans seismic design practices.
4. Findings on the Bay Bridge span failure.
5. Findings on the Cypress Viaduct collapse.
6. Findings on San Francisco Freeway Viaducts.
7. Findings on the Caltrans retrofit program.
8. Findings for other types of structures.

A discussion follows the statement of each finding that gives the rationale for the finding. Discussions are necessarily brief and they should be understood within the context of the technical materials presented in the balance of this Report.

As a prologue to these findings, the Board would like to express its appreciation to all those who directly and indirectly supported its efforts. The Board received the full cooperation and support of all individuals and institutions appearing before it. Individuals and organizations from State and local government, universities, professional associations and the public unselfishly contributed their time and expertise to the deliberations of the Board. Without exception, the individuals that appeared before the Board were thoughtful, gave the impression of being competent and knowledgeable, and were forthcoming. Officials and staff from all State agencies appearing before the Board were very cooperative in assisting the Board in its inquiry, including: Department of Transportation; Department of Conservation, Division of Mines and Geology; Highway Patrol; State Board of Registration for Professional Engineers; Department of Water Resources, Division of Dam Safety; Department of Finance; and, Seismic Safety Commission. The substantial effort of Caltrans and its staff is particularly noted and appreciated.

Findings on Seismology and Ground Motion

1 **The Loma Prieta earthquake was a Magnitude 7.1 earthquake with epicenter in the Santa Cruz Mountains over 60 miles from San Francisco and Oakland, 20 miles from San Jose, and 10 miles from Santa Cruz. The epicenter and region of strongest ground shaking was in a sparsely populated, mountainous area. Its ground motions were far from the most severe that can be expected as seismic dynamic loadings for bridges, either in the Bay Area or the State.**

The ground motions at the Cypress Viaduct and Bay Bridge were lower in intensity and duration during the Loma Prieta earthquake than those that can be expected for a Magnitude 8.3 on the San Andreas fault or a Magnitude 7+ on the Hayward fault, repetitions of earthquakes that have occurred historically. A report by the Working Group on California Earthquake Probabilities assesses the likelihood of a Magnitude 7 earthquake in the Bay Area as 50% between 1988 and 2018. This estimate represents a composite of 30-year probability estimates for Magnitude 7 earthquakes on the northern and southern segments of the Hayward fault, each assigned a 20% probability; for a Magnitude 7 on the Peninsula segment of the San Andreas, 20%; and less than 10% for a repeat of the 1906 Magnitude 8.3 on the San Andreas in the next 30 years.

2 **The Loma Prieta earthquake was anticipated by the Working Group on California Earthquake Probabilities, but not with high enough probability of occurrence (30% in the next 30 years) or confidence in the forecast (low) to have caused Caltrans or others to respond directly to this forecast.**

The 1988 Report of the Working Group on California Earthquake Probabilities assessed the likelihood of major earthquakes on the San Andreas and tributary fault systems. The Group was appointed by the U.S. Geological Survey and included many California scientists. These studies concluded that the San Andreas fault in the vicinity of Loma Prieta had an elevated potential for an earthquake of Magnitude 6.5-7.0. They concluded that this earthquake was the most likely one among those studied in the San Francisco Bay Area, at 30% probability of occurrence in the 30-year period to 2018, but they also concluded that they were not very confident in the estimate, giving it their lowest confidence rating. In the context of long-term probabilistic forecasts, the Loma Prieta earthquake was anticipated. The Working Group also assessed that there was a 20% probability of Magnitude 7 earthquakes on the northern and southern segments of the Hayward fault and on the peninsula segment of the San

Andreas fault. A repeat of the 1906 Magnitude 8+ earthquake was assessed as having a probability of less than 10% in the same period. These other earthquakes were forecast with moderate confidence.

3 The duration of the strong phase of ground shaking generated by the Loma Prieta earthquake was unusually short for an earthquake of Magnitude 7.1.

The duration of the strong phase of ground shaking may have been as short as one half of that expected for an earthquake of this magnitude. The fault rupture appears to have begun in the middle of the zone, which ruptured and progressed in both directions rather than the more normal process in which the rupture progresses in one direction only from one end of the zone to the other. Had the duration of the shaking been twice as long, the damaging effects undoubtedly would have been more severe. The intensity of ground shaking, as portrayed by peak acceleration measurements, showed unusual azimuthal and distance variations. These are probably reflections of the complex effects of the seismic source, geologic characteristics of the path outward from the source, and the effects of local geologic and site soils conditions. These spatial variations demonstrate the uncertainties of seismic hazard assessment.

4 Soft ground on the border of the San Francisco Bay amplified ground motions more than anticipated by current codes.

Damage observed in many areas was concentrated at soft ground sites. A soft soil (or soft ground) site, as used in the balance of this Report, is defined as a site underlain by several feet to several tens of feet of young bay mud. Damage in the Marina district, the most heavily damaged area of the City of San Francisco, the South of Market area, the Oakland Airport and waterfront, and the Cypress Viaduct were all located on sites with soft soils. Generally these were sites with landfill over bay muds, many completed some time ago without specific engineering consideration. No strong-motion records were obtained in the immediate vicinity of the Cypress Viaduct. Analysis of strong motion records from other sites, at comparable distances from the epicenter, indicate that the average levels of shaking (as evidenced by peak acceleration, peak velocity and/or 5% damped velocity response spectra) were of the order of 2 to 4 times greater in Oakland relative to a hard rock site on Yerba Buena Island across a range of frequencies of engineering importance. For selected narrow frequency bands, recordings of both the main earthquake and aftershocks reveal that ground motion amplification at some soft sites was eight to ten times that recorded at rock sites. The interaction of ground motion with the highly non-linear response process in structures may moderate the effect of these differences, particularly for higher ground motion levels. Lacking strong motion data from foundation sites at the Cypress Viaduct, and pending further studies, no final conclusion can be drawn about the detailed amplification characteristics of the soil deposits there, or possible resonance of motions at the individual foundation sites with specific parts of the structure at this time.

5 Soil liquefaction was observed in the region, but there is no evidence that it contributed to the Cypress Viaduct or Bay Bridge span failures.

There was no evidence presented to the Board that liquefaction occurred in the foundations of these structures; however, only limited field investigations were made. Liquefaction may have contributed to the performance of the Embarcadero Viaduct and other freeway viaducts, but through altering ground motions, not failure of bearing soils. Liquefaction, the transformation of loose saturated sandy materials into a fluid-like condition, was observed at many locations during the Loma Prieta earthquake, and contributed to the damage of structures at a number of locations. Particularly hard hit in the San Francisco Bay Area were developments on man-made artificial sandy fills in San Francisco and Oakland. These include the Marina District and the areas south of Market Street in San Francisco, the Oakland International Airport and Port facility, the Alameda Naval Air Station, Treasure Island, and the Bay Bridge approach at the east end. Elsewhere damage caused by liquefaction was concentrated principally in natural sediments. Damage from liquefaction results primarily from large horizontal and vertical displacements of the ground. These displacements occur because sand/water mixtures in a liquefied condition have virtually no shear strength and provide little or no resistance to compaction, lateral spreading, or downslope movement. Thus, ground of even the gentlest of slopes can move toward free surfaces such as shorelines, bluffs, river banks, and man-made cuts. In addition to the downslope displacements, the ground above the liquefied sediments commonly breaks into small blocks, which may tilt and cause vertical displacements between adjacent blocks. This permanent movement of the land surface can be devastating to surface structures, such as buildings, and to buried utilities, such as gas mains, water lines and sewers.

General Findings on Transportation Structures

6 **In addition to the tragic loss of life, the economic and social consequences of the Cypress Viaduct collapse and the Bay Bridge span collapse outweighed the costs of the damage itself.**

The damage caused to the Bay Bridge by the failed span at Pier E9 was modest. The cost to the community of losing the principal transportation artery between the San Francisco peninsula and the East Bay and the principal connector of I-80 north and I-880 south of the Bay Bridge (the Cypress Viaduct) was substantial, e.g., lost work time, lost productivity, increased traffic congestion, and increased commute time, possibly for years until the Cypress Viaduct is replaced. It is hard to comprehend the costs that would have been incurred by the community if both the BART Trans-bay Tube and the Bay Bridge had been out of service for an extended period.

7 **The available knowledge, seismic codes, and standards of practice of earthquake engineering have changed substantially from that of the periods when the Cypress Viaduct (early 1950s) and San Francisco-Oakland Bay Bridge (early 1930s) were designed and constructed.**

The methods of design and analysis of reinforced concrete and steel structures, the prescription of site ground motions, and the understanding of dynamic response and the capacities of structural elements and materials has changed considerably as a consequence of new information and observations of structural behavior in earthquakes. The approach to seismic design has changed from one of strength to one of strength and ductile capacity. The first introduction of ductile design of concrete frames for buildings was in the mid-1960s. After the 1971 San Fernando earthquake and a decade's limited research results, ductile design of reinforced concrete frames became common and accepted by the engineering profession in the mid 1970s. The main feature of ductile design is that a structure is designed so that it retains its ability to support loads, both gravity and seismic, after over-straining has occurred for repeated cyclic seismic deformations. Ductility is accomplished by the appropriate placement of steel reinforcing bars and ties, for example, to provide confinement of the concrete. Prior to 1971, Caltrans had not incorporated ductile design into its bridge design standards. After the demonstrations of the 1971 San Fernando earthquake, Caltrans changed reinforcing details for designs and work in progress and ductile detailing procedures were adopted.

8 Historically, the fiscal environment at Caltrans has inhibited giving the level of attention to seismic problems they require.

Many items, ranging from research on earthquake engineering to seismic retrofitting, were placed in low priority because of the limited resources. The history of Caltrans's seismic engineering group is illustrative of this environment. Following the 1971 San Fernando earthquake Caltrans established a seismic engineering group; by the middle of the decade it had been greatly reduced as Caltrans was forced to cut costs in many programs. Research on earthquake performance of bridges has been limited to occasional projects, generally those that supported specific needs. Until the last few years, few bridges have been instrumented to provide the basis for a satisfactory understanding of seismic response. This environment has improved greatly following the Loma Prieta earthquake. Attention, funds and personnel resources are being devoted to earthquake safety issues in amounts that are unprecedented. It is not clear that these long overdue improvements in support for earthquake engineering and seismic safety issues will continue once the immediate response to the earthquake has been completed and old restraints reassert themselves.

9 The Board finds that Caltrans has the reputation of being the best transportation agency among the States and a leader in bridge design.

Federal Highway Administration (FHWA) staff indicate that Caltrans is very well regarded for its quality of work, and has been so regarded for a long time. From the earliest time, AASHTO seismic design criteria followed the practices and recommendations of Caltrans. Caltrans has been and is active in the standards development process, and has staff participating in the projects of AASHTO, FHWA and Applied Technology Council (ATC).

10 No comprehensive analyses of the expected seismic performance of major transportation structures (e.g., Bay Bridge, Golden Gate Bridge, BART Trans-bay Tube) have been completed since their design by the authorities responsible for them.

The design of the Bay and Golden Gate Bridges in the early 1930s and BART in the late 1960s employed the state-of-the-art in earthquake engineering at the time of their design. However, in the intervening years much has been learned both through research and observations of structural performance in earthquakes. The changes in our understanding have touched every aspect of seismic design and structural performance—from specification of the ground motion and frequency characteristics at a site to the energy-absorbing characteristics of materials under cyclical loadings. Many practices previously thought to be conservative are now thought to be questionable. Engineers now have the ability to analyze large, complex systems using computers. The Bay Bridge was designed with slide rules, not computers. These structures are complex, with many seismic performance characteristics depending on interactions among elements that are not intuitive. This observation goes farther than these three structures. There are many bridges and transportation structures, particularly major interchanges, whose loss in an earthquake would have economic and social impacts far exceeding the cost of the damage itself. These warrant careful seismic safety scrutiny. Caltrans has begun this process, and has completed limited reviews of some structures, particularly interchanges.

11 Current federal criteria, when used for California transportation projects, may not be sufficiently conservative and inclusive of seismic concerns to meet the seismic safety needs of the State of California.

The Federal Highway Administration (FHWA) requires that the AASHTO Standard Specifications must be followed for projects to be constructed or rehabilitated with federal highway fund participation. Until recently, the 1983 AASHTO seismic requirements were optional for projects with federal participation, now they are mandatory. Federal criteria only supersede State specifications where the State specifications are less stringent than the federal requirements. Since Caltrans's seismic criteria have led AASHTO's, in effect Caltrans criteria are the ones enforced within California. For other transportation related projects where the State does not have criteria or is not involved, federal criteria, where they exist, will prevail for projects involving federal funding. Federal criteria are drawn to meet the needs of all the states, only a portion of which are subject to earthquake hazards as severe as those of California. The seriousness of California's earthquake hazards and its need for continued functioning of transportation systems pose special problems. Criteria drawn to respond to the national perception of the earthquake problem are likely to include compromises to accommodate many different perceptions of need.

12 There was no seismic instrumentation on the Bay Bridge, Cypress Viaduct, San Francisco Freeway Viaducts, or BART Trans-bay Tube. Only a few transportation structures were instrumented. This severely limits information on what the ground motions were and how the structures responded to these motions.

The lack of ground and structural response recordings limits engineering analysis of structural performance and therefore the ability to draw conclusions about the performance of these bridges. Ground motions during the earthquake at the Bay Bridge and the Cypress Viaduct have been estimated using recordings well away from the sites themselves. Caltrans has supported a limited program for the past few years by supporting the California Strong Motion Instrumentation Program (CSMIP). Support for instrumentation of transportation structures is at the same rate of funding (0.07% of equivalent permit value) as provided by the State fee on building permits. Coupled with federal funds, this has enabled a few structures in the Bay Area to be instrumented, including the Dumbarton Bridge, an overpass in San Jose, a BART Viaduct, and the Sierra Point Overpass. Because this program was only started recently, few bridges in California have instruments to record their response. When records are available, engineers can develop an understanding of actual performance and develop methods to anticipate response during more severe ground motion. The development of seismic design principles for buildings owes much to recordings of earthquake response of buildings, resulting in substantial improvements in design. Comparable advancements for bridge design will not be possible until more bridges have been instrumented and their performance observed.

Findings on Caltrans Seismic Design Practices

13 Caltrans does not have a management-directed seismic safety performance goal that must be met by all its structures.

The requirements of accommodating earthquakes are contained in design documents for use by engineers on individual projects, rather than as policy guidance requirements from the management of Caltrans or from the Legislature. It is stated by Caltrans officials that seismic safety is an integral part of design, along with many other issues. It is common for construction projects to have many criteria and constraints, all of which cannot be simultaneously met, particularly within budget and on time. The pressures of relieving traffic congestion and the limitations on funds led to severe budgetary problems for Caltrans. The 17-year period to implement the modest-cost cable restrainer program after the 1971 San Fernando earthquake suggests that seismic safety was not as pressing as other issues. It is natural that seismic requirements, when not specifically stated as part of policy, can become the subject of compromises as management makes allocations among competing uses for limited funds. Internal design guidance does not provide the same level of assurance of good seismic policies and practices as does a clear statement of minimum seismic safety goals for the agency, especially when there is no formal independent review process, as was true in Caltrans until recently. It is expected that a seismic safety goal established for Caltrans would vary with the degree of importance of the structure, with the highest importance afforded the most conservative criteria and careful design. Many factors may influence the degree of importance: cost to repair, economic consequences resulting from closure, redundancy of routes, delays and congestion resulting during repair.

14 Caltrans and the American Association of State Highway and Transportation Officials (AASHTO) seismic design codes had very low seismic requirements at the time of design of the Cypress Viaduct and the San Francisco Freeway Viaducts, both in comparison to those for buildings as specified by the Uniform Building Code of the same period and to current Caltrans requirements.

The Caltrans seismic coefficient, C, values were 0.02, 0.04 and 0.06 respectively for firm ground, soft ground and pile-founded structures at the time when the Cypress and San Francisco Freeway Viaducts were designed. The C value used for the design of these viaducts was 0.06. Since they were all built on pile foundations, the maximum value required at the time was used. From 1935 to 1949 the Uniform

Building Code for buildings specified that the coefficient should be either 0.08 for firm ground or 0.16 for soft ground. From 1949 to 1961 the UBC requirements were C=0.133 for the second story of a 2-story structure on firm ground and no special designation was made for soft ground. The Viaducts are not buildings; they have decks that are much more massive than typical building floors, less redundancy in vertical supports, and no walls or nonstructural elements that could provide added seismic capacity. Therefore, such structures may need even higher seismic design coefficients than buildings to achieve the same level of performance.

15 Caltrans bridge seismic design codes have improved substantially since the 1971 San Fernando earthquake, but have not been subjected to independent review.

After the experience of the 1971 San Fernando earthquake, Caltrans developed new seismic design specifications. These criteria introduced a modern approach to seismic design that recognized the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total bridge. The concept of a reduction factor, Z, for ductility and risk assessment was introduced to define seismic design forces obtained from modal analysis using realistic response spectra. The Caltrans seismic specifications have retained their present form since 1973, with regular refinements as new information became available. The Applied Technology Council published its FHWA-sponsored recommendations for bridge seismic resistant design in 1981. Their independent assessment led to recommendations that were based in large part on prior Caltrans seismic specifications. The technical basis for the Caltrans specifications is not well documented, and it is not known in some cases why technical decisions were made. Decisions on values used for the reduction factor were based on engineering judgement and observations of bridge performance in 1971. Caltrans seismic design criteria were not independently reviewed during their development and revision.

16 The basis for seismic design of Caltrans bridges before the Loma Prieta earthquake was that damage is acceptable as long as collapse is prevented. After the earthquake, the objective was modified to add that important structures will require only limited repair following major earthquakes.

Collapse prevention, with the inferred prevention of loss of life, was the principal objective of the Caltrans Bridge Design Provisions, applied uniformly without special considerations for important bridges. In the wake of the impacts of the Loma Prieta earthquake, Caltrans has indicated that they will introduce the importance of the structure into selecting the seismic design criteria for its design, with the goal of being able to restore vital transportation links quickly following a major earthquake. This will be translated into a performance standard that accepts limited, repairable damage for such structures.

17 Subsurface information customarily was not obtained by Caltrans in sufficient detail to enable a careful evaluation to be made of seismic loading conditions on foundations and the effects of soil-structure interaction.

The geotechnical engineering information available for the sites examined by the Board was not sufficiently detailed to allow careful evaluation of seismic loading conditions and soil-structure interaction. Caltrans generally accounts for site conditions by using site-specific spectral amplification ratios. Soil-structure interaction seldom appears to have been incorporated into the analysis, evaluation, and design of major structures until the last decade. For major projects, consideration of such interaction is appropriate and should be continued. There appears to be an under-representation of geotechnical expertise in the higher decision making ranks of Caltrans to complement the excellent skills in structural engineering and geology.

18 Most Caltrans concrete structures are of an age that they have nonductile detailing. Therefore, it should have been assumed by Caltrans that these were all at varying degrees of risk of failure in an earthquake.

It has been common knowledge within the structural engineering community for the past ten years that nonductile reinforced concrete structures are particularly earthquake vulnerable. Most Caltrans concrete bridges constructed before 1971 have nonductile details. Caltrans instituted design changes in 1971, following the San Fernando earthquake, that required new construction to utilize ductile details.

However, for the seismic upgrading of pre-1971 concrete bridges, a prior decision in the retrofit program dictated that the limited available funds should be used to install longitudinal restraints at the transverse expansion joints. This was done in an attempt to prevent failures of the type experienced in the 1971 San Fernando earthquake.

19 Many structures have been built that are deficient in their earthquake resistance. This has been caused by the slow development of new knowledge through limited research in earthquake engineering bridge design and the lag in putting research results into practice.

The seismic performance of structures has only recently received significant support for research, even though many building materials and systems had been observed as damage-prone in earthquakes for a long time. National support of earthquake research was small before the 1971 San Fernando earthquake. Following it, a focused, national research program in earthquake hazards reduction was developed, although it was not until 1975 that the program obtained sufficient resources to perform significant research on earthquake engineering topics. However, the current national level of support for earthquake engineering research is less than it was in the late 1970s. Only a small fraction of the national program is focused on experimental research, and only occasionally are projects supported on the seismic problems of bridges. Since 1971, Caltrans has supported a few specific projects at California universities, but their budgets and scope have been limited. Experience in structural research indicates that seldom is one study, or one series of experiments, sufficient to develop a full understanding of seismic performance. Thus, there has been very little research support relative to the needs for transportation structure design. Improvement in design practice is further hampered by the typical delays between the time something is discovered, either through research or observation of structural behavior in earthquakes, and the time at which the lessons learned are incorporated into common practice. Many developments in earthquake engineering applicable to bridge design have been developed for completely different types of structures, principally buildings. The wide participation of design and research engineers in the building code development process also fosters dissemination to and within the professions. These avenues for involving many engineers at many levels of practice are not as readily available for bridge design, because the institutional framework for development of seismic practices and the sharing of information with designers of buildings and researchers is not in place. Some Caltrans engineers are active in their professional organizations, both national and international, however, their participation is more limited than their counterparts in building design because of budget constraints. Caltrans's position in bridge design is so dominant within the State that there are relatively few other organizations designing bridges. While Caltrans has moved quickly to adopt some types of innovations, e.g. base isolation, it may be that the lag is much longer for information that appears on the surface to be more building-related.

20 Caltrans has implemented a number of actions to improve seismic design, including an independent review process for major projects.

Prior to the Loma Prieta earthquake, Caltrans did not systematically have independent technical reviews of their designs. It occasionally retained consultants when there were specific problems that needed resolution or when particular skills were needed for a design. On occasion it formed groups to review particular generic problems. However, design reviews were essentially internal. Following the Loma Prieta earthquake, Caltrans has taken or announced the following steps to improve its earthquake performance practices.

- Increased the staff size and resources of the Seismic and Structural Analysis unit of the Division of Structures.
- Appointed an Independent Review Committee for the repair and retrofit of the San Francisco Freeway Viaducts.
- Will establish a permanent Independent Seismic Advisory Board to review its criteria and design methodology.
- Will utilize independent peer review teams for major projects.
- Will establish and use an importance factor for the design of structures whose closure or failure would have major impacts.
- Will commission the Applied Technology Council to review its current seismic design criteria, including retrofit criteria.
- Committed resources to initiate a substantial earthquake engineering research program.

Findings on Bay Bridge Span Failure

21 The 50-foot-long upper and lower closure spans of the Bay Bridge over Pier E9 fell when the bolts failed that connected the pier with the 290' truss span to the east. Another span at pier E23, close to the eastern edge of the bridge, was near failure of a comparable type.

The closure spans linking the two trusses on Pier E9 had five inches of bearing on six-inch wide expansion-type support seats at each end, and had bolted connections at the east end. When the truss to the east broke free from its supports, the closure spans were pulled with it, and the motions were large enough to slide them off their support seats. As a result, the spans hinged down under gravity load, with the upper span coming down on the lower one. These then came to rest after striking the west truss-span connection to the pier.

22 The Bay Bridge was designed for 10% g earthquake accelerations, comparable to the levels specified in the 1930 Uniform Building Code for buildings.

The papers written shortly after the time of the design of the Bay Bridge indicate that this value was used; there is some lack of clarity on this point since the specifications for design reference 7.5%. The 10% value is consistent with the recommendations of building codes of the time. It should be noted that knowledge of damaging earthquake motions was very limited at this time. The first few measurements of strong ground motions were not made until the 1933 Long Beach earthquake.

23 The Bay Bridge appears to have no design or construction deficiencies as measured by the practices at the time it was built. There is no indication of subsequent maintenance deficiencies that contributed to failure of the span.

There is every indication that the Bay Bridge was an exemplary construction project for the period, conducted with quality materials and workmanship. The bridge's continued structural integrity appears to have been well ensured by regular maintenance.

24 Although some seismic structural rehabilitation had been completed on the Bay Bridge, there is no evidence that Caltrans was especially concerned about the earthquake collapse hazards posed by the Bay Bridge.

While there had been some seismic strengthening of the bridge with the installation of joint restrainers, rods and tie-downs, and steel restrainers incorporating elastomeric pads, there is no evidence that Caltrans had identified the Bay Bridge as any more earthquake vulnerable than the rest of its bridge inventory. Preparedness Plans developed by the Office of Emergency Services and Division of Mines and Geology for Hayward and San Andreas earthquakes included assumptions that major bridges would be out of service, but not because of failure or collapse of the bridges themselves. Rather, they assumed damage to approaches. It should be emphasized that these were formulated as planning assumptions for preparedness purposes and were not based on any engineering assessment of the expected performance of the bridges themselves.

25 There is no evidence that foundation failure contributed to the failure of the Bay Bridge span.

Examination of the underwater supports of the piers shows no signs of displacements at the soil surface or other signs of settlement, displacement or loss of bearing capacity. However, soil-structure interaction at the piers may have influenced the dynamic response of the bridge.

26 Given that the truss-to-pier connections on the east side of Pier E9 failed, the closure span would be considered in jeopardy of collapsing.

A five-inch movement of the 290' truss span to the east of Pier E9 relative to the pier would be enough to pull the closure spans off their seat supports. If the truss-to-pier connections failed, such a movement could easily have been expected in a major earthquake. There was no awareness on the part of Caltrans prior to the Loma Prieta earthquake that the truss-to-pier connection would fail.

27 The structural steps taken to repair the Bay Bridge appear to be appropriate for the short-term.

The approximate five-inch permanent displacement of the eastern span relative to Pier E9 was removed by jacking the trusses back into place. High strength bolts were used to attach the the trusses to the pier and larger seats for the replacement span were installed. The trusses at Piers E18 through E22 were also jacked back into position and connected to the piers with high strength bolts. Replacement restrainers were installed at these locations. The seismic adequacy of these repairs, and of the bridge itself, in the event of a severe earthquake needs to be investigated and any necessary long-term seismic upgrading completed expeditiously.

28 No engineering assessment of the dynamic, seismic performance of the Bay Bridge has ever been made.

A major effort would be required to make such an analysis. The Bay Bridge is a very large and complicated structure made of steel and concrete; it even has many wooden piles. It is several miles in length and is comprised of many separate, but connected structures, including several types of truss segments. It is geometrically complex, including a tunnel through Yerba Buena Island, and has foundations extending to rock and stiff soils through very soft, water-saturated soils. Steel, having substantial ductility, and trusses, having substantial redundancy, led to a very complex dynamic response. A dynamic analysis of the Bay Bridge is not straightforward because of these complexities and its results can not be intuitively inferred. Nevertheless, a seismic assessment is of primary importance.

29 The Bay Bridge may not be presumed to be adequately earthquake resistant just because it was only slightly damaged during the Loma Prieta earthquake and has since been repaired.

The level of ground shaking in the Loma Prieta earthquake was smaller in both duration and intensity than is expected in larger and closer earthquakes. The duration was not sufficient to excite all the different modes of response likely in a longer duration event. Nor was the level of shaking sufficiently close to that expected in major earthquakes to test the strength of bridge elements.

Findings on Cypress Viaduct Collapse

30 The Cypress Viaduct was designed and constructed to Caltrans seismic practices for reinforced concrete when built in the 1950s. It, and the San Francisco Freeway Viaducts, are brittle structures, possessing very little ductility, which was consistent with practices of the period.

These structures were designed to the Caltrans standards for seismic design of the time, which would now be termed nonductile design. AASHTO standards of the time did not contain specific seismic requirements. The ductile design of concrete dates from the mid 1960s, when a series of reinforcement details were developed that were thought to provide concrete structures with seismic capacity comparable to those of steel. Ductile design has been the subject of much research since then, with considerable development and changes in practice. It is now understood that reinforced concrete with inadequate confinement will rapidly lose its capacity to carry loads, either gravity or lateral, during cyclical motions. Early concepts of earthquake resistant design focused only on the strength of the structure. Modern approaches to design also consider dynamic behavior and maintenance of the ability of a damaged section to retain its load carrying capacity. When early post-tensioned bridges were built, there had been no experimental research or experience on their seismic performance.

31 The Cypress Viaduct appears to have been constructed according to plans and specifications with good quality materials and workmanship. There is no evidence of maintenance problems since construction that would affect its earthquake performance. The modifications to provide cable restrainers at the expansion joints appear to have been designed and installed to Caltrans specifications.

The material tests on concrete and reinforcing conducted after the earthquake indicate that their strengths meet or exceed the values specified in the design. One detail that may not have been constructed according to the design was noted: a bent of type B8 had shorter bars and anchorages as compared to the "as built" drawings. This could have had an effect on the failure of the structure, but it would have been minor at best. All things considered, the structure appears to have followed specifications and have been well constructed.

32 The Board is not aware of any evidence suggesting that Caltrans was specifically aware of the earthquake collapse hazards posed by the Cypress Viaduct.

The danger of collapse would not have been identified unless a thorough seismic analysis had been made, together with a failure analysis of this structure. After the near collapse of the retrofitted I-605/I-5 overcrossing structure during the 1987 Whittier Narrows earthquake, Caltrans embarked on a program to retrofit weak columns and evaluate the seismic performance of structures considering the structural system. Previously the retrofit program focused on cable restraints at expansion joints. The short time since the 1987 Whittier Narrows earthquake was not enough for Caltrans to identify the vulnerability of the Cypress Viaduct. It is, however, clear that Caltrans recognized much earlier that ductile detailing was required to achieve adequate seismic performance of reinforced concrete bridges, since it changed its general design practices after the 1971 San Fernando earthquake. This general observation did not translate into a specific understanding of the true seismic hazard posed by the Cypress Viaduct and did not cause it to be singled out as a high risk structure.

33 No evidence was presented to the Board that the foundation failed or that foundation problems contributed to the Cypress Viaduct collapse.

The foundations of the Cypress Viaduct are on a variety of soils types, e.g. engineered fill over bay muds, nonengineered fill over bay muds, and firm ground. Investigations indicate that there were no abnormal displacements of the soils at the foundations, nor were there indications of soil failure in the immediate vicinity. Supporting piles had not failed. The strong vibrations of the structure itself were sufficient to cause collapse.

34 **The Board concludes that a modern engineering seismic assessment of the Cypress Viaduct conducted before the earthquake, performed by a professional engineering organization in a manner consistent with the care and expertise usually exercised in evaluating such important structures, would have concluded that a collapse potential existed. Specifically, such an assessment would probably have concluded that:**

A. The Cypress Viaduct would collapse in a nearby major earthquake on the San Andreas or Hayward faults.

B. An earthquake with Loma Prieta's magnitude and location would probably not cause collapse, but would cause concern because of the weak, brittle nature of the structure.

C. Collapse would have been anticipated for the intensity of ground motion that did occur at the Cypress Viaduct site in the Loma Prieta earthquake, however, the extent of the collapse would probably not have been anticipated.

This structure had nonductile detailing of the reinforced concrete and inadequate confinement in joints as measured by today's standards of practice. Experience in past earthquakes identified such details as hazardous and would have caused an engineering reviewer to conclude that there were potential earthquake problems. An assessment of the shear and moment capacities of columns, beams and joints shows that they had in general less capacity than would be required by current Caltrans seismic design criteria for bridges or by the Uniform Building Code seismic design criteria for buildings. The number of hinges (three) in many bents would have led to questions on the capacity of the columns to accommodate large transverse inertial loads. It is likely that these deficiencies would would have lead to the conclusion that the Cypress Viaduct would collapse in strong shaking.

The Loma Prieta ground motions at the Cypress Viaduct site were stronger by at least a factor of two over what would have been predicted for the site based on commonly used attenuation relationships. Therefore, the degree of amplification of the ground motions on the soft foundation materials compared to firm ground would probably have been underestimated. Current code provisions do not assume such large amplifications, although recent research papers suggest that higher values should be used for very soft sites. However, these proposals had not been widely accepted by the design profession at the time of the earthquake.

Analyses performed after the earthquake indicate that the second story columns and pedestals were definitely subject to loads in excess of their capacity during the earthquake. These computations were based on elastic properties. Clearly, the multiple hinges (three) of some two-column upper level bents and the weak pedestals at the bottom of the two-hinged columns would have led to the conclusion that a few bents would fail, but it is doubtful that the entire structure would have been regarded as collapse-prone.

35 The demolition of a section of the standing Cypress Viaduct was very instructive and demonstrated the extent of collapse possible once local failure of a column or bent has occurred.

During demolition of the part of the Cypress Viaduct left standing, a four-bay, five-bent section (Bents 49-53) of the standing structure was demolished. A wrecking ball knocked out a corner column, the failure of which was sufficient to cause progressive collapse of the entire four-bay section. This tells much about the inadequacy of this type of structure to accommodate local failure. However, it should be noted that during demolition the opposite also occurred, i.e., a column was knocked completely out and no collapse of the upper deck ensued.

36 Tests indicate that retrofitting of the Cypress Viaduct columns and joints could have increased the seismic resistance of these elements.

After the earthquake, Caltrans contracted with the University of California at Berkeley to perform a series of experiments on a two-bay, three-bent (Bents 45-47) section of the Cypress Viaduct to determine its resistance to lateral cyclical loadings. Both the unretrofitted and retrofitted structure were tested to investigate approaches to improve performance of such structures and to provide guidance in retrofitting the San Francisco Freeway Viaducts. These tests were designed to determine two things: the seismic characteristics of the original structure prior to the earthquake, and the effectiveness of several proposed approaches to improving seismic strength of such freeway viaducts. The tests were not intended to measure the energy absorbing characteristics of the repairs, which are probably more important to overall seismic performance than the determination of strength. While the tests demonstrated that the strength of the columns and joints could be increased by relatively straightforward methods, they were not conclusive on the degree of increase in seismic capacity. These tests, and collateral research at the University of California at San Diego, did not address the important problems of frame joint/shear capacity and of degradation under dynamic, cyclic loadings.

Findings on San Francisco Freeway Viaducts

37 The San Francisco Freeway Viaducts are similar in design and construction to the Cypress Viaduct.

These viaducts (Embarcadero Viaduct, Terminal Separation Viaduct, Central Viaduct, China Basin Viaduct, Southern Freeway Viaduct, and Alemany Viaduct) in San Francisco were all built with the technology used for the Cypress Viaduct. They are the only other structures in the State of comparable design to the Cypress. The details of their designs are very similar, with many of the details of reinforcement placement identical to those of the Cypress. However, there are three key differences—first, there are several sections of the San Francisco Freeway Viaducts that are curved, whereas the parts of the Cypress Viaduct that failed were straight. Second, there are many more types of bents used in these structures, both individually and collectively. Third, the San Francisco Freeway Viaducts have prismatic rather than tapered upper frame columns and do not have pedestals. Engineering investigations performed to develop repair and retrofit approaches and designs have substantiated the similarity of design among the San Francisco Freeway Viaducts and the Cypress Viaduct.

38 The San Francisco Freeway Viaducts could be expected to suffer more severe damage, and possible collapse, if they had been subjected to the intensity of ground motions experienced by the Cypress Viaduct.

Observations of damage to some of the Embarcadero Viaduct bents and those in the other viaducts indicate a few shear failures of the second story columns, and more showed initiation of shear failure. None, however, collapsed. These San Francisco structures are estimated to have experienced ground motions in the range of 10%g to 15%g, less intense than those at the Cypress. No records were obtained at any of these structures. The Embarcadero Viaduct is on bay mud and had ground motions more intense than the other San Francisco Freeway Viaducts. Analytical investigations performed to support the design of repair and retrofit schemes of these structures confirm that many elements were substantially overstressed, with ductility demands too high for these nonductile structures. Cracks in the columns of the San Francisco Freeway Viaducts demonstrate what was probably the initial phase of damage to the Cypress Viaduct.

39 **The Caltrans repair and seismic retrofit of the San Francisco Freeway Viaducts is already underway. The retrofitting is expected to increase substantially the strength of the columns, but the precise degree of improvement in seismic resistance of the structures from these retrofits is not clear to the Board. The Board was unable to evaluate the specific details of the retrofit designs and programs for the individual viaducts in the time available, and considers them to be only short-term approaches.**

The experiments performed on a portion of the Cypress Viaduct before it was demolished clearly indicate that confinement of the columns and joints adds considerably to their strength. The degree of improvement and adequacy of these strengthening actions over the long-term are not known.

The materials provided to the Board do not present a compelling case that the procedures for upgrading the elements are necessarily a long-term solution to the seismic deficiencies of these structures. It may not be economically possible to retrofit these structures to the levels of seismic performance now required of new structures. Caltrans has appointed a special Independent Review Committee to review the repair and retrofitting of these structures. Caltrans should keep this Committee informed on all aspects of the six retrofit projects—their design and construction and the results of any relevant tests—and should request their advice and recommendations on the projects. The Committee should prepare a report that assesses the repair and retrofitting undertaken as a short-term solution and relates it to the long-term seismic performance of the viaducts.

Findings on Retrofit Program

40 **Caltrans has over 11,000 State-owned bridges within its jurisdiction, most of which were designed before basic understanding of earthquake engineering design was developed.**

Most of California's reinforced concrete bridges were designed and built before the 1970s and their seismic performance is suspect. In its recent seismic hazard assessment program, Caltrans has identified over 370 bridges in the high priority category for detailed seismic evaluation and potential retrofit.

41 **Caltrans instituted a seismic retrofit program in 1971 that, over the next 17 years, installed cable restrainers at expansion joints in over 1,200 bridges. Such restrainers are not generally sufficient to prevent collapse under very strong earthquake shaking.**

The 1971 San Fernando earthquake collapsed portions of the I-205/I-5 and I-5/SR-14 interchanges in Los Angeles County, modern structures then nearing completion, and severely damaged several other highway overpasses in the area. The response by Caltrans was: first, to adopt much better seismic design procedures for new concrete bridges; and, second, to systematically retrofit existing bridges throughout the State with cable restrainers, or other devices, to limit the opening that could occur at expansion joints. Examination of the damage and interpretation of the dynamic performance of the interchanges indicated that the greatest contributor to the damage was the differential movement at expansion joints, as adjacent parts of the structure moved in opposite directions, thus pulling a span off its supporting shelf and/or putting damaging loads into the columns. Caltrans concluded that the restraining motions at the expansion joints would keep the decks from falling and reduce forces in the supporting elements sufficiently to prevent collapse. The latter belief has been shown to be incorrect in the cases of the I-605/I-5 overcrossing during the Whittier Narrows earthquake and the Cypress Viaduct during the Loma Prieta earthquake.

42 The installation of cable restrainers under the Caltrans seismic retrofit program did not contribute to initiation of the collapse of the Cypress Viaduct. The precise influence of the cables on the failure process is not clear. The cable restrainers appear to have improved the behavior of San Francisco Freeway Viaducts, possibly saving some spans from collapse by limiting the relative displacements of the decks at the expansion joints.

There is no indication that the cable restrainers installed on the Cypress Viaduct contributed to the initiation of collapse, nor that they did anything to prevent it. The collapse was initiated by lateral, rather than longitudinal response. Engineering assessments of the strength of columns, the confinement of joints and the expected nonductile behavior of the structure leave little doubt that the capacity of the structural elements was exceeded during this earthquake and that they would have failed with or without cable restrainers. The characteristics of the mechanical behavior of cable restrainers is such that once the cable is taut, it transfers tension forces between the structures that otherwise would not have been so transmitted. In some cases, where an adjacent span collapsed, the restrainers on the Cypress Viaduct were pulled through the concrete members to which they were attached. These loads, transferred across the expansion joints, may have aided the collapse propagation, but the Board has made no conclusion regarding this mechanism. Several of the retrofit studies for the San Francisco Freeway Viaducts assessed their expected performance if the cable restrainers were not present. They indicate that the relative displacements at some expansion joints could have led to falling of the supported spans.

43 Caltrans began a second phase of seismic retrofitting following the 1987 Whittier Narrows earthquake in response to the near collapse of the I-605/I-5 overcrossing. This program was aimed primarily at strengthening single column bents of elevated structures and did not include the Cypress Viaduct.

A listing was made, following the Whittier Narrows earthquake, of bridges with single column bents that needed column retrofitting. The Cypress Viaduct was not on this list, because, being supported by pairs of columns, it was given lower priority than other structures supported by a line of single columns. Within program resource constraints, this was reasonable.

44 **Cities and counties within the State have responsibility for approximately 11,000 bridges and use the same criteria for design as Caltrans when federal or State funds are involved; their bridges can be expected to have the same seismic problems as those of the State.**

Cities and counties within the State have inventories of bridges comparable in number to those of the State. Cities and counties use Caltrans design and construction standards when there is either State or federal participation in the project, but it is not clear that they use them for other projects. The Board was unable to establish if cities and counties are required by law to use standards at least as conservative as those of Caltrans. Most of these city and county bridges were built in the same period as the State's, of comparable materials, and by the same contractor base. Many of the designers of these bridges and those responsible for administration of local bridge programs received their training as Caltrans employees. Thus, it is a reasonable conclusion that the types and extent of problems at the local level will be the same as the State's for comparable structures. The one factor moderating the problem is that local jurisdictions do not have the large, often unique, structures required for the State freeway system.

45 **An evaluation of the current Caltrans seismic retrofit program indicates that:**

A. The cable restrainer retrofit program addressed the first order failure mode for bridges, as identified in the 1971 San Fernando earthquake, and appears to have been an effective short-term, low-budget approach to improving the seismic performance of Caltrans bridges in relation to some, but not all aspects of response.

B. The single column reinforcement program appears to be reasonable for the short-term, if adequately planned and implemented in a timely manner.

C. The remaining retrofit program, currently being planned, will address the problem of multiple column bents and all bridges Statewide, and also appears to be reasonable for the short-term.

D. The complex response of bridges to earthquakes makes it unclear what specific retrofit program is best in the long-term, either from a

budgetary or seismic safety standpoint. It is clear that consideration of the entire structure, foundations and supporting soils is necessary to assess a retrofit approach.

The cable restrainer program grew out of an interpretation of the causes of failures in the 1971 San Fernando earthquake. A study of the failure of bridge spans in 1964 in both the Niigata, Japan and the Prince William Sound, Alaska earthquakes would have identified the serious implications of unrestrained joints. However, even if the problem had been identified by Caltrans, there is some doubt that resources would have been made available to fix the problem without the impetus of a California bridge collapse. The Caltrans single column phase of retrofitting directly responds to the fact that much more is known now about the design of earthquake-resistant reinforced concrete than was known when most California bridges were designed and built. By today's standards, the spacing of ties and confinement of concrete is inadequate in many pre-1971 reinforced concrete structures. Among the class of nonductile structures, the most pressing problems would reasonably be bridges supported on single column bents, since they have little redundancy. Caltrans's overall retrofit program now is directed at the performance of a structure as a whole, whereas it was previously focused on the resolution of the seismic problems of individual structural elements, particularly expansion joints. The Board feels that the retrofit program should continue to focus on the structure as a whole, including its foundations and supporting soils. Elements should not be abstractly considered outside the context of their use. Consideration of the structure as a whole will lead to different approaches to strengthening that offer better performance and economy. There are undoubtedly many good approaches yet to be identified and research on such should be a priority action before adoption of an overall approach to long-term retrofitting programs.

46 There are no widely accepted technical standards for seismic retrofit of bridges.

AASHTO does not have standards for either the seismic retrofit of bridges nor for the repair and seismic upgrading of damaged structures. There are at least three issues involved: first, development of criteria for performance of the upgraded or repaired structure, particularly as compared to newly-constructed structures; second, specific engineering requirements; and third, tests and procedures for specific application. There are a number of reports from FHWA, the Applied Technology Council, and the University of California at San Diego that address these issues. Caltrans has a set of procedures and details for use by designers. However, no code developing organization has developed technical standards for seismic upgrading. Except for unreinforced masonry, such standards do not exist for buildings either, but are currently under development.

Findings For Other Types of Structures

47 **A substantial number of California buildings and facilities are deficient in seismic resistance as measured by current standards. The fact that a particular type of structure has not yet been damaged in earthquakes does not necessarily indicate that its earthquake resistance is adequate.**

Seismic codes and design procedures have undergone substantial improvements over the years. This means that the seismic resistance of many structures is not up to modern requirements. The hazards posed by unreinforced masonry buildings have been recognized and efforts are being made to strengthen or remove them. However, other types of buildings and facilities constructed in the first 75 years of this century also have seismic deficiencies. The risk posed by these structures has not been given the study or action it deserves.

It is sometimes asserted that there has never been an earthquake failure of a specific type of structure and therefore it is seismically safe, but often such structures have not been exposed to strong shaking. California is fortunate that it has not had a major earthquake in an urban area since 1906. While there have been many damaging earthquakes since that time, few caused strong ground shaking of modern structures. When they have, the performance has sometimes been unsatisfactory. The simple fact is that the nature of construction, types of materials used, design principles, and types of buildings constructed have changed greatly since 1906. As engineers repeat a practice, there is a natural tendency to believe that the practice is a good one, even though it may be untested. The longer it is used the more confidence is placed in its validity and the less likely it is that it will be challenged. This can lead to complacency, particularly for institutions that have control of the full design and construction cycle of a particular type of structure. Seismic deficiencies in particular types of structures can be determined by engineering investigations with reasonable confidence without waiting for an earthquake to demonstrate the structure's shortcomings.

48 Independent, technical review is essential to achieve consistent excellence in civil engineering design and construction.

The American Society of Civil Engineers manual titled *Quality in Constructed Projects* states:

> Projects that should be peer reviewed are those that are complex, unique, or would have great consequence should failure occur. A project peer review should be conducted if the owner/client wants extra assurance that a project design of acceptable quality will be received at a fair cost and is prepared to pay for that added assurance by means of peer review.

The Board of Inquiry endorses this statement. The practice of civil engineering is not yet precise. The process of design of structures is still one that entails the making of many decisions and technical compromises.

Building codes were first developed as a means of ensuring that structures constructed by private owners met minimum safety standards. A code has two essential parts to be followed: the technical provisions and the administrative procedures. The administrative requirements include independent review of the design drawings, specifications, and calculations by the building department to ensure compliance with the technical provisions, and then periodic independent field inspection of the work in progress to maintain adequate craft performance and standards. The 1933 Long Beach earthquake caused considerable damage to many types of private buildings and to public school houses. The State legislature found that the building codes in the region were highly variable, with some having little or no seismic provisions, independent plan reviews, or field inspection. In response to the belief that all private buildings within the State should provide a minimum level of safety, the Riley Act was passed. It required that all jurisdictions adopt and enforce building codes. In response to the damage and collapse of school houses, the Field Act was passed, which required that special seismic standards be developed for schools that were more conservative than those for ordinary buildings and that their designs and construction be reviewed by the State for compliance with these standards. As a consequence, schools, as regulated by the Office of the State Architect, have performed very well in earthquakes during the intervening years.

In 1971 a number of hospitals failed in the San Fernando earthquake. Again it was found that public hospitals were not subject to code review, and that private hospitals were regulated as if they were ordinary buildings. The legislature determined that hospitals were of such importance to the community that it directed that standards for their design and construction be developed to ensure their operation after an earthquake and that a State agency review and approve their plans and construction. Following the failure of several dams in the

1920s, similar procedures were adopted by the State for the safety of dams.

These requirements for schools and dams have been effective and are neither onerous nor difficult to implement. In effect, the State has created special building department functions for classes of buildings that fall outside the province of local building departments. These experiences clearly demonstrate that design not subject to review often has higher earthquake damageability than does construction that is reviewed, and that the State can effectively provide seismic design review and achieve higher degrees of seismic safety and lower damageability. The civil engineering and design professions accept peer review and readily participate in it, realizing that it is an action that fosters better designs and that it is a useful approach to ensuring that problems are identified and resolved before and during construction.

A recent report to the State and Consumer Services Agency considered problems posed by the multiplicity of construction standards and practices of California State agencies [State and Consumer Services Agency, Jan. 1990]. It recommended that a single, independent agency perform the same functions for State owned buildings as a local building department does for its community. This would meet the requirements for independence that the Board feels to be vital and would ensure, if appropriate legislation is enacted, that every new State structure would adhere to acceptable seismic standards of design and construction.

49 The registration of professional engineers with a specialty in bridge design is not warranted.

Caltrans engineers that design bridges and other highway structures are licensed civil engineers. The Board finds no compelling need for a special license for bridge design, and cautions that the public would be ill-served by creating a proliferation of specialized structural engineering licenses. A civil engineer must have a given number of years experience designing buildings in order to qualify to take the Structural Engineer examination; bridge design experience does not fulfill the building design experience requirement.

50 Loss of life from and damage to currently existing substandard structures will dominate the impacts suffered in future California earthquakes.

The vast majority of the buildings and structures that will fail in future earthquakes exist now. The bulk of California structures were built before basic understanding of earthquake engineering and the nature of earthquake hazards was developed. It is only in the 1970s and 1980s that designers began to understand the scientific and engineering concepts necessary to provide adequate levels of earthquake hazard mitigation. While many of the structures built before this period provide adequate safety, there are others, some recognized and some unrecognized, that do not. California has noted the extreme hazard posed by unreinforced masonry buildings by passing Senate Bill 547 (Chapter 250) in 1986 that requires every city and county to inventory their high hazard buildings and develop a program to reduce their hazards. The SB 548 (Chapter 1941) program adopted in 1985 set up a comprehensive plan to address a wide range of hazards and to strive for substantial reduction in the overall danger by the year 2000. By adopting these acts, California has made a commitment to improving seismic safety.

51 Many structures are not subject to seismic codes or to review by an independent third party before construction.

Contrary to the beliefs of most of the public, the local building code does not apply to all structures within the community. Federal- and State-constructed buildings are exempt, as are non-building structures and industrial facilities that lie outside the administrative scope of the local building code. Those buildings not subject to the building code are usually subject to other safety regulations, e.g. OSHA. The owners often determine for themselves what building, as well as seismic, standards will be followed, and then self-regulate their compliance. Civil works, such as bridges and transportation structures, railways, and utility structures, are not included within the local regulatory framework. Often the institutions or governmental agencies constructing facilities are subject to no independent examination of the seismic safety standards they use and no plan and construction review for individual projects. Special legislation has subjected some types of buildings (public school houses and hospitals are notable), dams, and particularly high hazard structures (nuclear power plants and LNG facilities) to rigorous seismic design and construction review. This is not enough. The Board feels that all structures built within California should have to meet minimum seismic safety standards as reviewed by an independent entity.

52 Many State-owned structures are seismically substandard and many known hazardous conditions have not been addressed.

Reviews of the seismic safety of University of California facilities have been well documented in the press: UCLA's and U.C. Berkeley's collapse hazard buildings are an example. The California State University system faces similar problems. The 1987 Whittier earthquake, with its substantial damage to buildings at California State University at Los Angeles, the 1978 Santa Barbara earthquake with damage to U.C. Santa Barbara buildings, and the 1980 Livermore Valley earthquake with damage to buildings at Lawrence Livermore Laboratory of the University of California demonstrate the hazard. The seismic hazards of some State-owned office buildings, e.g. in Los Angeles and San Francisco, have also been well publicized.

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Chapter 5

Recommendations To Improve California's Earthquake Safety

The last of the Governor's five directives to the Board of Inquiry was to:

> ...make recommendations as to whether the State should modify the existing construction or retrofit programs for freeway structures and bridges in light of new information gained from this earthquake.

This chapter distills the findings of the Board's deliberations into eight specific recommendations for action that, in the Board's opinion, can improve earthquake safety in the State.

California has an earthquake problem—not just for its bridges, but for its entire constructed environment. While this earthquake may be remembered for the images of life loss and damage to the Cypress Viaduct and Bay Bridge, its impacts were much broader. Many possible actions were considered for recommendation by the Board. From among these the Board selected those few that should receive the highest priority. The following recommendations focus on practices of Caltrans with respect to transportation structures, but also include recommendations for other structures within the purview of other State and local agencies.

Earthquakes have occurred regularly in the past and will in the future. The Working Group on California Earthquake Probabilities estimates that the potential for one or more large earthquakes in the San Francisco Bay area is considered to be 50% for the 30-year period following January 1, 1988 [Dietrick, 1988]. For the southern San Andreas fault and San Jacinto fault, both in southern California, the potentials are respectively 50% and 60% for the same period. There are many other faults capable of and likely to produce large, damaging earthquakes. Thus, the implications of these recommendations are State-wide.

The Board of Inquiry has identified three essential challenges that must be addressed by the citizens of California, if they expect a future adequately safe from earthquakes:

- **Ensure that earthquake risks posed by new construction are acceptable.**
- **Identify and correct unacceptable seismic safety conditions in existing structures.**
- **Develop and implement actions that foster the rapid, effective, and economic response to and recovery from damaging earthquakes.**

These challenges are common sense—stop the increase, decrease the unacceptable, and prepare for the consequences. The attitude "don't fix it unless its broken" is common to us all. Through its inquiry, the Board has determined that "it" is broken and needs fixing. To be sure, we have done many things well from a seismic safety point of view, but we are faced with problems of sufficient seriousness and breadth that they warrant concerted, timely action, lest we are faced with a succession of future Boards of Inquiry asking yet again "What went wrong?"

These challenges address not only the issue of bridges, whose failure prompted the Board's formation, but also all the other constructed facilities upon which our modern economy and well-being depend. The Board might have limited its recommended actions only to those it believes necessary to correct problems with State-owned bridges. But to do so would have avoided the most fundamental of responsibilities—to provide for the public safety. The Board has interpreted its Charter in a broad sense and has made recommendations that are directed both at seismic issues for bridges and some of the larger issues of seismic safety facing the State.

The Board has developed eight recommendations for implementation. These recommendations identify what is to be done and by whom. Discussions follow the recommendations that provide the findings and arguments that led the Board to adopt them. The discussions are necessarily brief. They should be understood within the context of the findings of Chapter 4 and the technical materials presented in the balance of this Report. The findings on which these are based are given in detail in the recommendations of Chapter 4.

1 FOR ACTION BY THE GOVERNOR: Affirm the policy that seismic safety shall be a paramount concern in the design and construction of transportation structures. Specific goals of this policy shall be that all transportation structures be seismically safe and that important transportation structures maintain their function after earthquakes.

The most fundamental tenet of management is that an organization must know what is expected of it. The Board found that Caltrans does not have a specific seismic safety performance goal that must be met by all its structures. Requirements for accommodating earthquakes are contained in design documents for use by Caltrans engineers for individual projects, not as policy guidance or requirements from the management of Caltrans, from the Governor, or from the Legislature. The Board accepts that seismic safety is an integral part of the Caltrans design process, as are many other important issues. By implementing this recommendation, the State goes on public record that seismic safety of its transportation structures is not to be compromised.

The basis for seismic design of Caltrans bridges before the Loma Prieta earthquake was that damage is acceptable as long as collapse is prevented. After the earthquake, this objective was modified to add that important structures will require only limited repair following major earthquakes. Collapse prevention, with the inferred prevention of loss of life, was the principal objective of the Caltrans Bridge Design Provisions, without accommodations for the importance of individual bridges. In the wake of the Loma Prieta earthquake, Caltrans has indicated that they will introduce the importance of the structure into selecting the seismic design criteria for its design, with the goal of being able to restore vital transportation links more quickly following a major earthquake. This will be translated into a performance standard that accepts limited, repairable damage for such structures. The Board believes that the introduction of an importance factor is proper and warrants permanence.

The first goal of life safety is to be maintained for all structures, both new and existing. Fulfilling this goal requires assessing the seismic stability of existing bridges, principally those built before 1971. This is in process; their retrofitting, as required, has been initiated; and a plan is in preparation for completion.

The second goal, that the function of important structures is to be maintained, is a more restrictive one. Many factors may influence the degree of importance: community values, economic function performed, redundancy of routes, cost to repair, delays and congestion resulting during repair. It is expected that new important structures will be constructed to meet this goal with little disruption of the normal design and construction process, with the highest importance structures afforded the most conservative design. Correcting the seismic deficiencies of existing important structures is likely to be a long process requiring sustained effort.

The Board finds that the fiscal environment at Caltrans during the past two decades has inhibited giving the level of attention to seismic problems that they required. A policy statement is needed from the Governor that seismic safety shall be a paramount concern in the design and construction of transportation structures. It will provide the guidance required to design new structures and retrofit remaining substandard transportation structures under the unambiguous direction that seismic safety is not to be compromised.

2 **FOR ACTION BY THE GOVERNOR: Establish that earthquake safety is a priority for all public and private buildings and facilities within the State by taking the following actions:**

A. Propose legislation to ensure that every new facility in the State not otherwise subject to adequate seismic regulation and having the potential to cause substantial life loss during an earthquake be subject to compliance with adequate seismic safety standards for construction.

B. Require that seismic safety be a paramount concern in the design and construction of all State-owned structures. Specific goals of this policy shall be that all State-owned structures be seismically safe and that important State-owned structures maintain their function after earthquakes.

C. Initiate and fund a vigorous, comprehensive program of research to improve the capability in engineering and the physical and social sciences necessary to mitigate earthquake hazards and to implement the technology transfer and professional development necessary to hasten practical use of research results.

Life loss from and damage to currently existing substandard structures will dominate the impacts in future earthquakes in California. The bulk of California construction was built before basic understanding of earthquake engineering and the nature of earthquake hazards was developed. It was only in the 1970s and 1980s that designers began to understand the scientific and engineering issues necessary to provide adequate levels of earthquake hazard mitigation. But current knowledge of how to identify and mitigate the hazards posed by these structures is inadequate, and is unproven at best.

The same problems exposed by the Loma Prieta earthquake for some bridge structures exist for buildings and other structures. Unfortunately, the earthquake resistance of many structures is not subject to compliance with seismic codes or to review by an independent third party before construction, just as bridges were not. Contrary to the beliefs of most of the public, the local building code does not apply to all structures within the community. Federal- and State-constructed buildings are exempt, as are non-building structures and industrial facilities that lie outside the administrative scope of the local building code. Many State-owned and controlled structures are seismically substandard, and many known hazardous conditions have not been addressed.

The Governor should seek the same level of seismic performance for all structures as for transportation structures. In a real sense these are more pressing problems than those of bridges, since there are so many more of them, and since there is such wide variability in the practices used for their construction.

Experience in California earthquakes clearly demonstrates that design not subject to review often has higher earthquake damageability than does construction that is reviewed. The State has demonstrated that it can effectively provide seismic design review and achieve higher degrees of seismic safety and lower damageability. The civil engineering and design professions accept peer review and readily participate in it with the realization that it is an action that fosters better designs and that it is a useful approach to ensure that problems are identified and resolved before construction. State review of individual projects for earthquake safety has been implemented by the Office of Dam Safety since 1925 and the Office of the State Architect for school houses since 1933. These have been effective and are neither onerous nor difficult to implement. A recent report to the State and Consumer Services Agency considered problems posed by the multiplicity of construction standards and practices of California State agencies [State and Consumer Services Agency, Jan. 1990]. It recommended that a single, independent agency perform the same functions for State-owned buildings as a local building department does for its community. This would meet the requirements for independence that the Board feels is vital and would ensure, if appropriate legislation is enacted, that every new State structure would adhere to acceptable seismic standards of design and construction.

Research in earthquake hazards reduction has contributed much to our knowledge of improved building design and the need for high quality construction practices. A revolution in practice has begun that, if continued, promises greater safety and economy of construction. Its continuance is threatened, however, by shifts in national goals and priorities. To date, California has relied almost exclusively on federal support of research for this advancement, with research needs, goals and priorities set at the national level. In recent years a divergence has developed between the needs of California and the priorities of the federal programs as they have moved toward greater recognition of the needs of other states. Significantly, matters that need close attention to deal with California's vulnerability are not being emphasized, particularly the pressing ones of existing hazardous structures. If the impacts of future earthquakes are to be reduced, then Californians must commit resources to influence the direction and pace of research and implementation by:

- Strengthening support for research necessary to meet the future needs of engineering practice and public policy
- Focusing some capabilities on problem-oriented research
- Fostering more thorough and effective use of existing knowledge and research findings

The Seismic Safety Commission was created following the 1971 San Fernando earthquake to provide policy review and guidance to the Executive and Legislative branches of government on earthquake related issues. It has a membership of 20, who represent many different interests within the State, including the public, business, commerce, engineering, science, and State, county and local government. It has focused its staff resources on seismic safety issues of State government. If we are to foster better earthquake safety in the community at large, then we need independent assessments of current actions and an advocate for better policies. The Seismic Safety Commission, as a dual institution of the Executive and Legislative branches of government, must have the resources to perform these functions.

3 FOR ACTION BY THE GOVERNOR: Direct the Seismic Safety Commission to review and advise the Governor and Legislature periodically on State agencies' actions in response to the Recommendations of this Board of Inquiry.

To ensure that the recommendations of the Board of Inquiry are indeed acted upon will require monitoring the implementing actions of the State agencies. The Seismic Safety Commission is the natural institution within government to monitor performance. The Board has confidence that the Commission can satisfactorily discharge this responsibility.

4 FOR ACTION BY THE DIRECTOR OF THE DEPARTMENT OF TRANSPORTATION: Prepare a plan, including schedule and resource requirements, to meet the transportation seismic performance policy and goals established by the Governor. The plan shall include the timely seismic retrofitting of existing transportation structures.

The Board has found that many bridges have been built that are deficient in their earthquake resistance; fortunately only a small fraction pose a serious threat. These few warrant concerted action to bring them into conformance with the policy and goal of these recommendations.

Central to achieving the goal of the first recommendation is a planned program of actions, one that identifies what is to be done, by whom, when, and at what cost. A well-planned program will provide a basis for annual actions that allocates resources and personnel. The Board thinks it is essential to have a publicly available program plan that identifies the steps that will be taken over a period of time to achieve the goal of having adequately safe transportation structures. The plan should be periodically reviewed and revised to reflect changing conditions. Optimally, the plan should be completed, resulting in seismically safe structures, before the next major earthquake tests our actions. The Working Group on California Earthquake Probabilities has assessed that there is about 20% chance of a major earthquake in the Bay Area or on the southern San Andreas fault in the next 5 years, and a 60% probability in the next 20 years. These probabilities are guideposts for planning. Actions necessary for life safety should be completed within five years, and actions to maintain function should be completed within 20 years.

The Board believes that it is reasonable to expect Caltrans to complete the majority of the actions necessary to provide life safety for ordinary transportation structures within the next 5 years. Caltrans has stated that this is their plan. It will be more difficult to ensure that all important transportation structures be able to survive earthquakes without loss of function. The task of assessing and correcting the seismic deficiencies of these structures is more demanding, and may require as long as 20 years to complete. The Board expects that, after a preliminary evaluation of these structures to determine priority, full seismic analyses to determine expected performance in future earthquakes can be completed within five years. These analyses will then provide a basis for appropriate engineering design.

The early initiation of a vigorous research program on earthquake engineering is vital. Research can provide much of value to both the assessment of bridge performance and the development of economical, efficient, and workable seismic upgrading procedures. Early completion of research efforts, so that results are available early in the assessment and design process, is far better than to wait until later to find that new knowledge invalidates what has been done to date. Continuing research provides the improvements in knowledge required for steady progress toward the goals of safety and economy.

Caltrans's seismic safety plan should include constructive steps to limit the danger posed by substandard structures. Posting a substandard bridge as "having a potential for life loss in an earthquake" is not is an acceptable solution.

5 FOR ACTION BY THE DIRECTOR OF THE DEPARTMENT OF TRANSPORTATION: Form a permanent Earthquake Advisory Board of external experts to advise Caltrans on seismic safety policies, standards, and technical practices.

The Board finds that Caltrans has the reputation of being the best transportation agency among the States and a leader in bridge design. But no matter how good the organization, as engineers repeat a practice, there is a natural tendency to believe that the practice is a good one, even though it may be untested. The longer it is used, the more confidence is placed in its validity and the less likely it is that it will be challenged. An

Independent Advisory Board of qualified individuals can provide the periodic review and advice that would ensure that the policies and practices of Caltrans avail themselves of the best available information and thinking. The Advisory Board should have its membership drawn from private practice and research, including individuals experienced in design of bridges and buildings, and experts in all aspects of seismic design from geotechnical to structural engineering. If there had been external review of the strengthening program undertaken after the 1971 San Fernando bridge failures, it is doubtful that the hazard of nonductile concrete structures, such as the Cypress Viaduct would have been overlooked. The Advisory Board should meet at least semi-annually and be given access to the full operations of Caltrans. Its regular reports should review the earthquake engineering practices of Caltrans and recommend to the Director actions for improvement.

6 FOR ACTION BY THE DIRECTOR OF THE DEPARTMENT OF TRANSPORTATION: Ensure that Caltrans seismic design policies and construction practices meet the seismic safety policy and goals established by the Governor:

A. Review and revise standards, performance criteria, specifications, and practices to ensure that they meet the seismic safety goal established by the Governor and apply them to the design of new structures and rehabilitation of existing transportation structures. These standards, criteria, and specifications are to be updated and periodically revised with the assistance of external technical expertise.

B. Institute independent seismic safety reviews for important structures.

C. Conduct a vigorous program of professional development in earthquake engineering disciplines at all levels of the organization.

D. Fund a continuing program of basic and problem-focused research on earthquake engineering issues pertinent to Caltrans responsibilities.

The Board finds that Caltrans's bridge seismic design codes have improved substantially since the 1971 San Fernando earthquake, but lack independent verification. After the experience of the 1971 San Fernando earthquake, Caltrans developed new standards. These criteria introduced a modern approach to seismic design and recognized the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total bridge. The basis for Caltrans's standards is not documented, and it is not known why technical decisions were made. Caltrans seismic design criteria have not been tested by research and were not subjected to independent review. The Board believes that much will be gained if engineers outside of Caltrans are involved in the development of bridge design specifications and standards. Experience in the seismic performance of other types of structures has much to offer the design of transportation structures.

Prior to the Loma Prieta earthquake Caltrans did not systematically have independent technical reviews of its major projects or earthquake engineering practices and policies. It occasionally retained consultants when there were specific problems that needed resolution or specific skills needed in the design. On occasion, they formed groups to review particular generic problems. However, the review of designs was essentially internal.

The Board finds that independent, uncompromising technical review is essential to achieve consistent excellence in civil engineering design and construction. It agrees with and endorses the statement of the American Society of Civil Engineers as stated in its manual entitled *Quality in Constructed Projects:*

> Projects that should be peer reviewed are those that are complex, unique, or would have great consequence should failure occur. A project peer review should be conducted if the owner/client wants extra assurance that a project design of acceptable quality will be received at a fair cost and is prepared to pay for that added assurance by means of peer review.

Investigations of the seismic performance of structures has only recently become more than a limited subject for research, even though many building materials and systems had been observed as damage prone in earthquakes for a long time. National support of earthquake research was limited before the 1971 San Fernando earthquake. Following it, a focused, national research program in earthquake hazards reduction was formed, although it was not until 1975 that the program obtained sufficient resources to perform significant research on earthquake engineering topics. The national level of support for earthquake engineering research is currently less than in the late 1970s, not even accounting for inflation. Only a small fraction of the national program is focused on experimental research, and only occasionally are projects supported on the seismic problems of bridges. Since 1971, Caltrans has supported a limited number of sharply focused projects at California universities, but their number, budgets and scope have been limited. Thus, there has been relatively little in research support pertinent to the needs for transportation structure design.

The issue of improving practices is complicated by the typical delays between the time something is discovered, either through research or observation of structural behavior in earthquakes, and the time in which it becomes common practice. Many developments in earthquake engineering applicable to bridge design have been developed for totally different types of structures, principally buildings. For buildings, new information is often rapidly adopted by a single firm and then used by other firms. The Board believes that the lag between when information appears and when it becomes part of practice will be substantially shortened with an even more vigorous professional development program within Caltrans than is already in place, one that will include staff participation in the professional activities of the whole civil engineering community.

7 FOR ACTION BY THE DIRECTOR OF THE DEPARTMENT OF TRANSPORTATION: Take the following actions for specific structures:

A. Continue to sponsor and utilize the Independent Review Committee's technical reviews of the engineering design and construction proposed for the short-term repair and strengthening of the San Francisco Freeway Viaducts.

B. Develop a long-term strategy and program for the seismic strengthening of existing substandard structures, including the San Francisco Freeway Viaducts, that considers their overall behavior, the degree of seismic risk, and the importance of the structure to the transportation system and community.

C. Perform comprehensive earthquake vulnerability analyses and evaluation of important transportation structures throughout the State, including bridges, viaducts, and interchanges, using state-of-the-art methods in earthquake engineering.

D. Implement a comprehensive program of seismic instrumentation to provide measurements of the excitation and response of transportation structures during earthquakes.

The San Francisco double-deck viaducts could be expected to suffer severe damage and possibly collapse if they had been subjected to the intensity of ground motions experienced by the Cypress Viaduct during the Loma Prieta earthquake. The Caltrans repair and seismic retrofit of the San Francisco double-deck viaducts is already underway. The retrofitting is expected substantially to increase the strength of the columns, but the precise degree of improvement in seismic resistance of the structures is not clear.

Caltrans has appointed a special Independent Review Committee to review the repair and retrofitting of these structures. Caltrans should keep the Committee informed on all aspects of the six retrofit projects—their design and construction and the results of any relevant tests—and should request their advice and recommendations on the projects. The Committee should prepare a report giving its assessment of the repair and retrofitting undertaken as a short-term solution and how it relates to the long-term seismic performance of the viaducts.

The materials provided to the Board do not present a compelling case that the procedures for upgrading the the San Francisco Freeway Viaducts are necessarily a long-term solution to the seismic deficiencies of these structures. It may not be possible to retrofit these structures to the level of seismic performance now required of new structures. The Board is unable to evaluate the particular details of the retrofit designs and programs for the individual viaducts, but considers them to be only short-term approaches to repair. The degree of improvement and the adequacy of these strengthening actions for the long-term is not known.

No comprehensive seismic analyses of the expected seismic performance of major State transportation structures (e.g., Bay Bridge, Richmond-San Rafael Bridge) have been completed since their design by Caltrans. The design of the Bay and Golden Gate Bridges in the early 1930s employed the state-of-the-art in earthquake engineering at the time of their design. But in the intervening years much has been learned both through research on earthquake engineering and through observations of structural performance in earthquakes. The changes in our understanding have touched every aspect of seismic design and structural performance—from specification of the ground motion and frequency characteristics at a site to the energy absorbing characteristics of materials under cyclical loadings. Many practices previously thought to be conservative are now thought to be questionable.

These structures are too important to wait until a future earthquake to discover if they perform unacceptably. Earthquake engineering assessments should be completed to determine if their performance is acceptable and, if necessary, to determine how their seismic performance can be improved to an acceptable level. Such analyses should be based on a probabilistic assessment of the ground shaking likely at the site.

8 FOR ACTION BY TRANSPORTATION AGENCIES AND DISTRICTS:

Agencies and independent districts that are responsible for transportation systems—rail systems, highway structures, airports, ports and harbors—should:

A. Adopt the same seismic policy and goals established by the Governor for State transportation structures and implement seismic practices to meet them.

B. Perform comprehensive earthquake vulnerability analyses and evaluations of important transportation structures—e.g., the BART Trans-bay Tube and Golden Gate Bridge—using state-of-the-art methods in earthquake engineering, and install seismic instrumentation.

C. Institute independent seismic safety reviews for important structures.

D. Conduct a vigorous program of professional development in earthquake engineering disciplines at all levels of their organizations.

Many transportation agencies and districts are not administratively responsible to the Governor, and therefore can act independently. Their transportation structures (e.g. Golden Gate Bridge, BART Trans-bay Tube, Metro Rail) are just as important to the communities they serve as the transportation structures maintained by the State. The Board recommends that these organizations adopt the same policies, goals, and practices as recommended for Caltrans.

Federal criteria, when used for California transportation projects, may not be sufficiently conservative and inclusive of seismic concerns to meet the seismic safety needs of the State of California. Federal criteria only supersede State specifications where the State specifications are less stringent than the federal requirements. Caltrans' seismic criteria have led AASHTO's, in effect making Caltrans criteria the ones enforced for highway structures within California. However, for other transportation related structures, where the State does not have criteria, federal criteria are likely to prevail for projects involving State and federal funding. Federal criteria are drawn to meet the needs of all the states, only a few of which are subject to earthquake hazards as severe as those of California. The seriousness of California's earthquake hazards and its need for continued functioning of transportation systems pose special problems. These organizations should adopt seismic performance goals and standards comparable to those of the Caltrans-administered elements of our transportation system.

No comprehensive seismic analyses of the expected seismic performance of other major transportation structures (e.g. BART Trans-bay Tube designed in the late 1960s) have been completed since their design by the authorities responsible for them. For all the same reasons that Caltrans should assess its important structures, these organizations should also assess their structures' expected performance and take steps to ensure their adequacy.

Many of the structures owned and operated by these organizations, e.g. the Golden Gate Bridge, are unique and therefore pose difficult earthquake engineering problems of analysis. It is likely that several earthquakes will shake a structure before an earthquake sufficient to cause damage occurs. The installation of seismic instrumentation on these structures offers the opportunity to understand their actual performance during earthquakes and anticipate their performance in large earthquakes. Such information may prove to be critical in determining whether seismic upgrading is required to ensure adequate future earthquake performance.

The Board has made the case that independent review and expanded, vigorous professional development will benefit Caltrans practice and lead to better seismic performance of its structures The same reasoning has led the Board to conclude that all independent transportation agencies and districts should have independent seismic safety reviews of their projects and vigorous professional development programs in earthquake engineering at all levels of their organizations. Their structures serve vital functions within the community, just as do Caltrans's. They perform many of the same design and regulatory functions as does Caltrans for its structures. It is appropriate for the public to have the same level of concern for the seismic safety of these structures. It is just as important that independent transportation agencies have well qualified staff as it is for Caltrans.

Conclusions

The Loma Prieta earthquake should be considered a clear and powerful warning to the State of California. Although significant progress has been made in California during the past two decades to reduce earthquake risks through proper design and construction of State facilities, much more could have been done and awaits doing. More aggressive efforts to mitigate the consequences of earthquakes are needed in the continuing programs of construction and retrofitting of State structures if the disastrous potential of a great earthquake is to be minimized. The State of California must not wait for the next great earthquake, and the likely tens of billions of dollars in damage and thousands of casualties, to accelerate hazard-mitigation measures. Earthquakes will occur—whether they are catastrophes or not depends on our actions.

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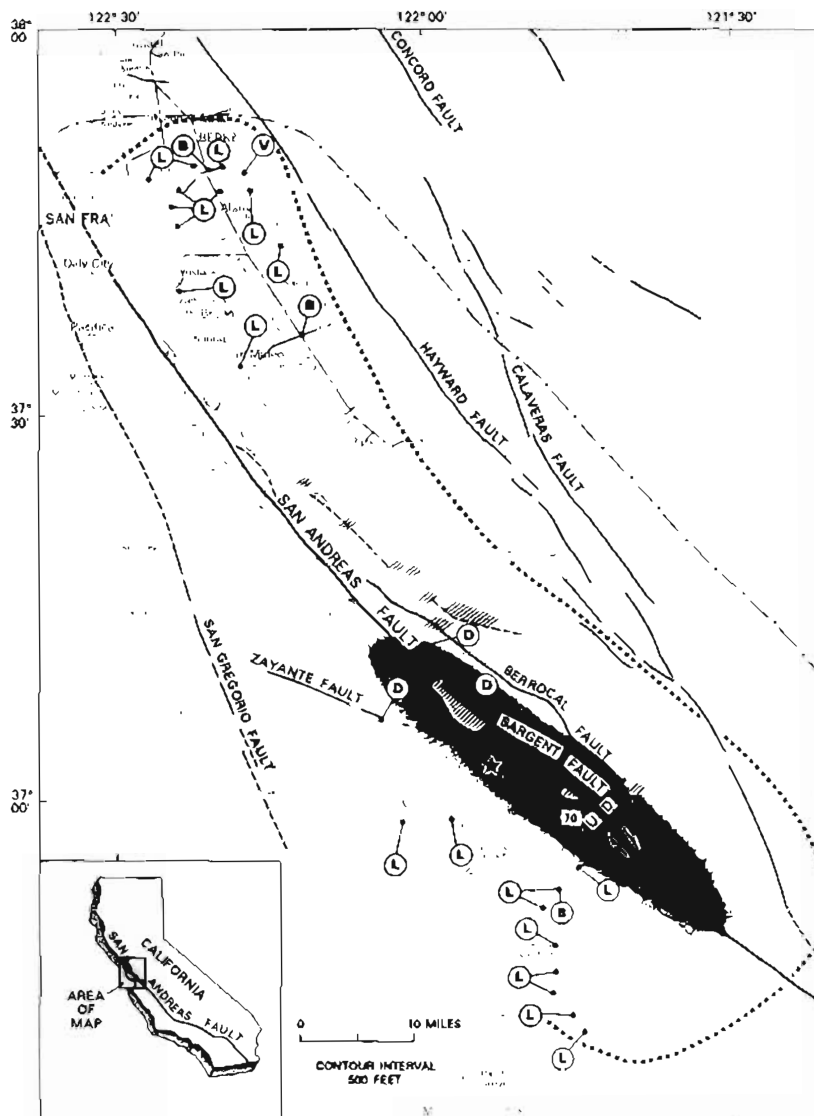
Chapter 6

Seismology and Ground Motion

The Loma Prieta earthquake of October 17, 1989 (5:04 P.M., Pacific daylight time) was the largest to occur in the San Francisco Bay area since the great earthquake of 1906.

The Loma Prieta earthquake was a result of rupture along a 25-mile-long (40 km.) segment of the San Andreas fault in the southern Santa Cruz Mountains, from the vicinity of Los Gatos southeast to the vicinity of Watsonville (Figure 6-1). The earthquake was named after the highest topographic point (3791 ft.) adjacent to the fault zone. The point of initial rupture, or hypocenter, was about 10 miles (16 km.) northeast of Santa Cruz at a depth of approximately 11.5 miles (19 km.). Maximum displacements on the fault, which dips to the southwest at an angle of about 70° were about 6 feet horizontally and 4 feet vertically; the southwestern



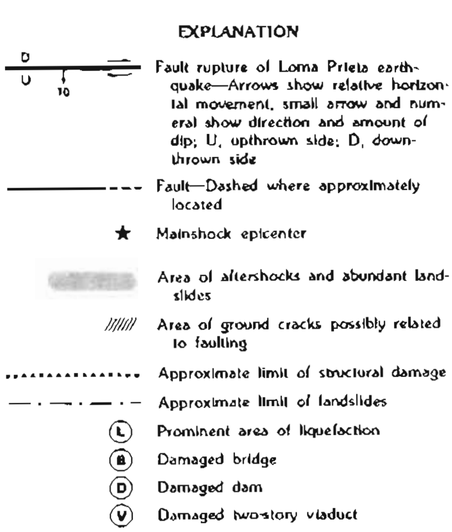


Figure 6-1. *Map of main shock epicenter, inferred fault rupture, area of aftershocks and abundant landslides, and approximate limits of structural damage [USGS, 1990].*

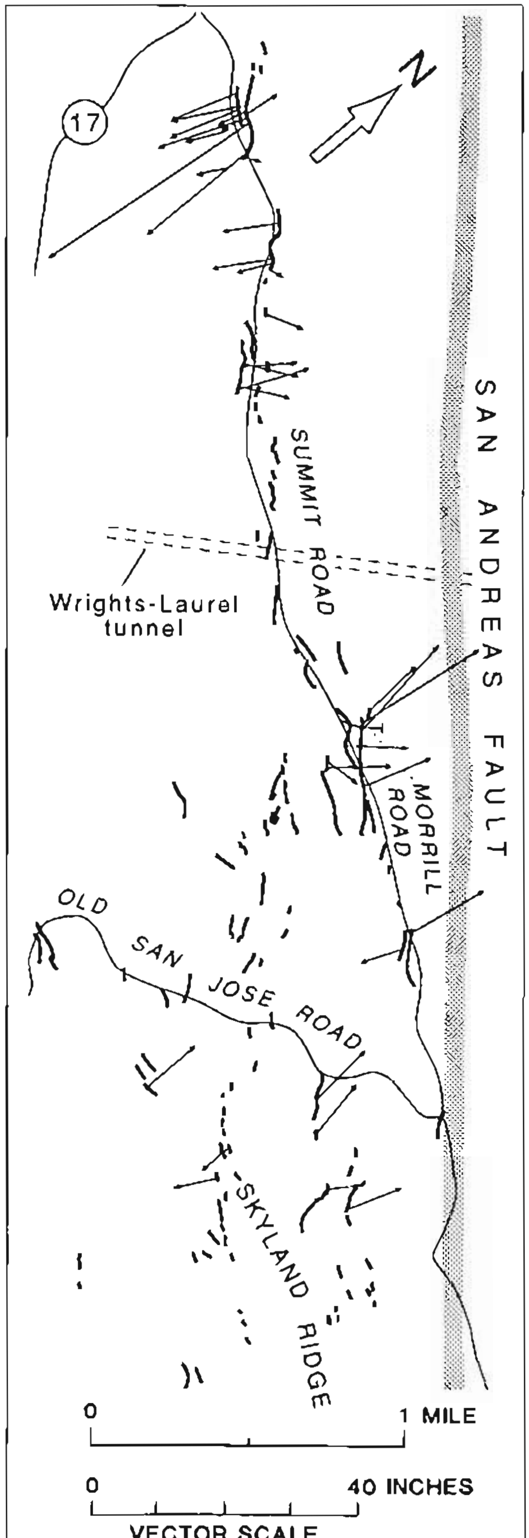


Figure 6-3. *Prominent surface cracks (heavy lines) and net-displacements (arrows) near Summit Road, Santa Cruz Mountains, not including cracks obviously related to landslides and local ground failure. The upper edge of the main fault rupture was about 3.7 mi (5.9 km) deep [from Plafker and Galloway, 1989]*

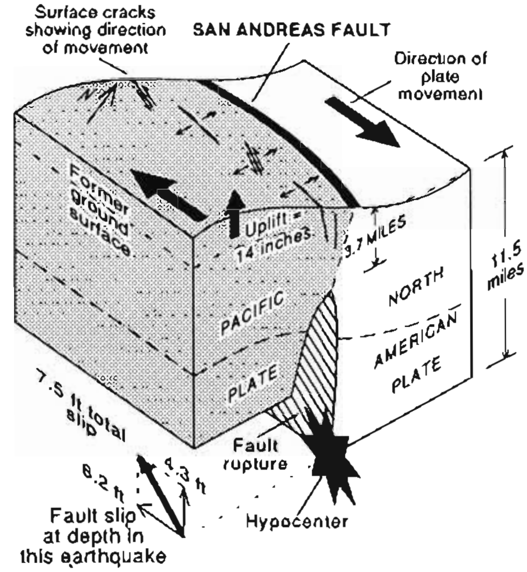


Figure 6-2. *Schematic diagram showing inferred motion on the San Andreas fault during the Loma Prieta earthquake [Pflafker and Galloway, 1989]*

side moved northwestward and upward with respect to the northeastern side. This Magnitude 7.1 earthquake was felt from Los Angeles north to the Oregon state line, and east to western Nevada [Pflafker and Galloway, 1989].

The fault rupture penetrated upward from the hypocenter to within about 3.7 miles (6 km.) of the ground surface, but did not break the ground surface; it was a blind rupture (Figure 6-2). Consequently a single, continuous trace of surface faulting was not found. Instead, a zone about 5 miles (8 km.) long and 2 miles (3.2 km.) wide along the fault zone contained numerous ground cracks, suggesting strain over a broad area (Figure 6-3). Both lithology and structures—for example, bedding planes—apparently controlled surface faulting [USGS, 1990]. Damage to both houses and roads was caused by these cracks, and very likely influenced where some landslides occurred [Pflafker and Galloway, 1989].

A Magnitude 5.0 aftershock occurred 33 hours after the main shock. Within 21 days, 87 aftershocks of Magnitude 3.0 or larger had occurred. The distribution of aftershocks defines the limits of ruptures that occurred on both the main fault plane and subsidiary fault planes (Figure 6-4). A pattern of aftershocks (Figure 6-4 cross section A-A') filled in a zone along the San Andreas fault that had been identified as a seismic gap before the earthquake (Figure 6-5) [Pflafker and Galloway, 1989].

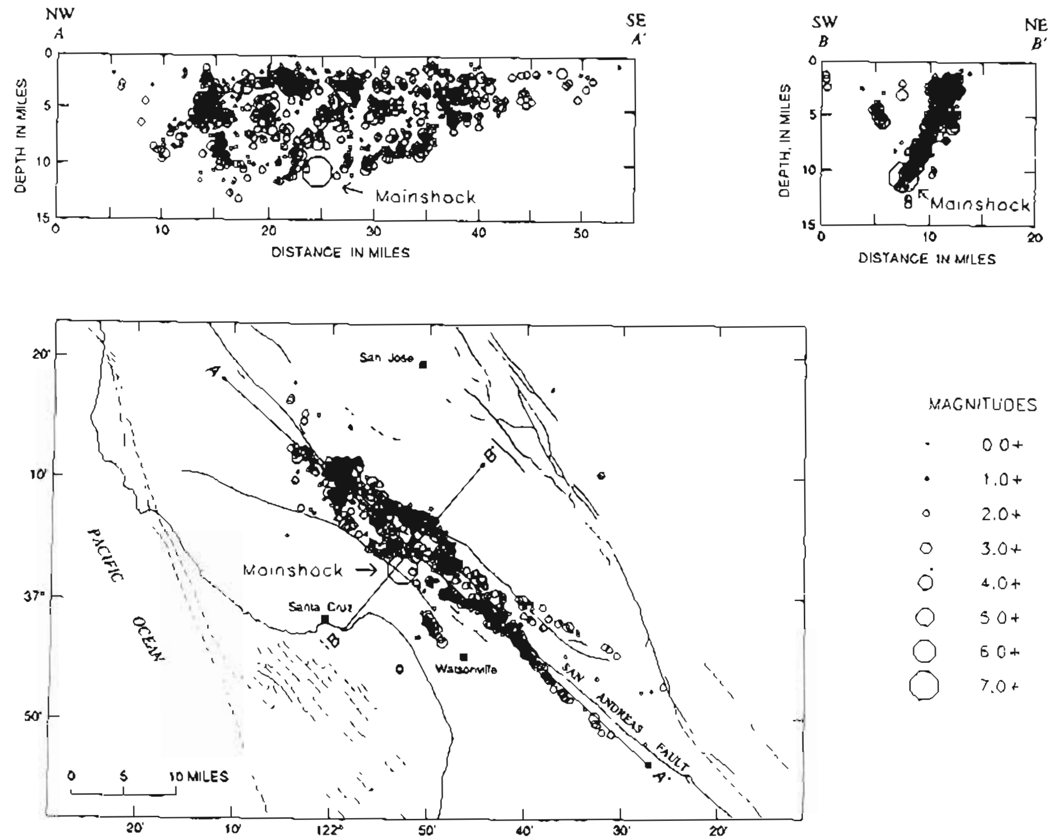


Figure 6-4. *Spatial distribution of aftershocks define rupture on both main fault plane and subsidiary structures. A) Section along fault B) Section transverse to fault showing dip of main fault plane and secondary faulting to the west. C) Map distribution of aftershocks showing complexity of fault zone involved [from Plafker and Galloway, 1989].*

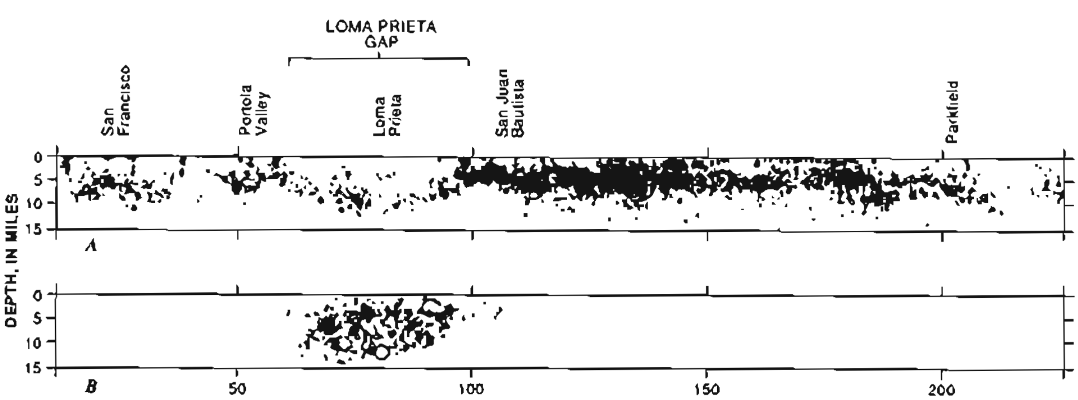


Figure 6-5. *Cross-section along the San Andreas fault from north of San Francisco to south of Parkfield. A) Background seismicity for 20 years prior to 1989 Dense activity south of San Juan Bautista is in creeping segment. North of San Juan Bautista the fault has been virtually aseismic since 1906. On the Loma Prieta segment the seismicity outlined a U-shaped area (Loma Prieta gap). B) Aftershocks of main shock (large circle) filled the former quiet zone of Loma Prieta gap [from Plafker and Galloway, 1989]*

The Loma Prieta earthquake was not unexpected, but occurred along a segment of the San Andreas fault previously recognized as having a high potential for an earthquake of Magnitude 6.5 to 7.0 [Dietrick, 1990].

Shaking intensity was VIII on the Modified Mercalli Intensity scale (MMI) over an area 30 mi (48 km) long and 15 miles (24 km) wide extending from Los Gatos to Watsonville and Santa Cruz (Figure 6-6). The zone of intensity VII extended 60+ mi (100 km) northwest to San Francisco and Oakland, and 30 mi (48 km) southeast to Salinas and Hollister. MMI VII is termed strong and is described by the types of effects observed: damage to weak unreinforced buildings, unreinforced masonry chimneys broken at roof lines; disruption of building contents; plaster cracked. MMI VIII is termed very strong shaking: damage to nonearthquake-resistant structures can be significant, with some collapses, particularly those in poor condition; damage to non-structural elements in modern, seismically-resistant buildings; and, substantial disruption of building contents and toppling of unanchored equipment. Within this region, free-field, peak horizontal accelerations of ground motion exceeded 0.6g close to the source and were as high as 0.26g at distances

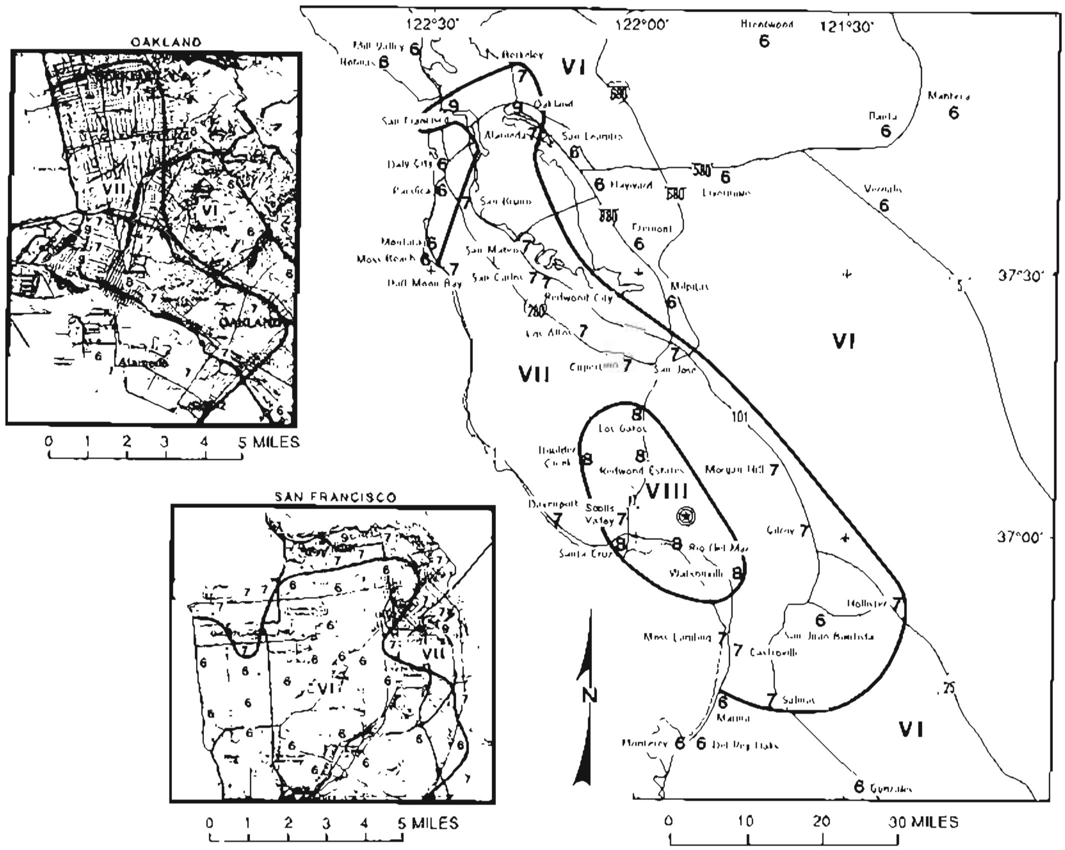


Figure 6-6. *Isoseismal map showing the distribution of Modified Mercalli Intensities for the Loma Prieta earthquake. Inserts show more detailed assessments in the cities of San Francisco and Oakland [from Plafker and Galloway, 1989]*

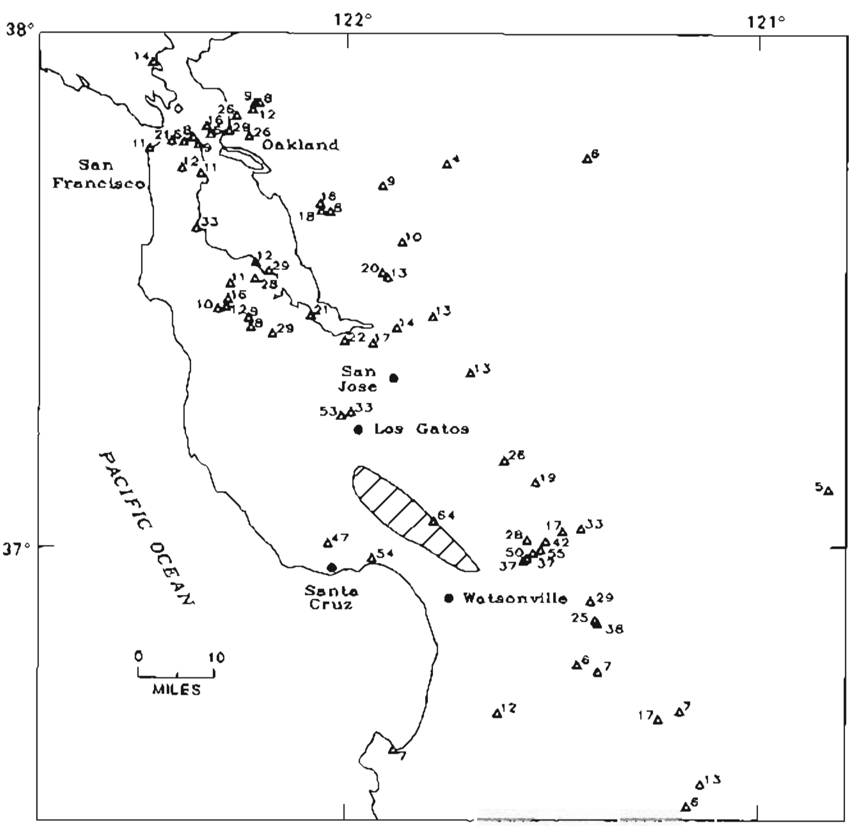


Figure 6-7. *Strong ground motions recorded during the Loma Prieta earthquake, expressed as percent of gravity. All are peak horizontal accelerations for free-field sites [from Plafker and Galloway, 1989].*

of 60 mi (96 km) (Figure 6-7) [Pflafker and Galloway, 1989; Shakal et al, 1989; Maley et al, 1989].

The nature of soil conditions, both in the epicentral region and in the Bay Area, played a very strong role in the damage observed and its distribution for this as well as for most California earthquakes. The ground motions in the Bay Area at soft soil sites, where much of the damage to bridges and viaducts occurred, were significantly greater than the motions recorded at nearby rock and stiff soil sites. *Civil Engineering Magazine* [March 1990] put it quite simply:

> Soil factors were the single most dominant issue in the Loma Prieta earthquake.

Liquefaction was widespread close to the source in Santa Cruz, Watsonville, and Moss Landing and in many areas in San Francisco, Treasure Island, Emeryville, and Oakland [EERI, 1990]. However, there is no evidence that liquefaction contributed to the failures at the Cypress Viaduct or the Bay Bridge span.

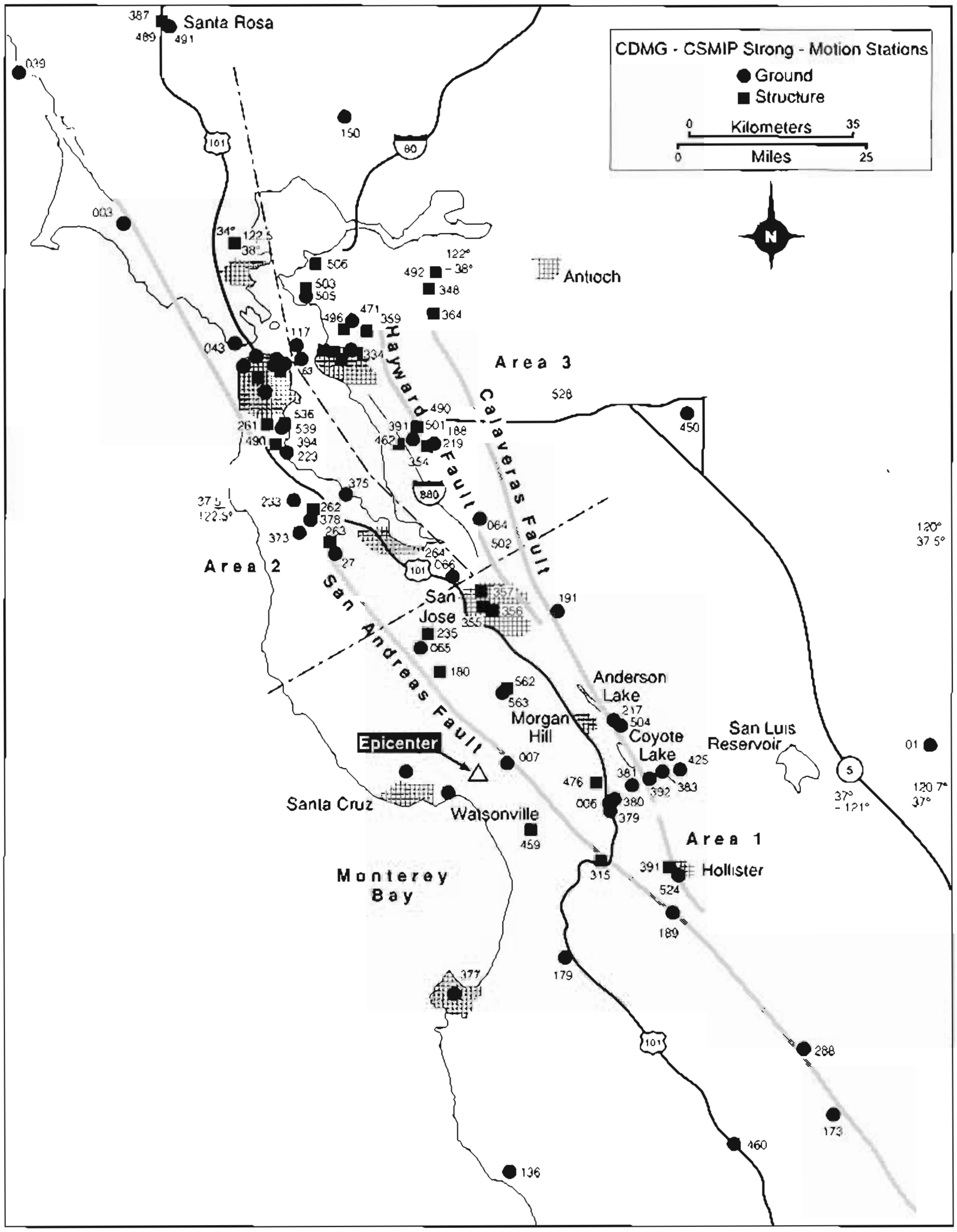


Figure 6-8. Locations of CSMIP stations that recorded the Loma Prieta earthquake [from Shakal et al., 1989]

Characteristics of Recorded Ground Motions

Recordings of earthquake ground motions were obtained at stations maintained by the California Strong Motion Instrumentation Program (CSMIP) [Shakal et al., 1989; CSMIP, 1989; and Huang et al., 1990] and at stations maintained by the U.S. Geological Survey (USGS) [Maley et al., 1989]. Figure 6-8 shows the locations of the CSMIP stations; USGS has 38 additional stations located over the same basic area.

Strong motion records were obtained during the main shock of the Loma Prieta earthquake at 98 free-field stations. Such instruments are located either in instrument shelters or on the ground floor of one- to three-story buildings that have no basements. Thirty-three of these recordings were at rock sites, ten were at soft soil sites and the remaining 55 recordings were on other soil sites. A soft soil site, as used in this Report, is defined as a site underlain by several feet to several tens of feet of young bay mud. Records were also obtained in basements of large buildings, at various floors of buildings, and at dams. However, none were obtained at any of the damaged bridges or viaducts.

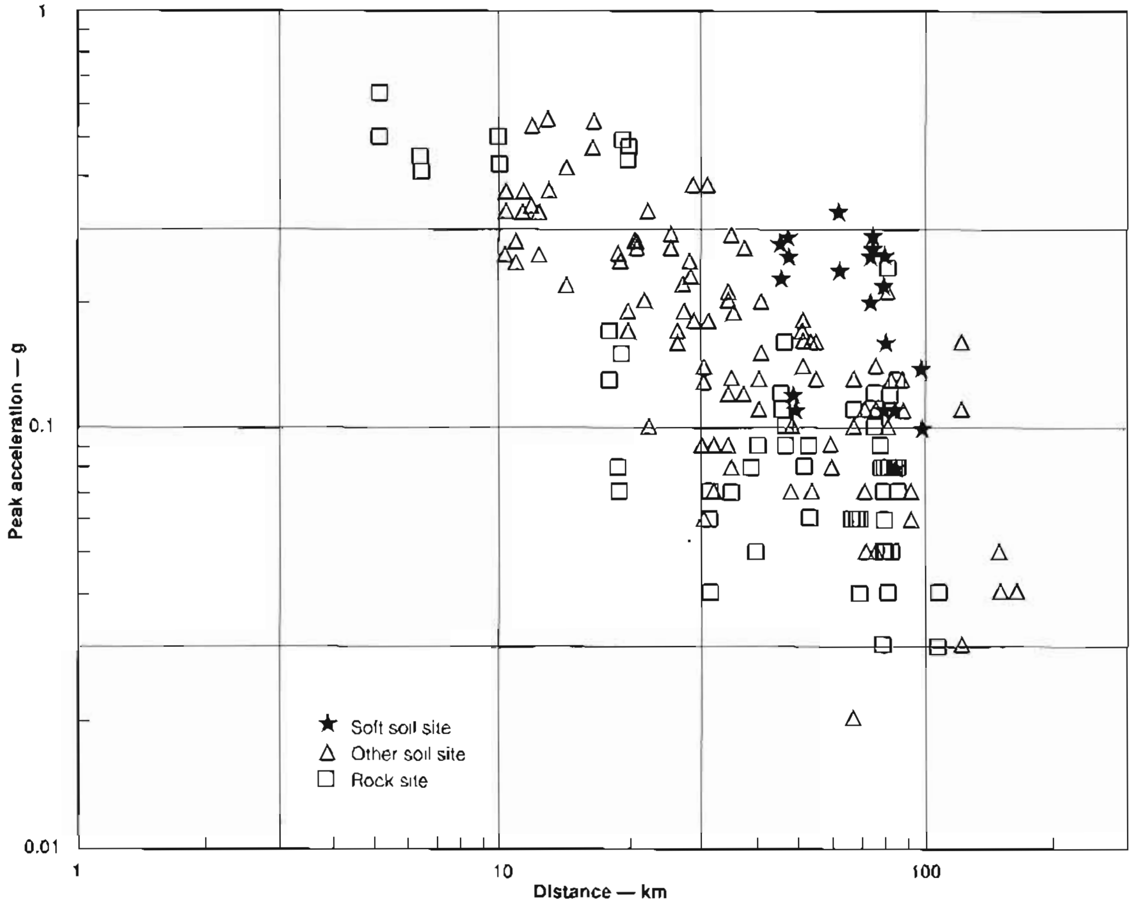


Figure 6-9. *Peak horizontal accelerations for free-field sites recorded during the Loma Prieta earthquake for rock, soil and soft soil sites.*

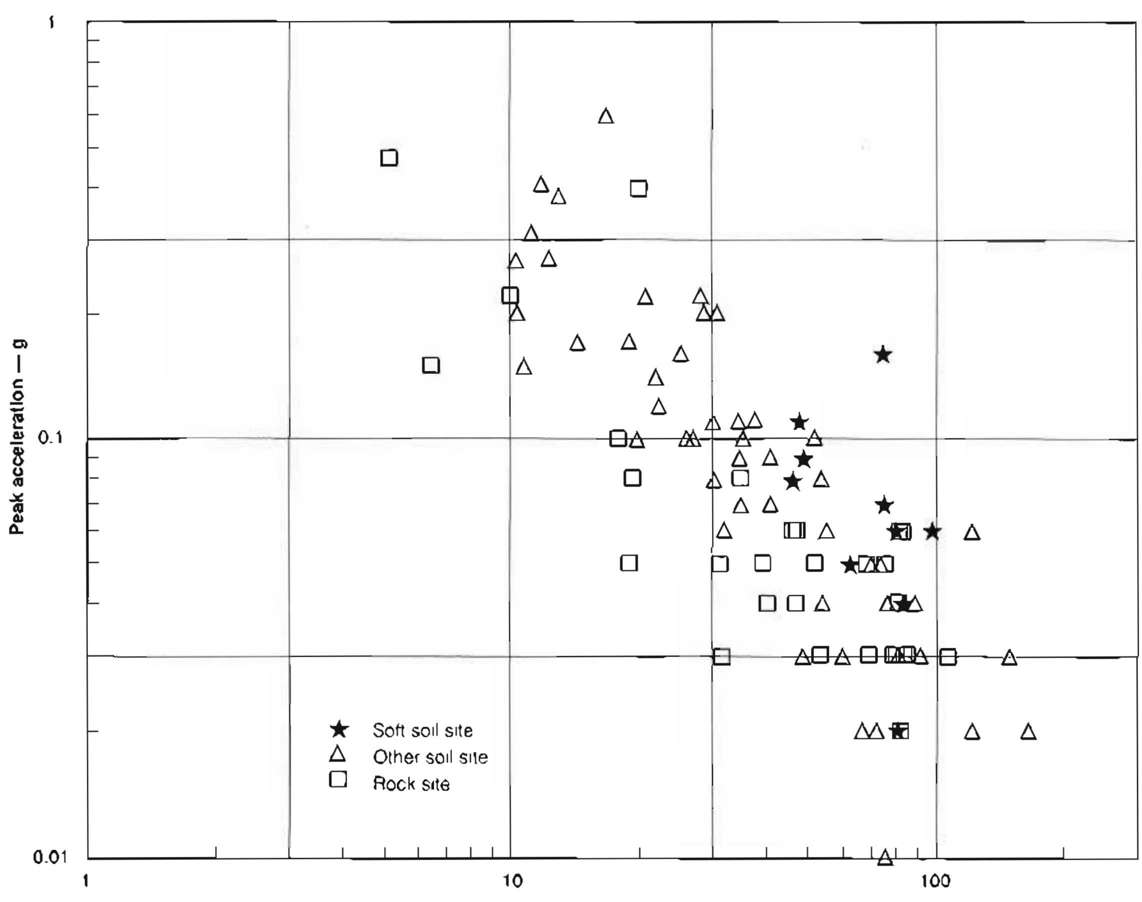


Figure 6-10. *Peak vertical accelerations for free-field sites recorded during the Loma Prieta earthquake for rock, soil and soft soil sites.*

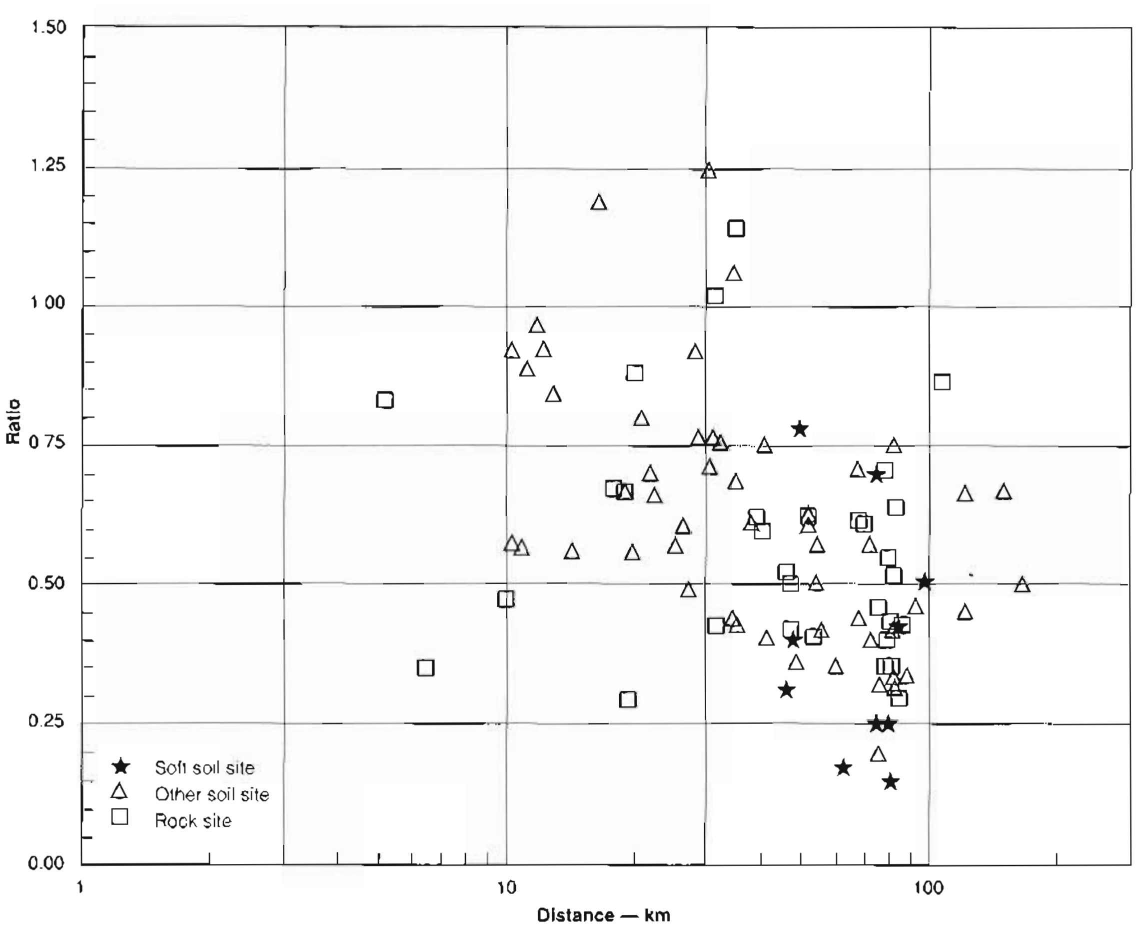


Figure 6-11. *Ratios of recorded peak vertical to horizontal accelerations for free-field sites recorded during the Loma Prieta earthquake for rock, soil and soft soil sites.*

The peak horizontal accelerations recorded at the "free-field" stations are presented in Figure 6-9. The corresponding peak vertical accelerations and the ratio of peak vertical acceleration divided by the peak horizontal acceleration are presented in Figures 6-10 and 6-11, respectively.

A reasonable question to ask is, "How well could the peak horizontal accelerations at rock and at soil sites, have been estimated using available attenuation relations?" Boore et al. [1990] showed that the equations originally developed by Joyner and Boore in 1981, on the average for all distances, underestimated the recorded motions on rock by about 40% percent and those on soils (other than soft soils) by about 60% percent. Other relations, such as those developed by Seed and Idriss [1982] provide, on the average, a closer estimate with those recorded during this earthquake (Figures 6-12). Figure 6-13 shows the residuals when the observed values are compared to those predicted by the Seed and Idriss attenuation relationship. Other available attenuation relationships (e.g. those summarized by Joyner and Boore [1988]) provide estimates that are comparable to those shown in the Figures 6-12 and 6-13. No generally applicable attenuation relationships have been developed for soft soil sites to date. The recorded peak horizontal accelerations at the soft soil sites are 2 to 3 times larger than those at the other sites.

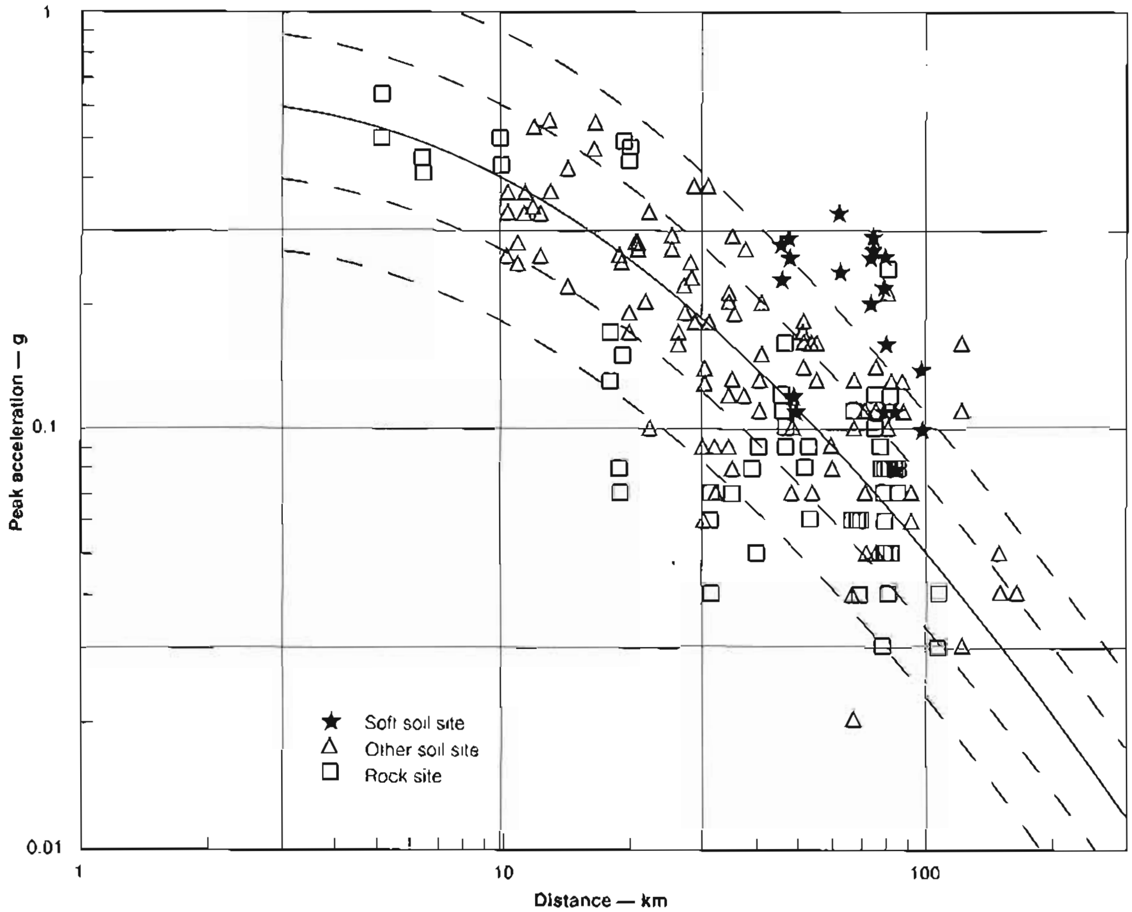


Figure 6-12. *Recorded accelerations and those calculated (median ± 2 s) using the relationships of Seed and Idriss [1982].*

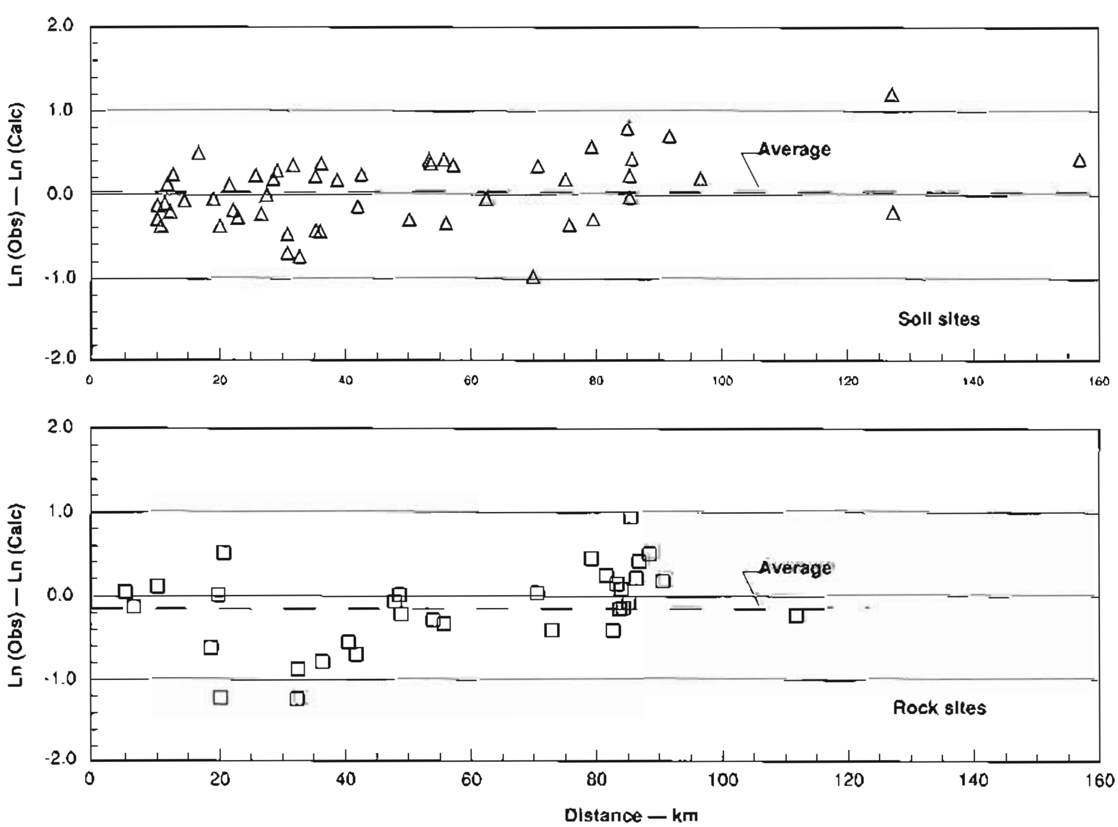


Figure 6-13. *Residuals between measured and computed horizontal accelerations using the attenuation relationship of Seed and Idriss [1982]*

Table 6-1. *Earthquake ground motion stations. Five-digit numbers are for stations operated by the CSMIP and four-digit numbers refer to stations operated by the USGS. Distance used herein is the closest distance from the recording station to the rupture surface at a depth of 5 km below the ground surface [Shakal et al., 1989; Maley et al., 1989].*

Station	Station Number	Distance (km)
Rock sites close to the source		
Corralitos	57007	5
Gilroy No. 1	47379	10
UC Santa Cruz	58135	20
Rock sites in San Francisco		
Diamond Heights	58130	76
Rincon Hill	58151	79
Pacific Heights	58131	80
Telegraph Hill	58133	81
Golden Gate	1678	82
Cliff House	58132	83
Soft soil sites south of San Francisco		
APEEL No. 2	1002	47
Foster City	58375	48
SF Airport	58223	64
Soft soil sites in Emeryville and in Oakland		
Emeryville	1662	81
2-story building	58224	76
Adjacent rock and soft soil sites		
Yerba Buena (rock site)	58163	78
Treasure Island (soil site)	58117	81

Selected Strong Motion Records

Twenty-eight of the free-field records have been fully processed and digitized at the time of this writing. A selected number of these recordings are reproduced to show the characteristics of the recorded motions at rock sites near the source and in San Francisco and at soft soil sites. The earthquake ground motion stations examined in this section are listed in Table 6-1.

The Corralitos, Gilroy No. 1 and the Santa Cruz stations are at rock sites close to the earthquake source. The Diamond Heights, Rincon Hill, Pacific Heights, Telegraph Hill, Cliff House, and Golden Gate stations are at rock sites close to the higher damage areas in San Francisco.

The APEEL #2 station in Redwood City and the stations in Foster City and at the San Francisco Airport are at soft soil sites. Damage in the vicinity of these recording stations was minimal during this earthquake. Nevertheless, it is of value to examine the characteristics of these recordings. The recording stations at Treasure Island, in Emeryville, and the other two stations in Oakland were each close to the areas of structural damage and significant liquefaction. The Yerba Buena Island site is a rock site close to and just south of the soft soil Treasure Island site, where significant liquefaction took place.

The soft soil sites listed in Table 6-1 generally consist of sandy fill underlain by young bay mud underlain by layers of dense to very dense sands and stiff to hard clays. The thickness of the fill ranges from about 10' to as much as 40'. The fill at most of these locations is typically loose to medium dense. The young bay mud ranges in thickness from about 5' to as much as 80', and at these depths is typically normally consolidated to lightly over-consolidated. Shear wave velocities in the young bay mud range from about 150'/sec at the top of the layer to about 500'/sec at the bottom of the thicker layers. Rock is encountered at depths ranging from about 300' to 500' at these soft soil sites.

Accelerations, Velocities, and Displacements

The accelerograms for the east-west (or near east-west) components of the recordings obtained at some of these stations are shown in Figure 6-14 (Corralitos, Gilroy No.1 and Santa Cruz), Figure 6-15 (rock sites in San

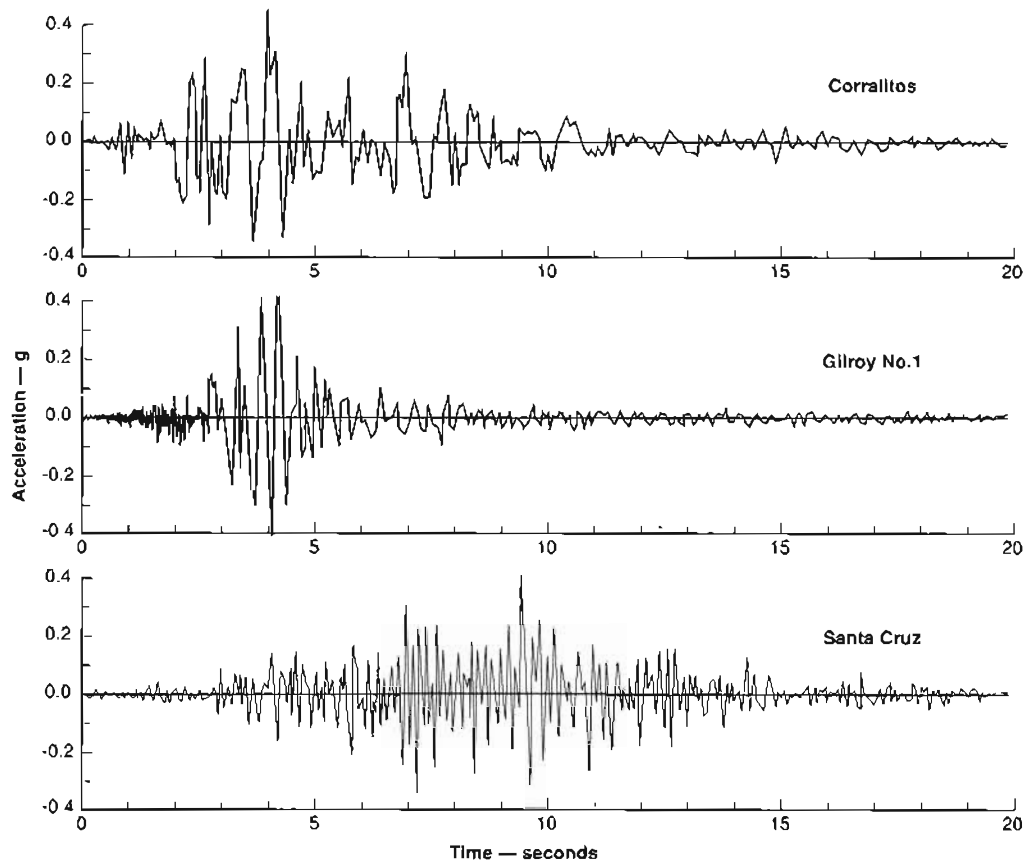


Figure 6-14. *Accelerograms of east-west components of motions recorded at rock sites within 20 km of the source.*

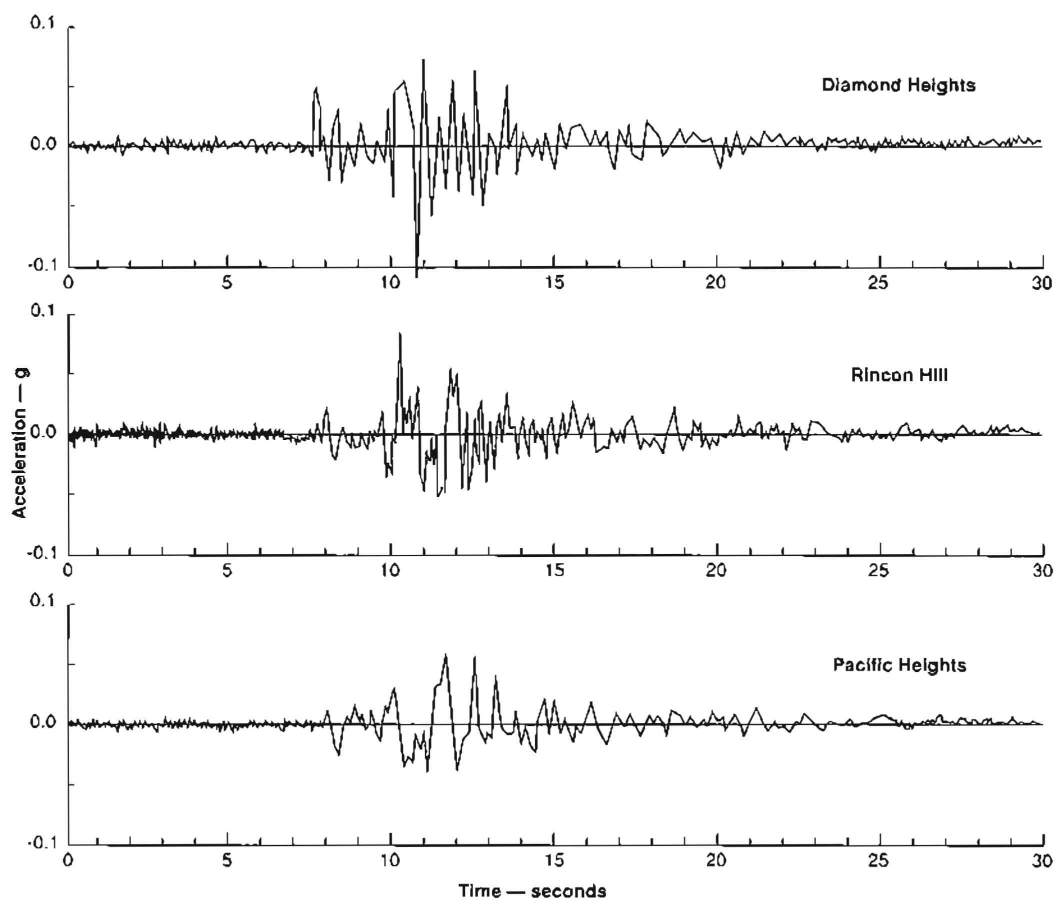


Figure 6-15. *Accelerograms of east-west components of motions recorded at rock sites in San Francisco.*

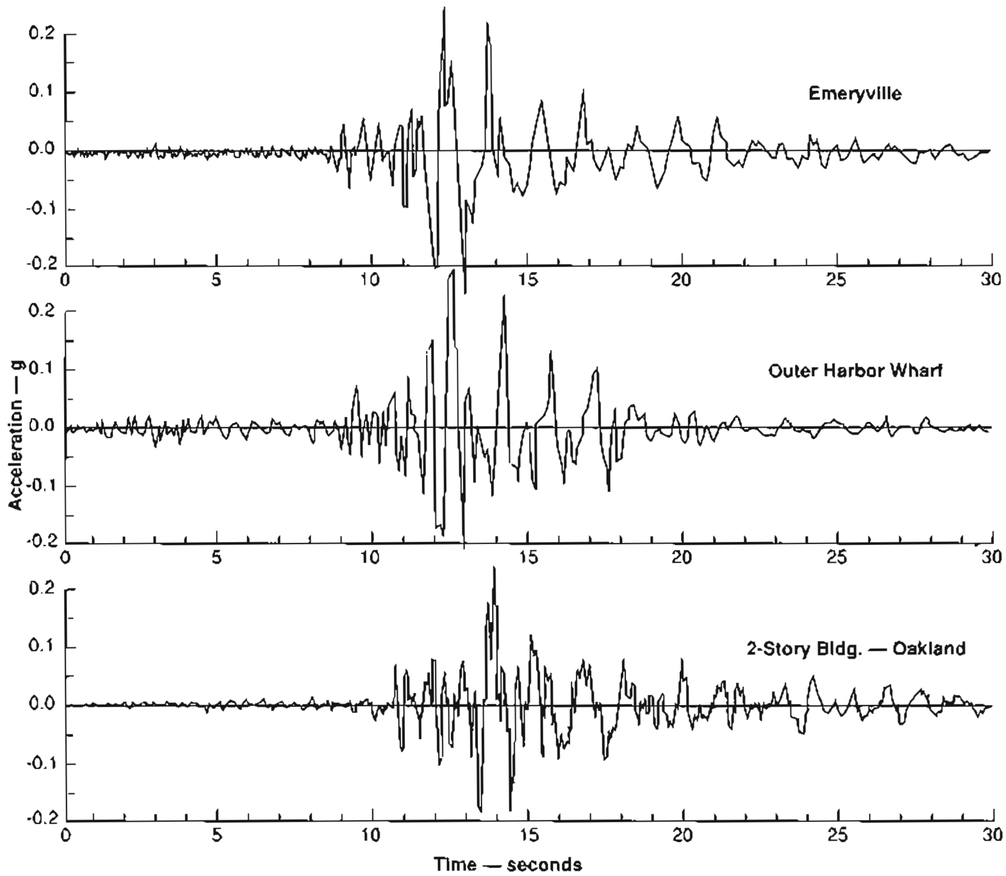


Figure 6-16. *Accelerograms of east-west components of motions recorded in Emeryville and in Oakland.*

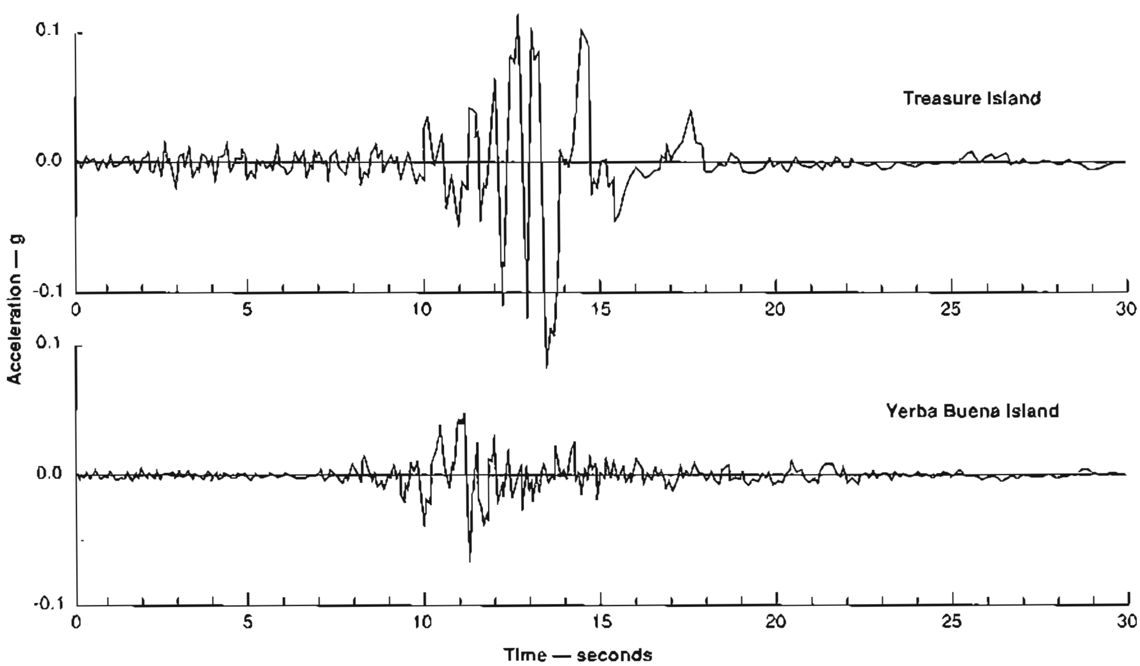


Figure 6-17. *Accelerograms of east-west components of motions recorded at Treasure Island and Yerba Buena Island.*

Francisco), Figure 6-16 (Emeryville and at the two soft soil sites in Oakland), and Figure 6-16 (Yerba Buena and Treasure Island).

The peak horizontal and vertical accelerations, velocities, and displacements of the motions recorded at the stations listed in Table 6-1 are presented in Table 6-2. The peak accelerations at rock sites both near the source and those at distances of about 80 km. are within the expected range for these values as predicted by common attenuation relationships. The ratio of peak horizontal velocity, v, divided by peak horizontal acceleration, a, for the rock sites near the

Table 6-2. *Peak accelerations, velocities and displacements for selected recorded ground motions. The peak accelerations listed in this table are "Volume II" accelerations*

Station	Distance (km)	Component	Peak Acceleration	Peak Velocity (cm/sec)	Peak Displacement (cm)
Corralitos	5	EW	.478	47.5	11.5
		NS	.629	55.2	9.5
		VERT	.439	18.6	7.77
Gilroy No. 1	10	EW	.442	33.8	6.32
		NS	.435	31.9	6.49
		VERT	.210	14.5	5.15
UC Santa Cruz	20	EW	.409	21.2	6.81
		NS	.441	21.2	6.61
		VERT	.331	12.0	6.72
Diamond Heights	76	EW	.113	14.3	4.31
		NS	.098	10.5	2.83
		VERT	.043	6.68	1.36
Rincon Hill	79	EW	.090	11.6	4.88
		NS	.080	7.34	2.62
		VERT	.029	3.97	1.85
Pacific Heights	80	EW	.061	14.3	4.88
		NS	.047	9.88	3.08
		VERT	.031	5.93	2.25
Telegraph Hill	81	EW	.092	9.59	2.76
		NS	.052	6.50	1.43
		VERT	.033	3.30	1.91
Golden Gate	82	EW	.243	35.5	7.42
		NS	.126	18.0	3.86
		VERT	.059	11.6	2.56
Cliff House	83	EW	.108	21.0	6.4
		NS	.075	11.2	3.70
		VERT	.062	7.51	1.57
APEEL No. 2	47	133	.227	35.9	5.68
		043	.277	53.1	10.4
		VERT	.086	7.79	1.06
Foster City	48	EW	.283	45.4	14.7
		NS	.257	31.8	6.28
		VERT	.103	8.38	3.57
SF Airport	64	EW	.332	29.3	5.92
		NS	.235	26.5	5.05
		VERT	.065	5.27	1.77
Emeryville	81	EW	.260	41.1	8.21
		NS	.214	21.5	3.75
		VERT	.060	5.00	.80
2-Story Building, Oakland	76	290	.243	37.9	8.05
		200	.191	20.0	3.92
		VERT	.144	6.19	1.30
Outer Harbor Wharf, Oakland	76	305	.271	42.3	9.17
		035	.287	40.8	9.88
		VERT	.066	10.5	1.83
Yerba Buena Island	79	EW	.067	14.7	4.12
		NS	.029	4.61	1.39
		VERT	.028	4.22	1.13
Treasure Island	81	EW	.159	33.4	12.2
		NS	.100	15.6	4.48
		VERT	.016	1.16	1.15

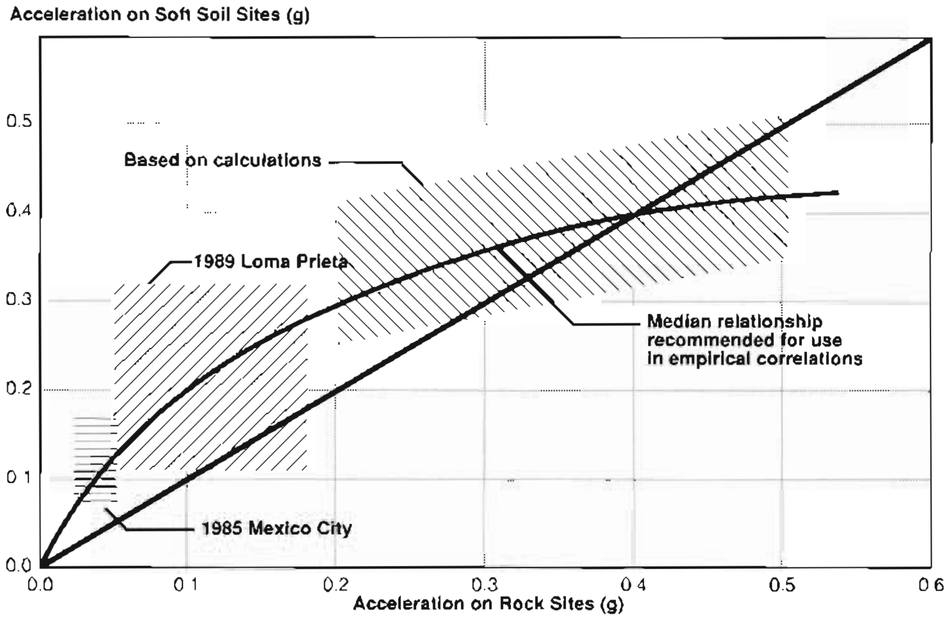


Figure 6-18. *Variation of accelerations on soft soil sites vs. rock sites [Idriss, 1990].*

source range from about 50 to 100 cm/sec/g with an average of about 60 cm/sec/g and ratio ad/v^2 (in which d is the peak displacement) for these rock sites ranges from about 2 to 6 with an average of about 3.5. These ratios are also within the expected range.

The peak horizontal accelerations at the soft soil sites are significantly higher than those at the adjacent rock sites. These trends are being examined by many researchers. For example, Jacob et al. [1990] shows a significant amplification of motion at soft sites compared to nearby sites not underlain by young bay mud during several aftershocks. Similar trends were reported by Jarpe et al. [1990] based on recordings at Yerba Buena and at Treasure Island during aftershocks. Idriss [1990] presented the results shown in Figure 6-18, which indicate that horizontal accelerations are amplified on soft soils for peak accelerations in the underlying rock less than about 0.4g. At higher levels of rock acceleration, he suggests a deamplification of the peak accelerations in the soft soils owing to nonlinear behavior. At very low levels of peak horizontal rock accelerations (say less than about 0.01g), the amplification can be of the order of 6 to 10, as evidenced by the recordings from aftershocks [Jacob, 1990; Jarpe et al., 1990]. Data from Mexico City in the 1985 Mexico earthquake for somewhat lower rock accelerations (.03g to .05g) show amplifications of soft soil accelerations of the order of 3 to 5. The results from the main shock of Loma Prieta indicate amplification ratios of the order of 2 to 3 as illustrated in Figure 6-18.

Spectral Ordinates

Frequency characteristics of earthquake ground motions are revealed by plots of spectral ordinates versus frequency. Spectral ordinates of the horizontal components of the motions recorded on rock sites at Corralitos, Gilroy No. 1 and at UC Santa Cruz are shown in Figure 6-19. Peak spectral ordinates of the motions at UC Santa Cruz occur at shorter periods than those of the other two motions. In addition, the motions at Corralitos seem to have higher spectral ordinates than the other two motions at periods of about 0.7 to about 1.1 sec. For the purposes of comparing these "near-source" motions to those of rock motions recorded in San Francisco and to the frequency content of the motions recorded at the soft sites, it is reasonable to average the spectral ordinates presented in Figure 6-19 to obtain an "average spectral curve" for motions recorded on rock within 20 km. of the source. This average is also shown in Figure 6-19. Figure 6-20 shows similar plots for the motions recorded at soft soil sites in Emeryville and in Oakland. Again, there are differences in the details; the average of the spectral ordinates is also shown in Figure 6-20. Similar averages were also obtained for the recordings at soft soil sites south of San Francisco and at rock sites in San Francisco (see Table 6-1 for the recordings included in each group).

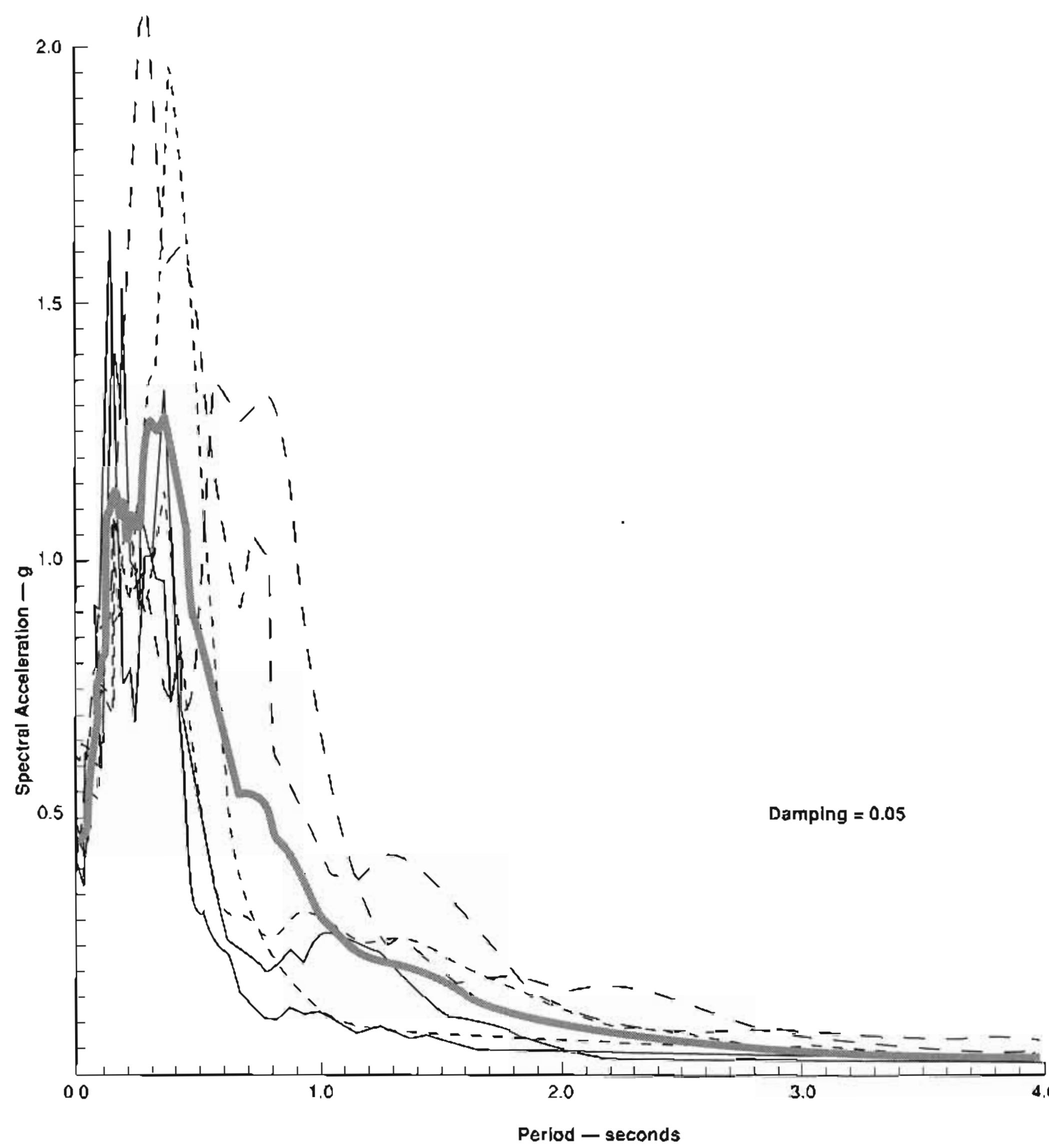


Figure 6-19. *Spectral ordinates for motions recorded within 20 km of the source at U C Santa Cruz (solid line), at Gilroy (dotted), Corralitos (dashed), and average for these records (heavy).*

Figure 6-21 presents the average spectral ordinates for the motions recorded at rock sites near the source, at rock sites in San Francisco, at soft soil sites south of San Francisco and at soft soil sites in Emeryville and Oakland. The average spectrum for near-source rock motions has a peak at a period of about 0.4 sec., while the average spectrum for rock motions in San Francisco is significantly flatter over a period range of about 0.3 to about 0.9 sec. The average spectral ordinates for the soft sites south of San Francisco are almost identical to those for the soft soil sites in the East Bay. This suggests that soft soil sites in the San Francisco-Oakland region, including the site where the Cypress Viaduct was damaged, and the Bay Bridge, experienced, on the average, similar levels of shaking during the main shock. It is interesting to note from Figure 6-21 that on a spectral basis, soft soil values are from 2 to 4 times those of rock in the region for frequency ranges of primary engineering importance. However, for some individual recordings and frequency ranges, this ratio was observed to be much larger, of the order of 10 for narrow frequency ranges [Borcherdt, personal communication, April 28, 1990].

It is also of interest to examine the spectral shapes (i.e., spectral ordinates nor-

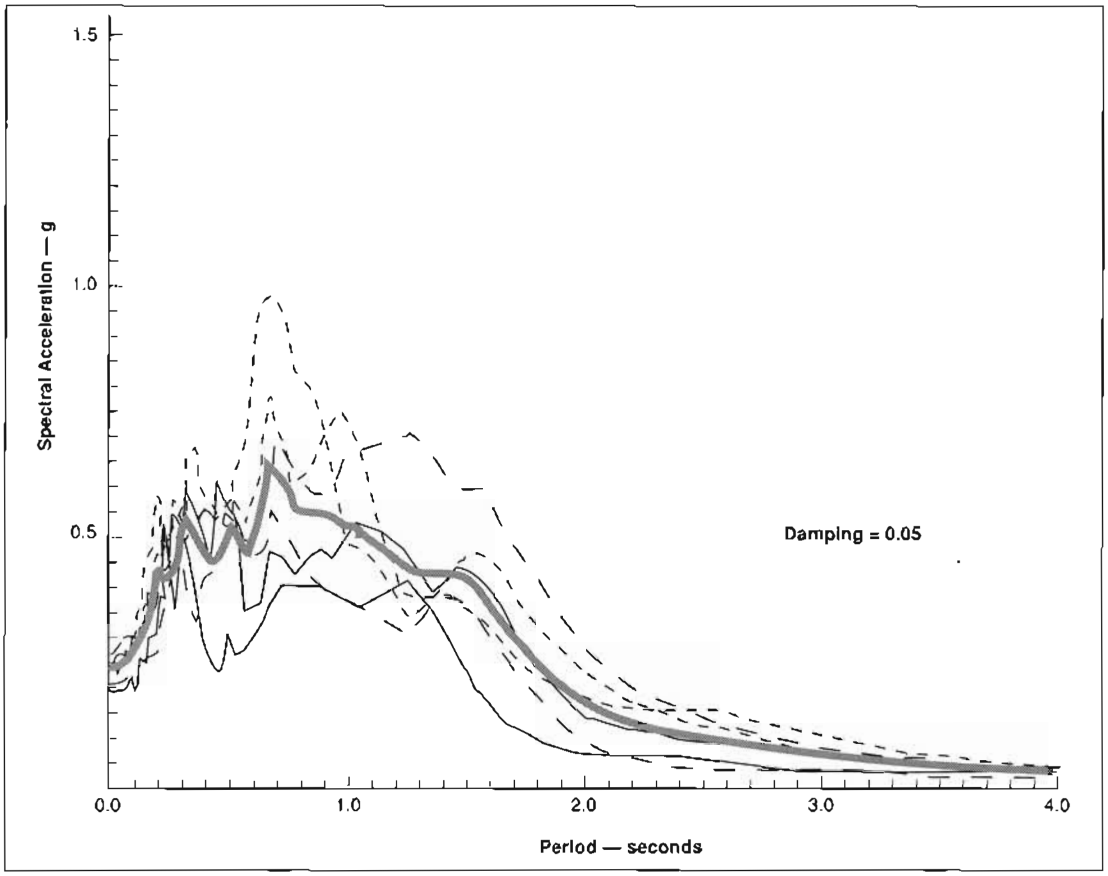


Figure 6-20. *Spectral ordinates for motions recorded at soft sites at Oakland 2-story building (solid line), at the Oakland Outer Harbor Wharf (dotted), Emeryville (dashed), and average for these records (heavy).*

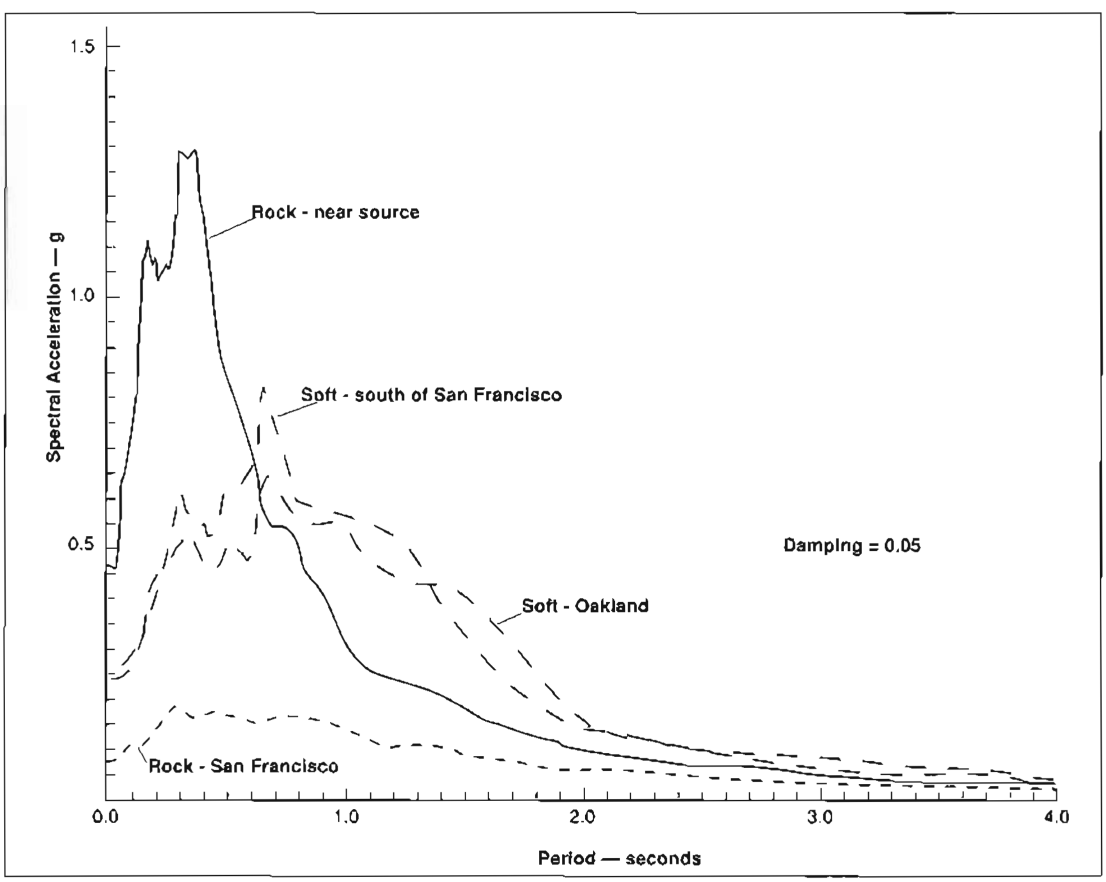


Figure 6-21. *Average spectral ordinates for motions recorded at rock sites and soft soil sites.*

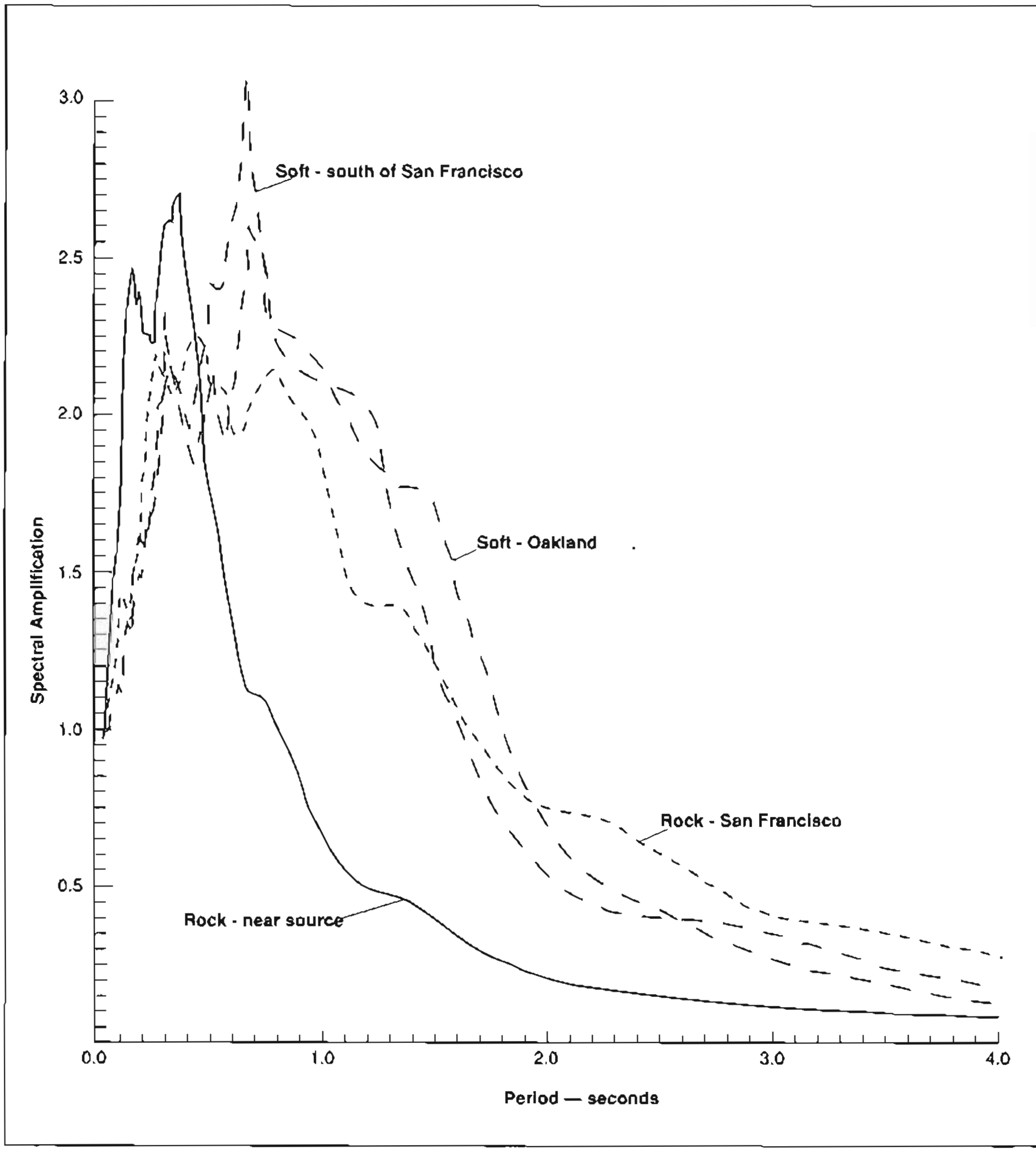


Figure 6-22. *Average normalized spectral shapes for motions recorded at rock sites and soft soil sites*

malized with respect to the peak acceleration associated with each average spectrum) of the spectra presented in Figure 6-21. The normalized spectral shapes are shown in Figure 6-22. The differences and the trends of the two average rock spectra and for the two soft soil spectra are as noted above for the corresponding absolute spectral ordinates. It is of particular interest to note that the frequency content of the rock motions in San Francisco is very similar to that of the two spectral shapes for the soft soil sites. This may have contributed to the increased amplification of motions at these soft soil sites during this earthquake. On the other hand, had there been soft soil sites in the vicinity of the source, the frequency content of the rock motions is unlikely to have been as much "in compliance" with those of the soft soil sites as it was for the distant motions during this earthquake. The level of shaking at soft soil sites closer to the source is very likely to be higher than that experienced at the Bay Area soft soil sites on October 17, but not in the same proportion to the ratios of the rock motions shown in Figure 6-21. The trend is more likely to be as suggested in Figure 6-18.

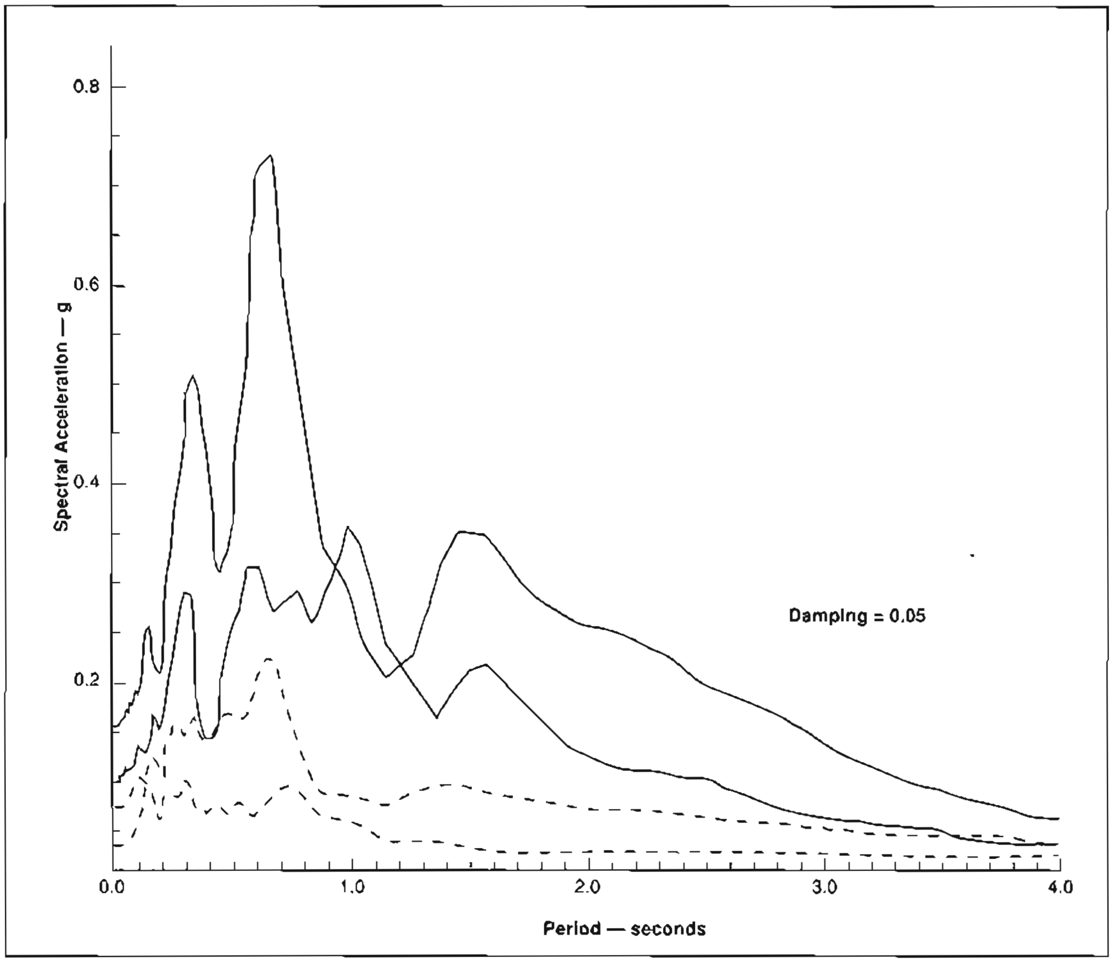


Figure 6-23. *Spectral ordinates for motions recorded at Treasure Island (solid) and Yerba Buena Island (dotted).*

Motions Recorded at Treasure and at Yerba Buena Islands

These adjoining islands are located in the middle of the San Francisco Bay (see Figure 9-4). Treasure Island is a man-made island that was formed in the 1920s by hydraulically placing sand over the young bay mud deposits that were underwater at the time. Yerba Buena Island is a natural island consisting mostly of rock outcrops; in fact, the suspension part of the Bay Bridge terminates at the west side of this Island and the truss part of the bridge terminates at the east side. Strong motion instruments had been placed by the CSMIP at one location in Yerba Buena Island and at one location in the middle of Treasure Island. Both instruments recorded the main shock on October 17; additional recordings were also obtained at both stations during a number of aftershocks [Jarpe et al, 1990].

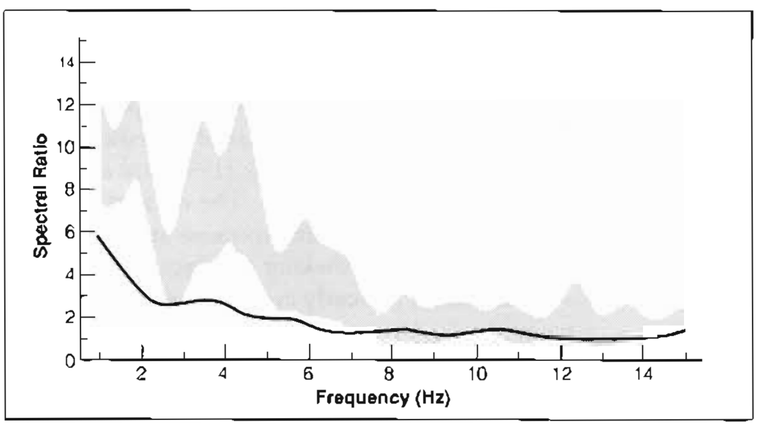


Figure 6-24. *Relative strong- and weak-motion site response of the Treasure Island and Yerba Buena Island stations. The shaded band is the 95% confidence region for the N-S- spectral ratio of the Treasure Island to Yerba Buena Island averaged for seven Loma Prieta aftershocks, and the thin line shows the N-S spectral ratio for the first five seconds (before apparent liquefaction) of the S-wave of the main shock [Jarpe et al., 1990].*

The recordings at Treasure Island and at Yerba Buena Island provide additional insight into the comparative behavior of soft soil sites and rock sites. The east-west accelerograms of the motions recorded during the main shock are shown in Figure 6-17, and spectral ordinates for the horizontal components at both islands are shown in Figure 6-23. The trends are similar to those shown in Figure 6-21 for the rock sites in San Francisco and for the soft soil sites. The ratio of the spectral ordinates recorded at

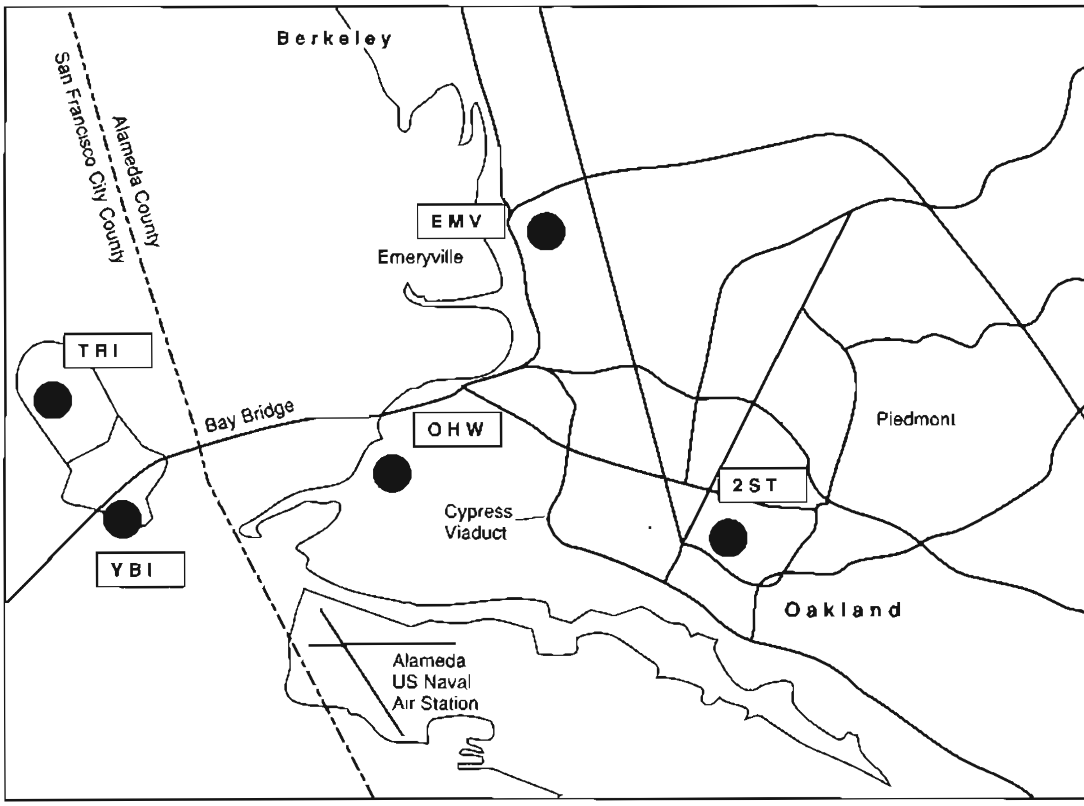


Figure 6-25. *Map showing the location of selected strong-motion recording sites in the San Francisco - Oakland area—Emeryville (EMV), Oakland, Outer Harbor Wharf (OHW), two-story building in Oakland (2ST), Treasure Island (TRI), Yerba Buena Island (YBI).*

Treasure Island divided by those recorded at Yerba Buena are shown in Figure 6-24 (Jarpe et al., 1990). The ratio of corresponding ordinates for the number of aftershocks examined by Jarpe et al. [1990] are also shown in Figure 6-24. The decrease in amplification of ground motions at soft soil sites as the level of shaking at adjacent rock sites increases is clearly evident from this figure.

Motions Near the Bay Bridge and Cypress Viaduct

Strong motion data on the Bay Bridge were not recorded for the earthquake or for major aftershocks. However, some ground motions recorded within a few kilometers of the bridge give indirect information on what the ground motions may have been like at the bridge site. Testimony presented to the Board on strong ground motion analyses suggests that significant differential displacement probably occurred between the west end of the Bay Bridge on the bedrock of Yerba Buena Island and the east end of the bridge on soft sediments [Hanks, testimony Jan. 4, 1990].

Hanks compared displacements oriented 79° east of north, approximately parallel to the eastern section of the Bay Bridge, as derived from strong motion recordings at sites on Yerba Buena Island and the Outer Harbor Wharf (about 1 km. SE of the east end of the bridge) (Figure 6-25). The displacement records were synchronized by shifting the arrival times of the S waves. The differential displacements were then determined by subtracting the motion recorded at the Yerba Buena site from that recorded at the Outer Harbor Wharf site. The result shows relative extension of 11.06 cm (4.34 in.) and about 6 cm (2.4 in.) relative shortening between the two sites; see Chapter 9, Figure 9-18 for more discussion. Similar ground motions were recorded at both the downtown Oakland site, about 6 km (4 mi) east-southeast of the east end of the bridge, and at the Emeryville recording site, about 3 km (1.8 mi) northeast of the east end of the bridge, so that it is likely that the record at the Outer Harbor Wharf site is a close representation of ground motion at the east end of the bridge. It may be noted, however, that these relative displacements were significantly smaller (5 or more times) than

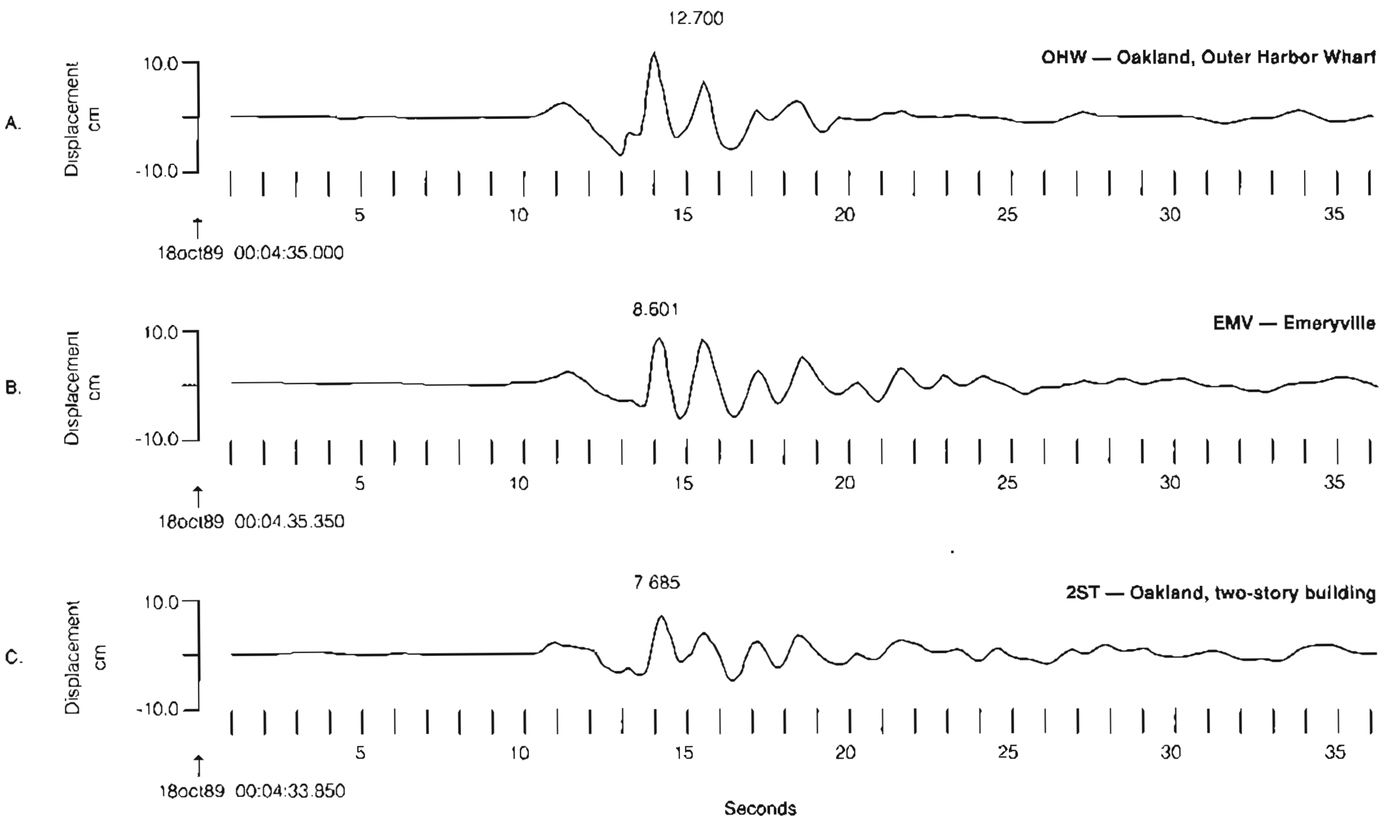


Figure 6-26. *Displacements determined from strong motion records for three sites surrounding the collapsed portion of the Cypress Viaduct on I-880 show remarkable coherence. (A) Oakland, Outer Harbor Wharf, (B) Emeryville; (C) Oakland, ground floor of 2-story building. See Figure 6-25 for locations of sites.*

the maximum thermal expansions for which the bridge was designed.

As noted earlier, it is estimated that all soft soil sites in San Francisco and the East Bay experienced, on average, similar levels of shaking during the main shock. This observation is further supported by the strong motion records from three sites surrounding the collapsed portion of the Cypress Viaduct, which display a remarkable coherence (Figure 6-26) [Hanks, testimony Jan. 4, 1990]. The three records include that recorded at the Outer Harbor Wharf site in Oakland, about 2 km west of the collapsed structure; that recorded at the Emeryville site, about 2.5 km north of the collapsed structure; and that recorded at the downtown Oakland site, about 2.5 km east of the collapsed structure. In Figure 6-26 the displacements records are shown shifted slightly on the time scale to place the arrival of the first large signal on each record at about 10.5 seconds. The coherence is apparent, and if each record is overlain over the other, the similarity of both main and more minor wave forms in the three records is even more evident. Inasmuch as the collapsed section of the Cypress Viaduct lies nearly at the center of the triangle formed by these three stations, it is fair to assume that the ground motion under this section of the Cypress Viaduct must have been very similar to that recorded at the three stations.

Potential for Damaging Earthquakes in the San Francisco Bay Area

Earthquakes have occurred throughout most of California in the past and more are to be expected in the future. The pattern of small earthquakes, even in as short a period as two years, demonstrates how widespread earthquakes are in the state (Figure 6-27) [Hill et al., in press 1990]. The distribution of faults on which displacement occurred in historical time or in the relative recent geological past (Figure 6-28) is another indicator of where future earthquakes might occur [Hill et al., in press 1990].

The location of significant earthquakes that occurred between 1769 and 1987 are shown in Figure 6-29, and Table 6-3 lists those earthquakes of Magnitude 7 or greater

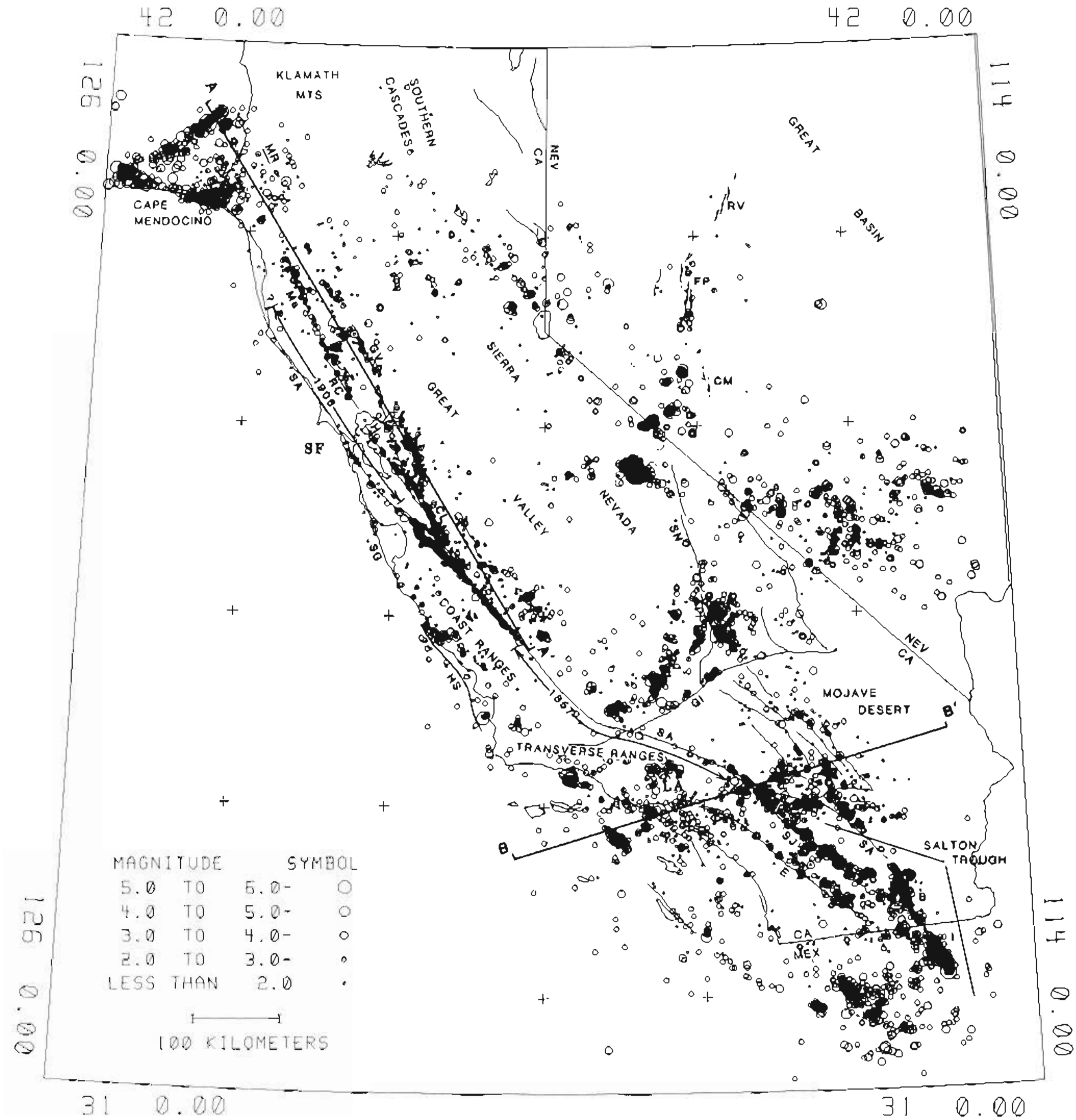


Figure 6-27.
Earthquakes of less than M6 in California, Nevada and northern Baja California, 1980-1981. This distribution of seismicity indicates that most of California is subject to earthquakes. [from Hill et al., in press 1990].

that have occurred in California during that period [Ellsworth, in press, 1990]. An estimate of the conditional probability of major earthquakes occurring along segments of the San Andreas fault and its major branches is shown in Figure 6-30, see also Table 6-4, [USGS, 1990].

The potential for one or more large earthquakes on faults of the San Andreas system in the San Francisco Bay area is considered to be 50% for the 30-year period following January 1, 1988. This estimate represents a

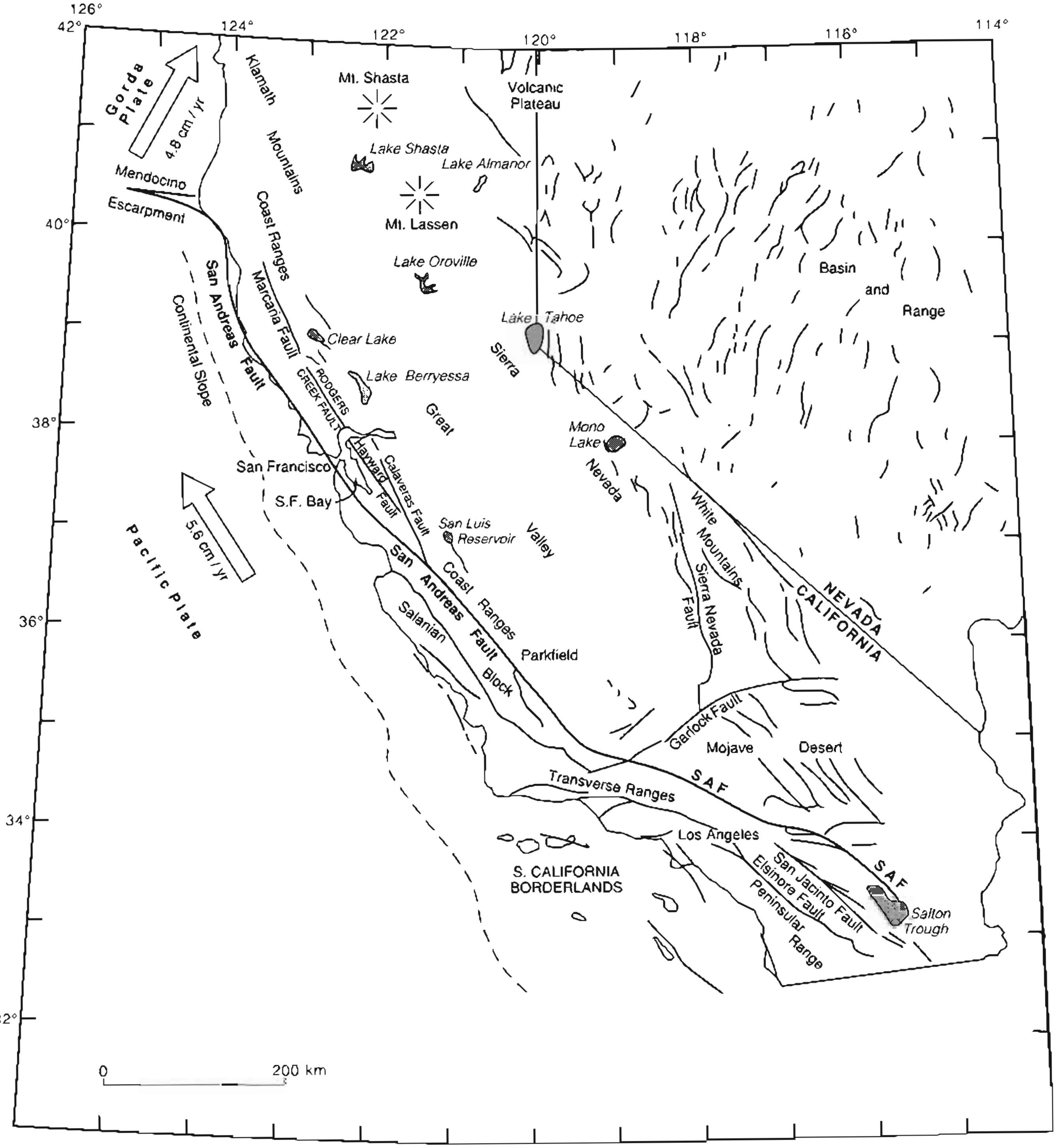


Figure 6-28. *Simplified map showing faults in California and Nevada on which earthquakes have occurred or might occur. Large arrows off coast show direction and rates of motion of Pacific and Gorda plates relative to North American plate, the fundamental plate motions that result in earthquakes in California [from Ellsworth, in press 1990].*

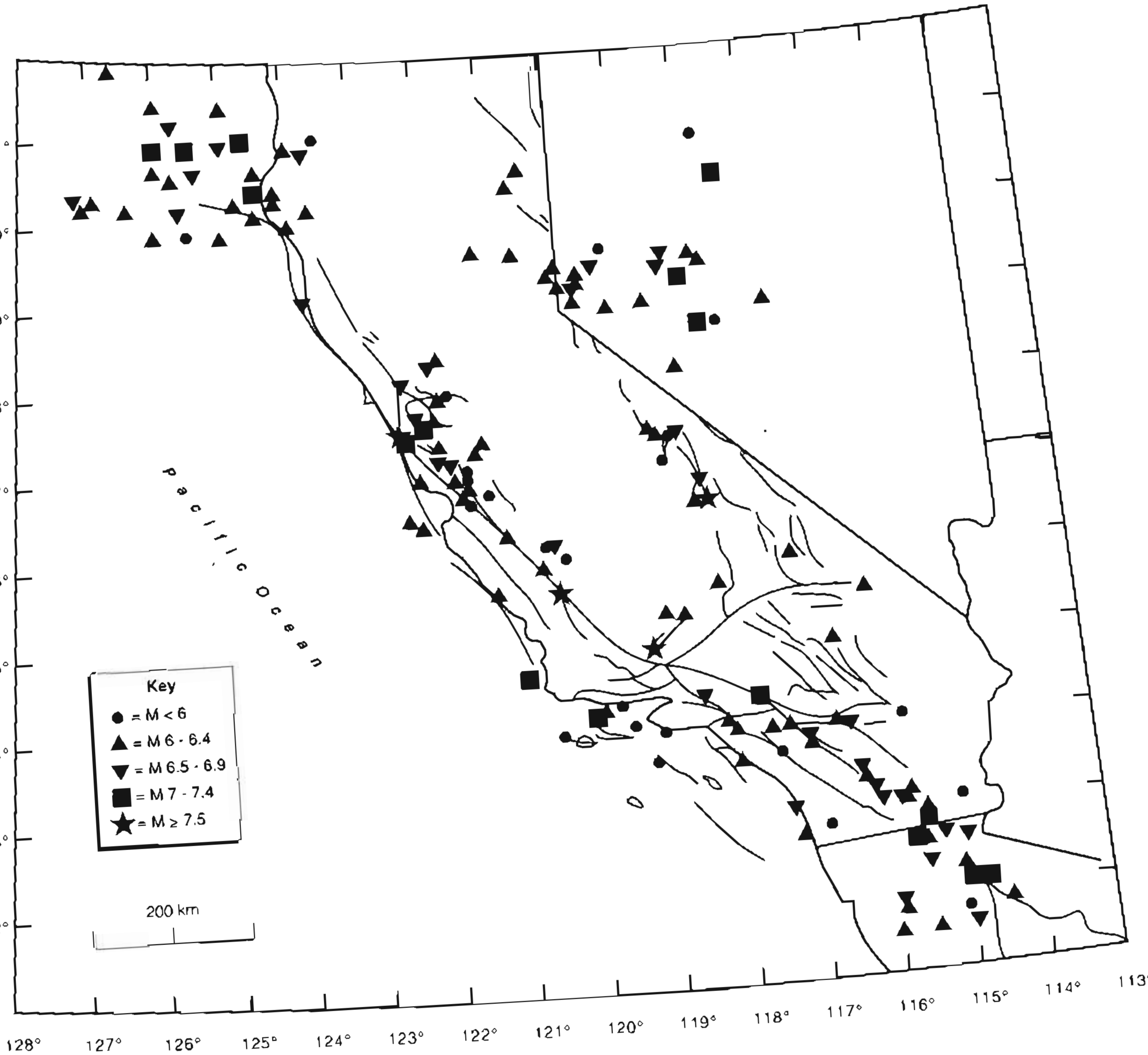


Figure 6-29. *Significant earthquakes of California, Nevada and northern Baja California, 1769-1987. Circle: M6; triangle M6-6.4; inverted triangle: M6.5-6.9; square: M7-7.4 star: M7.5 [from Ellsworth, in press 1990]*

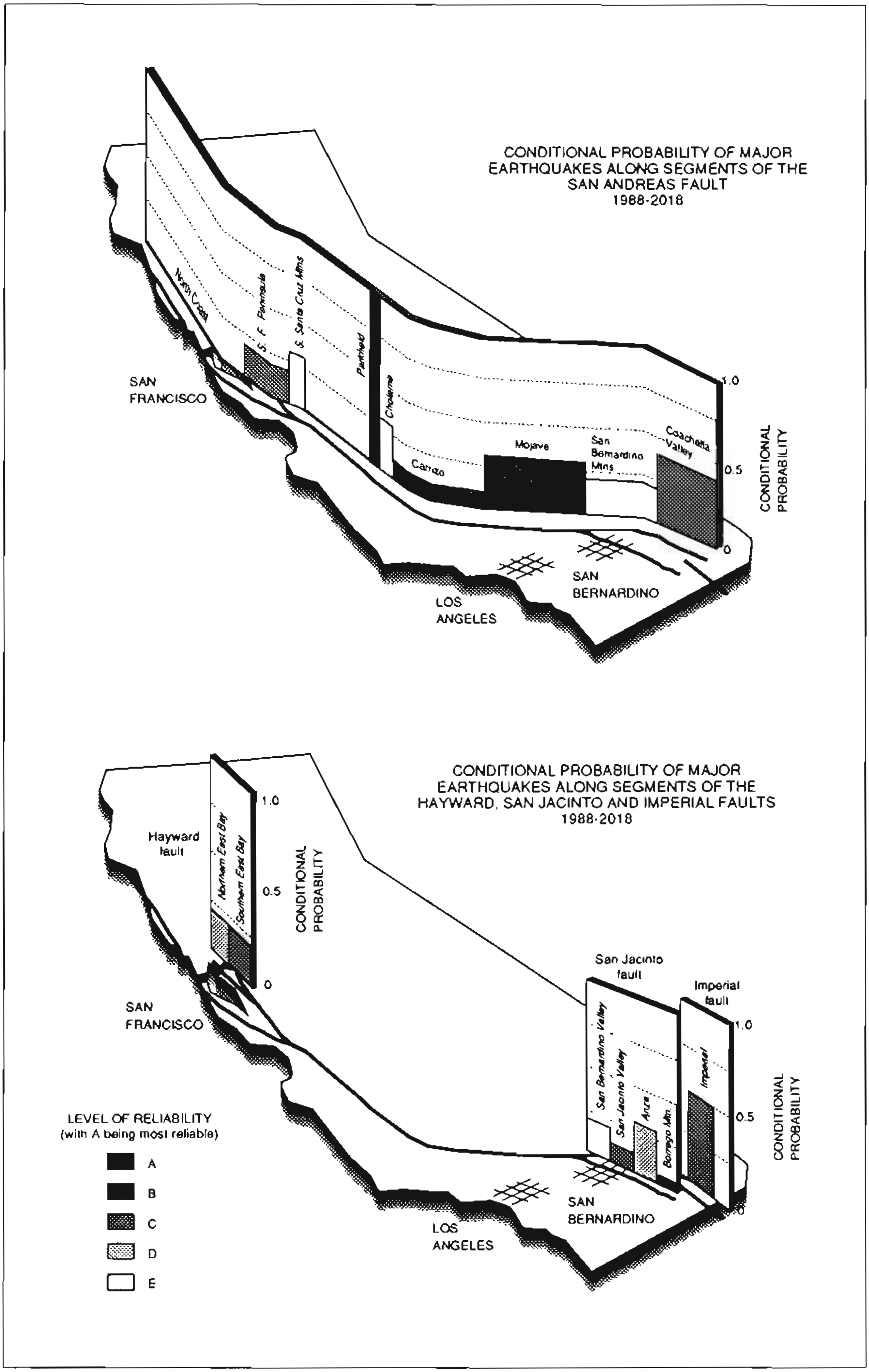


Figure 6-30. *Diagram showing conditional probabilities of major earthquakes along the San Andreas fault and its major branches [from The Working Group on California Earthquake Probabilities, 1988]*

Table 6-3. *Earthquakes larger than Magnitude 7 in California, 1769-1989*

Date	GMT Time	Magnitude	Locality
1812/12/8	15:00	7	Wrightwood
1812/12/21	19:00	7	Santa Barbara Channel
1838/6/?	P.M.	7	San Francisco Peninsula
1857/1/9	08:00	8.25	Ft. Tejon
1868/10/21	15:53	7	Hayward Fault
1872/3/26	10:30	7.6	Owens Valley
1899/4/16	13:40	7	West of Eureka
1906/4/18	13:12	8.25	Great San Francisco Earthquake
1922/1/31	13:17	7.3	West of Eureka
1923/1/22	09:04	7.2	Cape Mendocino
1927/11/4	13:50	7.3	Southwest of Lompoc
1940/5/19	04:36	7.1	Imperial Valley
1952/7/21	11:52	7.7	Kern County Earthquake
1980/11/8	10:27	7.2	West of Eureka
1989/10/18	00:04	7.1	Loma Prieta

Table 6-4. *Probability of one or more large earthquakes on the San Andreas fault system.*

Geographic Region of the Fault	Expected Magnitude	Probability for intervals beginning 1/1/88			
		5 yr	10 yr	20 yr	30 yr
San Francisco Bay Area	7	0.1	0.2	0.3	0.5
Southern San Andreas Fault	7.5-8	0.1	0.2	0.4	0.6

composite of probabilities of large earthquakes on two segments of the Hayward fault immediately east of San Francisco Bay (20% each); the Peninsula segment of the San Andreas fault west of San Francisco Bay (20%), and the much longer segment of the fault, which includes the Peninsula segment, on which occurred the 1906 earthquake (10%) (Figure 6-31) [USGS, 1990]. Additionally, the Healdsburg-Rodgers Creek fault, a northwestward extension of the Hayward fault has recently been assessed as having a significant potential for a Magnitude 7 earthquake [Budding et al., 1989].

The levels of ground motion that may be generated during these earthquakes can be estimated using concepts discussed earlier in this Chapter.

The overall damage potential in the Bay Area may be examined in a relative sense by using the preliminary maps produced by Evernden [1990]. He calculated Modified Mercalli Intensities (MMI) in and around the San Francisco region for four earthquakes (Figures 6-32 through 35).

- An earthquake with Loma Prieta's size and location.
- A repeat of the 1906 San Francisco earthquake.
- A San Francisco peninsula earthquake, Magnitude 7.0.
- A Hayward fault earthquake, magnitude 7.0.

The intensities estimated for the Bay margins in the vicinity of Oakland and San Francisco are MMI IX in the latter three earthquakes, whereas intensity VIII did not extend that far north for either the projection or occurrence of the Loma Prieta earthquake. MMI IX corresponds to violent shaking and is described by the term "general panic," and as causing heavy damage to nonseismically reinforced structures, including many collapses, and damage to seismically-resistant structures. While these maps are not predictions of what will specifically happen in the respective earthquakes, they indicate the degree to which the effects of the Loma Prieta earthquake were less than those that can be expected in other large Bay Area earthquakes.

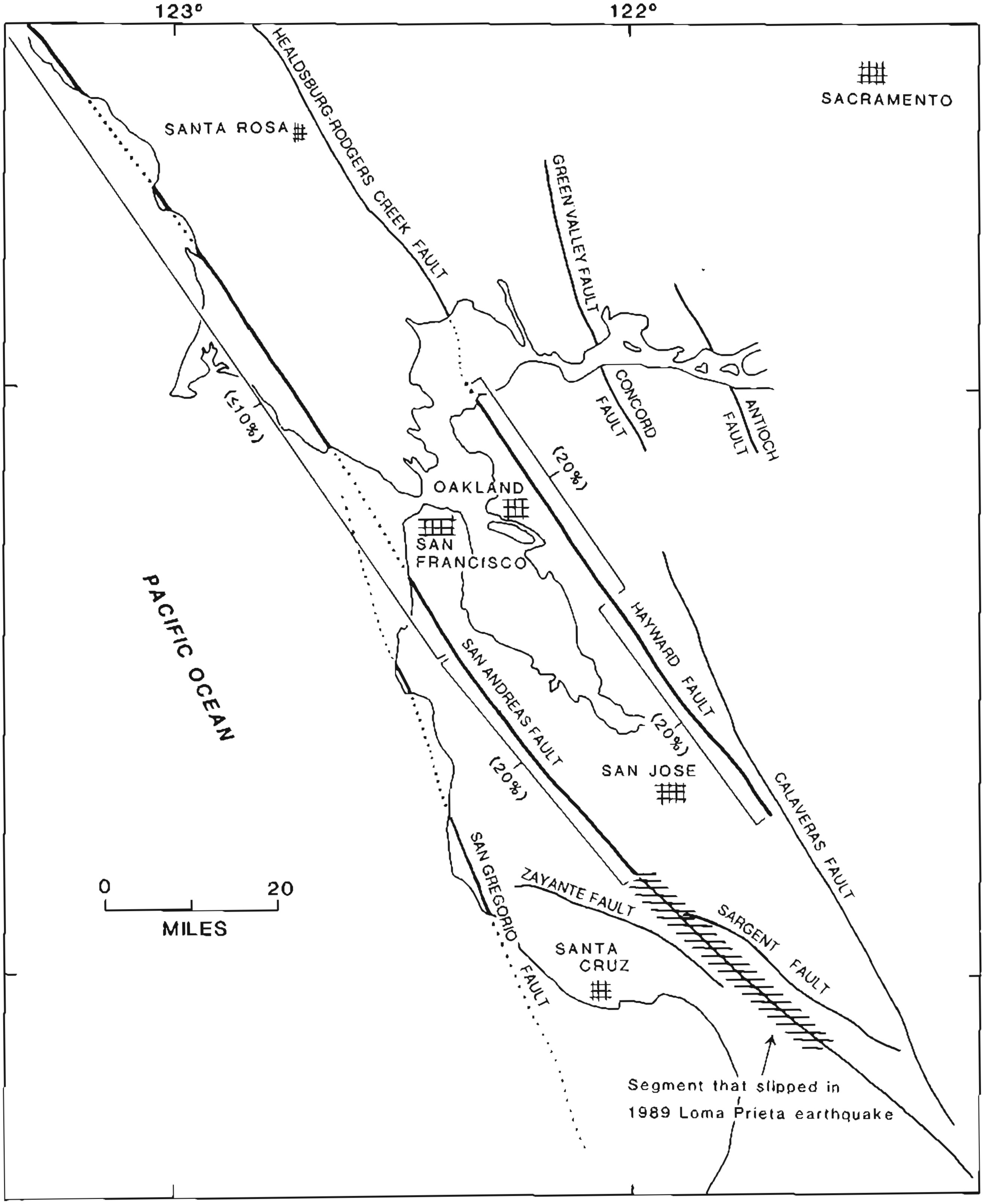


Figure 6-31. *Diagram showing conditional probabilities of major earthquakes in the Bay Area [from The Working Group on California Earthquake Probabilities, 1988]*

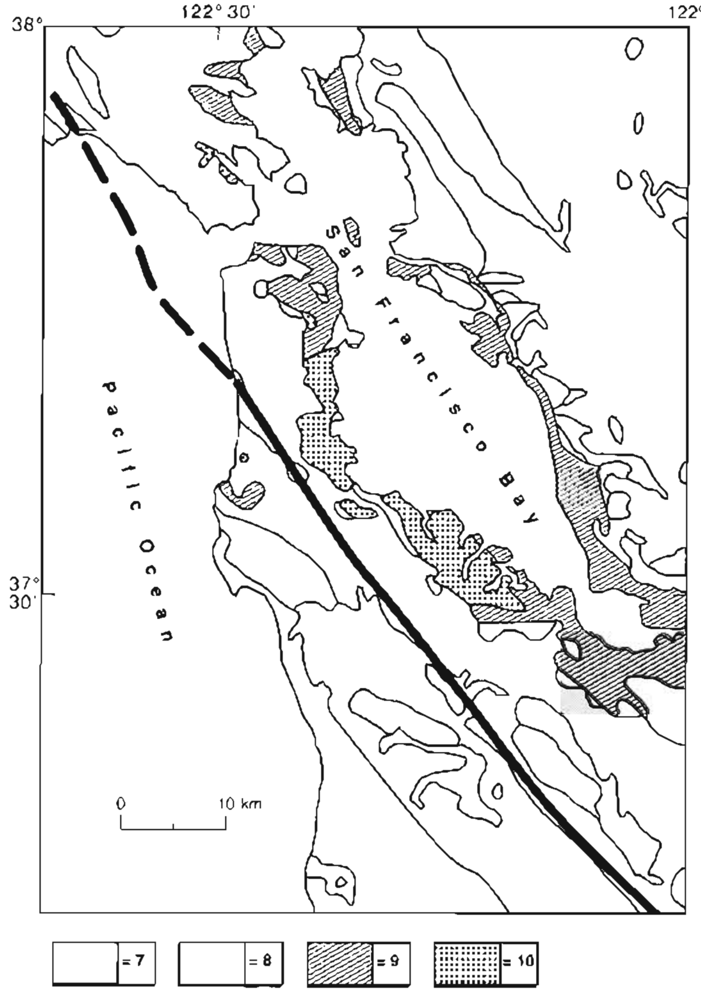


Figure 6-32. *Map showing the calculated Modified Mercalli Intensity for a repeat of the 1906 San Francisco earthquake on the San Andreas fault (heavy line) [Evernden, in press 1990].*

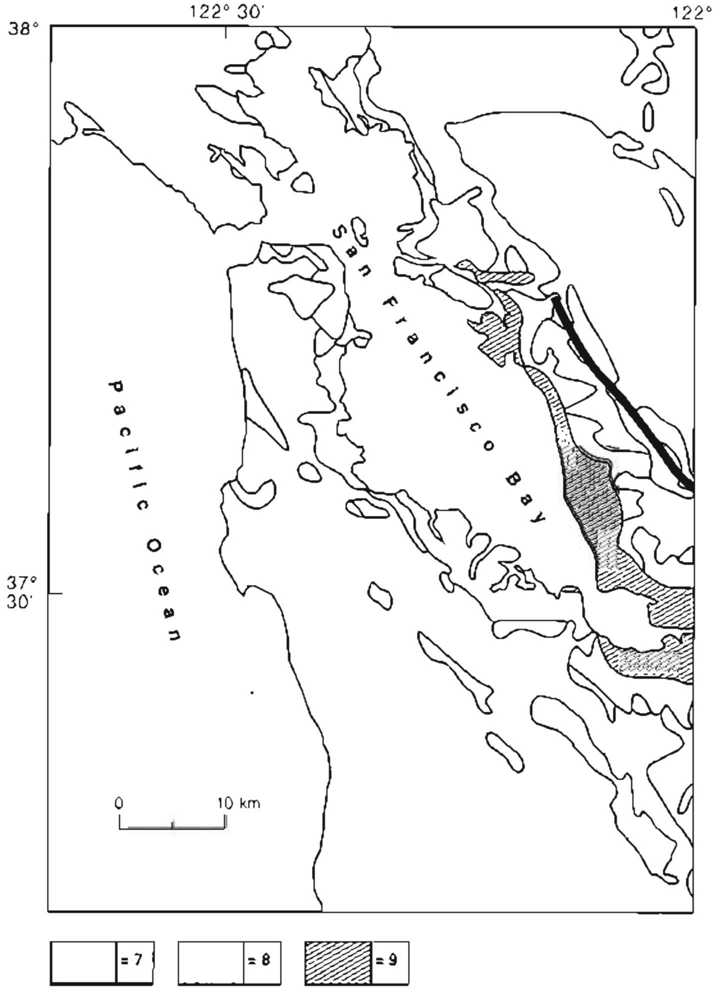


Figure 6-33. *Map showing the calculated Modified Mercalli Intensity for an earthquake on the Hayward fault (heavy line) [Evernden, in press 1990]*

Conclusions

The Loma Prieta earthquake of October 17, 1989 was not the expected great earthquake for the region—that is, a repeat of the 1906 San Francisco earthquake. Such a great earthquake is still to come in the San Francisco Bay region. After about 50 years of seismic quiescence since 1906, numerous moderate-size earthquakes ranging from M 5+ to M 6+ have begun to occur, and earthquakes of about M 7 to M 7.5 may become more frequent, as they were in the decades before 1906.

While there is much yet to be learned from ongoing research investigations of the impacts of this earthquake, it is clear that there will be a major influence on the development of earthquake engineering in the next few years, with potentially major impacts on earthquake engineering practices. The Loma Prieta earthquake occurred along a segment of the San Andreas fault previously recognized as having a high potential for an earthquake of Magnitude 6.5 to 7.0. There were many other specific earthquake probabilities reported at the same time by the Working Group on California Earthquake Probabilities. Such a situation presents the engineering and public policy communities with the difficult challenge of how to react to such information in order to moderate the impacts of future earthquakes. As the science of forecasting improves, so will the opportunities to use such forecasts.

The effects of soft sediments on ground motion are well demonstrated, including

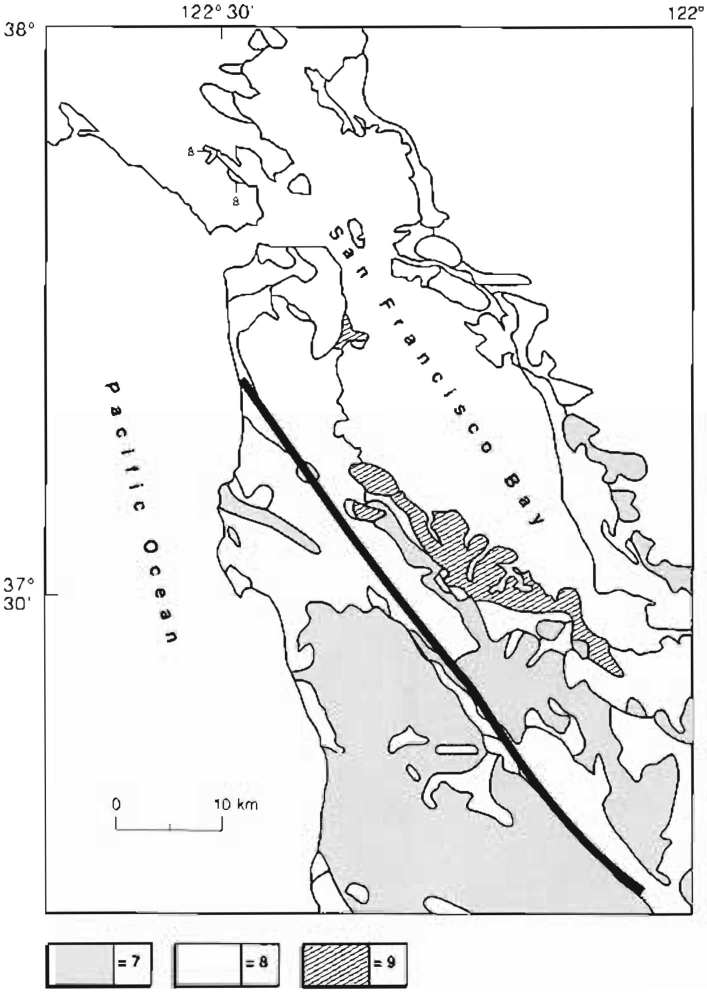


Figure 6-34. *Map showing the calculated Modified Mercalli Intensity for an earthquake on the Peninsula segment of the San Andreas fault (heavy line) [Evernden, in press 1990]*

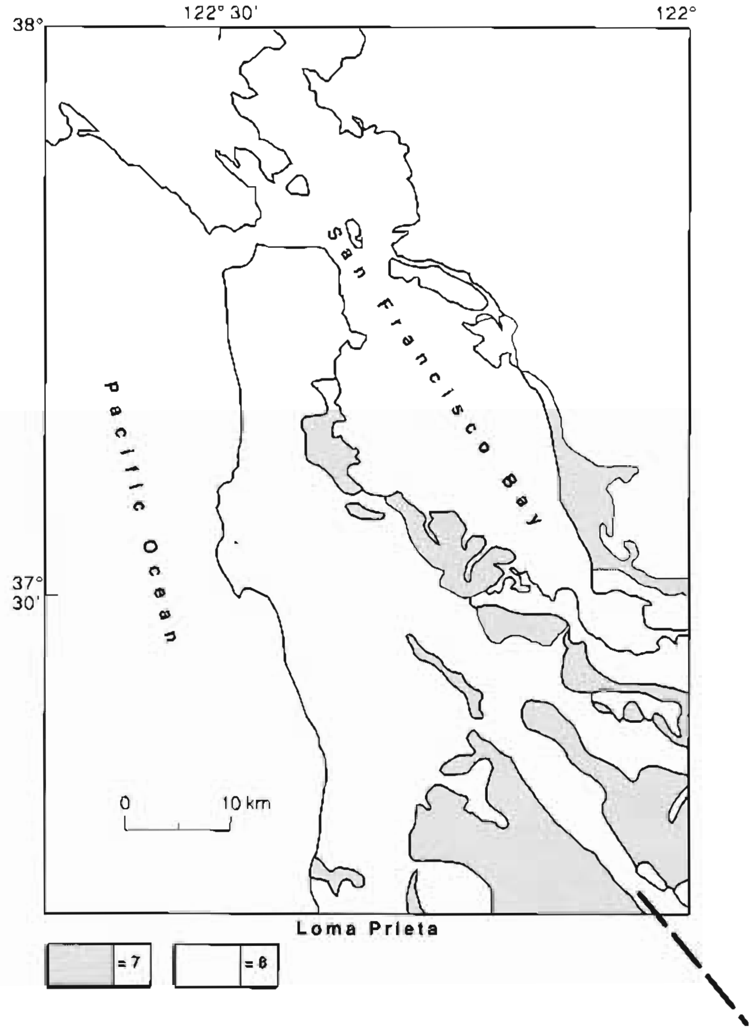


Figure 6-35. *Map showing the calculated Modified Mercalli Intensity for a Loma Prieta size and location earthquake (heavy line) [Evernden, in press 1990].*

amplification of motions of from 2 to 4 times that on bedrock. The relation of ground motion periods to the presence and depth of soft sediments is well illustrated in the records. The motions during this earthquake at soft sites were clearly more than anticipated, even by the most up-to-date building codes. The recordings (only a handful) obtained at soft soil sites during the earthquake constitute the largest set of recordings of significant shaking ever obtained for such sites. These data and those obtained from the larger aftershocks will be used by the engineering professions to refine procedures for estimating these motions and for improving relevant code provisions.

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Chapter 7

Seismic Design Codes in California

The 1933 Long Beach earthquake provided great impetus to the improvement of seismic design practices and adoption and enforcement of seismic building code provisions. The Long Beach earthquake was a Magnitude 6.2 shock that killed several hundred people and caused significant damage, particularly to school buildings. Prior to the 1933 earthquake, most cities in California did not have building codes, and if they did, they were not very faithfully enforced. The Long Beach earthquake prompted the State Legislature to pass the Riley Act, which mandated that cities adopt and enforce building standards at least as conservative as those in the Uniform Building Code (UBC), and the Field Act, which required all public school buildings be designed to resist earthquake forces.

The first code specifically applicable to national bridge design was published in 1931 by the American Association of State Highway Officials (AASHO), which later changed its name to American Association of State Highway and Transportation Officials (AASHTO). Caltrans developed the first criteria within the U.S. for design of bridges to resist seismic forces and incorporated them into their design guidelines in 1940. Caltrans has always compiled its own seismic design criteria, presumably because earthquake problems in California were recognized as severe. AASHTO first incorporated specific seismic considerations into its code in the 1961 edition, adopting the 1943 Caltrans provisions. Generally AASHTO has followed the Caltrans seismic design criteria since then, but has lagged behind. For design purposes, elevated freeway structures are treated as bridges.

Early code requirements for seismic design employed a horizontal force equal to some fraction of the weight of the structure. After the Field Act, school buildings were required to be designed to withstand a horizontal force equal to 10% of the weight of the building. Over the years, as research provided new knowledge about earthquakes—structural dynamics, soil dynamics, strengths of building materials and performance of structural components—building codes were modified to reflect the increased knowledge. However, there has always been a time lag between the development of new knowledge through research and the subsequent modification of codes, whether they are for buildings or bridges.

The following discussion compares the development of design requirements of the UBC, AASHTO, and Caltrans after setting the stage by discussing earthquake engineering research on buildings and bridges.

Research on Reinforced Concrete Bridge Structures

The evolution of the design of concrete bridge structures has depended on two more or less independent channels of research. One channel of research has studied the properties of concrete and reinforcing steel, the properties of reinforced concrete columns, beams, walls, slabs, etc., and the appropriate methods of design for structural elements to resist the forces imposed on them. This research has been carried out in almost all industrialized countries. In the United States it has been carried out at universities and private and governmental research laboratories. The major thrust of this research took place in the years following 1910 and focused almost entirely on the ability of reinforced concrete structures to withstand static loads. The second channel of research has studied the occurrence of destructive earthquakes, the nature of ground shaking generated by earthquakes of different magnitudes and at various distances, the response of buildings to ground shaking, and methods of design to resist earthquake forces. Practically none of this earthquake research has been specifically directed at the seismic design of bridges.

Research in reinforced concrete construction began in Europe and has about a 150-year history. In the United States a major program in experimental research was started in 1903 at the University of Illinois and has continued for many decades. Research also has been carried out at some state experimental research laboratories, at the Portland Cement Association Research Laboratory, and at various other universities and private laboratories.

California was one of the first states in which reinforced concrete construction was employed. In 1889 Ransome built a four-story building in San Francisco using slabs and beams cast as a unit. During the first three decades of this century reinforced concrete buildings were constructed in California without much thought being given to earthquake forces. An example of such construction was the Veterans Administration Hospital in Sylmar that collapsed during the 1971 San Fernando earthquake and caused 49 casualties.

Research on the ability of structures to withstand earthquake forces lagged behind research on the ability of structures to withstand static (gravity) loads. Earthquake engineering as a scientific discipline is a relatively new field. After the 1906 San Francisco earthquake, there was a general feeling of lack of direction. Charles Derleth, Professor of Structural Engineering at the University of California-Berkeley, was the author of the paper "The Effects of the San Francisco Earthquake of April 18, 1906 on Engineering Construction" which appeared in the 1907 *Transactions of the American Society of Civil Engineers*. In it he said "An attempt to calculate earthquake stress is futile. Such calculations can lead to no practical conclusions of value." At that time very little was known about earthquakes. For example, the earthquake magnitude scale was not developed until the 1930s; strong-motion accelerographs for recording destructive ground motions were developed in the early 1930s and the first recordings were not obtained until 1933; engineering education did not include structural dynamics; and there were no computers for calculating the response of structures to earthquakes. Thus, the engineering profession in 1907 had no background knowledge that would enable them to deal with earthquakes. Not that engineers had ignored lateral forces; following the devastating 1868 earthquake in the East Bay, many engineers in the Bay Area adopted the use of bond iron to provide reinforcing for masonry, among other strategies to provide structural continuity. After the 1906 earthquake, the proposition of the eminent French engineer Eiffel, of Eiffel Tower fame, was to treat design for earthquakes the same as design for winds, and he suggested a wind speed for such use. This suggestion was followed in San Francisco by some engineers. Progress was slow until the 1925 Santa Barbara and 1925 Tokyo earthquakes. The 1933 Long Beach earthquake was the real watershed in the development of seismic building practices in California. After 1933, buildings were designed in California to accommodate earthquake loads by providing resistance to static horizontal forces and the designs were carried out based on knowledge developed for reinforced concrete structures under conventional static loading.

In the 1950s research began on the ability of reinforced concrete members to withstand oscillatory bending moments, such as those that would be produced by earthquake shaking. This research showed that to prevent brittle fracture of the reinforced concrete elements, it is necessary to change the method of reinforcing, in particular to use closely-spaced reinforcing bar ties to constrain the main reinforcing bars in regions of large bending moments. This information first appeared in a book published by the Portland Cement Association titled *Design of Multistory Reinforced Concrete Buildings for Earthquake Motions* (1961). Over the next 15 years research on ductile reinforced concrete accelerated understanding of how to design and analyze structures subjected to dynamic earthquake loadings. Structural engineers in California gradually adopted the recommendations set forth in the 1961 book and those set forth in subsequent research, particularly research that followed the 1971 San Fernando earthquake. It is generally felt that ductile design has greatly improved the ability of concrete buildings to withstand severe earthquake motions. In the 1970s, this information began to be used in the design of bridges. Ductile concrete design developed too late, however, to be used in the design of the Cypress Viaduct and the San Francisco Freeway Viaducts.

Two basic problems have been that the level of research effort on earthquake engineering has not been commensurate with the size of the problem and that the information provided by research lagged behind the need for it; the lag time for bridges was approximately 20 years. As a consequence, California is more vulnerability to earthquakes than it should be. The Golden Gate Bridge and the Bay Bridge were identified by researchers as deserving seismic analysis and seismic instrumentation in the 1980s, but proposals to instrument and/or analyze them were not supported by research organizations or the owners. In an effort to strengthen the research in earthquake engineering, a consortium of eight California universities established the California Universities for Research in Earthquake Engineering (CUREe) in 1989. The members of the consortium are the five campuses of the University of California at Berkeley, Davis, Los Angeles, Irvine, San

Diego plus Stanford University, California Institute of Technology, and the University of Southern California. The objective of CUREe is to speed up the development of new knowledge in earthquake engineering, to decrease the lag time in putting the results of research into practice, and to strengthen earthquake engineering education.

Uniform Building Code (UBC)

The Uniform Building Code is published by the International Conference of Building Officials and is considered to the most up to date of several national codes regarding seismic requirements for buildings. It first included a reference to earthquake design in its 1927 edition, but it did not prescribe any design requirements. The 1930 edition stated:

> "The following provisions are suggested for inclusion in the Code by cities located within an area subject to earthquake shocks. The design of buildings for earthquake shocks is a moot question but the following provisions will provide adequate additional strength when applied in the design of buildings or structures."

The requirements for design in 1930 were that a building should be designed to resist a horizontal force F at every elevation:

$$F = C\,W$$

where W is the dead weight plus the live load above the elevation under consideration, and the seismic coefficient C has the values:

$$C = \begin{cases} 0.075 & \text{when the foundation rests on material upon which a load of 2 tons/ft}^2 \text{ or more is allowed} \\ 0.10 & \text{when the foundation rests on material upon which a load of less than 2 tons/ft}^2 \text{ is allowed or when foundation is on piles} \end{cases}$$

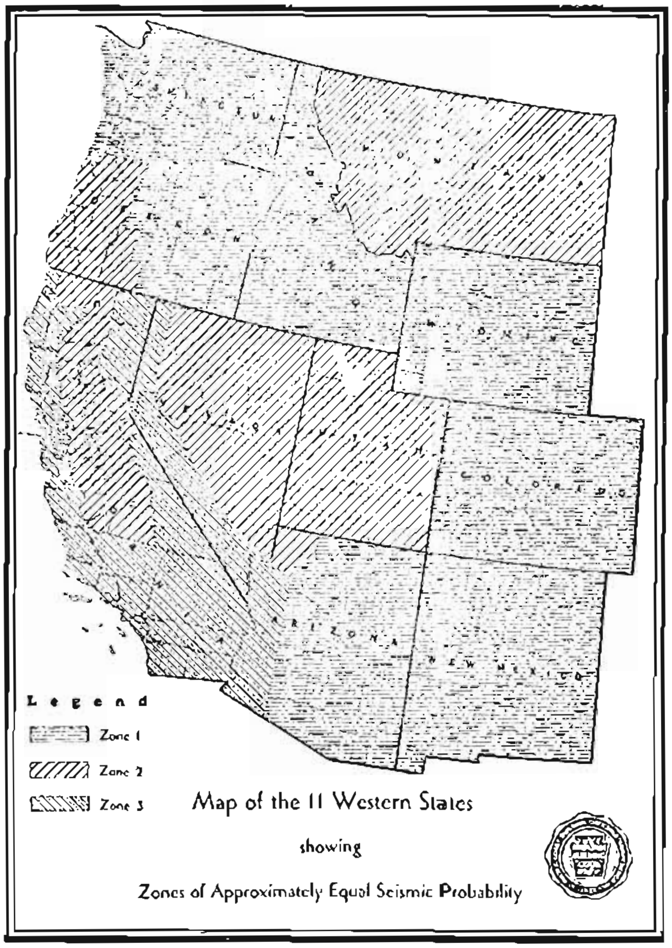
In 1935 the UBC requirements for seismic design were changed, and the new provisions remained in effect until 1948. The relevant portions of the 1946 UBC are reproduced in Figure 7-1. This version of the UBC required that all buildings be designed to resist a horizontal force, F, at every elevation:

$$F = C\,W$$

where W is the dead weight plus half of the live load above the elevation under consideration, and the seismic coefficient C has the values:

$$C = \begin{cases} 0.02 & \text{when the foundation rests on soil having a safe bearing value of 1 ton/ft}^2 \text{ or more, in Zone 1} \\ 0.04 & \text{when the foundation rests on soil having a safe bearing value less than 1 ton/ft}^2\text{, or is supported on piles, in Zone 1} \end{cases}$$

These values of C are for Zone 1 on the Zoning Map (Figure 7-1). For Zone 2, regions of intermediate seismicity, the values



(b) Horizontal Force Formula. In determining the horizontal force to be resisted, the following formula shall be used:

F=CW

where "F" equals the horizontal force in pounds.

"W" equals the total dead load plus one-half the total vertical designed live load,

at and above the point of elevation under consideration, except for warehouses and tanks, in which case "W" shall equal the total dead load plus the total vertical designed live load at and above the point or elevation under consideration. Machinery or other fixed concentrated loads shall be considered as part of the dead load.

"C" equals a numerical constant as shown in Table No. 23-A

TABLE NO. 23-A—HORIZONTAL FORCE FACTORS

Part or Portion	*Value of "C"**	*Direction of Force*
The structure as a whole and every portion not itemized in this table**	.02 on soil, over 2000 lbs. .04 on soil up to 2000 lbs.	Any direction horizontally
Bearing walls, non-bearing walls, partitions, curtain walls, enclosure walls, panel walls	05 With a minimum of five pounds per square foot.	Normal to surface of wall
Cantilever parapet and other cantilever walls, except retaining walls	25	Normal to surface of wall
Exterior and interior ornamentations and appendages	.25	Any direction horizontally
Towers, tanks, towers and tanks plus contents, chimneys, smokestacks, and penthouses when connected to or a part of a building	.05	Any direction horizontally

*See map on page 266 for zones. The values given "C" are minimum and should be adopted in locations not subject to frequent seismic disturbances as shown in Zone 1. For locations in Zone 2, "C" should be doubled. For locations in Zone 3, "C" should be multiplied by four.

**Where wind load as set forth in Section 2307 would produce higher stresses, this load should be used in lieu of the factor shown.

Figure 7-1. *Seismic design requirements reproduced from the 1946 Uniform Building Code.*

of C should be multiplied by 2; and for Zone 3, regions of high seismicity like the San Francisco Bay area, the values of C should be multiplied by 4. Thus the values of C would have been 0.08 and 0.16 for the San Francisco-Oakland region. Note that Zone 3 is the highest zone of the 1946 map; Zone 4 was not introduced until the 1970s.

The 1949 UBC incorporated a fundamental change in the way C values were assigned. It required that each story of a building be designed to resist a horizontal force F = CW, where W is the dead weight tributary to the story under consideration and C, for Zone 3, is

$$C = \frac{0.15}{N+4.5} \times 4$$

where N is the number of stories above the story under consideration. For example, a two-story structure must be designed so that the second story can resist a seismic force of 0.13 times the weight of the roof and tributary walls. The first story should be designed to resist a force 0.11 times the weight of the second floor plus the roof and tributary walls. These values are presumably for firm ground. The 1949 UBC made no provision for differences in earthquake forces that may result from differences in foundation soils, whether soft or firm.

In 1961 further modifications were made to the seismic requirements, of the UBC. The seismic requirements were modified again as a consequence of the February 9, 1971 San Fernando earthquake, which was centered on the northern boundary of the City of Los Angeles. Additional modifications of the code have been made as research produced new knowledge, and the 1988 edition of the UBC has a greatly expanded seismic design chapter.

Caltrans Seismic Design Criteria, 1940-1971

Caltrans (formerly the California State Highway Department) has always compiled its own seismic design criteria, presumably because it recognized that California had a more severe earthquake problem than did the other states. It formulated the first code requirement in the United States in 1940 for design of bridges to resist seismic forces. Earthquakes were considered in design of bridges before the first formal codes. In 1933, the design calculations and specifications used by Caltrans for the San Francisco-Oakland Bay Bridge required that a static force of 0.075W (or 0.10W as has been stated in some reports as the basis for design) be applied as an earthquake load. It appears that these values were either taken from the 1930 UBC or used a common source.

The seismic design criteria used as the basis for the design of the Cypress Viaduct in Oakland and the San Francisco Freeway Viaducts were contained in the prevailing edition of Caltrans's standard specifications. The seismic criteria used for all the viaducts were essentially identical and were based on criteria adopted by Caltrans in 1943. These criteria remained unchanged until some modifications in 1965, which were followed by substantial amendments after the 1971 San Fernando earthquake.

The specific seismic criteria implemented by Caltrans between 1940-65 are quoted below:

1940 "Provision shall be made for seismic stresses resulting from earthquake. The seismic force shall be considered as an assumed horizontal force applied at the center of mass in any direction that will produce a maximum stress in the member considered. The assumed horizontal force shall be a percentage of the dead load and will be determined by the Designing Engineer."

1943 "...structures...shall be designed to resist a seismic force (F) in accordance with the following formula: F = CW, where F is the seismic force to be applied horizontally in any direction at the center of gravity of the weight of the structure, W= dead load of the structure, and C is:

$$C = \begin{cases} 0.02 & \text{for structures founded on spread footings with a bearing capacity exceeding 4 tons/f}^2 \text{ or better} \\ 0.04 & \text{for structures founded on spread footings with a bearing capacity less than 4 tons/f}^2 \\ 0.06 & \text{for structures founded on pile foundations} \end{cases}$$

1965 "...structures...shall be designed to resist earthquake forces (EQ) in accordance with the following equations:

EQ = KCD

where

EQ = force applied horizontally at the center of gravity of the structure. This force shall be distributed to supports according to their relative stiffnesses.

K = Numerical coefficient representing energy absorption of the structure:

= 1.33 for bridges where a wall with a height to length ratio of 2.5 or less resists horizontal forces applied along the wall.

= 1.00 for bridges where single columns or piers with a height to length ratio greater than 2.5 resists the horizontal forces.

= 0.67 for bridges where continuous frames resist horizontal forces applied along the frame.

C = $0.05(T)^{-1/3}$ (maximum value of C = 0.10).

= Numerical coefficient representing structure stiffness.

T = $0.32(D/P)^{1/2}$ for single-story structures only.

= Period of vibration of structure.

D = Dead load reaction of structure.

P = Force required for one inch horizontal deflection of structure.

The EQ forces calculated above shall never be less than 0.02D. Special consideration shall be given to structures founded in soft materials capable of large earthquake movements, and to large structures having massive piers."

At the time the Cypress Viaduct and the San Francisco Freeway Viaducts were designed in the 1950s, Caltrans's seismic force requirements were sufficiently small that most bridges would automatically have satisfied them—that is, when a bridge was designed to carry its own weight and the traffic loads, it would have more earthquake resistance than that required by these values of C, especially since the transverse and longitudinal forces were not applied simultaneously.

Caltrans Post-1971 Seismic Requirements

In 1971, after the San Fernando earthquake and the consequent collapse of several freeway structures, Caltrans:

- Doubled the design forces for frames on spread footings, returning the design lateral forces to approximately those required in the 1943 criteria
- Increased the design forces for frames on pile foundations by a factor of 2.5
- Introduced ductile detailing requirements

Caltrans also initiated development of new design criteria to incorporate technical developments in earthquake engineering related to:

- Ground motion attenuation
- Soil effects (local geology)
- The dynamic response of bridge structures

These efforts led to the development of

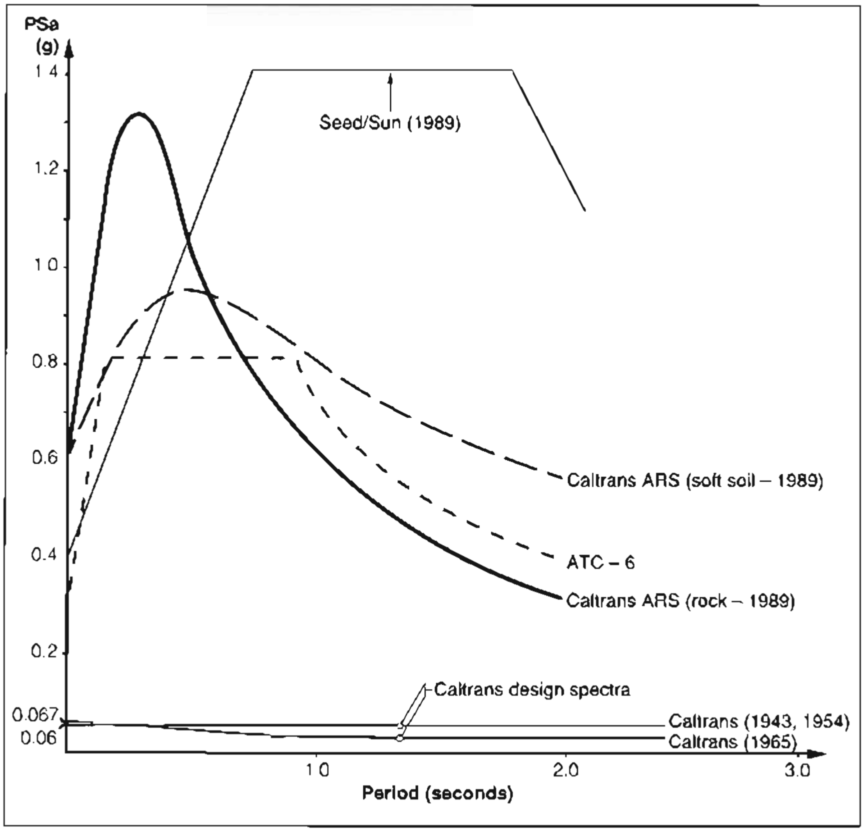


Figure 7-2.
Caltrans ARS elastic spectra (1989) and design spectra (1965, 1954, 1943) for multi-pier bents. Note that the ARS spectra (upper curves) and pre-1971 design spectra are not strictly comparable, since the ARS spectra are divided by Z to determine design values; see Figure 7-3.

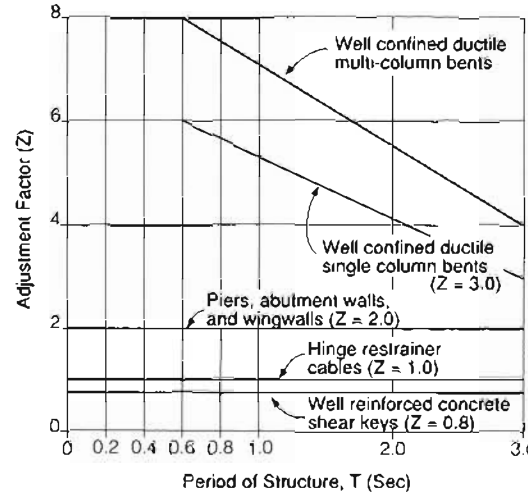


Figure 7-3. *Adjustment for ductility and risk assessment—Z factor.*

Table 7-1. *Summary of reduction factors.*

	Caltrans	ATC-6	ATC-3
Frame type	Z[1]	R[1]	R[2]
Single colum	6	3	2.5
Multiple column	8	5	7

Notes: 1. Bridge structure, maximum value
2. Building structure

the Caltrans ARS Spectra where A, R and S relate to the maximum expected bedrock acceleration (A), the normalized rock response (R), and the soil amplification spectral ratio (S). In 1978, the Applied Technology Council (ATC), in a project sponsored by the National Science Foundation, published ATC-3, *Tentative Provisions for the Development of Seismic Regulations for Buildings*. These seismic building provisions were developed by consulting engineers, researchers and federal and state agency representatives [ATC, 1989]. In another project, sponsored by the Federal Highway Administration, ATC in 1982 published ATC-6, *Seismic Design Guidelines for Highway Bridges*; these guidelines were the recommendations of a team of nationally recognized experts, composed of Federal and State agency representatives, consulting engineers, and researchers [ATC, 1981]. The 5% damped Caltrans ARS Elastic Response Spectra, the ATC-6 Soil Type III Ground Motion Spectra (anchored to a peak ground acceleration of 0.5g), and the equivalent allowable stress design spectra used by Caltrans in the period from 1943 to 1965, are presented in Figure 7-2.

The Caltrans Bridge Design Specification [State of Calif., Dept. of Trans., various dates] reduces the ARS Elastic Response Spectra to a Strength Design Spectra through the use of an adjustment factor (Z). The Caltrans' adjustment factors (Z) for ductility and risk assessment as a function of period and structure type/component is reproduced in Figure 7-3. The rationale behind the values of the Caltrans Z factors is not documented. ATC-3 and ATC-6 divide elastic spectral ordinates by a response modification factor (R) to obtain a strength design spectrum.

The development of the ATC-3 and ATC-6's R factors was based on the redundancy, ductility, and over strength provided by the various systems. Table 7-1 summarizes the Caltrans Z factors, the ATC-3 R factors for buildings, and the ATC-6 R factors for bridges. A multiple-column bent with well detailed ductile columns was assigned the highest R value of 5 in ATC-6, compared with a value of 8 used by Caltrans for the same system. For single-column bents, the Caltrans Z factor of 6 is twice the ATC-6 R factor of 3, and 2.4 times the ATC-3 R factor for an inverted pendulum building.

In Figure 7-4, strength design spectra for multi-column bents on soft soil sites have been generated for the Caltrans seismic

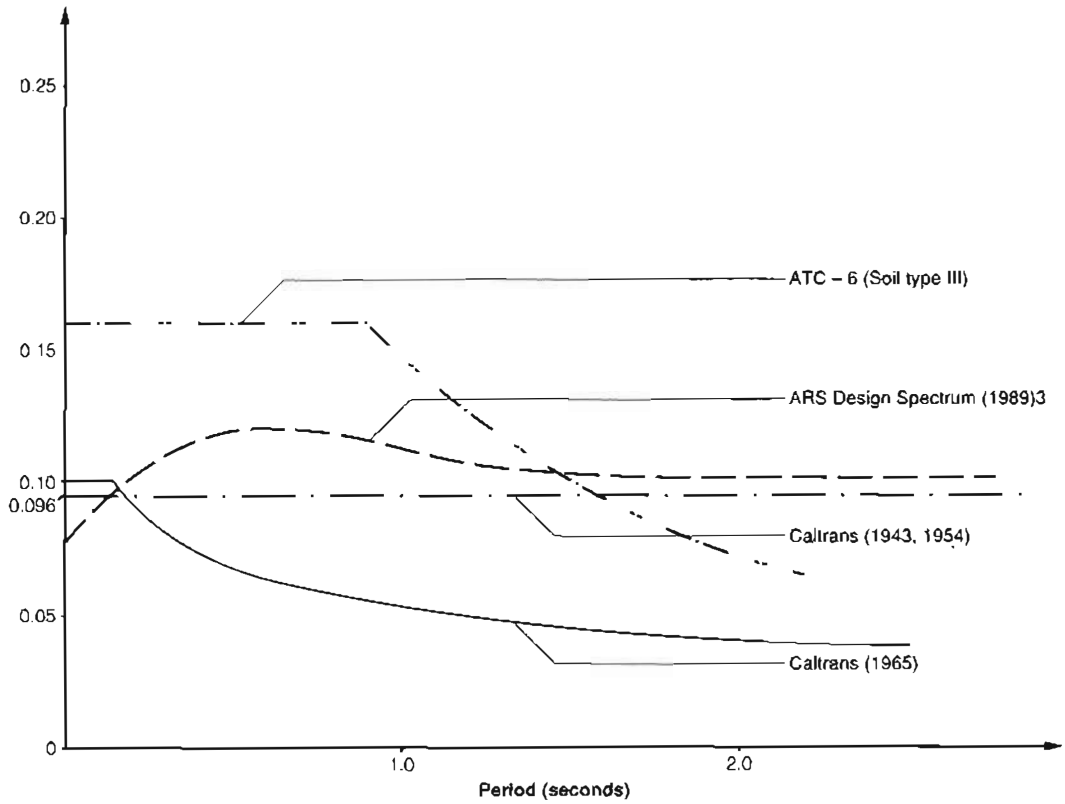


Figure 7-4. *Caltrans strength design spectra (1943, 1954 1965, 1989) and ATC-6 strength design spectrum (1981) for multi-column bents on soft soil sites.*

requirements of 1943, 1954, 1965, and 1989 and the ATC-6 guidelines of 1982.

Current Caltrans minimum strength requirements are virtually identical to the 1943 requirements, the only difference being the ductile detailing requirements in the current bridge design specifications, which are enough to cause significant improvements in expected performance. For periods less than 1 second, the ATC-6 spectral ordinates are approximately 50% greater than the pre-1965 Caltrans requirements, but for periods exceeding 1.5 seconds, the ATC-6 spectral ordinates are less than the pre-1954 Caltrans requirements. Although the current Caltrans ARS spectra account for site effects, earthquake shaking intensity, and so on, in a very reasonable manner, their strength design spectra appear to be unconservative, especially in the short period range [Uang and Bertero, 1988].

AASHTO Seismic Design Codes for Bridges

The national code for design of bridges is prepared by AASHTO. This code was first issued in 1931 and the official title of the code now is *AASHTO Standard Specifications for Highway Bridges*. The Federal Highway Administration and its predecessor, the Bureau of Public Roads, required that all bridges constructed with federal funds be designed in accordance with these specifications. The AASHTO code, like the UBC, is the publication of a national nonprofit organization and each State Highway Department could adopt, or not adopt, elements of the code.

Prior to 1941, AASHO bridge design specifications did not mention earthquake loads. The 1941, 1944 and 1949 editions of the AASHO code simply stated that: "Structures shall be proportioned for earthquake stresses." However, there was no recommendation or criterion as to how the earthquake force was to be determined or how it was to be applied to the structure.

AASHTO codes did not incorporate seismic provisions until the 8th edition (1961), although these were first issued in 1958 as "interim" provisions. The 1961 AASHTO code essentially implemented the 1943 Caltrans seismic procedures. In general, the AASHTO code since 1961 has followed the Caltrans seismic design specifications, but has lagged behind. In 1975 the AASHTO code was expanded to include the earthquake criteria developed by Caltrans in

Table 7-2. *Caltrans, AASHTO, and UBC seismic design requirements (1940-1965) for the Cypress and San Francisco Freeway Viaducts founded on bay mud.*

Year	Single Pier[1]			Multi-Pier[1]		
	Caltrans	AASHTO	UBC	Caltrans	AASHTO	UBC
1940			0.16W			0.16W
1943	0.06W		0.16W	0.06W		0.16W
1946	0.06W		0.16W	0.06W		0.16W
1949[2]	0.06W		0.11W	0.06W		0.09W
1954[3]	0.06W		0.11W	0.06W		0.09W
1958	0.06W	0.06W	0.11W	0.06W	0.06W	0.09W
1961	0.06W	0.06W	0.11W	0.06W	0.06W	0.13W
1962[4]	0.06W	0.06W	0.11W	0.06W	0.06W	0.13W
1965[5]	0.05W[6]	0.06W	0.11W	0.033W[6]	0.06W	0.13W

Notes:
1. Base shear coefficient for single pier and multi- pier (2+) bents supporting one and two roadway decks, respectively; all supported on pile foundations. UBC comparisons are for one and two story structures.
2. Cypress Viaduct designed, first double-deck viaduct.
3. Terminal Separation Viaduct designed, first of the five San Francisco double-deck viaducts.
4. Southern Freeway Viaduct designed, last of the five San Francisco double-deck structures.
5. China Basin Viaduct designed.
6. Assumed natural period of 1 second (T).

1973. In 1983, following the completion of substantial research sponsored by the Federal Highway Administration and Caltrans, the AASHTO *Guide Specification for Seismic Design of Highway Bridges* was published. The 1983 AASHTO code retained the requirements of the 1975 code, but allowed the designer the option of using the *Guide Specification*. The 1989 AASHTO code contains the same reference to the optional use of the 1983 *Guide Specification:*

> In regions where earthquakes may be anticipated, structures shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure in accordance with the following criteria or AASHTO Guide Specifications for Seismic Design of Highway Bridges.

Comparisons Among Codes

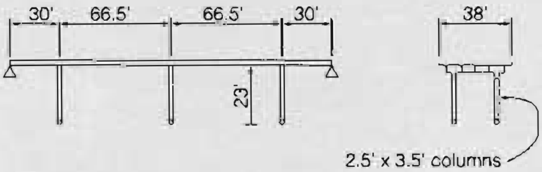
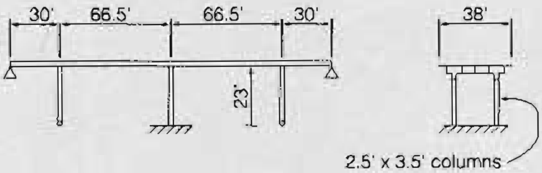
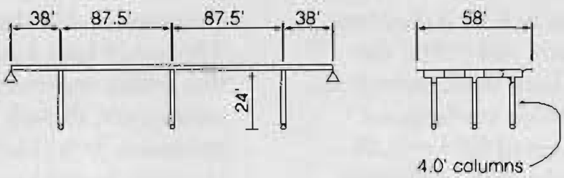
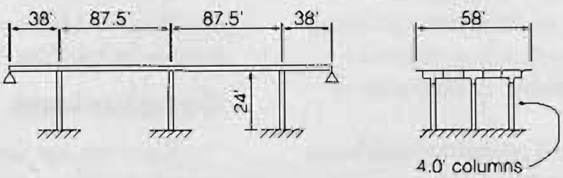
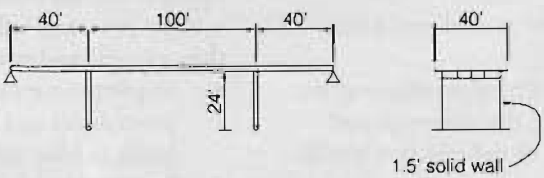
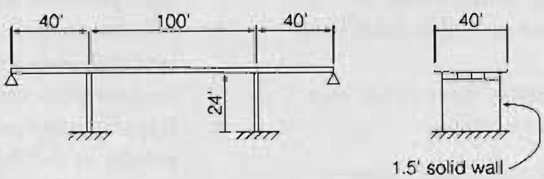
All the Bay Area double-deck freeway viaducts are pile-supported and were designed by Caltrans for the 1943 level of lateral force (0.06W). Caltrans and AASHTO seismic design requirements for pile-supported single- and multi-pier frames are compared in Table 7-2 The Uniform Building Code (UBC) minimum design lateral forces for one- and two-story structures, for the period 1946-65 are also presented in Table 7-2 under the single and multiple pier headings. The UBC minimum design forces are between 50% and 290% higher than those required by Caltrans.

Caltrans's introduction of seismic force requirements related to structural configuration (Table 7-3) in 1965 lead to a significant reduction (approximately 50%) in the design lateral forces for those structures with natural periods exceeding 1.0 second.

At the time that the Cypress Viaduct was designed, little was known about earthquake effects on bridges. It would have been natural for bridge designers to refer to experience in building design, as codified in the UBC, even though bridges differ in significant ways from buildings. For considerations of seismic design, the double-deck Cypress Viaduct differed from a two-story building in the following ways: the story heights were taller; the floor decks were much heavier; there were fewer columns per unit of floor area; and there were no walls or other nonstructural elements that would would provide extra strength and energy dissipation during an earthquake. These differences indicate that the seismic requirements of the building code would be less adequate for a freeway structure than for a building.

Had Caltrans used the seismic requirements of the UBC in the design of the

Table 7-3. *Summary of Caltrans (1963) seismic c_k factors for bridges.*

Type of bridge	Elevation / Typical section	Transverse period & "K"	Longitudinal period & "K"	Transverse factor K x C	Longitudinal factor K x C
Two-lane tee beam Pin-end columns		1.4 sec. 0.67	3.5 sec. 0.67	3.0%	2.2%
Two-lane tee beam Pin-end columns Center bent fixed		1.2 sec. 0.67	2.6 sec. 0.67	3.1%	2.4%
Four-lane box girder All pin-end columns		0.7 sec. 0.67	0.7 sec. 0.67	3.7%	3.7%
Four-lane box girder All fixed-end columns		0.4 sec. 0.67	0.4 sec. 0.67	4.7%	4.7%
Closed abutment All pin-end piers		0.03 sec. 1.33	1.0 sec. 0.67	13.3%	3.3%
Closed abutment All fixed-end piers		0.03 sec. 1.33	0.5 sec. 0.67	13.3%	4.2%

Cypress Viaduct, it would have been designed for a seismic coefficient of 0.16 if they had used the pre-1949 values, and 0.09 if the 1949-1958 values had been used, instead of 0.06. The natural question is whether a seismic design coefficient of 0.09 or 0.16 would have provided adequate earthquake resistance for the Cypress Viaduct. It is impossible at this date to determine precisely what the outcome of such a design would have been, but it is possible to speculate as follows:

1. The columns would probably have been somewhat larger and stronger to resist bending moments.
2. Similar "hinge joints" would probably have been used.
3. The same inadequacies of reinforcing-bar details would exist in the columns, and the same deficiency in column ties would exist.
4. Seismic resistance up to the point of initial cracking would probably have been increased.
5. The structure still would have failed in a brittle manner if overstressed.

Although no seismic recordings were made at the Cypress Viaduct site during the Loma Prieta earthquake, it is estimated that the peak acceleration of the ground was on the order of 25% g or more. If the structure had retained its integrity, then the estimated horizontal accelerations of the upper deck would have been between 0.5 and 0.7 of the acceleration of gravity. These motions, being much larger than the 0.09 or 0.16 design values, would have cracked column sections and produced column failure in the 0.09 structure, and in the 0.16 structure as well. This could have lead to a collapse similar to that which occurred during the Loma Prieta earthquake, though it might have been less extensive. It is clear that even the 0.16 structure would have been inadequate to survive a moderate to large earthquake on the Hayward fault or on the adjacent portion of the San Andreas fault.

Conclusions

Caltrans has been at the forefront of seismic bridge analysis and design technology and will no doubt continue to be. Their seismic practices will continue to improve if they expand and strengthen their efforts to:

- Implement state-of-the-art analysis procedures and emerging technologies (such as base isolation).
- Instrument bridges to determine how they perform in earthquakes.
- Initiate analytical and experimental research activity to determine how bridges perform in earthquakes.
- Rapidly incorporate these research results in the Bridge Design Specifications.
- Interact with the earthquake engineering community in both the United States and abroad to benefit from what is learned in earthquake engineering for other types of structures.

Chapter 8

The California Bridge Seismic Retrofit Program

Early History

The 1971 San Fernando earthquake provided dramatic evidence that bridges were vulnerable in earthquakes. It showed clearly that some bridges were not seismically resistant—including those that used then-current seismic design requirements, whose construction had just been completed. There were two bridge-related fatalities. Reviews of damage pointed to two major problems in bridge design to resist seismic forces: lack of ductility in concrete bridges owing to improper detailing (by today's standards) of the steel reinforcement; and lack of restraint at abutment and interior expansion joints, which permitted large relative motions of the deck structure to occur (Figures 8-1 and 8-2).

Expansion joints are provided in long structures to accommodate the movements caused by temperature-induced changes

Figure 8-1. *Many bridge columns constructed of reinforced concrete failed during the 1971 San Fernando earthquake because of a lack of ductility.*

Figure 8-2. *The reinforced concrete box girder transition bridge from the westbound I-210 to the southbound I-5 collapsed during the 1971 Fernando earthquake The supporting columns may have given way first or they may have been brought down by the deck spans sliding off their 14-inch seats at the expansion joints*

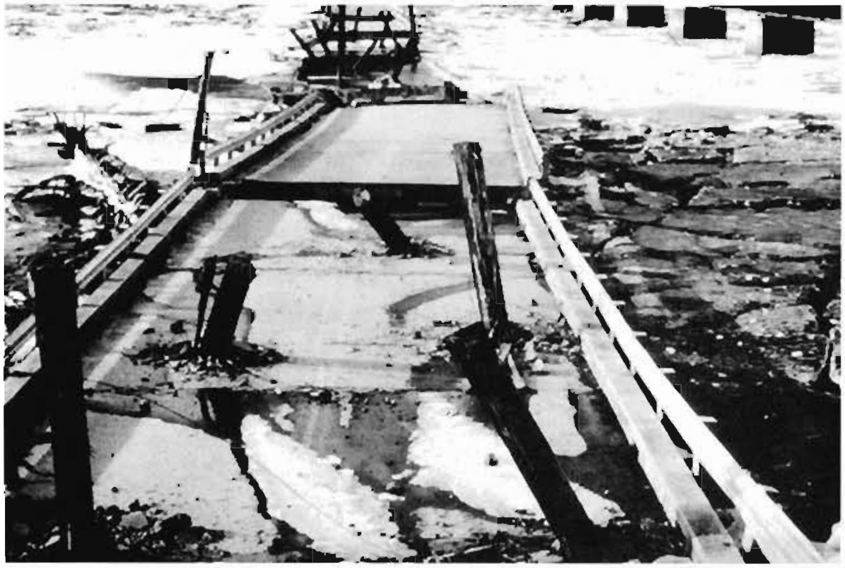


Figure 8-3. *The Twentymile River Bridge was one of several that collapsed during the 1964 Alaska earthquake. Spans of this 825-foot concrete girder bridge fell off their timber pile bents into the water*

Figure 8-4. *One of the bridges damaged in the 1964 Niigata (Japan) earthquake was the 12-span Showa Bridge. Two piers in the middle of the river failed, causing three spans to drop. Two other spans slipped off piers that remained upright*

in structural member length; such joints transfer mainly gravity loads. The relative movements across expansion joints in the San Fernando earthquake were sufficient for some bridges to displace the roadway decks off their seats, causing falling of the no-longer-supported deck, and causing supporting columns to fail. As discussed in Chapter 7, by 1971 Caltrans had not yet incorporated ductile detailing into their design specifications and practices developed through research in the previous decade. Nor had they incorporated experiences in bridge behavior from the Alaska and Niigata earthquakes of 1964 (Figures 8-3 and 8-4), which had revealed many of the deficiencies that were observed in the San Fernando earthquake.

The performance of bridges in the San Fernando earthquake caused revision of Caltrans seismic design practices for new bridges and initiation of a program to correct the seismic deficiencies of existing bridges. Immediately after the earthquake, Caltrans modified their bridge design specifications and procedures to incorporate more strength, ductility and continuity [State of Calif., Dept. of Trans., 1971]. These changes were incorporated into bridges then under design. As a result, bridges built since 1971 are generally considered to be seismically resistant.

Caltrans embarked on a program to retrofit the State-owned bridges to address the hazard posed by the large inventory of pre-1971 bridges. Their interpretation of the damage that occurred during the San Fernando earthquake was that the lack of restraint to the bridge's deck structure allowed excessive relative movement at interior and abutment expansion joints. This interpretation was reinforced by several succeeding earthquakes where damage was attributed to the lack of restraint: the 1980 Ferndale (Figure 8-5) and 1985 Palm Springs earthquakes (Figure 8-6). Caltrans concluded that an efficient and economical procedure to improve seismic performance of the pre-1971 State-owned bridges would be to tie the segments of the bridges together across their expansion joints and to tie the bridges to the abutments. They envisaged the retrofitted deck as a continuous beam in the horizontal plane which would transfer much of the earthquake load into the abutments and not to the columns. In this

Figure 8-5. *Two spans of the Fields Landing Overhead, which had not been fitted with joint restrainers, slipped off their narrow seat supports during the 1980 Ferndale earthquake*

manner the displacements of the columns would be limited and the columns protected from lateral displacements beyond their capacity [Degenkolb, 1978; State of Calif., Dept of Trans., testimony Mar. 15, 1990].

Caltrans developed several devices [Degenkolb, 1978; Mancarti, 1984] to accomplish the retrofit. They used either steel cables or bars passing through a joint and attached to each side with special anchorages (Figure 8-7). Some slack was built in that permitted thermal movements, but large openings, sufficient to move one section off the bearing seat of the second during an earthquake, were intended to be restrained by tensile forces developed in the cables or bars. These devices were designed using calculations; laboratory tests were used only to determine the stress-strain relations of the cables and bars [Degenkolb, 1978].

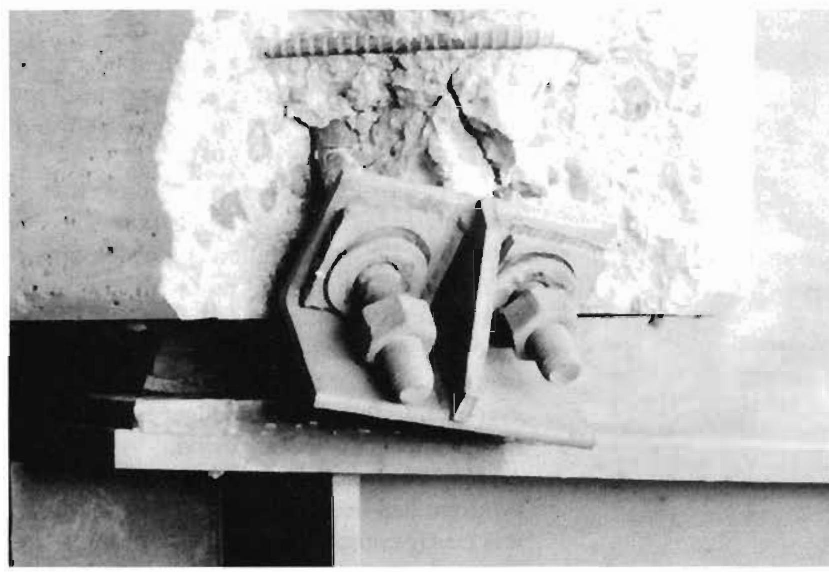


Figure 8-6. *Although the Whitewater Overcrossing had been retrofitted with cable restrainers at the abutments, an improperly connected cable caused the restrainer at one abutment to fail during the 1985 Palm Springs earthquake. This permitted a movement of several inches to occur, which damaged the abutment seat support.*

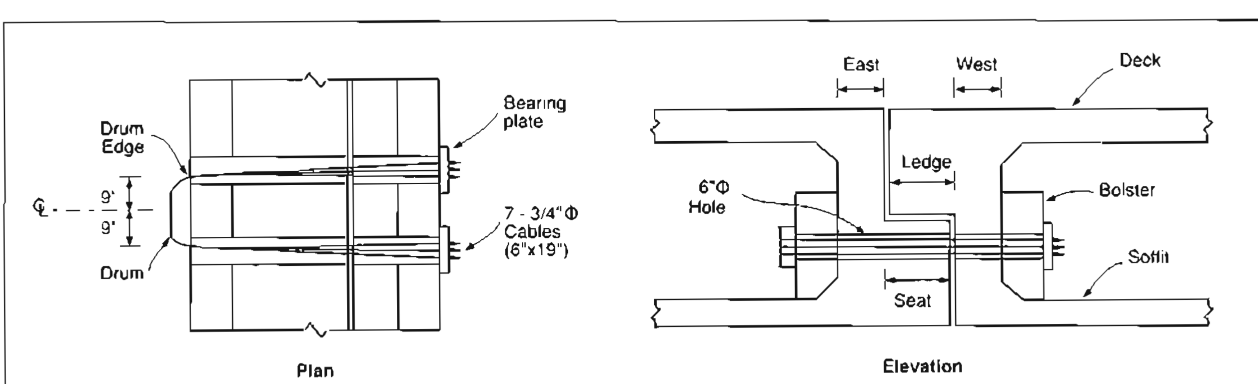


Figure 8-7. *This diagram shows an example of a joint restrainer device that Caltrans developed after the 1971 San Fernando earthquake for the purpose of keeping bridges together at their expansion joints Tests on this particular device carried out 12 years after the retrofit program began showed that it failed by pulling through at the diaphragm, rather than in the preferred mode of cable elongation*

Vulnerable bridges were identified by a State-wide survey; joint restrainers were installed on the Cypress Viaduct in 1977 and on the Bay Bridge in the 1970s.

The retrofit program was not given a particularly high priority by Caltrans. It took until 1987 to complete, by which time 1,261 bridges had been fitted with restrainers at a cost of $54 million. This was a relatively modest level of expenditure compared to the overall budget of the agency over the period, even noting the extreme budget reductions in effect for part of that period.

In 1984, after the joint restrainer program had been underway for 12 years and was nearly complete, Caltrans initiated a series of experiments at UCLA to determine how the restrainers behaved [Selna et al., April 1989; Selna et al., Sept-Oct. 1989]. The main experiments were done at full scale on a portion of a reinforced concrete box girder bridge that consisted of an expansion joint and an appropriate length of the deck structure on each side. Three types of restrainers were tested, and, while all reached their design load, two failed in the anchorages rather than in the desired, more ductile mode of cable or bar elongation. Another experiment showed that cable ductility was considerably reduced because of a stress concentration where it wrapped around the edge of a drum support. These experiments, whose total budget was $191,000, indicated that there were some technical problems with the joint restrainers, and designs were subsequently improved. Regarding the earlier retrofits using the older design, Caltrans expressed to the Board that they were aware of the deficiencies in some of the early installed restrainers and have stated that their upgrade is part of the ongoing retrofit program [State of Calif., Dept of Trans., testimony Jan. 18, 1990].

Caltrans' viewed the joint restrainers as sufficient to prevent bridge collapse for most pre-1971 bridges. A 1985 paper [Zelinski, 1985] by a Caltrans' retrofit expert stated:

> It should be noted that column retrofits have been avoided by designing restraining features in a manner which circumvents collapse due to column damage. Review of behavior of retrofitted bridges during earthquakes will determine whether features will need improvement and whether columns need to be considered for retrofitting.

Material presented to the Board supported this view, although it also revealed that Caltrans had a continuing concern about the seismic capacity of nonductile columns in pre-1971 bridges. However, this appears to have been more a concern with serviceability than with collapse, which explains the low priority that column upgrading received in the seismic retrofitting plan. A 1978 paper by a Caltrans engineer [Degenkolb, 1978] described several schemes for retrofitting columns: wrapping with prestressed steel hoops or wire that is covered with a protective coat of shotcrete, and jacketing with a welded steel shell infilled with grout. In 1979, a State-wide survey of single column State-owned bridges was made. About 780 bridges were identified that might have

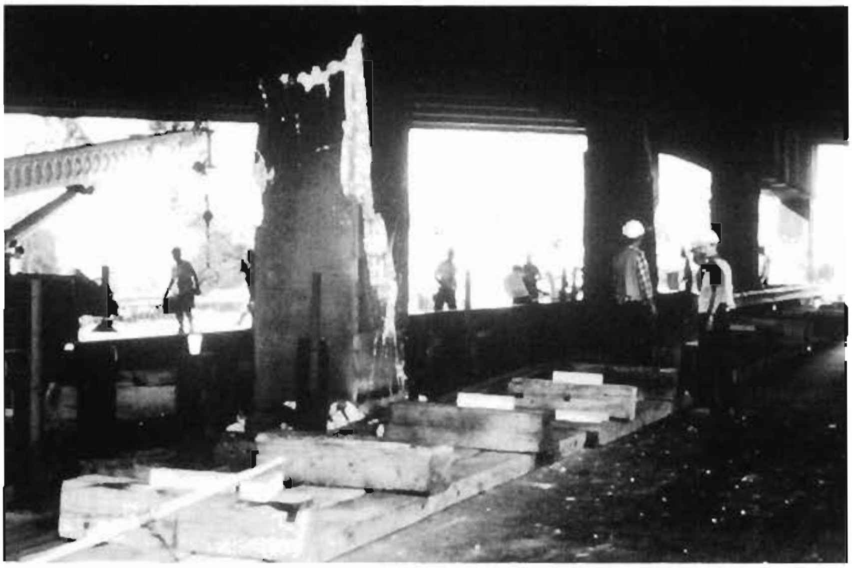


Figure 8-8. *The near collapse of a 5-column bent supporting the I-605 crossing over the I-5 during the 1987 Whittier Narrows earthquake provided the impetus to initiate the state-wide program to retrofit bridge columns and other supporting elements*

required retrofitting. A few years later, Caltrans began participation in a multi-agency funded experimental program at the National Bureau of Standards (now the National Institute of Standards and Technology) on large scale testing of bridge columns; their contribution was $180,000. This program, which began in 1983, encountered difficulties, and a planned series of tests on retrofitted columns was never carried out [Sate of Calif., Dept. of Trans., testimony Jan. 18, 1990]. In 1987 Caltrans agreed to fund research on column retrofitting proposed by researchers from U.C. San Diego [State of Calif., Dept. of Trans., correspondence file on U.C. San Diego column research]. A desired 6-year program at a total cost of $940,000 was reduced in funding and scope (to $386,000 over 3 years) because Caltrans could not justify the high research cost, since it did not have financial authority to embark on a program of column retrofit [Priestley, et al., March 1988]. Ultimately, the $386,000 came from a Federal Highway Administration grant to Caltrans.

Recent Events

The Whittier Narrows earthquake of October 1987 was a significant event for the bridge retrofit program, even though it was only a Magnitude 5.9 earthquake. A five-column bent supporting the crossing of the I-605 freeway over the I-5 freeway nearly collapsed (Figure 8-8) during the morning commute. Subsequent analysis indicated that membrane tension in the deck provided by the joint restrainer cables provided the margin of seismic capacity by which collapse was averted [Priestley, et al., May 1988]. Caltrans agrees that only slightly more shaking would have degraded the columns enough for complete failure to occur [Roberts, testimony Mar. 15, 1990]. This was hard evidence that joint restrainers are not sufficient by themselves to prevent collapse of some bridges supported on nonductile columns, even for a modest magnitude earthquake and relatively short duration of moderate ground shaking.

Caltrans reacted quickly to this near tragedy. In December 1987, they set aside funding of $64 million for a column retrofit program. The recently initiated research program at U.C. San Diego was expanded in scope to a total budget of $741,000 and extended a year to 1991 [State of Calif., Dept. of Trans., correspondence file on U.C. San Diego column research]. Another project at U.C. San Diego [State of Calif., Dept. of Trans., correspondence file on U.C. San Diego I-605/I-5 overcrossing research] was funded to investigate the I-605/I-5 separator ($148,000). Money for both these research programs again came from Federal Highway Administration grants to Caltrans. Even though the I-605/I-5 separator had multiple column bents, the increased efforts were directed toward bridges with single column bents. Because of their lack of redundancy, Caltrans viewed these as even more of a risk than those with multi-column bents [State of Calif., Dept. of Trans., testimony Dec. 14, 1989].

Caltrans identified about 400 single column bridges as candidates for retrofit under the new program. They used selection and prioritization procedures developed in-house over the preceding years. A design methodology using the steel shell jackets was developed, based partly on preliminary results from the U.C. San Diego research program. As described in Caltrans' Memo to Designers 20-4 [State of Calif., Dept. of Trans., Nov. 28, 1989(c)], a typical retrofit design would provide fixity only at the bases of selected columns. Jacketing and increases in the moment capacity of the footing (expensive) were only to be used where deemed necessary for stability of the bridge. The other columns would be jacketed as necessary to maintain axial and shear integrity only, permitting a pinned condition to develop at the base. Full height jackets would be employed only if shear failure was considered possible. The retrofit design process uses linear response spectrum-type analyses, with several steps required to arrive at appropriate fixity conditions and to account for open and closed conditions at the joints. This process is somewhat ad hoc; development of full nonlinear analyses procedures has been supported by Caltrans but has not yet reached the practical stage.

The single column retrofit program was just getting underway when the Loma Prieta earthquake occurred. The tragic collapse of the Cypress Viaduct reiterated the vulnerability of bridges with nonductile columns and other supporting elements; this time nonductile joints played an important role in the failure. Damage to five other double-deck viaducts in San Francisco caused them to be closed after the earthquake. Caltrans is collaborating with independent consultants on the design of repairs for these and one other viaduct (see Chapters 11 and 12) at a total estimated repair cost of over $100 million. A series of tests was carried out on a standing segment of the Cypress Viaduct (see Chapter 10) by Caltrans and researchers at U.C. Berkeley at a cost of $3 million. The tests were conducted to evaluate various column and joint retrofit schemes. They were intended to guide and validate the repair methodologies being used on the San Francisco Freeway Viaducts. Results of these tests are discussed in Chapter 10.

Heightened concern about bridge safety after the Loma Prieta earthquake has prompted several State-wide retrofit programs [State of Calif., Dept. of Trans., February 6, 1990 and Feb. 7, 1990]

- Caltrans has proposed an accelerated schedule for retrofitting the 392 bridges that have single column bents (Table 8-1). They now place the total cost of this program at $142 million.
- Caltrans has put the rest of the State-owned bridges on the retrofit agenda (Table 8-1). Current estimates are that 700 bridges, mostly ones with multi-column bents, will require retrofit at a cost between $200 million and $300 million. To support this research effort, a Caltrans-funded research program is beginning at U.C. Berkeley that will examine retrofit strategies for bridges with multi-column bents.
- Through Senate Bill SB36X, the State has mandated a seismic retrofit program for locally-owned (city and county) bridges (Table 8-1). Caltrans will imple-

Table 8-1. *Summary of State-wide retrofit program*

	No. of Bridges	Total Cost	Start Date	End Date
State-Owned Bridges				
Joint Restrainers	1,261	$54 million	1971	1987
Single-Column Bents	392	$142 million	1990	1994
Multi-Column Bents	700	$200-$300 million	1990	1994
City and County Bridges	1,500	$75-$100 million	1990	1994
Total	**3,853**	**$471-596 million**		

ment this program, except in Santa Clara and Los Angeles Counties, where the county governments will be in charge. An initial phase will install joint restrainers and retrofit bridges with single column bents, where needed, with the remaining bridges to follow. Preliminary figures from Caltrans indicate that 1,500 bridges will require retrofit at a total cost of $75 to $100 million.

These programs represent a total new commitment of about $500 million to the State-wide seismic retrofitting program of State- and locally-owned bridges. The program is planned for completion by 1994. This is an ambitious undertaking, particularly noting that historical expenditures have been modest by comparison both in total and annual rates. Appropriations for the entire program have yet to be made; SB36X contained $60 million for retrofitting State bridges and $20 million for the local effort, far short of the estimated cost.

Conclusions

The succession of earthquakes from 1971 San Fernando through 1989 Loma Prieta have clearly shown that pre-1971 bridges can be very vulnerable to earthquake damage. Caltrans' early assumption that joint restrainers were sufficient to prevent collapse under strong shaking was incorrect, as least for some bridges. The near collapse of the I-605/I-5 overcrossing during the moderate 1987 Whittier Narrows earthquake and the collapse of the Cypress Viaduct during the 1989 Loma Prieta earthquake made this point very clearly. Yet, joint restrainers may have been the effective way to spend the amount of money available to improve seismic safety. In testimony before the Board, Caltrans stated that the joint restrainers prevented the spans of many bridges from collapsing during the Loma Prieta earthquake [State of Calif., Dept. of Trans., testimony Mar. 15, 1990], and the Board finds this view plausible.

On the other hand, it may not have been wise to expect joint restrainers to prevent collapse for each of the great number of bridge types on which they were installed. Consider the Cypress Viaduct. While the restrainers were effective in keeping the decks on their seats, they could not be expected to reduce the earthquake loads to the columns for such a long structure. Little of the earthquake-induced loads could be transferred to the abutments, since the abutments were so far apart (over 1.5 miles). Recognition of this would have forced a closer examination of the structure, which may have revealed the inadequacies that caused it to collapse during the Loma Prieta earthquake.

The pace of the bridge retrofit program prior to the Whittier Narrows earthquake was slower than it should have been. While few would advocate diverting traffic safety improvement funds to earthquake retrofit, with 5,000 traffic fatalities per year on California's highways, a good case could be made that the average of $3.5 million spent per year (1972 to 1987) to retrofit obviously hazardous bridges was too small a portion of the average annual budget for new construction. The issue, however, is moot since the joint restrainer program finished before any major earthquake struck and since Caltrans may not have embarked on a column retrofit program any sooner than it did (following the Whittier Narrows earthquake), being of the opinion that joint restrainers were sufficient to prevent bridge collapse.

Caltrans put off for too long initiation of research on the retrofit program, and then it was too narrowly focused. Their major research effort on joint restrainers began in 1984, 12 years after the first restrainers were installed. This research showed the devices to be much less ductile than previously thought. Although no earthquake damage, including Whittier and Loma Prieta, has demonstrated any difficulty with joint restrainers, it is not clear whether many have been required to behave in a ductile manner during earthquakes, which may have revealed problems. The research program at U.C. San Diego on column strengthening and the one beginning at U.C. Berkeley on multi-column bents are crash programs driven by the urgent needs imposed by the post-Loma Prieta retrofit program. Conducting research under such conditions is not ideal. Had research programs been supported over the years, even at modest levels, with some portion reserved for basic research, it is likely that safer and more economical retrofit strategies would now be available. Caltrans should recognize the value of continuing basic and applied research efforts on earthquake engineering topics and make commitments to long-term knowledge development.

Caltrans appears to be on the right track regarding the current retrofit efforts, although the Board does not have the resources to properly evaluate the retrofit design procedures being used or proposed. Because of the large amount of money anticipated to be spent on retrofit of bridges during the next five years, it would be prudent to immediately initiate an external technical review of all retrofit design procedures and pertinent results from experimental programs as they become available. The Board believes that every bridge retrofit should consider the behavior of the entire structural system, not just individual elements.

The Loma Prieta earthquake, as devastating as it was to the Cypress Viaduct, San Francisco Freeway Viaducts, and the Bay Bridge, did not have ground shaking at these sites that was nearly as intense or as long as is expected in other highly likely earthquakes for the region. The Board cautions that there is still lack of experience for bridge behavior during the very strong and long duration shaking that would result from a major earthquake. The long-term process of understanding the impacts of earthquake ground motions has just begun. Research and experience have much yet to teach on how to design and construct new bridges and upgrade existing ones.

Chapter 9

San Francisco-Oakland Bay Bridge

History of the Bay Bridge

At least since 1851, when William Walker, editor of the San Francisco Herald, proposed that a causeway and pontoon bridge be built to the Oakland shore, visionaries dreamed of a bridge spanning the Bay to join San Francisco with the East Bay.

In 1928, President Herbert Hoover and California Governor C.C. Young jointly appointed a commission to make preliminary studies and recommend "a solution of the state and interurban traffic needs between the counties of San Francisco and Alameda across San Francisco Bay, reconciling these with the needs of national defense and the national interests of navigation" [Dillon, 1979].

In August 1930, the Hoover-Young Commission issued its report. The report recommended the current Bay Bridge location, from Rincon Hill in San Francisco via Yerba Buena Island to San Antonio estuary on the Oakland shore. The major considera-

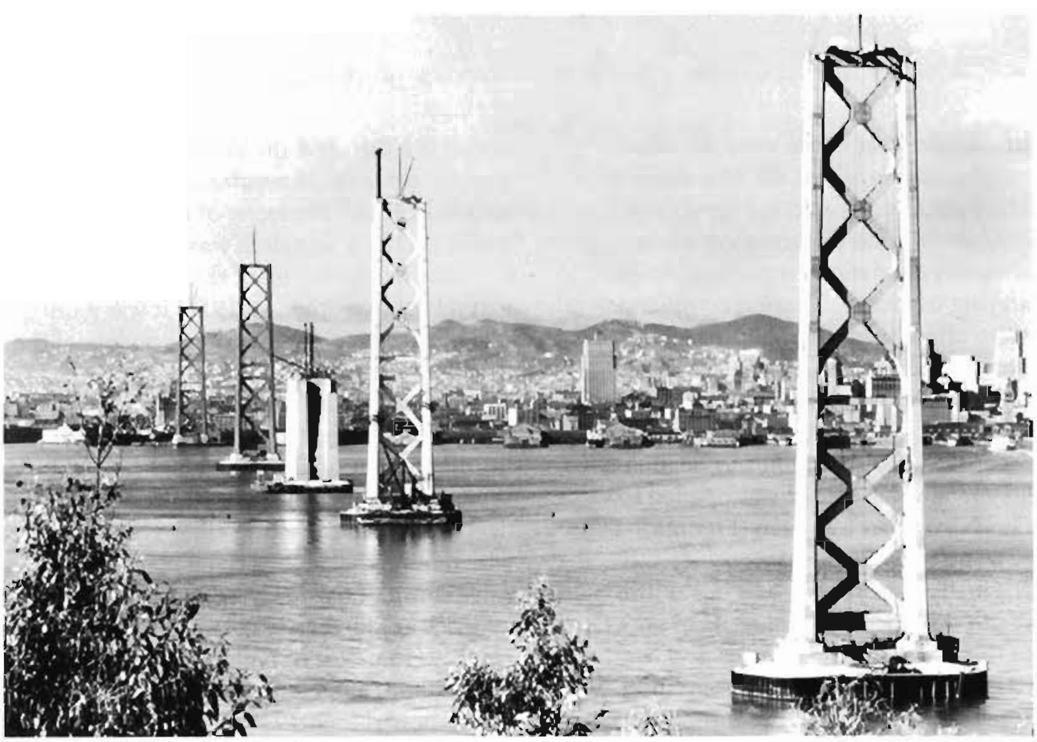


Figure 9-1. *West Bay Crossing towers before construction of cables and deck spans*

Figure 9-2. *Hanging platforms used to construct cables of West Bay Crossing.*

tions in choosing the route were the adequacy of the soil on which the foundations would be built, its engineering feasibility, and economics. The Commission made several recommendations regarding design and carrying capacity. These recommendations were effectively adopted through enactment of Chapter 9, Statutes of 1933, which authorized the Department of Public Works to construct a bridge across the San Francisco Bay. Construction was begun on July 9, 1933 and the bridge was opened to traffic on November 12, 1936 (Figures 9-1 and 9-2).

Originally, automobile and bus traffic ran in both directions on the six-lane upper deck of the structure. The lower deck was reserved for trucks and electric commuter trains operated by the Key System. In 1958, the Key System discontinued train runs across the Bay, and the present configuration of five lanes of westbound traffic on the upper deck and five lanes of eastbound traffic on the lower deck was initiated.

The bridge is one of the engineering marvels of our time. In 1956, it was named one of the seven engineering wonders of the world by the American Society of Civil Engineers. At the time it was built, it was the world's longest steel structure, consuming 18 percent of all the steel fabricated in the United States in 1933. It included the deepest pier in the world (Pier E3, 235'), which nevertheless had to be planted on a substratum of hard clay and sand because bedrock at 300 feet, was impossible to reach. Total cost of building the bridge was $78 million, a phenomenal sum in the depression-ridden 1930s.

Figure 9-3. *Aerial view of the San Francisco-Oakland Bay Bridge.*

Alignment, Structural Configuration, and Foundation Conditions

The San Francisco-Oakland Bay Bridge, shown in the aerial photograph of Figure 9-3, consists of two sections—a West Bay Crossing from San Francisco to Yerba Buena Island and an East Bay Crossing from Yerba Buena Island to Oakland. The alignment of these crossings, with the connection on Yerba Buena Island, is shown in Figure 9-4. The total distance along the alignment from the San Francisco anchor of the West Bay Crossing to Pier E39 of the East Bay Crossing is 4.35 miles. The bridge is of double-deck design, and the deck widths vary from 60' to 72'.

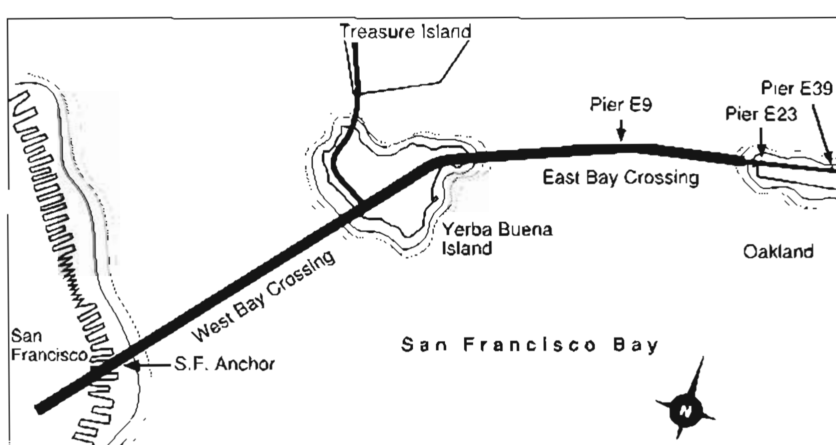


Figure 9-4. *Plan view of San Francisco-Oakland Bay Bridge alignment.*

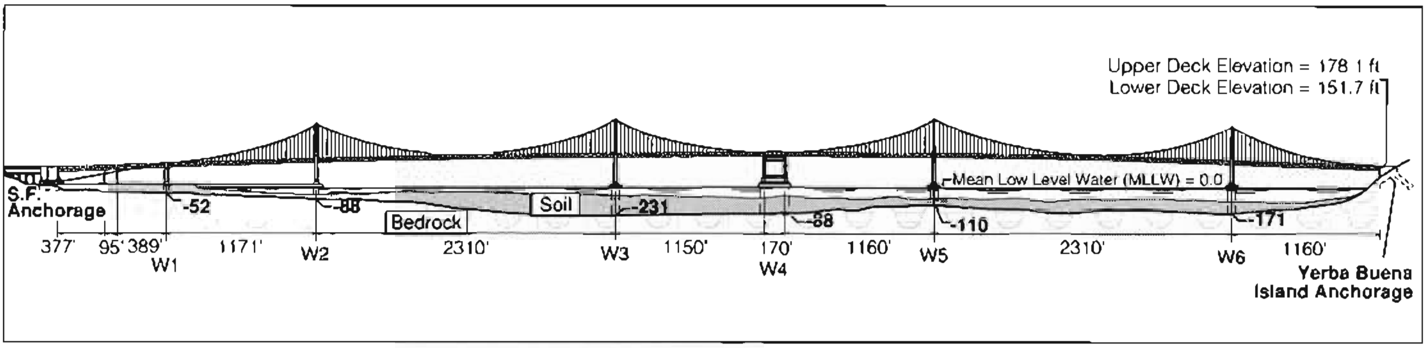


Figure 9-5. *West Bay Crossing.*

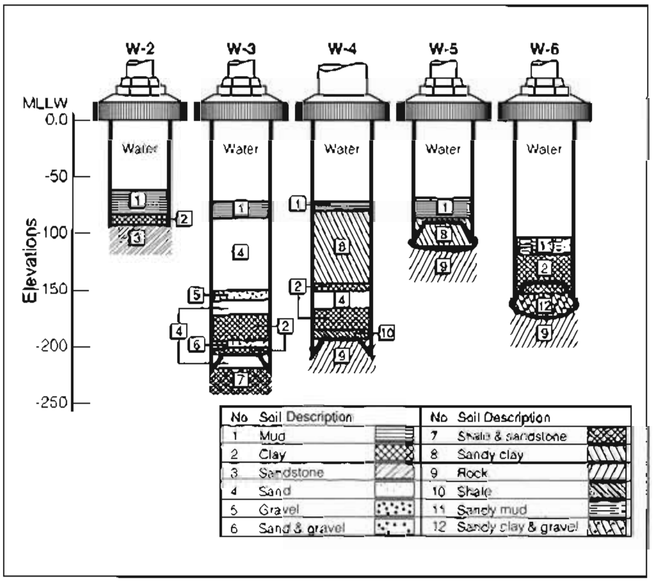


Figure 9-6. *Soil conditions under West Bay Crossing at Pier locations W-2 through W-6*

West Bay Crossing

The West Bay Crossing consists of a twin suspension structure (Figure 9-5). Its total length from the San Francisco anchorage to the Yerba Buena Island anchorage is 1.95 miles. Both anchorages and the main supporting piers, W1 through W6, are founded on rock (Franciscan sandstone). The piers extend vertically through mean low-level water depths and soil layers as shown in Figure 9-6. The surface soil layer of mud consists generally of a comparatively soft silt that ranges in depth from approximately 10' to 20'. Below the mud layer are strata of consolidated clays, sands, gravels, and shales.

East Bay Crossing

The East Bay Crossing (Figure 9-7) consists of four shallow simple-span trusses on Yerba Buena Island from Pier YB1 to Pier E1, a long cantilever truss structure from Pier E1 on the Island to Pier E4 in the Bay, five deep simple-span trusses from Pier E4 to Pier E9, fourteen shallow simple-span trusses from Pier E9 to Pier E23, and a number of simple-span deck systems that use steel and concrete stringers supported on transverse concrete bents from Pier E23 to Pier E39. Among the many determinants of this layout were the navigation requirements of the U.S. War Department, which resulted in the 1400-foot span immediately to the east of Yerba Buena Island having a vertical clearance of 185' above mean higher high water level and the three contiguous 510' spans to the east each with a minimum clearance of 165'. For the remaining portion of the bridge to the east, which is over comparatively shallow water, the principal factor was that of balancing costs of superstructure and substructure so as to secure the greatest economy. The total length of the East Bay Crossing from Pier YB1 to Pier E39 is 2.14 miles.

To provide anchorage against longitudinal forces in the bridge superstructure system, the bridge was designed with the following elements:

- Two simple-span trusses from YB1 to YB3 are pin-connected into common fixed shoes at YB2 (one on the north side, the other on the south side) and anchored by fixed shoes at YB1.
- Two simple-span trusses from YB3 to E1 are pin-connected through common fixed shoes at YB4 and anchored by fixed shoes at E1.
- The entire cantilever truss system from E1 to E4 is anchored at E1.
- Five deep simple-span trusses from E4 to E9 are pin-connected through common fixed shoes at E5, E6, E7, and E8 and anchored at E9.

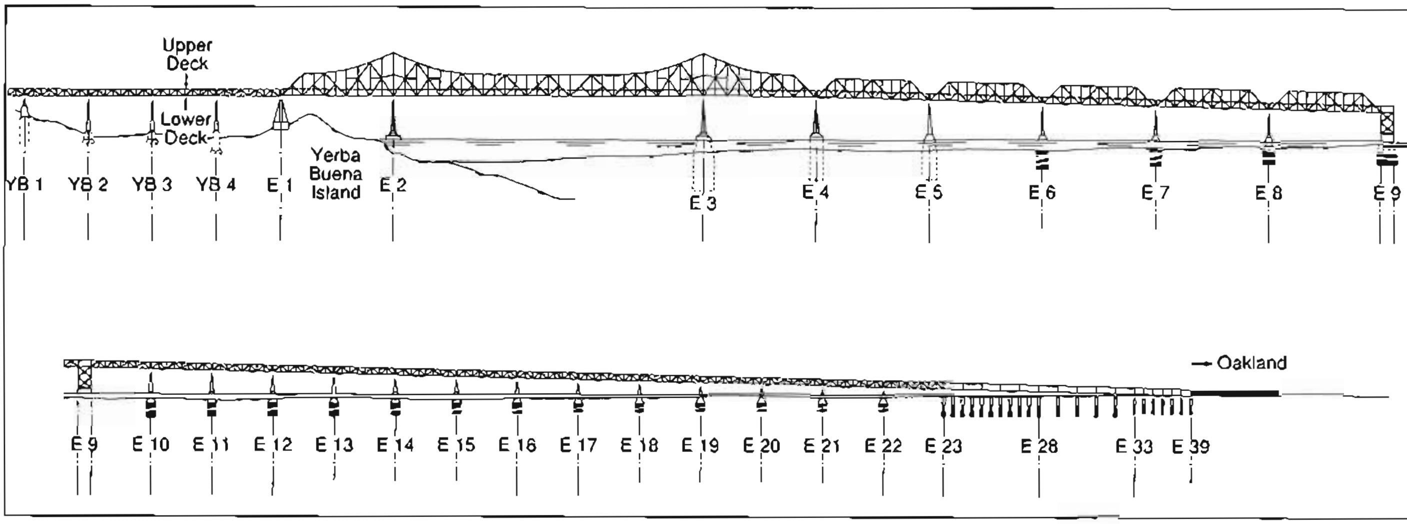


Figure 9-7. *East Bay Crossing.*

- Two simple-span trusses from E9 to E11 are pin-connected through common fixed shoes at E10 and anchored by fixed shoes at E9.
- Six simple-span trusses from E11 to E17 are pin-connected through common fixed shoes at E12, E13, E14, E15, and E16 and anchored by fixed shoes at E17.
- Each of the six simple-span trusses from E17 to E23 is anchored by fixed shoes at its east end and is provided with roller type expansion shoes at its west end.

The pier supporting structures at anchor point locations YB1, E1, E9, E17, E18, E19, E20, E21, E22, and E23 consist of the following (Figure 9-8):

1. Tall reinforced concrete piers at locations YB1 and E1, which are founded on bedrock at the approximate elevations +4' and +30', respectively. (All elevations are given relative to the mean low-level water surface.)
2. Reinforced concrete piers at locations E17, E18, E19, E20, E21, and E22, all of which are founded at approximately -45' elevation on timber piles driven to elevations in the approximate range -115' to -120'.
3. A reinforced concrete bent at location E23, which is founded at approximately -45' on timber piles driven to approximately -115' elevation.
4. A rigid steel tower at location E9 which rests on a reinforced concrete pier at elevation +25', which in turn is supported at elevation -45.3' on timber piles driven to elevation -122.2'.

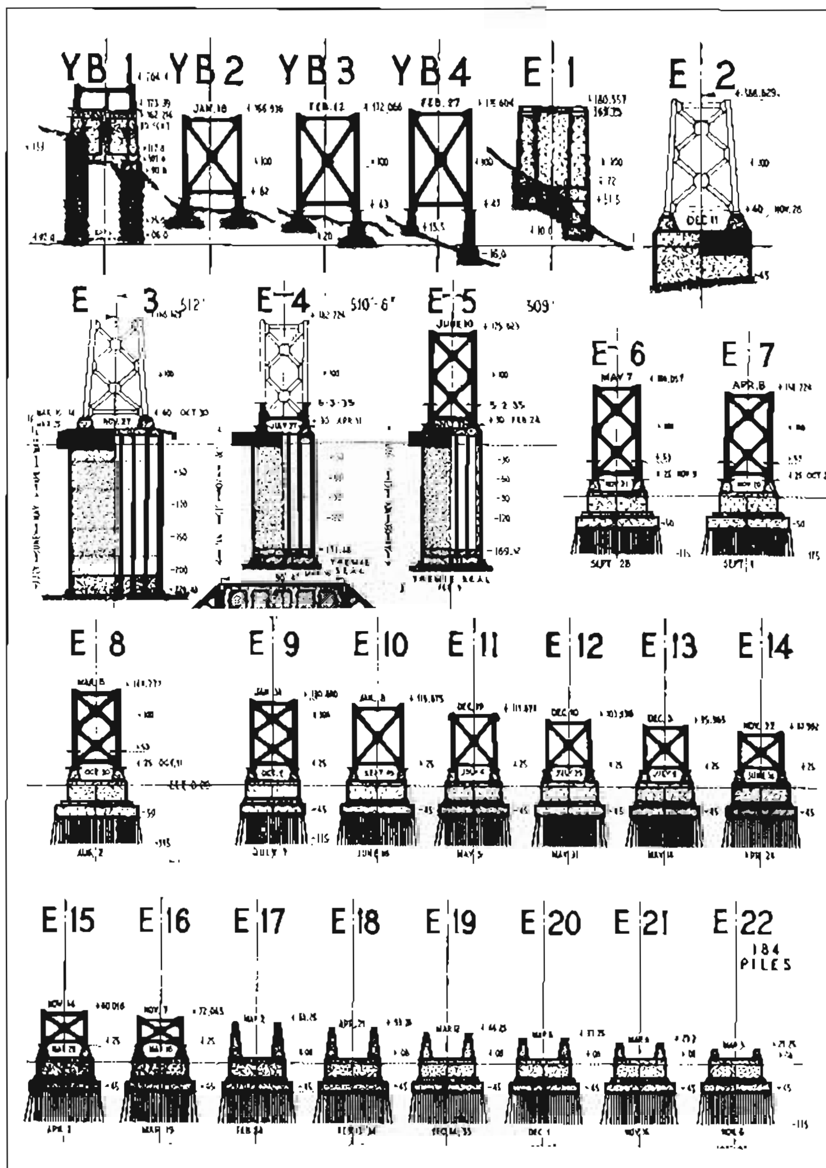


Figure 9-8. *East Bay Crossing support structures from Pier YB1 through Pier E22.*

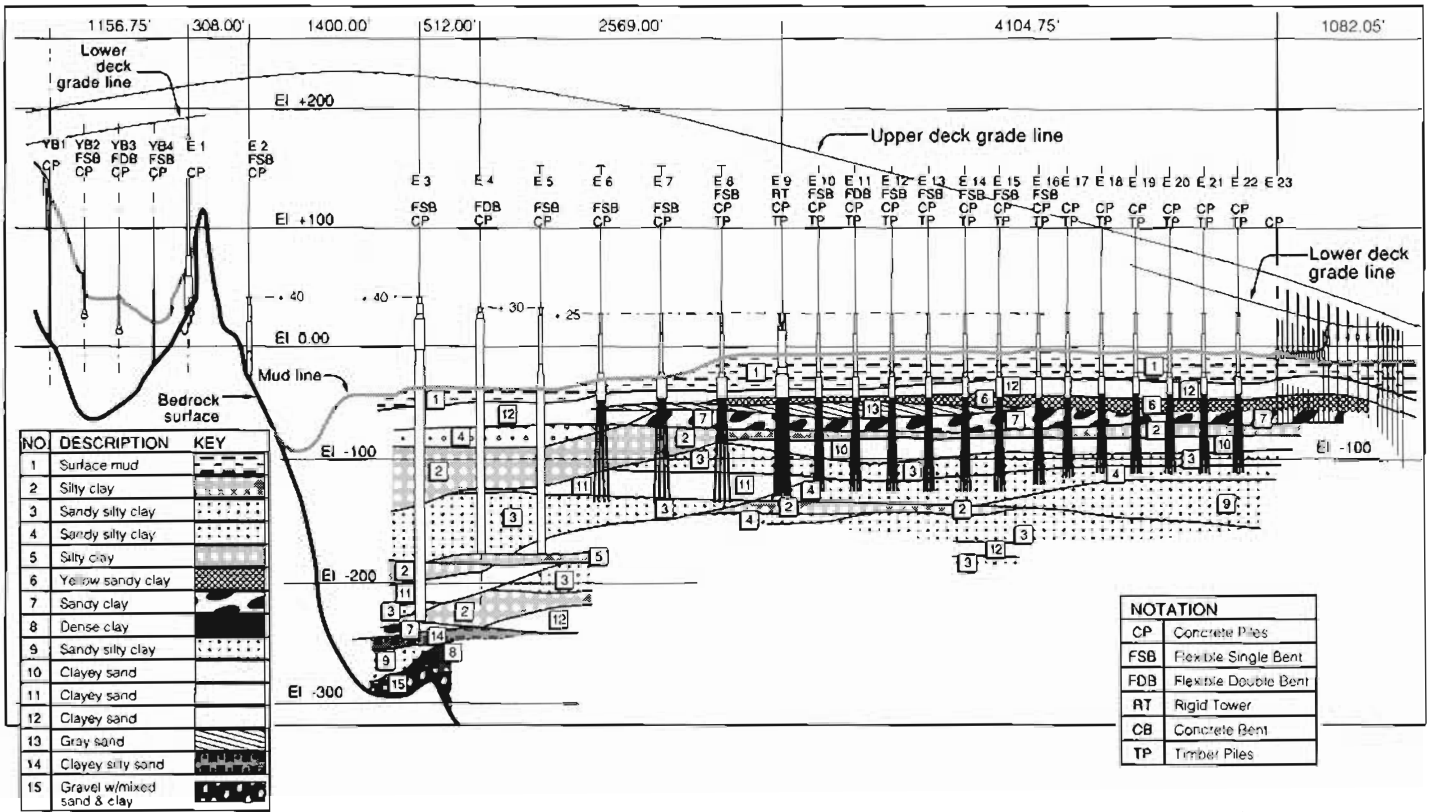


Figure 9-9. *Soil profile and support systems of the East Bay Crossing.*

The pier supporting structures at locations YB2 through YB4, E2 through E8, and E10 through E16, which provide very little longitudinal constraint to the superstructure, consist of the following:

1. Single steel diagonally-braced bents, aligned in the transverse direction, at locations YB2, YB4, E2, E3, and E5. These are founded at the approximate elevations +62, +47, +40, +40, and +30', respectively, on concrete piers at locations YB2, YB4, and E2 and on concrete caisson piers at locations E3 and E5. The concrete piers at locations YB2, YB4, and E2 extend down to the approximate elevations +25.0, -16.0, and -45', respectively, while the concrete caisson piers at locations E3 and E5 extend down to the approximate elevations -235' and -177', respectively. The concrete piers at locations YB4 and E2 are founded on bedrock, while the concrete piers at location YB2 are founded in sand. Both concrete caisson piers at locations E3 and E5 terminate in strata of sand and gravel.
2. Single steel diagonally-braced bents, aligned in the transverse direction, at locations E6, E7, E8, E10, E12, E13, E14, E15, and E16, all of which are founded at the approximate elevation +25' on concrete piers, which, in turn, are supported at elevations ranging from -50' to -45' on timber piles driven to elevations ranging from -125 to -114'.
3. Double steel diagonally-braced bents, aligned transversely and placed side by side in the longitudinal direction at locations YB3, E4, and E11, which are founded at the approximate elevations +43, +30, and +25', respectively. Two concrete piers at location YB3 terminate in sand at the approximate elevations +20' and zero, a concrete caisson pier at location E4 terminates at the approximate elevation -177.8' in soil, and a concrete pier at location E11 founded at elevation -44.9' on timber piles driven through soft sediments to the approximate elevation -115.9'. These double-bent supports provide for longitudinal movements arising from temperature change and stress. The lengths of the bridge segments contributing to the movements at these locations are 0.22, 0.94, and 0.44 miles, respectively.

Figure 9-8 shows the supporting structures of the East Bay Crossing described above. Figure 9-9 shows their positions relative to the soil profile. Note that because the bedrock surface is very deep east of Pier E2, most of the bridge substructures terminate in the soils. Figure 9-10 illustrates schematically the support conditions provided by the supporting structures shown in Figure 9-8. Note that all supporting struc-

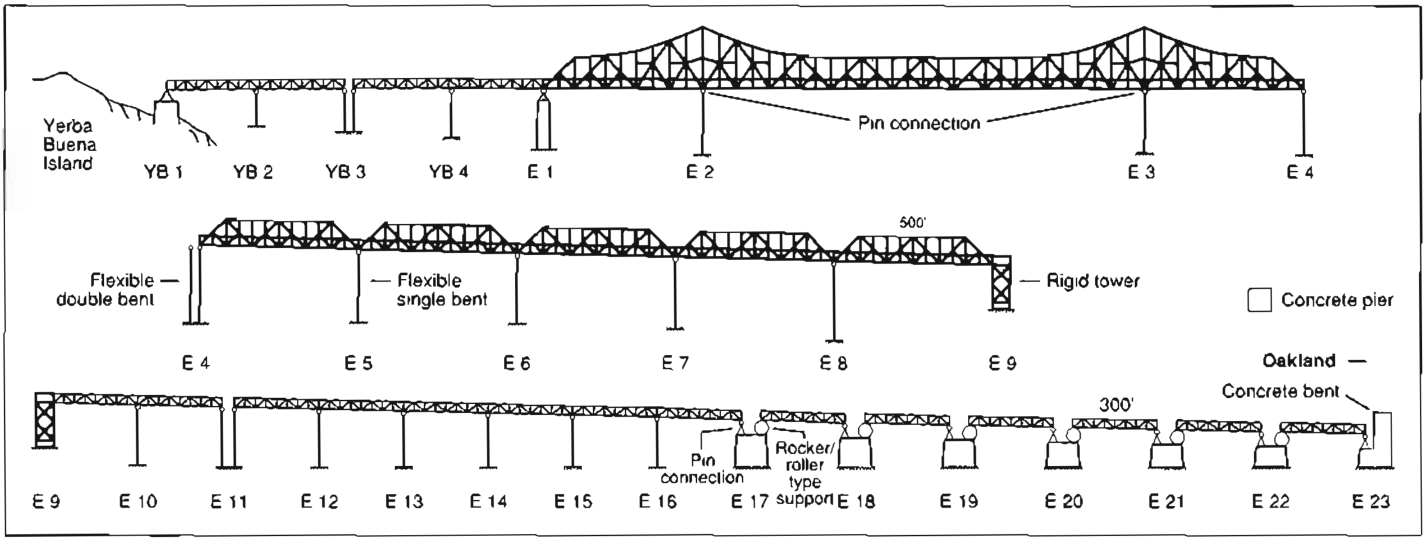


Figure 9-10. *East Bay Crossing support conditions.*

tures are very rigid in the transverse direction; thus, each one carries nearly all of its own tributary transverse loading of the superstructure. As previously pointed out, the longitudinal loadings from Pier YB1 to Pier E23 are carried almost completely by the anchor piers at locations YB1, E1, E9, E17, E18, E19, E20, E21, E22, and E23.

Seismic Retrofit of the East Bay Crossing

Seismic retrofits were installed in the East Bay Crossing of the San Francisco—Oakland Bay Bridge in 1976 as part of the Caltrans expansion joint retrofit program. Retrofit consisted of the following:

1. Rods and tiedowns were installed near the east ends of the concrete stringer spans at the lower deck level of all bents E23 through E27.
2. Rods were installed near the east ends of the steel stringer spans at the upper deck level at bent locations E25, E27, E29, and E31.
3. Rods and tiedowns were installed at the east ends of the concrete stringer spans at the upper deck ramp level of all bents E34 through E38.
4. Steel restrainers incorporating elastomeric pads were installed at all expansion-shoe locations E17 through E22.

Damage only occurred to those areas where retrofit measure Number 4 was used. Figure 9-11 illustrates the basic mechanism of retrofit measure Number 4. It consists of a steel restrainer beam (21" WF 68 lbs/ft) in the transverse direction flexibly attached to two rigid anchor fixtures through four one-inch diameter steel (A325) rods placed at

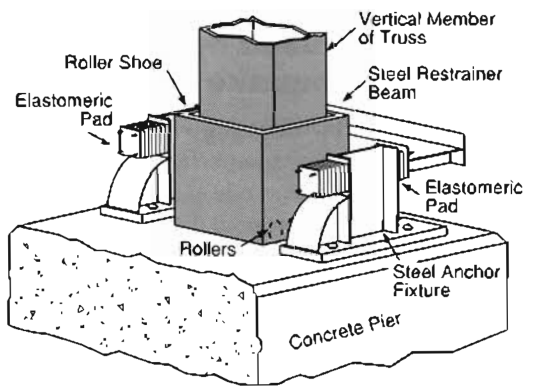


Figure 9-11. *Restrainer retrofit installed on the expansion sides of Piers E17 through E22, rollers are at the base of the vertical member of the truss within the roller shoe.*

each end of the restrainer beam. Elastomeric pads were installed as shown to cushion the bolt forces exerted to the rigid anchor fixtures. When initially installed, a clearance of approximately 3-4" was provided between the restrainer beam and the roller shoe. Consequently, the roller shoe can freely move eastward 3-4" relative to the concrete pier before coming into contact with the restrainer beam. A larger relative eastward displacement will engage the restraining mechanism. Note that this mechanism provides no constraint to relative movements in the westerly direction. The intent of the restrainer system is to prevent the roller shoe from moving off its support in the easterly direction. A fixed shoe for the adjacent span to the west is attached to the pier immediately to the west of the roller shoe. Keeper plates are provided on the base plates to prevent relative movements of the shoes in the transverse direction.

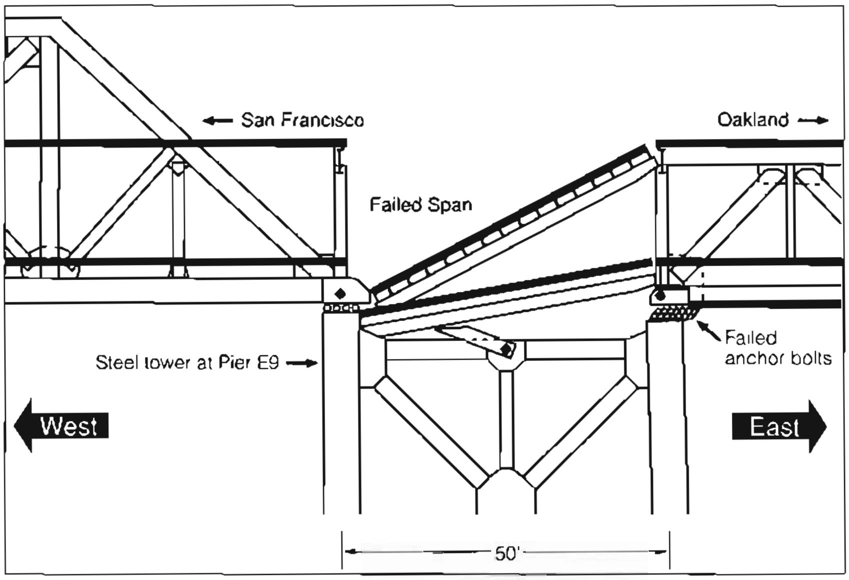


Figure 9-12. *Positions of upper and lower decks at Pier E9 after the earthquake*

Figure 9-13. *Aerial view of collapsed upper and lower decks.*

Damage Caused by the Loma Prieta Earthquake

The West Bay Crossing—the suspension section—had very superficial damage: a crack in the concrete at one pier, a crack in one of the east collars around the cable at center span, and minor damage to some outer expansion fingers at the expansion joints. The East Bay Crossing had one link span fall at both roadway levels, causing the bridge to be closed for one month. Damage to the East Bay Crossing focused on the fallen span, on failed bolts attaching the deck trusses to their piers, and on fracturing of concrete and inelastic elongation of reinforcing bars at the bases of the pedestals of Pier E17.

Collapse of Upper and Lower Decks at Pier E9

During the earthquake, the upper and lower decks of the bridge immediately above Pier E9 collapsed as shown in Figure 9-12. As previously described, E9 serves as an anchor pier in the longitudinal direction for the entire bridge superstructure from Pier E4 to Pier E11, a distance of 3,176'. Therefore, when compared with each of Piers E4 through E11, Pier E9 is very rigid and capable of carrying very large loads. The steel tower and the top of the concrete pier at E9 are visible in the aerial photograph of Figure 9-13. The plan dimensions of the tower are 50.0' by 66.0' in the longitudinal and transverse directions, respectively, as measured center-to-center of the columns.

Figure 9-14 shows the as-built structural arrangement at the top of the steel tower at Pier E9. The two (north and south) shallow trusses on the Oakland side are pin-connected to fixed shoes bolted to support plates rigidly attached to the upper ends of the two east-side columns of the tower. These fixed shoes must carry the resultant longitudinal force developed in the superstructure from the west side of Tower E9 to Pier E11, a distance of 632'. The two deep trusses on the San Francisco side are pin-connected to two roller shoes resting on support-plates rigidly attached to the upper ends of the west-side columns of the tower. Each roller shoe is rigidly connected into a diagonal strut attached at the center of the corresponding top chord member of the tower as shown in Figure 9-14. The longitudinal component of the resultant force in the two diagonal struts is approximately equal to the resultant longitudinal force developed in the superstructure from Pier E4 to the west side of Pier E9, a distance of 2,544'. Because the tributary distance to the west (2,544') is much greater than the tributary distance to the east (632'), the resultant longitudinal force carried into the west side of the tower will normally be much greater than the resultant longitudinal force carried into the east side. Because of this much larger force, the west-side force is carried into the tower through the two strut members, while the east-side force is carried directly into the tower through the two fixed shoes on the east-side columns.

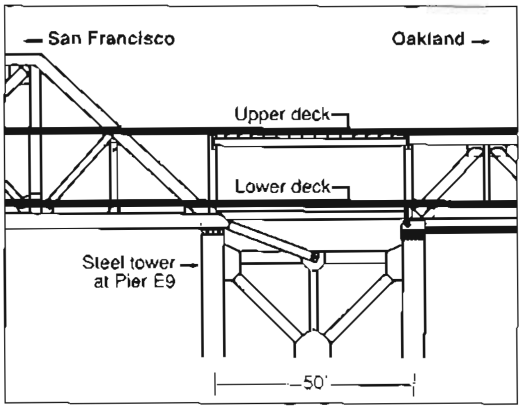


Figure 9-14. *As-built positions of upper and lower decks*

Figure 9-16. *Vertical and transverse supports at west end locations of the four upper deck stringers*

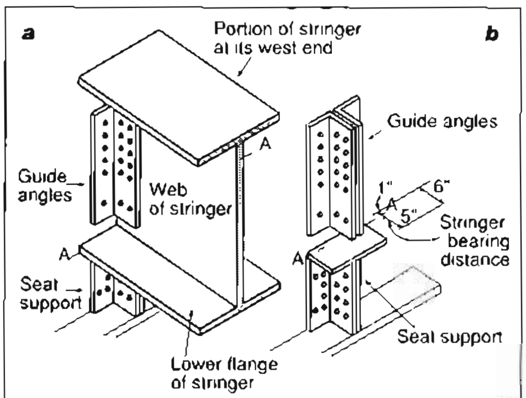


Figure 9-15. *Support conditions at west end of deck stringer: view A shows stringer in position, view B shows stringer removed.*

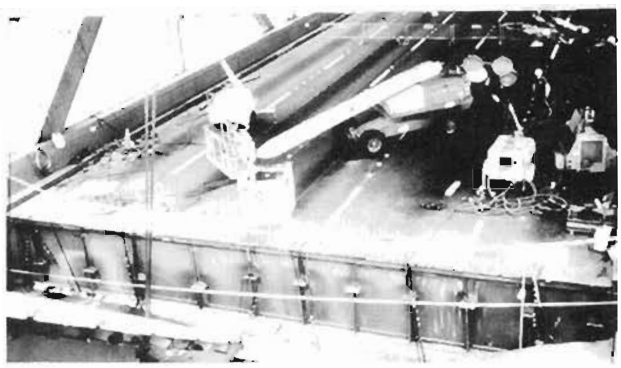


Figure 9-17. *Vertical and transverse supports at west end stringer locations of both upper and lower decks.*

The upper deck floor system, which collapsed during the earthquake, consisted of four longitudinal stringers supporting transverse floor joists which, in turn, supported the concrete deck. The lower deck floor system, which also collapsed during the earthquake, consisted of eleven longitudinal stringers directly supporting the concrete deck—no transverse floor joists were used. Figure 9-15a and 9-15b show the support conditions at the west end of each stringer. The lower flange of each stringer simply rested on top of a stiffened seat support having the longitudinal dimension of 6 inches, as shown in Figure 9-15b. Since a clearance of approximately 1" was provided at the west end of each stringer, its actual lower-flange bearing distance on the seat support in the longitudinal direction was about 5". The web of each stringer was constrained in the transverse direction by the two guide angles shown in Figure 9-15b. Neither the seat support nor the transverse web support provided mechanical constraint in the longitudinal direction. At the east end of each stringer, in both the upper and lower deck systems, the lower flange was bolted to a fixed seat, thus providing essentially a pinned-connection anchorage.

During the earthquake, the seismically induced, resultant, longitudinal, inertia force in the 632 feet of tributary superstructure from the west side of Tower E9 to Pier E11 was transmitted into the two fixed shoes on the east side of the tower. This force was sufficiently large to shear off the 40 1-inch diameter bolts used to anchor both fixed shoes to the tower (Figure 9-12). This shear failure allowed the shoes to move eastward relative to their tower supports by an amount at least equal to the 5" stringer bearing distance shown in Figure 9-15b, thus permitting both upper and lower deck systems to fall off their west-end supports into the positions shown in Figures 9-12 and 9-13. Figure 9-15a shows the seat and transverse web supports at the west-end locations of the four upper-deck stringers immediately following the earthquake. These supports experienced various amounts of damage, as indicated by the close-up photograph in Figure 9-16b. The seat and transverse web supports at the west-end stringer locations of both the upper and lower deck systems, all of which were damaged, are shown in Figure 9-17. While the east-end

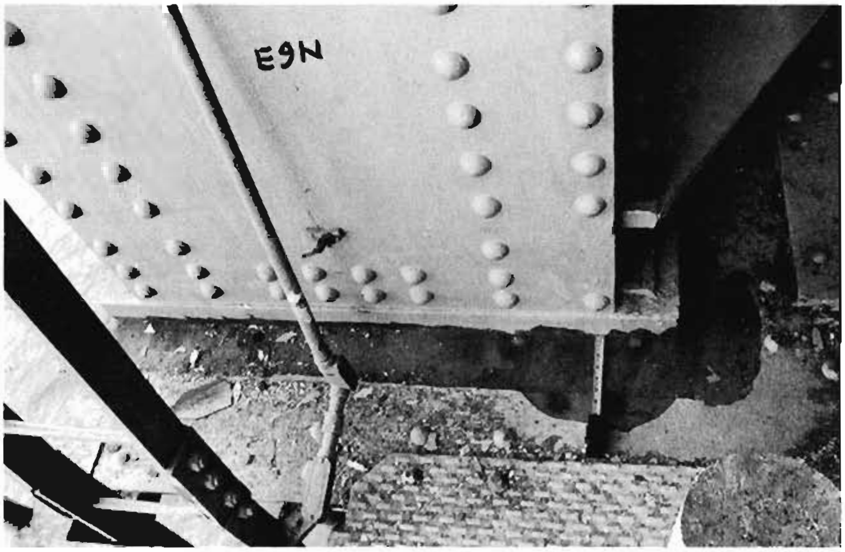


Figure 9-18. *Anchor shoe on top of the northeast column of Tower E9 after the earthquake.*

lower-flange bolted connections of all upper and lower deck stringers did not fail completely, they were severely damaged due to the large deck rotations that occurred during collapse.

Figure 9-18 shows the fixed shoe on top of the northeast column of Tower E9 following the earthquake. The amount of permanent slippage of the shoe relative to its tower support was evident by the orange-colored undercoating paint exposed by the permanent displacement of the shoes. The permanent slippage was measured to be approximately 5 1/2" in the easterly direction and 1" in the northerly direction. The maximum slippage in the easterly direction was at least 7", since the surface coating of paint had been rubbed off over the entire 7" distance from the original shoe position to the easterly edge of the supporting base plate. The permanent slippages of the fixed shoe on the top of the southeast column of Tower E9 differed only slightly from those described above for the fixed shoe on top of the northeast E9 column. The maximum slippage of this fixed shoe in the easterly direction was also at least 7".

Fixed-Shoe Anchor Bolt Failures, Piers E18-E22

During the earthquake, all 24 1-inch diameter bolts attaching the north and south fixed shoes to their pier supports sheared off at each of the locations E18 through E22. These shear failures allowed the shoes to slip back and forth in the east-west direction; keeper plates mounted on top of the base plates prevented slippages in

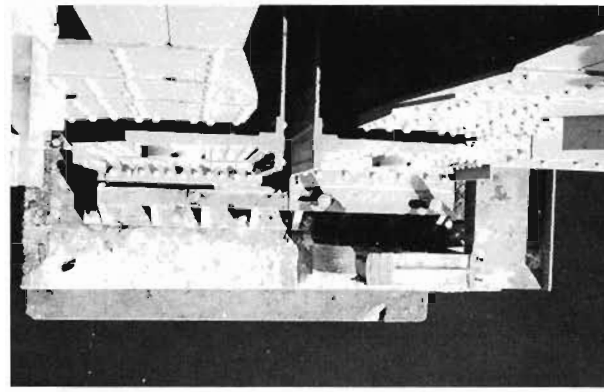


Figure 9-19. *Damaged restrainer device on the south side of Pier E21.*

Table 9-1. *Permanent slippages of fixed shoes at E18 through E22.*

Pier E18	North shoe 3/4"	South shoe 1/8"
Pier E19	North shoe 1"	South shoe 1 1/2"
Pier E20	North shoe 2"	South shoe 2"
Pier E21	North shoe 2 1/4"	South shoe 2 5/8"
Pier E22	North shoe 5"	South shoe 2 1/2"

the north-south direction. Since the expansion shoes at each of the locations E17 through E21 had been retrofitted with restrainer devices of the type shown in Figure 9-11, the easterly displacements of these shoes relative to their respective pier supports were partially controlled, even though the restrainer devices were considerably damaged. Figure 9-19 illustrates such damage to a restrainer device located on the south side of Pier E21; an expansion shoe and a fixed shoe show on the right and left sides, respectively, in this same figure. This partial control of the relative displacements at the expansion shoes directly affected the amounts of slippage that occurred at the fixed shoes. The permanent slippages of the fixed shoes on their supports at all locations E18 through E22 were in the westerly direction. The amounts of these permanent slippages, as reported by Caltrans, are shown in Table 9-1.

The maximum peak-to-peak slippage of each fixed shoe on Piers E18 through E22, which includes motion to the east and to the west of the original position, was about 10" as evidenced by sliding scratch marks in the paint. The maximum slippage to the east was limited to 4 1/2" because of the presence of a keeper plate. Consequently, the average of the maximum slippage values to the east was a little less than this limiting value (4 1/2"). The average of the maximum slippage values to the west was about 5 1/2".

Figure 9-20. *View of Pier E17 from the water.*

Rivet Failures

The rivets anchoring the upper-deck span E23-E24 to the east side of Pier E23 sheared off, allowing the span to move relative to its pier supports. The span's transverse floor beam at the west end, which was connected by rivets to the pier through its lower flange at the north and south pier bearing plates, came to rest about 2" east and 1 1/2" north of its original position. Since the longitudinal stringers of the upper deck closure span at pier E23 were attached to the floor beam, they moved outward along their west-end support seats (detail similar to that shown in Figure 9-15) as a result of the slippage. This movement was large enough that the stringer webs had moved outside of the guide angles, and only 1/4" of longitudinal bearing distance remained on the seat support following the earthquake. Thus, a deck collapse, similar to the one that occurred at Pier E9, nearly took place at Pier E23.

Concrete Fractures and Yielding of Reinforcing Bars at Pier E17

Pier E17, which serves as an anchor pier for the bridge superstructure from Pier E11 to Pier E17, a distance of 1740', experienced damage at the bases of its two concrete pedestals during the earthquake. The photograph in Figure 9-20 shows these two tapered pedestals in their positions immediately below the superstructure and resting on a common concrete base. Figure 9-21 shows elevation and plan views of the pier with sectional views (A-A and B-B) providing structural details. The pedestals are 53'3" high, have outside base dimensions of 17'10" by 20'10", are hollow with inside base dimensions of 11'10" by 14'10", have a uniform wall thickness of 36", and are provided with reinforcing steel both vertically and horizontally.

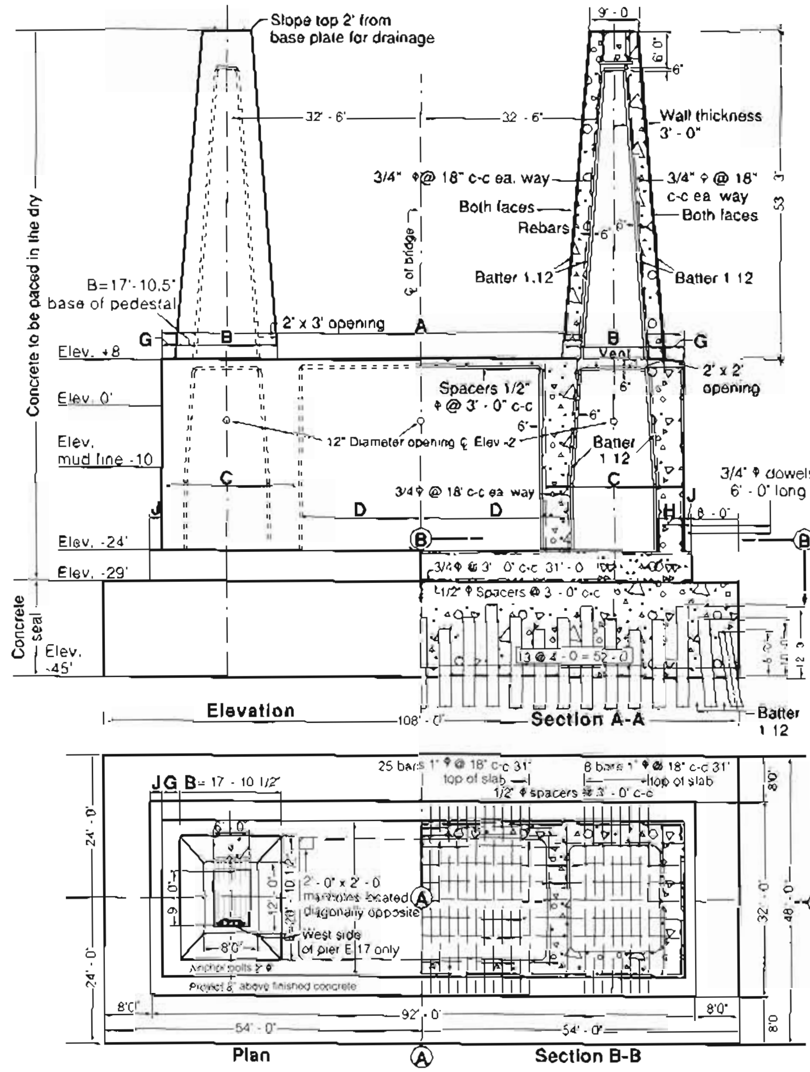


Figure 9-21. *Structural details of Pier E17*

During the earthquake, the tops of the two pedestals moved sufficiently in horizontal directions relative to their bases to cause serious fracturing of the concrete and yielding of the vertical reinforcing bars immediately above the bases at Elev. +8'. The fracture patterns of the concrete (Figure 9-22) were confined to the base locations indicated. Only two of the eight corner locations suffered no apparent fracturing of the concrete; while, as seen from the fracture patterns, relatively large chunks of concrete broke from the pedestals at the other six corner locations. At the southwest corner of the south pedestal, the concrete broke away sufficiently to expose two vertical reinforcing

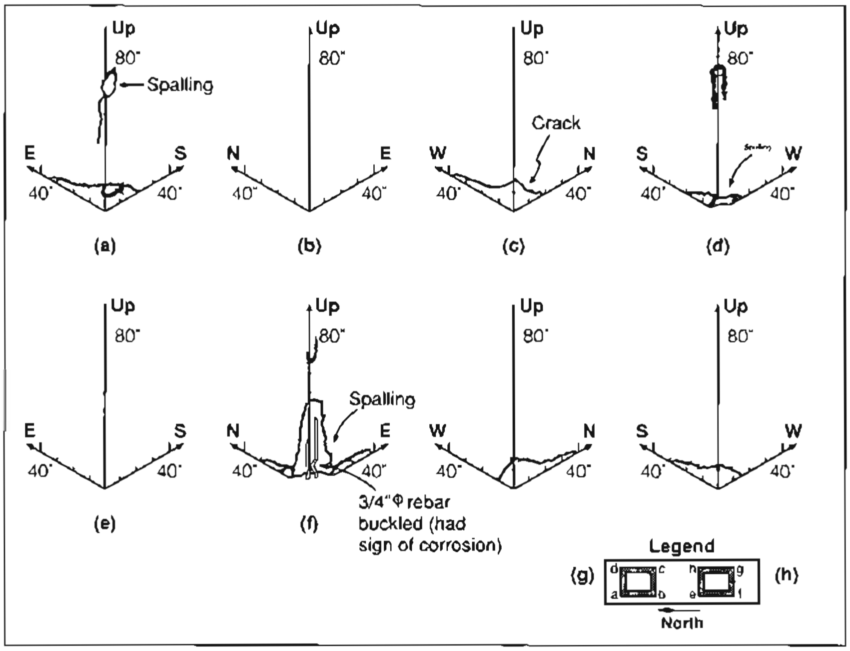


Figure 9-22. *Cracking patterns at the corners of the bases of pedestals at Pier E17.*

bars at a lap splice, as seen in the photograph of Figure 9-23. The spalling of the concrete at this location was caused by the outward buckling of one of these bars, which was straight before the earthquake as evidenced by the bar's corrosion-marked initial position in the concrete. In order for this bar to buckle under compression, it first had to undergo inelastic yielding in tension. The photograph in Figure 9-24 shows the buckled shape in a close-up view. The bar buckled outward about 1 1/2" over a length of about 12". To buckle in this manner it is estimated that the inelastic elongation of the bar was about 1/2". Since the concrete fractures in the pedestals were limited to the base locations, the pedestals appear to have rocked, essentially as rigid bodies, about their bases, causing bar elongations of the type described above. These elongations allowed uplift separations to occur between the bases of the pedestals and their common concrete pier support at Elev. +8'. Following the earthquake, separation cracks were clearly visible at this elevation around the entire perimeter of the south pedestal and around most of the perimeter of the north pedestal. In view of the estimated 1/2-in. inelastic bar elongation, it is reasonable to believe the uplifts were of a similar magnitude. Clearly, the pedestal base overturning moments reached yield levels during the earthquake.

Close observation of the exposed reinforcing bars reveals serious corrosion. All vertical reinforcing in the pedestals needs further investigation.

Figure 9-23. *Spalled concrete and exposed bars at SE corner of south pedestal at Pier E17. Two bars are shown. the bar to the left terminated before entering the base; the other one, which buckled, passed from the base and lapped the other bar.*

Figure 9-24. *Close-up view of the buckled bar shown in Figure 9-23.*

Repair of Damage

The collapsed upper and lower deck systems above Pier E9 (Figure 9-12) were removed in sections and new deck systems were constructed.

The replacement lower deck system was constructed using 11 longitudinal steel stringers having an arrangement similar to the original deck. Five 33" WF 141 lbs/ft stringers were used on the south side, and six 33" WF 130 lbs/ft stringers were used on the north side; the spacing of the former was greater than that of the latter. These 11 stringers now support a newly-constructed precast lightweight concrete deck.

The replacement upper deck system was constructed similar to the lower deck system, except that 12 equally-spaced 33" WF 130 lbs/ft stringers were used. A newly-constructed precast lightweight concrete deck has been placed on top of the twelve stringers. This deck replacement system differs from the original, which had only four longitudinal stringers supporting transverse floor joists on which a concrete deck had been placed.

Both ends of all stringers in the newly-constructed deck systems are mounted on elastomeric bearing pads, which are bonded to both the stringer lower flanges and their bearing seats. All stringers are bolted to their seats at both ends using two 1 1/4" diameter bolts per connection, thus providing positive attachments at both east and west ends. Slotted bolt holes, however, are provided at the west end of each stringer to accommodate movements caused by temperature changes. The newly-constructed bearing seats at the west ends of all stringers have an 8" longitudinal seat width rather than the 6" width used originally as shown in Figure 9-15b. Should the fixed-shoe bolts on the east-side columns fail again during a future earthquake, the west end positive stringer attachments would most likely fail, allowing the stringers to move off their supports. Recognizing this possibility, a secondary support system is planned for installation by Caltrans that will prevent the deck systems from dropping should the stringers move eastward more than 8" relative to their seat supports.

Prior to the above described deck repairs, the bridge superstructure from the east side of Tower E9 to Pier E11 was jacked back into its original position at the top of the two east columns of Tower E9, and the fixed shoes on these columns were reconnected using 1-inch diameter A325 high-strength bolts. The final repair operation at Pier E9 was the installation of new curb and railing sections.

Repair of the earthquake damages at Pier E18 through E22 consisted of jacking the displaced deck spans back into position, installing new 1-inch diameter A325 high-strength bolts in the fixed shoes, and replacing the damaged restrainer devices. The upper deck stringers in the closure span at Pier E23, which had nearly moved off their seat supports, were repositioned and provided with longer bearing seats.

Repair of damage to the pedestals of Pier E17 has not been done as of this writing. However, Caltrans has plans for retrofitting this pier through jacketing of the pedestals.

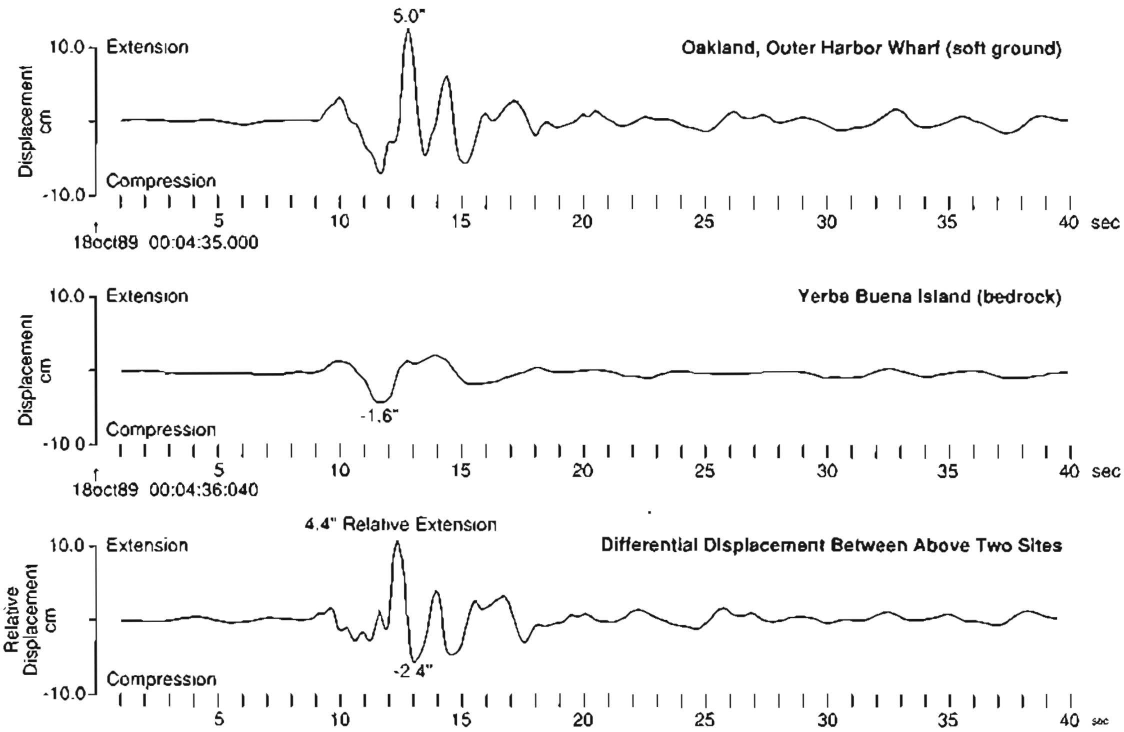


Figure 9-25. *Time-histories of displacement in the direction 79° east of north.*

Assessment of Seismic Loads Experienced During the Earthquake

A complete seismic safety assessment of the San Francisco-Oakland Bay Bridge is beyond the scope of this Report, since it would require in-depth dynamic analyses of both the East Bay Crossing and the West Bay Crossing, including their foundation conditions. However, a limited assessment of the seismic loads has been made that focuses on the longitudinal anchorage forces in the fixed shoes at the piers where damage occurred during the Loma Prieta earthquake. In conducting this assessment, recognition is made of the fact that the East Bay Crossing is almost completely uncoupled longitudinally at Piers YB3, E4, and E11 through the use of double transverse steel bents and at Piers E17-E22 through the use of expansion shoes. This form of uncoupling was provided to greatly limit thermal stresses from temperature variations of ±30°F from the assumed normal temperature of 62°F. Further, recognition is made of the fact that temperature variations of ±30°F will produce unconstrained length changes of ±27" in a 2.14-mile length of steel structure, which corresponds to the distance from Pier YB1 to E39.

Considering that these length changes are very large compared with the Loma Prieta earthquake-induced maximum relative displacements of +4.4" and -2.4" between a strong-motion recording station on Yerba Buena Island and a similar station on the outer harbor wharf in Oakland (Figure 9-25), it is concluded that the longitudinal anchorage forces were produced primarily by seismically-induced inertia forces in the bridge superstructure, rather than by earthquake-induced pier differential displacements at the piers. The role of inertial forces is further supported by the evidence that there was no observable permanent pier separations when the bridge spans were jacked back into their original positions on their respective support piers.

Equivalent Static Seismic Coefficients

To assess the adequacy of the fixed-shoe connections at Piers E9, E17, E18, E19, E20, E21, E22, and E23 as originally constructed and as rebuilt following the earthquake, equivalent static seismic coefficients have been calculated in accordance with the definitions

$$C_u \equiv \frac{nV_u + \mu w L_f}{wL_I} \qquad (1)$$

and

$$C_d \equiv \frac{nV_d}{wL_I} \qquad (2)$$

in which n is the total number of bolts anchoring both the north and south fixed

Table 9-2. *Evaluation of equivalent static seismic coefficients C_u and C_d.*

Pier No.	n	d in.	L_f ft.	L_l ft.	Original Bolts V_u-k	V_d-k	C_u	C_d	Comments	Replacement Bolts V_u-k	V_d-k	C_u	C_d
E9	40	1	170	632	38	19	0.22	0.07	F	65	32.5	0.32	0.12
E17	48	2	290	1740	115	57	0.23	0.12	DNF	—	—	—	—
E18	24	1	145	290	38	19	0.33	0.09	F	65	32.5	0.46	0.16
E19	24	1	145	290	38	19	0.33	0.09	F	65	32.5	0.46	0.16
E20	24	1	145	290	38	19	0.33	0.09	F	65	32.5	0.46	0.16
E21	24	1	145	290	38	19	0.33	0.09	F	65	32.5	0.46	0.16
E22	24	1	145	290	38	19	0.33	0.09	F	65	32.5	0.46	0.16
E23	8	2	145	290	115	57	0.33	0.12	DNF	—	—	—	—

F=failed; DNF=did not fail; d=bolt diameter; n=number of bolts

shoes to a specified pier; V_u is the ultimate shearing strength of one bolt under the single-shear condition existing at each fixed shoe; V_d is an allowable design single-shear force per bolt taken herein as $V_u/2$, which is reasonable in engineering practice; w is the gravity load from the combined dead-weight of the bridge superstructure plus a small percentage of the live-load as measured per unit of longitudinal length, i.e., $w = 16.1 + 1.2 = 17.3$ k/ft; L_l is the tributary longitudinal length of superstructure contributing to the longitudinal anchor force being carried by the two fixed shoes; μ is the sliding coefficient of friction between each fixed shoe and its pier support taken herein equal to 0.3; and L_f is the tributary longitudinal length of superstructure contributing to the vertical reaction force from the gravity load transmitted to the pier by both fixed shoes.

In accordance with the definitions of Equations 1 and 2, C_u and C_d are equivalent static seismic coefficients corresponding to the shear-failure loading condition and a specified design loading condition, respectively. Note that under the specified design loading condition, friction is not taken into consideration and the design single-shear load per bolt V_d is taken as $V_u/2$. Coefficients C_u and C_d as defined above have been evaluated for the fixed shoes, using bolt strengths for both the original bolts and the replacement bolts, at Pier E9 and at Piers E17 through E23 as given in Table 9-2. The values of V_u used in this table are those values obtained by tests on original bolts removed from the bridge and new bolts (A325) taken from the replacement bolt supply. These tests were conducted at the University of California, Berkeley, under the direct supervision of Abolhassan Astaneh, Associate Professor of Civil Engineering.

Since the original fixed shoe bolts failed at Pier E9 and at each of Piers E18 through E22, the corresponding C_u values in Table 9-2 provide a reasonable estimate of the maximum seismically-induced longitudinal acceleration (an average along the length) that actually developed in each tributary length (L_l) of superstructure. Accordingly, the maximum average longitudinal acceleration that developed in the 632' length of tributary structure to Pier E9 was about 0.22g (g = acceleration of gravity), and the maximum average longitudinal acceleration that developed in each 290' length of tributary superstructure to Piers E18 through E22 was about 0.33g. If the ultimate shearing strengths (V_u) of the original bolts had been higher, the maximum average longitudinal accelerations would have been higher than those values given above—how much higher cannot be determined from the data available. Because the original bolts did not fail at Piers E17 and E23, the corresponding C_u values (0.23 and 0.33) simply indicate that the maximum average longitudinal accelerations were less than 0.23g and 0.33g.

While the 48 2" diameter fixed-shoe bolts at Pier E17 did not fail, indicating an average longitudinal acceleration in the 1740' of tributary superstructure less than 0.23, yield-level overturning moments did occur at the bases of the piers two concrete pedestals limiting the shearing forces transmitted to the fixed-shoe bolts.

Because the ultimate shear strengths (V_u) of the replacement bolts are much higher than those of the original bolts, the corresponding C_u values of the repaired structure are much higher than those of the original structure. The replacement bolt C_u values equal to 0.32 and 0.46 in Table 9-2 indicate that the maximum average longitudinal accelerations in the corresponding tributary lengths of the superstructure would have to reach 0.32g and 0.46g, respectively, to produce shear failures in the new bolts. Had these replacement bolts been installed prior to the Loma Prieta earthquake and had the higher g-levels not been reached during the earthquake, the bolts would not have failed. However, considering that the Loma Prieta earthquake produced only moderate levels of ground shaking at the San Francisco-Oakland Bay Bridge site, much higher g-levels could occur under maximum credible earthquake conditions, provided the corresponding fixed shoes and pier support systems (steel tower and concrete piers) can carry the larger loads. If they cannot, more serious failures can be expected to occur.

The design equivalent static seismic coefficient C_d as defined by Equation 2 simply indicates what fraction of the tributary weight (wL_1) can be carried longitudinally through the fixed shoes under the specified design condition that the combined longitudinal anchor force carried through both shoes not exceed the value $F_d = nV_d = nV_u/2$. As shown in Table 9-2, the original-bolt C_d values at Pier E9 and at Piers E18 through E22 are 0.07 and 0.09, respectively; while at Piers E17 and E23, they equal 0.12. These coefficient values agree reasonably well with the original seismic design requirement of the San Francisco-Oakland Bay Bridge [Purcell et al., 1936]. As stated by the design engineers C. H. Purcell, C. E. Andrew, and G. B. Woodruff: "The stresses in the superstructure were calculated for seismic forces equal to 10 percent of gravity acting in any direction." For unknown reasons, this quoted design requirement differs from that stated in a booklet issued by the State of California, Department of Public Works entitled *San Francisco-Oakland Bay Bridge, Design Specifications, Superstructure*, 1933, which states: "The forces due to earthquake shall be assumed as a horizontal force equal to 7 1/2 percent of gravity."

While the original fixed-shoe anchorages examined above essentially satisfy the original design requirement, they were clearly inadequate in resisting the forces developed during the Loma Prieta earthquake. As shown in Table 9-2, the replacement bolt C_d values at Pier E9 and at Piers E18 through E22 are 0.12 and 0.16, respectively.

Dynamic Seismic Coefficients

The pseudo-acceleration response spectrum for a given ground acceleration time-history and for a specified value of damping can be used effectively to estimate the maximum acceleration induced in a structure when it responds essentially as a single degree of freedom (SDOF) system. While the East Bay Crossing cannot in general be modelled in this simplified way, some segments can be so modelled under certain conditions. For example, the longitudinal response of each simple-span structure between Piers E17 and E22 could be modelled in this approximate manner, since the roller expansion shoes at the west end of each span uncouple it horizontally from the continuing structure to the west. Thus, the superstructure mass of one span can be lumped at the top of its anchor pier, giving essentially a SDOF system when the response levels are sufficiently low so that the expansion shoe restrainer devices do not interact. The single concrete-pier/timber-pile supporting system would provide longitudinal restraint to this mass. After estimating the spring constant of this restraint, including soil-structure interaction, and after selecting an appropriate damping value, the SDOF system would be defined. Then, using a realistic pseudo-acceleration response spectrum, one can estimate the maximum horizontal anchorage force carried through the fixed shoes. Because the supporting system is quite stiff, anchor forces much larger than those represented by the Cu values in Table 9-2 can be expected under maximum credible earthquake conditions. This statement is supported by the fact that:

- Peak free-field accelerations produced at the surface of the ground near the eastern end of the East Bay Crossing were about 0.25g.
- These peak accelerations could be expected to be much higher, by a factor of possibly 2, under maximum credible seismic conditions.
- Dynamic structural response could, for the stiffer systems, produce even higher accelerations in the bridge superstructure system.

Under maximum credible seismic conditions, large nonlinear effects would be experienced in the foundations, thus reducing their stiffnesses and increasing their damping values. Clearly, rigorous dynamic analyses, including modelling of nonlinear effects and radiation damping are required to properly assess the maximum seismic forces produced in the bridge elements and to assess their performances under such conditions.

Conclusions

A comprehensive seismic safety assessment of each major bridge crossing San Francisco Bay should be implemented under a high-priority program sponsored by Caltrans, and retrofit measures should be taken immediately thereafter, as needed, to ensure satisfactory performance of each bridge under both moderate and maximum credible earthquake conditions. Each seismic safety assessment should include the following components:

1. A seismic hazard analysis to establish the annual probability of exceedence relation for peak free-field ground acceleration on firm soil and/or bedrock at the bridge site.
2. Analyses of the vertical and horizontal free-field ground motions, as a function of soil depth, at each bridge-support location under both moderate and

maximum credible earthquake conditions. New borings, using currently acceptable standards, should be drilled to bedrock at selected pier locations.

3. Three-dimensional dynamic analyses of the complete bridge system (superstructure and substructure), including soil-structure interaction, when subjected to the free-field ground motions at all support locations under both moderate and maximum credible earthquake conditions.
4. Assessment of the performance (linear and nonlinear) and safety of each structural element in both the superstructure and substructure systems under both moderate and maximum credible earthquake conditions.
5. Identify the deficiencies on all bridge systems that have the potential to cause unsatisfactory performance under moderate and/or maximum credible earthquake conditions.
6. Develop retrofit measures that would remove the deficiencies and give reasonable assurance of satisfactory performance.

All components of the seismic safety assessment should be carried out with rigor and completeness using modern state-of-the-art methodologies. Strong-motion instrumentation should be installed on all major San Francisco Bay bridges to allow monitoring of their dynamic responses during future earthquakes.

Chapter 10

The Cypress Viaduct Collapse

History of Cypress Viaduct

The Cypress Viaduct was California's first continuous double-deck freeway structure. Each deck of the Viaduct carried 4 lanes of traffic. Before construction of the Cypress Viaduct, the Nimitz Freeway terminated at street level. Traffic between the Bay Bridge and the Nimitz had to traverse city streets and railroad tracks. The Cypress Viaduct, from 9th Street in the south to 34th Street in the north, connected the two transportation links (Figure 10-1).

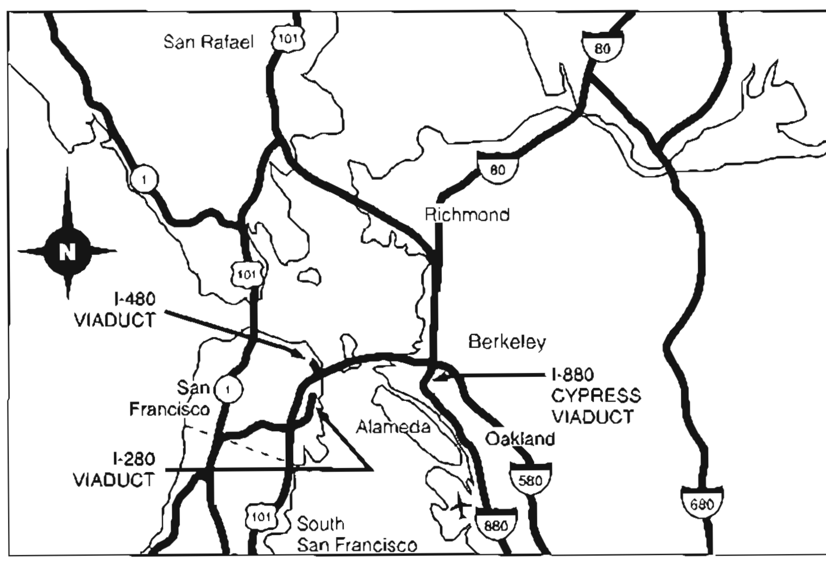


Figure 10-1. *Location of the Cypress Viaduct portion of Interstate 880.*

Construction of the Cypress Viaduct involved three separate contracts on which work began in August, 1954 and was completed in August, 1957 (Figure 10-2). Total construction cost was $10,200,000—the structure cost $8,700,000 and the roadwork cost $1,500,000. Upon completion, the

Figure 10-2. *Cypress Viaduct before collapse*

Figure 10-3. *Cypress Viaduct after collapse.*

Cypress Viaduct was heralded as "a solution required to provide for the passage of 50,000 vehicles per day through a congested area without undue delay, and yet make it possible to efficiently construct the relieving facility within the confined space immediately adjacent to traffic." [*Highway Magazine*, 1958]. During the Magnitude 7.1 Loma Prieta earthquake, a large portion of the structure collapsed (Figure 10-3). The collapse extended from Bent 63 in the south through Bent 112 in the north. Only Bents 96 and 97 remained standing.

Shortly after the earthquake a series of six contracts was awarded by Caltrans for demolition and removal of the Cypress Viaduct structure and resurfacing of frontage roads (Figure 10-4). The first contract was awarded on October 31, 1989 and the final contract on January 10, 1990. The total cost of the demolition was just over $3.5 million. With the completion of the last contract in January, the Cypress Viaduct was removed completely.

Figure 10-4. *Cypress Viaduct during demolition.*

Design of the Cypress Viaduct

Caltrans began preliminary design of the structure in 1949 and construction took place between 1954 and 1957. During the conceptual design phase a double-deck structure was selected over two single-level structures because of right-of-way costs [*Design News*, 1956].

The Cypress Viaduct contained several new and unusual design features, including a large number of different bent types and different hinge arrangements. It is believed that the original structural system was designed with many hinges and joints in order to simplify the analysis and interpretation of behavior, as well as to provide for movements due to creep, shrinkage, temperature, and prestressing, and for future construction additions. The many hinges and joints, coupled with the inadequate seismic design criteria and forces existing in the early 1950s, made the structure highly susceptible to damage or collapse in a strong earthquake.

The Cypress Viaduct was analyzed and designed between 1949 and 1954, during a period when little was known about seismic design of reinforced concrete structures. It should also be noted that the use of prestressed concrete in bridges was new in the United States when the Cypress Viaduct was designed. The first major U.S. bridge

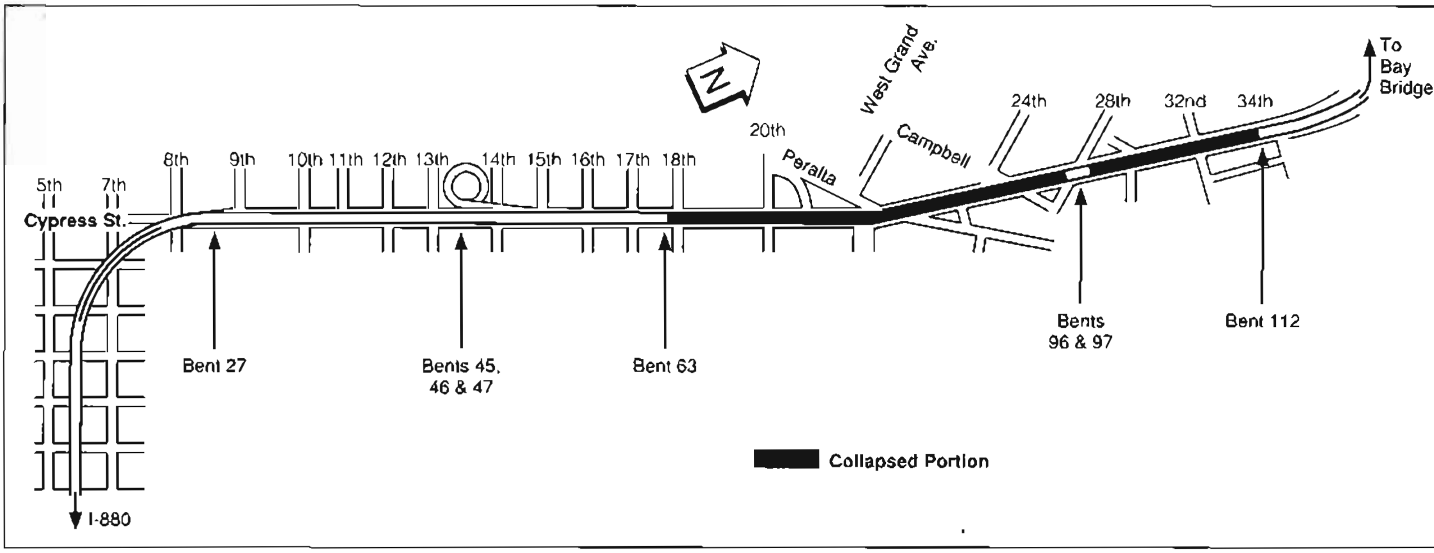


Figure 10-5. *Plan of Cypress Viaduct showing damaged bents and undamaged bents.*

application of prestressed concrete was completed in Philadelphia in 1949. More significantly, the Cypress Viaduct was designed before research had developed procedures for obtaining ductility in over-stressed structural members, and, therefore, the columns and joints failed in a brittle manner when overloaded.

Seismic Design Criteria Used By Caltrans in Design of Cypress Viaduct

The Caltrans seismic design criteria in effect during 1949 to 1954 were introduced in 1943. They stipulated that structures be designed to resist a seismic force in accordance with the formula F = CW, in which F = seismic force applied in any direction at the center of gravity of the structure; W = dead load of the structure; and C = coefficient as here specified: .02 for structures founded on spread footings with soil capacities of 4 tons per square foot or more; .04 for structures founded on spread footings with soil capacities less than 4 tons per square foot; and .06 for structures founded on pile foundations. The above were to be applied as static loads. Since the Cypress Viaduct was founded on pile foundations, it would have been designed for F = .06W. The design did not incorporate ductility, which was not common until the 1970s.

Other Seismic Design Criteria in Effect at Time of Cypress Viaduct Design

It is interesting to compare the Caltrans seismic requirement with that specified in the 1946 Uniform Building Code (UBC), which specified the following earthquake design lateral forces:

	Firm Soil	Soft Soil
Oakland-San Francisco	(.02x4)W	(.04x4)W
Sacramento	(.02x2)W	(.04x2)W

The UBC used a 4 (Zone 3 of the UBC map) or 2 (Zone 2) multiplier to account for earthquake zone location. Thus, UBC would have required 0.16W for the Cypress Viaduct design, whereas Caltrans used 0.06W. If Caltrans had followed the UBC by multiplying by 4 for zone location, they would have used (.06x4)W = .24W in the design of the Cypress Viaduct.

A study of the AASHO/AASHTO seismic design codes indicates that up until 1961, AASHO/AASHTO had no specific requirements for seismic design, except that earthquake forces should be considered when they exist, and an allowable overstress of 33.3% was permitted when earthquake stresses were included in a load combination. A history of seismic requirements in the AASHO/AASHTO national code for bridges and in the Caltrans bridge design criteria may be found in Chapter 7 of this Report.

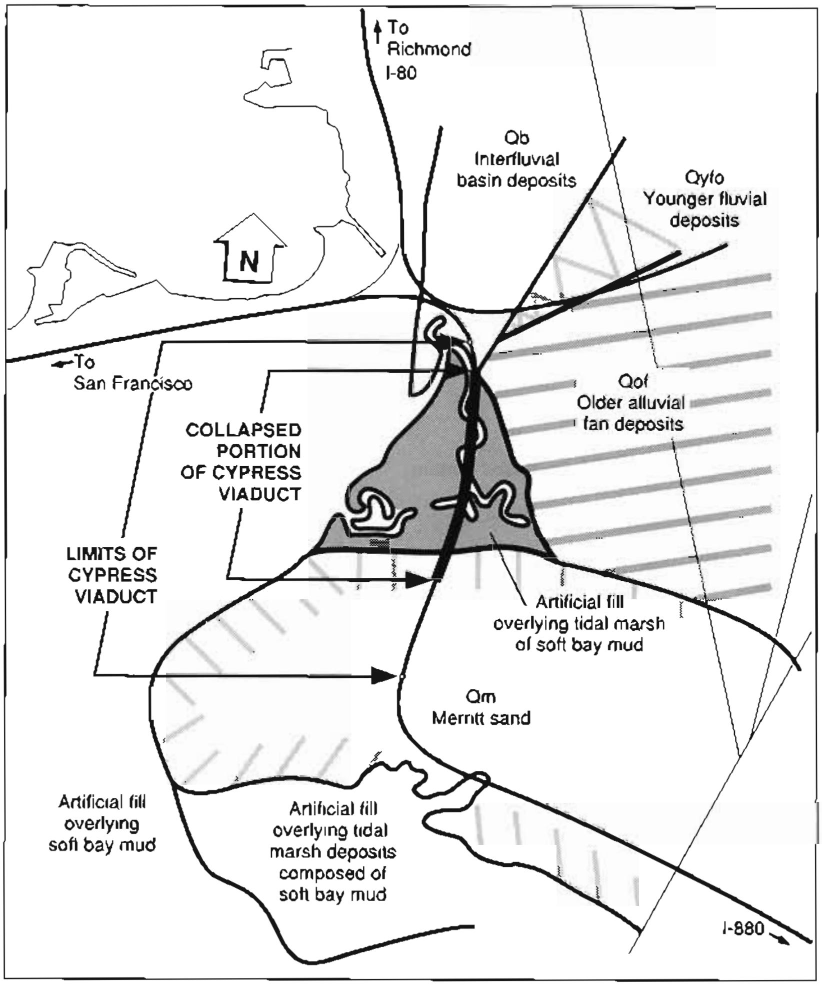


Figure 10-6. *Geologic map of Cypress Viaduct site.*

Description of Cypress Viaduct Structure

The Cypress Viaduct was a reinforced concrete structure with some prestressing and two levels of elevated roadway. The box girder roadway was supported by a series of 83 two-story bents, extending from Bent 29 through Bent 111. Bents 63 through 112 collapsed in the October 17th earthquake; only Bents 96 and 97 remained standing (Figure 10-5). The majority of the bents were constructed of reinforced concrete. However, a number of the bents had post-tensioned concrete transverse girders at the top level. In some cases these were used to provide for possible future widening of the roadways, necessitating an increase in top girder transverse spans from about 65' to as long as 100', and the removal of some of the top story columns. This could be done with relative ease using prestressed girders, but not with reinforced concrete top girders. The following description of the Cypress Viaduct structure draws heavily, and in some cases directly, on the report prepared by researchers from the University of California at Berkeley: Nims, Miranda, Aiken, Whittaker, and Bertero [Nims et al., 1989].

Box girders of multi-cell reinforced concrete had spans varying from 68 to 90 feet between supporting bents. At both deck levels, the box girders were monolithically connected to the bents. A variety of bent types were used with several hinge combinations in the upper story columns. Hinges also existed at the connection of the lower columns to the footings. In addition to hinges in the bents, transverse joints were provided across the entire width and depth of the multicell box girder bridge system in every third span to allow for longitudinal expansion and contraction.

Site Conditions

Geologic maps of the area indicate that about 6,000' of the Cypress Viaduct was located on the bay plain at an approximate elevation of 10'. Alignment at the ground surface was underlain by unconsolidated surficial sedimentary deposits [Helley 1972, Radbruch, 1957] (Figure 10-6). The southerly one-third of the Cypress Viaduct was founded on Merritt sand, a loose to relatively dense deposit of beach and wind-blown, fine-grained sands with varying amounts of silt and clay and with lenses of sandy clay and clay. The Merritt sand is about 60 feet thick in the subject area and abruptly thins at its northern margin.

The northerly two-thirds of the Cypress Viaduct, the major portion that collapsed, was founded on about 7' of dense-to-stiff artificial fill overlying a pre-existing triangu-

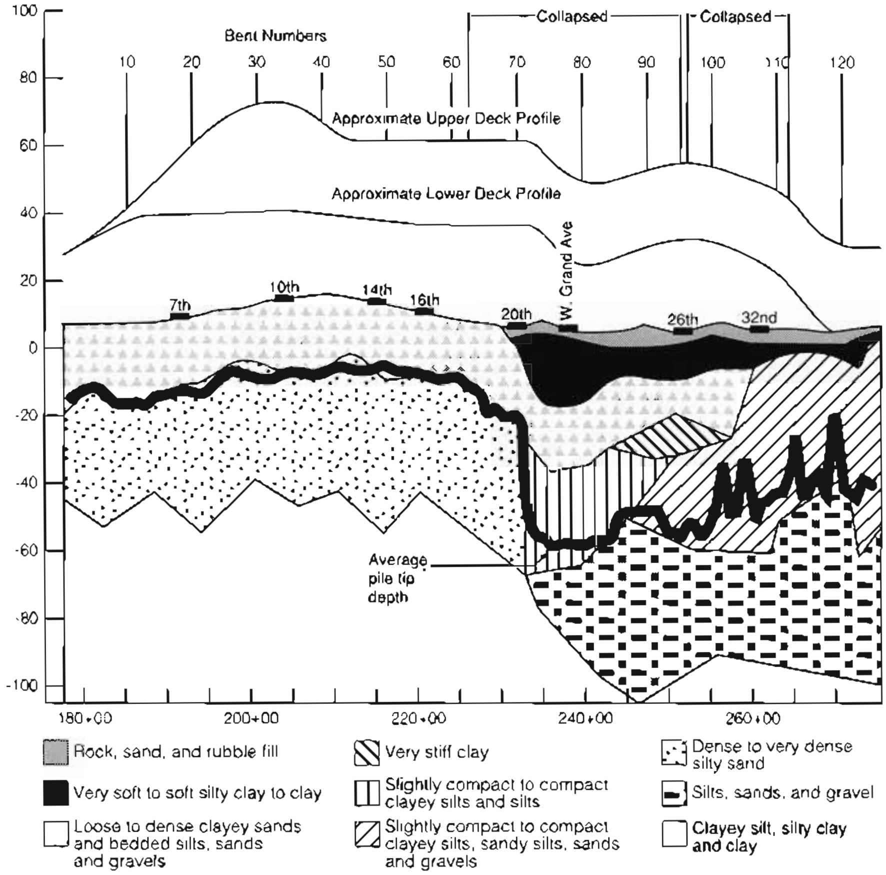


Figure 10-7. *Soil profile of Cypress Viaduct based on Caltrans boring information from 1953-1990.*

Table 10-1. *Types of soil underlying Cypress Viaduct.*

Geologic Unit	Description
Bay mud	Soft mud becoming firmer with depth; thickness range from 0-100 feet.
Merritt sand	Fine-grained silty sand of beach and wind blown origin; maximum thickness 65 feet.
Temescal formation	Clayey gravel; sandy, silty clay and sand-silt-clay mixtures; alluvial fan material brought down from the Berkeley Hills; maximum thickness 60 feet; grades laterally into Merritt sand.
Alameda formation	Clay, silt, sand and gravel; consolidation increases with depth; maximum thickness over 1,000 feet.
Franciscan assemblage	Sandstone, shale and various metamorphic rocks.

lar-shaped tidal marsh composed of soft bay mud and with old slough channels paralleling the west side of the viaduct structure. Boring logs indicate that the thickness of the soft bay mud ranges from 20 to 25 feet. The presence of this marsh is shown on current geologic maps and was shown in an 1878 Atlas Map of the Alameda County area by Thompson and West.

Based on generalized, published geologic data [Radbruch, 1957; Trask, 1951], the typical sequence of subsurface geologic units in the Cypress area, from top to bottom, is as shown in Figure 10-7 and Table 10-1.

Surficial deposits underlying the soft marsh deposits in the vicinity of Cypress Viaduct include [Helley, et al, 1972]:

1. Older alluvial fan deposits, composed of silt, sand and gravel to the east of the marsh deposits and generally extending to depths of several hundred feet or more.
2. Younger fluvial deposits, composed of fine sand silt and clay to the north of the marsh, generally extending to depths of less than 15 feet.
3. Interfluvial basin deposits to the north of the bayward edge of the plain composed of plastic organic-rich clay and silty clay generally extending to depths of less than 10 feet.

These uncemented sediments tend to interfinger with depth down to bedrock level at elevation -510 to -565 feet.

Two borings drilled along the Cypress Viaduct alignment encountered Franciscan bedrock, consisting of graywacke sandstone, shale and siltstone, at about elevation -515' and -555', with the bedrock surface sloping gently to the north. This data and data from other borings in the area suggest that the trough or low point of the bedrock depression between Yerba Buena Island and the East Bay Hills may be somewhere in the vicinity of the Cypress Viaduct.

The southerly boring encountered a 60' interval of Merritt sand; overlying a 140' thickness of fairly competent clayey silt, silt and sandy silt with a few scattered intervals of sand-silt-gravel mixtures; overlying about 320' of clayey silt, silty clay and clay with intervals of silt-sand-gravel mixtures.

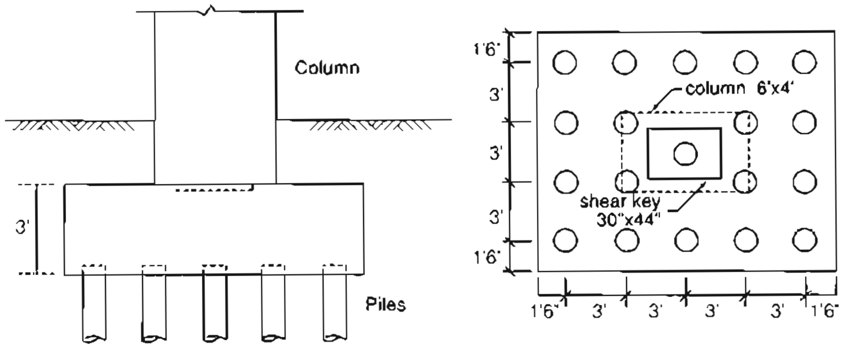


Figure 10-8. *Representative pile cap.*

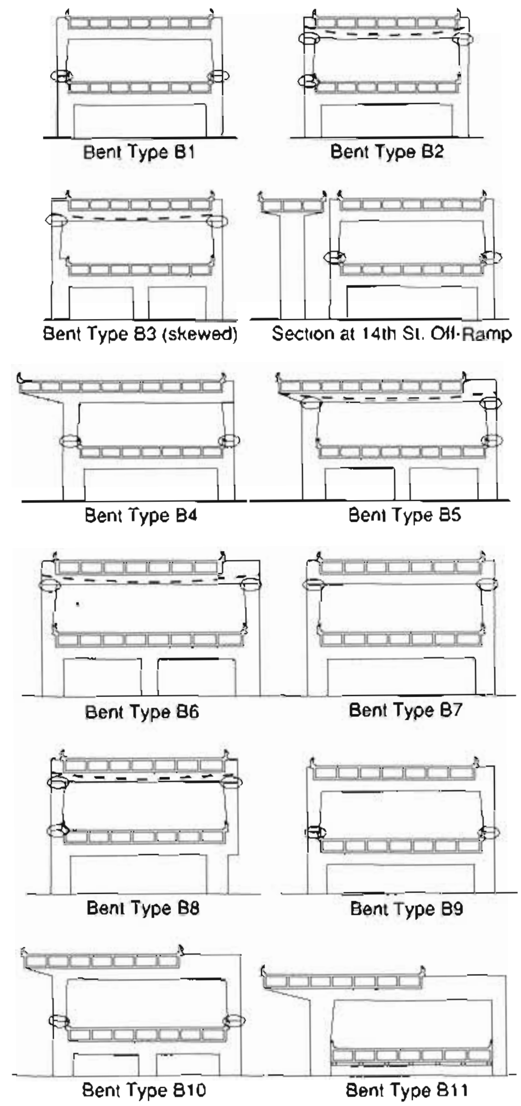


Figure 10-9. *Elevations of the 11 different bent types used in the Cypress Viaduct, circles indicate shear key (hinge) locations, dashed line indicates post-tensioning cables.*

The northerly boring encountered 8' of fill overlying 8' of soft silty clay (bay mud); overlying about 50' of slightly compact to compact clay-silt-sand-gravel mixtures; overlying a sequence of sediments similar to that encountered below the Merritt sand in the southerly boring with the exception that the lowermost 110' of the sequence encountered sand, gravel and gravelly clayey silt.

There was no reported evidence of foundation failure or liquefaction following the earthquake. There are no known faults in the immediate proximity of the Cypress Viaduct site. The extent of collapse is nearly identical to the limits of the buried tidal marsh deposits, with two exceptions: the southerly portion of the collapsed section extended approximately 500' south onto the Merritt sand foundation, and the short-skewed crossing at 26th Street, which is founded over the buried tidal marsh, did not collapse.

Bent Foundations

The bents were supported on pile foundations. The as-built drawings [State of Calif., Dept. of Trans., various dates] provide the following information concerning the foundations:

- Piles were approximately one foot in diameter spiral welded pipe piles. The number of piles used varied from bent to bent.
- Each column base was on a separate foundation.
- Bents with two lower columns had from 18 to 35 piles for each column base.
- Bents with three lower columns had from 9 to 21 piles for each column base.
- The design pile loading was 45 tons.

Pile lengths varied significantly along the Viaduct and can be divided into two distinct groups. From Bent 34 to Bent 71, the average pile length is approximately 15'. From Bent 72 to Bent 111, the average pile length is approximately 50'. The change between the two groups is dramatic: at Bent 71 the average pile length is approximately 21', while at Bent 72 the average pile length is approximately 51'. Pile lengths are charted in Figure 10-7. A representative pile cap is shown in Figure 10-8. There is no evidence of failure of the foundation system or that the foundation contributed to the failure of the bents.

Table 10-2. *Bent Types—Tenth St. to north abutment.*

No.	Bent Type	No.	Bent Type	No.	Bent Type
32	B1	60	B5	88	B1
33	B1	61	B5	89	B1
34	B1	62	B6	90	B1
35	B1	63*	B1	91	B1
36	B1	64	B1	92	B1
37	B1	65	B1	93	B1
38	B1	66	B1	94	B1
39	B1	67	B1	95	B3
40	B1	68	B1	96**	B3
41	B1	69	B1	97**	B3
42	B1	70	B7	98	B3
43	B1	71	B2	99	B1
44	B1	72	B2	100	B1
45	B1	73	B8	101	B1
46	B1	74	B8	102	B1
47	B1	75	B2	103	B1
48	B1	76	B2	104	B1
49	B1	77	B2	105	B1
50	B1	78	B2	106	B1
51	B1	79	B2	107	B9
52	B1	80	B2	108	B9
53	B1	81	B1	109	B10
54	B1	82	B1	110	B10
55	B1	83	B1	111	B10
56	B4	84	B1	112***	B11
57	B5	85	B1	113	B11
58	B5	86	B1	114	B11
59	B5	87	B1		

* Southernmost collapsed bent
** Standing bents
*** Northernmost collapsed bent

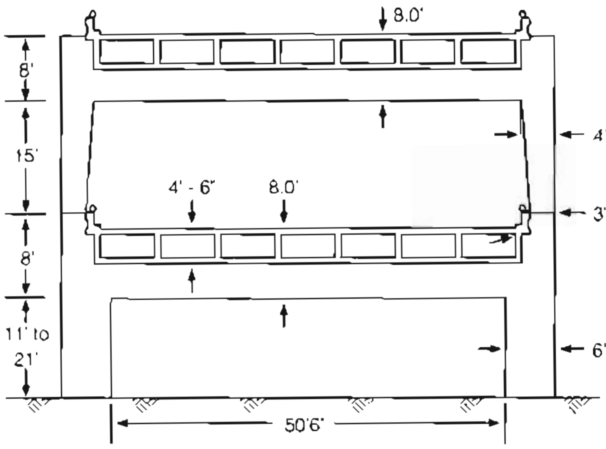


Figure 10-10. *Typical bent dimensions, type B1 bent*

Figure 10-11. *B1 bent with no external damage*

Types of Bents

Eleven different bent types were used in the design of the Cypress Viaduct. The majority of bents, in terms of both the total number surveyed and the number collapsed, consisted of two portal frames (i.e., with columns pinned at their bases), one mounted on top of the other. All of the different bent types identified in the investigation are illustrated in Figure 10-9. Much of the following description of the structure is taken from the U.C. Berkeley report [Nims et al., 1989]. Table 10-2 catalogs all of the bent types observed from Bent 32 to Bent 114. This includes bents on either side of the collapsed region as well as all of the failed bents.

Many of the bents had some superelevation. However, superelevation did not appear to contribute to the failure and is not shown in the representative figures.

B1 Bents. Fifty-three of the total 83 bents were type B1 bents—two portal frames, one mounted on top of the other. Typical bent dimensions are shown in Figure 10-10. An example of a B1 bent with no external damage is shown in Figure 10-11. The upper frame is connected to the lower frame by shear keys ("hinges") at the base of the upper frame columns. The columns of the lower frame are connected to the pile caps by shear keys. Figures 10-12, 10-13, and 10-14 show the B1 bent reinforcement details from the as-built plans. This detailing will be referred to in subsequent sections.

B2 Bents. The second most common bent design, B2—of which there were a total of eight—is shown in Figure 10-15. The B2 bent design was characterized by three shear keys in the upper level and a post-tensioned upper girder. The east column of these bents is continuous from the ground level to the shear key beneath the upper girder. The west column has two shear keys in the upper level: one above the lower girder and one beneath the upper girder. Thus, the upper west side column is a pin-ended column, and therefore resistance to lateral loads in the

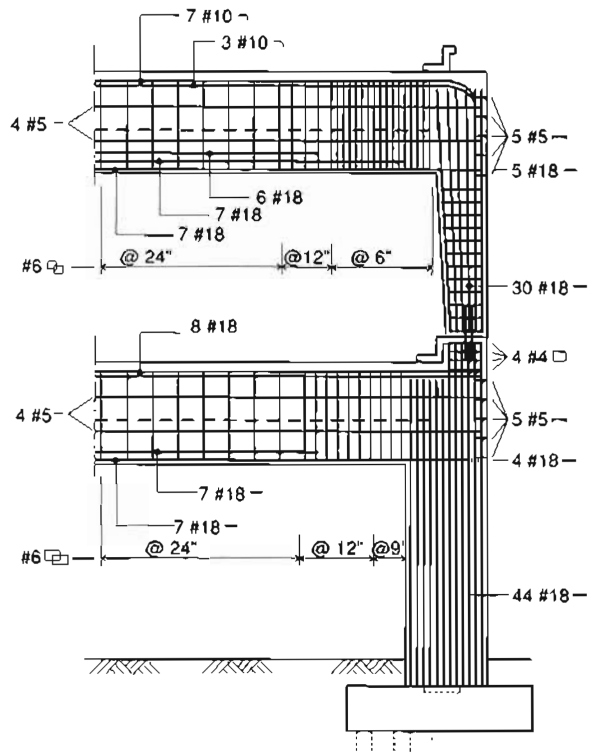


Figure 10-12. *Reinforcement layout of type B1 bent.*

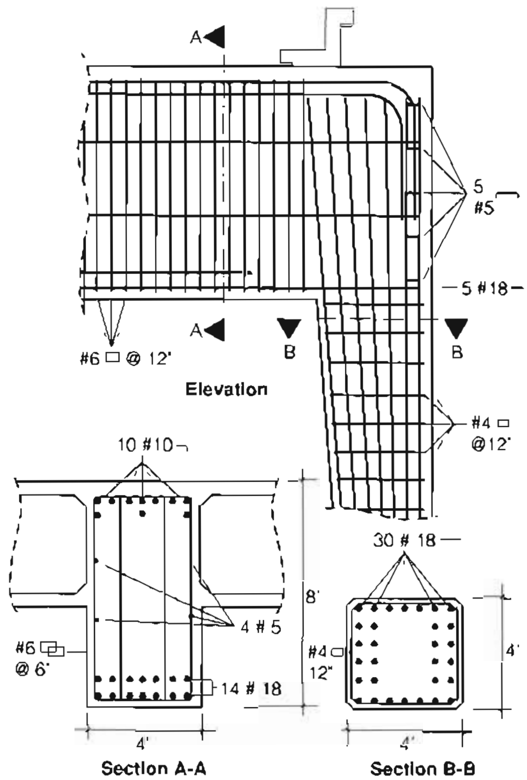


Figure 10-13. *Upper level joint detail of type B1 bent.*

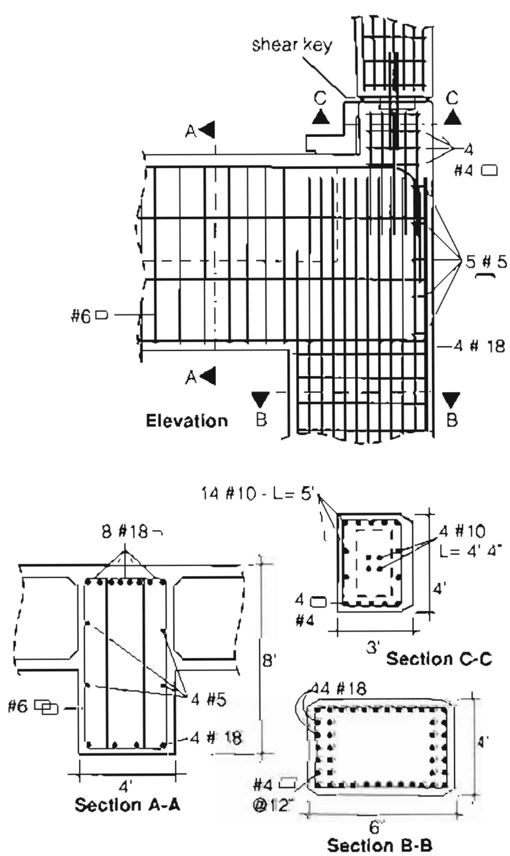


Figure 10-14. *Lower level joint detail of type B1 bent.*

upper level deck is provided by the east column only. The columns of the lower frame are connected to the pile caps by shear keys.

B3 Bents. A third bent design, B3, was used for Bents 95-98. This design is of particular significance because Bents 96 and 97 remained standing after the earthquake. The 96-97 span was the only span along the entire portion of the Viaduct from Bent 63 to Bent 112 in which the upper level roadway did not collapse. This bent design is shown in Figure 10-9 and is characterized by three columns supporting the lower roadway, two continuous external columns supporting the upper roadway and a post-tensioned upper girder. The only shear keys in this design are at the connections of the two upper level columns to the underside of the upper girder and at the connections of the three lower level columns to the pile caps. Bents 95-98 are skewed with respect to the north-south axis of the roadway. This skew accommodates 26th Street, which passes under the Viaduct between Bents 96 and 97.

Other Bents. At the northern end of the Viaduct, the upper and lower roadway alignments diverge, with the lower roadway returning to ground level and the upper

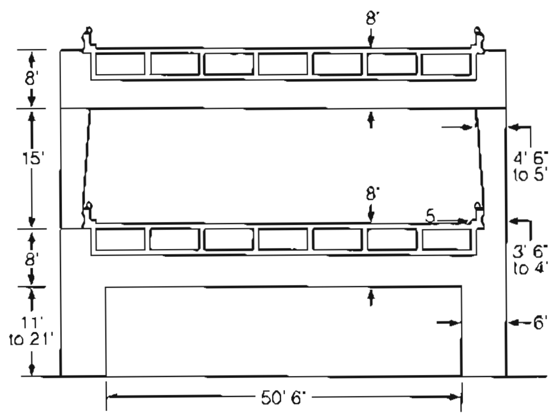


Figure 10-15. *Typical bent dimensions, type B2 bent.*

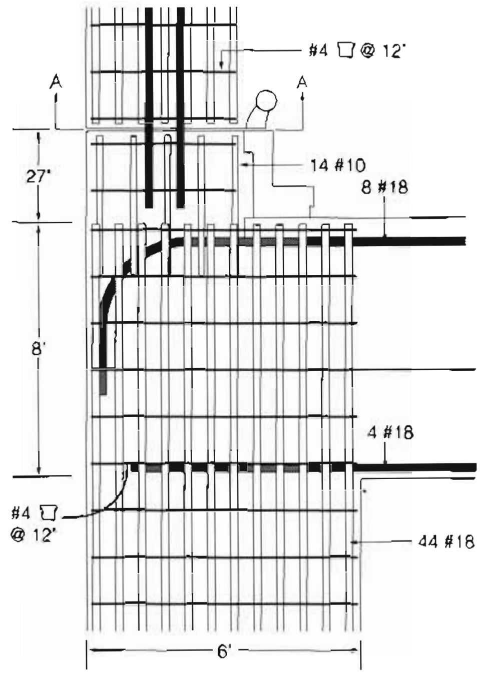


Figure 10-16b. *Typical connection detail 2—B1 bent lower girder-to-column connection.*

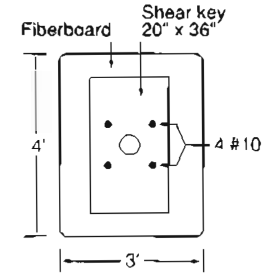


Figure 10-16a. *Typical connection detail 1—shear key.*

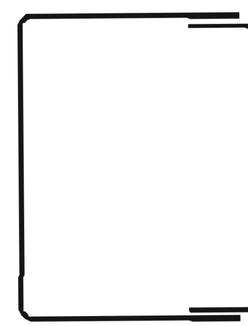


Figure 10-16c. *Typical connection detail 3—configuration of transverse reinforcement in the joint region.*

roadway heading northwest toward the Bay Bridge distribution structure. The four bents at this end of the Cypress Viaduct, Bents 112-115, are of particular interest because Bents 113-115 did not fail. This bent design is referred to as B11. Bents 112-115 are single-level portal frames, and support only the upper roadway. The columns in these bents are taller than those of the other bent types. Bent 112 was the northernmost collapsed bent. This bent design is also shown in Figure 10-9. There are seven other bent designs that occur only once or twice in transition regions, such as where ramps connect to the upper or lower roadways (Table 10-2, Figure 10-9).

Connection Details

The four main connection details that were used in the Cypress Viaduct are discussed below.

Connection Detail 1—Shear Key. A typical shear key detail at the base of an upper column is shown in Figure 10-16a. The cross-section dimensions of the column are reduced at the pin joint by the inclusion of a 1/2" thick layer of expansion joint material at the joint. The shear key dimensions specified in the as-built drawings for the B1 bents are 20" x 36" in plan by 2-5/8" deep. For the other bent types, the narrow plan dimension of the shear key varied between 20" and 30" and the wide plan dimension varied between 24" and 40"; the specified depth was consistently 2-5/8". Evidence from site surveys suggests that the shear keys were deeper than the specified 2-5/8".

The longitudinal reinforcement in the lower columns (typically 44 #18 bars) terminates 27" below the expansion joint material. The only vertical reinforcement bars continuous through the joint are 4 #10 bars, each 52" long, in the shear keys near the center of the column. Some of the columns on the west side of the Cypress Viaduct contain a drain pipe (either 4" or 6" in diameter) that is continuous through the center of the shear key.

The shear keys between the lower columns and the pile caps are similar to the shear keys described above. The typical depth specified on the as-built drawings is 2-5/8"; the narrow dimension ranges from 20" to 30"; the wide dimension varies from 24" to 40"; and 4 #10 bars go through the joint.

Connection Detail 2—Lower Girder-to-Column Moment Connections. The joint detail at the junction of the lower girder and the lower column, immediately below the shear keys, is shown in Figure 10-16b. These joints are found in the B1 bents below both shear keys and in the B2 bents below the lower west shear key. This region provides bearing support to the upper column. The lower girder negative reinforcement (8 #18 bars) enters the top of the joint region and bends down into the column. The bends start near the column centerline and all bars are bent in the same

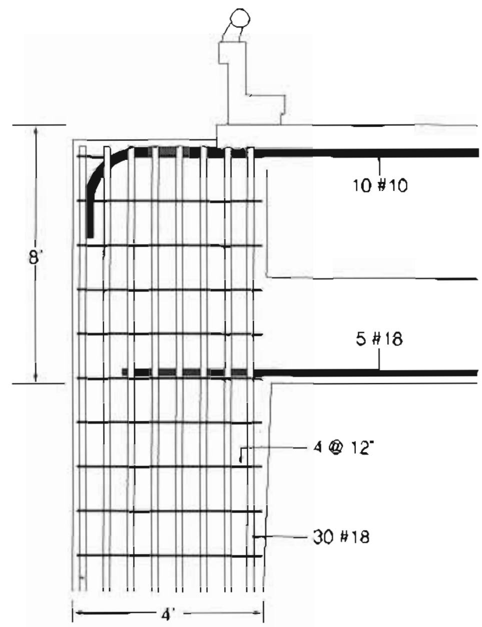


Figure 10-17. *Typical connection detail 4—B1 bent upper girder-to-column connection.*

plane. These bars are bent 5' into the lower column. The reinforcement from the bottom of the girder that enters the joint region consists of 4 #18 bars, and these bars terminate in the joint region of the column. The lower column longitudinal reinforcement (typically 44 #18 bars) is not bent over into the girder, but terminates longitudinally in the joint region. Fourteen longitudinal #10 bars, 5' long, are spliced to the #18 bars; these #10 bars terminate immediately below the shear key. The lateral reinforcement in the joint region is #4 bars at 12".

Connection Detail 3—Lower Girder-to-Continuous-Column Connection in the B2 Bents. The lower girder-to-continuous column connections are on the east side of the B2 bents, where the columns are continuous between the ground and the upper girder. This detail is similar to connection detail 2 described above, except that some of the column longitudinal reinforcement is continuous through the lower girder-to-column joint. The cross-section of the upper column is smaller than that of the lower column.

The as-built drawings show 44 longitudinal #18 bars in the lower column and 22 longitudinal bars (4 #18 and 18 #10) in the upper column. The upper column bars on the north, south, and east faces are shown as weld-spliced to bars in the lower column, although the standard approach at the time was to lap-splice column reinforcement.

Field observations, however, do not agree with the as-built drawings and indicate that the two layers of 7 #18 bars on the east face of the lower east column extended into the upper east column. These bars appeared to be continuous through the upper column and terminated just below the upper girder to upper column expansion joint. No welded splices were apparent, nor was reduction in bar size observed.

The transverse reinforcement in the lower joint region, inferred from the as-built drawings, consists of #4 ties on 12" centers. In some locations, these ties appeared not to be continuous perimeter hoops, but rather being of the configuration shown in Figure 10-16c. The ends of the single #4 ties were hooked 90 degrees but were not bent into the column core.

Connection Detail 4—Upper Girder-to-Column Connection in the B1 Bents. The upper girder-to-column moment joint of a B1 bent is shown in Figure 10-17. The reinforcement in the top of the girder (10 #10 bars) is bent down into the column as described above for detail 2. The longitudinal reinforcement in the column terminates in the upper girder-to-column joint and is not bent over into the girder. The reinforcement in the bottom of the girder (5 #18 bars) terminates horizontally in the joint region of the column.

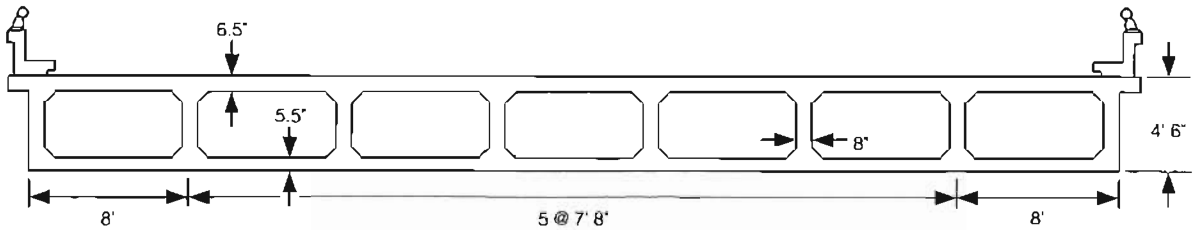


Figure 10-18. *Typical roadway section*

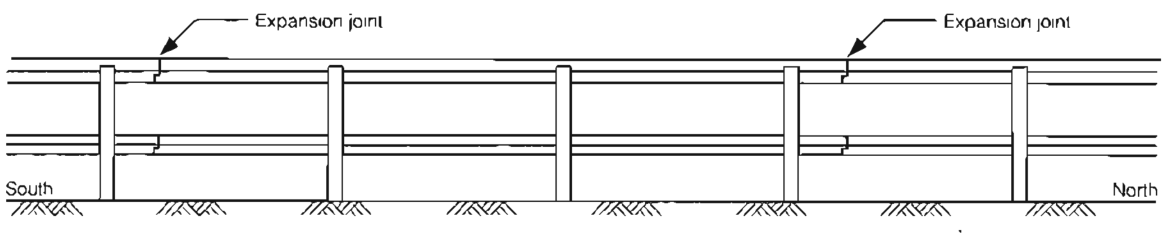


Figure 10-19. *Elevation showing typical location of expansion joints.*

Roadway Deck

The roadway is 52' wide and has four traffic lanes. The typical roadway section is shown in Figure 10-18. The upper and lower decks are similar. The roadway is a seven cell concrete box girder. Overall, the deck is 54'4" wide and 4'6" deep. The cell top is 6-1/2" thick, the bottom is 5-1/2" thick and the webs are 8" thick. The spans between the bents vary from 68' to 90' [State of Calif., Dept. of Trans., various dates].

Expansion Joints

Typically, expansion joints are placed in the roadway every three spans. Table 10-3 lists the locations of the deck expansion joints from Bent 32 to Bent 115. Expansion joints are located 15'-20' north of the bent listed.

Figure 10-19 shows an elevation of a portion of the Cypress Viaduct illustrating the typical location of expansion joints. A detailed elevation of one of these joints is shown in Figure 10-20. The seat of the expansion joint is a 6" outstand and the joint includes 1-1/4" of expansion material. The net bearing length for the supported roadway deck is 4-3/4".

Table 10-3. *Locations of expansion joints.*

Bent No.	Bent No.	Bent No.
32	62	92
35	65	95
38	68	98
41	71	101
44	74	104
47	77	106
50	80	109
54*	83	112
56	86	115
59	89	

*Bent 54 has its expansion joint 15 to 20 feet south of the bent.

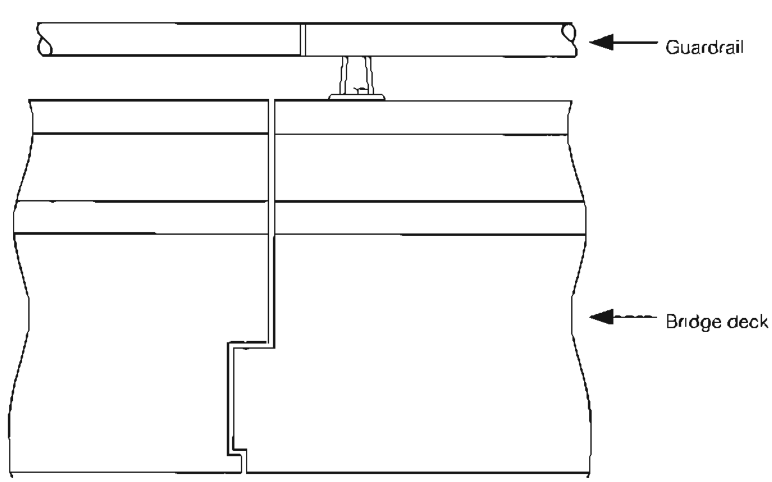


Figure 10-20. *Detail of expansion joint elevation.*

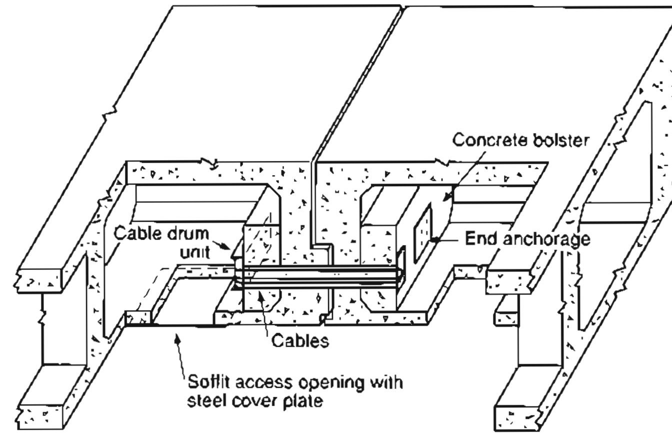


Figure 10-21.
Typical expansion joint retrofit.

Seismic Retrofit of Cypress Viaduct in 1977

As part of the retrofitting program Caltrans instituted following the San Fernando earthquake of 1971, longitudinal restrainers were installed in the Cypress Viaduct in 1977 at all transverse joints in the box girder bridge superstructures [Gates, Dec. 11, 1989]. The retrofitting detail employed in all the expansion joints on the Cypress Viaduct surveyed by the U.C. investigative team is shown in Figure 10-21 [Nims et al., 1989].

The typical restrainer consisted of seven cables tied to anchor plates which bear on the end-diaphragm of the box cells. The anchorage consisted of a plate approximately 12" square and 1-1/2" thick. The cables and anchors were installed in three locations across the width of the deck: in the two exterior cells and in the middle cell (Figure 10-22). They were installed through manholes cut into the soffit of the box girder. This is a typical retrofitting detail [Degenkolb, 1978], and is consistent with what was observed by the survey teams. Typically, the cables used in the California retrofitting program are 3/4" pre-formed 6x19 galvanized cables (Federal Spec. RR-W-410C), and are designed to resist (elastically) a force equal to 25% of the weight of the lighter deck section connected. The swaged end fittings are required to develop the minimum breaking strength of the cable.

The longitudinal restrainers were designed in 1974-75 using interim criteria developed immediately after the San Fernando earthquake and dated March 15, 1971 [Gates, Dec. 11, 1989]. Caltrans used these interim criteria until about 1976, when a more comprehensive approach was developed, which was similar to the Caltrans criteria for new transportation structures.

The 1971 interim criteria used in Cypress Viaduct retrofit design included the following [Gates, Dec. 11, 1989]:

> Restraining features such as keys or restrainer cables were designed to a force level equal to .25 times the contributing dead load applied to the feature. Design was at the working stress level, with an allowance of 33% overstress. It was felt that such a design provided resisting capacity up to .50g at ultimate strength. At the Cypress Viaduct the seismic retrofit consisted of the addition of cable-restrainer units to all expansion joints in the structure. The average joint restrainer configuration used 3/4" cables arranged into three separate double-cable units. At the average joint a total of 7x2x3 or forty-two cables provided restraint. Concrete bolsters were added to strengthen the diaphragms in the vicinity of the restrainer units.

It is important to emphasize that, following the 1971 San Fernando earthquake, Caltrans elected to use the limited available retrofit funds only to install longitudinal restrainers at the transverse expansion joints in box girder bridge spans. This policy decision was made based on Caltrans's perception of the most immediate and critical needs and to prevent failures of the

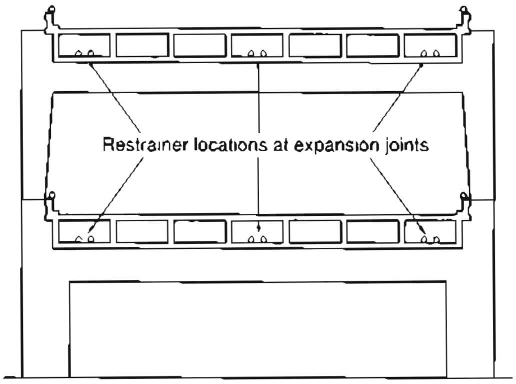


Figure 10-22. *Section of a typical bent showing cable restrainer locations.*

type experienced during the 1971 earthquake. Unfortunately, no detailed global, regional, and local analyses of the entire structural system of the Cypress Viaduct were made at that time or up until its failure on October 17, 1989 to determine if other weaknesses existed. Caltrans changed to ductile concrete detailing design procedures after San Fernando. They knew that their pre-1971 structures were nonductile, but did not fully address their seismic deficiencies.

Earthquake Damage and Failure Modes Sustained by the Cypress Viaduct

Several investigative teams viewed the extensive damage and collapse of the Cypress Viaduct in the days following the Loma Prieta earthquake. As a result of these surveys, several reports were published. The most extensive surveys were performed by Caltrans and by a team from the University of California at Berkeley [Nims et. al., 1989]. The following description of damage sustained by the Cypress Viaduct draws heavily on the work of the U.C. team [Nims et al., 1989]. Their survey began at Bent 32 (10th Street) in the south and continued through Bent 114 (32nd Street) in the north and included both sides of the Viaduct (Figure 10-5). A detailed bent-by-bent description of damage can be found in the report by Nims et al. Table 10-4 summarizes damage sustained by each bent.

Table 10-4. *Bent-by-bent damage summary.*

Bent Nos.	Bent Type	Bent Condition
32-45	B1	Standing, little observed damage.
46-47	B1	Standing, little observed damage, shear cracks in east and west faces of upper deck, location of ambient vibration test.
48	B1	Standing, little observed damage.
49	B1	Standing, cracking in the critical region.
50	B1	Standing, cracking in the critical region, pounding with exit ramp.
51-54	B1	Standing, cracking in the critical region.
55	B1	Standing, cracking in the critical region.
56	B4	Standing, extensive cracking in the critical region.
57-61	B5	Standing, extensive cracking in the critical region.
62	B6	Standing, southernmost standing bent, extensive cracking in the critical region.
63-69	B1	Upper level collapsed, upper girder flat on lower girder, typical B1 failure.
70	B7	Upper level collapsed, upper girder flat on lower girder.
71-72	B2	Upper level collapsed, upper girder flat on lower girder.
73	B8	Upper level collapsed, cantilever failed, upper girder flat on lower girder.
74	B8	Upper level partially collapsed, cantilever failed, transition between upper girder flat and upper girder tilted.
75-80	B2	Upper level partially collapsed, upper girder tilted, pin-ended column remained in place.
81-94	B1	Upper level collapsed, upper girder flat on lower girder, typical B1 failure.
95	B3	Upper level collapsed, upper girder flat on lower girder, east column displaced to north.
96-97	B3	Standing, extensive cracking in lower-girder lower-column joint, evidence transverse cycling.
98	B3	Upper level collapsed, upper girder flat on lower girder, east column displaced to north.
99-103	B1	Upper level collapsed, upper girder flat on lower girder, typical B1 failure.
104	B1	Upper level collapsed, extensive damage to lower girder, upper girder flat on lower girder, both upper and lower decks are on the ground at the expansion joint north of 104, typical B1 failure.
105	B1	Upper and lower levels collapsed, upper girder flat on lower girder, lower girder on ground.
106	B1	Upper level collapsed, extensive damage to lower girder, upper girder flat on lower girder, typical B1 failure.
107	B9	Upper level collapsed, upper girder flat on lower girder, similar to a B1 failure.
108	B9	Upper level collapsed, upper girder flat on lower girder, shear failure in lower east side column, similar to a B1 failure.
109-111	B10	Upper level collapsed, upper girder flat on lower girder, similar to a B1 failure.
112	B11	Collapsed, shear in girder. Abutment showed evidence of transverse motion.
113	B11	Standing, little observed damage, deck sheared completely south of this bent.
114	B11	Standing, little observed damage.

Figure 10-23. *Overall view of the collapse between Bents 84 and 90 (east side)*

Figure 10-24. *Overall view of the failure of the west side of Bent 90*

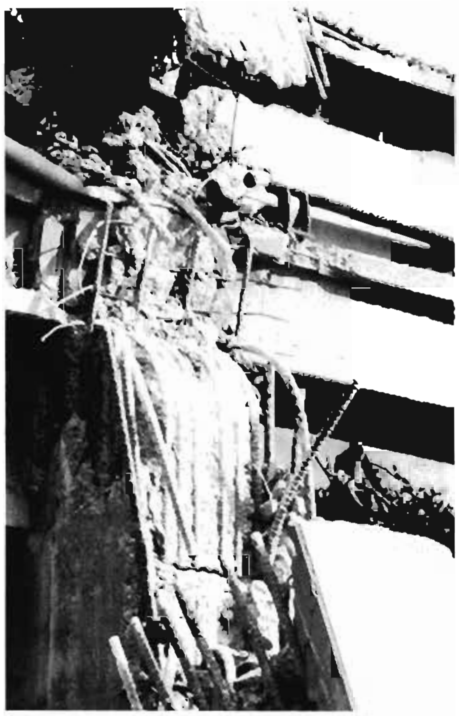


Figure 10-25. *Close-up of the stub and joint regions of Bent 90 (west side).*

Figure 10-26. *East shear key of Bent 108 after the collapse.*

Damage to B1 Bents

Fifty-three of the 82 bents surveyed were type B1 bents. The B1 bents suffered the most damage and failed in a consistent manner throughout the collapsed portion of the Cypress Viaduct. In this portion of the upper roadway, all B1 bents collapsed completely and relatively intact onto the lower roadway (Figure 10-23). The lower columns remained standing, supporting the lower roadway, with the exception of Bents 104-106 (see "Lower Girder Failure," below).

Failure occurred in the lower girder-to-column joints on both sides of the bent, with initial failure of the stub region. The failure surface was defined by the curved negative reinforcement bent down into the joint (Figures 10-24 and 10-25). The shear key did not fail in pure shear, as evidenced by the cone of concrete still attached to the four #10 bars that extended through the key into the lower column (Figure 10-26). The upper girder-to-column joint sometimes failed completely, but in other cases was just severely cracked. Almost all the damage in this upper joint seems to have been produced as a result of the collapse of the upper deck onto the lower deck. The #4 transverse ties in the joint region, as well as those in the column region, either failed or were severely

Figure 10-27. *Overall view of the west side of the collapse between Bents 72 and 76.*

damaged. Examples of this type of B1 bent failure are Bents 63-69, Bents 81-94, and Bents 99-103.

Damage to B2 Bents

The eight type B2 bents also failed in a consistent manner over the entire collapsed section of the Viaduct. In each case the top east column (with one top shear key) failed completely, and the west column (with shear keys top and bottom) either tilted, but did not collapse, as the upper roadway rested on the lower roadway (Figure 10-27 and 10-28), or failed completely and fell to the ground relatively undamaged (Figure 10-29). The lower columns in all B2 bents remained standing and the lower girder suffered only minor damage. In the post-tensioned upper girder however, the unbonded tendons frequently snapped and punched through the concrete cover and anchorage plate (Figure 10-30). The damage to the lower girder-to-column joint on the east side was very similar to that of the B1 bents. The only difference resulted from an additional outer layer of longitudinal reinforcement that was continuous through the joint and which bent back on itself as the joint failed (Figure 10-31). Because of the reduced section formed by the expansion joint material and the loss of anchorage of the four #10 dowel bars, the upper shear keys failed in a manner similar to the lower shear keys in the B1 bents. Column failure was in the plane of the shear key (Figures 10-32 and 10-33). Bent type B2 failure is illustrated by Bents 71 and 72, where the upper deck dropped completely, and Bents 75-80, where the upper deck tilted onto the east side of the lower deck.

Figure 10-28. *Rotations at the upper part of Bent 80.*

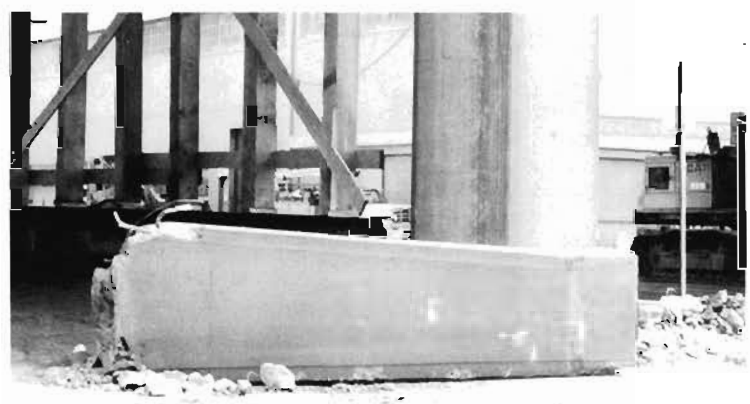


Figure 10-29. *Nearly undamaged upper west column of Bent 71 on the ground*

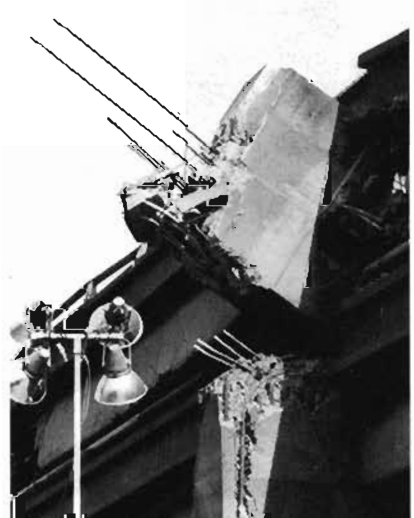


Figure 10-30. *Close-up of protruding unbonded post-tensioning cables.*

Figure 10-31. *Example of failure of the continuous connection of a B2 bent.*

Figure 10-32. *Lower shear key of the upper west column of Bent 71*

Figure 10-33. *Upper shear key of the upper west column of Bent 71.*

Figure 10-34. *Retrofitting cables and anchorage at the expansion joint north of Bent 62*

Figure 10-35. *Damage below the west shear key, Bent 27*

Damage to Expansion Joints and Retrofitting

The expansion joints in the collapsed region were heavily damaged. In the uncollapsed portion, expansion joints suffered severe to minor damage. Most of the damage appeared to be secondary, and resulted from the loss of column support to the upper deck. The retrofits failed either by the anchorage pulling out or by the snapping of the cables, usually at about mid-length. In some places there was evidence of concrete crushing at the lower side of the hole through which the cables passed, indicating that the anchorage failed while resisting the downward motion of the dropping deck. During field investigation, examples of expansion joint damage or retrofitting failures were observed in the expansion joints north of Bent 24, Bent 62, and Bent 112. Figure 10-34 shows damage at the expansion joint north of Bent 62.

Extensive Cracking in Critical Region

Extensive cracking was observed in many of the uncollapsed bents in the critical region of the lower girder-to-column joints immediately below the shear key at the base of the

Figure 10-36. Cracking below the west shear key, Bent 55.

upper column (Figure 10-35). The most serious evidence of this cracking was in the uncollapsed B1 bents, where the cracks followed the same path as the eventual failure surface (Figure 10-36). This same pattern of cracking was observed in many other bents, both collapsed and standing (Figure 10-37), and was indicative of very serious damage—to the point of imminent collapse. Bents 27-29 and Bents 32-62 exhibited this type of extensive cracking.

Figure 10-37. Cracking below the east shear key, Bent 56.

Figure 10-38. Bent 50 and off-ramp where there was evidence of pounding.

Pounding

Debris was found on the ground at Bent 50 between the southbound 14th Street exit ramp and the upper roadway, suggesting pounding between the ramp and roadway (Figure 10-38). The clearance between the exit ramp and the roadway appeared to be about 6-8". Pounding was observed only at Bent 50.

Damage to B8 Bents - Cantilever Failure

The two B8 bents, Bents 73 and 74, had three shear keys, as did the B2 bents. However, the upper east columns were not supported directly on the lower columns, but were supported on a cantilever extension of the lower girder, both of which failed completely. Figure 10-39 shows the cantilever section failure.

Figure 10-39. Failure of the cantilever section of Bent 73 (east side)

Damage to Skewed B3 Bents

The four B3 bents, Bents 95-98 were the only bents with continuous columns on both sides of the roadway. Bents 96 and 97 were the only two bents between Bents 63-112 to remain standing. The B3 bents were skewed

Figure 10-40. *General view of standing Bents 96 and 97 (east side).*

Figure 10-41. *Upper part of Bent 97 (west side)*

Figure 10-42. *Overall view of the collapse between Bents 104 and 107 (east side).*

Figure 10-43. *Failure of the lower girder at Bent 104*

Figure 10-44. *Close-up of the failure of the lower girder in Bent 104 (west side)*

in order to accommodate 26th Street, which passes under the Viaduct at an angle. Bents 95 and 98 were only partially skewed to the Viaduct, and the upper level failed at those bents in a manner similar to the failure mode of the B2 bents. Failure occurred as a result of shear failure of the deck, not at an expansion joint. Bents 96 and 97 were both very skewed to the Viaduct and remained standing (Figure 10-40). Both bents were severely cracked, however, in the critical lower girder-to-column joint region (Figure 10-41).

Lower Girder Failure

The lower girder of Bents 104-106 failed next to the girder-to-column joint region, causing the girder to drop to within a few feet of the ground (Figure 10-42). The upper deck collapsed onto the lower deck in a typical B1 type failure. These were the only bents that exhibited this type of lower girder failure. Figures 10-43 and 10-44 show the failure of the lower girder of Bent 104 in the region west of the column joint.

Figure 10-45. *Shear failure of the east lower column of Bent 108*

Figure 10-46. *Close-up of the shear failure of the east lower column of Bent 108*

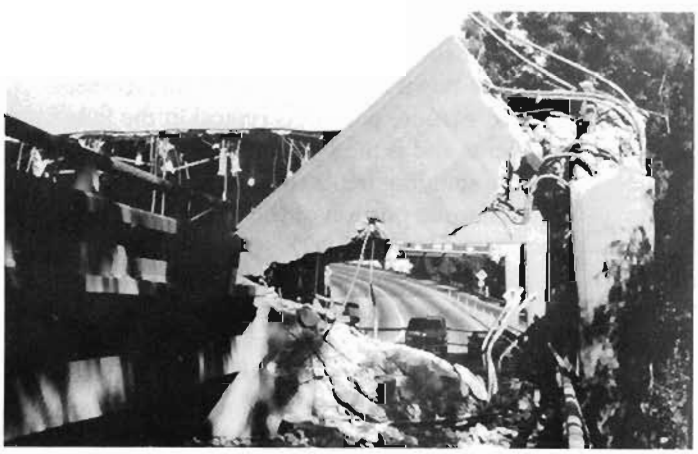


Figure 10-47. *Shear failure in upper girder of Bent 112 (east side).*

Figure 10-48. *Shear failure of roadway just south of Bent 113*

Failure of Columns in Shear

At only one location, Bent 108, did a column fail in shear. Bent 108 was a B9 bent, which is similar to a B1 bent but with a shorter lower column height. Figures 10-45 and 10-46 show the shear failure of the east column of Bent 108.

Failures of Girder in Shear, Box Girder Deck in Shear, and Rocker Bearings

At only one location, the northernmost collapsed Bent 112, did the upper girder fail in shear (Figure 10-47). Bent 112 was a type B11 bent, with the bent supporting only the upper roadway. The lower deck was supported directly on the ground with rocker bearings. The upper roadway box girder deck failed in shear just to the south of Bent 113 (Figure 10-48). The rocker bearing retainer plate moved about 1" in the transverse direction (Figure 10-49) and one of the bolts was sheared off.

Figure 10-49. *Close-up of rocker bearing at Bent 112 abutment indicating transverse motion*

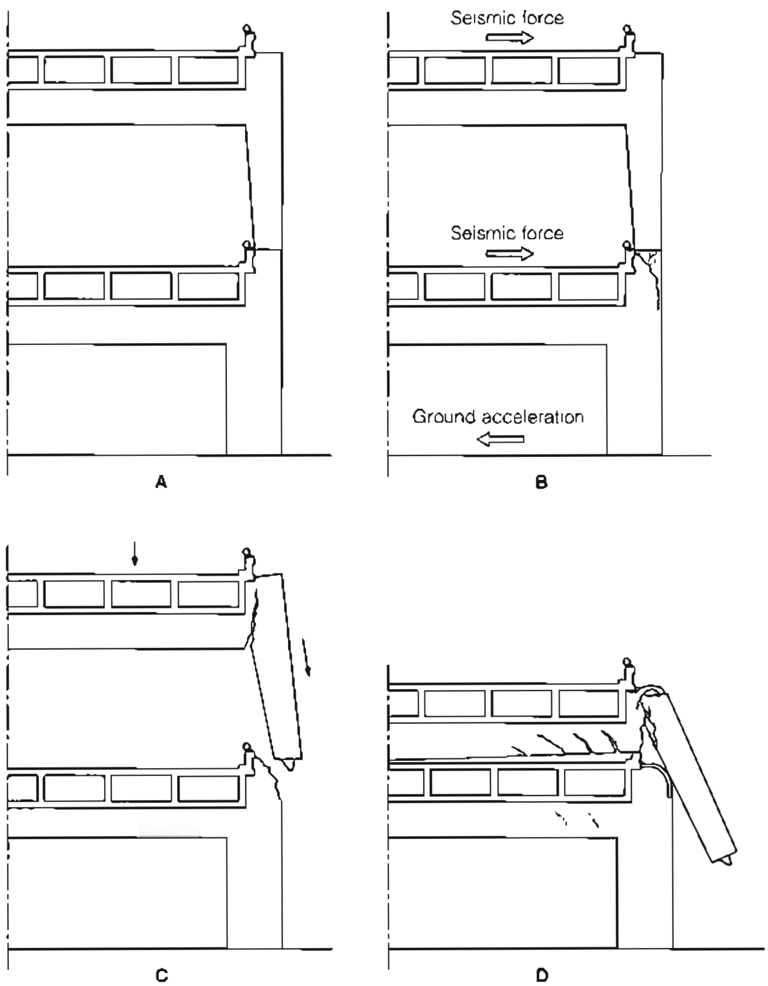


Figure 10-50. *Typical failure sequence of a B1 bent.*

Typical Bent Failure Mode

Bent damage has been cataloged and described by the U.C. investigators [Nims et al., 1989]. The description of failure modes, both typical and nontypical, presented below draws heavily, and in some cases directly, on their work.

All but seven of the 48 collapsed bents failed in the same way (Table 10-1). The common failure mode was characterized by a failure surface that developed in the stub region, and in some cases in the lower girder-to-column joint (Figure 10-50). This failure surface formed above both of the lower girder-to-column joints of the failed B1 bents, and in the continuous (east) lower girder-to-column joints of all the failed B2 bents. The failure surface is defined by the line of the lower girder negative reinforcement (8 #18) that bent down through the joint region and terminated inside the joint. This failure plane in the B1 bents extended from the stub region to the lower outside corner of the joint, approximately to the level of the underside of the lower girder. The lower portion of the failure crack was approximately coincident with the tail of the bent-down negative girder reinforcement. This failure pattern is typical in the failed bents and is the typical pattern of cracking in the standing bents to the south of the collapsed portion of the Viaduct (Bents 27-29 and Bents 55-62).

Damage and secondary failures in other locations within the frames were more variable. The condition of the upper girder-to-column joints of the B1 bents varied from complete failure to only moderate damage. Bent 108, for example, failed in the lower girder-to-column joints, but the upper east joint—although significantly cracked—did not fail, even though the upper deck had collapsed completely onto the lower deck. Similarly, damage to the upper columns was quite variable. Some of the columns were completely split vertically while others were essentially intact. Some crack patterns in the lower girder-to-column joints of the standing bents are shown in Figures 10-35, 10-36, and 10-37.

The lower girder negative reinforcement consisted of 8 #18 bars. These bars were bent down into the lower column inside the outer layer of longitudinal reinforcement. Typically, there were two layers of 7 #18 bars in the outer column face, with the 8 #18 girder bars bent down between these layers. No dimension is given on the as-built drawings [State of Calif., Dept. of Trans., various dates] for the spacing of these two

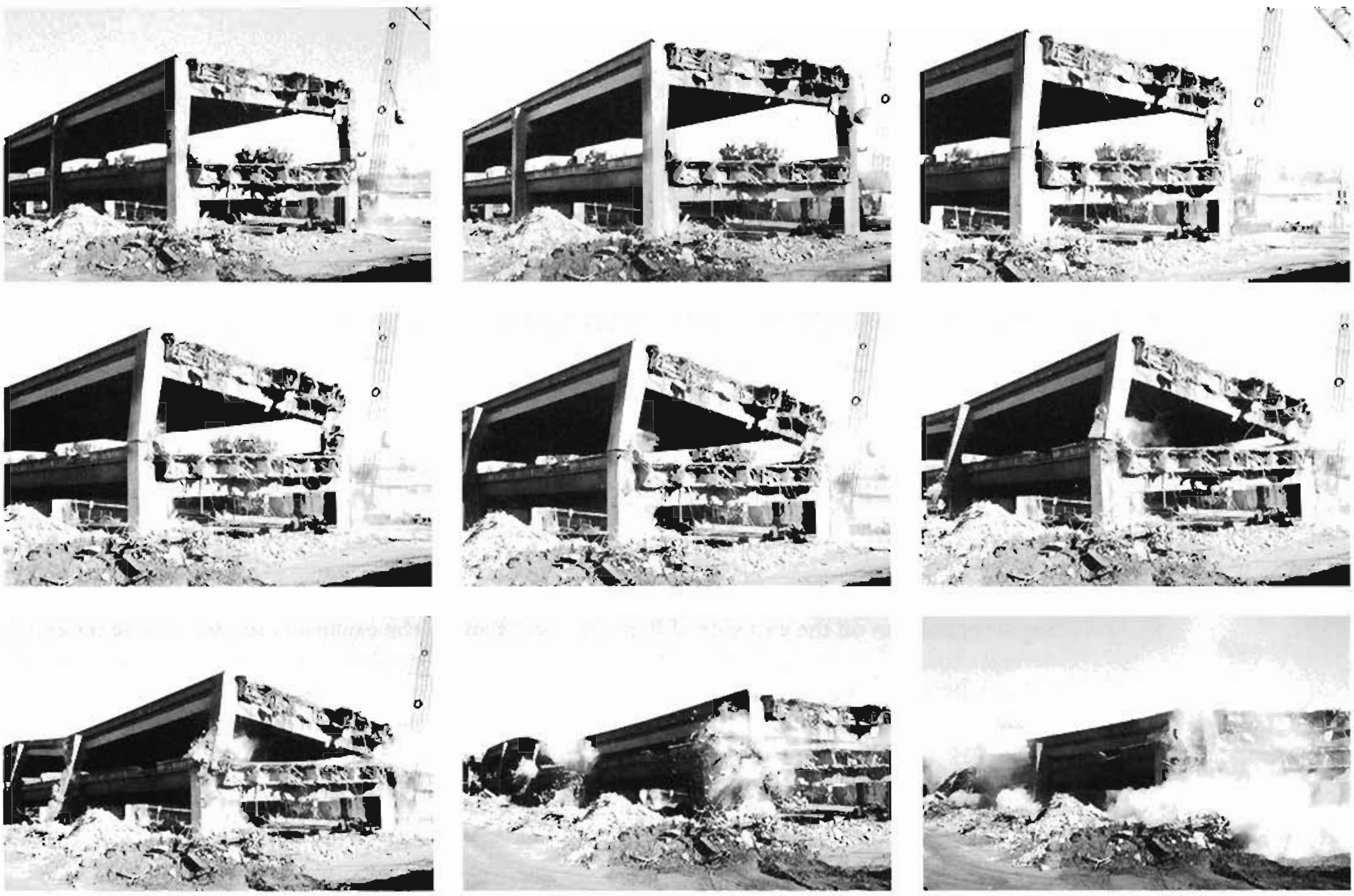


Figure 10-51. *Failure sequence after demolition of a single column*

layers, but from field inspection it appeared that the layers were less than 6" apart. In most of the B1 bents a cone was formed below the shear key, although in some cases a wedge was formed instead. The closely spaced 8 #18 bars formed a plane of weakness in a region where the shear stresses were high and the reinforcing bars were poorly restrained. The large number of bars disrupted the diagonal shear capacity of the concrete. In addition, the 8 #18 bars were poorly restrained and tried to kick out, which contributed to the vertical crack in the column over the tails of the #18 bars. The 8 #18 bars also provided a smooth ramp down which the cone or wedge was subsequently able to slide.

The transverse steel in this girder-to-column joint was the only reinforcement that directly provided restraint to the girder negative reinforcement and resisted the tension forces originated by the transverse shear forces. Consequently, this transverse steel was the only reinforcement (at most 5 #4 ties) that could prevent the failure in the stub region i.e. the movement of the failure wedge downward and outward. This reinforcement was clearly not adequate to resist the loads imposed on the stub and lower girder-to-column joint regions.

The common failure mode is illustrated by the sequence shown in Figure 10-50. A diagonal crack forms in the stub and lower girder-to-column joint regions. This crack follows a plane of weakness in the joint region defined by the plane of the bent-down negative girder reinforcement. Then the gravity and seismic forces push the upper column down and away from the joint, resulting in the collapse of the upper deck. Much of the damage to the upper columns, roadway and girders was subsequently caused by the impact of the upper deck on the lower deck.

This mode of failure is vividly demonstrated in the photo sequence of Figure 10-51, which was taken after the earthquake and during the demolition of an uncollapsed segment of the Cypress Viaduct. The uncollapsed segment consisted of four spans supported by five bents—Bent numbers 49, 50, 51, 52, and 53. A wrecking ball struck the top story column on the east side of Bent 49 several times to initiate its failure. When the column began to fail, a progressive collapse ensued and resulted in total failure of the entire five-bent structure. Failure followed the typical failure mode observed in the sections that collapsed during the earthquake.

Nontypical Bent Failure Modes

Bents 73 and 74 are B8 bents and are shown in Figure 10-9. They are similar to type B2 bents, except that the upper east columns were supported on a cantilever extension of the lower girder. A shear failure of the lower girder cantilever stub led to the failure of these bents (Figure 10-39). The failure surface may have been initiated by flexure. The reinforcement in the top of the cantilever terminated close to the inner face of the supported column above it. The resistance of the cantilever to sliding shear was basically no more than the shear strength of the concrete.

Bent 112 was the northernmost collapsed bent and was a B11 bent. This bent supported the upper roadway only, and the lower roadway terminated at the north abutment. Figure 10-47 shows the east side of Bent 112 after collapse—the column remained standing but the girder failed in shear. Bent 112 was the only bent where girder failure caused the bent to collapse. The expansion joint approximately 20 feet north of Bent 112 failed during the earthquake. Three sets of U-shaped cable restraints tied the decks together longitudinally at the expansion joints. The outer two sets suffered anchorage failure, while strands in the central pair of cables snapped. North of the expansion joint, the roadway deck failed in shear (Figure 10-48).

Standing Bents in the Cypress Viaduct

Debris on the ground at Bent 50 suggested pounding between the bent at the level of the upper roadway and the adjacent exit ramp (Figure 10-38). The clearance between the exit ramp and the bent appeared to be about 6-8"; inspection showed no signs of damage consistent with this level of lateral displacement in the bent. If the bent had deformed 4" to 6" more, it would probably have collapsed. A review of the as-built drawings [State of Calif., Dept. of Trans., various dates] showed that the connection of the exit ramp column to the pile cap was a joint that incorporated a shear key. Therefore, this joint had little moment capacity. Pounding appears to be primarily the result of rigid body rotation of the exit ramp about a north-south axis at the top of the pile cap.

The span of the upper roadway between Bents 96 and 97 was the only portion of the upper deck between Bents 63 and 112 that survived the earthquake. Bents 95-98 were B3 bents and were skewed to the north-south axis of the Viaduct (Figure 10-40). The skew alignment of the bents was to accommodate 26th Street, which passes under the Viaduct between Bents 96 and 97. The B3 bents have three columns at the lower level and shear keys at the junction of the upper girder and upper columns. The reinforcement in the outer face of the east and west columns is continuous through the lower girder-to-column joint region. These were the only bents in the collapsed portion in which column continuity was provided on both sides of the Viaduct. Although Bents 95 through 98 were essentially identical, only Bents 96 and 97 survived. Factors that may have contributed to the collapse of Bent 95 and 98 are the collapse of the adjacent B1 bents (Bents 94 and 99) and the influence of the expansion joints between Bents 95 and 96 and between Bents 98 and 99.

Results of Analytical Investigations

Several analytical investigations have been made of the Cypress Viaduct since the earthquake of October 17, 1989. These have included static or dynamic analyses of the following analytical models:

- Local (joint) level by Priestley and Seible, Moehle et al., and Krawinkler.
- Regional level (single bent) by Wilson, Priestley and Seible, Moehle et al., Krawinkler, Lew et al., Einashai et al., and Werne et al.
- Global level (three bents or more) by Mahin et al. and by Werne et al.

Many of these analytical investigations, as well as others, are still in progress. In some cases only preliminary results are available based on various assumptions for material properties and models, seismic input, and other factors. For example, design values of f'_c = 4,000 psi for concrete compressive strength and f_y = 40,000 psi for reinforcing steel yield stress were assumed by some investigators, while samples taken from the structure after the earthquake and tested at U.C. Berkeley laboratories gave f'_c = 6,000 to 7,000 psi and f_y = 42,000 to 50,000 psi for various steel bar sizes.

There are no ground motion records available from the Cypress Viaduct site, therefore various dynamic analyses have used ground motion records from sites within 1-2 miles: (1) the outer harbor wharf in Oakland to the west (CSMIP Station 427); (2) near a multistory building in Emeryville to the north (USGS Station 1662); or (3) near a two-story building in Oakland to the east (CSMIP Station 224); and (4) in one early study [Wilson, 1989] the ground motion at the San Francisco Airport several miles away was used.

The fact that ground motion records are from sites other than the Cypress Viaduct should be kept in mind when interpreting or comparing results from the different analytical studies discussed below. Nevertheless, these analytical investigations offer valuable insight into the structural response and collapse of the Cypress Viaduct in the earthquake.

Static Analyses

Static analyses of bent type B1 (Figures 10-10 to 10-14), with two hinges at the bottoms of the upper-story columns have been performed by Priestley et al.; Moehle et al.; Krawinkler; and Lew et al. Even though their assumptions vary somewhat, their calculations tend to confirm the bent failure mode described previously. Twentynine B1 bents and eight B2 bents collapsed. Static analyses of individual bents consisted of the following steps:

1. Analyze for a known total vertical dead load W.
2. Analyze for unknown lateral loads 2F at the top-story level and F at the bottom-story level, giving a total horizontal base shear at the footings due to earthquake of V_{EQ} = 3F [Priestley and Seible, Nov. 9, 1989].
3. Determine internal shear and moment capacities of members and joints at critical locations.
4. Use the results of Steps 1, 2, and 3 to determine the force F and thus 2F and V_{EQ} = 3F to initiate first failure at the weakest location and subsequent sequences of failure until the structure collapses.

Inherent in each of the above steps are certain assumptions and uncertainties. As an example, consider bent B1, which has been analyzed by Priestley et al. [Nov. 9, 1989], Moehle [testimony, Dec. 14, 1989], and Krawinkler [1990]. In Step 1 the total dead load W tributary to the bent can be calculated accurately for a longitudinal span of about 80 ft and is about W = 2,800 kips [Priestley et al., Nov. 9, 1989], evenly divided between the top and bottom girders. A linear elastic analysis of the bent frame for this loading can be made based on uncracked gross section or cracked section member properties to determine values of the horizontal shear H_w at the critical shear key hinges, as well as bending moments M_w and shear V_w at all locations in the girders and columns. Uncertainties in this are the member stiffnesses, cracked sections, hinge restraints, and the long-time effects of creep and shear key slip on the horizontal shear H_w and member M_w and V_w values. Neglecting these, calculated values of H_w = 145 and 130 kips were found by Priestley et al. [Nov. 9, 1989] and Moehle [Dec. 9, 1989], and including some of the uncertainties, a range of 100 to 200 kips was predicted by Krawinkler [1990].

For Step 2 the common assumption of lateral forces, 2F and F, at top- and bottom-story levels as a first- mode approximation is used in an equivalent static analysis to represent an earthquake. These can be compared to the results of dynamic analysis discussed later. For this case the values H_c, M_c, and V_c in the structure and base shear V_{EQ} can be calculated in terms of F using the same type of frame analysis as in Step 1. At each upper level shear key hinge H_c = F and the base shear V_{EQ} = 3F.

In Step 3 internal ultimate capacities are calculated based on code formulas or simple strength calculations and critical points are identified. For B1 type bents, these are the horizontal shear key strength of the pedestal H_n and the $+M_n$ in the top girder at the face of the top girder-column joint. Calculated values of H_n = 272, 270, and 280 kips were found by Priestley et al. [Nov. 9, 1989], Moehle [testimony, Dec. 14, 1989] and Krawinkler [1990], respectively. Calculated values of $+M_n$ = 3,580 and 3,800 ft kips were found by Priestley et al. and Krawinkler based on the insufficient bond anchorage into the top joint of the bottom five #18 bars in the top girder.

The calculations of H_n and M_n are based on ACI code or committee formulas and various materials assumptions and can only be verified by experiment. H_n is a key value in predicting the lateral loads to cause collapse. In the last Step 4 of the static analysis, the results of Steps 1, 2, and 3 are combined. The horizontal shears at the shear key hinge are additive on one side of the bent and subtractive on the other side. Taking the critical additive case, $H_w + H_e = H_n$, in which $H_e = F$ and $F = H_n - H_w$, giving values of F = 127 and 140 kips found by Priestley et al. and Moehle, and a range from 80 to 180 kips by Krawinkler. In the following, assume that F = 127 kips found by Priestley et al. is a representative value, then due to the lateral earthquake load the horizontal shear in the top story 2F = 254 kips, and the structure base shear V_{EQ} = 3F = 381 kips = (381/2,800) W= 0.14W. From the predictions for F by Moehle and Krawinkler it can be seen that this value for V_{EQ} could vary from 0.09W to 0.19W. The above upper-story shear values of 2F and the base shear values V_{EQ} found analytically should be compared with the experimental value of 2F=465 kips sustained by the non-retrofitted test structure described below under "Experimental Investigations."

When one shear key pedestal fails, the total top-story shear of 2F must be carried by the single pedestal on the other side, given that it did not fail on the same cycle. The shear that can be transmitted is limited by the maximum capacity of $+M_n$ = 3,580' kips in the top girder at the upper joint face due toinadequate bond anchorage of the bottom bars. Priestley et al. calculate V_{EQ} = 0.11W, in which case a catastrophic collapse (Figures 10-50 and 10-51) would occur after only one or two cycles of lateral earthquake motion.

Priestley et al. have made similar static calculations to those above for other bents, which failed at higher lateral loads. It should be noted that they used design values of f'_c = 4,000 psi for the concrete and f_y = 45,000 psi for the reinforcing steel in their calculations, whereas subsequent tests of the concrete at U.C. Berkeley indicated that f'_c = 6,000 to 7,000 psi. For bent B2, their analysis indicates that initial damage occurs by developing a flexural plastic hinge at the negative moment end of the bottom transverse girder at a base shear V_{EQ} = 0.17W. A full mechanism then developed at V_{EQ} = 0.21W, involving plastic hinges at both ends of the bottom girder, resulting in collapse; at this point the shear strength at the base of the continuous upper column was exceeded. In addition, Priestley et al. performed an inelastic finite element analysis of a two-dimensional reinforced concrete model of the shear key pedestal region which predicted crack formation quite well and a higher-than-ACI-code-predicted shear failure load of H_n = 390 kips.

Bertero [testimony, Dec. 14, 1989] stated that his static analysis indicated that one of the two B8-type bents in the structure with a slight bottom girder cantilever overhang would have failed at a lower lateral load than the B1 bents. Failure was attributed to vertical cracks due to negative moment initiating in the short cantilever of

Table 10-5. *Frequencies and periods in transverse direction.*

	1st Mode		2nd Mode	
	Frequency (Hz)	Period (sec.)	Frequency (Hz)	Period (sec.)
Wilson [1989]	2.4	0.42	6.1	0.16
Moehle [1989]	2.6	0.38	6.9	0.14
Krawinkler [1990]	2.4, 2.1	0.42, 0.47		
Lew et al. [1990]	2.5	0.40	6.4	0.15

the lower girder. This cracking was followed by a flexural failure owing to the fact that all of the #18 bars indicated on the drawings were not actually carried beyond this critical section. This was determined from analysis of photographs taken after the collapse.

Lew et al. [1990] also have made extensive static analyses of bents B1, B2, and B8 similar to those reported above by Priestley et al. and Bertero. They used f'_c = 6,000 psi, resulting in slightly higher values of V_{EQ} than those found by Priestley et al. Their calculated values for lateral base shears V_{EQ} to initiate failures were 0.20W for B1 and 0.25W for B2. For B8 they reported that if the bars specified in the drawings actually existed and had sufficient anchorage, failure would not have initiated there. However, if, as reported by Bertero, they did not, failure could have initiated at a base shear V_{EQ} as low as 0.10W.

Based on the review of the published static analyses, even though Bent B8 might have been the first bent to fail, the level of ground shaking experienced by the structure was sufficient to cause damage to other bents, and so it is believed that the failure of the 29 B1 bents under transverse earthquake motion was the primary cause of the collapse of the Cypress Viaduct.

Dynamic Analyses

Results of dynamic analyses were reported to the Board of Inquiry by Wilson [1989], Krawinkler [1990], Einashai et al. [1989], and Lew et al. [1990] for single-bent models and by Mahin and Moehle [testimony Dec. 27, 1989] for the three- bent test structure, and preliminary results were presented to the Board on March 1, 1990 by Werne [testimony Mar. 1, 1990] for static and dynamic analyses of one-, three-, and multiple-bent models.

Dynamic analyses determine one or more of the following:

1. Frequencies, periods, and mode shapes.
2. Maximum displacements and internal forces and moments resulting from a given input response spectrum at the foundation.
3. Time history of displacements and internal forces and moments resulting from a given input time history at the foundation.

Somewhat different analytical model of the structure have been employed by different investigators utilizing one- or two-dimensional elements and different assumptions for the stiffness properties of the elements and the hinges. Calculated frequencies and periods for the first two modes in the transverse direction are summarized in Table 10-5. These values are in good agreement with experimental values obtained from ambient measurements and forced vibration field tests discussed below. The two values given by Krawinkler for the first mode include one for a model with uncracked stiffness properties for all elements and the other with cracked stiffness properties for the columns only. Wilson and Moehle used uncracked properties, and all three used linear elastic responses.

Krawinkler [1990] also performed response-spectra and time- history response analyses using ground motion records 1.2

miles to the north at Emeryville (USGS Station 1622) on a soil formation site similar to that existing at the collapsed northern portion of the Cypress Viaduct and from a two-story building in Oakland 1.2 miles to the east (CSMIP Station 224) on a soil formation site similar to that existing in the damaged, but not collapsed, southern portion of the Cypress Viaduct. Results of Krawinkler's dynamic analyses for the Emeryville record indicate a predominant first-mode response with maximum total story shear demands for the cracked column model of about 730 kips in the top story and 1,300 kips in the bottom story. The ratio of these maxima is 1,300/730 = 1.78 which is considerably greater than the 1.50 assumed in the static analysis. The maximum story shears found using the Oakland record were very similar to those found using the Emeryville record. Consequently, the elastic response of the structure to these two records cannot explain the differences between the behavior of the northern and southern sections of the Cypress Viaduct in the two different soil formations. However, Krawinkler speculates that because the elastic strength demand of the Oakland response spectra decreases significantly between a period of 0.45 to 0.60 seconds while the Emeryville record does not, the decrease in structure stiffness and increase in period due to cracking and deterioration may have decreased the strength demand of the southern portion significantly, but not the northern portion of the Viaduct structure. The base shear of 1,300 kips represents about an equivalent:
$V_{EQ} \sim (1{,}300/2{,}800)W = 0.46W$.

A preliminary report by Moehle [testimony Dec. 14, 1989] on a linear dynamic analysis of the three-bent test structure using the response spectra at the Oakland Wharf (CSMIP Station 427), 1.2 miles west of the Cypress Viaduct, gave maximum total story shears per bent of about 850 kips in the top story and 1,100 kips in the bottom story, or a ratio of 1,100/850 = 1.29, which is less than the 1.50 assumed. The base shear of 1,100 kips represents a $V_{EQ} \sim (1{,}100/2{,}800)W = 0.39W$. The dynamic analysis results given by both Krawinkler and Moehle indicate their maximum elastic top-story shear demands (730 and 850 kips) greatly exceed the static capacities calculated by Priestley et al., Moehle, and Krawinkler to be 254, 280, and 160 to 360 kips. These analyses all indicate that failure would be expected for this nonductile structure.

Preliminary results by Wilson [1989] for a dynamic analysis of a single bent, modelled with two-dimensional finite elements and subjected to a ground motion record obtained at the San Francisco Airport, the only one available at the time, demonstrated similar dynamic amplifications. The maximum input ground acceleration used was 0.30g, which amplified to 0.44g at the lower deck and 0.70g at the upper deck.

Additional dynamic analyses are in progress by several investigators at the time of this writing. However, it is evident from the above results that the Cypress Viaduct could not survive the ground motions generated by the Magnitude 7.1 earthquake of October 17, 1989 if the capacities were as predicted by the static analyses.

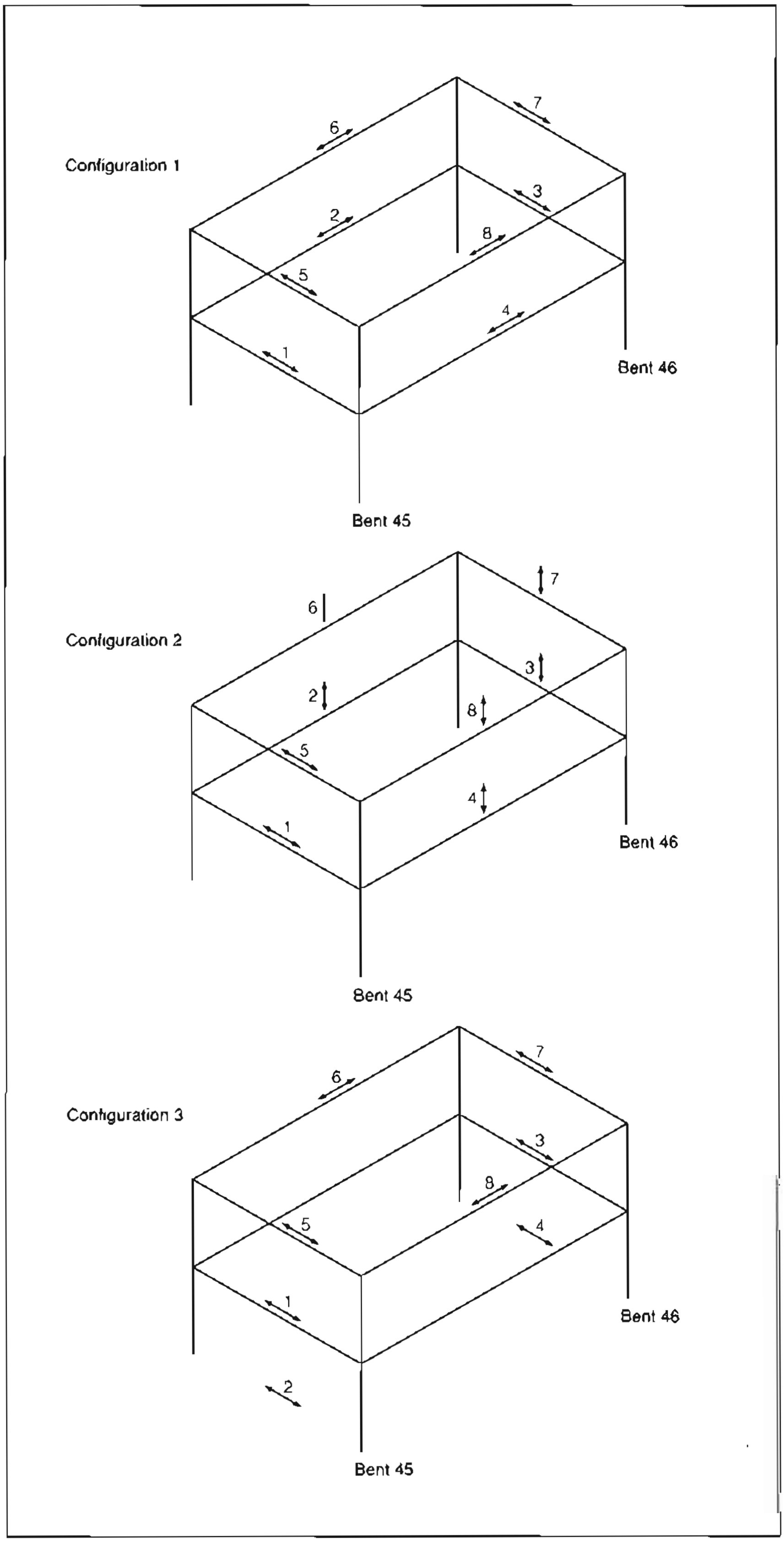


Figure 10-52. *Instrument configurations for ambient vibration tests, arrows show location and orientation of Kinemetrics Ranger seismometers.*

Experimental Investigations

Between October 24 and December 27, 1989, researchers from the University of California at Berkeley, under the direction of Mahin, Moehle, and Stephen, carried out several experiments on the southern portion of the Cypress Viaduct that remained standing. Ambient vibration measurements were made on October 24, during the early stages of demolition, and these were followed by forced vibration experiments and static load tests on a 2-span segment between Bents 45 and 47, which had been left standing after the rest of the structure was demolished. Investigations were conducted as part of a research agreement between Caltrans and U.C. Berkeley.

Ambient Vibration Measurements

The ambient vibration measurements were made on the undamaged span between Bents 45 and 46. These bents were in their original state, and no retrofitting of any kind existed at the time the ambient measurements were made. Configuration of the instruments on the span between Bents 45 and 46 and results for transverse, longitudinal, and vertical frequencies, periods and mode shapes are discussed in detail by Nims et al. [1989]. Their results are summarized in Figure 10-52 and in Tables 10-6 and 10-7. The measured transverse first-mode frequency and period of 2.5 Hz and 0.39 seconds are in good agreement with the computed values of 2.4 to 2.6 Hz and 0.38 to 0.42 seconds for the uncracked analytical model. Other first-mode periods from

Table 10-6. *Ambient vibration: frequencies and mode shapes for horizontal modes.*

Mode	1st Transverse	2nd Transverse	1st Longitudinal	2nd Longitudinal	1st Torsional
Period (sec)	0.39	0.22	0.65	0.33	0.27
Frequency (Hz.)	2.5	4.5	1.6	3.0	3.7
			Mode Shapes		
Upper Level	1.00	1.00	1.00	1.00	1.00
Lower Level	0.45	-0.51	0.52	-0.57	0.50
Ground	0.00	0.00	0.00	0.00	0.00

Table 10-7. *Ambient vibration. frequencies and mode shapes for vertical modes.*

Mode	1st Vertical	2nd Vertical
Period (sec)	0.19	0.17
Frequency (Hz.)	5.3	6.0
Mode Shapes	1st vertical	2nd vertical
Channel 2	0.29	1.00
Channel 3	0.07	0.12
Channel 4	0.29	1.00
Channel 6	0.88	0.52
Channel 7	0.24	0.25
Channel 8	1.00	0.56

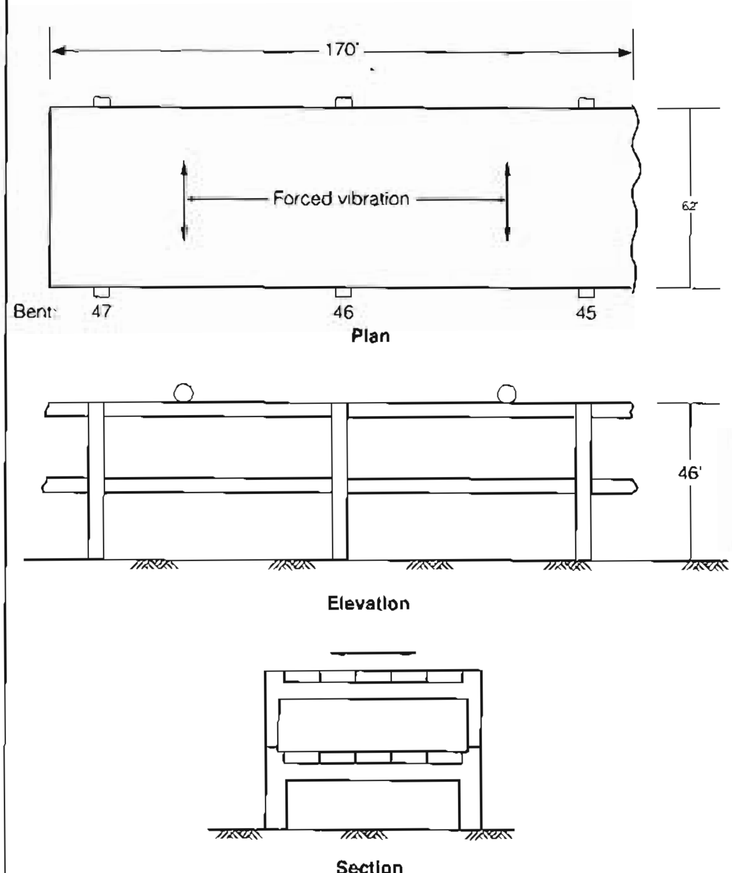


Figure 10-53. *Schematic view of Cypress Viaduct test structure.*

Tables 10-6 and 10-7 are 0.65, 0.27 and 0.19 seconds for longitudinal, torsional and vertical vibrations, respectively.

Experimental Investigations Performed on Three-Bent Test Structure

The tested segment of the original Cypress Viaduct consisted of three 62' wide bents—Bents 45, 46, and 47—plus a 170' length of the top and bottom seven-cell box girder bridge deck. The top deck was 46' above the ground. A schematic of this test structure is shown in Figure 10-53. Note that the original top and bottom box girder bridge deck overhangs south of Bent 45 were cut back to 20' to make them symmetrical with the original overhangs north of Bent 47. Bents 45, 46, and 47 were typical type B1 bents (Figures 10-9 and 10-10), which static and dynamic analyses predicted would fail under transverse earthquake ground motion in a manner shown in Figures 10-50 and 10-51, with the weak link being the shear resistance in the pedestal below the shear key hinge at the bottom of the upper story columns (Figure 10-14).

Initially, forced vibration tests and static load tests were conducted on the original unretrofitted test structure. After these tests were completed, three different retrofit schemes were installed, one on each of the three bents. A detailed description of these retrofit schemes has been presented to the Board in a preliminary letter report [Mahin et al., 1989]. The retrofitted test structure was then subjected to forced vibration tests and then to increasing static cyclic load excursions and corresponding displacements to evaluate the different retrofitting schemes.

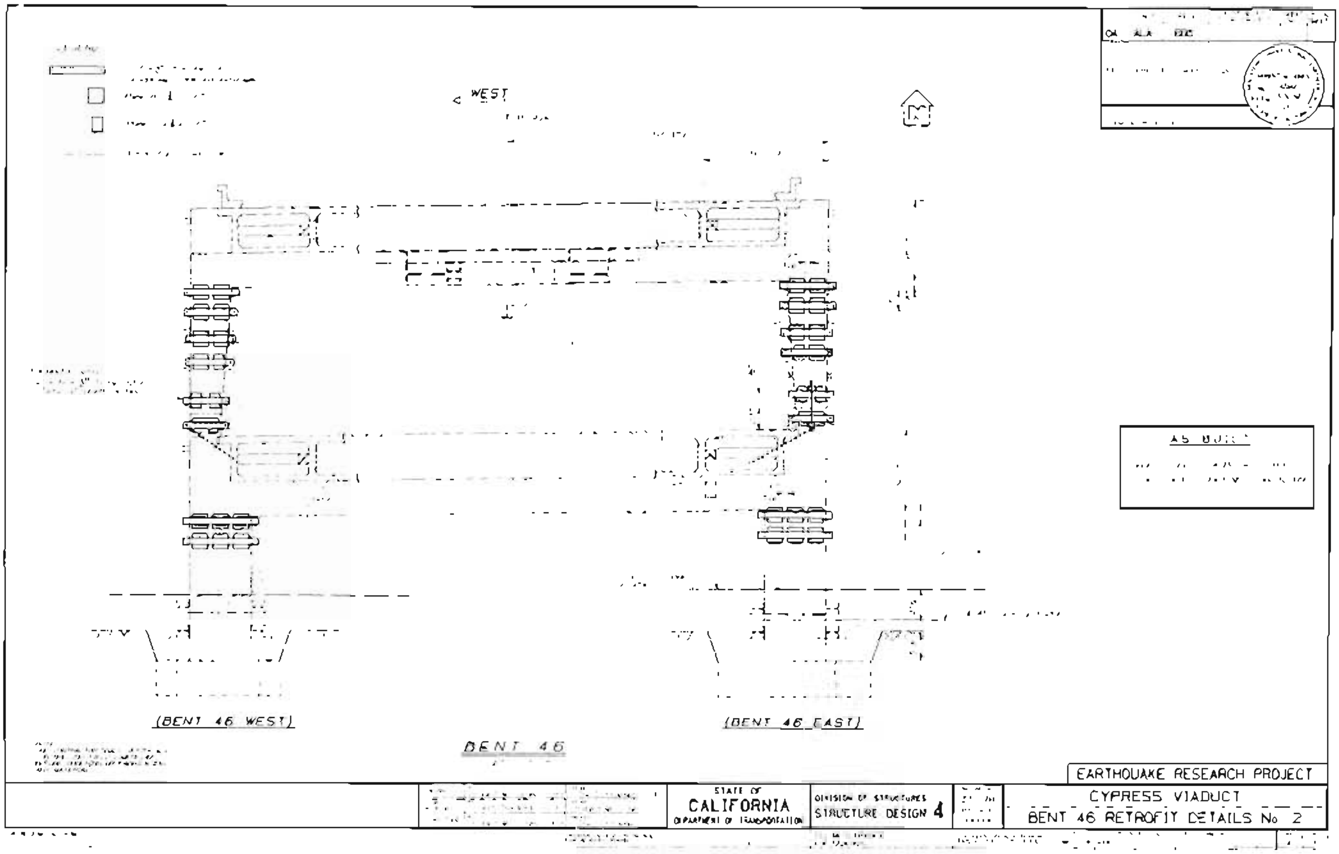


Figure 10-54. *Caltrans as-built drawing, Bent 45 retrofit details.*

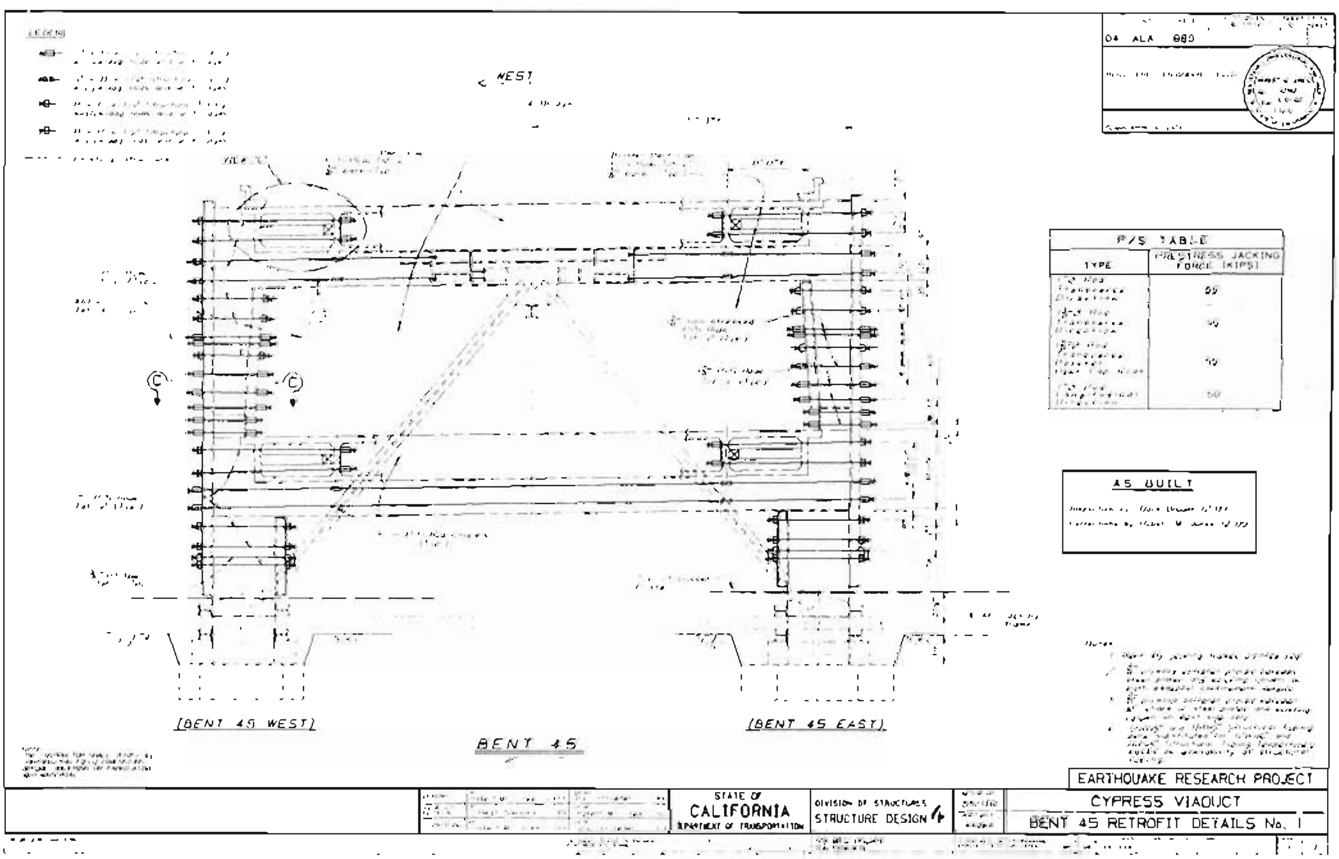


Figure 10-55. *Caltrans as-built drawing, Bent 46 retrofit details.*

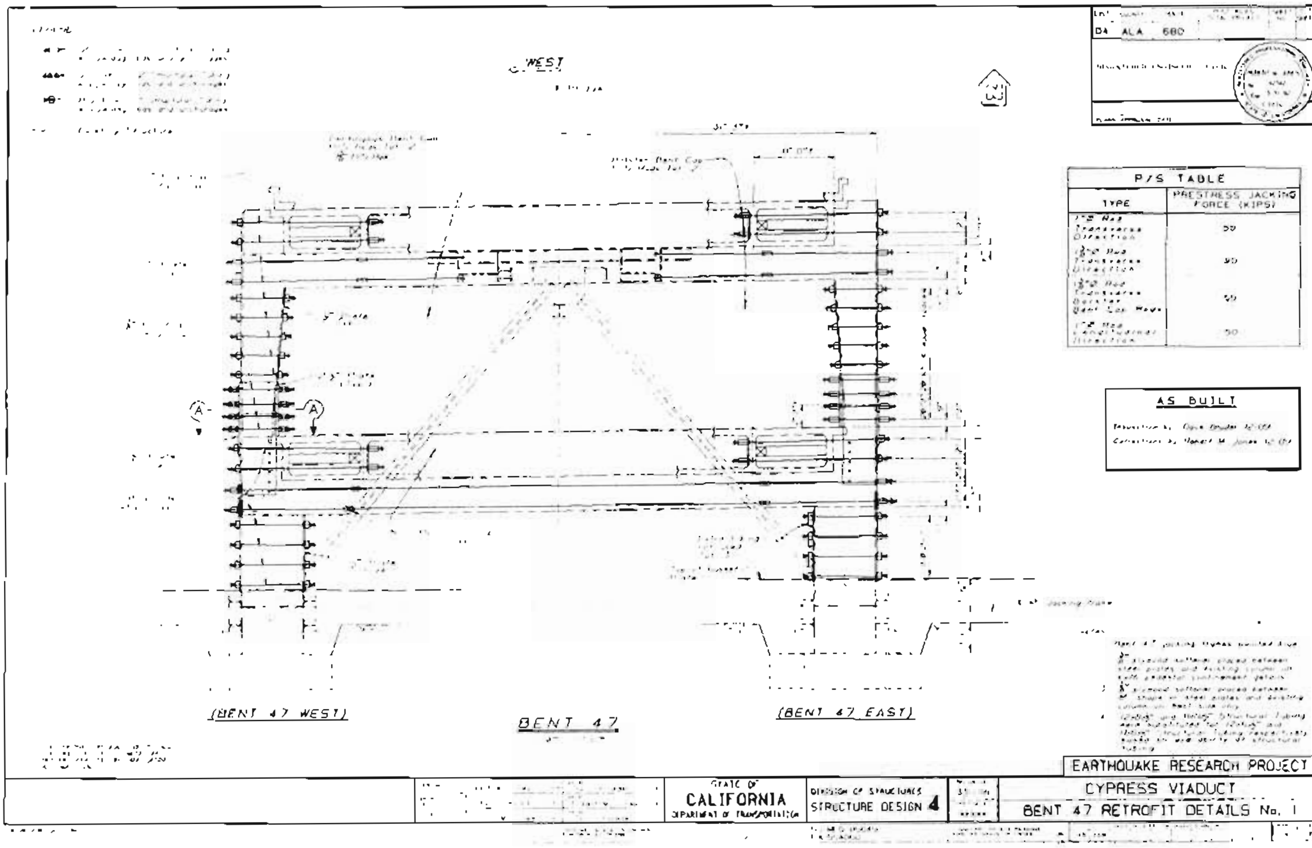


Figure 10-56. *Caltrans as-built drawing, Bent 47 retrofit details.*

Table 10-8. *Comparison of experimental and analytical test results.*

	1st Mode		2nd Mode	
	Frequency (Hz)	Period (sec.)	Frequency (Hz)	Period (sec.)
Experimental*	2.5	0.40	6.5	0.15
Analytical*	2.6	0.38	6.9	0.14

*As reported by Moehle, Dec. 14, 1989

The loading apparatus for the static load tests consisted of pairs of large steel "A" frames, which straddled each of the bents as shown in Figures 10-54, 10-55, and 10-56. Hydraulic jacks with a nominal capacity of 700 kips each were installed at the tops of each "A" frame. These jacks were arranged so that lateral loads in the transverse east or west directions could be applied at the top deck level, with maximum values of 1,400 kips per bent, totalling to 4,200 kips for the three-bent structure. Ninety channels of data were recorded, but only available and pertinent preliminary results will be discussed in this Report. A final technical report on this research will be published later in 1990.

Tests on Nonretrofitted Test Structure. Prior to being retrofitted, the 3-bent test structure was subjected to a series of forced vibration tests by means of centrifugal shakers mounted on each of the two top deck spans (Figure 10-53). Measured experimental frequencies and periods for the first two modes in the transverse direction are compared in Table 10-8 with calculated values by Moehle from Table 10-5 and indicate excellent agreement.

After the forced vibration tests were completed, steel collars were installed around each of the six pedestals just below the shear key hinge joints. These collars could be left unclamped to offer no restraint to pedestal cracking or clamped to restrain this cracking.

Table 10-9. *Static load results for two cases of pedestal clamping.*

	One Unclamped Pedestal	All Pedestals Clamped
Maximum load per bent (kips)	465	600
Top deck displacement (in.)	0.72	1.02

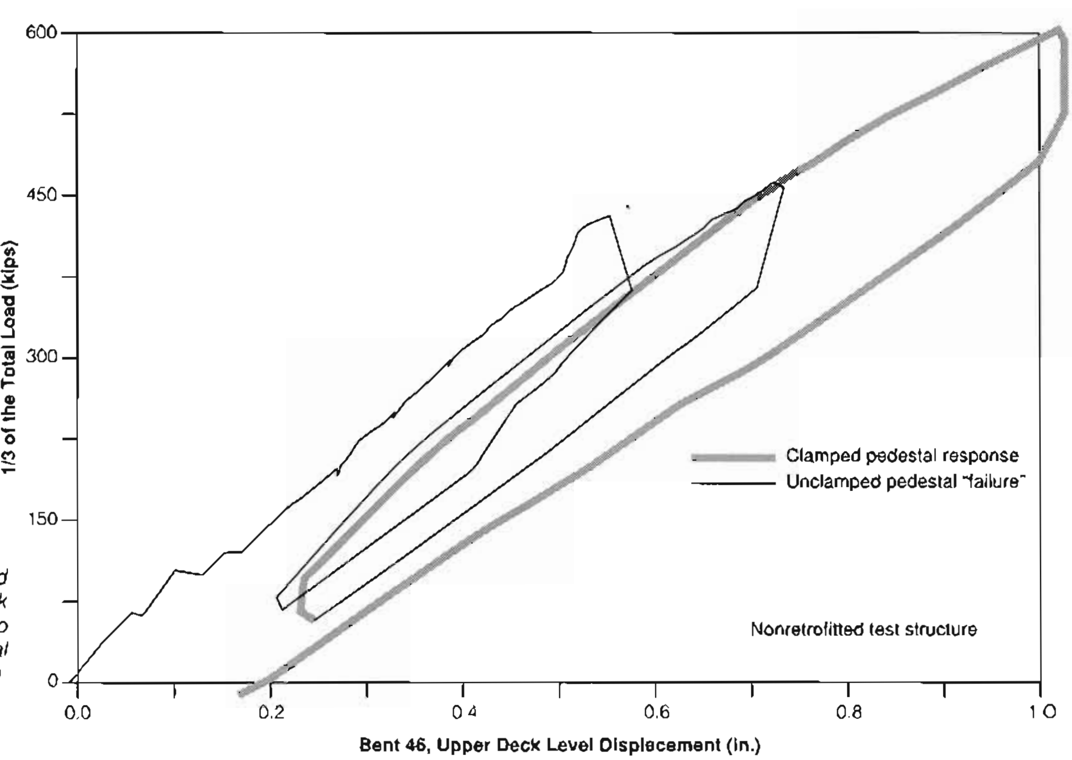


Figure 10-57. *Load versus upper deck displacement for two cases of pedestal clamping*

Shores were installed to support the decks in case of sudden failure under lateral loads. The structure was then subjected to static lateral loads in one direction only, first with only one (west column of Bent 46) of the six pedestals unclamped and second with all six pedestals clamped. All three bents were loaded equally. A plot of nominal load per bent vs. upper deck level displacement is given in Figure 10-57 for the two cases of clamping. Results of these tests are shown in Table 10-9.

For the one pedestal unclamped case, the loading was stopped at 465 kips when critical hairline cracks in the unclamped pedestal began to form. Partial unloading to about 75 kips left a displacement of 0.20". For the all pedestals clamped case, the loading was stopped after a significant overload to 600 kips. After a complete unloading, a residual displacement of 0.20" remained.

It is important to note here that for the one pedestal unclamped case—the unclamped pedestal corresponding to conditions at the time of the earthquake—the test "capacity" of 465 kips considerably exceeds the calculated ACI code value "capacities" of 254, 280, and 160 to 360 kips from static analyses discussed above, but it is still considerably less than the elastic top-story shear demands of 730 and 850 kips found from the two dynamic analyses.

Tests on Retrofitted Test Structure. Three different retrofitting schemes (Figures 10-54, 10-55, and 10-56) were installed on Bents 45, 46, and 47. These tests were conducted to determine the performance of the retrofits when subjected to increasing cyclic lateral loading at the top deck.

The cyclic loading history is shown in Figure 10-58. The initial load of 300 kips was selected to seat the loading apparatus. Cycles to 600 kips were below the elastic top-story seismic shear demands of 730 and

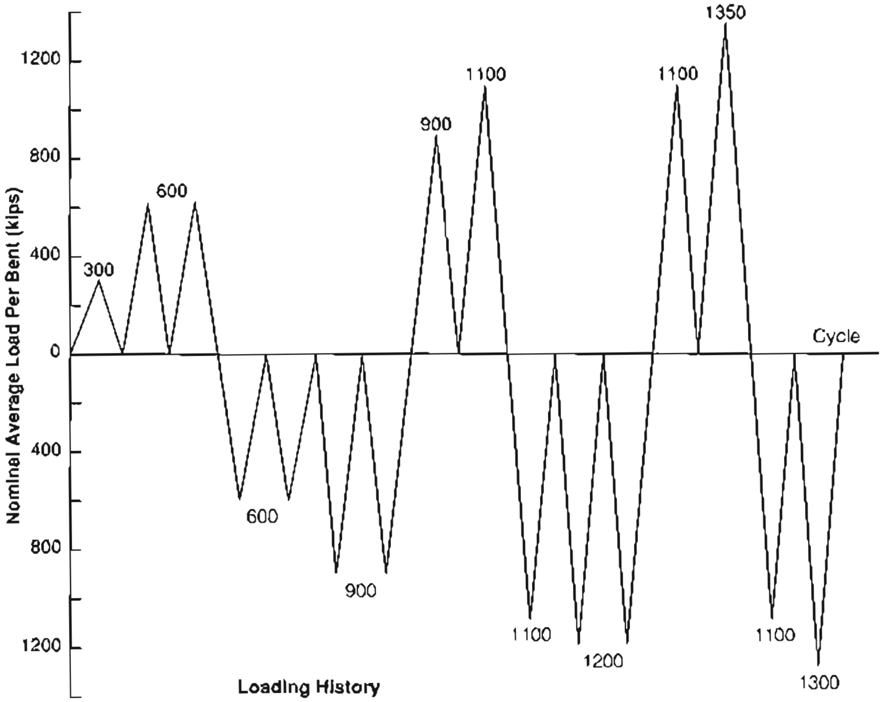


Figure 10-58. *Loading history of upper deck of test structure.*

850 kips predicted by the two dynamic analyses, but above the test load of 465 kips that caused cracking in the unclamped pedestal in the nonretrofitted test structure. The 900 kip load level was close to the top-story seismic shear demand of 850 kips predicted from the dynamic response spectrum analysis. Loads above this level of 900 kips were then applied cyclically to 1,100, 1,200, 1,100, 1,350, 1,100, and 1,300 kips.

Figure 10-58 shows the load-displacement hysteretic loops of the top deck for Bent 46, which indicates nearly linear behavior up until a bent load of 900 kips. Considerable cracking and some yielding had occurred prior to this load level. Under increasing load, the structure stiffness decreased and showed a pronounced drop at a bent load of about 1,100 kips and a maximum displacement of about 3". Figure 10-59 shows that up to this load level the hysteretic loops were stable and repeatable. On the last excursion to the west, the bent load reached 1300 kips at a displacement of 6". During a final excursion to the east, the structure reached its full yield capacity at a bent load of about 1,330 kips or a total three-bent structure load of 3(1,300) ~ 4,000 kips. Figure 10-59 shows that large inelastic deformations and a maximum displacement of nearly 10" took place, with more than 80% of this occurring in the top story.

At this maximum displacement, the

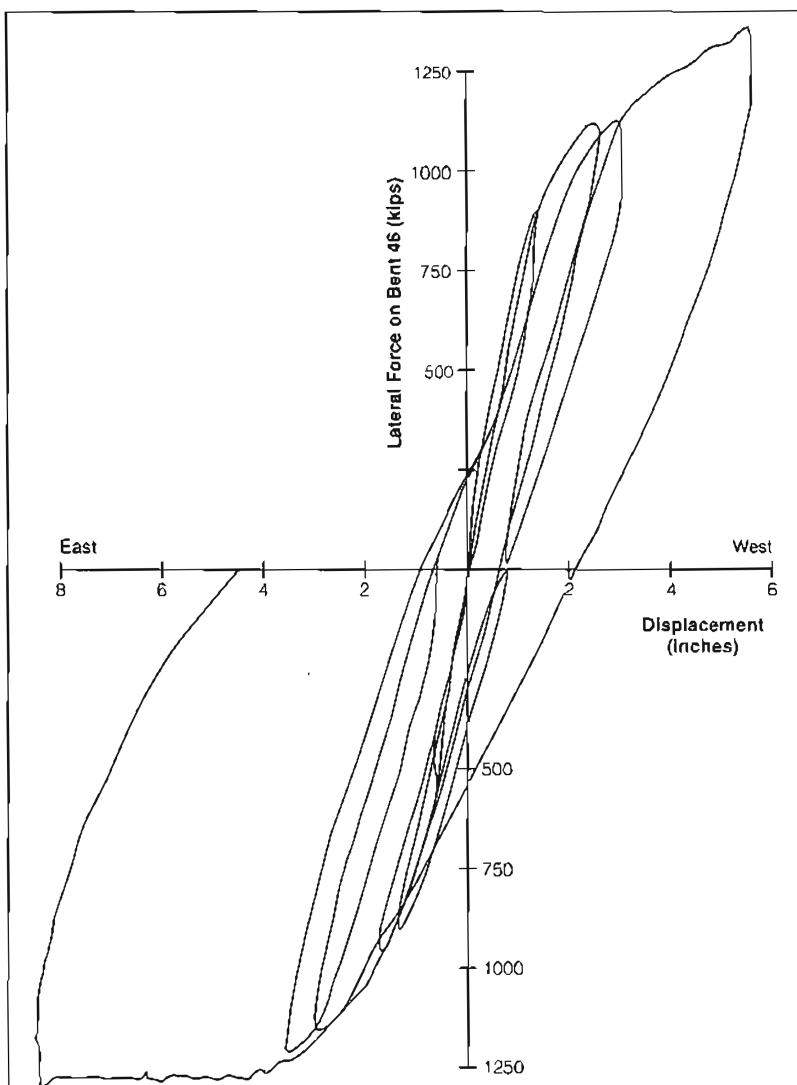


Figure 10-59. *Lateral load-displacement plot for Bent 46.*

structure continued to maintain its lateral and vertical load capacity. The structure sustained severe damage in the top girder-to-column joints in all frames and significant damage at the lower girder-to-column joints. Unloading left a residual displacement of about 5".

As a final note, it is of interest to compare the maximum bent load and displacement attained by the retrofitted structure of 1,330 kips and 10" in Figure 10-59 with that attained by the unretrofitted structure of 465 kips and 0.72" given in Table 10-9, both of which should be compared with the elastic top-story shear demands of 730 and 850 kips found from the two dynamic analyses.

Conclusions

1. Analysis and design of the Cypress Viaduct were performed between 1949 and 1954, when little design information was available on dynamic effects, realistic lateral forces and ductile design and ductile detailing of reinforced concrete structures to resist earthquake effects.
2. The Cypress Viaduct was designed and constructed to meet the required seismic design criteria and forces existing at the time. However, in terms of current seismic design criteria, based on research, development and experience since 1954, the design was deficient in several respects:
   a. The three dimensional structural system contained many hinges and joints to simplify its analyses and interpretation its of behavior, as well as to provide for movements resulting from creep, shrinkage, temperature and prestressing and future construction additions. Consequently, the structure lacked redundancy, which made it highly susceptible to damage or collapse in a strong earthquake.
   b. The structure lacked the ductility required in present designs and detailing of ductile reinforced concrete. By today's standards, the Cypress Viaduct had inadequate and incorrectly detailed transverse reinforcement in both the columns and the joint regions, poor reinforcement anchorage and splice detailing, and no confinement reinforcement in critical regions.
   c. The structure was brittle, nonductile, and lacked the energy-absorbing capacity required to resist strong cyclic earthquake motions.
3. Following the San Fernando earthquake of 1971 a decision was made to first utilize the limited funds available for retrofitting to install only longitudinal restrainers at the transverse expansion joints in bridge decks. This was done for the Cypress Viaduct in 1977, but unfortunately no detailed comprehensive analyses of the entire structure system were made to determine if other weaknesses existed. Such analyses, with methods available in 1977, would have predicted the failure of the Cypress Viaduct under a ground motion equiva-

lent to that experienced in the Loma Prieta earthquake of October 17, 1989 or greater.

4. Static and dynamic analyses performed since the earthquake by several investigators using analytical models consisting of a single joint, a single bent, or multiple bent segments indicate that the calculated seismic demands required to initiate failure from the earthquake to be greater than the available structural capacities. The predominant failure mechanism in most bents was the development of a critical diagonal tension crack in the lower girder to upper column pedestal or joint region produced by a horizontal shear force. The failure surface followed the plane defined by the bent down lower girder negative reinforcement in the joint region. Gravity and seismic forces then pushed the upper columns down and away from the joint, resulting in the collapse of the upper deck. Once the collapse of one or more bents was initiated, progressive collapse of other bents along the length of the viaduct probably ensued as demonstrated after the earthquake during the demolition of a four-span, five-bent segment.

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Chapter 11

San Francisco Freeway Viaducts

There are six Freeway Viaduct structures (elevated freeways) in San Francisco (Table 11-1)—Terminal Separation, Embarcadero, Central, Alemany, Southern Freeway, and China Basin Viaducts—all of which are comparable in design and construction to the Cypress Viaduct in Oakland. None of the six San Francisco Freeway Viaducts collapsed, but several were severely damaged. Damage in most cases involved spalling and diagonal cracking of the concrete in the columns and girder-to-column connections. Cracking patterns were similar to those exhibited in the standing bents of the Cypress Viaduct and were consistent with the failure planes in the collapsed portions of the Cypress.

Immediately after the earthquake, Caltrans assessed the condition of the viaducts, closed damaged structures, and shored damaged sections of the Terminal Separation, Embarcadero, Central, Southern, and China Basin Viaducts. The Alemany Viaduct, which was undamaged, has remained open. Figure 11-1 shows the location of all six San Francisco viaducts.

Table 11-1. *San Francisco Freeway Viaducts.*

Freeway	Highway Designation	Date of Original Design
Double-Deck		
Terminal Separation	I-480	1954
Embarcadero	I-480	1956, 1962
Central Viaduct	US-101	1957
Albany Viaduct	US-101	1958
Southern Freeway Viaduct	I-280	1962
Single-Deck		
China Basin Viaduct	I-280	1965, 1969

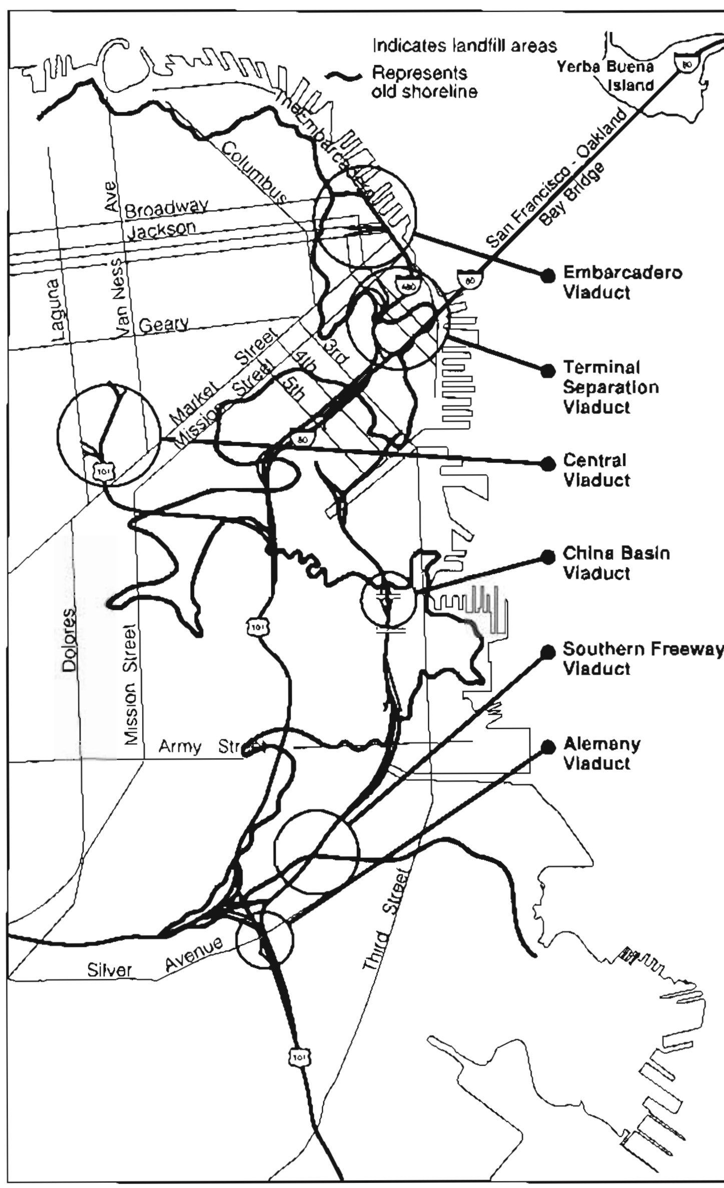


Figure 11-1. *Locations of the six San Francisco Freeway Viaducts.*

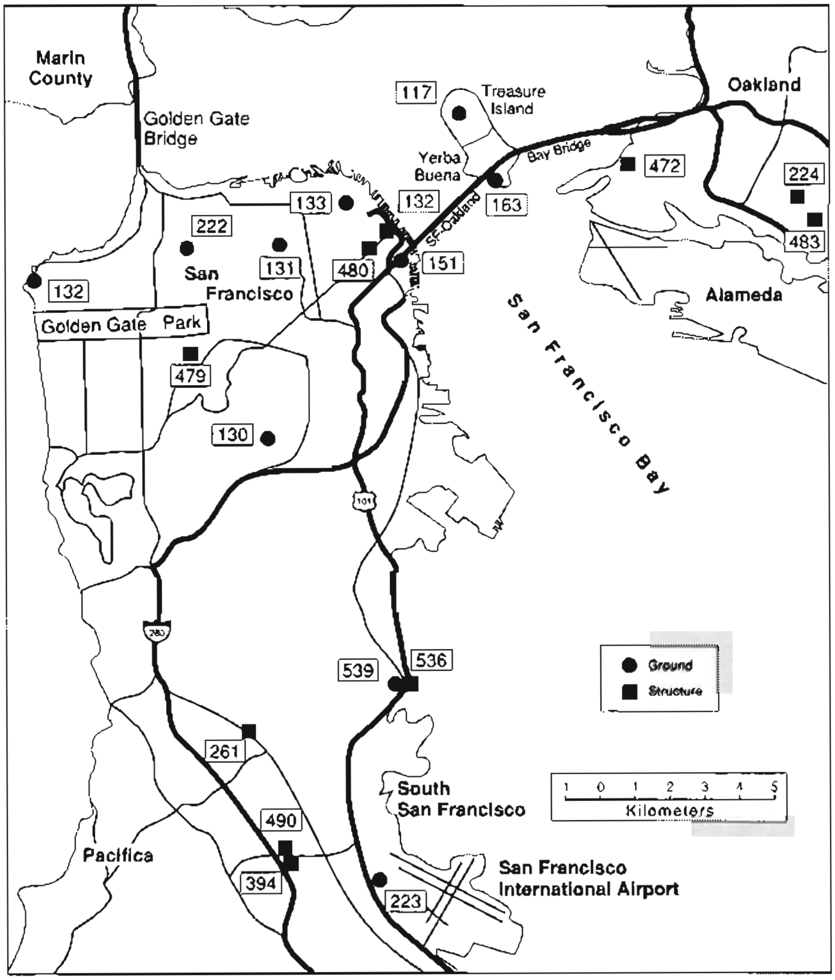


Figure 11-2. *Locations of Strong Motion Instrumentation Program recording stations*

117 *Treasure Island*
130 *SF—Diamond Heights*
131 *SF—Pacific Heights*
132 *SF—Cliff House*
133 *SF—Telegraph Hill*
151 *SF—Rincon Hill*
163 *Yerba Buena Island*
222 *SF—Presidio*
223 *SF International Airport*
224 *Oakland—2-story bldg.*
261 *So. SF—4-story hospital*
394 *San Bruno—9-story gov. bldg.*
472 *Oakland—24-story residential bldg.*
479 *SF—6-story UCSF bldg.*
480 *SF—18-story commercial bldg.*
483 *Oakland—24-story residential bldg.*
490 *San Bruno—6-story office bldg.*
532 *SF—47-story office bldg.*
536 *So. SF—Sierra Pt. overpass*
539 *So. SF—Sierra Pt*

The epicenter of the Magnitude 7.1 Loma Prieta earthquake was approximately 60 miles south of San Francisco. The duration of the strong motion was generally less than 10 seconds. Five California Strong Motion Instrumentation Program (CSMIP) stations (Figure 11-2), located between one and five miles of the six San Francisco Freeway Viaducts, recorded strong motion from the Loma Prieta earthquake. A summary of the strong motion records collected at these five stations is presented in Table 11-2. Levels of peak horizontal ground acceleration were greater at the fill/bay mud sites (0.15g average) than at the rock sites (0.08g average) [Shakal et al., 1989]. In addition, the duration of the strong motion shaking was significantly greater at the fill/bay mud sites (5 to 10 seconds) than at the rock sites (2 to 5 seconds). The relationship between the freeway locations and the bay mud fill is shown in Figure 11-1.

From an engineering point of view, it is clear that in terms of acceleration intensity, at both rock and bay mud sites, the Loma Prieta was a minor-to-moderate earthquake, and that if the buildings and bridges/freeways in San Francisco had been subjected to a moderate-to-severe earthquake, with a strong motion duration between 10 and 20 seconds, damage would have been much greater and more widespread.

Table 11-2. *Summary of strong motion records near the San Francisco Freeway Viaducts*

Location, CSMIP Station	Epicentral Distance (km)	Site Geology	Record Component	Ground Acceleration (g)	Strong Motion Duration (sec)
Rincon Hill #58151 (Fremont & Harrison)	95	Sandstone/ shale	90x H 360x H V	0.09 0.08 0.03	< 5 sec.
#58480	95	Fill over bay mud	350x H V	0.17 0.04	<10 sec.
Embarcadero #58532	96	Fill over bay mud	0x H V	0.13 0.08	<10 sec.
Telegraph Hill #58133	97	Sandstone/	90x H 360 H V	0.08 0.06 0.03	< 5 sec.
Pacific Heights	97	Sandstone/ shale	270 H 360x H	0.06 0.05 0.03	< 5 sec.

Table 11-3. *Description of San Francisco Freeway Viaducts.*

Freeway	Approximate Length of Structure [1]	Number Of Bents	Type Of Bents	Deck Span	Approximate Height	Foundation
Terminal Separation	N/A	127	Double-deck	60'-120'	Varies	Piles
Embarcadero	5200'	66	Double-deck	75'-100'	65'	Piles
Central Viaduct	5400'	60	Double-deck	80'-100'	60'	Piles
Alemany Viaduct	1500'	17	Double-deck	72'-100	50'	Piles
Southern Freeway Viaduct	5700'	63	Double-deck	95'	50'	Piles
China Basin Viaduct	600' [2]	14	Single-deck	65'-110'	0-60'	Piles

[1]Defined as the length of roadway supported by individual bents
[2]Length of the distribution structure

Description of the Six San Francisco Freeway Viaducts

All of the San Francisco Freeway Viaducts are double-deck structures, with the exception of the China Basin Viaduct, which is a single-deck structure. All six viaducts are of reinforced concrete, are constructed with multiple column bents, and are typically founded on piles. Some of the viaducts incorporate post-tensioned transverse bent girders at the upper deck level. The design and details for these six structures are similar to one another [State of Calif., Dept. of Trans., various dates], and are also similar to those of the collapsed Cypress Viaduct. Table 11-3 provides a summary of the physical characteristics of each of these structures. Note that all are founded on piles.

In the 1970s, Caltrans retrofitted all six San Francisco Freeway Viaducts with joint restrainer cables as part of the post-San Fernando earthquake expansion joint retrofit. In 1984, Caltrans added externally-mounted post-tensioned rods to the exterior face of the upper level column-to-girder joint in a number of the San Francisco Freeway Viaducts in an attempt to increase the moment capacity of the joint. Following a peer review and further analysis, Caltrans judged this latter partial-upgrading solution to be ineffective.

The Terminal Separation Viaduct links I-80 with I-480 (Embarcadero Viaduct) and links Beale, Mission and Main Streets in downtown San Francisco (Figure 11-3). The structure is composed of single-column and multi-column bents with a maximum of

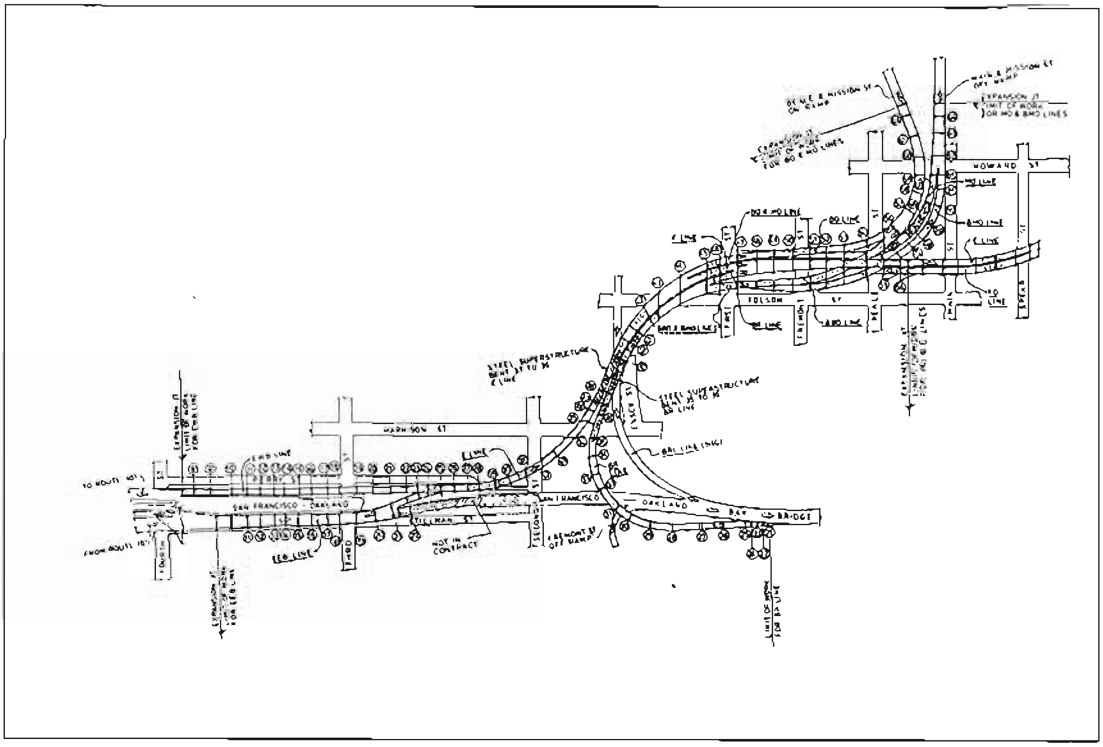


Figure 11-3. *Location of Terminal Separation Viaduct.*

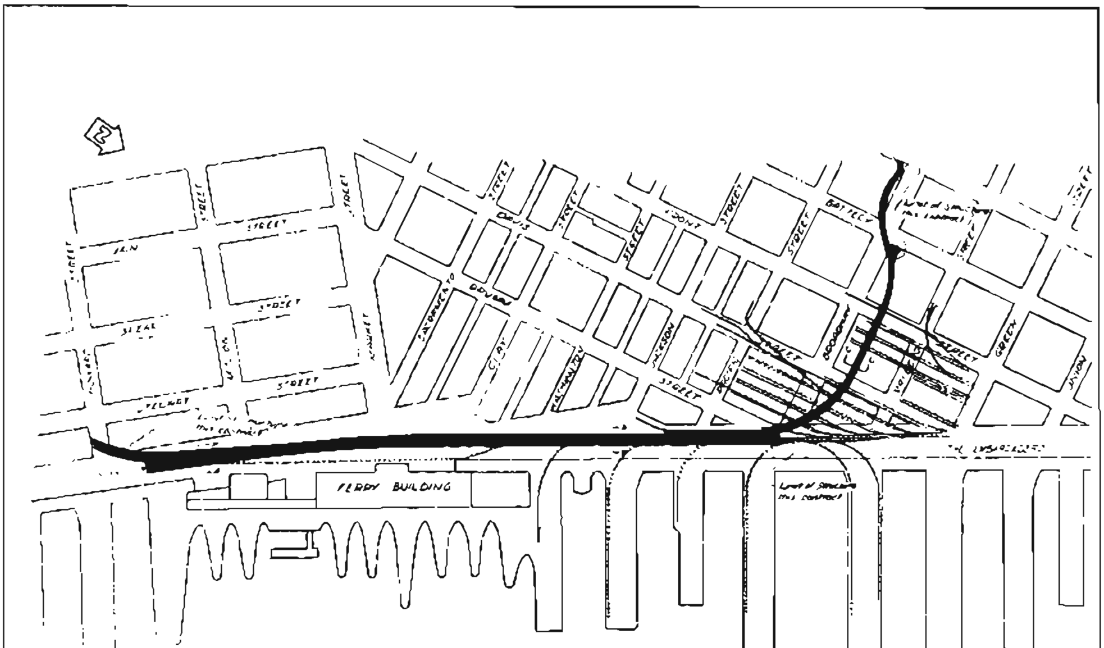


Figure 11-4. *Location of Embarcadero Viaduct.*

three levels of framing. The Embarcadero Viaduct (I-480) carries traffic from I-80 as far north as Broadway and as far west as Sansome Street in San Francisco (Figure 11-4). This viaduct is a multi-column, double-deck structure. The Central Viaduct provides a series of on-ramps and off-ramps for Highway 101 (Figure 11-5). The structure is composed of single-column and multi-column bents with a maximum of two tiers (double-deck) of framing. The Alemany Viaduct is the interchange between Highways 101 and 280 (Figure 11-6) and is a multi-column, double-deck structure. The

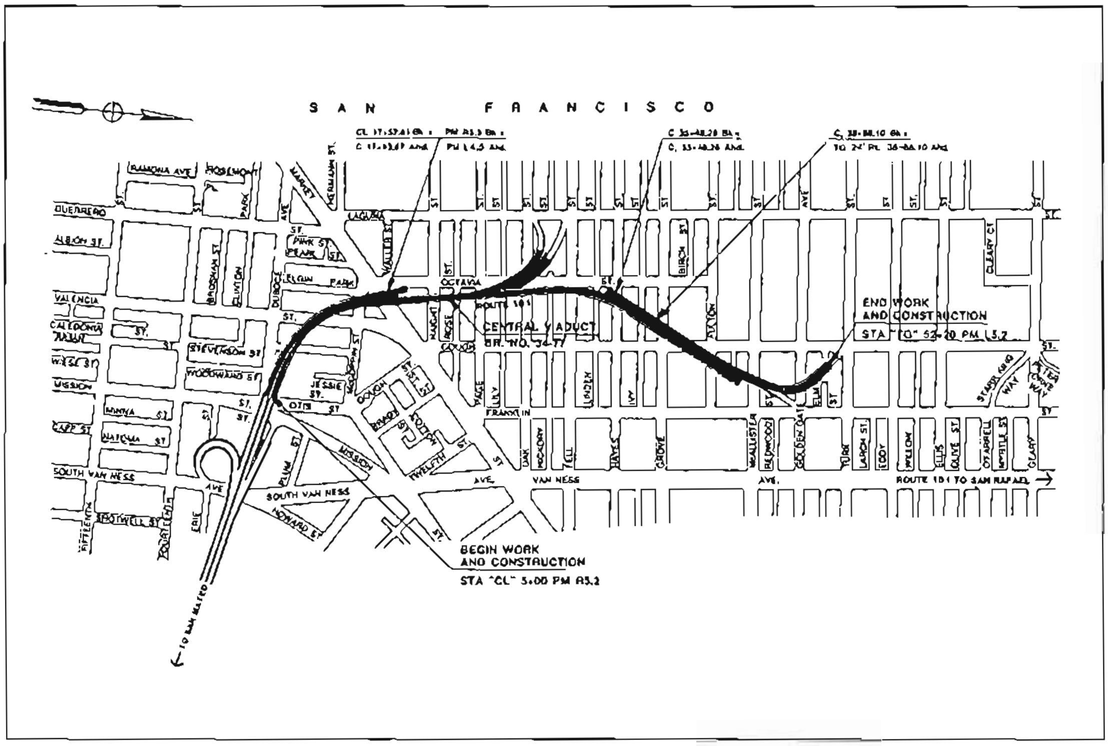


Figure 11-5. *Location of Central Viaduct.*

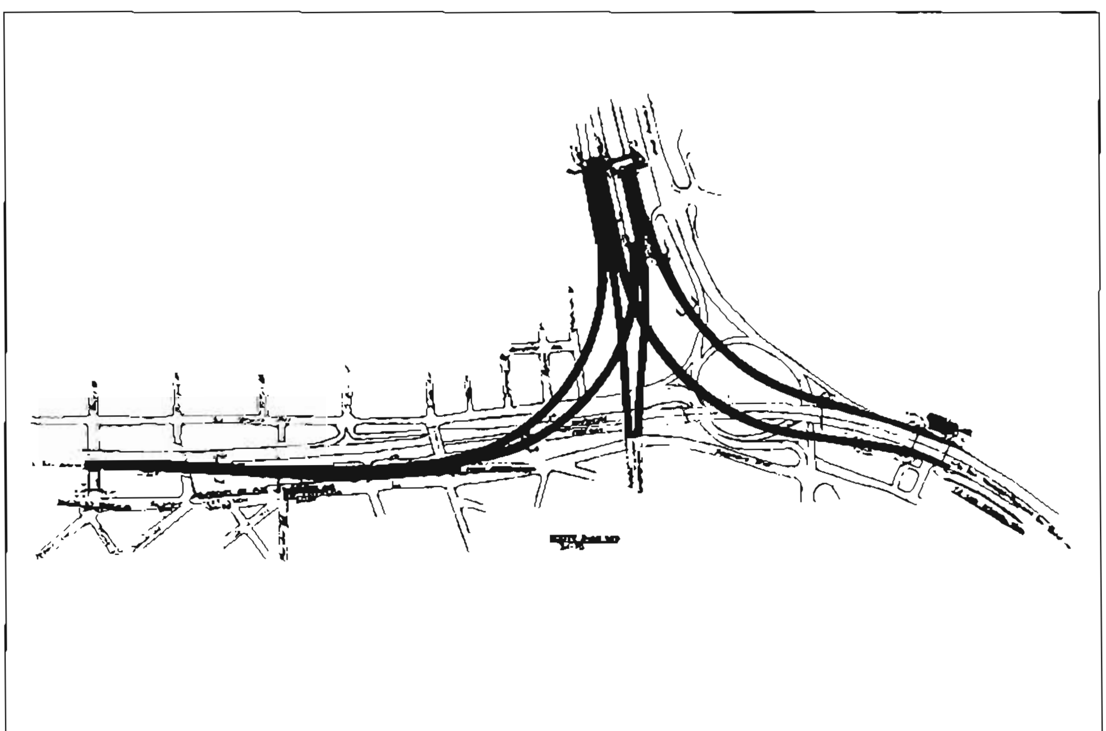


Figure 11-6. *Location of Alemany Viaduct.*

Southern Freeway Viaduct on Highway 280 (Figure 11-7) is composed of single-column and multi-column bents with a maximum of two levels (double-deck) of framing. The China Basin Viaduct on Highway 280 (Figure 11-8) is composed of multi-column bents of varying heights that support one and two levels of framing.

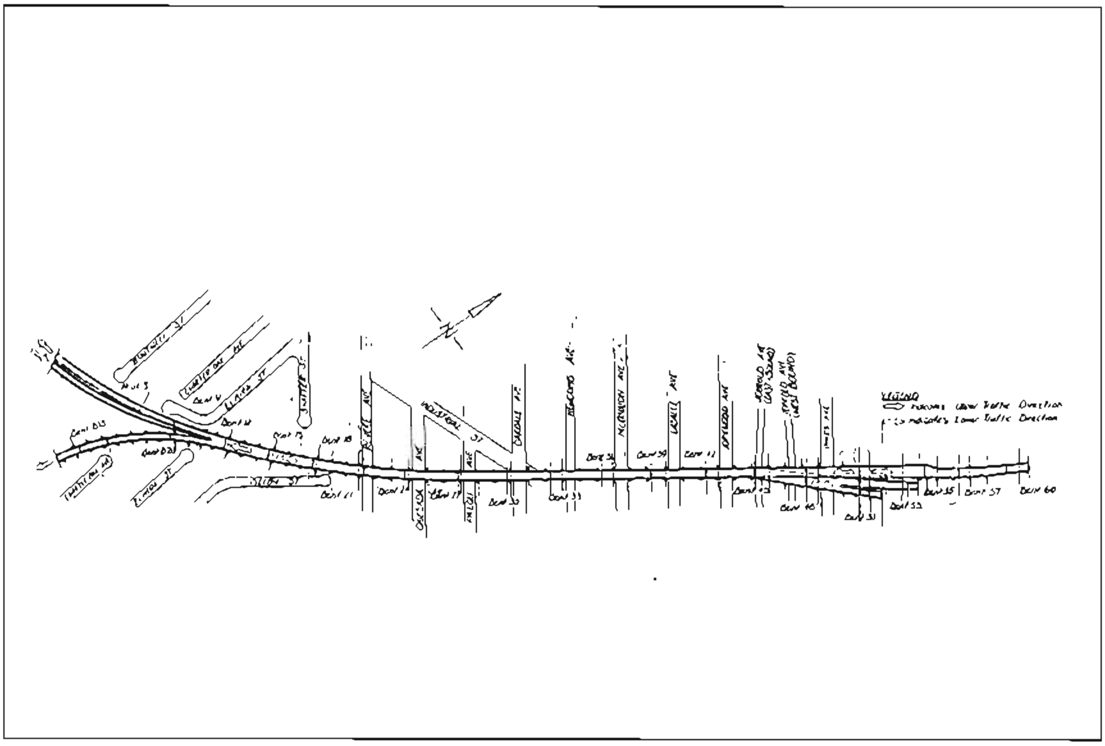


Figure 11-7. *Location of Southern Freeway Viaduct.*

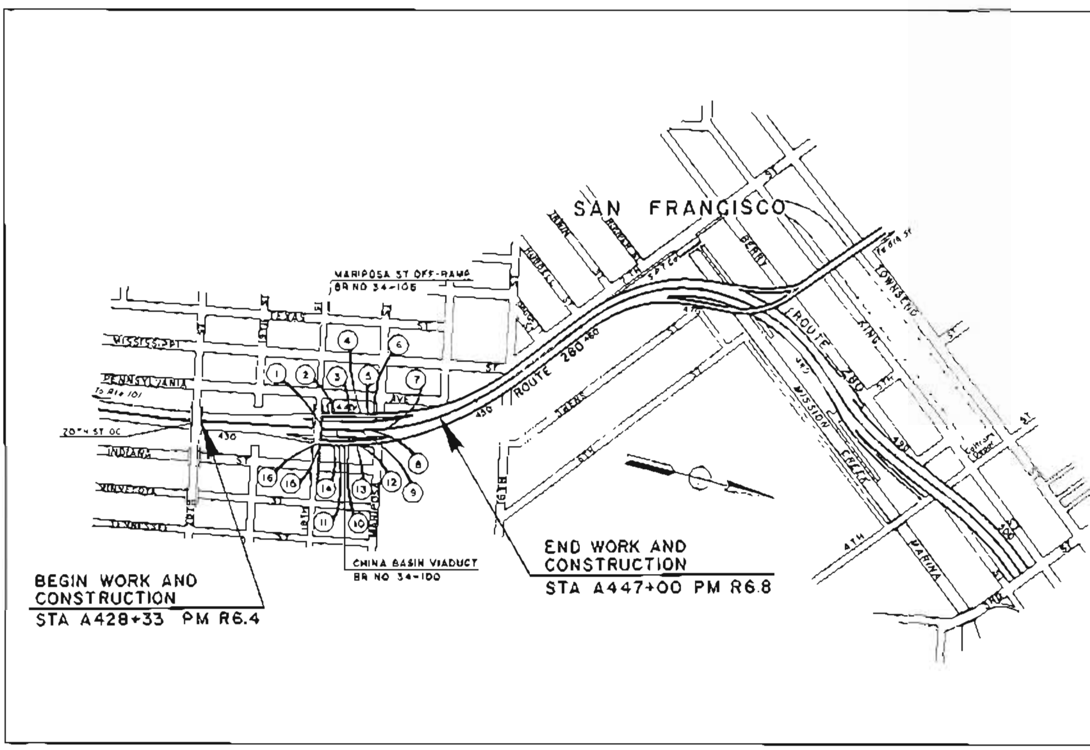


Figure 11-8. *Location of China Basin Viaduct*

Lateral Load Resisting Systems

The San Francisco Freeway Viaducts utilize moment resisting frames for seismic resistance in both the transverse and longitudinal directions. Transverse load resistance (perpendicular to the roadway) is provided by frame action of transverse bents. Longitudinal resistance (parallel to the roadway) is provided by frame action between the deck structure and the girders and columns of the transverse bents. The two-level (double-deck) bents have varying configurations selected to satisfy both the geometry of the roadway and the constraints on support locations. All of the supporting bents are of concrete construction, reinforced in accordance with the conventions of the 1950s. During the design of these viaducts, no consideration was given to ductile detailing.

The structures are separated with expansion joints along the road length at typical intervals of two or three bents to accommodate longitudinal thermal movements. The joints have been retrofitted with seismic restraining cables as part of the Caltrans cable restrainer program. These expansion joints, together with varying roadway widths, geometry, and tributary areas produce a complex structural system, particularly with regard to the seismic loading of individual bents. Complicating the behavior further is the variable bent stiffness resulting from variations in bent span and spacing, and variations in column and girder size. Any evaluation of these complex systems must rely on the Caltrans as-built drawings, since the original design calculations are no longer available.

Transverse Bents

These bents were primarily designed to satisfy vertical load requirements and were provided with nominal lateral load capacity. The gravity loads and spans dictated both girder and column sizes, while the post-tensioning of the top girders frequently dictated the locations of hinges, or rotation joints, within the various bents. The hinges reduced the seismic resistance of the bents by concentrating the seismic resistance into a few locations that would then experience large ductility demands during earthquake shaking.

The bents can be categorized by column hinge locations, resulting in four basic types:

Type I — A two-level portal frame with hinges at the base of each column at each level. These hinge locations create a stacked frame configuration.

Type II — A lower level portal frame with hinges at the column bases and with a single-column cantilevering vertically to the upper deck. The cantilevered column is hinged at the underside of the upper deck and the other column in level 2 is hinged top and bottom. This configuration places all of the seismic demand on the cantilevered column in the upper level. There is no redundancy in this frame type.

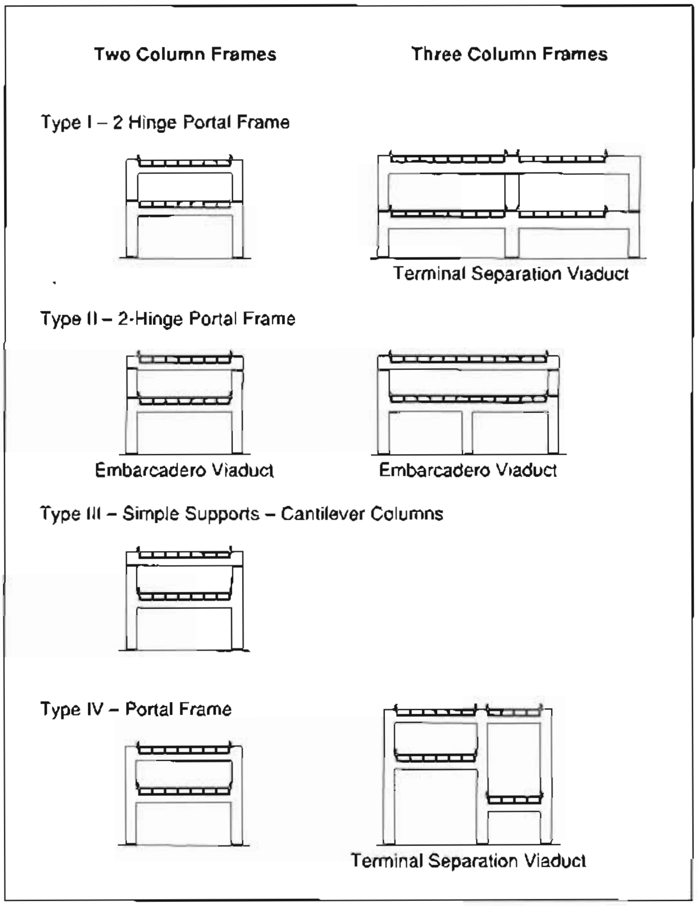


Figure 11-9. *Multiple column bent types.*

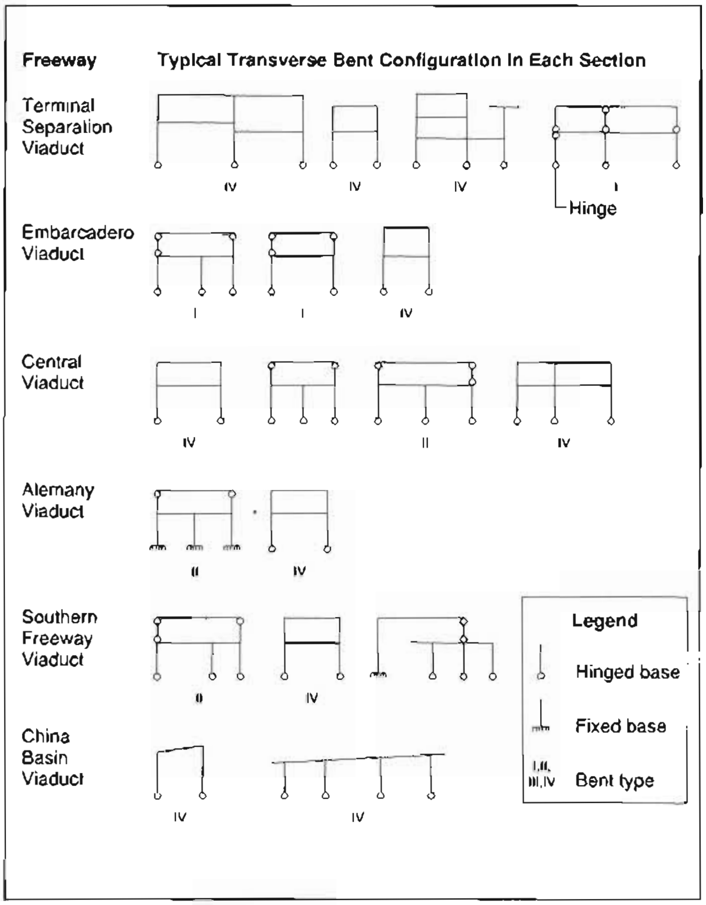


Figure 11-10. *Configuration of two-level transverse bents.*

Type III A lower level portal frame with hinges at the column bases. Two cantilever columns extend to the upper deck with hinges under the upper deck.

Type IV A two-level portal frame with hinges only at the base of the lower columns. All other joints resist moment.

Both two-column and three-column bents can be classified as Type I to IV (Figure 11-9). The hinges, especially in Type II, create an unbalanced frame response and force the single columns to carry all the seismic loads.

The six San Francisco Freeway Viaducts each contain several bent types. The predominant types are summarized in Figure 11-10. The Terminal Separation Viaduct is unique because of the roadway geometry and the heights of the bents. The Embarcadero, Central and Alemany Viaducts all have bents similar to one another, and the Southern Freeway's bents are in part similar. The China Basin Viaduct Interchange is a special two-level crossover and its frames are nontypical.

Bent Details

Reinforcing details for the transverse bents in the six San Francisco Freeway Viaducts are basically the same, and are similar to the Cypress Viaduct in Oakland [State of Calif., Dept. of Trans., various dates]. Representative bents from the Terminal Separation Viaduct (Figure 11-11), the Embarcadero Viaduct (Figure 11-12) and the Central Viaduct (Figure 11-13) illustrate the similarity of reinforcement.

Typically the bent girders were designed for heavy vertical loads with reinforcing that is sized and positioned for these loads. Only nominal top and bottom girder reinforcing is provided for lateral load induced moments with few top bars extending into and anchored at the knee joints, and with few bottom bars also extending into the knee joints. This pattern of reinforcing was common through the 1950s and early 1960s.

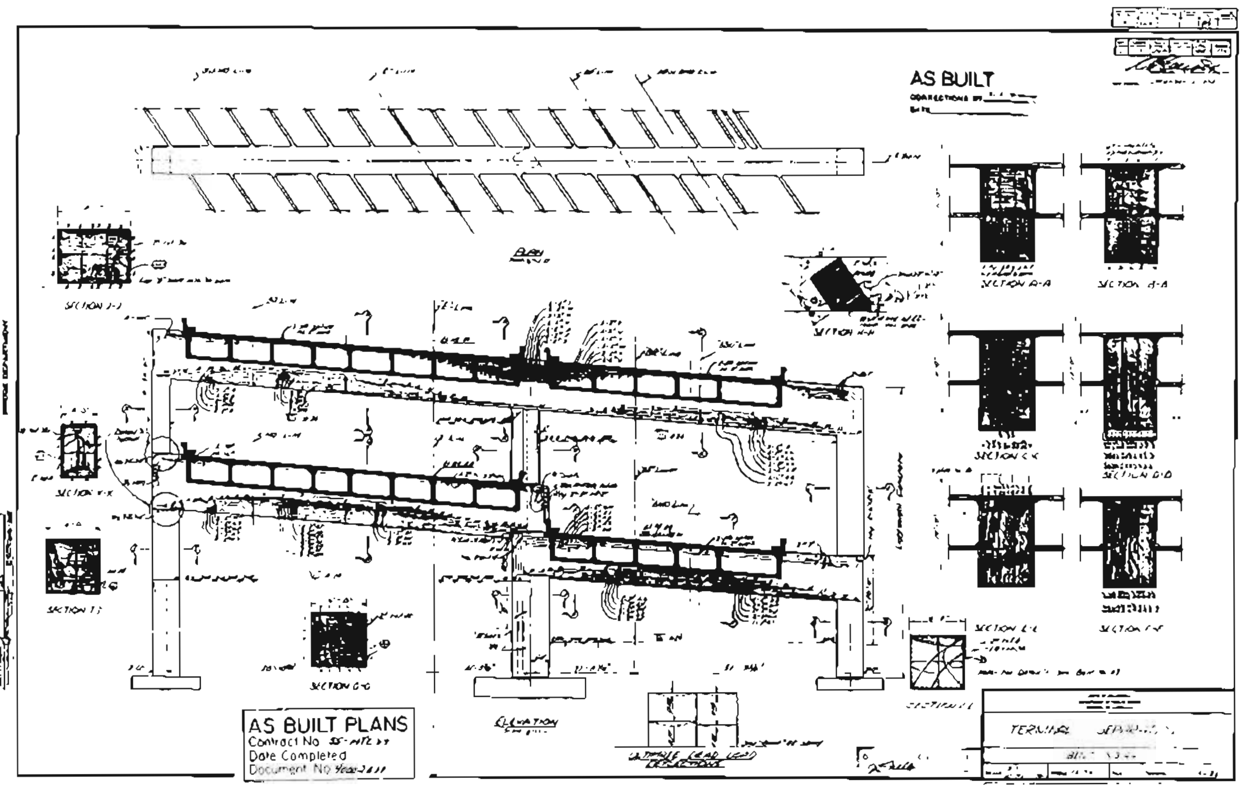


Figure 11-11. *Terminal Separation Viaduct, transverse Bent 44.*

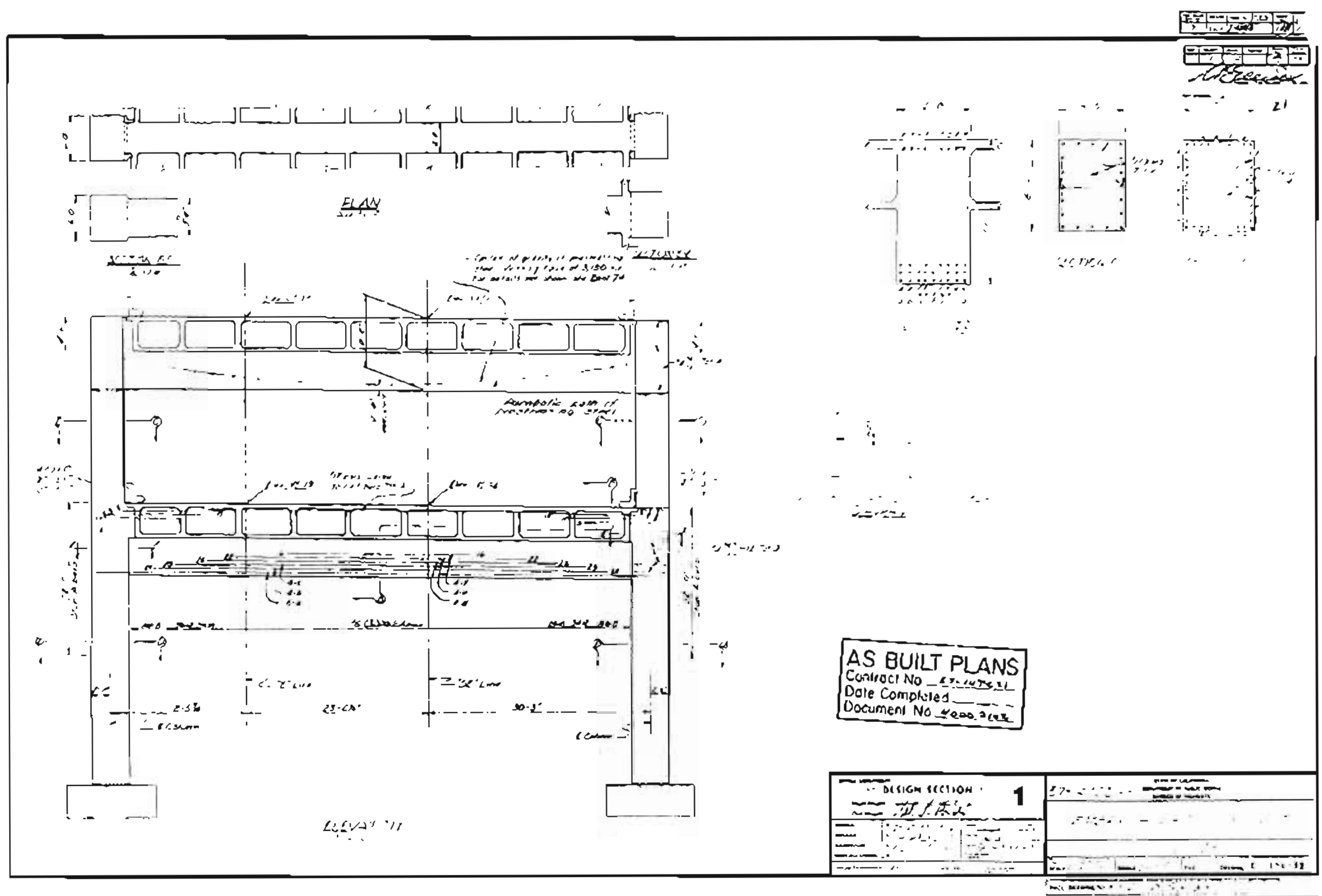


Figure 11-12. *Embarcadero Viaduct, transverse Bent 78.*

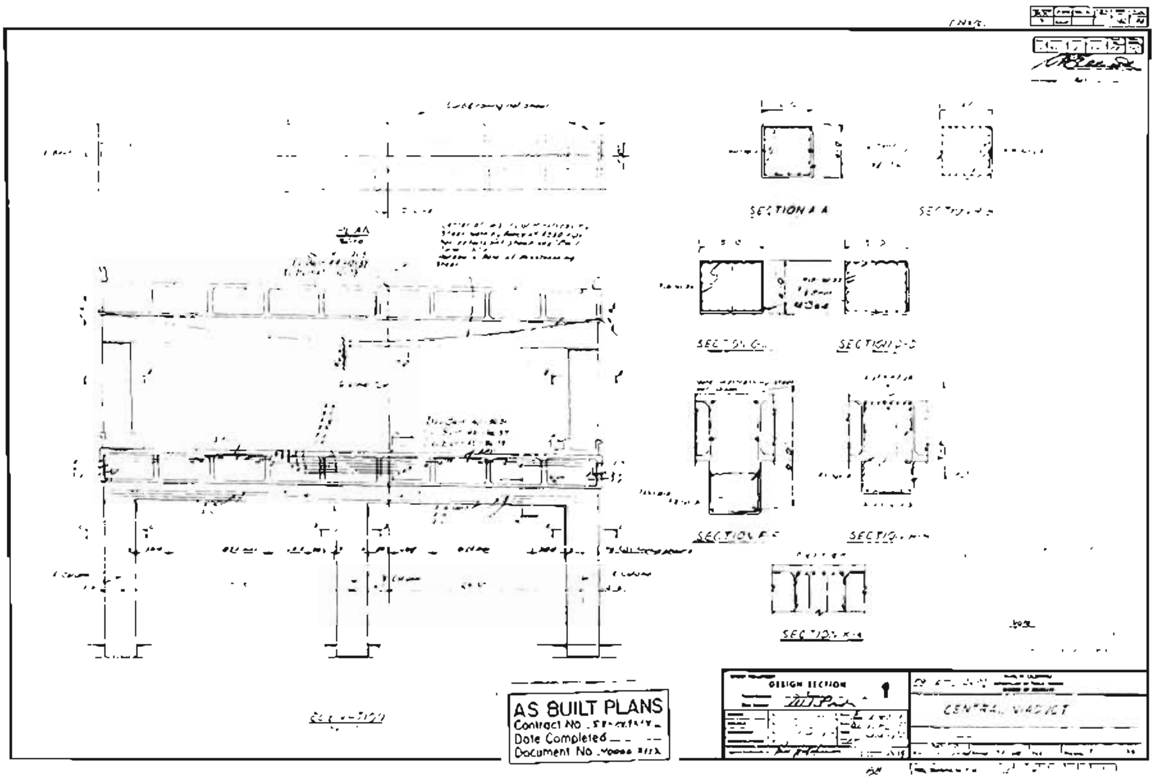


Figure 11-13. *Central Viaduct, transverse Bents 43-45.*

The seismic regulations in the 1950s and 1960s permited a 33% increase in the allowable stresses under seismic loads. Because of this increase, the nominal reinforcement provided for the vertical load was generally sufficient for seismic loading resulting in no special or additional reinforcement. Consequently, the bents possess a much reduced seismic capacity with respect to current standards.

Special confining reinforcement for columns, joints and girders, important for ductility, was not used in any of the San Francisco Freeway Viaducts. The details of frame knee joints are virtually identical in the six viaducts, indicating no change in detailing philosophy during the design period of these viaducts from 1954 to 1965. Figures 11-14, 11-15 and 11-16 for the Terminal Separation Viaduct, Embarcadero Viaduct, and Central Viaduct, respectively, show little joint confinement at both top- and mid-level frame joints. Column hoop ties provided were also minimal, and inadequate by standards for ductile concrete.

The lack of shear reinforcement in the columns, joints, and girders resulted in the concrete members of the viaducts being weak in shear and torsion. The poor anchorage of the girder reinforcement in the columns produced sections that are weak in flexure and knee joints that have low shear capacity. In general, the bents form a weak-column strong-girder system, just the opposite of the current philosophy in design of buildings that prefers a strong-column weak-girder system. The columns and joints in the viaduct frames will generally fail first in a nonductile manner during an earthquake, prior to girder yielding, which increases the likelihood of collapse.

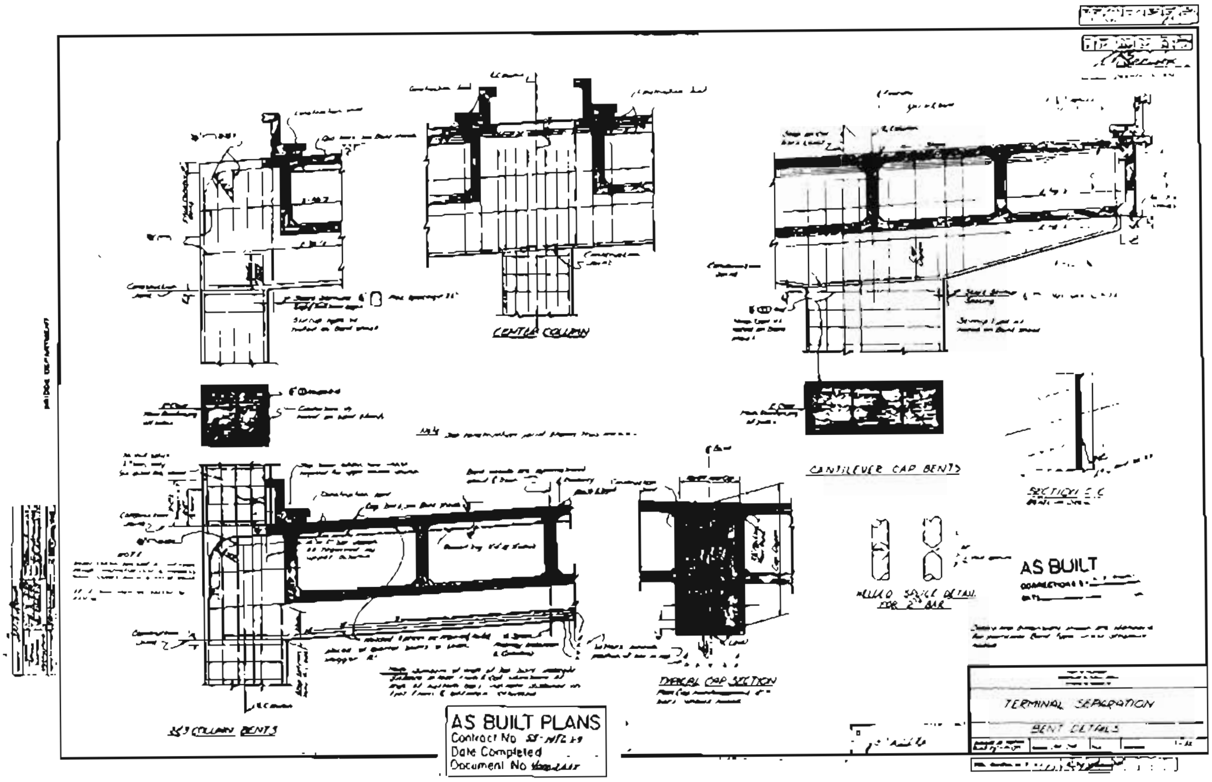


Figure 11-14. *Terminal Separation Viaduct, transverse bent details.*

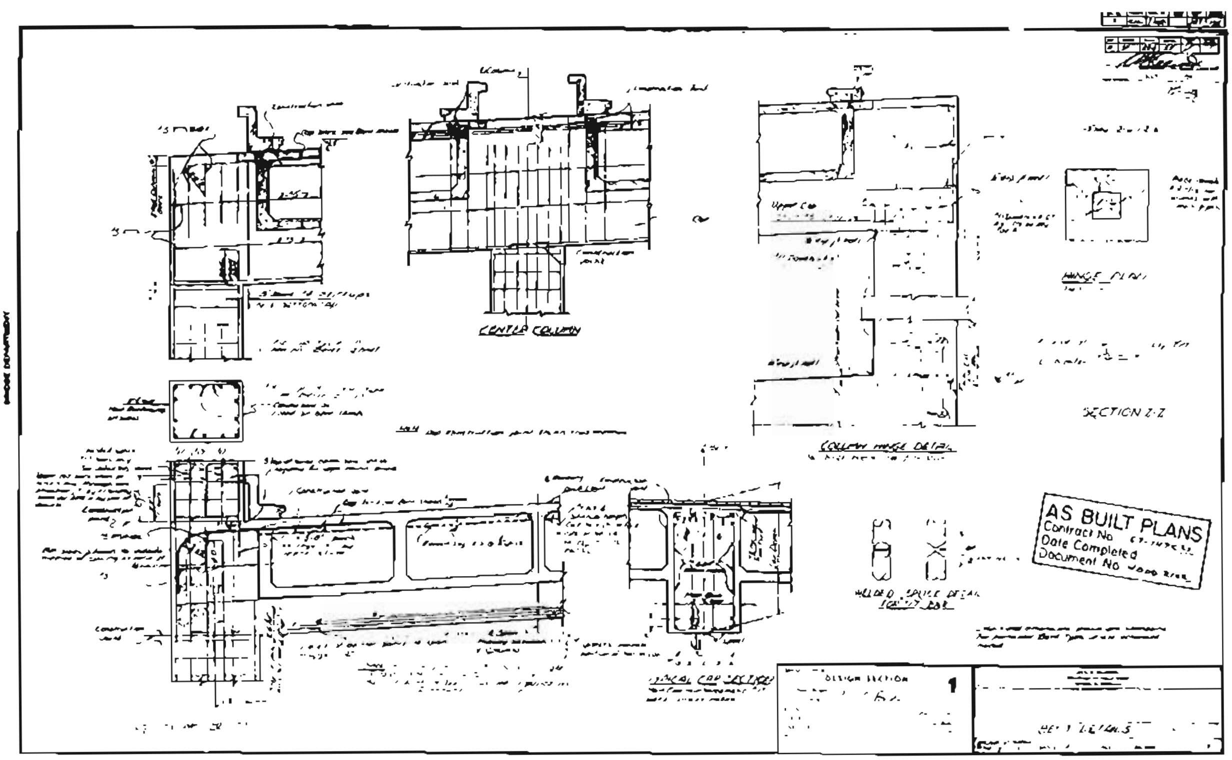


Figure 11-15. *Embarcadero Viaduct, transverse bent details.*

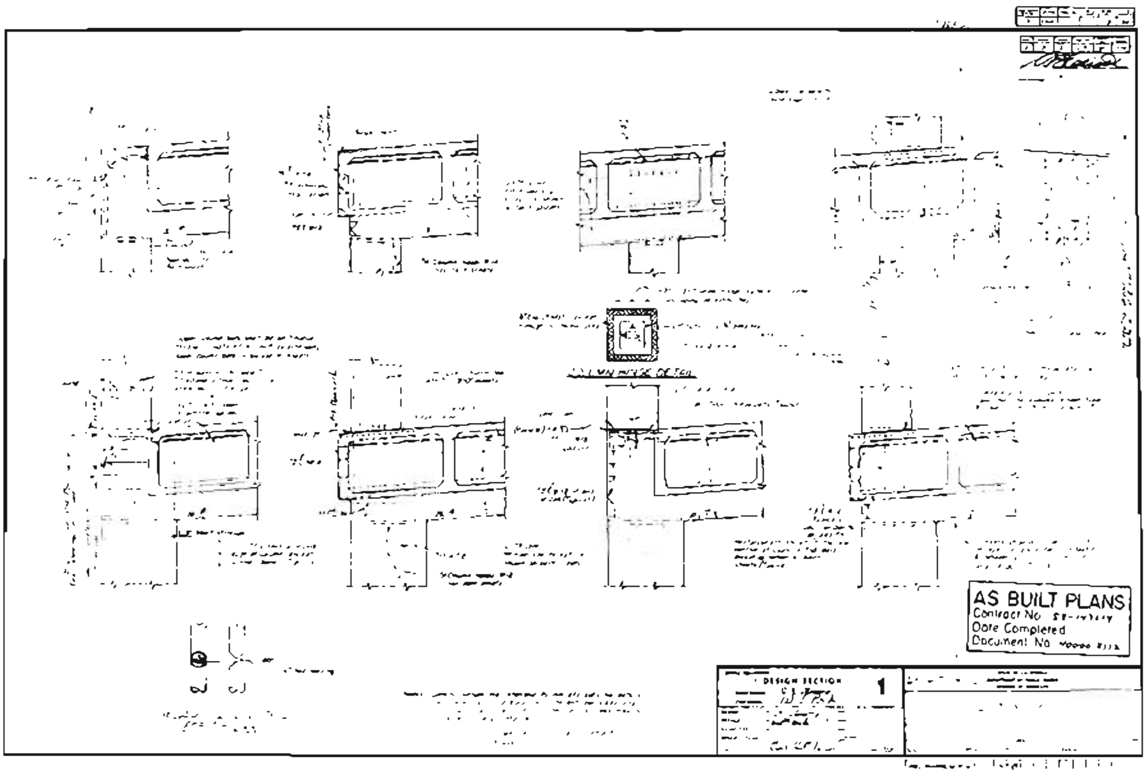


Figure 11-16. *Central Viaduct, transverse bent details.*

Longitudinal Frames

Seismic forces in the longitudinal direction of the roadway are resisted by a moment frame created by the roadway box girder in flexure, the transverse girders in torsion, and the columns in flexure. These frames do not appear to have been designed for seismic loads, but were simply designed as a normal box girder deck frame with acknowledgment of the added capacity attributed to the 1/3 stress increase allowed for seismic loading. No additional strength for seismic loads was provided in the longitudinal direction. Typically, the edge of the box girder deck section terminates on the inboard side of the bent columns to accommodate roadway clearance. Under seismic loading in the longitudinal direction, this frame configuration induces torsion in the column-to-girder knee joint, the outrigger girders, and the columns, however no special torsion reinforcement was provided to resist torsional moments.

In the longitudinal direction, bent types I, II, III and IV, with their various locations of column hinges, form frames of somewhat unconventional configuration. Figure 11-17 illustrates four different frame geometries resulting from the hinge locations. Frames of bent type II, with hinges at the top and bottom of the upper level columns, produce a frame with unbalanced moment resisting capacity. Tributary longitudinal loads to this unstable frame are transferred to the frame on the other side of the roadway through in-plane action of the deck, which relies on the cable restrainers to prevent large in-plane twisting of the deck. The load transfer increases the ductility demand on the cantilever columns. Longitudinal frames for other bent types are stable, but their strength and ductility are insufficient.

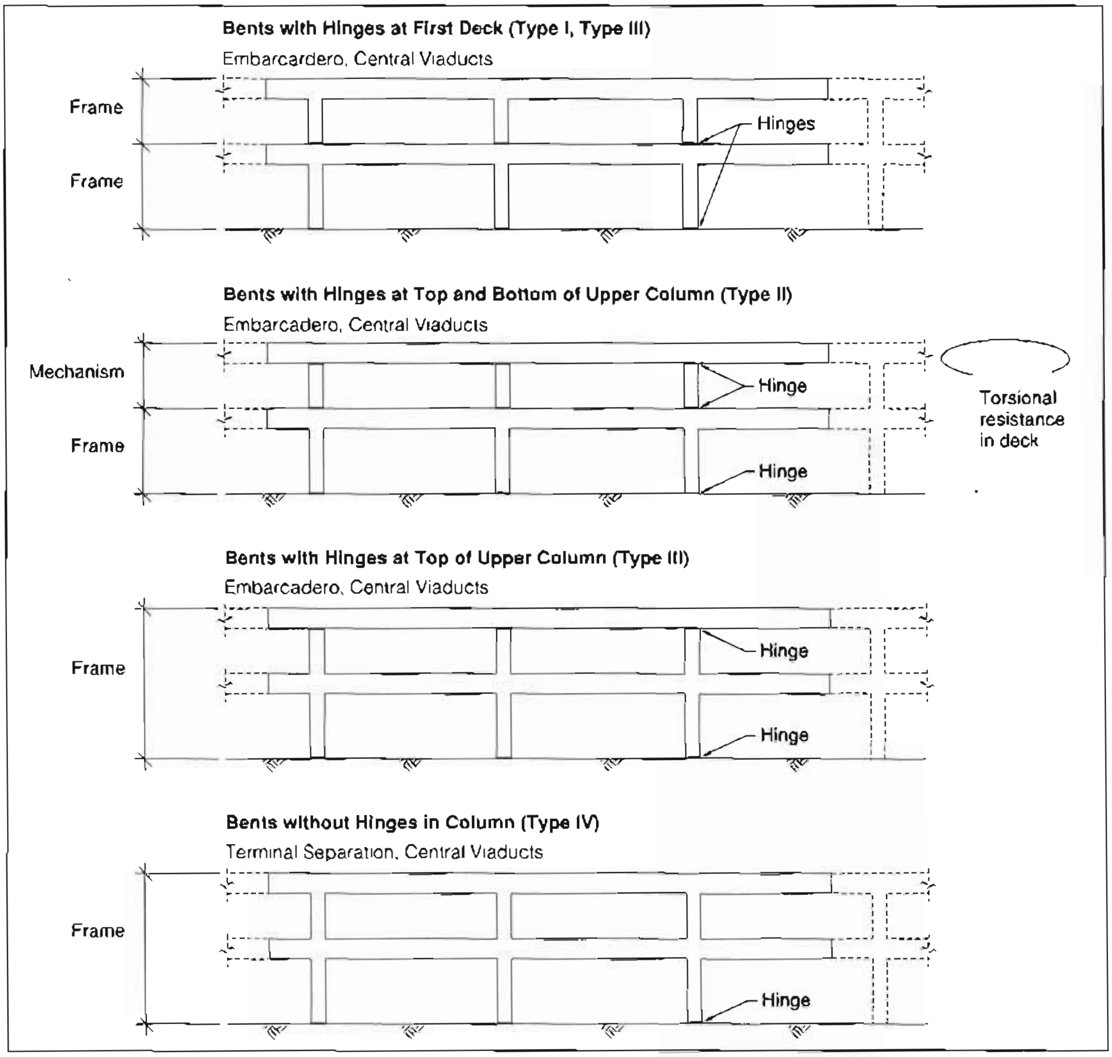


Figure 11-17. *Configuration of two-level longitudinal frames.*

Bent Foundations

All San Francisco Freeway Viaduct two-story bents are supported on pile caps that are founded on piles. The bent column base is usually hinged at the footing cap preventing moment transfer to the cap and piles. Shear transfer is accomplished by a shear key and by nominal vertical reinforcing dowel bars (4 #5 bars) located at the center of each column, Figure 11-18. The lateral load capacity of the column at the foundation connection is limited to the capacity of the shear key way and dowels.

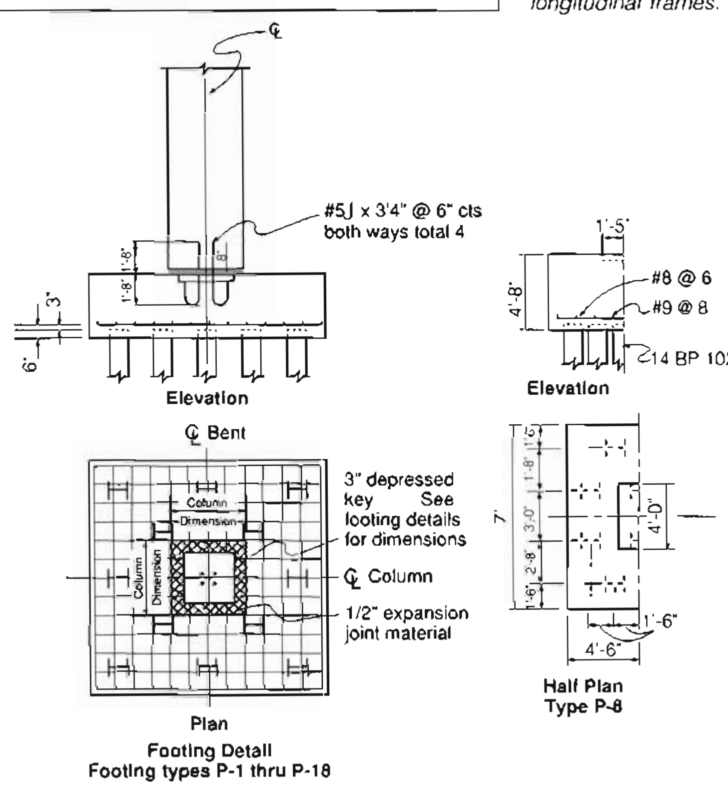


Figure 11-18. *Bent foundation details.*

Table 11-4. *Summary of primary damage to San Francisco Freeway Viaducts*

Freeway	Damaged Bents	Description of Damage
Terminal Separation Viaduct	44	Girder shear fracture
	48	Girder shear fracture
	51	Girder shear fracture
Embarcadero Viaduct	75	Lower knee joint, shear fracture
	77	Lower knee joint, shear fracture
	78	Lower knee joint, shear fracture
	79	Lower knee joint, shear fracture
Central Viaduct	42	Upper column, shear failure
	43	Upper column, shear failure
	44	Upper column, shear failure
	45	Upper column, shear failure
	48	Upper column, shear failure
Alemany Viaduct	—	No damage
Southern	48	Upper column, shear fracture
	51	Lower knee joint, shear fracture
	52	Lower knee joint, shear fracture
	5	8Joint pounding, 8-1/2 inch movement
China Basin Viaduct	32	Lower outrigger bent, shear fracture
	N1-35	Top outrigger bent, shear fracture

Earthquake Damage Sustained by the San Francisco Freeway Viaducts

Five of the six San Francisco Freeway Viaducts suffered damage in the Loma Prieta earthquake. Although none of this damage resulted in actual collapse, the damage was severe enough to require emergency shoring of all five damaged viaducts. Table 11-4 is a summary of the significant damage (as documented by Caltrans), and indicates problems with shear capacity of both columns and frame knee joints. These observed fractures occurred at relatively low levels of ground acceleration [Shakal et al., 1989], indicating a low overall seismic capacity of these viaduct structures.

In addition to Caltrans's surveys of damage, several other brief reviews of damage have been undertaken by EERI [1989], Preistley and Seible [Dec. 3, 1989A; Dec. 3, 1989B]; Caltrans consultants [T.Y. Lin Intl., 1990; Bechtel, 1990; CH2M Hill, 1990; DeLeuw, Cather & Co., 1990; Parsons Brinckerhoff, 1990; Tudor Engineering, 1990]; and by Astaneh et al. [1989]. A brief discussion of major damage to five of the San Francisco Freeway Viaducts follows.

Geometrically the most complex of the San Francisco structures, the Terminal Separation Viaduct makes the transition from a single-level viaduct to two and three levels with crossovers. The geometry of the structure creates numerous outrigger girders, many of which show serious shear fractures (Figure 11-19). Some of these cracks apparently existed prior to the Loma Prieta earthquake while others are new cracks induced by the earthquake. These cracks imperil the vertical load capacity of the girders and Caltrans has elected to temporarily shore up the damaged bents.

The primary damage to the Embarcadero Viaduct was located between Bent 75 and Bent 79, although other bents also sustained some damage. Figure 11-20 illustrates the complexity of the structural

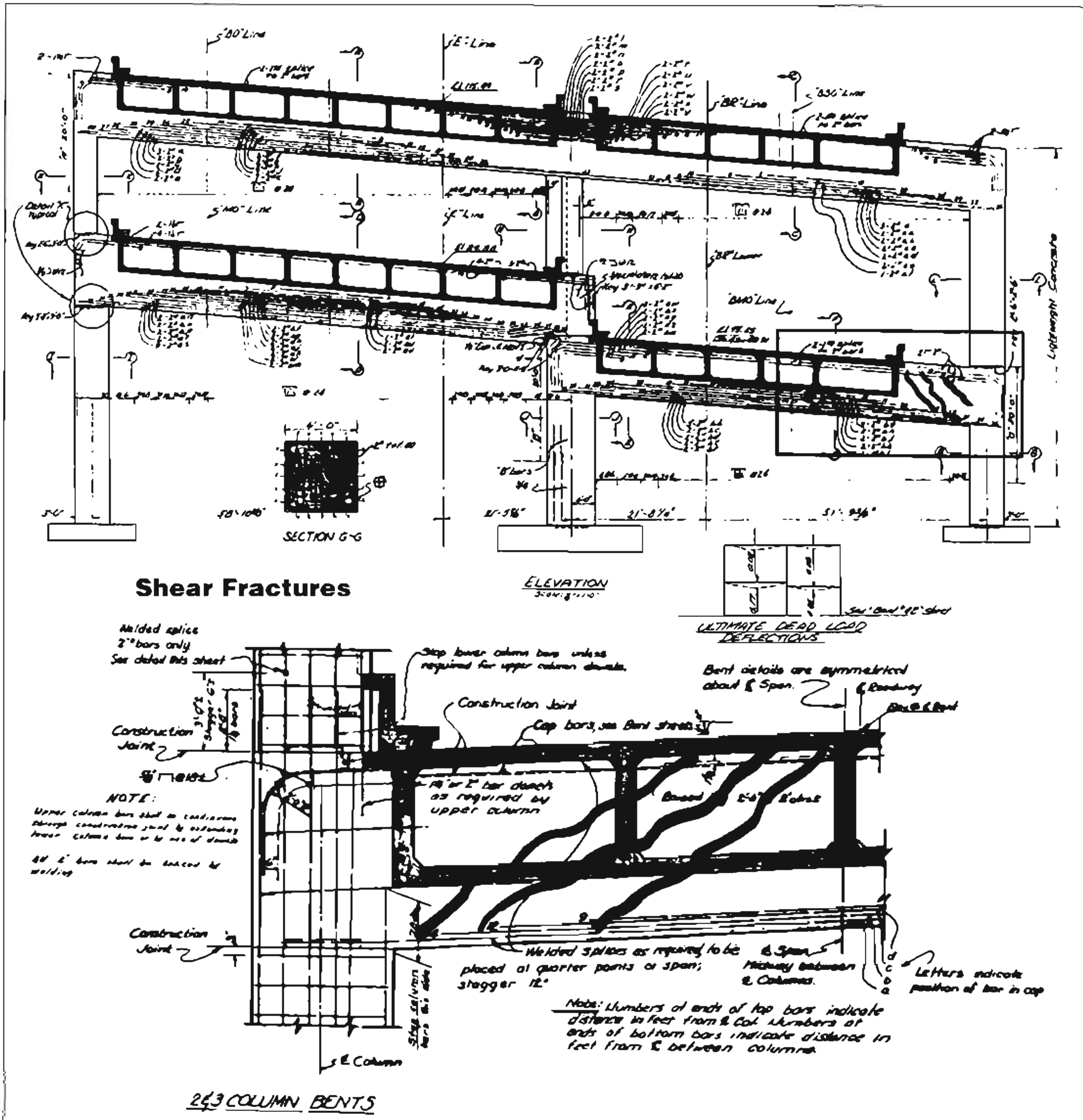


Figure 11-19. *Earthquake damage to Terminal Separation Viaduct Bent 44; detail is from opposite side.*

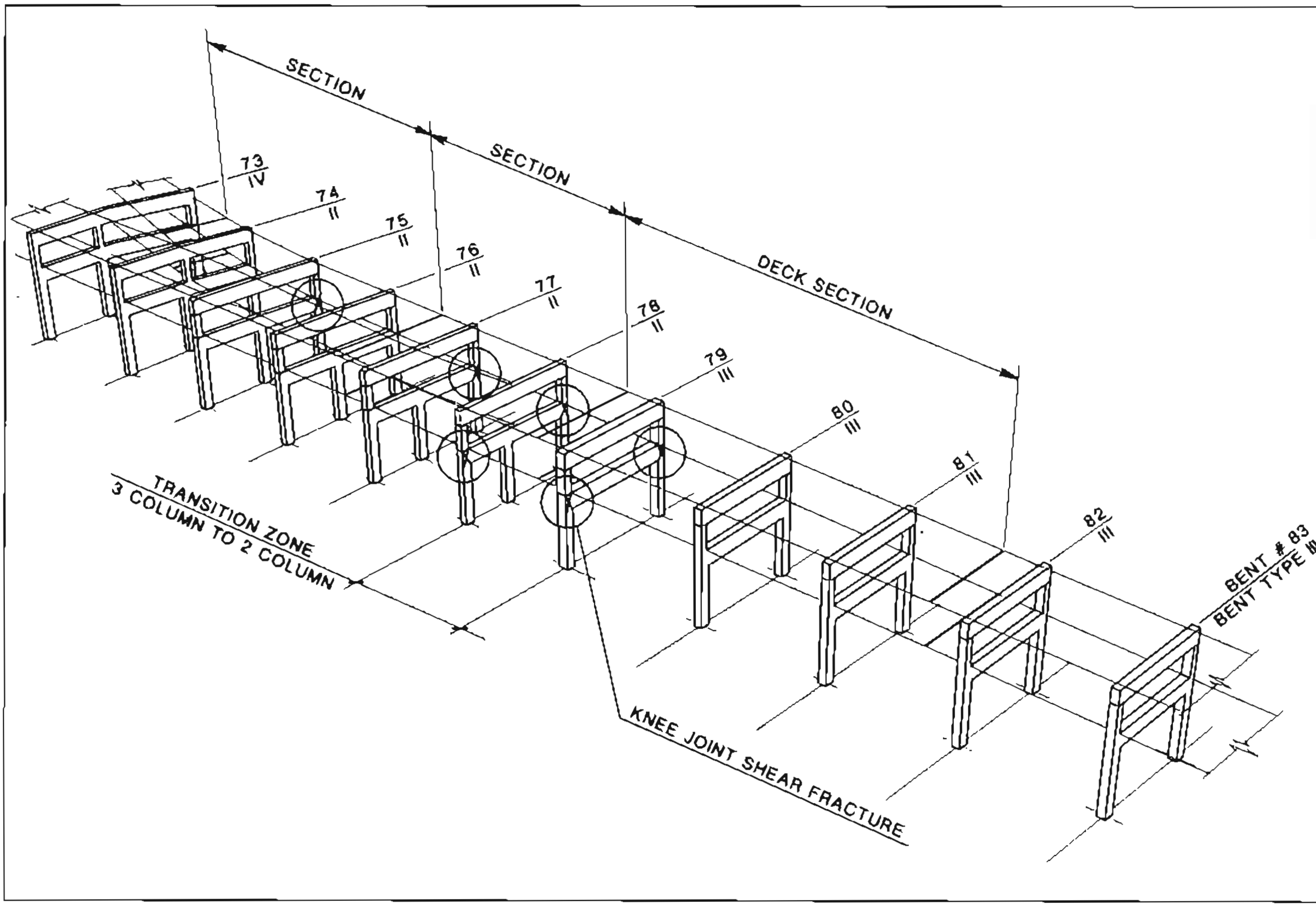
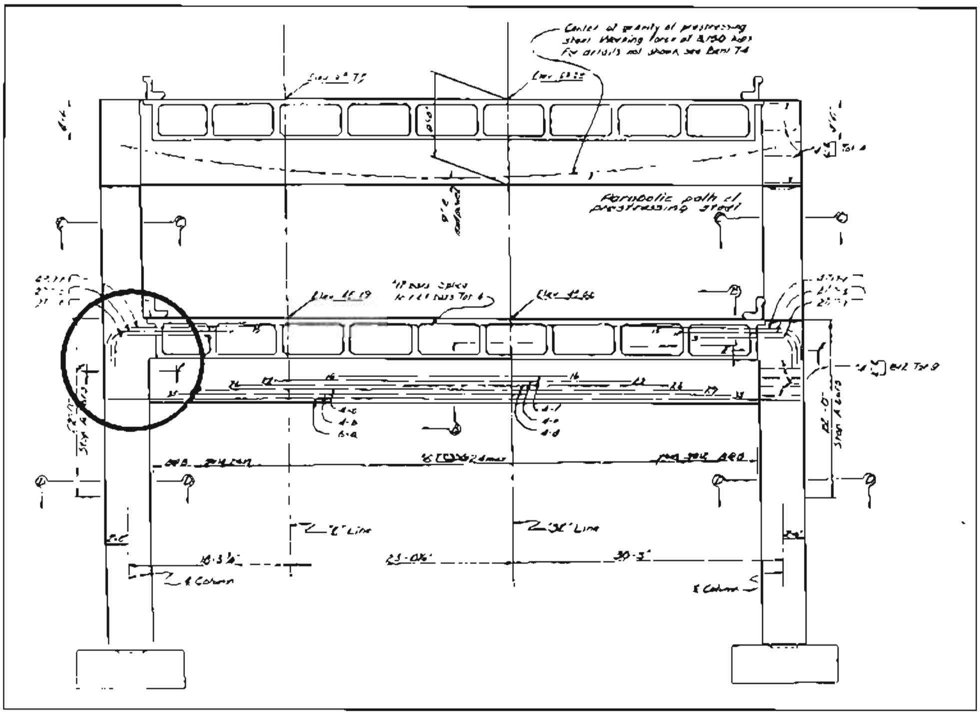


Figure 11-20. *Earthquake damage to Embarcadero Viaduct, partial isometric view.*

systems with varying roadway width, roadway deck expansion joints, and bent types and spans. The primary damage was knee joint shear fracture occurring at the mid-level knee joint on either side of Bents 78 and 79. Other knee joint fractures occurred on the west side at Bents 75 and 77. The nature of the fractures indicates strong lateral motion, but, because of the opposite inclination of shear cracks in adjacent bents, probably not all of the bents were translating in-phase nor did they fracture on the same cycle. The joint fractures occur in bents that are near the transition between three-leg bents and two-leg bents, and also in a zone of roadway narrowing. The two-leg bents, Bent 78 and Bent 79, are bent types II and III, respectively. Figure 11-21 illustrates the damage at Bent 78.



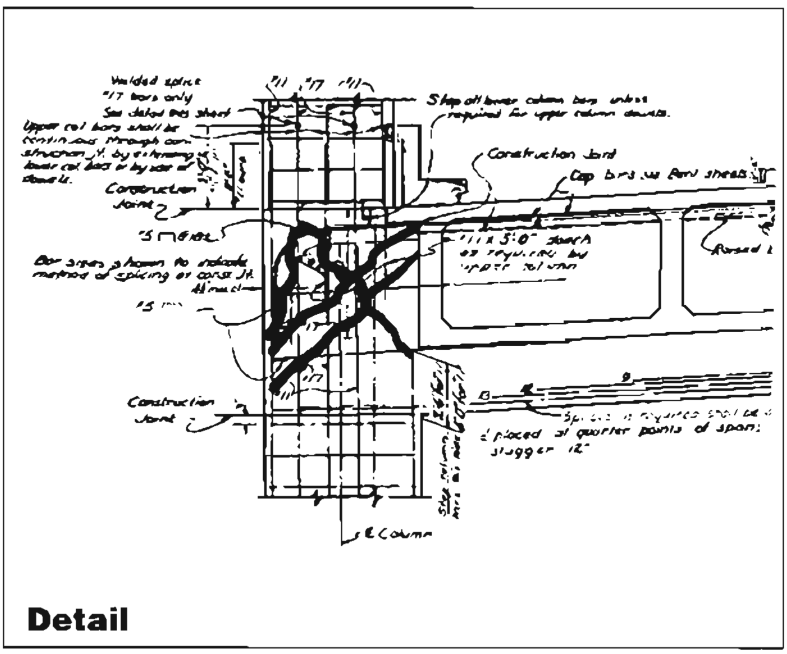


Figure 11-21. *Earthquake damage to Embarcadero Viaduct Bent 78.*

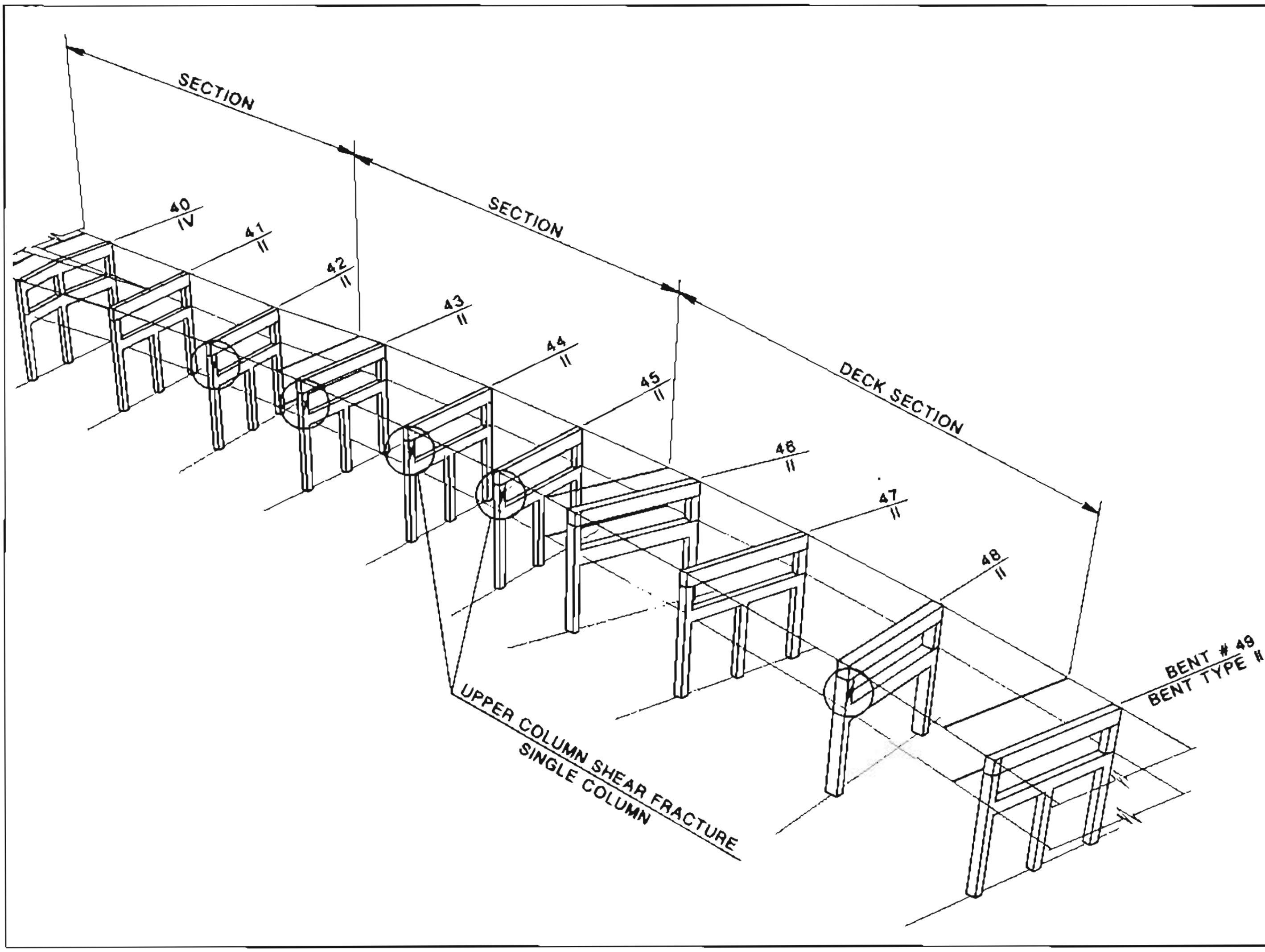
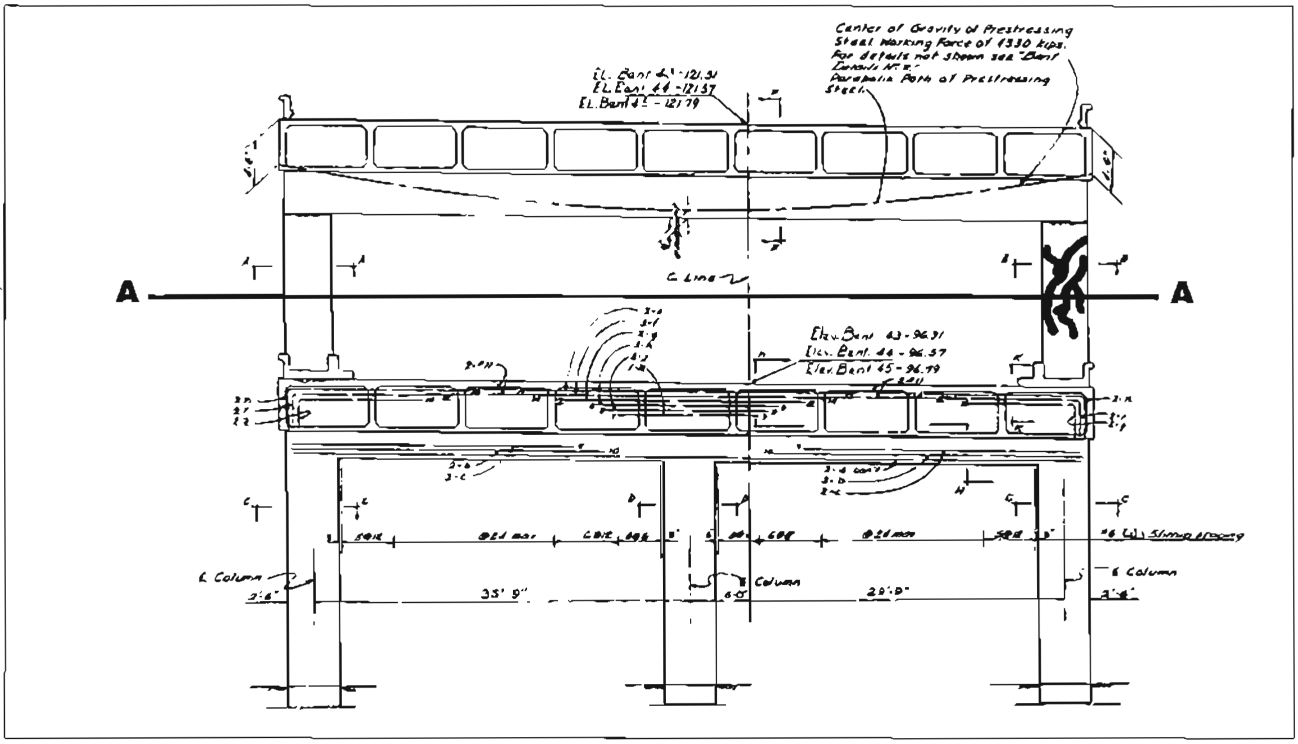


Figure 11-22. *Earthquake damage to Central Viaduct, partial isometric view.*

The Central Viaduct was damaged between Bent 42 and Bent 48. All of these bents are of type II configuration, each with one cantilevered upper level column. Figure 11-22 provides an isometric view of the structure between Bents 40 and 48. The damage was a midheight shear fracture occurring in the upper level single cantilevered columns at Bents 42, 43, 44, 45, and 48. Figure 11-23 illustrates the damage at Bent 43.



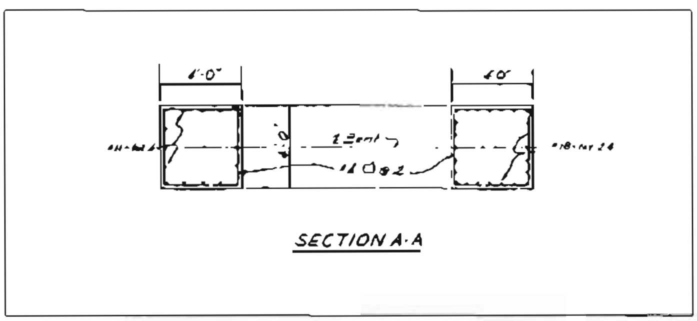


Figure 11-23. *Earthquake damage to Central Viaduct Bent 43*

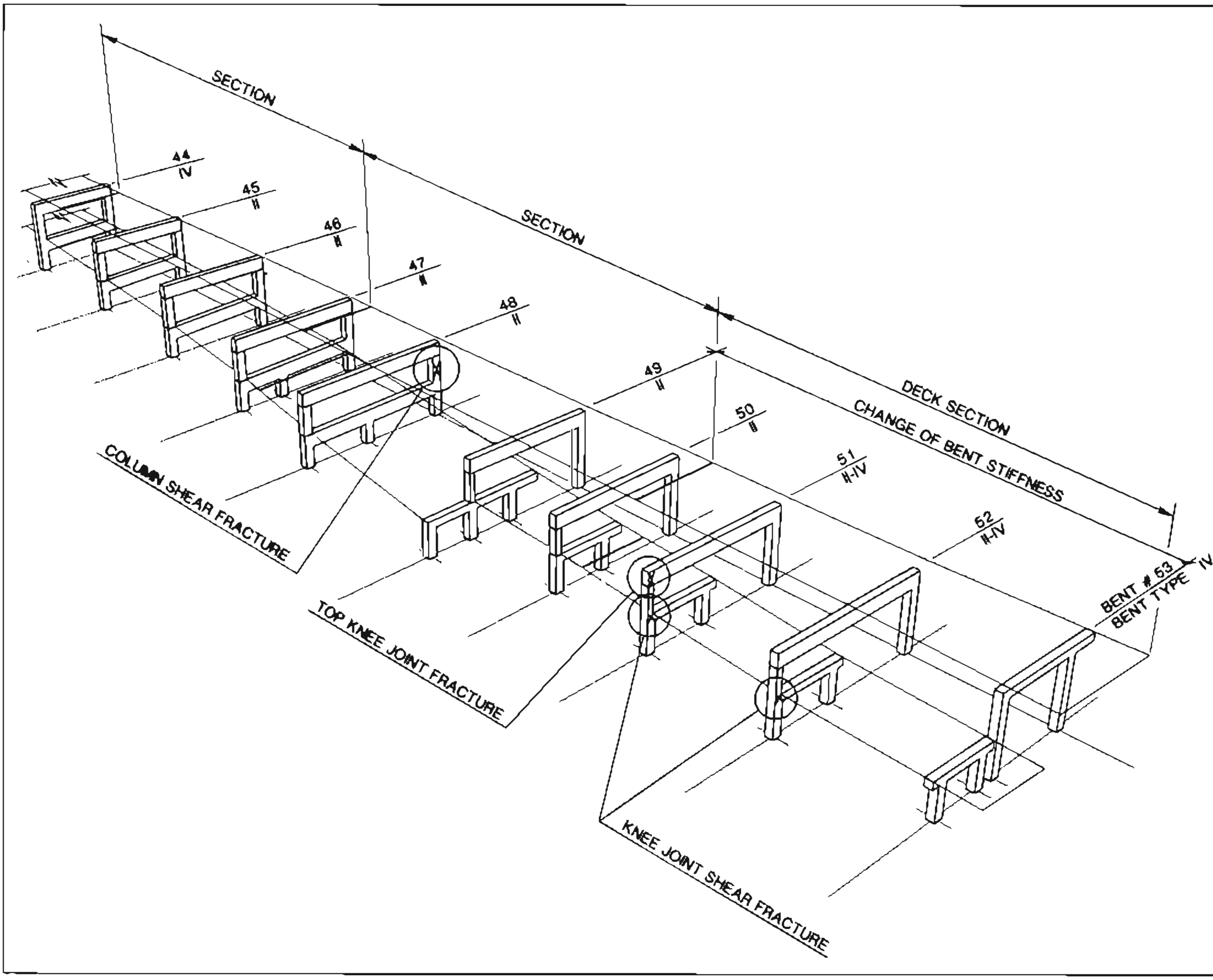
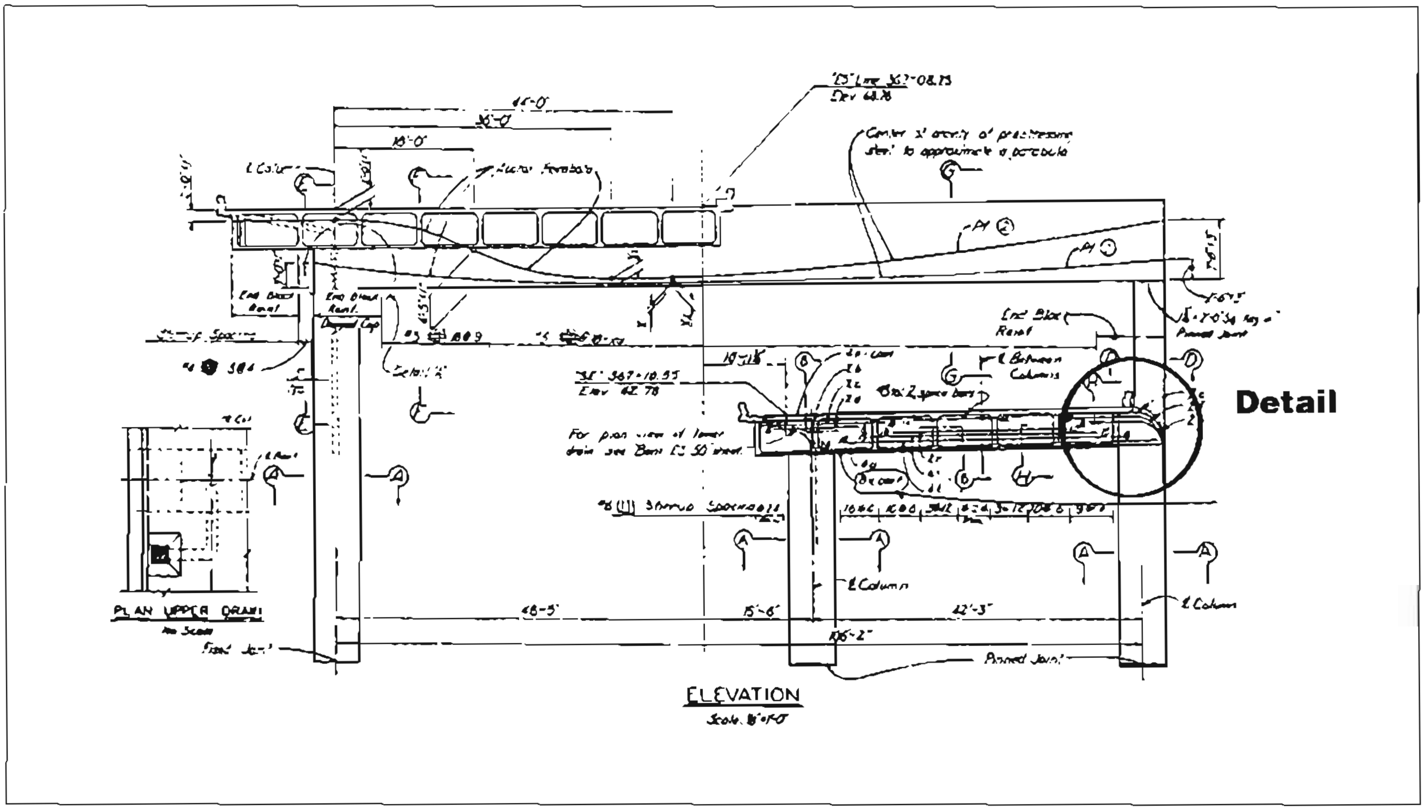
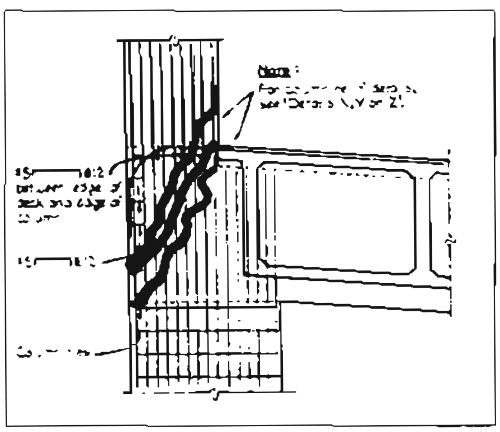


Figure 11-24.
Earthquake damage to Southern Freeway Viaduct, partial isometric view

The area of damage to the Southern Viaduct occurred at a zone where the roadway makes a transition from a two-level structure to a single-level structure (Figure 11-24). Damage resulted from the significant transverse motion of Bents 51 and 52 which fractured the east side mid-level knee joints. These transverse moment frames are unique with a large two-story frame cradling a one-leg frame below. Bent 48 also underwent significant lateral motion that fractured the upper columns. Figure 11-25 illustrates the damage to Bent 51; Figure 11-26 shows Bent 48 with the column shear fracture and a substantial 2" to 3" lateral movement.

Damage to the China Basis Viaduct was





Detail (reverse view)

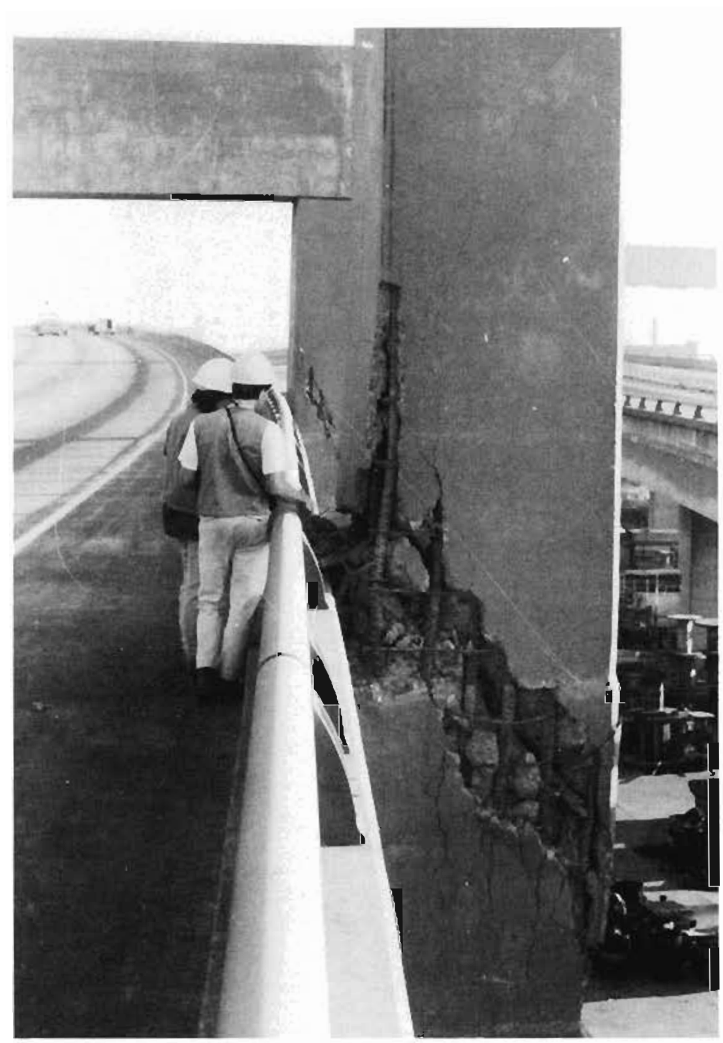


Figure 11-25. *Earthquake damage to Southern Freeway Viaduct, Bent 51*

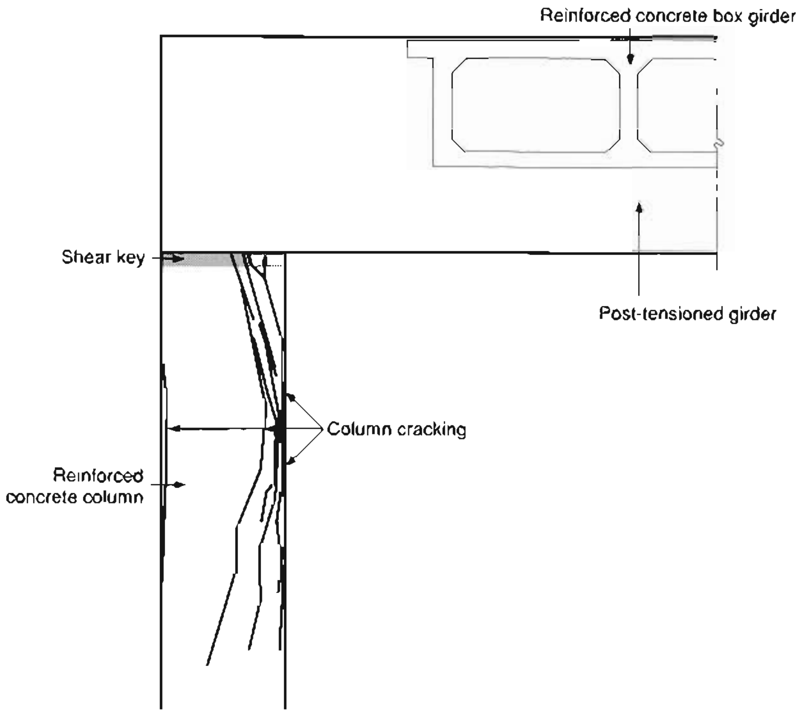
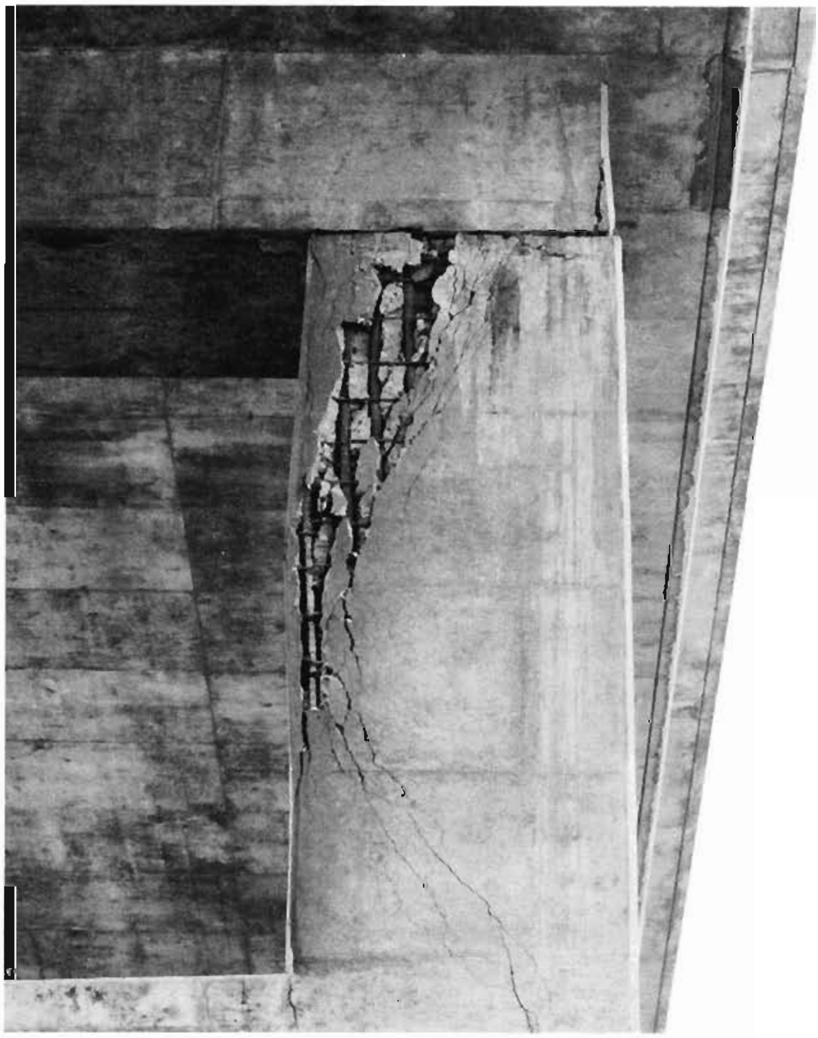


Figure 11-26. Earthquake damage to Southern Freeway Viaduct, Bent 48, lower photo shows reverse view of upper figure.



primarily limited to Bent 32 and to Bent N1-35, which is shown in Figure 11-27. Both bents show shear fractures of the upper outrigger knee joint.

A summary of significant bent damage is shown in Figure 11-28, where it can be seen that all failures were shear failures either of columns or of frame knee joints. These failures are indicative of inadequate shear capacity caused by insufficient and poorly anchored transverse ties in both the columns and the knee joints. The location of damage also correlates well with site conditions, particularly areas of soft soils or fills where ground motions were intense.

Based on damage observed to the viaducts following the Loma Prieta earthquake, it can be concluded that the expected performance of the San Francisco structures in a moderate-to-major earthquake would will be at best poor, and perhaps even catastrophic.

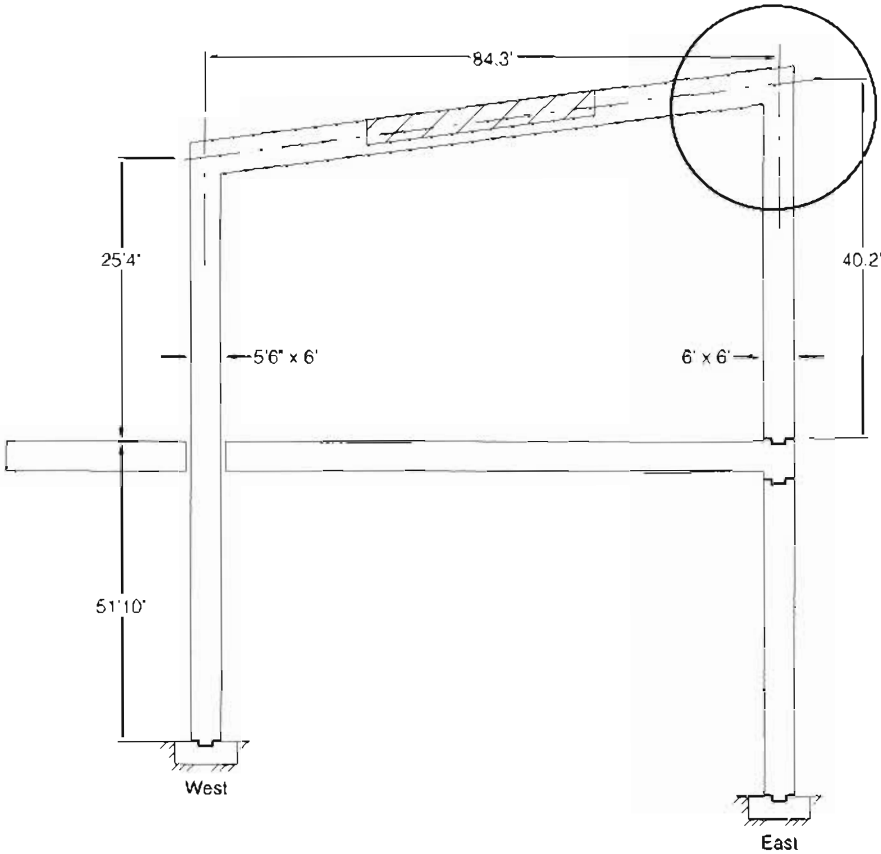


Figure 11-27. *Earthquake damage to China Basin Viaduct Bent N1-35, lower photo shows reverse view of upper figure*

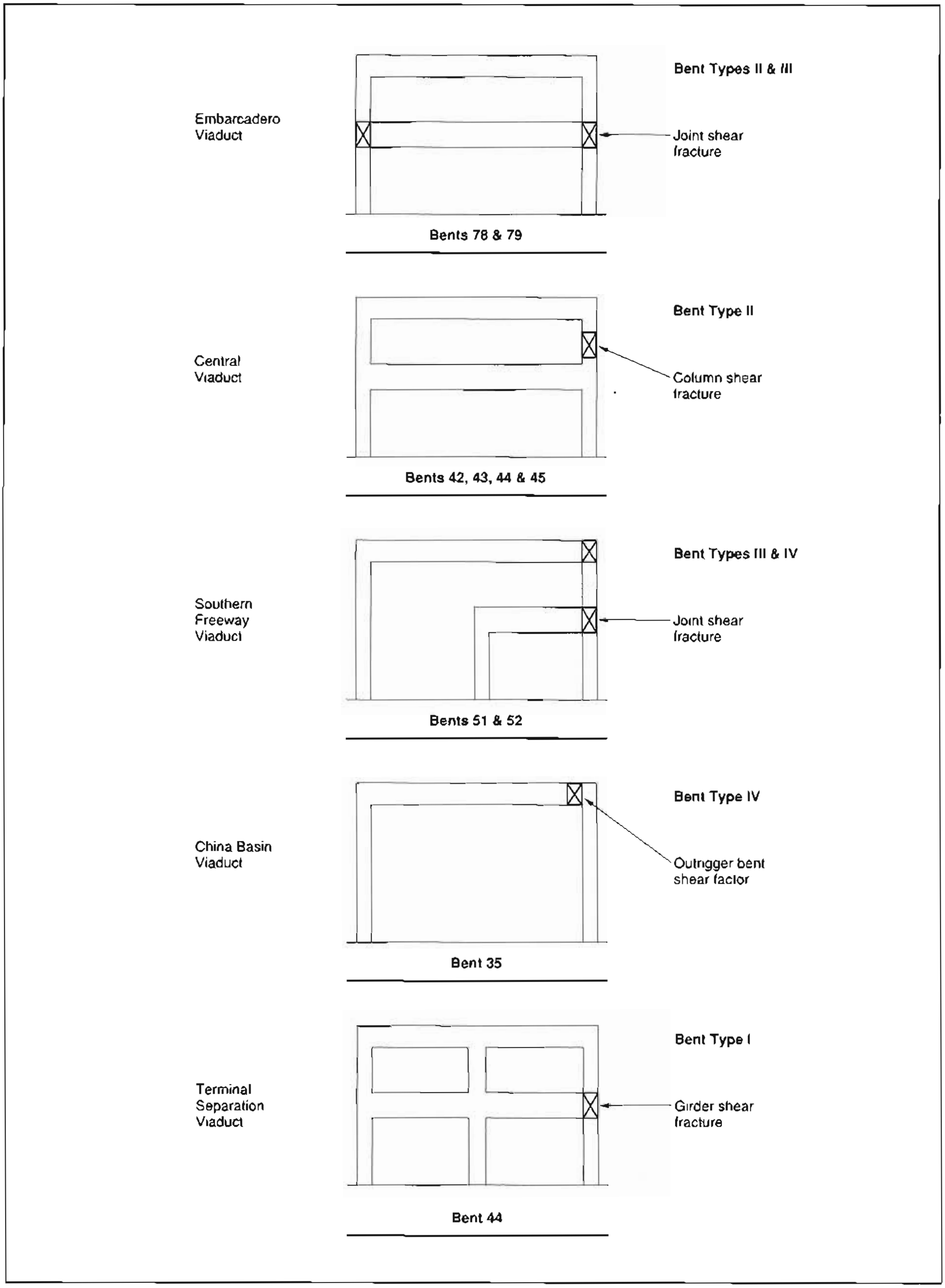


Figure 11-28. *Summary of damage patterns in double-deck San Francisco Freeway Viaducts.*

Conclusions

The Loma Prieta earthquake of October 17, 1989 was, for the San Francisco Freeway Viaducts, a minor-to-moderate earthquake with recorded peak ground accelerations less than 0.15g and not more than 5 to 10 seconds of strong motion shaking. All of the freeway structures, with the exception of the Alemany Viaduct, were damaged during the earthquake and subsequently closed to traffic. If these structures had been subjected instead to a Richter Magnitude 7+ earthquake nearby on either the Hayward or San Andreas faults, damage would most likely have been widespread and catastrophic with most, if not all, of the elevated freeway structures collapsing.

The San Francisco Freeway Viaducts are composed of single-column and multi-column bents, typically with two-tiers, but with a maximum of three tiers, of framing supporting two levels of roadway. The transverse lateral load resisting system in the multi-column bents typically consists of a pinned-base single-story portal frame with one or more columns cantilevering to the upper level girder. The reinforcement in the columns and girders is generally poorly detailed by current standards and reflects the engineering profession's lack of understanding regarding the inelastic response of reinforced concrete members at the time when these viaduct structures were designed. In the longitudinal direction, none of the six freeway viaduct structures have a planar lateral load resisting system. The lack of redundancy and the inadequate reinforcement detailing are two of the major seismic deficiencies in these freeway structures.

Damage to the individual viaducts varied and included shear cracking in columns, girders and joints, torsional cracking in outrigger bents, anchorage failure of the girder reinforcement, and shear key failure, among others. Many of the crack patterns are similar to those observed in the collapsed and damaged portions of the Cypress Viaduct.

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Chapter 12

Repair and Upgrade of the San Francisco Freeway Viaducts

The six San Francisco Freeway Viaducts (Terminal Separation Viaduct, Embarcadero Viaduct, Central Viaduct, Alemany Viaduct, Southern Freeway Viaduct, and the China Basin Viaduct) are vital links in the transportation system of the region. Five of these viaducts were damaged in the Loma Prieta earthquake (the only undamaged viaduct was the Alemany). Caltrans began immediately to plan for the repair and seismic upgrading of these structures so that they could be put back in service as soon as practical. They retained six different engineering consultants, one for each structure, to prepare plans for their repair and seismic retrofitting.

The Cypress Viaduct and all six of the San Francisco Freeway Viaducts were among the 109 bridges designated for cable restrainer retrofitting after the 1971 San Fernando earthquake. Some of the restrainers installed in the Cypress Viaduct failed during the October 17, 1989 earthquake as a result of spans collapsing. Either the assembly punched through the end wall diaphragm or the cables themselves snapped [Nims et al., 1989]. None of the San Francisco Freeway Viaduct restrainers failed during the earthquake. In 1984, Caltrans added externally-mounted post-tensioned rods to the exterior face of the upper level column-to-girder joint in a number of the Viaducts in an attempt to increase the moment capacity of the joint. Following a peer review and further analysis, Caltrans judged this partial upgrading solution to be ineffective.

With few exceptions, the damage to Viaduct columns was limited to repairable cracking. The damage was so severe to several Southern Freeway Viaduct columns that the existing columns are being demolished and new columns built. Cracks of all widths are typically being repaired using epoxy grout injection. The ability of the epoxy grout to completely penetrate and fill these cracks is unknown, particularly for those cracks that formed during the earthquake that have closed under the weight of the structure (e.g., shear cracks in the column-to-girder joints). Complete penetration of these cracks by epoxy grout is virtually impossible. Some tests of repaired members indicate clearly that their strength and stiffness is less than that of the uncracked element and that the energy dissipation characteristics of the repaired element degrade quickly following re-cracking of the wall [Wong et al., 1975].

The balance of this chapter discusses the criteria and initial approach to the repair and seismic retrofitting of the six San Francisco Freeway Viaducts.

Caltrans Seismic Upgrade Criteria

Caltrans's objective for upgrading the six San Francisco Freeway Viaducts is to "...produce structures that are safe and resistant to collapse...," but notes that the upgraded structures will have "...the potential for serious damage and possible closure following an earthquake..." [State of Calif., Dept. of Trans., Letters/Memoranda to Consultants, various dates]. This is consistent with the intent of the Caltrans Bridge Design Specification (BDS). That is, for bridges designed for the minimum seismic requirements specified in the BDS, it is assumed that they will suffer moderate-to-major damage during severe earthquake shaking. The resulting damage may be so extensive that the bridge may have to be closed for long-term repairs, or even demolished and rebuilt.

The criteria set by Caltrans for the development of the upgrading schemes for the San Francisco Freeway Viaducts were outlined in a series of letters and memoranda sent to all six consultants [State of Calif., Dept. of Trans., Letters/Memoranda to consultants, various dates]. In these, Caltrans noted that:

> ...The semi-permanent retrofit analysis/design should be based on the use of externally attached devices which can be incorporated into a future final rehabilitation project. The decision not to use permanent-type solutions is primarily based on the lack of research to verify the procedures and analyses being used to retrofit these structures. Although we expect this current rehabilitation to produce structures which are safe and resistant to collapse, the potential for serious damage and possible closure following an earthquake remains. Upon completion of research currently underway and/or soon to be initiated, we expect to determine more appropriate site-specific analytical tools which will permit a more damage-resistant retrofit.
>
> The scheme(s) which you choose should be ones which can be enhanced in the future (i.e., add ribs to steel plates, install additional prestress rods, locally strengthen webs and flanges of attached beams, etc.), and encased in concrete to provide an improved appearance. In essence, external steel devices would seem to fit this concept, whereas concrete devices would seem to be more difficult to improve at a later date.
>
> There may be localized special areas where a more permanent type improvement may be required. Such solutions would be acceptable; however, your analysis must show convincing evidence for this solution. In addition, permanent type replacements are appropriate where your field investigations establish that damage has caused structural degradation beyond repair.
>
> Isolating and damping devices are an acceptable alternative to the semi-permanent devices discussed above. However, these force reducing devices are considered permanent. The analysis should be based on site-specific soil data and response, and must consider the increased deformation created by said device. Isolation devices should be installed at this time if they are a viable option ...

The upgrading criteria specified by Caltrans included the following:

Soil Response Spectra. "...The Bridge Design Specification response curves used in the two orthogonal directions produce an acceptable level of forces for a ductile design...use of other site-specific response curves is acceptable, subject to our review..."

Ductility Demand. "...The intent is to reduce all ductility demands to a maximum level of 4.0. This may be impossible at all locations. Localized demands up to 6 can be acceptable. These locations must be identified and justification documented. Justification examples could be, but not limited to:

1. Columns, lightly reinforced in the vertical direction, which are translating less than one-quarter the column dimension in the direction of movement;
2. A column surrounded by other columns having acceptable ductility demands (i.e., redundancy). In addition, there may be isolated columns which require replacement, or other permanent-type fix, where retrofit measures are impossible or impractical. A permanent fix is acceptable under those conditions..."

Analytical Model Modifications. "...The designer is the judge for determining when to allow columns to pin as a means of arriving at a retrofit solution. Pins can be allowed to form at fixed connections where lap-spliced column bars exist if the Class P casing method is used to confine those locations. Where the as-builts show the option of lapped splices or continuous bars at a footing, the lap option can be assumed. That was the general method of construction at the time..."

Analytical Model Boundary Conditions. "...Restraints and/or releases at model ends must represent realistic boundary conditions. Torsional fixity at hinges is expected unless applied torsion is greater than dead load reaction (unlikely in concrete structures). Springs representing the stiffness of the adjacent frame should be used in both the tension and compression models if the model ends at an expansion hinge..."

Hinge Restrainers. "...The analysis must include an evaluation of existing hinge restrainers and transverse joint shear capacity. The number of restrainers must be evaluated using the Memo to Designers 15-10 method and increased if there is a deficiency. However, the total cable force must not exceed the tensile capacity of the superstructure (which could be controlled by minimal reinforcing steel development length at hinge diaphragms). Longer, more elastic cables, in combination with hinge seat extenders, may be required in lieu of tightly tied joints because of limited superstructure tensile capacity. Transverse shear capacity must also be assured. Both seat extenders and transverse keys could be externally mounted, providing traffic clearances are not restricted..."

Concrete Shear Strength. "...Based on tests at UCSD, there appears to be substantial reserve strength above design values. It would be appropriate to boost the nominal design strength by 50%. However, shear capacity is greatly influenced by axial compression. No increase should be used when axial loads are less than $0.1\ F'_C A_g$. Concrete shear strength should be reduced in accordance to AASHTO as axial load approaches zero and/or becomes tensile due to overturning effects. Applied shear should be computed from elastic forces or from $1.5\ \mu$ whichever is smallest..."

Column Plastic Moment. "...The plastic moment is dependent upon column bars being fully developed into caps and footings to allow yield of the bar. Insufficient development can be improved to assure yield by prestressing the concrete in the development area. The moment capacity must be reduced or considered zero according to your best judgment when development is not assured. Ductility demands are affected accordingly. The nominal moment of lap spliced connections in plastic hinging zones should be reduced 25% if well confined (i.e., Class F casing) or considered zero if partial confinement exists (i.e., Class P casing)..."

Footing Capacity. "...It is possible that footing capacity may be exceeded because of increased seismic forces or increased retrofit structure stiffness not anticipated in the original design. Overturning effects at bents can significantly increase axial load (good for column shear; detrimental for piles). The column connection, footing, and pile capacities must be examined and improved, if required. Ultimate axial capacity of piles can be used for intermittent earthquake loads. Capacity varies with end condition (bearing or friction) and unsupported length ..."

Joint Connections. "...All joints must be evaluated for continuity based on rebar development. Deficient connections must be improved to resist expected seismic forces. Lap splices to be jacketed for confinement to produce a safe pin or grouted solid to insure plastic hinging..."

Seismic Upgrade Experimental Data

Caltrans and its consultants are basing their upgrading strategies in part on the experimental work of Mahin, Moehle and Stephen [Mahin et al., 1989] at the Cypress Viaduct during the months of November and December 1989, and on the research work of Priestley [Chai et al., 1990] at the University of California at San Diego.

Mahin et al. at the University of California at Berkeley retrofitted three standing bents of the Cypress Viaduct. Each bent was retrofitted with a different strengthening technique. In all three bents, post-tensioning bars were installed parallel to the upper and lower girders to provide additional girder moment capacity and to provide some confinement in the column-to-girder joint region. The main features of each strengthening technique were:

Bent 45: Vertical steel wide flange (WF) strong-backs attached to the exterior face of the columns; vertical discontinuous WF strong-backs attached to the interior face of the columns; strong-backs on column faces stitched together. Upper column pedestals reinforced with steel plates on the interior face to be stitched to the external WF strong-backs.

Bent 46: Lower girder-to-column joint reinforced with rock anchors extending downward through the joint at 30° to the horizontal. Upper column pedestals reinforced by means of thick steel plates post-tensioned to the column.

Bent 47: Upper column pedestal reinforced with steel plates that extended up into the bottom portion of the upper column. Steel plates attached to the interior and exterior faces of the column and post-tensioned to it.

In their preliminary report, Mahin et al. noted problems with each of the strengthening schemes. Furthermore, their tests were performed at a psuedo-static rate (their only real alternative) whereas the earthquake loading is dynamic in nature. It is well established that the static and dynamic behavior of concrete elements can vary. These tests generated very useful information on three different upgrading strategies, but they did not produce a strengthening scheme for the San Francisco Freeway Viaducts.

Priestley has tested different upgrading techniques for circular, nonductile cantilever columns as part of the Caltrans single column/pier retrofitting program. As of this writing, testing has yet to begin on the upgrading of rectangular columns using elliptical steel jackets—the approach adopted for the retrofit of columns.

Seismic Upgrade Analysis and Design Methodology

All six consultants used elastic finite element analyses as the basis for developing their upgrading solutions for the San Francisco Freeway Viaducts. Frame elements were typically modeled as 3-D beam-column elements (6 degrees of freedom per node) and bracing elements were modeled as 3-D truss elements (3 degrees of freedom per node). Analyses were performed in accordance with the BDS using:

- Compression models (expansion joints hinged around a horizontal axis in the plane of the bent and a vertical axis; torsion and three translational force resultants transmitted through the joint) and tension models (expansion joints linked by a linear spring; torsion and vertical shear force and transverse horizontal shear force transmitted through the joint) to simulate the effect of the axial restrainers.
- A response spectrum approach using the ARS spectra (recommended by Caltrans), normalized to a peak ground acceleration of 0.5g, or the Seed/Sun soft soil spectrum [Seed and Sun, 1989] recommended by Dames & Moore. Figure 12-1 illustrates the ARS and Seed/Sun spectral details.
- Static and dynamic (transverse seismic and longitudinal seismic) analysis result combinations to yield design force and moment resultants.

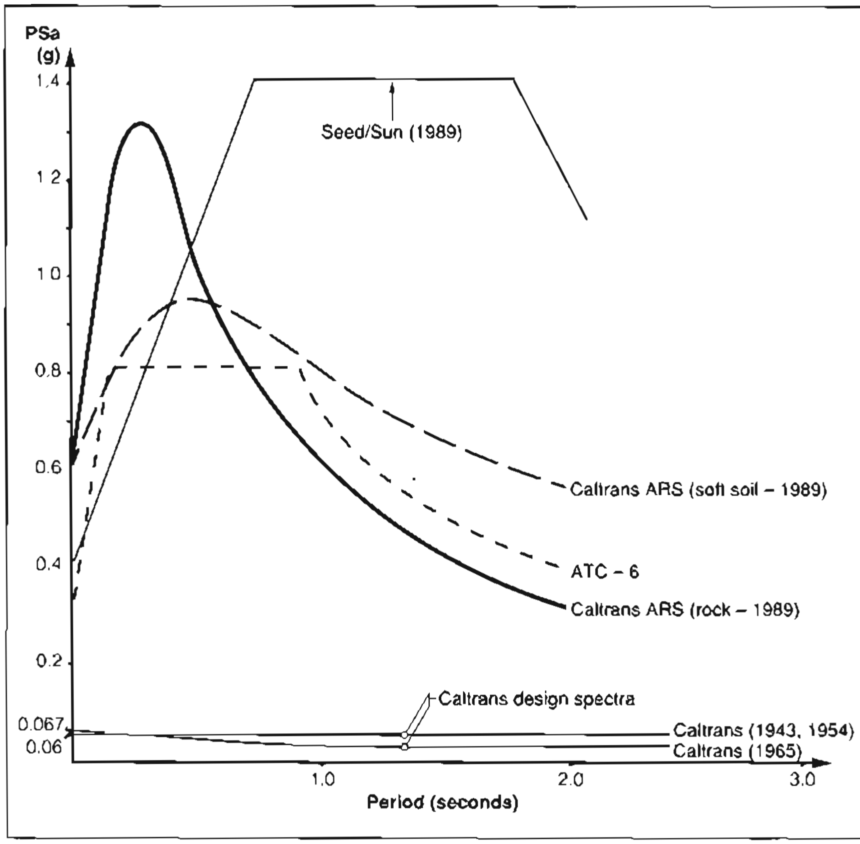

Figure 12-1. *Caltrans ARS spectra (1989) and design spectra (1943, 1954, 1965) for multi-pier bents.*

Using the results of the elastic response spectrum analyses, Caltrans directed their consultants to upgrade the freeway structures using a procedure similar to the following:

1. Analyze the existing freeway structure, and,
   A. For cases where the elastic flexural demand (D) was less than 1.5 times the section's flexural capacity (C), that is, $D/C < 1.5$, no strengthening or confinement was required.
   B. For cases where $1.5 < D/C < 4$, the section was to be confined and its shear capacity increased.
   C. For cases where $D/C > 4$, the section was to be confined and strengthened to reduce the D/C ratio to less than 4.
2. Re-run the analysis using the modified strengths and stiffnesses to estimate the new D/C ratios.
3. Iterate as required to typically reduce the D/C ratios to less than 4.
4. Design the joints and connections using capacity design procedures, that is, using those forces associated with plastic hinge formation in the adjacent members.

General Discussion of the Upgrading Approaches

The collapse of the Cypress Viaduct, a structure whose reinforcement details are virtually identical to those in the six San Francisco Freeway Viaducts, indicated that these structures had very limited lateral strength, were poorly detailed by current standards, and had little or no redundancy. The retrofitting approach selected must accommodate these basic facts about the structures' characteristics.

The selection of any upgrading scheme for any building or structure should address three major issues:

1. The compatibility of the strength-deformation relationships of the existing structural system and the proposed upgrading system (that is, ensuring that the upgrading system can protect the existing vulnerable structural system).
2. The choice of appropriate intensities and durations of earthquake shaking.
3. The selection of the damage criteria associated with number 2.

Caltrans specified the level of earthquake shaking defined by the ARS spectral ordinates and the corresponding damage criteria—serious damage is permitted but collapse is excluded. The compatibility of the upgrading scheme with the existing structure, and the other shortcomings of the viaduct structures noted below, were appropriately left up to the design consultants. Caltrans also specified that the particular seismic upgrading system for each structure also should take into account or provide:

1. Limited strength and negligible ductility in the Viaducts' transverse framing.
2. Absence of planar longitudinal framing in all six viaducts.
3. Relatively high lateral transverse stiffness of the viaducts, especially those incorporating multi-column bents.
4. Effects of variations in the lateral strength and stiffness of the transverse frames along the length of the viaduct

(for example, two-column bents between three-columns bents).

5. Lack of redundancy in the viaducts' structural framing.
6. Failure modes observed in the Cypress Viaduct during the Loma Prieta earthquake.
7. An acceptable, stable, plastic hinge formation sequence in the upgraded viaduct.
8. The damage incurred by the viaduct structures during the Loma Prieta earthquake.

As a first step, elastic analysis is a valuable tool and can be used to identify potential hinging zones and critical regions. The second step should involve the static load-to-collapse analysis of the typical transverse and longitudinal bents to predict more accurately:

- Ultimate strength of the individual frames.
- Sequence of plastic hinge formation.
- Local ductility demands required to develop the desired global displacement ductility.
- Maximum frame displacements associated with the critical ductility demand—to be used to determine the performance limits of the upgrading system.

Having established this information, the strength, stiffness and deformation characteristics of the upgraded system can be chosen—the required information cannot be predicted using elastic analysis. Once the upgrading scheme has been selected and designed, it should be reanalyzed using the procedures noted above (and 3-D nonlinear analysis) to verify the performance of the upgraded system. The evaluation of the retrofitting designs of these nonredundant, nonductile reinforced concrete frame structures should not rely solely on elastic analysis.

The design criteria set by Caltrans for the seismic upgrading of the San Francisco Freeway Viaducts appear, at the outset, to be reasonable. The major difference between their upgrading criteria and the BDS is limiting the adjustment factor (Z) to 4 from the value of 8 in the BDS. The Z factor is intended to account for material overstrength, redundancy in the structural system, and ductility (energy dissipation capacity) in the structural system. Limiting the demand-to-capacity (D/C) ratios to 4 acknowledges the lack of redundancy and ductility in the single and multi-pier bents. The upgrading criteria state that if the D/C ratio does not exceed 1.5, no action is required—arguing, perhaps, that the ultimate strength of a member is 1.5 times its nominal strength. This is a reasonable assumption, provided that nominal rather than measured material properties are used. However, for these upgrades, Caltrans has instructed its consultants to use the measured concrete strengths rather than the design compressive strength; in this case, a difference of nearly 60%. Linear elastic finite element computer programs have been used to analyze the expected performance of the viaducts. Elastic analysis can be used to predict the location of the first plastic hinge to form in a frame under a specified loading pattern, but

Table 12-1. *Characteristics of the proposed retrofitting schemes for the San Francisco Freeway Viaducts.*

Terminal Separation Viaduct	
Single-Column Bents:	
Transverse direction	Elliptical steel column jackets; footing strengthening
Longitudinal direction	Tubular steel X-braces; footing strengthening
Multi-Column Bents	
Transverse direction	Energy dissipators on tubular steel braces; tie beam at grade; post-tensioned rods to increase joint and girder shear capacity; post-tensioned rods through shear keys to increase shear capacity
Longitudinal direction	Tubular steel X-braces
Non-typical Bents	Reinforced concrete shear walls between closely spaced single column bents
Expansion Joints	Steel WF expansion joint seats and cable restrainers
Embarcadero Viaduct	
Multi-Column Bents	
Transverse direction	Steel channel sections post-tensioned to columns for column and joint confinement
Longitudinal direction	Steel channel sections post-tensioned to columns for column and joint confinement
Non-typical Bents	Reinforced concrete shear walls between closely spaced single column bents
Expansion Joints	Internal concrete diaphragm bolsters; 8" pipe hinge restrainers and cable restrainers
Central Freeway Viaduct	
Single-Column Bents	
Transverse direction	Elliptical steel column jackets
Longitudinal direction	Elliptical steel column jackets
Multi-Column Bents	
Transverse direction	Elliptical steel column jackets; column and girder steel plate strengthening; shear key strengthening
Longitudinal direction	No upgrade
Expansion Joints	Internal concrete diaphragm bolsters; cable restrainers
Alemany Viaduct	
Multi-Column Bents	
Transverse direction	Flat plate stiffened steel column joint and girder jackets; footing strengthening
Longitudinal direction	No upgrade
Expansion Joints	Internal concrete diaphragm bolsters; 8" pipe hinge restrainers and cable restrainers
Southern Freeway Viaduct	
Single-Column Bents	
Transverse direction	Elliptical steel column jackets
Longitudinal direction	Elliptical steel column jackets
Multi-Column Bents	
Transverse direction	Elliptical steel column jackets; flat plate steel joint jackets; post-tensioned rods to increase girder capacity; footing strengthening
Longitudinal direction	Tubular steel X-braces; footing strengthening.
Expansion Joints	Internal concrete diaphragm bolsters; 8" pipe hinge restrainers and cable restrainers
China Basin Viaduct	
Multi-Column Bents	
Transverse direction	Flat plate post-tensioned steel column and joint jackets; elliptical steel column jackets; footing strengthening; separation of girders from columns and addition of corbels and restrainers; external vertical post-tensioning of column
Longitudinal direction	No upgrade
Expansion Joints	Cable restrainers

rarely anything more. The assumptions regarding plastic hinge locations and ductility demands cannot be justified using elastic analysis because the hinge formation sequence, and the subsequent load redistribution, can preclude hinge formation in certain members and force hinges into others. These assumed values and analytical approaches warrant careful consideration during the design review process.

Specific Upgrading Approaches

Table 12-1 reviews the basic characteristics of the upgrade approaches for each of the Viaducts. In these descriptions, the terms "transverse" and "longitudinal" refer to the orientation of the bents with respect to the roadway: the transverse direction is perpendicular to the roadway and the longitudinal direction is parallel to the roadway.

Transverse Upgrade

All of the upgrading schemes developed to date for framing perpendicular to the roadway (transverse direction) are based on:

- Elastic finite element analysis alone (with the exception of Terminal Separation Viaduct for which some 2-D nonlinear analysis has been completed).
- Caltrans's specified adjustment factors (Z) for flexure.
- Jacketing of columns, joints and plastic hinge zones to achieve either confinement, an increase in flexural strength, or an increase in shear strength, or a combination thereof.

- Post-tensioning joints, columns and girders.
- Footing strengthening through increased footing size, new piles, etc.

That is, all of the upgrading approaches stiffen and strengthen the freeway structures with the exception of the upgrade of the transverse framing in the Terminal Separation Viaduct. For these frames, installation of energy dissipators has been proposed to protect the nonductile frame elements by reducing the displacement response of the structure.

Column Jacketing

Steel jacketing of rectangular columns is proposed by all of the consultants. The objectives of jacketing the columns vary from viaduct to viaduct and include:

- Improving the inelastic response of the critical regions by substantially increasing the ultimate concrete compression strains through confinement.
- Preventing buckling of the longitudinal column reinforcement.
- Increasing the flexural and shear strength of the concrete section.

The effectiveness of the steel jackets, in particular the elliptical jackets, in the critical regions to resist column shear and flexure is dependent upon the capacity of the jacket being developed at the cross-section under consideration.

One detail used by three of the consultants involves terminating the column jacket short (1/2" to 2") of the girder/joint and creating a horizontal plane of weakness in the column between the relatively stiff joint region and jacketed column. Damage to the column during long-duration earthquake shaking will tend to be concentrated in this plane of weakness and in the surrounding concrete. Shear failure (sliding or diagonal tension) initiating from the jacket termination location must be precluded. Both of these issues require careful consideration because they could profoundly affect the inelastic response of the Viaducts.

Column-to-Girder Joint Jacketing/Post-Tensioning

Several of the San Francisco Freeway Viaducts suffered damage in the column-to-girder (knee) joint regions. Jacketing and/or post-tensioning of the knee joint region is proposed by all six consultants. The objectives of jacketing/post-tensioning the joint regions should be:

- Improve the shear capacity of the concrete in the joint region by introducing a triaxial compression field in the joint region through post-tensioning.
- Improve the anchorage of the girder reinforcement (especially the bottom reinforcement that is curtailed in both the span and the joint region) in the joint, and to reduce possible yield penetration into the joint by introducing a biaxial compression field in the girder near the column-girder interface.
- Reduce the shear stresses in the joint region through the addition of steel side plates that are post-tensioned to the column.

The frame joint regions are critical components in the lateral load resisting system. The typical failure mode of knee joints such as these is shear. Because shear is a nonductile failure mechanism, these joints should be designed using capacity design procedures based on the maximum forces that can be delivered to the joint by the beams and columns.

Shear Key Strengthening

The initiation of collapse of most of the Cypress Viaduct bents can be traced directly to cracking that emanated from the shear key atop the pedestals above the lower roadway bent cap. Although the locations of the shear keys in the six San Francisco Freeway Viaducts are typically directly above or below the bent caps, the shear keys remain a plane of weakness in the frames. The upgrading scheme must provide for the adequate strengthening of the shear keys. The new detail must be capable of developing the shear force associated with the flexural capacity of the column, and the hinge deformation associated with the maximum inelastic response of the entire frame.

Conclusions

The repair and seismic retrofit of the viaducts is already underway. Retrofitting is expected to substantially increase the strength of the columns, but the precise degree of improvement in seismic resistance of the structures is not clear. The above discussion focuses on principles of design, not on the specifics of whether the approaches proposed for the Viaducts are ones that will yield adequate seismic performance in future earthquakes.

The Board of Inquiry is unable to evaluate the particular details of the retrofit designs and programs for the individual viaducts without performing detailed studies, but considers them to be only short-term approaches to repair. The materials provided to the Board do not present a compelling case that the procedures for upgrading the elements are necessarily a long-term solution to the seismic deficiencies of these structures. Evaluation of the appropriateness of the retrofits requires close examination of the specific characteristics of each design and consideration of many factors that could influence its performance.

Caltrans has appointed an Independent Review Committee to review the repair and seismic retrofitting of these structures as proposed by the six consultants. The Board of Inquiry defers to their judgements on the specifics of the design instructions and the designs themselves. The Board has confidence that the Committee's technical judgements will be well informed and appropriate. Caltrans should keep the

Committee informed on all aspects of the six retrofit projects—their design and construction and the results of any relevant tests—and should request their advice and recommendations on the projects. The Committee should prepare a report giving its assessment of the repair and retrofitting undertaken as a short-term solution and how it relates to the long-term seismic performance of the viaducts. The Board of Inquiry urges the Committee to pay special attention to the appropriateness of column and joint jacketing schemes, the strengthening of the shear keys, and to consideration of the overall seismic response of the structures.

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Appendix

Testimony and Presentations Received by the Board Of Inquiry at Seven Public Hearings

NOVEMBER 28, 1989 — Sacramento

Caltrans

Robert K. Best
Director

James Roberts
Chief, Division of Structures

James H. Gates
Structural Mechanics Engineer
Division of Structures

Department of Conservation, Division of Mines and Geology

Joseph I. Ziony
Assistant Director

Anthony Shakal
Supervising Geologist for Earthquake Engineering

DECEMBER 14, 1989 — Oakland

Caltrans

James Roberts
Chief, Division of Structures

James H. Gates
Structural Mechanics Engineer
Division of Structures

Other testimony received

Abolhassan Astaneh
Associate Professor of Civil Engineering
University of California, Berkeley

Vitelmo V. Bertero
Professor of Civil Engineering and Director, Earthquake Engineering Research Center
University of California, Berkeley

Richard Hendricks
Design Chief
Alameda County Department of Public Works

Jack Moehle
Associate Professor of Civil Engineering
University of California, Berkeley

J. David Rogers
Principal
Rogers/Pacific, Inc.

Paul Veisze
Consultant
Hammon, Jensen, Wallen, and Associates

JANUARY 4, 1990 — Burlingame

Caltrans

James Roberts
Chief, Division of Structures

James H. Gates
Structural Mechanics Engineer
Division of Structures

Robert N. McDougald
Chief, Maintenance Operations Branch
District 4

San Francisco Bay Area Rapid Transit District

Matthew M. McDole
Manager of Engineering

Mark Chiu
Supervising Structural Engineer

San Francisco Bay Conservation and Development Commission

William Travis
Deputy Director

Joseph P. Nicoletti
Vice-Chair
Engineering Criteria Review Board

U.S. Geological Survey

James Dietrich
Project Chief

Thomas Hanks
Project Chief

Other testimony received

Joseph J. Litehiser, Jr.
Secretary
Seismological Society of America

Ronald Mayes
Vice President
Computech Engineering Services, Inc.

Dan Mohn
Chief Engineer
Golden Gate Bridge District

JANUARY 17-18, 1990 — Pasadena

Caltrans

James Roberts
Chief, Division of Structures

James H. Gates
Structural Mechanics Engineer
Division of Structures

City of Los Angeles

Karl Deppe
Chief Engineer
Earthquake Division

Robert Horii
Department of Building & Safety

JANUARY 17-18, 1990 — Pasadena *continued*

Structural Engineers Association of Southern California

Edwin H. Johnson
President, Structural Engineers Association of Southern California
Vice-Chairman
Atkinson, Johnson & Spurrier, Inc.

Earl L. Pitkin
Consulting Civil & Structural Engineer

Per T. Ron
Vice President
Johnson, Nielson & Associates

Other testimony received

Ahmed M. Abdel-Ghaffar
Professor of Civil Engineering
University of Southern California

Clarence R. Allen
Professor of Geology & Geophysics
California Institute of Technology

M. J. Nigel Priestley
Professor of Structural Engineering
University of California, San Diego

Ronald F. Scott
Professor of Civil Engineering
California Institute of Technology

Lawrence G. Selna
Professor of Civil Engineering
University of California, Los Angeles

Nabih Youssef
Director of Structural Engineering
Albert C. Martin & Associates

FEBRUARY 8, 1990 — Sacramento

Caltrans

James Roberts
Chief, Division of Structures

James H. Gates
Structural Mechanics Engineer
Division of Structures

Ray J. Zelinsky
Senior Bridge Engineer
Division of Structures

California Highway Patrol

D. O. Helmick
Deputy Commissioner

Sgt. Rick James
Investigator

Sgt. Tom Shelton
Chief Investigator

State Board of Registration for Professional Engineers

Al Blaylock
President

Darlene Atkinson Stroup
Executive Director

FEBRUARY 8, 1990 — Sacramento *continued*

Other testimony received

Mary Bufkin
U. S. General Accounting Office

Vernon H. Persson
Chief, Division of Safety of Dams
California Department of Water Resources

MARCH 1-2, 1990 — Burlingame

Caltrans

James Roberts
Chief, Division of Structures

James H. Gates
Structural Mechanics Engineer
Division of Structures

Adlai F. Goldschmidt
Senior Engineering Geologist
Transportation Laboratory

Kenneth A. Jackura
Senior Materials and Research Engineer
Transportation Laboratory

David G. Heyes
Senior Engineering Geologist
District 4

Presentations by Caltrans consultants for San Francisco retrofit projects

T. Y. Lin International, San Francisco

Charles Seim
Principal Engineer

Esmond Chan
Project Manager

Mark Ketchum
Project Manager

Ron Zimmerman
Senior Engineer

Keith Bull
Dames & Moore (subcontractor)

J. P. Singh
Geospectra Consultants (subcontractor)

Bechtel Corporation, San Francisco

Karl Weidner
Engineering Manager

Wen S. Tseng
Principal Engineer

CH2M Hill, Emeryville

Gordon Elliott
Principal Engineer

DeLeuw, Cather & Company, San Francisco

Tom Barron
Vice President

Greg Orsilini
Structural Engineer

MARCH 1-2, 1990 — Burlingame *continued*

Ray Wong
Chief, Structural Engineering

Parsons Brinckerhoff, San Francisco
Jack Belvedere
Vice President

Tudor Engineering, San Francisco
Tom O'Neil
Vice President

Jean-Michel Benoit
Chief, Bridges and Structures
Morrison-Knudsen Engineers, Inc. (subcontractor)

Lawrence Livermore National Laboratory
Roger Werne
Associate Director for Engineering
Applied Mechanics Department

Gerald Goudreau
Applied Mechanics Department

Other testimony received

William Holmes
Member, Board of Directors
Structural Engineers Association of California

James R. Libby
President
James R. Libby and Associates
Consulting Structural Engineers

Jack Moehle
Associate Professor of Civil Engineering
University of California, Berkeley

Loring A. Wyllie, Jr.
Chairman of the Board
H. J. Degenkolb Associates

Peter Yanev
Vice President
EQE Engineering

MARCH 15, 1990 — Sacramento

Caltrans

James Roberts
Chief, Division of Structures

Martin Kiff
Budget Officer

Presentations by Caltrans consultants for San Francisco retrofit projects

DeLeuw, Cather & Company, San Francisco
Tom Barron
Vice President

Greg Orsilini
Structural Engineer

MARCH 15, 1990 — Sacramento *continued*

Tudor Engineering, San Francisco
Tom O'Neil
Vice President

Jean-Michel Benoit
Chief, Bridges and Structures
Morrison-Knudsen Engineers, Inc.
(subcontractor)

Other testimony received
David Cabrera
Principal Program Budget Analyst
California Department of Finance

Greg Kay
Lawrence Livermore National
Laboratory
Applied Mechanics Department

Dale A. Schauer
Group Leader for Applied
Mechanics
Lawrence Livermore National
Laboratory

L. Thomas Tobin
Executive Director
California Seismic Safety
Commission

Annotated Bibliography

The references listed below are those that were sent or made available to all of the Members of the Board of Inquiry. The list also includes reference material consulted by Board Members in preparation of this Report. Catalogued material presented to the Board and carrying bracketed State archival catalog numbers are available for inspection at the Office of Planning and Research, 1400 Tenth Street, Sacramento, California.

Altshuler, E. *B.A.R.T. Civil & Structural Design Criteria Update.* Bay Area Rapid Transit District, material submitted to the Board of Inquiry and dated September 30, 1983 [VAR030].

Synopsis: Revisions to BART design criteria.

American Association of State Highway and Transportation Officials (AASHTO). *Standard Specifications for Highway Bridges, 1977-1989.* American Association of State Highway and Transportation Officials, excerpts from various years from 1977 through 1989 [CT004].

Synopsis: Material provided by Caltrans and excerpted from Standard Specifications for Highway Bridges, adopted by the American Association of State Highway and Transportation Officials.

American Association of State Highway Officials/American Association of State Highway and Transportation Officials (AASHO/AASHTO). *Standard Specifications for Highway Bridges, 1949-1975.* American Association of State Highway Officials/American Association of State Highway and Transportation Officials, excerpts from various years from 1949 through 1975 [CT003].

Synopsis: Material supplied by Caltrans and excerpted from Standard Specifications for Highway Bridges, adopted by the American Association of State Highway Officials (through 1961) and American Association of State Highway and Transportation Officials.

American Institute of Architects, San Francisco Chapter. *The Embarcadero: Addressing the Issues.* February 15, 1990 [VAR051].

Synopsis: A proposal to the City and County of San Francisco that offers an alternative to repair of the Embarcadero freeway, suggesting that a surface parkway is a feasible means for meeting long-term goals of the city.

American Society of Civil Engineers. *Quality in the Constructed Project: A Guideline for Owners, Designers and Constructors.* Volume 1 (excerpts), American Society of Civil Engineers [VAR045].

Synopsis: Manual for practicing professional engineers; portions excerpted for Board members were Sections 13.6-13.9, concerning project peer reviews.

Applied Technology Council. *Tentative Revisions for the Development of Seismic Regulations for Buildings.* Report No. ATC-3-06, Applied Technology Council, April 1989.

Applied Technology Council. "Seismic Resistance of Highway Bridges," *Proceedings of Second Joint U.S.-New Zealand Workshop.* Applied Technology Council, 1986 [PAS009].

Synopsis: Contains about 30 papers related to seismic resistance of highway bridges. Papers were presented at the second bilateral workshop held in San Diego, California, May 8-10, 1985. The workshop was attended by 12 experts from New Zealand and more than 40 participants from the United States. The report contains recommendations for future research and summaries of research projects being conducted in both countries.

Applied Technology Council. *Seismic Design Guidelines for Highway Bridges*. Applied Technology Council, October 1981, revised June 1986 [PAS007].

Synopsis: Guidelines for the seismic design of highway bridges. The guidelines are the recommendations of a team of nationally recognized experts, composed of consulting engineers, academicians, and state and federal highway engineers from throughout the United States. These guidelines incorporate new research findings since the 1973 Caltrans provisions and the 1975 AASHTO specifications. An extensive commentary documenting the basis for the guidelines and an example illustrating their use are included. These guidelines were used to redesign twenty-one existing bridges to increase seismic safety. Bridges were redesigned for four different acceleration coefficients (0.1, 0.2, 0.3, 0.4). Redesigns indicate an average cost increase of 6.3%, with several bridges showing no increase, and a few bridges indicating cost increases of as much as 45%.

Applied Technology Council. *Seismic Retrofitting Guidelines for Highway Bridges*. Report No. FHWA-RD-83-007 and ATC-6-2, U. S. Department of Transportation, Federal Highway Administration, December 1983.

Applied Technology Council. *Seismic Retrofit Guidelines for Highway Bridges*. Applied Technology Council, August 1983 [PAS010].

Synopsis: Guidelines for the seismic retrofitting of highway bridges recommended by a team of nationally recognized experts. The guidelines include preliminary screening procedures, methods for evaluating an existing bridge in detail including the calculation of seismic capacity/demand ratios, and potential retrofitting measures for the most common seismic deficiencies. Special design requirements for various retrofitting measures are also provided. An extensive commentary documenting the basis for the guidelines and an example problem illustrating their use are included. The retrofitting concepts cover prevention of failures of bearings and expansion joints, reinforced concrete columns, piers, and footings, abutments and mitigation of liquefaction and soil movement.

Applied Technology Council. *Comparison of United States and New Zealand Seismic Design Practices for Highway Bridges*. Applied Technology Council, August 1982 [PAS008].

Synopsis: Material presented at a three-day workshop, "Seismic Design of Highway Bridges," held in Wairakei, New Zealand. The participants included 16 members from the United States and 35 from New Zealand. The report also contains research recommendations. The presentations at the workshop consisted of summaries of all aspects and innovative procedures used in New Zealand design practice, comparisons of United States and New Zealand design practices, and the comments and observations of the U.S. team.

Applied Technology Council. *Earthquake Resistance of Highway Bridges, Proceedings of a Workshop*. Applied Technology Council, November 1979 [PAS011].

Synopsis: Twenty-three papers related to earthquake resistance of highway bridges that were presented at a workshop held in San Diego, California, during January 29-31, 1979. The proceedings also contain recommendations for future research.

Applied Technology Council. *Retrofitting Existing Bridges*. Report No. ATC-6-1, Working Group 4, proceedings of a conference on earthquake resistance of highway bridges, San Diego, CA, January 29-31, 1979.

Astaneh, Abolhassan. "Addendum to: Damage to the Bay Bridge Caused By the October 17, 1989 Loma Prieta Earthquake: A Preliminary Report to the Board of Inquiry." Testimony presented to the Board of Inquiry January 4, 1990 [VAR019].

Synopsis: Oral responses to specific questions asked by Board members during A. Astaneh's presentation to the Board of Inquiry on December 14, 1989.

Astaneh, Abolhassan and William MacCracken. "Damage to the Bay Bridge Caused By the October 17, 1989 Loma Prieta Earthquake: A Preliminary Report to the Board of Inquiry." Testimony presented to the Board of Inquiry December 14, 1989 [VAR008].

Synopsis: Brief summary of damage to the Bay Bridge. A brief discussion of the bridge and damage to Pier E-9, the areas between E-17 and E-23, and damage to seismic restrainers.

Astaneh, Abolhassan, Vitelmo V. Bertero, Bruce A. Bolt, Stephen A. Mahin, Jack P. Moehle, and Raymond B. Seed. *Preliminary Report on the Seismological and Engineering Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake*, Report No. UCB/EERC-89/14, University of California, Berkeley, October 1989 [UC001].

Synopsis: Preliminary report on the Loma Prieta earthquake primarily by Professors Astaneh, Bertero, Bolt, Mahin, Moehle, and Seed from the University of California at Berkeley. Topics covered include: seismological and geotechnical considerations of the earthquake, preliminary observations on the performance of concrete freeway structures, initial observations on the damage to the San Francisco Bay Bridge, and preliminary observations on the performance of buildings.

Bachtold, Burch C. and Robert N. McDougald. "Bay Bridge Pavement Cracking and Sand Boils." Set of slides accompanying testimony presented to the Board of Inquiry March 2, 1990 [CT056].

Synopsis: Set of 4 slides that show views of Bay Bridge approaches, highlighting pavement cracking, extrusions, and sand boils.

Bachtold, Burch C. "Memorandum to Alan R. Pendleton, Executive Director, San Francisco Bay Conservation & Development Commission." Material submitted to the Board of Inquiry and dated February 9, 1990 [CT053].

Synopsis: In response to a letter from BCDC, Bachtold, District Director of Caltrans District 4, invites participation by the BCDC Engineering Criteria Review Board in planning an in-depth study of earthquake performance of the San Francisco-Oakland Bay Bridge.

Barron, T. E. "I-280 Southern Viaduct (North): Discussion of Seismic Retrofit Strategy." DeLeuw, Cather & Company, testimony presented to the Board of Inquiry March 2, 1990 [CT075].

Synopsis: Report of presentation made to the Board of Inquiry explaining the retrofit strategy employed by the consultant for the I-280 Southern Viaduct, computer modeling assumptions, and fully elastic model results (without retrofit). The presentation includes drawings of retrofit details, slides of bent damage, and an artist's rendering of the completed retrofit.

Barron, T. E. "Presentation by DeLeuw, Cather & Company." DeLeuw, Cather & Company, testimony presented to the Board of Inquiry March 15, 1990 [CT074].

Synopsis: Summary of seismic benchmark study of the Southern Viaduct, for the purpose of checking the reliability of the model used by the project consultant. The conclusion reached by the consultant is that the model did indeed predict damage where it actually occurred during the Loma Prieta earthquake.

Bay Area Rapid Transit District. "Transbay Tube Earthquake Performance, October 17, 1989." Testimony presented to the Board of Inquiry January 4, 1990 [VAR026].

Synopsis: Copies of slides and overhead projector plates presented to the Board of Inquiry.

Bay Area Rapid Transit District. *B.A.R.T. Civil & Structural Design Criteria for Aerial and Underground Structures.* Bay Area Rapid Transit District, undated [VAR031].

Synopsis: Complete sections from the BART Design Criteria Manual on aerial structures, cut and cover subway structures, and earthquake design criteria for subways.

Bayol, Greg. "Symbol of Strength: Repair of the Bay Bridge," *Going Places.* California Department of Transportation, January/February 1990 [VAR078].

Synopsis: This article, written by Caltrans' District 4 Public Affairs Officer, describes the work of Caltrans District 4 engineers and maintenance crews in the repair of the Bay Bridge following the Loma Prieta earthquake.

Bechtel Corporation. "Testimony to the Board of Inquiry on Retrofit Plans for the San Francisco Terminal Separation Viaduct." Testimony presented to the Board of Inquiry March 2, 1990.

Beck, James L., Theodore M. Christensen, Allen Ely, M. J. Nigel Priestley, Ronald F. Scott, Frieder Seible, and Lawrence G. Selna. *Independent Seismic Design Review of the H.O.V. Viaduct No. 2, I-110 Harbor Project 15.* California Department of Transportation, December 19, 1989 [CT052].

Synopsis: Independent Review Panel's analysis of Caltrans design and Contractor's alternative for Harbor Freeway bridges and recommendations for changes to minimize damage in an "extreme-event" earthquake.

Benoit, Jean-Michel. "Central Viaduct, Loma Prieta Earthquake Seismic Benchmark Study." Tudor Engineering and Morrison-Knudsen Engineers, Inc., material submitted to the Board of Inquiry and dated March 14, 1990 [CT073].

Synopsis: Summary of seismic benchmark study of the Central Viaduct, for the purpose of checking the reliability of the model used by the project consultant. The conclusion reached by the consultant is that the model did indeed predict damage where it actually occurred during the Loma Prieta earthquake.

Benoit, Jean-Michel. "Central Viaduct Analysis." Morrison-Knudsen Engineers, Inc., testimony presented to the Board of Inquiry March 2, 1990 [CT065].

Synopsis: Outline of presentation to Board of Inquiry by Morrison-Knudsen Engineers, a subcontractor to Tudor Engineering for their retrofit planning for the Central Freeway Viaduct.

Bertero, Vitelmo V. "Testimony to the Board of Inquiry on Damage Assessment of the Cypress Viaduct." Testimony presented to the Board of Inquiry December 14, 1989.

Best, Robert. "Presentation graphics on Embarcadero freeway repair alternatives, for Senate Transportation Committee hearing in San Francisco." February 16, 1990 [CT054].

Synopsis: Indicates cost, operational considerations, time to complete, and schematic plans for four alternatives.

Boore, D. M., L. Seekins, and W. B. Joyner. "Peak Accelerations from the 17 October 1989 Loma Prieta Earthquake," *Seismological Research Letters*. Seismological Society of America, Vol. 60, No. 4, pp. 151-166, October-December, 1989.

Borcherdt, R. D., J. F. Gibbs, and K. R. Lajoie. *Maps Showing Maximum Earthquake Intensity Predicted in the Southern San Francisco Bay Region, California, for Large Earthquakes on the San Andreas and Hayward Faults*. USGS Miscellaneous Field Studies Map MF-709, scale 1:125,000, United States Department of the Interior, U. S. Geological Survey, 1975.

Borcherdt, R. D. "Effects of Local Geology on Ground Motion Near San Francisco Bay," *Bulletin of the Seismological Society of America*. Seismological Society of America, Vol. 60, No. 1, pp. 29-61, February 1970.

Brady, A. G. and P. N. Mork. *Loma Prieta, California, Earthquake: Processed Strong Motion Records*. Volume I, Open File Report 90-XXX (pre-release draft), United States Department of the Interior, Geological Survey, February 1990 [USG008].

Synopsis: Draft report of sets of computer-processed strong motion data records of the earthquake on October 17, 1989, recorded at stations in the USGS Cooperative Network of permanent stations.

Brown, Harold, Chairman, Los Angeles County Earthquake Commission. *San Fernando Earthquake, February 9, 1971*. County of Los Angeles, November 1971 [VAR039].

Synopsis: Reflects the views of the Los Angeles County Earthquake Commission that was appointed by the Los Angeles County Board of Supervisors to examine what happened during the earthquake, to assemble facts, to draw conclusions and make recommendations as to what actions could and should be taken in advance of future earthquakes to minimize casualties, physical effects, and disruptions. The Commission met about 25 times and received testimony from and examined more than 35 witnesses. Dr. Housner was one of the seven members. The recommendation for highway structures were: "Present standards of earthquake for highway bridges and other roadway structures should be revised and improved to conform with the current state of knowledge of earthquake engineering and should provide sufficient resistance to survive very strong shaking without collapse."

Buckle, I. G., Ronald L. Mayes, and M. R. Button. *Seismic Design and Retrofit Manual for Highway Bridges*. Report No. FHWA-IP-87-6, U. S. Department of Transportation, Federal Highway Administration, May 1987 [PAS012].

Synopsis: A comprehensive manual for seismic design and retrofit of highway bridges written especially for beginners. This guide is based on seismic design guidelines by Caltrans (1985), the Applied Technology Council (ATC-6, 1981), and AASHTO (1983). The emphasis is on short- and medium- span bridges that are typical of current design practice throughout the United States. It presents the basic principles of seismology, structural dynamics, and structural form as they relate to highway bridge structures. Seismic design concepts are presented that highlight the importance of simplicity, symmetry, and integrity of bridges. Examples of acceptable structural form and form to be avoided are included. A methodology for seismic retrofitting is also presented, which includes discussion of bridge evaluation procedures, capacity/demand ratios, and design concepts for strengthening and upgrading existing bridges. The guide emphasizes that the bridge design philosophy in the U.S. is based on the use of Response Modification Factors. Several design examples are presented. Abutment design procedure, along with examples, are presented in an appendix.

Cabrera, David. "Priority Setting Process: Caltrans Operations and Capital Projects." State of California, Department of Finance, Testimony presented to the Board of Inquiry March 15, 1990 [VAR064].

Synopsis: Compilation of various state budget information with regard to Caltrans, including a calendar for the budget cycle, an overview of how budget priorities are set, a summary of recent Caltrans budget requests for research and earthquake activities, an explanation of revenue adjustments related to possible passage of Propositions 108 and 111, proposed budget adjustments assuming passage of Propositions 108 and 111, and schematics of the transportation planning and programming process, prepared by Caltrans.

California Transportation Commission. "Evaluation of the Proposed Department of Transportation Budget (fiscal years 1987-88 through 1989-90)." California Transportation Commission, March 23, 1989 [VAR033].

Synopsis: Annual report by the California Transportation Commission analyzing the Caltrans capital outlay budget proposal.

Celebi, Mehmet, Erdal Safak, A. Gerald Brady, Richard Maley, and Vahid Sotoudeh. *Integrated Instrumentation Plan for Assessing the Seismic Response of Structures—A Review of the Current USGS Program.* USGS Circular 947, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1987 [USG002].

Synopsis: Discusses procedures to be followed in instrumenting a structure to obtain maximum structural response data with minimum number of instruments. Three examples of buildings instrumented are discussed. Two case studies of data derived from the 1979 Imperial Valley earthquake are also discussed. Appendix D is a draft scheme for instrumenting the San Francisco-Oakland Bay Bridge.

CH2M Hill. "Testimony to the Board of Inquiry on Retrofit Plans for the San Francisco China Basin Viaduct." Testimony presented to the Board of Inquiry March 2, 1990.

Chai, Y. H., M. J. N. Priestley, and F. Seible. *Computer Program on Strength and Ductility of Circular Bridge Columns.* University of California, San Diego, January 11, 1990 [UC002].

Synopsis: Describes the theory and use of a computer program COLRET developed at the University of California at San Diego. The program calculates the strength and ductility of circular bridge columns under seismic loading. The program also calculates the capacity of the column after retrofit with an oversized steel casing with grout in-fill.

Cluff, Lloyd S., Chairman, Seismic Safety Commission. *California At Risk: Reducing Earthquake Hazards, 1987-1992.* Report SSC 89-02, State of California, Seismic Safety Commission, September 1, 1989 [VAR067].

Synopsis: 1989 annual report on the implementation of the California Earthquake Hazards Reduction Act of 1986. The Seismic Safety Commission works with representatives of 43 state agencies to refine and implement the program's initiatives.

Cohen, David, Doug Menuez, and Ron Grant Tussy, Editors. *Fifteen Seconds.* The Tides Foundation, 1989 [VAR016].

Synopsis: A compilation of black and white and color photographs related to the Loma Prieta earthquake. Includes such photos as faces of the victims, rescue effects, damage to buildings, bridges, freeways, inspection by the political authorities, news coverage, etc.

Converse, Frederick J. "Dynamic Tests on San Francisco Bay Mud," *Trans-Bay Tube Technical Supplement to the Engineering Report, Appendix B*_Prepared for Bay Area Rapid Transit District, Parsons, Brinckerhoff-Bechtel-Tudor, Consulting Engineers, June 21, 1960 [VAR029].

Synopsis: Summary of dynamic tests made on San Francisco Bay mud for the design of the BART trans-bay tube.

Dames & Moore Consultants. *The October 17, 1989 Loma Prieta Earthquake*. Dames & Moore, 1989 [VAR015].

Synopsis: Brief coverage of various aspects of Loma Prieta earthquake, especially the geotechnical aspects of the earthquake.

Davis, James F., John H. Bennett, Glenn A. Borchardt, James E. Kahle, Salem J. Rice, and Michael A. Silva. *Earthquake Planning Scenario for a Magnitude 8.3 Earthquake on the San Andreas Fault in the San Francisco Bay Area*. Special Publication 61, California Department of Conservation, Division of Mines and Geology, 1982 [DMG006].

Synopsis: A detailed planning document for a magnitude 8.3 earthquake on the San Andreas Fault in the San Francisco Bay Area, similar to the 1906 event, to portray the probable consequences of catastrophic earthquakes. It contains a map showing isoseismal areas and areas where ground failure may occur. Also contains map of the Bay area showing major highways and airports that would be damaged. It predicts no damage to Golden Gate and Bay Bridges, except damage to northern approach to Golden Gate and eastern and western approaches to Bay Bridge. However, major damage to San Mateo Bridge and its eastern approach is predicted. No major damage to I-280, Embarcadero and Nimitz Freeways are projected. A fatality count between 3,000 to 11,000 and a property loss of about $40 billion are projected.

Degenkolb, O. H. *Retrofitting of Existing Highway Bridges Subject to Seismic Loading—Practical Considerations*. Report No. ATC-6-1, proceedings of a conference on earthquake resistance of highway bridges sponsored by the Applied Technology Council, San Diego, CA, January 29-31, 1979.

Degenkolb, Oris. "Retrofitting Techniques for Highway Bridges," *Bridge Notes*. California Department of Transportation, Vol. 20, No. 1, August 1978 [CT019].

Synopsis: Describes the retrofitting work by Caltrans following the 1971 San Fernando earthquake. Topics discussed are prioritizing retrofitting work, hinge and bearing restrainers, restrainer details, installation of restrainers and column retrofitting concepts.

DeLeuw, Cather & Company. "Testimony to the Board of Inquiry on Retrofit Plans for the San Francisco Southern Freeway Viaduct." Testimony presented to the Board of Inquiry March 2, 1990.

Dieterich, J. H. "The Loma Prieta Earthquake: Implications for Future Earthquakes and Hazard Mitigation," *EOS*. American Geophysical Union, Vol. 71, No. 8, p. 271, February 20, 1990.

Dillon, Richard, Thomas Moulin, and Don DeNevi. *High Steel*. Celestial Arts, 1979.

Synopsis: Historical account of the conception, planning, and construction of the San Francisco-Oakland Bay Bridge and the Golden Gate Bridge. Many historical photographs and drawings are included. The authors state that the bridges were designed to serve the transportation needs of the area while being flexible enough to withstand major earthquakes.

Diridon, Rod, Chairman, Metropolitan Transportation Commission. *Year 2005 Bay Area Travel Forecasts, Technical Summary*. Metropolitan Transportation Commission, 1988.

Drago, Jim. "Back In Business: The Bay Area Rebounds from the Devastating Loma Prieta Earthquake," *Going Places*. California Department of Transportation, January/February 1990 (A) [VAR077].

Synopsis: This article, written by Caltrans' Public Information Officer, commends the work of Caltrans crews in repairing earthquake damaged transportation facilities following the Loma Prieta earthquake and the speed with which the Bay Area's transportation network was restored to operations. It is followed by another related article by Lisa Covington describing special efforts made by various offices and divisions within Caltrans after the earthquake.

Drago, Jim. "Field Lab: Caltrans and the University of California Use a Section of the Cypress Viaduct to Test Earthquake Retrofitting Concepts," *Going Places*. California Department of Transportation, January/February 1990 (B) [VAR079].

Synopsis: This article, written by Caltrans' Public Information Officer, describes the field testing procedures used by Caltrans and University of California researchers on an undamaged section of the Cypress Viaduct structure. The purpose was to test several column retrofit concepts that might then be employed on similar double-deck structures in the Bay Area. The article states that the tests confirmed Caltrans' belief that the planned retrofit techniques would give added strength to the Bay Area structures.

Earthquake Engineering Research Center. *Preliminary Report on the Seismological and Engineeirng Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake.* Report No. 89-14, University of California, Berkeley, October 1989 .

Synopsis: This is a preliminary report on engineering and seismology prepared by faculty of the University. It covers all engineering aspects of the earthquake, including the Cypress Viaduct and Bay Bridge, performance of engineered buildings, and site failure and liquefaction. It reviews preliminary seismological and geological issues. Preliminary lessons learned are presented.

Earthquake Engineering Research Institute. *Preliminary Reconnaissance Report, Loma Prieta Earthquake, October 17, 1989.* Report No. 89-03, Earthquake Engineering Research Institute, November 1989 [VAR007].

Synopsis: This is a preliminary report, containing about 50 pages, by the reconnaissance team of scientists and engineers sent by the Earthquake Engineering Research Institute (EERI) to investigate the effects of the Loma Prieta earthquake. It describes geological and seismological aspects of the quake, and identifies numerous locations where liquefaction, ground settlement and landslides occurred. Damage to transportation systems is discussed only briefly and only in general terms.

Einashai, A., J. Bommer, and A. El-Ghazouli. *The Loma Prieta (Santa Cruz) Earthquake of 17 October 1989: Seismological, Geotechnical and Structural Field Observations.* ESEE Research Report No. 89/11, Imperial College of Science & Technology, London, England, December 1989 [VAR053].

Synopsis: Reconnaissance report on the Loma Prieta earthquake by the research staff of Imperial College of London, England. It has a special appendix on the Cypress viaduct. Linear and nonlinear analyses were performed of a typical bent and concluded that excessive shear forces caused the collapse. The analyses also concluded that the collapse occurred within the first five seconds of the earthquake.

Ellsworth, W. L. "Historical Seismicity," in Wallace, R. E., ed., *The San Andreas Fault System.* USGS Professional Paper 1515, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1990 (in press).

EQE Engineering, Inc. *The October 17, 1989 Loma Prieta Earthquake.* EQE Engineering, Inc., 1989 [VAR006].

Synopsis: A 40-page report that includes several black and white photos of damaged structures. The topics covered include seismicity and geology, commercial structures, industrial facilities, lifelines, transportation, residential structures, and the fires following the earthquake.

Evernden, J. F. "Predicted and Observed Ground Motion for Past and Expected California Earthquakes," EOS. American Geophysical Union, 1990 (in press).

Fairweather, Virginia. "The Next Earthquake," *Civil Engineering.* March 1990 [VAR061].

Synopsis: This article questions whether the money and political backing for needed earthquake retrofit programs, research, and legislative changes will fade as memories of the October 1989 earthquake subside.

Fratessa, Paul F., LeRoy Crandall, Kyle McKinsey, Edwin Johnson, Roy Johnston, Albert Ridley, and John Worsley. *A Review of the California State Prison Construction Program.* State of California, Seismic Safety Commission, April 4, 1988 [VAR069].

Synopsis: Report to the Seismic Safety Commission from the Committee on the State Prison Construction Program, which was established by the Seismic Safety Commission in 1985 to "determine whether the State's prison construction program complies with appropriate seismic safety design and construction standards and practices."

Fung, George G., Richard J. LeBeau, Eldon D. Klein, John Belvedere, and Adlai F. Goldschmidt. *The San Fernando Earthquake, Field Investigation of Bridge Damage.* California Department of Transportation, February 1971 [CT016].

Synopsis: Results of a field investigation of bridge damage in the aftermath of the 1971 San Fernando earthquake. The investigative team consisted of four bridge designers and a geologist. For each damaged bridge, selected drawings and logs of test borings, along with photos showing the damage, are provided. Most of the major damage occurred within the limits of two contracts under construction, namely, the Route 5/210 Interchange and the Route 5/14 Interchange. The Route 5 (Truck Lane)/405 separation structure also suffered severe damage and was demolished. A total of five major bridges were required to be entirely replaced. The report described damage to a total of 25 bridge structures.

Futtrup, Harold A. *Comments on Proceedings of January 17-18, 1990 of Governor's Board of Inquiry.* Material submitted to the Board of Inquiry and dated January 30, 1990 [VAR046].

Synopsis: Thoughts of a Board meeting attendee regarding proceedings of the meeting. The author provides comments and drawings of a suggested alternative to wire rope restrainers and other mechanical devices.

Gates, James H. *Summary of Earthquake Engineering at Caltrans.* California Department of Transportation, February 22, 1990, material submitted to the Board of Inquiry on March 1, 1990 [CT059].

Synopsis: Description of the Structural and Seismic Analysis Unit (SASA) within the Caltrans Division of Structures, including general responsibilities and committee organization.

Gates, James H. *Summary of the Local Bridge Seismic Retrofit Program.* California Department of Transportation, February 7, 1990, material submitted to the Board of Inquiry on February 8, 1990 [CT042].

Synopsis: Summary of plans to inspect approximately 11,229 local highway bridges and retrofit those found deficient, pursuant to SB 36X.

Gates, James H. *Summary of the State Bridge Seismic Retrofit Program.* California Department of Transportation, February 6, 1990, material submitted to the Board of Inquiry on February 8, 1990 [CT041].

Synopsis: Summary of retrofits completed and future plans for expansion joints and bearings, single column bridges, and other state bridges.

Gates, James H. *Caltrans and AASHTO Seismic Design Philosophy.* California Department of Transportation, January 3, 1990 (A), material submitted to the Board of Inquiry on January 4, 1990 [CT031].

Synopsis: Description of design philosophy used by Caltrans since 1971 and pages from Caltrans Bridge Design Specifications and AASHTO Guide Specification.

Gates, James H. *Foundation Data: Embarcadero and Distribution Structures (San Francisco) and Struve Slough (Watsonville).* California Department of Transportation, January 3, 1990 (B), material submitted to the Board of Inquiry on January 4, 1990 [CT032].

Synopsis: Copies of portions of contract plans showing foundation data and foundation locations for the bridge structures.

Gates, James H. *The Loma Prieta Earthquake—Summary Slides.* California Department of Transportation, material submitted to the Board of Inquiry and dated December 12, 1989 [CT017].

Synopsis: Listing and set of 48 slides made immediately after the earthquake by Caltrans Peqit Team engineers, UCSD Professor Priestley, and UCB Professor Astaneh. The slides illustrate damage to the Cypress structure, Bay Bridge, Struve Slough bridges, and the San Francisco double-deck structures. The set includes a map of the region highlighting the areas of greatest damage, views of various bents from each structure, Bay Bridge repair actions, and summary information.

Gates, James H. *Bay Bridge—Summary of 1976 Seismic Retrofit.* California Department of Transportation, December 11, 1989 (A), material submitted to the Board of Inquiry on December 14, 1989 [CT026].

Synopsis: Description and as-built plans of the seismic retrofit installed in 1976 and designed in 1974-75.

Gates, James H. *Cypress Viaduct—Summary of 1977 Seismic Retrofit.* California Department of Transportation, December 11, 1989 (B), material submitted to the Board of Inquiry on December 14, 1989 [CT022].

Synopsis: Summary and drawings of retrofit installed in 1977 and designed in 1974-75 using criteria developed after the 1971 San Fernando Earthquake.

Gates, James H. *Struve Slough Bridge—Description of Damage.* California Department of Transportation, December 11, 1989 (C), material submitted to the Board of Inquiry on December 14, 1989 [CT023].

Synopsis: Summary of damage to Bridge No. 36-88 R/L, built in 1964.

Gates, James H. *Summary of the Expansion Joint Retrofit Program.* California Department of Transportation, December 11, 1989 (D), material submitted to the Board of Inquiry on December 14, 1989 [CT028].

Synopsis: Summary of the Caltrans expansion joint retrofit program (formerly called Phase 1), prepared by Ray Zelinski.

Gates, James H. *Summary of the Single-Column Retrofit Program.* California Department of Transportation, December 11, 1989 (E), material submitted to the Board of Inquiry on December 14, 1989 [CT029].

Synopsis: Summary and prioritization scheme of the Caltrans single-column retrofit program (formerly called Phase 2).

Gates, James H. *Bay Bridge—Summary of Damage.* California Department of Transportation, December 8, 1989 (A), material submitted to the Board of Inquiry on December 14, 1989 [CT027].

Synopsis: Summary and drawings of damage at Pier E9, E17 through E22, and E23.

Gates, James H. *Cypress Viaduct—Summary of Bent Framing Configurations.* California Department of Transportation, December 8, 1989 (B), material submitted to the Board of Inquiry on December 14, 1989 [CT020].

Synopsis: Diagrams describing the bent configurations along the Cypress Viaduct, prepared by Caltrans' Office of Structure Maintenance and Investigations.

Gates, James H. *Cypress Viaduct—Summary of Partial Damage at Bents 74-80, 96-97.* California Department of Transportation, December 8, 1989 (C), material submitted to the Board of Inquiry on December 14, 1989 [CT021].

Synopsis: Photographs and descriptions of partially damaged sections of the Cypress Viaduct.

Gibbs, J. F., T. E. Fumal, and R. D. Borcherdt. *In-Situ Measurements of Seismic Velocities at Twelve Locations in the San Francisco Bay Region.* Geological Survey Open-File Report 75-564, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1975.

Hall, John F. "Structural Behavior During the Loma Prieta Earthquake," *Engineering & Science.* California Institute of Technology, Volume LIII, Number 2, pp. 13-19, Winter 1990 [VAR071].

Synopsis: This article, written by the Technical Secretary to the Board of Inquiry, discusses the nature of the strong shaking generated by the Loma Prieta earthquake and how some particular and typical structures fared. Photographs and sketches are used to illustrate the major points. Emphasis in the article is on structures that were built when earthquake effects were poorly understood. The author states that these structures represent California's most pressing earthquake problem.

Hanks, Thomas C. "Testimony to the Board of Inquiry on Strong Ground Motion During the Loma Prieta Earthquake." Testimony presented to the Board of Inquiry January 4, 1990.

Hanks, Thomas C. *The National Earthquake Hazards Reduction Program—Scientific Status.* USGS Bulletin 1659, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1985 [USG001].

Synopsis: Examines present body of scientific understanding and data bases pertinent to the objectives of National Earthquake Hazards Reduction Program (NEHRP) created under Public Law 95-124. Generally speaking, the USGS has responsibility for the scientific elements of the program, and the NSF has responsibility for the engineering elements. FEMA is charged with planning and coordinating the entire program. The NBS has responsibility for developing building codes.

Helley, E. J., K. R. Lajoie, and D. B. Burke. *Geologic Map of Late Cenozoic Deposits, Alameda County, California.* Map MF-429, United States Geological Survey, 1972.

Helmick, D. O. "A Preliminary Report to the Board of Inquiry on the 1989 Loma Prieta Earthquake Induced Freeway Collapse." State of California, Department of California Highway Patrol, testimony presented to the Board of Inquiry February 8, 1990 [VAR043].

Synopsis: Summary of information obtained from interviews of 160 Bay Bridge failure eyewitnesses and 195 eyewitnesses to the Cypress Viaduct collapse.

Hendrix, Richard. *Letter to the Board of Inquiry Giving Eyewitness Account of Cypress Failure.* Testimony presented to the Board of Inquiry December 14, 1989 [VAR037].

Synopsis: Eyewitness testimony on failure of the Cypress structure; presenter was traveling on the structure when the earthquake struck.

Heyes, David G. "Geology and Soils in the Vicinity of the San Francisco-Oakland Bay Bridge and Cypress Structure Site." California Department of Transportation, testimony presented to the Board of Inquiry March 1, 1990 [CT062].

Synopsis: Text of testimony presented to the Board of Inquiry by Caltrans' District Geologist for District 4.

Hill, D. P., J. P. Eaton, and Lucy Jones. "Seismicity," in Wallace, R. E., ed., *The San Andreas Fault System*. USGS Professional Paper 1515, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1990 (in press).

Hofer, Rudolph, Jr. "Seismic Design Proves Effective in San Francisco," *Modern Steel Construction*. American Institute of Steel Construction, Inc., January-February 1990 [VAR044].

Synopsis: Describes major structural failures during the Loma Prieta earthquake, arguing that the earthquake demonstrated that current seismic designs, particularly of structural steel, performed well.

Holmes, William T. "Address to the Board of Inquiry." Structural Engineers Association of California, testimony presented to the Board of Inquiry March 2, 1990 [VAR060].

Synopsis: Text of a letter read to the Board of Inquiry by Mr. Holmes, representing the Structural Engineers Association. The letter suggests ways in which SEAOC seismic design experience, which has been primarily focused on buildings, could be useful in improving the seismic design of new and retrofitted bridges.

Hough, S. E., P. A. Friberg, R. Busby, E. F. Field, K. H. Jacoby, and R. D. Borcherdt. "Sediment-Induced Amplification and the Collapse of the Nimitz Freeway," Nature. MacMillan Magazines, Ltd., 1990 (in press).

Housner, George W. "Earthquake Considerations in the Design of the Trans-Bay Tube," Trans-Bay Tube Technical Supplement to the Engineering Report (Appendix A). Prepared for Bay Area Rapid Transit District, Parsons, Brinckerhoff-Bechtel-Tudor, Consulting Engineers, May 27, 1960 [VAR028].

Synopsis: Summary of earthquake considerations for the design of the BART Trans-Bay Tube.

Huang, M. J., T. Q. Cao, U. R. Vetter, and A. F. Shakal. *Second Interim Set of CSMIP Processed Strong Motion Records from the Santa Cruz Mountains (Loma Prieta) Earthquake of 17 October 1989*. Report OSMS 90-01, California Department of Conservation, Division of Mines and Geology, February 1, 1990 [DMG007].

Synopsis: Plots of the processed data for an additional 15 records are presented. These consist of uncorrected accelerations, instrument and baseline-corrected accelerations, velocity and displacement plots, response spectra, Fourier amplitude spectra, and absolute acceleration spectra for 0, 2, 5, 10, and 20 percent damping values.

Idriss, I. M. *Response of Soft Soil Sites During Earthquakes*. Proceedings, a memorial symposium to honor Professor H. B. Seed, University of California, Berkeley, May 1990.

Imbsen, R.A. and Joseph Penzien. *Evaluation of Energy Absorption Characteristics of Highway Bridges Under Seismic Conditions*. Volumes I and II, Earthquake Engineering Research Center, University of California, Berkeley, September 1986 [PAS013].

Synopsis: This two-volume report describes the computer program NEABS-II (Nonlinear Earthquake Analysis of Bridge System). The program uses a nonlinear beam-column element for reinforced concrete bridge columns to evaluate energy absorption characteristics using a three-dimensional yield surface. A gapped tension-compression, tie-bar element having bilinear force-displacement relationships was developed to model expansion joints and restrainer cables. To demonstrate the use of the program, three California bridges were analyzed. These bridges are: 1) the Fields Landing Overhead, a four-span reinforced concrete box-girder bridge that suffered major damage in 1980; 2) Jeffrey Road Overcrossing; and 3) the East Connector Overcrossing. These nonlinear analyses were compared with linear elastic analyses. Nonlinear analyses showed responses as much as six times those obtained from the linear analyses. A complete listing of the computer program is provided in Volume II.

Imbsen, R. A. "Seismic Design of Highway Bridges Workshop Manual," *Design and Retrofitting Concepts*. Report No. FHWA-IR-81-2, U. S. Department of Transportation, Federal Highway Administration, January 1981.

Jackura, Kenneth. "Cypress Viaduct Soil Profile." California Department of Transportation, testimony presented to the Board of Inquiry March 1, 1990 [CT060].

Synopsis: Copies of overhead projections used during a presentation to the Board of Inquiry which provided information on soil conditions and liquefaction potential near the Cypress Viaduct.

Jarpe, S. P., L. J. Hutchings, T. F. Hank, and A. F. Shakal. "Selected Strong- and Weak-Motion Data from the Loma Prieta Earthquake Sequence," *Seismological Research Letters*. Seismological Society of America, Vol. 60, No. 4, pp. 167-176, October-December, 1989.

Jennings, Paul C. *Engineering Features of the San Fernando Earthquake of February 9, 1971*. Report No. EERL 71-02, California Institute of Technology, June 1971 [PAS022].

Synopsis: This report contains nine papers prepared by staff and students in earthquake engineering within the Division of Engineering and Applied Science at the California Institute of Technology. George Housner and Paul Jennings authored or coauthored five of the nine papers. The report recommended that the bridge design standards of the day were inadequate and should be revised.

Johnson, E. H. "Statement to the Governor's Board of Inquiry by Representative of the Structural Engineers Association of California." Testimony presented to the Board of Inquiry January 18, 1990 [VAR023].

Synopsis: Commends Caltrans' present design-review process and recommends design review by qualified engineering firms. Comments that prior to 1971, U.S. lateral force levels were much lower than those in Japan. Includes several photos of the damage of Nimitz Freeway.

Joyner, W. B. and D. M. Boone. *Measurement, Characteristics and Prediction of Strong Ground Motion*. Proceedings, Specialty Conference on Earthquake Engineering and Soil Dynamics II—Recent Advances in Ground Motion Evaluation, Park City, Utah, American Society of Civil Engineers, June 1988.

Kay, Greg and Dale A. Schauer. "Embarcadero Structure: Computer Modeling and Engineering Analysis Update." Lawrence Livermore National Laboratory, material submitted to the Board of Inquiry and dated March 15, 1990 [VAR065].

Synopsis: Copies of presentation slides and graphics which summarize the results of an analysis performed by LLNL of the adequacy of the Embarcadero retrofit, based upon information gained from the Cypress test section analysis. The analysis concluded generally that the retrofitted structure would perform better than the unretrofitted structure, but that further study was needed of the beam to column intersection in bents.

Keeley, Fred, Chair, CSAC Earthquake Relief Legislative Task Force. *Loma Prieta Earthquake Unmet Needs Action Plan*. County Supervisors' Association of California, March 8, 1990 [VAR080].

Synopsis: Report by a task force comprised of county supervisor representatives of the ten disaster struck counties to prove unmet need for further assistance, beyond the relief provided by state and federal appropriations following the Loma Prieta earthquake. The conclusion of the task force is that an additional $2.9 billion in public funds will be required to restore the earthquake devastated region. The funding needs cover twelve major categories. The largest single area of need identified is funding for hazard mitigation projects, particularly fire and safety retrofits in public buildings. The combined estimated cost of these projects totals $2.3 billion.

Krawinkler, Helmut. *Preliminary Analysis of Cypress Structure*. Material submitted to the Board of Inquiry and dated January 1990 [VAR049].

Synopsis: Preliminary analysis of a typical bent of the Cypress Viaduct is presented. Static and dynamic analyses of this bent were performed using the "DRAIN-2D" computer program. The Emeryville record was used as input for the dynamic analysis. The analysis showed that seismic loads on the structure exceeded the computed capacity by a large margin. Therefore, cascading was not needed to cause collapse.

Larson, T. D. *Statement from Federal Highway Administration*. U. S. Department of Transportation, Federal Highway Administration, material submitted to the Board of Inquiry and dated January 19, 1990 [VAR038].

Synopsis: Written statement of policy considerations and actions of the Federal Highway Administration with regard to the Loma Prieta earthquake.

Lew, H. S. (Editor). *Performance of Structures During the Loma Prieta Earthquake of October 17, 1989.* NIST Special Publication 778 (ICCSSC TR11), U. S. Department of Commerce, National Institute of Standards and Technology, January 1990 [VAR062].

Synopsis: Report based primarily on the data gathered by a team representing the Interagency Committee on Seismic Safety in Construction, which surveyed the damage to buildings, utilities, and transportation structures immediately after the Loma Prieta earthquake. The team found that most structures designed in accordance with modern codes and standards performed well without serious structural damage. However, there were many concrete and masonry buildings and highway structures in the San Francisco Bay area which were not designed according to modern seismic design codes and which did not perform well. It discusses in detail the collapse of the Cypress Viaduct and Struve Slough Bridge and damages to the Bay Bridge, Interstate 280, Embarcadero Freeway, and U.S. Highway 101. The report presents two dimensional finite element analyses of two bents to obtain mode shapes and fundamental periods. Static stresses are calculated in three different bents. A finite element analysis of a 9 span portion was also performed to obtain vertical and longitudinal periods.

Lin, T. Y., International. "Testimony to the Board of Inquiry on Retrofit Plans for the San Francisco Embarcadero Freeway Viaduct." Testimony presented to the Board of Inquiry March 1, 1990.

Litehiser, Joseph J. "Statement on Behalf of the Seismological Society of America Before the Governor's Board of Inquiry." Testimony presented to the Board of Inquiry January 4, 1990 [VAR020].

Synopsis: Statement and recommendations on behalf of the Executive Committee of the Society.

Longinow, A., R. R. Robinson, and K. H. Chu. *Retrofitting of Existing Highway Bridges Subject to Seismic Loading—Analytical Considerations.* Report No. ATC-6-1, proceedings of a conference on earthquake resistance of highway bridges sponsored by the Applied Technology Council, San Diego, CA, January 29-31, 1979.

Mahin, Stephen, Jack Moehle, and Roy Stephen. *Static Load Tests of the I-880 Cypress Viaduct—Retrofit Phase, Preliminary Letter Report.* December 27, 1989 [CT034].

Synopsis: Preliminary results of field tests on Cypress Viaduct bents using three retrofit schemes. A static cyclical loading maximum of 1/4 of dead weight was applied. Finally, a load equal to dead weight was applied. It was concluded that all schemes strengthened the structure. The results indicate that the schemes are (most to least preferable) as follows: 1) rock bolt retrofit, 2) steel strong-back retrofit, and 3) steel collar retrofit.

Maley, R., A. Acosta, F. Ellis, E. Etheredge, L. Foote, D. Johnson, R. Porcella, M. Salsman, and J. Switzer. *U.S. Geological Survey Strong-Motion Records from the Northern California (Loma Prieta) Earthquake of October 17, 1989.* Geological Survey Open-File Report 89-568, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, October 1989 [USG005].

Synopsis: Provides traces obtained from strong-motion accelerographs at thirty-eight USGS stations located at epicentral distances ranging from 27 to 115 km. Included are records from six extensively instrumented structures—five buildings and one dam. The design characteristics and drawings of the instrumentation schemes for the six structures are also given.

Mancarti, G.D. *New Concepts in Earthquake Retrofitting of Highway Bridges.* State of California, Department of Transportation. August 1978.

Mayes, Ronald, et al. (ATC/EERI/NCEER Bridge Reconnaissance Team). *Reconnaissance Report, Bridge Structures, October 17, 1989 Loma Prieta Earthquake.* Jointly published by Applied Technology Council, Earthquake Engineering Research Institute, and National Center for Earthquake Engineering Research, November 16, 1989 [VAR002].

Synopsis: Report by group of engineering experts who toured bridge structures immediately after the earthquake and made a series of findings and recommendations.

McDole, M. M., and M. M. Chiu. "BART Presentation Slides." Testimony presented to the Board of Inquiry January 4, 1990 (A) [VAR034].

Synopsis: Slides related to testimony regarding observations on the performance of the BART Trans-Bay Tube during and after the earthquake.

McDole, M. M., and M. M. Chiu. "BART Trans-Bay Tube Performance and Related Issues." Testimony presented to the Board of Inquiry January 4, 1990 (B) [VAR018].

Synopsis: Summary of testimony relating to observed performance of the BART Trans-Bay Tube and background on construction of the tube.

McDougald, Robert N. "Structural Damage and Repairs to the San Francisco-Oakland Bay Bridge." Testimony presented to the Board of Inquiry January 4, 1990 [CT033].

Synopsis: Description of damage and repairs to sections of the Bay Bridge following the October 17, 1989 earthquake.

McNutt, Steve. "Loma Prieta Earthquake, October 17, 1989, Santa Cruz County, California," *California Geology*. California Department of Conservation, Division of Mines and Geology, January 1990 [VAR024].

Synopsis: Gives an overview of the Loma Prieta earthquake. Provides a list of California earthquakes, M>6.5, and significant aftershocks. Notes that in Loma Prieta the rupture was 31 miles compared to 1906 San Francisco earthquake rupture of 280 miles long.

Merritt, J. L. *Application of the SATURN Program to the Evaluation of Effects on Ground Modification of Earthquake Shaking with Specific Application to the Response of the Cypress Viaduct*. BDM International, Inc. (White Paper), November 16, 1989 [VAR054].

Synopsis: This document is essentially a proposal for the analysis of structures similar to the Nimitz Freeway (Interstate 880). It specifically proposes to use the SATURN computer program, a three-dimensional finite element program that handles sophisticated problems of "structure-medium interaction," for structures founded in or on natural earth media. It is professed that this is the only "mature, scientifically-qualified and available procedure" for handling the liquefaction problems.

Moehle, Jack P. "Slides from Presentation to Board of Inquiry." Letter dated March 29, 1990 and set of slides presented to the Board of Inquiry March 1, 1990 [VAR082].

Synopsis: Set of 55 slides showing tests performed on Cypress structure after the Loma Prieta earthquake and copies of charts, diagrams, and other presentation materials presented to the Board of Inquiry.

Moehle, Jack P. "Experiments on the Cypress Street Viaduct and Indications Regarding Performance During the 17 October Earthquake." Testimony presented to the Board of Inquiry December 14, 1989 [VAR009].

Synopsis: Working drawings, photographs, and overhead projection summaries of tests performed on the Cypress structure.

Mohn, Daniel E. *Letter concerning performance of the Golden Gate Bridge during the 1989 Loma Prieta Earthquake*. Material submitted to the Board of Inquiry and dated December 20, 1989 [VAR017].

Synopsis: Summary of observed performance of the Golden Gate Bridge, previous retrofits, instrumentation, and studies of seismic design of the bridge.

Munroe, T. *The Economic Impact of the Loma Prieta Earthquake of 1989: A Preliminary Look*. Pacific Gas & Electric Company, 1989 [PAS006].

Synopsis: A brief report on the economic impact of the Loma Prieta earthquake by the chief economist for Pacific Gas & Electric. It is estimated that the physical damage may reach $10 billion, which is only a fraction of a percent of total physical asset base. Income and job losses are only short-term, and the author sees some long-term benefits such as renewed attention to transportation problems.

Naramore, Sharon A. and Fred Y. Feng. *Field Tests of Large Diameter Drilled Shafts: Part I, Lateral Loads*. Report No. FHWA/CA/SD-88-02, California Department of Transportation, in cooperation with the U. S. Department of Transportation, Federal Highway Administration, March 1990 [CT071].

Synopsis: Report of field tests of large diameter drilled shafts under loads, conducted on the Century Freeway (I-105) site. The project objectives were to evaluate the feasibility and cost of using eight foot diameter shafts, and to determine the lateral load capacity.

Nathe, Sarah (editor). "The Once and Future Quake: All About Loma Prieta," *Networks: Earthquake Preparedness News*. Governor's Office of Emergency Services, Bay Area Regional Earthquake Preparedness Project, Volume 5, Number 1, Winter 1990 [VAR075].

Synopsis: This is a special edition of the quarterly newsletter, devoted entirely to the Loma Prieta earthquake. Included are articles on the geology of the earthquake, the damage to structures, the earthquake's effects on institutions and businesses, and its emotional impacts on people.

Nims, D. K., E. Miranda, I. D. Aiken, A. S. Whittaker, and V. V. Bertero. *Collapse of the Cypress Street Viaduct as a Result of the Loma Prieta Earthquake*. Report No. UCB/EERC-89/16, University of California, Berkeley, November 1989 [UC003].

Synopsis: Provides a description of damage to the Cypress Viaduct and design information on the structure, including dimensions of typical bents, typical reinforcement at the joints, and identification of bent types for all bents. It provides bent-by-bent damage description and frequencies and mode shapes of Bents 45 and 46 obtained through ambient vibration measurements after the earthquake. Contains several photographs of damage.

O'Neill, Thomas J. "Material Presented at Governor's Board of Inquiry." Tudor Engineering Company, testimony presented March 2, 1990, materials submitted March 6, 1990 [CT064].

Synopsis: Copies of overhead projections used by Tudor Engineering presenters to describe the scope and objectives of the contract retrofit project for the Central Freeway Viaduct in San Francisco.

Ohashi, M., T. Fujii, E. Kuribayashi, and T. Tazaki. *Inspection and Retrofitting of Earthquake Resistance Vulnerability of Highway Bridges—Japanese Approach*. Report No. ATC-6-1, proceedings of a conference on earthquake resistance of highway bridges sponsored by the Applied Technology Council, San Diego, CA, January 29-31, 1979.

Pacific Aerial Surveys. *Aerial Oblique Mosaic of the Cypress Structure Following the Loma Prieta Earthquake of October 17, 1989*. Hammon, Jensen, Wallen & Associates, photographs taken October 26, 1989 [VAR004].

Synopsis: Black and white aerial photograph using aerial film by Kodak, 9 X 9 Cartographic Camera, 8-1/4-inch focal length by Zeiss.

Pacific Coast Building Officials Conference. *Uniform Building Code, 1946 Edition (excerpts)*. Pacific Coast Building Officials Conference, January 1, 1946 [VAR050].

Synopsis: Excerpts of specifications for lateral bracing from the Uniform Building Code standards in effect at the time of the Cypress Viaduct design and construction.

Parsons, Brinckerhoff. "Testimony to the Board of Inquiry on Retrofit Plans for the San Francisco Alemany Freeway Viaduct." Testimony presented to the Board of Inquiry March 2, 1990.

Parsons, Brinckerhoff-Bechtel-Tudor, Consulting Engineers. "Earthquake Effects - General," *Trans-Bay Tube Technical Supplement to the Engineering Report*. Bay Area Rapid Transit District, July 1960 [VAR027].

Synopsis: Summary of earthquake effects studied relative to the BART Trans-Bay Tube design and construction.

Parsons, Brinckerhoff-Tudor-Bechtel, Consulting Engineers. *Soil Investigation, Transbay Tube*. BART Publication B7200, Bay Area Rapid Transit District, Undated [VAR032].

Synopsis: Complete report on soil investigation for design and construction of BART Trans-Bay Tube.

Patrick, Robert L. *Additional Earthquake Issues*. Letter to Dr. George Housner dated January 23, 1990 [VAR057].

Synopsis: Raises some issues related to seismic safety in California, such as the hazard of chemical plants in the Los Angeles basin, shortfalls in the current building codes, lack of effort to educate homeowners of seismic risks, problems with high-rise buildings, and disruption of business and industry.

Pendleton, Alan R. *Background Information on the Commission and Its Engineering Criteria Review Board (Memorandum Report).* San Francisco Bay Conservation and Development Commission, December 22, 1989, material submitted to the Board of Inquiry January 4, 1990 [VAR003].

Synopsis: Staff memorandum describing the BCDC, its Engineering Criteria Review Board (ECRB), the relevance of the Bay Plan to review of structures, and projects reviewed by the ECRB.

Persson, Vernon. "Presentation Materials to the Board of Inquiry." Testimony presented to the Board of Inquiry February 8, 1990 [VAR041].

Synopsis: Copies of overhead projection presentation material related to testimony.

Persson, Vernon. *Letter to Professor Housner Regarding Consultant Boards.* December 11, 1990 [VAR042].

Synopsis: Description of use of consultant boards by Division of Safety of Dams and listings of composition of example boards.

Pitkin, Earl L. and Per T. Ron. "Report to the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake: Damage to Bridge Structures in Santa Cruz." Testimony presented to the Board of Inquiry January 18, 1990 [VAR021].

Synopsis: Reports that inspection of 15 bridges in the Santa Cruz area showed that they performed well. Color photos of 6 major bridges are provided, however, photo quality is poor.

Plafker, George and John P. Galloway, Editors. *Lessons Learned from the Loma Prieta, California, Earthquake of October 17, 1989.* USGS Circular 1045, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1989 [USG003].

Synopsis: Advances the theme that geologic conditions strongly influence damage, since geology determines where fault ruptures are likely to occur, how hard the ground will shake, where landslides will occur, and where the ground will sink and crack. It also draws parallel between the 1906 and 1989 quakes by noting that the pattern of damage is very similar in both cases.

Priestley, M. J. Nigel. *Independent Seismic Design Review of the HOV Viaduct No. 2—I-110 Harbor Project 15, Preliminary Report to Caltrans.* Material submitted to the Board of Inquiry January 17, 1990 [CT036].

Synopsis: Provides conclusions and recommendations of the independent consultant panel convened by Caltrans to evaluate the seismic capacity of the high occupancy vehicle viaduct (H.O.V.) to be built over the existing Harbor Freeway in Los Angeles. The panel evaluated two designs, one by Caltrans and an alternate design by the contractor and concluded that the proposed structure could survive the Caltrans design level earthquake without collapse. The panel proposed some changes to both designs, but does not indicate any preference for one design over the other.

Priestley, M. J. Nigel and Frieder Seible. *The Central Viaduct, A Damage and Repair Assessment.* Final Report to Caltrans and Federal Highway Administration under Emergency Contract Number 59K978, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, December 3, 1989 (A) [CT038].

Synopsis: Describes the damage to Bents 22, 43, and 48 of the Central Viaduct. Calculation for capacity checks for these bents are also provided. The damage pattern here is noted to be different from those seen at Cypress or Embarcadero. It is estimated that accelerations of about 0.14g would produce the damage observed.

Priestley, M. J. Nigel and Frieder Seible. *The China Basin Viaduct, A Damage and Repair Assessment of Bents No. 32 and N1 35.* Final Report to Caltrans and Federal Highway Administration under Emergency Contract Number 59K978, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, December 3, 1989 (B) [CT039].

Synopsis: Describes the damage to Bents 32 and N135 at China Basin. Provides calculations of dead load stresses, section strengths, lateral capacity, and suggestions for retrofit. The lateral estimated loads necessary to cause the observed damage for Bents 32 and N135 are, respectively, 0.10g and 0.40g.

Priestley, M. J. Nigel and Frieder Seible. *Route 980, Bent #38, A Damage and Repair Assessment.* Final Report to Caltrans and Federal Highway Administration under Emergency Contract Number 59K978, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, November 20, 1989 [CT037].

Synopsis: Describes the damage to Bent 38 of the Interstate 980 southbound connector in Oakland. Calculations are provided for dead load stresses and section capacities. Suggests joint repair measures.

Priestley, M. J. Nigel and Frieder Seible. *Collapse of the Cypress Viaduct, Final Report to Caltrans Office of Structures Design.* Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, November 9, 1989 [CT014].

Synopsis: Damage analysis provided for Types I, II, and II bent systems. Provides reinforcement layout for knee joints and calculations for member capacities. States that the crack patterns at Embarcadero are indicative of the mechanism developed at Cypress. Failure mechanisms for all three bent types involve shear failure in the upper columns.

Priestley, M. J. Nigel. "Damage to the I-5, I-605 Separator," *Earthquake Spectra*, Volume 4, No. 2. Earthquake Engineering Research Institute, May 1988.

Priestley, M. J. Nigel and Frieder Seible. *Retrofitting Bridge Columns—Additional Tests.* Research proposal to Caltrans, March 1988.

Priestley, M. J. Nigel, F. Seible, and Y. H. Chai. *Seismic Retrofitting of Bridge Columns.* Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, 1988 [CT035].

Synopsis: Report on the preliminary tests conducted at University of California at San Diego on the seismic retrofitting of bridge columns as a result of shear failure of columns of the I-5/I-605 separator during the Whittier earthquake. Two types of retrofit schemes are considered. The first one suitable for circular or rectangular column consists of confining the column by steel jacketing. The second method suitable for circular or oval columns consists of wrapping a prestressing wire or tendon and covering them by mortar or epoxy coating. Preliminary results indicate preference for steel-jacketing.

Prud'homme, Rachel. "George W. Housner: How It Was," *Engineering & Science.* California Institute of Technology, Volume LIII, Number 2, pp. 26-35, Winter 1990 [VAR073].

Synopsis: This article is a transcript of excerpts from three days of interviews of Dr. Housner by Rachel Prud'homme. The interviews, conducted in 1984 as part of the Oral History Project of the Caltech Archives, were a recording of remembrances by the man known as "the father of earthquake engineering." The excerpts included in the article trace the development of earthquake-safe building standards in California—and probably the rest of the world as well.

Psomas, Timothy G. *Opposition to Bridge Structural Engineer Licensing Proposal.* Letter to Dr. George Housner, California Council of Civil Engineers & Land Surveyors, dated April 10, 1990 [VAR076].

Synopsis: States the strong opposition of the California Council of Civil Engineers & Land Surveyors to creation of a special license category for bridge structural engineers, as suggested in testimony to the Board of Inquiry by Albert Blaylock, President of the Board of Registration for Professional Engineers & Land Surveyors. In support of this opposition, the author includes a copy of testimony presented to the Board of Registration by Council member Arthur R. McDaniel and a position paper presented to the Board of Registration by the Professional Engineers in California Government.

Purcell, C. H., C. E. Andrew, and G. B. Woodruff. "East Bay Crossing of the Bay Bridge," *Engineering News Record.* November 26, 1936.

Raab, Norman C. and Howard C. Wood. "Earthquake Stresses in the San Francisco-Oakland Bay Bridge," *ASCE Transactions.* American Society of Civil Engineers, Paper No. 2123, Vol. 106, 1941 [VAR005].

Synopsis: Describes the analysis of Bay Bridge subjected to earthquake loads assumed as 0.10g. horizontal acceleration at 1.5 sec. period with a displacement amplitude of 2.2 in. The seismic stresses are of the same order as those from the design wind loads. The paper states that the resonance of the bridge under seismic loads is improbable.

Radbruch, Dorothy H. *Areal and Engineering Geology of the Oakland West Quadrangle, California.* Map I-239, United States Department of the Interior, U. S. Geological Survey, 1957.

Randall, J. R. and Greg Armendariz. "Report to the Board of Inquiry on Bridges in Santa Clara County." Material submitted to the Board of Inquiry and dated January 19, 1990 [VAR035].

Synopsis: Description of damage observed to bridges maintained by the County of Santa Clara and cities within the county.

Requa, Mark L., Chairman. *Report of the Hoover-Young San Francisco Bay Bridge Commission to the President of the United States and the Governor of California.* 1930 [VAR048].

Synopsis: Excerpts from the feasibility report on the Bay Bridge by the nine-member commission apparently appointed by President Herbert Hoover and California Governor C. C. Young and charged with finding a solution to the need for a transportation artery across San Francisco Bay that would be acceptable to all parties and would be economically possible. The commission considered several routes for the proposed bridge and recommended the present route and a design not too different from the present one.

Rinaldi, Angela (Project Editor), Larry Armstrong (Photo Editor), Craig Turner (Text Editor), and the staff of the Los Angeles Times. *The 1989 San Francisco Bay Earthquake, Portraits of Tragedy and Courage.* Los Angeles Times, 1989 [VAR056].

Synopsis: Compilation of color photographs and essays of the Loma Prieta earthquake, prepared by the staff of the Los Angeles Times. Most photos and essays are human interest subjects, but some pictures of the Bay Bridge and Nimitz Freeway (Interstate 880) are included as well.

Roberts, James E. *UBC Seismic Code Requirements, 1940 - 1953 Era.* State of California, Department of Transportation, Division of Structures, material submitted to the Board of Inquiry and dated April 7, 1990 [CT077].

Synopsis: Information requested by Dr. Housner regarding the Uniform Building Code (UBC) seismic code during the period of the design of the Cypress Viaduct structure. Attachments to the transmittal letter include an explanation of the Caltrans seismic code of 1949 and how it related to the UBC standards; a copy of portions of the 1943 UBC specifications pertaining to the seismic design of structures; a copy of the portions of the 1952 UBC specifications pertaining to the seismic design of structures; and a brief history of earthquake codes in California, excerpted from the 1980 code of the Structural Engineers Association of California (SEAOC).

Roberts, James E. "*Presentation by Caltrans to Governor's Board of Inquiry.*" Testimony presented to the Board of Inquiry March 15, 1990 [CT070].

Synopsis: Summary of final presentation session by Caltrans to the Board of Inquiry. The presentation includes remarks on an historical perspective to the bridge failures, reiteration of Caltrans' seismic design standards and philosophy, Caltrans' actions following the Loma Prieta Earthquake, and a final summary. Several attachments are incorporated into the presentation, including: (1) a brief explanation and timeline of the Caltrans budget process; (2) a summary of expended funds for structure related research; (3) a summary of annual expenditures for bridge retrofit projects; (4) copies of articles from Caltrans publications which provide insights into the development and theory behind the original design for the Cypress Street Viaduct structure; (5) a Caltrans in-house article which discusses the theory behind the seismic retrofit program; (6) a letter from Jim Gates presenting some of his thoughts on the inquiry process; and (7) a listing of Caltrans bridges damaged during the Loma Prieta Earthquake.

Roberts, James E. *Response to Recommendation of the Board of Inquiry.* Letter to Professor George W. Housner dated March 6, 1990 [CT066].

Synopsis: Letter to the Chairman of the Board of Inquiry describing formation by Caltrans of a Peer Review Team of experts in seismic and structural research, structural design, bridge design, seismic and structural analysis, and structural design education. The team will review the plans for the retrofit of the six San Francisco viaducts. This letter is in response to the recommendation of the Board of Inquiry in its Progress Report to the Governor.

Roberts, James E. "Report on Caltrans Research Budgets and Research Studies." Testimony presented to the Board of Inquiry January 18, 1990.

Robinson, R. R., A. Longinow, and K. H. Chu. *Seismic Retrofit Measures for Highway Bridges, Vol. 1, Earthquake and Structural Analysis.* Report No. FHWA-TS-79-216, U. S. Department of Transportation, Federal Highway Administration, April 1979 (A).

Robinson, R. R., A. Longinow, and D. S. Albert. *Seismic Retrofit Measures for Highway Bridges, Vol. 2, Design Manual.* Report No. FHWA-TS-79-217, U. S. Department of Transportation, Federal Highway Administration, April 1979 (B).

Robinson, R. R., E. Privitzer, A. Longinow, K. H. Chu. *Structural Analysis and Retrofitting of Existing Highway Bridges Subjected to Strong Motion Seismic Loading*. Report No. FHWA-RD-75-94, U. S. Department of Transportation, Federal Highway Administration, May 1975.

Rogers, J. David. *Outline of Testimony Given Before Assembly Transportation Committee*. Material submitted to the Board of Inquiry and dated November 7, 1989 [VAR001].

Synopsis: Report on history and seismic design concepts of Cypress Viaduct structure, as presented to the Assembly Transportation Committee and the Board of Inquiry.

Schnabel, P. B., J. Lysmer, and H. B. Seed. *SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites*. Report No. UCB/EERC-72/12, Earthquake Engineering Research Center, University of California at Berkeley, December 1972.

Scott, Mel. *The San Francisco Bay Area*. University of California Press, Berkeley, California, 1985.

Scott, Ronald F. "Statement to the Governor's Board of Inquiry." Testimony presented to the Board of Inquiry January 17, 1990 [VAR022].

Synopsis: Report to the Board of Inquiry on the Independent Seismic Design Review of H.O.V. Viaduct No. 2 (I-110 Harbor Project 15), prepared for Caltrans.

Scott, Ronald F. "Liquefaction," *Engineering & Science*. California Institute of Technology, Volume LIII, Number 2, pp. 21-25, Winter 1990 [VAR072].

Synopsis: This article attempts to present answers in layman's language to a series of public questions on soil liquefaction, which was identified as a contributor to the degree of damage observed in the Loma Prieta earthquake. Specifically, the author defines soil liquefaction, describes the conditions that give rise to it, identifies locations in the Los Angeles area where it might potentially occur, discusses the associated hazards, and suggests possible means of alleviating the hazard to structures from soil liquefaction.

Scott, Stanley. *Independent Review of Critical Facilities: With Special Emphasis on State-Federal Relationships and Dam Safety*. Report SSC 81-01, State of California, Seismic Safety Commission, January 1981 [VAR068].

Synopsis: Report based on a paper presented to the Western States Seismic Policy Council meeting in Sacramento on April 11-12, 1980. The report examines the need for independent review of critical facilities and defines the composition and responsibilities of panels for such review.

Seed, H. B. and J. I. Sun. *Implications of Site Effects in the Mexico City Earthquake of September 19, 1985 for Earthquake-Resistant Design Criteria in the San Francisco Bay Area of California*. Report No. UCB/EERC-89-03, Earthquake Engineering Research Center, University of California at Berkeley, March 1989.

Seed, H. B. and I. M. Idriss. *Ground Motions and Soil Liquefaction During Earthquakes*. Monograph No. MNO-5, Earthquake Engineering Research Institute, 1983.

Seed, H. B. and Idriss. *Rock Motion Accelerograms for High Magnitude Earthquakes*. Report No. UCB/EERC-69/07, Earthquake Engineering Research Center, University of California at Berkeley, April 1969.

Seim, Charles, Mark Ketchum, J. P. Singh, Esmond Chan, Keith Bull, and Ron Zimmerman. "Golden Gate Bridge Seismic Investigation and Embarcadero Viaduct Seismic Evaluation and Strengthening Methods." T.Y. Lin International, testimony presented to the Board of Inquiry March 1, 1990 [CT063].

Synopsis: Description of the scope and objectives, with other comments as appropriate, of two projects of T.Y. Lin International being performed under contract to the Golden Gate Bridge District and Caltrans, respectively.

Selna, Lawrence G., L. J. Malvar, and R. J. Zelinski. "Box Girder Bar and Bracket Seismic Retrofit Devices," *ACI Structural Journal*. American Concrete Institute, Vol. 86, No. 5, September-October 1989 [VAR011].

Synopsis: Provides results of full-scale test program at UCLA on the two types of joint restrainers for bridge decks, namely, the bar restrainer and the cable restrainer. A weak link in this program is the failure in the box girder hinge diaphragm. Does not indicate any preference.

Selna, Lawrence G., L. J. Malvar, and R. J. Zelinski. "Bridge Retrofit Testing: Hinge Cable Restrainers," *Journal of Structural Engineering*. American Society of Civil Engineers, Vol. 115, No. 4, April 1989 [VAR012].

Synopsis: The testing program at UCLA on the C-1 cable restrainers that Caltrans installed on most bridges, is described. The tests were full-scale tests and included drum. The test results indicate that the actual capacity of the restrainers is slightly greater than the design value and the failure occurs in the reinforced concrete hinge diaphragm.

Shakal, A., M. Huang, M. Reichle, C. Ventura, T. Cao, R. Sherburne, M. Savage, R. Darragh, and C. Petersen. *CSMIP Strong Motion Records from the Santa Cruz Mountains (Loma Prieta) Earthquake of October 17, 1989.* Report OSMS 89-06, California Department of Conservation, Division.of Mines and Geology, November 17, 1989 [DMG001].

Synopsis: Presents plots of all records obtained from 93 CSMIP stations extending up to 175 km from the epicenter, consisting of a total 125 records (3 components). The 93 stations include 53 ground-response stations and 40 extensively-instrumented structures.

Shangle, Barbara J., Editor. *Earthquake 7.1: San Francisco Bay Area, October 17, 1989.* LTA Publishing Company and United Press International, 1989 [VAR025].

Synopsis: Compilation of extensive color photos of Loma Prieta earthquake along with the narrative reports from various United Press International reporters from October 17, 1989 through October 23, 1989. Datelines in San Francisco, Santa Cruz, Oakland, Mission San Juan Bautista, etc.

Shelton, T. K. *Photographs of the San Francisco-Oakland Bay Bridge.* State of California, Department of the California Highway Patrol, material submitted to the Board of Inquiry and dated March 8, 1990 [VAR063].

Synopsis: Reprints of 20 photographs which were presented to the Board at its hearing of February 8, 1990. The photographs show various views of the collapsed and damaged portions of the Bay Bridge and repair of the damage.

Smith, C. L. (retired Caltrans employee). *Letter to Ben Williams enclosing various memos and articles.* December 29, 1989 [VAR055].

Synopsis: A retired Caltrans employee criticizes Caltrans' way of doing things. He raises issues such as steel vs. concrete bridges, seismic vulnerability of freeways, and putting freeways underground.

Spittler, T. E. and R. H. Sydnor. "Landslides and Ridge-Top Failures Associated with the Epicentral Area of the Loma Prieta Earthquake of October 17, 1989, Santa Cruz County, California," *EOS.* American Geophysical Union, Vol. 71, No. 8, p. 290, February 20, 1990.

State of California, Department of Conservation, Division of Mines and Geology. *Plots of the Processed Data for the Interim Set of 14 Records from the Santa Cruz Mountains (Loma Prieta) Earthquake.* Report OSMS 89-08, California Department of Conservation, December 13, 1989 [DMG004].

Synopsis: Plots of the processed data for 14 selected records, each consisting of three components, were presented. Data consist of uncorrected accelerations, instrument and baseline-corrected acceleration, velocity and displacement plots, response spectra, Fourier amplitude spectra, and absolute acceleration spectra for 0, 2, 5, 10 and 20 percent damping values.

State of California, Department of Conservation, Division of Mines and Geology. "Strong Ground Shaking from the Loma Prieta Earthquake of October 17, 1989 and Its Relation to Near-Surface Geology in Oakland Area." California Department of Conservation, testimony presented to the Board of Inquiry November 28, 1989 [DMG002].

Synopsis: This report first describes the Loma Prieta earthquake. Figures are given to show after-shocks, focal-mechanism solution, peak horizontal acceleration values with distance, selected strong-motion records, contour plot of the thickness of Bay mud, response spectra of records near Oakland and strongly shaken zone (0.4g or greater) for the Loma Prieta earthquake. Also given are maps for strongly shaken zones for anticipated magnitude 7 earthquakes on the San Andreas fault near San Francisco and on the Hayward fault near Oakland.

State of California, Department of Conservation, Division of Mines and Geology. *Second Quick Report on CSMIP Strong-Motion Records from the October 17, 1989 Earthquake in the Santa Cruz Mountains.* California Department of Conservation, October 25, 1989 [PAS003].

Synopsis: This second quick report presents accelerograms that were recovered within eight days after the earthquake, from about 90 CSMIP stations.

State of California, Department of Conservation, Division of Mines and Geology. *Quick Report on CSMIP Strong-Motion Records from the October 17, 1989 Earthquake in the Santa Cruz Mountains.* California Department of Conservation, October 19, 1989 [PAS002].

Synopsis: Accelerograms from about 28 CSMIP stations that were collected within two days after the earthquake are provided. These records include ground-response stations and those on the structures.

State of California, Department of Conservation, Division of Mines and Geology. *Fault-Rupture Hazard Zones in California.* Special Publication 42, California Department of Conservation, Revised 1988 [DMG003].

Synopsis: Describes special study zones, as defined by the Alquist-Priolo Special Studies Zones Act. This Act generally prohibits the location of structures for human occupancy across the traces of active faults. Under the Act, the State Geologist is required to delineate special studies zones, in which development is to be prohibited unless special geologic investigations determine that structures are safe from surface displacements.

State of California, Department of Public Works. "San Francisco-Oakland Bay Bridge, Design Specifications, Superstructure." State of California, Department of Public Works, various dates.

State of California, Department of the Highway Patrol. *Photographs of the Cypress Structure.* Photographs taken October 17, 1989 [VAR052].

Synopsis: Copies of 26 aerial photographs taken of the Cypress Structure on October 17, 1989, approximately 20 minutes after the earthquake, by Officer Guinn of the CHP Golden Gate Air Operations Unit.

State of California, Department of Transportation. "Caltrans and Division of Structures Organization." Testimony presented to the Board of Inquiry March 1, 1990 (A) [CT058].

Synopsis: Organization charts of Caltrans and the Division of Structures and listing of functional responsibilities of the Division of Structures.

State of California, Department of Transportation. *Project Plans, Alemany Viaduct.* Material submitted to the Board of Inquiry March 1, 1990 (B) [PAS015].

Synopsis: As-built plans for the Alemany Viaduct in San Francisco. Consists of approximately 150 engineering drawings and technical specifications of bridge structures built for the viaduct. Drawings are dated from approximately 1953 through approximately 1958.

State of California, Department of Transportation. *Project Plans, Central Viaduct.* Material submitted to the Board of Inquiry March 1, 1990 (C) [PAS016].

Synopsis: As-built plans for the Central Viaduct in San Francisco. Consists of approximately 300 engineering drawings and technical specifications of bridge structures built for the viaduct. Drawings are dated from approximately 1951 through approximately 1977.

State of California, Department of Transportation. *Project Plans, China Basin Viaduct, Unit #1.* Material submitted to the Board of Inquiry March 1, 1990 (D) [PAS017].

Synopsis: As-built plans for the China Basin Viaduct in San Francisco. Consists of approximately 150 engineering drawings and technical specifications of bridge structures built for the viaduct. Drawings are dated from approximately 1965 through approximately 1969.

State of California, Department of Transportation. *Project Plans, China Basin Viaduct, Unit #2.* Material submitted to the Board of Inquiry March 1, 1990 (E) [PAS018].

Synopsis: As-built plans for the China Basin Viaduct in San Francisco. Consists of approximately 150 engineering drawings and technical specifications of bridge structures built for the viaduct. Drawings are dated from approximately 1965 through approximately 1985.

State of California, Department of Transportation. *Project Plans, Embarcadero Freeway Viaduct.* Material submitted to the Board of Inquiry March 1, 1990 (F) [PAS020].

Synopsis: As-built plans for the Embarcadero Freeway Viaduct in San Francisco. Consists of approximately 150 engineering drawings and technical specifications of bridge structures built for the viaduct. Drawings are dated from approximately 1955 through approximately 1963.

State of California, Department of Transportation. *Project Plans, Southern Freeway Viaduct.* Material submitted to the Board of Inquiry March 1, 1990 (G) [PAS014].

Synopsis: As-built plans for the Southern Freeway Viaduct in San Francisco. Consists of approximately 300 engineering drawings and technical specifications of bridge structures built for the viaduct. Drawings are dated from approximately 1964 through approximately 1974.

State of California, Department of Transportation. *Project Plans, Terminal Separation Viaduct.* Material submitted to the Board of Inquiry March 1, 1990 (H) [PAS019].

Synopsis: As-built plans for the Terminal Separation Viaduct in San Francisco. Consists of approximately 250 engineering drawings and technical specifications of bridge structures built for the viaduct. Drawings are dated from approximately 1954 through approximately 1975.

State of California, Department of Transportation. *Retrofit Project—China Basin Route 280 Viaduct, General Plan.* Material presented to the Board of Inquiry March 1, 1990 (I) [CT067].

Synopsis: General plan drawings and photographs for retrofit of the China Basin Viaduct.

State of California, Department of Transportation. *Status of Selected Bridges.* Material submitted to the Board of Inquiry and dated February 28, 1990 [CT057].

Synopsis: One-page statistical summary of number of bridges, total length, bridge retrofit cost, and total retrofit cost, by Caltrans District.

State of California, Department of Transportation. *Cypress Demolition Slides.* Material submitted to the Board of Inquiry and dated February 22, 1990 [CT055].

Synopsis: Set of 10 slides that show the collapse of a portion of the Cypress Viaduct during demolition operations, taken by an employee of the demolition contractors. The photographer is not known.

State of California, Department of Transportation. *San Francisco Double Deck Sites.* Material presented to the Board of Inquiry February 8, 1990 [CT068].

Synopsis: Large map (23" X 36") of the eastern portion of San Francisco, highlighting the locations of the six double-deck viaducts.

State of California, Department of Transportation. *Summary of the State Bridge Seismic Retrofit Program.* Material submitted to the Board of Inquiry February 6, 1990.

State of California, Department of Transportation. *Summary of the Local Bridge Seismic Retrofit Program.* Material submitted to the Board of Inquiry February 7, 1990.

State of California, Department of Transportation. "Cypress Viaduct, Log of Test Borings." February 1990, material submitted to the Board of Inquiry on March 1, 1990 [CT061].

Synopsis: Set of 9 test boring log sheets and drawings for areas surrounding the Cypress structure.

State of California, Department of Transportation. *Status of Bridges Maintained by Caltrans.* Material submitted to the Board of Inquiry December 14, 1989 [CT030].

Synopsis: Brief summary of number of bridges maintained by Caltrans and numbers needing retrofit under single-column and multiple-column retrofit programs.

State of California, Department of Transportation. *The Loma Prieta Earthquake.* Video tape made by Caltrans engineers immediately after the earthquake, material presented to the Board of Inquiry December 13, 1989 [CT018].

Synopsis: Video tape made by Caltrans engineers of the Cypress Viaduct structure, Bay Bridge, and Embarcadero immediately after the earthquake, October 18-22, 1989.

State of California, Department of Transportation. *Sample of Typical Retrofit Plan: Project Plans for Construction on State Highway in Los Angeles County (Culver City at Route 405).* Plan Set CRMA-5090(10), December 11, 1989 [CT049].

Synopsis: Sample of plans for a typical retrofit project.

State of California, Department of Transportation. *Sketches of Proposed Future Repairs at Bent E9 (Bay Bridge).* December 6, 1989 [CT025].

Synopsis: Engineering drawings of planned repairs.

State of California, Department of Transportation. *Sample of Typical Retrofit Plan: Project Plans for Construction on State Highway in Los Angeles County (Route 57/60 Separation).* Plan Set F-P057(19), December 4, 1989 [CT048].

Synopsis: Sample of plans for a typical retrofit project.

State of California, Department of Transportation. "Presentation by Caltrans to Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake." Testimony presented to the Board of Inquiry November 28, 1989 [CT002].

Synopsis: History of seismic requirements of AASHO/AASHTO code; history of Caltrans seismic design criteria; description and interpretation of Cypress Viaduct failure; description and interpretation of Bay Bridge failure.

State of California, Department of Transportation. *Cypress Viaduct.* Video tape made by Caltrans engineers, October-December 1989 [CT069].

Synopsis: Video tape (approximately 120 minutes in length) of testing of the Cypress structure by U.C. Berkeley team and demolition of portions of the structure that were not damaged during the earthquake.

State of California, Department of Transportation. *Photographs of structural failures and cracking.* Photographs taken October 19-25, 1989 [CT040].

Synopsis: Photographs of various examples of cracking and structural failures throughout the earthquake area.

State of California, Department of Transportation. *Soil Borings and Foundations Exploration Information for Bay Bridge and Approaches.* Various dates, 1967-71 [CT051].

Synopsis: Detailed drawings showing foundation information for Bay Bridge and surrounding area.

State of California, Department of Transportation. *Selected Contract Plan Sheets for the Bay Bridge.* October 1933 [CT024].

Synopsis: Historical plans and drawings of Bay Bridge design.

State of California, Department of Transportation. *'As-Built' Plans of the Cypress Structure and Bay Bridge.* Various dates [CT001].

Synopsis: Original plans for the Cypress Structure and Bay Bridge.

State of California, Department of Transportation. *Bridge Details Resulting From Experience Gained From the Los Angeles Earthquake.* 1971.

State of California, Department of Transportation. Correspondence file on column retrofit research at U.C. San Diego. Various dates.

State of California, Department of Transportation. Correspondence file on U.C. San Diego research on the I-605—I-5 overcrossing. Various dates.

State of California, Department of Transportation, Division of Structures. "Current Seismic Design Procedures (11/89): Bridge Design Aids." Testimony presented to the Board of Inquiry November 28, 1989 (A) [CT010].

Synopsis: Excerpts from current Bridge Design Aids manual for internal use by Caltrans engineers.

State of California, Department of Transportation, Division of Structures. "Current Seismic Design Procedures (11/89): Design Specifications and Commentary." Testimony presented to the Board of Inquiry November 28, 1989 (B) [CT008].

Synopsis: Excerpts (loads, foundations, substructures, and reinforced concrete sections) from current Caltrans internal seismic design manuals, including commentaries, for use in Caltrans projects.

State of California, Department of Transportation, Division of Structures. "Current Seismic Design Procedures (11/89): Memos to Designers." Testimony presented to the Board of Inquiry November 28, 1989 (C) [CT009].

Synopsis: Excerpts from current manual of memos to designers on seismic design procedures, for internal use by Caltrans engineers.

State of California, Department of Transportation, Division of Structures. "Cypress Viaduct—Miscellaneous Publications." Testimony presented to the Board of Inquiry November 28, 1989 (D) [CT011].

Synopsis: Various articlês about the Cypress structure, written before 1959.

State of California, Department of Transportation, Division of Structures. "Seismic Design Procedures and Specifications, 1940 to 1968." Material submitted to the Board of Inquiry November 28, 1989 (E) and having various dates [CT005].

Synopsis: Excerpts (loads, foundations, substructures, and reinforced concrete sections) from various historical editions of Caltrans internal seismic design manuals, for use in Caltrans projects. Excerpts included are from 1940, 1943, 1946, 1949, 1953, 1963, 1964, 1965, 1966, 1967, and 1968.

State of California, Department of Transportation, Division of Structures. "Seismic Design Procedures and Specifications, 1971 Detail and Design Booklet." Testimony presented to the Board of Inquiry November 28, 1989 (F) [CT006].

Synopsis: Bridge Details Resulting from experience gained from the San Fernando earthquake of February 9, 1971.

State of California, Department of Transportation, Division of Structures. "Seismic Design Procedures and Specifications, 1972 to 1984." Testimony presented to the Board of Inquiry November 28, 1989 (G) [CT007].

Synopsis: Excerpts from Caltrans internal manuals, explaining seismic design procedures and specifications for use in Caltrans projects.

State of California, Department of Transportation, Division of Structures. *Bay Bridge—Seismic Design Excerpts.* Circa 1933 [CT015].

Synopsis: Historical information on Bay Bridge design

State of California, Department of Water Resources, Division of Safety of Dams. *Statutes and Regulations Pertaining to Supervision of Dams and Reservoirs.* State of California, Department of Water Resources, 1989 [VAR040].

Synopsis: Authorizing statutes and regulations governing the Division of Safety of Dams.

State of California, Office of Emergency Services. *Loma Prieta Earthquake: Homes/Businesses Damaged/ Destroyed.* State of California, Office of Emergency Services, December 14, 1989 [VAR013].

Synopsis: Summary statistics reported by county offices of emergency services of buildings damaged and people killed and injured from the earthquake.

State of California, State and Consumer Services Agency. *State Construction Standards and Policies.* Report prepared under the order of S. Clinton, State of California, January 1990 (undated).

Steinbrugge, Karl V., John H. Bennett, Henry J. Lagorio, James F. Davis, Glenn Borchardt, and Tousson R. Toppozada. *Earthquake Planning Scenario for a Magnitude 7.5 Earthquake on the Hayward Fault in the San Francisco Bay Area.* Special Publication 78, California Department of Conservation, Division of Mines and Geology, 1987 [DMG005].

Synopsis: A detailed planning document for the maximum credible earthquake of magnitude 7.5 on the Hayward fault. It provides detailed damage scenarios for all types of buildings, transportation facilities and utility lifelines. It contains a map of the Bay area showing major highways that would be damaged. It predicts that soil and structure failures at the Bay Bridge east approach would close the bridge over 72 hours. It predicts damage to I-280 and Embarcadero Freeway but does not specifically mention Nimitz Freeway.

Stroup, Darlene Atkinson and Al Blaylock. *Transcript of Presentation by the Board of Registration for Professional Engineers and Land Surveyors.* State of California, Department of Consumer Affairs, testimony presented to the Board of Inquiry on February 8, 1990 and dated March 5, 1990 [VAR066].

Synopsis: Verbatim transcription of the presentation made by representatives of the Board of Registration for Professional Engineers and Land Surveyors before the Board of Inquiry at its February 8, 1990 meeting. The presentation summarizes a report currently being prepared by the Board of Professional Engineers on the structural failures of the Cypress Viaduct and Bay Bridge. In addition, members asked several questions about current professional engineer registration and certification practices and procedures.

Tobin, L. Thomas. "Board of Inquiry Presentation." California Seismic Safety Commission, testimony presented to the Board of Inquiry March 15, 1990 [VAR074].

Synopsis: This is a topical summary of the presentation made at the March 15, 1990 meeting of the Board of Inquiry by L. Thomas Tobin, Executive Director of the California Seismic Safety Commission. The presentation described the duties and responsibilities of the Seismic Safety Commission, highlighted major programs of the Commission, offerred several recommendations to the Board of Inquiry on behalf of the Commission, and presented conclusions relevant to the Board of Inquiry's charge, based on experience of the Commission.

Tobin, L. Thomas. *Letter to Robert Best, Director, Caltrans, regarding use of independent peer review panels.* California Seismic Safety Commission, February 6, 1990 [VAR047].

Synopsis: Recommends, on behalf of the Seismic Safety Commission, that Caltrans have earthquake retrofit designs and repairs reviewed by an independent panel of engineering peers.

Toppozada, Tousson R., John H. Bennett, Glenn Borchardt, Richard Saul, and James F. Davis. "Earthquake Planning Scenario for a Major Earthquake on the Newport-Inglewood Fault Zone," *California Geology*. California Department of Conservation, Division of Mines and Geology, April 1989, pp. 75-84 [PAS001].

Synopsis: Gives a summary of planning scenarios for a major earthquake of magnitude 7 on the Newport-Inglewood fault.

Trask, Parker D. and Jack W. Rolston. "Engineering Geology of San Francisco Bay, California," *Bulletin of the Geological Society of America*. Geological Society of America, Vol. 62, pp. 1079-1110, 1951.

Travis, William. "Presentation to the Governor's Board of Inquiry on the Loma Prieta Earthquake." San Francisco Bay Conservation and Development Commission, testimony presented to the Board of Inquiry January 4, 1990 [VAR014].

Synopsis: Four major points made during presentation, including recommendations to the Board.

Tudor Engineering. "Testimony to the Board of Inquiry on Retrofit Plans for the San Francisco Central Freeway Viaduct." Testimony presented to the Board of Inquiry March 2, 1990.

Uang, C. M. and V. V. Bertero. *Implications of Recorded Ground Motion on the Seismic Design of Building Structures*. Report No. UCB/EERC-88-13, Earthquake Engineering Research Center, University of California at Berkeley, September 1988.

United States Geological Survey Staff (Branches of Engineering, Seismology, and Geology, and Western Regional Geology). "Apparent Lithologic and Structural Control of Surface Faulting During the Loma Prieta Earthquake of October 17, 1989," *EOS*. American Geophysical Union, Vol. 71, No. 8, p. 290, February 20, 1990.

United States Geological Service. "The Loma Prieta, California, Earthquake: An Anticipated Event," *Science*. Vol. 247, pp. 286-293, January 1990 [PAS004].

Synopsis: Summarizes the causes and effects of the Loma Prieta earthquake. Studies in the past predicted high probabilities for the fault rupture of the San Andreas fault segment in the southern Santa Cruz Mountains segment. This earthquake, while anticipated, carried no obvious foreshocks to forewarn of the event. The extent of the damage was also anticipated because of bay mud and man-made fills.

United States Geological Survey. *Corrected Acceleration, Velocity, and Displacement at 200, SPS*—Various record sets, United States Department of the Interior, U. S. Geological Survey, October 18, 1989 [USG007].

Synopsis: Sets of strong motion data records of the earthquake on October 18, 1989 (GMT), recorded at stations in Emeryville, the Transamerica Building in San Francisco, and the Abutment Building of the Golden Gate Bridge.

United States Steel Corporation. *The San Francisco-Oakland Bay Bridge*. United States Steel Corporation, 1936 [PAS021].

Synopsis: This report was written soon after the completion of the Bay Bridge in 1936. The construction started in May 1933 and was completed in November 1936 at a cost of about $78 million and using over 200,000 tons of steel. The report essentially describes the construction procedures, including fabrication and erection.

Ward, Peter L. and Robert A. Page. *The Loma Prieta Earthquake of October 17, 1989: What Happened ... What is Expected ... What Can Be Done.* United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1989 [USG004].

Synopsis: A brief geologic view of what caused the Loma Prieta earthquake and implications for future California earthquakes.

Werne, Roger, Alan Copeland, Gerry Goudreau, Greg Kay, Roger Logan, Dave McCallen, Bob Rainsberger, and Dale Schauer. "Cypress Structure: Computer Modeling and Engineering Analysis Update." Lawrence Livermore National Laboratory, testimony presented to the Board of Inquiry March 2, 1990 [VAR058].

Synopsis: Copies of overhead projections used by Lawrence Livermore Laboratory staff in a presentation to the Board of Inquiry, which described the supercomputer-aided nonlinear analysis used by Lawrence Livermore to analyze the Cypress Viaduct structure. The goal was to improve Caltrans' understanding of the failure by evaluating concrete and rebar designs, developing failure models, modeling bridge expansion joints, and developing a three dimensional soil liquefaction model.

Whittaker, A. S., C. M. Uang, and V. V. Bertero. *Earthquake Simulator Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Eccentrically Braced Steel Structure.* Report No. UCB/EERC-87-02, Earthquake Engineering Research Center, University of California at Berkeley, July 1987.

Wiedner, Karl and Wen S. Tseng. "Caltrans Terminal Separation Project (I-480), San Francisco: Presentation by Bechtel to the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake." Bechtel Corporation, testimony presented to the Board of Inquiry March 2, 1990 [CT076].

Synopsis: Report of presentation made to the Board of Inquiry explaining the retrofit strategy employed by the consultant for the I-480 Terminal Separation project, computer modeling assumptions, and inspection reports of the structure. The presentation includes drawings of retrofit details, photographs of bent damage, and a brochure on Bechtel Corporation's Added Damping and Stiffness Elements (ADAS) earthquake energy dissipation device for structures.

Williams, Ben A. "Report to the Board of Inquiry on Results of Survey of Locally Maintained Bridges Damaged by the Loma Prieta Earthquake." Testimony presented to the Board of Inquiry February 8, 1990 [VAR036].

Synopsis: Description of damage observed to bridges maintained by local governments in the nine counties declaring an emergency after the earthquake, based on survey responses received from them.

Wilson, Edward L. *Dynamic Analysis of a Typical Test Bent of the Cypress Viaduct.* November 30, 1989 [VAR081].

Synopsis: In-progress follow-up to Wilson's October 29 preliminary report titled "A Static and Dynamic Analysis of a Typical Test Bent of the Cypress Viaduct." Diagrams illustrate the finite element mesh, dimensions, and material properties. Data is also presented on the dynamic response of the structure.

Wilson, Edward L. *Preliminary Report: A Static and Dynamic Analysis of a Typical Bent of the Cypress Viaduct.* October 29, 1989 [VAR010].

Synopsis: A two-dimensional elastic analysis was performed of a typical bent of the Cypress Viaduct (bent type B2), using two-dimensional finite elements. Static stresses in the bent were reported for dead load and a lateral load of 20% of dead load. A dynamic analysis was performed using the accelerations recorded at the San Francisco Airport as base input. The maximum ground acceleration was 0.30g.

Wong, T. Y., V. V. Bertero, and E. P. Popov. *Hysteretic Behavior of Reinforced Concrete Framed Walls.* Report No. UCB/EERC-75-23, Earthquake Engineering Research Center, University of California at Berkeley, 1975.

Working Group on California Earthquake Probabilities. *Probabilities of Large Earthquakes Occurring in California on the San Andreas Fault.* Geological Survey Open-File Report 88-398, United States Department of the Interior, U. S. Geological Survey, United States Government Printing Office, 1988 [USG006].

Synopsis: Describes the work by the National Earthquake Prediction Evaluation Council, which consists of twelve eminent geologists and seismologists. The group provided probabilities for large (magnitude 7 or greater) earthquakes on the major faults of the San Andreas fault system. It provided probabilities for 5-year, 10-year, 20-year, and 30-year time intervals. Because of insufficient data, similar probabilities for other faults are not provided.

Wyllie, Loring A., Jr. "Comments on the Behavior of Reinforced Concrete Structures During Earthquakes." Testimony presented to the Board of Inquiry March 1, 1990 [VAR059].

Synopsis: Text of a presentation to the Board of Inquiry on behalf of the American Concrete Institute Committee 318 and H.J. Degenkolb Associates, Engineers. The presentation describes ductility and other properties of reinforced concrete, as used in bridge structures.

Zelinski, Ray J. "Outline of Presentation to Board of Inquiry." California Department of Transportation, testimony presented to the Board of Inquiry February 8, 1990 [CT050].

Synopsis: Description of a "typical retrofitting strategy" and summary information on San Francisco double-deck viaduct retrofit projects.

Zelinski, Ray J. *Request for Information on Semi-permanent Retrofit of Double Decker Viaducts.* Letter sent to all San Francisco retrofit project consultants, California Department of Transportation, January 8, 1990 [CT047].

Synopsis: Request for various informational items relative to retrofit projects of San Francisco area structures closed following the Loma Prieta earthquake, defining the scope of work to be performed by the consultants.

Zelinski, Ray J. *Information Memo #4 on Semi-permanent Retrofit of Double-Decker Viaducts.* Letter sent to all San Francisco retrofit project consultants, California Department of Transportation, December 30, 1989 [CT046].

Synopsis: Guidelines for retrofit of San Francisco area structures closed following the Loma Prieta Earthquake.

Zelinski, Ray J. *Information Memo #3 on Semi-permanent Retrofit of Double-Decker Viaducts.* Letter sent to all San Francisco retrofit project consultants, California Department of Transportation, December 10, 1989 [CT045].

Synopsis: Guidelines for retrofit of San Francisco area structures closed following the Loma Prieta Earthquake.

Zelinski, Ray J. *Information Memo #2 on Semi-permanent Retrofit of Double-Decker Viaducts.* Letter sent to all San Francisco retrofit project consultants, California Department of Transportation, December 5, 1989 [CT044].

Synopsis: Guidelines for retrofit of San Francisco area structures closed following the Loma Prieta Earthquake.

Zelinski, Ray J. *General Guidelines for Semi-permanent Retrofit of Double-Decker Viaducts—Information Memo #1.* Letter sent to all San Francisco retrofit project consultants, California Department of Transportation, November 22, 1989 [CT043].

Synopsis: First of a series of four informational guideline letters sent to the six primary consultants for San Francisco retrofit projects. The four letters (dated November 22, December 5, December 10, and December 30, 1989), provide basic instructions on the retrofit parameters and assumptions that the consultants are to use in their designs.

Zelinski, Ray J. "California Department of Transportation Bridge Earthquake Retrofitting Program," *U.S./New Zealand Workshop.* May 1985 [PAS005].

Synopsis: Describes Caltrans seismic retrofitting program since the 1971 San Fernando earthquake. The retrofitting program consisted primarily of installing cable restrainers at the joints to prevent longitudinal separation of spans. A total of 1,247 bridges were retrofitted by 1985 out of a total number of 12,000 bridges in California. The bridges to be retrofitted were selected according to a prioritizing scheme that considered frequency of earthquakes in the area, replacement cost, retrofit cost, detour length, average daily traffic, defense and emergency routes, and the facility crossed by the bridge. The design of cable restrainers was assisted through elastic dynamic and equivalent static analyses.

__________. "Shift Happens," *Engineering & Science*. California Institute of Technology, Volume LIII, Number 2, pp. 2-12, Winter 1990 [VAR070].

Synopsis: This article is an overall summary and analysis of "California's best-studied earthquake." The focus is on the instrumental record and previous predictions of a major earthquake in the Loma Prieta epicentral region. Extensive descriptions of some of the new recording and analysis instruments is provided.

Annotated List of References Maintained at the California Institute of Technology and Obtained from Caltrans Press Room Inventory

Note: The material listed below was obtained from the Caltrans press room and was available for inspection by members of the Board of Inquiry at the California Institute of Technology in Pasadena. Reference numbers refer to the inventory numbers assigned by Caltrans.

A-1) *Cypress Street Viaduct —10th St. to Distribution.* July 26, 1957.

A-2) *Cypress Street Viaduct —Earthquake Upgrading.* August 26, 1977.

A-4) *Cypress Street Viaduct —Market to 11th St.* October 6, 1955.

A-6) *Cypress Street Viaduct —Magnolia to 17th Street.* June 10, 1967.

A-9) *Embarcadero Freeway Viaduct* (5 parts).

Howard Street to Broadway, Test Pile. c1956.

Howard Street to Broadway. c1958.

Clay-Washington Street Ramps. c1965.

Transverse Earthquake Restrainers. December 9, 1985.

B-1) "Cypress Street Project," *California Highways& Public Works.* July/August 1957. Article about the project.

B-2) *Design News #5.* February 14, 1956. House organ article about prestressed caps at Cypress Street Viaduct with provisions for future widening.

B-3) *Design News #8.* April 1956. House organ article about plastic scale model used to study two way deflections at Cypress Street Viaduct.

B-4) *Design News #21.* March 19, 1967. House organ article about full scale load test of a bent cap at Cypress Street Viaduct.

B-5) "California Double Decks & Prestressed Freeway," *Engineering News Record.* September 25, 1955. Article about the first stage construction.

B-6) "California's First Double Deck Freeway," *Engineering News Record.* July 18, 1967. Article about the second stage construction project.

B-7) "Cypress St. Viaduct, Major Construction Project," *Highway Magazine.* March 1958. Article about construction, by the resident engineer.

B-8) "California Cypress St. Viaduct, Major Construction Project," *Better Roads.* November 1958. Article about construction, by the resident engineer.

B-11) "District IV Freeways Make Great Strides," *California Highways & Public Works.* March-April 1955. Article includes 'skyways' under construction.

bb) *Specifications for Design of Highway Bridges, 1949.* AASHO specifications as modified by the Division of Highways. Mimeograph format.

C) *Bridge Maintenance Inspection Reports and related correspondence—Cypress Street Viaduct.* 33 items dated April 10, 1957 through April 11, 1989.

D-1) Memo (dated Oct. 19, 1987) to Leo Trombatore, Caltrans Director, from W. E. Schaefer, Deputy Director for Project Development. Issue memo recommending acceleration of Phase 2 retrofit program to $16 million per year.

D-2) Memo (dated Oct. 22, 1987) to Governor George Deukmejian, via Secretary of Business, Transportation and Housing Agency, from Leo J. Trombatore. Issue memo recommending acceleration of Phase 2 retrofit program to $16 million per year. Approved at Agency level 12-4-87.

D-3) Memo (dated Oct. 26, 1987) to R. J. LeBeau from Jim Gates. In house memo: Percent of Phase 2 needs by north/south split; report of Phase 1 attached, 1262 bridges @ 54.2 million; description of Phase 2 attached.

D-4) Memo (dated Nov. 25, 1987) to W. E. Schaefer from Jim Roberts, Chief, Division of Structures. Expounds on value of Phase 2 program.

D-5) Memo (dated Dec. 14, 1987) to All District Directors, from Department of Transportation, Division of Highways, Allan Hendrix, Chief. 1988 STIP sets aside $64 million for Phase 2 retrofit over 4 year period.

D-6) Memo (dated Jan. 4, 1988) to Supervisors & Above, from Division Of Structures, J. E. Roberts, Chief. $16 million per year approved for Phase 2. Get projects ready beginning July 1988.

E-1) Fact Sheet (10-23-89). Chronologic listing of retrofit funding appeals, following Whittier earthquake of 1987.

E-2) Status—Listing of damaged bridges, updated daily: 10-18 9:00am, 10-18 10:55am, 10-20 8:55am, 10-23, 10-24 6:45 am, 10-26 9:30am, 10-31 8:00am, 11-3 9:00am, 11-6 7:20am, 11-8 7:00 am.

E-3) Seismic Research (10-23-89). One page status of seismic research since 1971.

E-4) Scenario of effects due to hypothetical loss of various transportation facilities from a magnitude 7.4 seismic event on the Hayward Fault; Jim Gates', DOS, response to Jack Bennett, Division of Mines and Geology. A planning exercise.

F-1) 3-23-88—Correspondence to Stephen Moore, citizen, from Jim Roberts. Describes Phases 1 and 2 of seismic retrofit program. (Exhibit 7A)

F-2) 6-7-88—Correspondence to Goveror Deukmejian from Stephen Moore. Critical of viaduct aesthetics and hinge seats and recommends putting Phase 2 on hold. (Exhibit 11)

F-3) 6-15-88—Correspondence to S. Moore from J. Roberts, referral from Governor, Response to 6-7-88 correspondence. (Exhibit 7B)

F-4) 7-18-88—Correspondence to J. Roberts from S. Moore. Response to Roberts' response of 6-15-88. Still critical of structural adequacy of Phase 1 retrofit techniques. (Exhibit 2)

G-12) State Transportation Improvement Plan. Annual listing of every highway capital improvement project planned to begin within the next 5 years.

H-1) Memo 10-23-89 to All Employees from W. E. Schaeffer. Thanks for immediate response to earthquake disaster.

J) "Theory of California Seismic Bridge Design & Analysis for the Beginner." A paper by J. Roberts and B. Marony, DOS, c1989.

K) Bridge Maintenance Inspection Reports—San Francisco-Oakland Bay Bridge, East Bay Spans. (36 reports dated 12-80 through 8-8-89)

L) "Bridge Seismic Retrofit Program for California Highway System." Technical paper describing seismic retrofit program. By J. E. Roberts, c1988.

M-1) Summary of Retrofit Project Expenditures, by fiscal years, by county and district. (Project status, Phase 1 retrofit)

N) "New Concepts in Earthquake Retrofitting on Highway Bridges." By G.D. Mancarti, DOS 1984. A

paper presented to 6th Northwest Bridge Engineer's Seminar, October 1984.

O & O-2) "California DOT Bridge Earthquake Retrofitting Program." By R. J. Zelinski, c1985. A paper presented to US/Japan and US/New Zealand seismic workshops.

P) "Retrofitting Techniques for Highway Bridges," Bridge Notes. Vol. 20, #1, August 1978. House organ article by Oris Degenkolb.

Q) "Retention of Project Records." In house instruction from Memos to Designers.

R) Report of Status of Phase 2 retrofit Program, May 23, 1988.

R-1) Five in house memos proposing candidate structures and funding for Phase 2 retrofitting.

U-1) *Bridge Design Practice*—In house manual, correspondence course on Load Factor Design for bridges.

U-2) *Bridge Design Specifications*—In house manual, AASHTO Specifications as modified by Caltrans.

U-3) *Bridge Computer Manual*—In house manual, instructions for various computer programs available.

U-4) *Bridge Design Details*—In house manual, instructions for draftsmen and reduced copies of standard drawings available.

U-5) *Bridge Memos to Designers*—In house manual, instructions for bridge designers.

U-6) *Bridge Design Aids*—In house manual, bridge design charts.

U-7) *Bridge Design System*—In house manual,instructions for computer programs available.

U-8) *Bridge STRUDL*—In house manual, instructions for STRUDL structural analysis program.

W) Listing of Phase 2 retrofit projects under design as of September 21, 1989. Same as part 10 of item R.

X) "Bridge Details Resulting from Experience Gained from L.A. Earthquake of February 9, 1971," March 16,1971. New seismic criteria to be applied to structures under design and construction.

Y) Map—"Peak Acceleration from Maximum Credible Earthquakes in California," acceleration contours. Map by Division of Mines and Geology.

1—Phase 2 Retrofit Program correspondence file, July 28, 1986 through August 23, 1989.

2—Whittier Narrows Earthquake of 1987 research project correspondence file, November 19, 1987 through August 18, 1989.

4—Definitions of seismic related terms, by Professional Engineering Registration Program.

6—Final minutes of Peer Review Panel meeting, 8-21-89, a volunteer group of structural specialists reviewing seismic accommodations on the Embarcadero Viaduct.

7—"Something New," *California Highways and Public Works*. January-February 1957. Article about double deck viaducts under construction in Oakland and San Francisco. Was also on the press room table at 1120 N Street, but was not numbered or catalogued.

8—In-house memo, 10-26-89, to J. E. Roberts from A. P. Bezzone re: lateral column ties and AASHTO Code, responding to Jim See's accusations.

8a—Letter of clarification to California Transportation Commission, 10-26-89 by Jim See.

9—Phase 2 earthquake retrofit projects, total $63.3 million. Status of Phase 2 projects. An update of part 10 in item R.

14—"Nimitz Freeway Collapse," Los Angeles Times. October 26, 1989. newspaper article.

15—Recap of earthquake related closures, 10-18-89, Caltrans teletype listing roads closed due to earthquake.

20—Project Consultants. List of consulting Engineers hired for emergency repair design work.

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Figures 3-2 through 3-4, Charles C. Thiel Jr.; Figure 3-6, Caltrans; Figure 3-7, Pacific Aerial Surveys; Figure 3-8, Nims/Aiken/Miranda; Figures 3-11 through 3-13, Caltrans; Figures 8-1 through 8-6, National Information Service for Earthquake Engineering, Caltech; Figure 8-8, Caltrans; Figures 9-1 through 9-3, Moulin Studios, San Francisco; Figure 9-13, Caltrans; Figure 9-16a, Caltrans; Figure 9-16b, Abolhassan Astaneh; Figure 9-17, Caltrans; Figures 9-18 through 9-20, Abolhassan Astaneh; Figures 9-23 and 9-24, Abolhassan Astaneh; Figure 10-2, Nims/Aiken/Miranda; Figures 10-3 and 10-4, Caltrans; Figures 10-23 through 10-49, Nims/Aiken/Miranda; Figure 10-51, Caltrans; Figure 11-21, Caltrans; Figure 11-23, Caltrans; Figures 11-25 through 11-27, Caltrans.

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