THE RACE TO SEISMIC SAFETY

PROTECTING CALIFORNIA'S TRANSPORTATION SYSTEM

Submitted to the
Director, California Department of Transportation

by the
Caltrans Seismic Advisory Board
Joseph Penzien, Chairman

December 2003

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:

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

Competing Against Time
Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake

It is the policy of the State of California that seismic safety shall be given priority consideration in the allocation of resources for transportation construction projects, and in the design and construction of all state structures, including transportation structures and public buildings.

Governor George Deukmejian
Executive Order D-86-90, June 2, 1990

The safety of every Californian, as well as the economy of our state, dictates that our highway system be seismically sound. That is why I have assigned top priority to seismic retrofit projects ahead of all other highway spending.

Governor Pete Wilson
Remarks on opening of the repaired Santa Monica Freeway
damaged in the 1994 Northridge earthquake, April 11, 1994

The Seismic Advisory Board believes that the issues of seismic safety and performance of the state's bridges require Legislative direction that is not subject to administrative change.

The risk is not in doubt. Engineering, common sense, and knowledge from prior earthquakes tells us that the consequences of the 1989 and 1994 earthquakes, as devastating as they were, were small when compared to what is likely when a large earthquake strikes directly under an urban area, not at its periphery. Geological science makes it clear that such an event will happen.

California must complete this race. When great earthquakes occur, we must be ready—no matter what part of the state is heavily shaken. To do any less is to concede responsibility and accept the consequences of loss of life and widespread economic disruption when truly large California earthquakes occur.

Earthquakes measure our actions, not our words.

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Protecting California's Transportation System
Caltrans Seismic Advisory Board, 2003

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Joseph Penzien, Chairman
Frieder Seible, Vice-Chairman
Bruce A. Bolt
I.M. Idriss
Joseph P. Nicoletti
F. Robert Preece
James E. Roberts

Charles C. Thiel Jr., Editor

December 2003

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Joseph Penzien
Chairman

Report Organization

This report is organized in two parts. Sections 1-3 review the findings and recommendations of the Seismic Advisory Board and are meant for all readers. Sections 4-13 provide technical details of the basis for the recommendations of the SAB. For reference purposes, Attachment 1 reproduces Governor George Deukmejian's Executive Order D-86-90, and Attachment 2 provides the findings and recommendations from the two immediate predecessors of this report: *Competing Against Time*, and *The Continuing Challenge*. In many ways, the findings of those reports are still current and warrant examination. The Seismic Advisory Board has considered carefully these previous findings and recommendations, and still supports them today as current and worthy of guiding the future state program.

Seven Recommendations for a Safer Transportation System

The Seismic Advisory Board has reviewed current and past Caltrans bridge seismic design practices and makes the following seven recommendations to help California achieve a safe transportation system:

1. **Seismic Saftey Policy.** The California Legislature should establish as state policy the current Caltrans practice: "It is the policy of Caltrans—to the maximum extent feasible by present earthquake engineering practice—to build, maintain, and rehabilitate highway and transportation structures so that they provide an acceptable level of earthquake safety for users of these structures."

2. **Non-State-Owned Bridge Retrofits.** The Legislature should provide timetables for the seismic retrofit of non-state-owned bridges so that those bridges requiring retrofit are completed within the next 5 years. The standards for non-state-owned bridges should be the same as for state-owned bridges.

3. **Design Standards.** Caltrans should maintain its standards for construction and retrofit of bridges and other transportation structures to provide life safety for all structures and functionality for lifeline and other important structures following an earthquake. Further, Caltrans should maintain its current policy that seismic-related design and construction issues be independently reviewed to ensure compliance with these standards. Selective seismic peer reviews should be conducted under policies and procedures reviewed by the Seismic Advisory Board (SAB).

4. **Regular Safety Reassessment.** Caltrans should regularly reassess the seismic hazard and engineering performance of bridges, including existing, retrofitted, and new structures. Caltrans should determine, as measured by the then-current state of knowledge, whether bridges and transportation structures can be expected to perform in an acceptable manner under earthquake shaking.

5. **Toll Bridge Seismic Safety Program.** The Toll Bridge Seismic Safety Program needs to be completed efficiently and without further delay.

6. **Problem-Focused Investigations.** Caltrans should continue its commitment to problem-focused seismic investigations at or above its current level.

7. **Emergency Response.** Caltrans should maintain its rapid response capability to evaluate, repair, and restore damaged bridges, regardless of the cause—whether natural or terrorist.

Executive Summary

Will future California earthquakes again cause destruction of portions of California's transportation system, or will their impacts be controlled to limit the damage and disruption any large earthquake will cause?

This is the key question addressed by the Caltrans Seismic Advisory Board in this report. Much has been accomplished, but more remains to be done. The highest priority goals are repairing and retrofitting state-owned and state-maintained critical bridges. However, there are still hundreds of other vulnerable bridges owned by local or regional agencies that require analysis and retrofit. The Caltrans Seismic Advisory Board (SAB) has assessed the current state of affairs and concluded that there are seven priority recommendations that need to be acted on if the state is not to re-experience the calamities of the past—loss of life, collapse of highway bridges, and billions of dollars in disruptions to California's economy.

The Seismic Advisory Board believes that unless the seven SAB-recommended actions are taken, the answer to the above question is that the state faces destruction of many bridges and highway transportation structures in a large earthquake, not an outcome of limited impacts. Modern state bridges have yet to experience a Big One—a major earthquake in an urban area, as happened in the San Francisco Bay Area in 1906. Ongoing tectonic deformations in California make it absolutely certain that it will happen. No one knows how soon, or where it will occur. Will we be ready? The race is on and we are in it whether we choose to run or not.

It is the conclusion of the Seismic Advisory Board that the state is at a crossroad. It must maintain a focus on seismic safety or pay the consequences. As every Californian is aware, the state's budgetary crisis is severe. However, reducing financial support for bridge seismic design has been shown to have devastating consequences for the state. Following the 1971 San Fernando earthquake, Caltrans expanded its earthquake engineering research support, initiated a retrofit program with cable restrainers, and administratively formed an earthquake engineering group. Later in the decade, when the Governor and the Legislature changed priorities under budgetary pressures, the group was disbanded and the assigned department staff scattered. Earthquake safety for California's highways was once again just another competitor for state highway funds—not a priority. The Loma Prieta and Northridge earthquakes—both of which caused catastrophic bridge failures—underscored the cost of this change in priorities.

In 2002, Assembly Bill 2996 eliminated the separate Seismic Safety Retrofit Account and put seismic safety projects once again in direct competition with construction and maintenance projects under the State Highway Account. The process of de-emphasizing seismic safety has already begun.

The SAB is using this report, which it has entitled *The Race to Seismic Safety*, to urge both Caltrans and the state Legislature to codify and institutionalize practices that will lead to adequate seismic performance of the state's transportation structures.

It is truly a race against the certainty of future earthquakes. If California is to win this race, then as a state we must reinforce our resolve and redouble our efforts to build a highway transportation system that can deliver statewide life-safe performance in a large earthquake.

The issue is not to increase significantly the resources applied to the task, but to complete the job begun in earnest in 1989, resolve those safety issues not yet addressed, and maintain the seismic safety commitment so that highway bridges and other transportation structures will perform at an acceptable level of safety and functionality in future earthquakes.

The Seismic Advisory Board believes that the issues of seismic safety and performance of the state's bridges require Legislative direction that is not subject to administrative change.

The risk is not in doubt. Engineering, common sense, and knowledge from prior earthquakes tells us that the consequences of the 1989 and 1994 earthquakes, as devastating as they were, were small when compared to what is likely when a large earthquake strikes directly under an urban area, not at its periphery. Geological science makes it clear that such an event will happen.

California must complete this race. When great earthquakes occur, we must be ready—no matter what part of the state is heavily shaken. To do any less is to concede responsibility and accept the consequences of loss of life and widespread economic disruption when truly large California earthquakes occur.

Earthquakes measure our actions, not our words.

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Section 1

Status of Bridge Safety

Every Californian who drives is aware of the significant, statewide effort to seismically retrofit bridges. To date, the Caltrans race to seismic safety has resulted in the retrofit of over 2,000 of the most vulnerable highway bridges in an effort to improve their earthquake performance and provide a life-safe and reliable highway transportation system.

However, over time, memory of disastrous events is diminished and actions to moderate or avoid them in the future often become less resolute. This is certainly the experience with seismic hazard in California. Before the 1971 San Fernando earthquake, it was assumed that operational loads on bridges presented more severe structural requirements for bridges than did earthquakes. The San Fernando earthquake of 1971 (magnitude 6.6) dramatically illustrated the error of this assumption and the public paid for the consequences in out-of-service roadways. Following the San Fernando earthquake, there was a concerted effort to retrofit bridges, but the resolve eventually diminished. While the interest of Caltrans remained strong, the resources were unavailable.

The 1987 Whittier Narrows earthquake rekindled interest in highway bridge safety when a bridge failed in that moderate magnitude 6.0 earthquake. Only after the 1989 Loma Prieta earthquake (magnitude 6.9), with its spectacular bridge failures, was there a fundamental change in attitude at the state level with respect to the seismic safety of transportation structures. The 1994 Northridge (magnitude 6.7) earthquake reinforced commitment to achieving adequate bridge performance for California. It is now almost 10 years since the Northridge earthquake, so the same decay of interest is of critical concern. The state should reaffirm its commitment to maintaining a seismically safe highway system.

The effort to produce a seismically safe transportation system requires quantifying the seismic hazard of sites and predicting the response of new and existing structures to earthquakes. It is not technically possible or economically practical to expect that all bridges will be undamaged when an earthquake occurs. Indeed, the seismic performance goal is that a standard bridge will remain life-safe, but may be significantly damaged, possibly beyond repair, and important bridges on lifeline routes remain in service. The demand following an earthquake is on the restoration of the economic functioning of the community, in which highways and bridges provide vital links, and without which recovery is impeded.

The Governor's Board of Inquiry on the 1989 Loma Prieta earthquake warned in 1990 that the effort to make bridges safe would take significant, sustained investments in retrofit construction and research to learn how to build better bridges. Following the Loma Prieta earthquake, the California Department of Transportation (Caltrans) established the Caltrans Seismic Advisory Board (SAB) to provide advice on seismic policy and technical practices. The devastating 1994 Northridge earthquake again illustrated both the extent of the problem and the consequences of not taking action. The SAB was charged with advising the Director of Caltrans on issues of importance in achieving the Caltrans obligation to provide seismic safety for California's transportation structures. In the 1994 SAB report to the

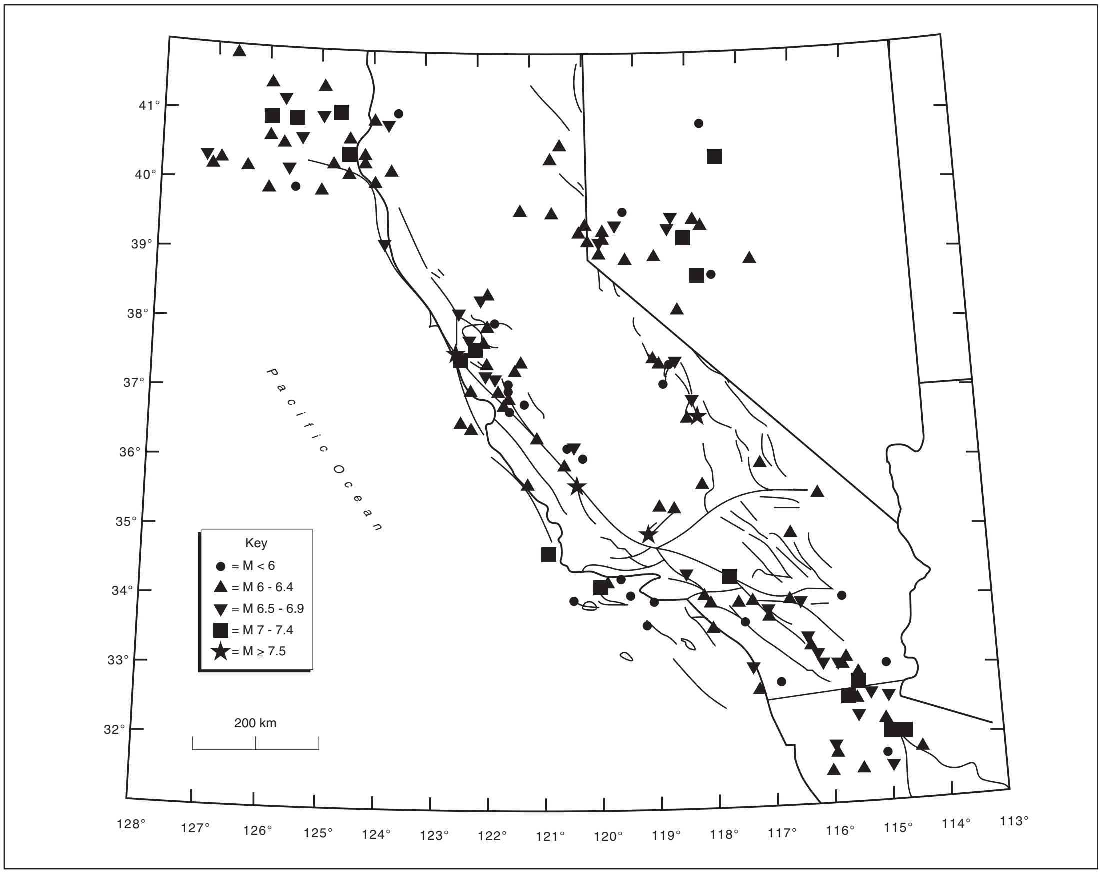


Figure 1-1. *Significant large eartquakes in California since the 1800s.*

Director of Caltrans, *The Continuing Challenge* (Housner et al. 1994), the Seismic Advisory Board urged both Caltrans and Governor Pete Wilson to take action on many fronts to rectify the earthquake hazard posed by bridges. The response of Caltrans is to be commended, and great strides have been made.

Today, however, the state's fiscal and administrative environments have changed. With California's deepening budget deficit crisis, seismic performance of the transportation system could again be pushed to a back burner. The Seismic Advisory Board believes strongly that bridge seismic design and the safety of transportation structures should be a priority for the State of California.

This report, *The Race to Seismic Safety*, was prepared by the Seismic Advisory Board to document the accomplishments and advances made by Caltrans since 1989 and to provide guidance for resolving outstanding safety issues so that highway bridges will perform at an acceptable level in future earthquakes. As of this writing (December 2003), there is still much to do to ensure the safety and reliability goals for California's approximately 24,000 state and local highway bridges.

1.1 Actions Following the 1971 San Fernando Earthquake

The seriousness of highway earthquake safety was first identified in 1971 when the Interstate 5 (I-5) Golden State Freeway Interchange in the San Fernando Valley collapsed during the magnitude 6.6 San Fernando earthquake. The interchange was under construction at the time of the earthquake. The San Fernando earthquake occurred at the north end of the San Fernando Valley, where a limited number of bridges were in place, and its impacts on bridges were concentrated in this area. Because of the design characteristics of the I-5 Interchange, it was determined that there were two principal failure modes:

- Failure of the high, single-column supports of the connectors and overcrossings.
- Separate sections of the bridge could move independently, accumulating excessive relative displacements at the supporting movement joints greater than the joints could accommodate. This resulted in span unseating and bridge collapse under gravity loads.

These findings led to a statewide Caltrans retrofit program to install cable restrainer devices on existing highway bridges. Cable restrainers limit the relative displacement across joints, and thereby prevent individual sections from falling. Many other seismic issues relating to state and local bridges were identified, and research into their resolution initiated. Budget limitations in the late 1970s eventually limited the principal actions by Caltrans to installation of cable restrainers.

1.2 Actions That Followed the 1987 Whittier Narrows Earthquake

In 1987, the relatively small magnitude 6.0 Whittier Narrows earthquake nearly caused the collapse of a freeway bridge, again over Interstate 5. This time it was a bridge of common type—it was not high and had short multiple-column supports. This magnitude 6.1 earthquake, and its accompanying ground motion, was smaller than was then commonly believed by engineers to represent a threat to bridges and transportation structures. This unexpected damageability caused an accelerated effort by Caltrans to understand how bridges perform in earthquakes and how to better design them so they do not fail. The new effort following the 1987 earthquake was modest and consistent with state and Caltrans budget priorities at that time. The Column Retrofit Research Program, already underway, provided the technical means to respond to the bridge performance problems exposed in the earthquake and the impetus to continue.

1.3 Advances Following the 1989 Loma Prieta Earthquake

The wake-up bell tolled tragically on the evening of October 17, 1989 when the magnitude 7.0 Loma Prieta earthquake caused unprecedented damage to bridges throughout the San Francisco region. The two-level elevated Cypress Street Viaduct collapsed in Oakland, killing 41 people. A section of the San Francisco-Oakland Bay Bridge failed, causing the bridge to be out of service for 30 days. Highway bridge closures caused immeasurable economic losses and disruption—even for bridges over 60 miles from the epicenter, where seismic ground motion was low enough that only a few of the most vulnerable buildings were damaged at that distance.

Following the 1989 Loma Prieta earthquake, Governor George Deukmejian appointed a Board of Inquiry to assess transportation deficiencies and how to fix them. The Board's work resulted in the influential report entitled *Competing Against Time* (Housner et al. 1990; see also Attachment 1 to this report). Governor Deukmejian issued Executive Order D-86-90 to implement the Board's principal recommendations. The order stated:

> *It is the policy of the State of California that seismic safety shall be given priority consideration in the allocation of resources for transportation construction projects, and in the design and construction of all state structures, including transportation structures and public buildings.*

With this Executive Order, earthquake safety issues became, for the first time in California, a specified primary goal for all construction—not a secondary or tertiary goal that would be addressed only if the funds were available. Subsequently, the Legislature and the public approved a bond issue, which provided the resources needed to begin addressing and resolving the problem of how to design bridge and transportation structures to perform acceptably under the influence of strong earthquake ground motions.

1.4 Advances Following the 1994 Northridge Earthquake

In 1994, the earthquake bell again rang. The magnitude 6.7 Northridge earthquake struck the San Fernando Valley with losses that added up to over $50 billion in direct damage, several times more than in the Loma Prieta earthquake. Damage to highway bridges was far greater than expected by the public, although retrofitted bridges performed well and the damage was generally limited to those that had not yet been retrofitted. One of the primary freeway arterials of the region, the Santa Monica Freeway (I-10) on the Wilshire corridor failed. Herculean efforts by Caltrans and its contractors repaired and restored the freeway within 90 days. In sharp contrast, it took years to replace the Cypress Street Viaduct after 1989 (still not completed when the Santa Monica Freeway was already back in service).

The Santa Monica Freeway was only one of the freeways damaged by the Northridge earthquake. Caltrans efforts to repair damage to SR-118, I-5, and SR-14 were rapid and effective. The Northridge earthquake occurred in a region of Los Angeles that had already been tested by the San Fernando earthquake 23 years earlier. Ground shaking in the Northridge earthquake at the Santa Monica Freeway was modest enough that only the most vulnerable nearby buildings were affected, yet Santa Monica Freeway bridges failed. Many of the bridges damaged in 1994 had already been identified as requiring retrofit. Unfortunately, the earthquake occurred before the work was done.

1.5 Completing the Task: Building the Seismic Safety of California's Bridges and Transportation Structures

The Governor's Board of Inquiry warned in 1990 that the effort to make bridges safe would take significant, sustained investment in retrofit construction and research to learn how to build better bridges. Only parts of the Board's 1990 recommendations had been implemented by the time of the 1994 Northridge earthquake. The Caltrans Seismic Advisory Board, comprised of many of the members of the earlier Board of Inquiry, assessed the risk in its 1994 report *The Continuing Challenge*, in which the SAB urged Caltrans and Governor Pete Wilson to take action on many fronts. In response to these recommendations, Caltrans and the state government renewed and redoubled their

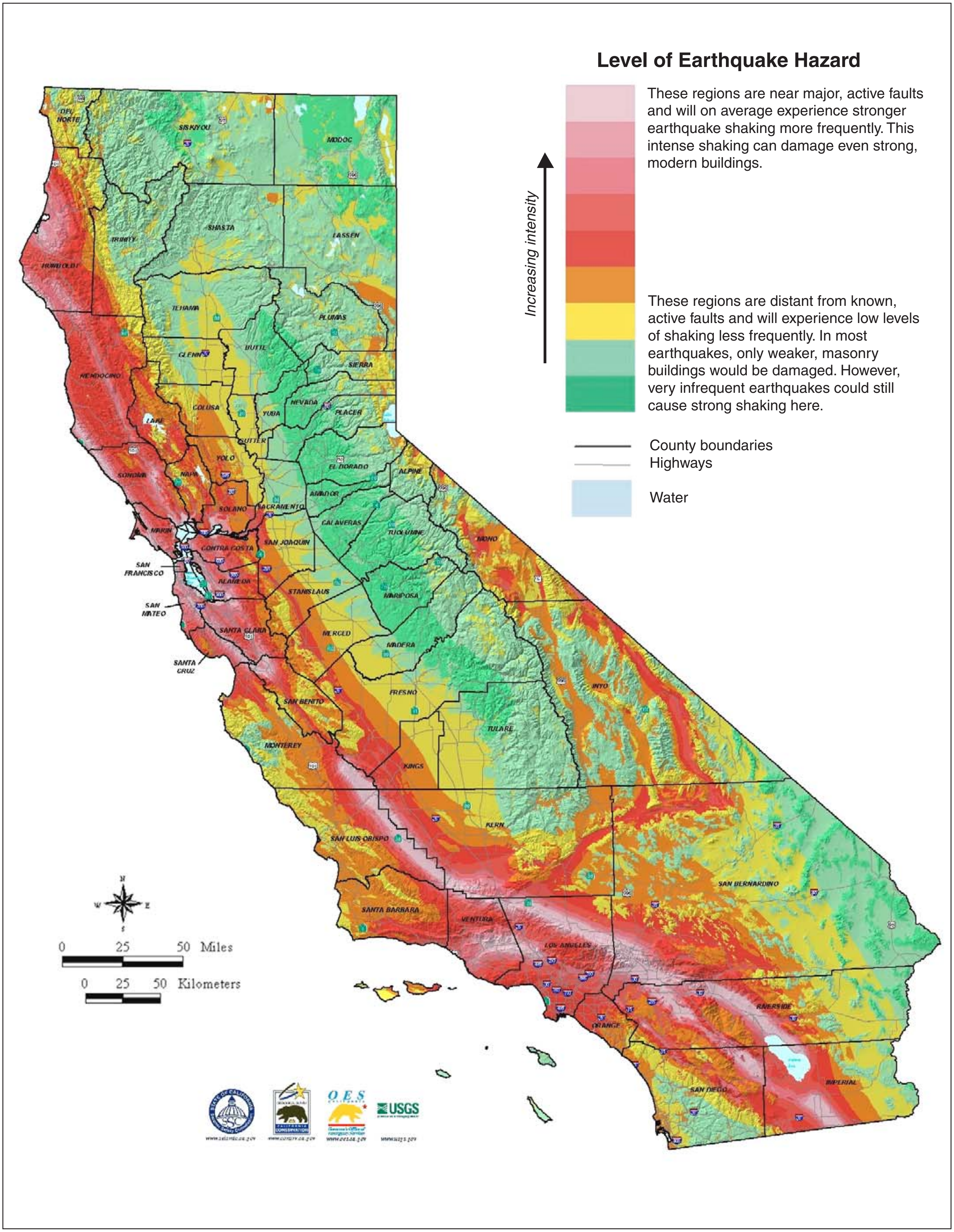


Figure 1-2. *Seismic hazard map (Caltrans).*

efforts with a significant design and construction program to mitigate the hazards posed by California bridges.

There can be no question that the seismic hazard of the State of California has an impact on every one of its citizens, both in the form of direct damage and loss, or from earthquake-caused taxes and disruption of economic activity. A plot of major earthquakes since the 1800s (Figure 1-1) shows how seismically active California is. This is a short time period of experience. The probabilistic hazard map (Figure 1-2) gives a clearer view of the state's risk.

The regions affected by the recent 1994 Northridge and 1989 Loma Prieta earthquakes were large, but greater magnitude earthquakes are in the offing (USGS 1988), and may be focused directly under population centers instead of some distance away.

At the time of the present report (December 2003), Caltrans has substantially achieved its highest priority goals in repairing and retrofitting state-owned and state-maintained critical bridges. Nevertheless, there are still many bridges that require analysis and retrofit to newly updated criteria.

From the evidence available, the SAB concludes that other state and local agencies have only a fair to poor record to date in mitigating the seismic hazard of their bridges and transportation structures—with the exception of Santa Clara and Los Angeles counties, which have essentially completed the task. Interestingly, these two counties were the hardest hit by the 1989 and 1994 earthquakes.

Significant seismic performance deficiencies of major bridges identified following the San Fernando, Loma Prieta, and Northridge earthquakes are being addressed by Caltrans. Seismic retrofit programs are underway or completed for most of the major toll bridges. The replacement of the San Francisco-Oakland Bay Bridge East Spans and the construction of the new Benicia-Martinez Bridge are in process. The new toll bridge at the Carquinez Strait is complete. The Golden Gate Bridge District is moving ahead with retrofit work on the Golden Gate Bridge. Considering how high the seismic threat is, it is essential that efficient completion of these major toll bridge seismic safety projects be completed without further delay. For other state-owned bridges, the first wave of seismic retrofit has reduced the damageability of these bridges to an acceptable life-safe (that is, non-collapse) level. But what of the rest? Many existing bridges, accepted by preliminary screening, should be reevaluated in light of current understanding of their structural performance. In addition, a number of gaps in the seismic safety of the state's bridges and transportation structures may still remain.

It has been nine years since the 1994 Northridge earthquake, our most recent wake-up call. Much remains to be done, and just like in the past, the resolve to complete the job is waning with the passage of time. What remains to be done is both prodigious and vital. The race is on to complete the task before the next Big One strikes and finds us unprepared.

Section 2

Recommendations for Action

The Seismic Advisory Board submits the following seven Recommendations to make California's transportation system life-safe and, for important bridge structures, ensure that they continue functioning after an earthquake.

These Recommendations are intended to increase the seismic safety of California's bridges and transportation structures and avoid the complacency caused by the absence of recent major, damaging earthquakes that force us to pay attention. These recommendations are action-driven. It is the SAB's intent that after a future major California earthquake, the state will not have to explain why the highway transportation system did not perform as the public expected.

2.1 Seven Transportation Recommendations of the Caltrans Seismic Advisory Board

1. **Seismic Safety Policy.** The California Legislature should establish as state policy the current Caltrans practice: "*It is the policy of Caltrans—to the maximum extent feasible by present earthquake engineering practice—to build, maintain, and rehabilitate highway and transportation structures so that they provide an acceptable level of earthquake safety for users of these structures.*"

Discussion: Such practice has been the policy of Caltrans since the 1989 Loma Prieta earthquake, but it has not been articulated as an agency mission or responsibility.

To ensure that the public's interest in safety is met, the state has committed in a continuing way—through enforcement of independent technical review—to the safe construction of schools, hospitals, and public and private buildings. The Caltrans SAB considers it appropriate that highway structures be held to the same standard of performance as other constructed facilities vital to the safety and robust functioning of the economy.

The Seismic Advisory Board recommends that the above Caltrans policy be formally adopted by the Legislature to ensure adequate seismic performance of the state's highway system now and in the future. Current Caltrans actions remain consistent with this recommendation, as originally directed by Executive Order D-86-90.

The Seismic Advisory Board believes that the issues of seismic safety and performance of the state's bridges require Legislative direction that is not subject to administrative change.

Caltrans is also responsible for non-highway bridges in California, but its responsibility is limited to those structures where a failure could adversely impact the state highway system, e.g., railway bridges over state highways. However, where railway bridges do not affect the state's highways, the state does not impose safety requirements for their design and construction. It is important to note that the economic recovery of the state after a damaging earthquake may be limited by the performance of these other bridges, and the speed with which other transportation modes are restored to pre-earthquake service condition.

Implementing this seismic safety policy will require a continuous commitment of resources similar to that of the last decade.

2. **NON-STATE-OWNED BRIDGE RETROFITS.** The Legislature should provide timetables for the seismic retrofit of non-state-owned bridges so that those bridges requiring retrofit are completed within the next 5 years. The standards for non-state-owned bridges should be the same as for state-owned bridges.

Discussion: Caltrans has proceeded well in upgrading the seismic safety performance of its bridges. However, many local (city and county) agencies have not addressed their bridge safety issues with the same completeness. This is despite the fact that the Federal Highway Administration (FHWA) and Caltrans funded assessment of non-state-owned bridges, identified those that potentially require seismic retrofit, and developed retrofit plans for those needing them. Caltrans even provided that the state and federal governments would reimburse 100 percent (80 percent federal, 20 percent state) of the design and construction cost as a response to recent earthquake recovery efforts. Yet many jurisdictions have been slow in responding and addressing the safety of their bridges. As of the beginning of 2003, local agencies are no longer automatically receiving the 20 percent state matching funds since the Seismic Safety Retrofit Account was eliminated by AB2996. Local agencies are now required to provide 20 percent of the total cost of the retrofits from their respective allocations of State Highway Account funds. As a result, local actions to complete the retrofit task have been greatly reduced.

Currently, only about 37 percent of non-state-owned bridges with identified deficiencies have been retrofitted and an additional 12 percent are under construction. This compares with an over 90 percent completion rate for state-owned bridges.

Even if users of the highway system do not understand who controls what bridges, they should not be misled into thinking that seismic safety issues have been addressed for these structures when in fact a non-state-owned portion of the system has been unresponsive—even when costs were covered.

The objective of retrofit of non-state-owned highway bridges is to provide a reliable highway transportation system in the post-earthquake period, regardless of the jurisdictional issue of who is responsible for its individual elements. Evidence suggests that retrofit of the remainder of these local bridges will not happen unless it is mandated. Many local agencies control bridges that form the "backbone" of the transportation system, and without Legislative mandate, these bridges will probably not be retrofitted before the next big earthquake strikes.

3. **DESIGN STANDARDS.** Caltrans should maintain its standards for construction and retrofit of bridges and other transportation structures to provide life safety for all structures and functionality for lifeline and other important structures following an earthquake. Further, Caltrans should maintain its current policy that seismic-related design and construction issues be independently reviewed to ensure compliance with these standards. Selective seismic peer reviews should be conducted

under policies and procedures reviewed by the Seismic Advisory Board (SAB).

***Discussion*:** It is important to the economy and safety of the people of California that all bridges and other transportation structures in the state highway system, whether existing or newly constructed, have predictable seismic performance. A key question related to performance is "What level of safety is high enough?" While it is economically impractical to make every transportation structure fully damage-resistant, it is practical to expect that the lives of the users of these structures be as safe as is required for critical building structures, such as schoolhouses and hospitals. To achieve the desired performance, it is important that expected seismic performance be *independently* verified. Currently, it is Caltrans practice to use internal technical review, not independent external peer review, for most retrofit and new design. These are effective practices, but need policy direction to ensure continued action.

4. **Regular Safety Reassessment.** Caltrans should regularly reassess the seismic hazard and engineering performance of bridges, including existing, retrofitted, and new structures. Caltrans should determine, as measured by the then-current state of knowledge, whether bridges and transportation structures can be expected to perform in an acceptable manner under earthquake shaking.

Discussion: Caltrans has made great strides in the past decade in transforming California's state highway system into one with predictable, good seismic performance. During this period, a major retrofit effort has upgraded over 2,000 bridges (about 18 percent of the state inventory). Much has been learned about the technical issues of seismic performance. In some cases, there has been an improvement in our understanding of the reliability of retrofit approaches. Some structural details that were previously accepted without retrofit are now suspect. At the same time, there has been significant change in our understanding of the earthquake risk as geologists and seismologists have gained more understanding of California's seismic hazards. It is vital that the process of providing a reliable transportation system reflect these improvements in knowledge and understanding, both of the engineering performance of structures and the seismic hazard at the site.

Because a bridge has been reviewed as acceptable at one time or has been retrofitted, there is no assurance that it will still provide acceptable performance when knowledge and understanding of seismic performance have improved. The SAB believes that periodic reassessments should be initiated by new technical developments or on a set timetable—perhaps every 10 years since construction was completed. Under Federal Highway Administration (FHWA) requirements and the standards of the American Association of State Highway Officials (AASHO), every two years Caltrans conducts assessments of the structural condition of every state bridge. If the mandate is broadened to include visual assessment of expected seismic performance, where appropriate for the type of bridge or condition, then this program is an ideal vehicle through which to regularize seismic performance assessments. It has the potential advantage to the federal government that at small expenditures for assessment, large disaster recovery costs could be avoided.

5. **TOLL BRIDGE SEISMIC SAFETY PROGRAM.** The Toll Bridge Seismic Safety Program needs to be completed efficiently and without further delay.

Discussion: The design basis for the Toll Bridge Seismic Safety Program was well-developed and benefited from research findings. The engineering models and analysis procedures used are expected to reliably predict their performance. Extensive independent technical peer reviews give added confidence that the completed designs for those toll bridges are technically appropriate. While the seismic retrofit of most of the toll bridges is complete, retrofit construction is still underway on the Richmond-San Rafael Bridge and the West Spans of the San Francisco-Oakland Bay Bridge. Construction of the new Benicia-Martinez Bridge and the new East Spans of the San Francisco-Oakland Bay Bridge are underway. The work on toll bridges should be completed as a priority, so that it will be done before the next big earthquake.

6. **PROBLEM-FOCUSED INVESTIGATIONS.** Caltrans should continue its commitment to problem-focused seismic investigations at or above its current level.

***Discussion*:** A cornerstone of the significant improvements in bridge design in the past decade has been the commitment of Caltrans to a vigorous research program. Striking changes in seismic design and construction practices within Caltrans have resulted from problem-focused research and investigations to resolve critical issues, and to understand why bridges performed as they did in recent California earthquakes. Highly focused research has provided an economical means by which the seismic hazard of bridges has been mitigated.

The good performance of retrofitted bridges in the Northridge earthquake can be directly attributed to Caltrans research programs. The evolving post-Loma Prieta earthquake design and retrofitting practices used by Caltrans appear to be sound. No significant damage has been reported to the 60 bridges retrofitted by Caltrans in regions of strong shaking since the start of the post-1987 retrofit program. The retrofit techniques used included procedures that were developed principally from the research program completed for jacketing of bridge columns.

The innovative approaches in analysis and design developed from research have allowed most of the major bridges—including most of the toll bridges—to have their seismic performance improved by retrofit rather than replacement, with significant cost savings. In the case where retrofit was not appropriate (e.g., the East Spans of the San Francisco-Oakland Bay Bridge), techniques developed by Caltrans research efforts have allowed new design approaches that have improved seismic performance. These Caltrans efforts have brought about not only better understanding of the technical and professional issues of design and seismic performance of structures, but have also achieved economy in providing an acceptable level of seismic performance. The SAB believes it is vital that both the state and Caltrans continue this research commitment.

7. **EMERGENCY RESPONSE.** Caltrans should maintain its rapid response capability to evaluate, repair, and restore damaged bridges, regardless of the cause—whether natural or terrorist.

Discussion**:** Earthquakes are not the only possible cause of bridge failure. Accidents or terrorist acts can damage bridges in many of the same ways that earthquakes do. The technical response to restore a bridge damaged from either cause is similar. The means of making such assessments is rapidly changing and improving. For some structures, it may be possible to use real-time instrument response recordings to aid in making hard decisions on whether or not to continue to use a possibly damaged structure. As the capability to perform instrument-based evaluation procedures increases, such instrumentation should be integrated into the design and construction of highway transportation structures to facilitate rapid condition assessment. Development and implementation of effective recovery procedures will not only assist Caltrans in rapidly restoring its structures to service, but also improve the effectiveness and economy of Caltrans response and recovery assistance to other agencies, cities, and counties under the direction of the Office of Emergency Services.

2.2 Conclusions of the Seismic Advisory Board

It is the conclusion of the Seismic Advisory Board that the state is at a crossroad. It must maintain a focus on seismic safety, or pay the horrific consequences. As every Californian is aware, the state's budgetary crisis of 2003 is severe.

The 1971 San Fernando earthquake exposed the severity of the hazard posed by bridges in earthquakes. Caltrans expanded earthquake engineering research support, initiated a retrofit program with cable restrainers, and administratively formed an earthquake engineering group. However, later in the decade, when the Governor and the Legislature changed priorities under budgetary pressures, the program was curtailed and the assigned department staff scattered. Earthquake safety for California's highways was once again just another competitor for state highway funds—not a priority. The Loma Prieta and Northridge earthquakes made clear the potential cost of this change in priorities. The SAB believes that the discontinuation of the Seismic Safety Retrofit Account in 2002 could be the precedent for delays in the Local Bridge Seismic Retrofit Program unless action is taken.

Earthquakes measure our actions, not our words. If we are to win this race against future earthquakes, then as a state we will have to reinforce our resolve and redouble our efforts to yield a highway transportation system that can withstand the test. The issue is not to increase significantly the resources applied to the task, but to complete the job begun in earnest in 1989, resolve those safety problems not yet addressed, and maintain that commitment so that highway bridges will perform at an acceptable level in future earthquakes.

The race is on. We cannot stop until we reach the goal. The state must complete the race before the next severe earthquake so that it will not have to explain a failure to act. To do any less is to concede the race and accept that the consequences will be loss of life and widespread and severe economic disruption when truly large California earthquakes occur.

Section 3

Key Questions of Public Concern

The state is faced with a decision on whether to implement the seven Recommendations of the Seismic Advisory Board as contained in this report, and maintain the current effort to achieve adequate seismic performance of bridges, or to lessen or curtail these efforts. Stated plainly, the choice is whether to complete the formidable task of seismically protecting our transportation system or allow its safety to diminish and decay.

Anticipating the future is difficult—particularly when there are big choices to be made. There are, however, questions that can be addressed now that may help in understanding the dimensions of the choices to be made. Six questions are assessed below, ones that are, in the SAB's judgment, issues essential to understanding the choice between increasing bridge seismic safety efforts or abandoning them mid-stream. The SAB's answers to, and discussion of, these key questions are predicated on the assumption that the priority for seismic safety of bridges becomes a competing priority for Caltrans and local transportation agencies, not a number one priority as it has been for the past 15 years.

1. How will the transportation system perform in the coming big earthquake?

The answer depends on the type of bridge, and the service it provides. Given that current retrofit construction underway is completed, it is expected that major bridges on designated lifeline routes will perform well (termed critical bridges, for which there are no alternatives, e.g., San Francisco-Oakland Bay Bridge; there are relatively few of these bridges in the state). It is expected that these bridges will be closed for a short period of time while they are inspected and minor repairs completed to ensure safety.

Following a large earthquake, the SAB expects that many Standard bridges near the epicenter will be sufficiently damaged as to be out of service for a period of time, and some may require replacement. Collapse is not expected for most of these bridges, but repair for some may not be economical. Within urban areas, the differences between the performance of retrofitted bridges and those awaiting retrofit may be extreme, with retrofitted bridges performing much better.

2. Why is improving seismic safety a good investment?

The highway system is a vital element of the functioning of the California economy. If an earthquake damages the transportation system severely enough, there will be harsh impacts on California's economy and the market position of its businesses will suffer. It is clear from observations of California and foreign earthquakes that protecting the transportation system from catastrophic earthquake impacts saves lives and guards the state's ability to prosper and compete economically. Retrofit and strengthening structures protects them not just from the effects of earthquakes, but from acts of terrorists as well.

3. Why haven't the programs for retrofit of local bridges been completed?

A much smaller percentage of locally-owned bridges have been seismically retrofitted than have state-owned bridges.

Caltrans and the Federal Highway Administration (FHWA) provided the assessment, design, and construction funds for seismic retrofit of non-state-owned bridges as part of the post-earthquake recovery process. Seismic retrofit plans for most bridges that need it are complete. Now, however, in 2003, local communities must provide 20 percent of construction costs of a bridge project from their share of the State Highway Account. Consequently, many local communities see the more imminent need of maintenance and new construction projects; thus seismic retrofit progress for non-state-owned bridges has slowed to a snail's pace.

4. Has the retrofit program to date been effective?

Yes. It has been effective in that the seismic safety of those bridges assessed and modified is expected to be significantly better than those that have not been retrofitted. The state's goal, from Governors Deukmejian to Gray Davis, for seismic performance for the vast majority of state bridges is that they not represent a life safety threat to the public. The 1994 Northridge earthquake dramatically illustrated the success of retrofit—the 60 bridges retrofitted by Caltrans in the region of strong shaking showed no significant damage and could be reopened to traffic the following day.

5. Should we spend more money on seismic safety?

Yes. The job is not yet done if the people of California want their transportation system to be reliable for use during and after earthquakes. Many of the bridges in the state could be significantly compromised following an earthquake. Simply put, the job is only partially done. There is much more to do. Many retrofits now underway and planned will not be completed unless the resources are available. This is particularly true for remaining work on local bridges and toll bridges.

6. Why are the East Spans of the San Francisco-Oakland Bay Bridge being replaced rather than retrofitted?

The economic impacts of the closure of the Bay Bridge were massive following the 1989 Loma Prieta earthquake—to say nothing of the social disruption. The major damage was limited, and the bridge was returned to service in just one month. Notwithstanding, the damage was more severe than the obvious dropped deck spans on Pier E-9, which was the immediate reason for closure. Piers and other supports on the east section of the bridge were damaged, and their seismic capacities significantly reduced. Structural vulnerability assessments of the East Spans showed that more than 75 percent of the bridge members did not meet current deformation or strength capacity requirements and, if retrofitted, would need to be replaced under full traffic.

The San Francisco-Oakland Bay Bridge is near the Hayward fault, which the USGS recently assessed as having a 27 percent probability of a magnitude 6.7 or greater earthquake in the next 30 years. If this happens before replacement of the East Spans, currently under construction, is completed, there is a potential for portions of the existing eastern portion of the bridge to collapse. The San Francisco-Oakland Bay Bridge is heavily used for most of the day, and many lives are likely to be lost in any significant collapse.

The likelihood of earthquake ground motions sufficient to cause collapse of a portion of the existing East Spans of the San Francisco-Oakland Bay Bridge is so high that the only reasoned approach was to either seismically retrofit the bridge or to replace it to ensure an acceptable level of reliability and safety. The retrofit option was deemed too expensive and less reliable. For the sake of public safety during a lengthy retrofit period and a seismically more reliable new structure, the decision was made to replace, rather than retrofit, the East Spans. (Note that the West Spans, the suspension portion of the bridge, is not being replaced, but retrofitted as the most economic way to provide for acceptable levels of performance.)

Questions Following Future Earthquakes

No matter how well or poorly bridges perform in future earthquakes, questions will be raised about performance.

The 1989 Loma Prieta earthquake initiated the beginning of the process of resolving the seismic performance issues of California's highway transportation system. In his 1989 charge to the Board of Inquiry, Governor George Deukmejian directed that five specific issues arising from the 1989 Loma Prieta earthquake be addressed. To the Governor's original list of issues, the Board of Inquiry added another, on safety of bridges.

1. Determine why bridges failed in the earthquake.
2. Determine whether these failures were or could have been foreseen.
3. Advise on how to accurately predict possible future bridge and structure failures.
4. 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. 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.
6. Are California's roadways earthquake-safe?

These are the same issues likely to be raised after future earthquakes if the task of implementing the recommendations in this report falls short. The answers the state will give then will depend on actions it takes now and in the immediate future.

The balance of this report provides a review of the state-of-the-art and the state-of-practice in seismic design of bridges. These sections are not necessarily complete, either in coverage or scope. The Seismic Advisory Board provides the following sections of this report as the technical basis upon which it makes its seven Recommendations.

Section 4

San Francisco-Oakland Bay Bridge East Spans Seismic Safety Project

4.1 Background of East Spans Retrofit/ Replacement Issues

The San Francisco-Oakland Bay Bridge (Figure 4-1) carries 10 lanes of Interstate traffic on two decks. More than 280,000 vehicles use the bridge each day. This aging structure was first opened to the public in 1936. It was designed according to elastic theory to resist approximately 0.10g horizontal acceleration, which was the best understanding of the seismic hazard at that time (Caltrans 1998).*

At 5:04 pm on October 17, 1989, both spans of the double-deck roadway above Pier E9 experienced partial collapse under the seismic motions from the Loma Prieta earthquake (Figure 4-2). The damage cost a human life and closed the bridge for 30 days, causing millions of dollars per day in economic losses to the Bay Area.

The Loma Prieta earthquake had a moment magnitude of 6.9. Its epicenter was nearly 100 kilometers southwest of the bridge. An earthquake of that magnitude that far away was thought to present a relatively minor threat to the San Francisco-Oakland Bay Bridge compared to a larger, and much more likely, earthquake on either the San Andreas (15 km west) or the Hayward fault (8 km east).

Following the Loma Prieta earthquake, Caltrans began a multi-year seismic vulnerability assessment and retrofit project of all major bridges in California, including the San Francisco-Oakland Bay Bridge (SFOBB) East

Figure 4-1. West (foregreound) and East Spans of the existing San Francisco-Oakland Bay Bridge.

Figure 4-2. Collapse of one span of the eastern section of the San Francisco-Oakland Bay Bridge during the 1989 Loma Prieta earthquake.

* Sections 4.1 through 4.3 are from Seible et al. (2003).

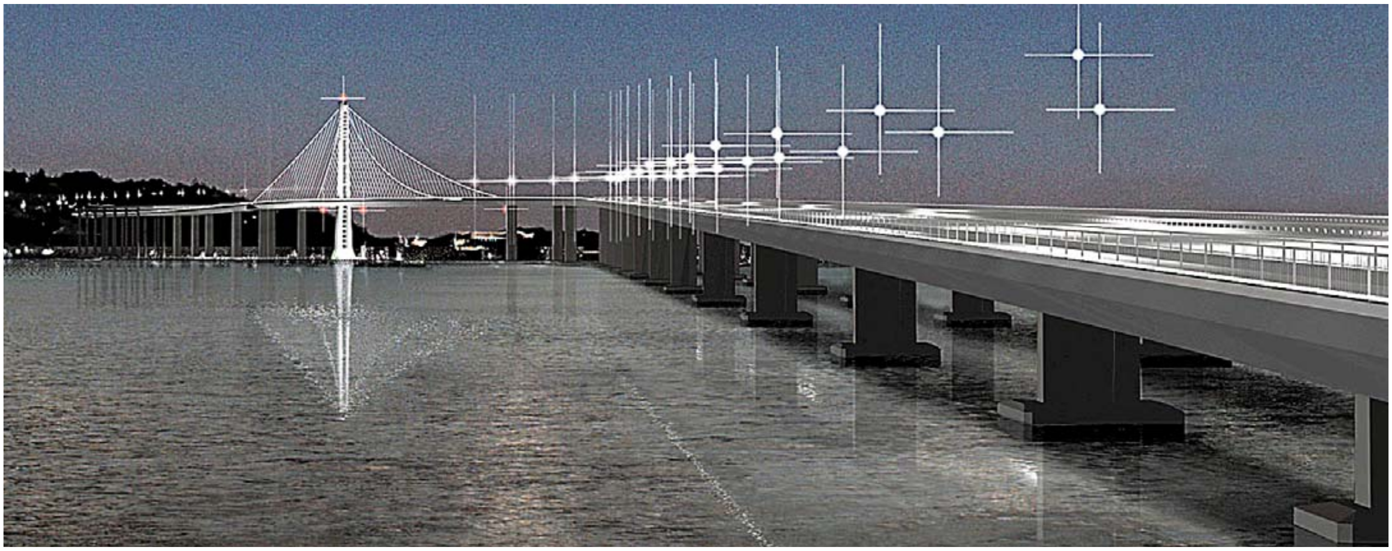


Figure 4-3. *Rendering of the new East Spans of the San Francisco-Oakland Bay Bridge at night (Lin, T.Y. International 2000b).*

Spans (Seible 2000). Caltrans first evaluated retrofit of the existing East Span steel truss structure. Retrofit of the existing span proved to be extremely expensive and of questionable reliability in terms of performance of the retrofit. Notwithstanding, a limited seismic retrofit of the existing East Span was completed to provide some protection from failure should low level earthquake ground motion occur before the replacement bridge was complete. Due to the difficulty of implementing a retrofit plan under full traffic, Caltrans determined that a replacement structure would provide a seismically more reliable alternative to retrofit. This decision resulted in the SFOBB East Spans Replacement Project. This bridge replacement project is the largest bridge project in California's history (estimated at close to $3 billion). The principal objective is for the bridge to be safe and reliable and be returned to service as quickly as practical without interruption of trans-Bay traffic following a big earthquake.

4.2 New East Spans Replacement Project

The Caltrans concept for the SFOBB East Spans Replacement Project consisted of two parallel reinforced concrete viaducts. These viaducts would extend from Oakland to Yerba Buena Island. The Metropolitan Transportation Commission (MTC), which represents the nine Bay Area counties and acts under authority granted by the California Legislature, petitioned the Legislature to approve an additional budget to include a “signature span” and various amenities to the Caltrans concept. MTC appointed an Engineering and Design Advisory Panel (EDAP), a panel of worldwide-recognized experts in bridge design, to develop recommended guidelines for the design of the new SFOBB East Spans.

4.2.1 EDAP Panel Recommendations

Among other recommendations of the Engineering and Design Advisory Panel, the following recommendations had a major impact on the design of the new bridge (Caltrans 2002):

- The new East Spans and the retrofitted West Spans should be designed to provide postearthquake “lifeline service.”
- The new East Spans should have a cable-supported main span with a single vertical tower with single or multiple legs in the transverse direction and single or multiple planes of supporting cables.
- The new East Spans should not be double-decked.
- The cable or suspension tower on the East Spans should not be taller than the suspension towers on the existing West Span.
- The new East Spans should have bicycle and pedestrian lanes.

Following these and other recommendations, Caltrans selected and contracted with the joint venture of T.Y. Lin International/ Moffatt & Nichol to develop 30 percent designs for two alternative bridges: one with a cable-stayed main span and one with a self-anchored suspended main span (Caltrans 1998; Goodyear and Sun 2003; Tang et al. 2000). After careful evaluation, and mainly due to its better assimilation with existing suspension

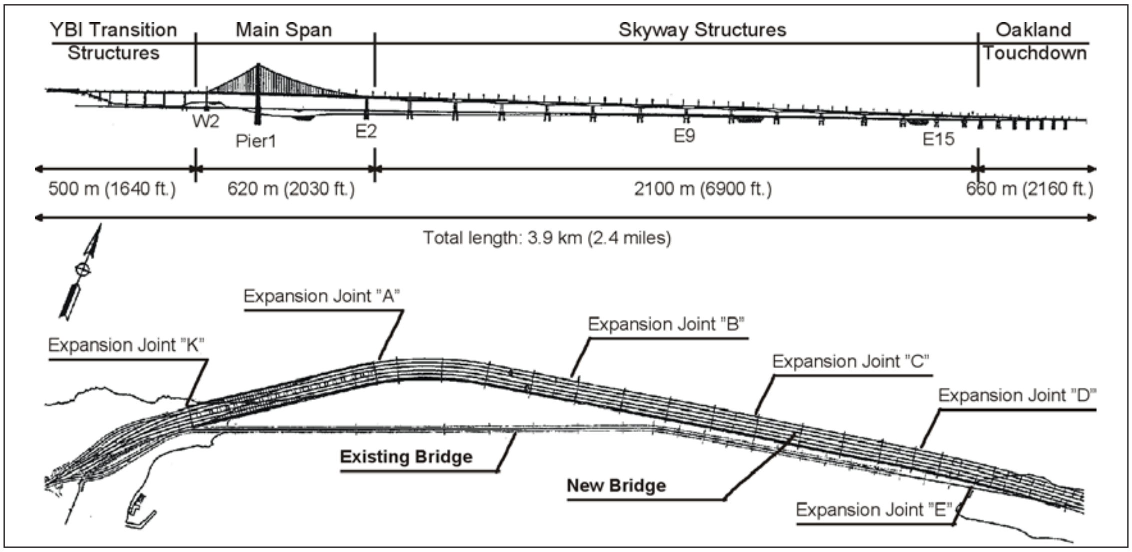


Figure 4-4. *Schematic representation of the new East Spans of the San Francisco-Oakland Bay Bridge.*

bridges in the San Francisco Bay Area, the self-anchored suspension (SAS) bridge was chosen (Figure 4-3).

4.2.2 Design of New East Spans

The new East Spans (Figure 4-4) consist of four distinct structures:

- The Oakland landing, or touchdown, structure.
- A segmental concrete box girder crossing called the Skyway.
- A self-anchored suspension Signature Span.
- A series of multi-cell post-tensioned concrete box girder bridges providing the transition to the tunnel on Yerba Buena Island (Tang et al. 2000).

The new East Spans will feature parallel roadways and will be built next to the existing East Span bridge, which will be dismantled after the new bridge is opened to traffic.

Skyway

The Skyway consists of two parallel segmental precast concrete viaducts with a typical span of 160m, grouped in frame units of three or four piers per frame, separated by expansion joints. The haunched single cell box girder cross-section has a depth of 5.5m at midspan (9.9 m at the pier) and features two vertical webs spaced 8.5 m apart.

The total deck width of 25 m is reached using overhangs of 8.3 m on each side. The deck is post-tensioned in the longitudinal and in the transverse directions, while the webs are post-tensioned longitudinally and vertically.

The cast-in-place hollow rectangular reinforced concrete piers of the Skyway rely on highly confined corner elements for inelastic deformation capacity and on connecting structural walls for stiffness and strength (Figure 4-5). The Skyway piers imitate the main span tower in geometry and architectural treatment through cover concrete articulation, helping to maintain a consistent visual theme throughout the entire bridge (Hines et al. 2002). The Skyway piers have heights ranging from 36 m at Pier E3 to 14 m at Pier E16 with monolithic connections to the superstructure and to the foundations.

Signature Span

The Signature Span (or main span) of the new eastern portion of the San Francisco-Oakland Bay Bridge will be the world's largest self-anchored suspension (SAS) bridge. The SAS Signature Span consists of a 385 m front span and a 180 m back span. The single tower is 160 m tall and is made up of four steel shafts (tapered stiffened box members) connected with intermittent shear links along its height. The tower pile cap is positioned at water level supported by thirteen 2.5 m diameter cast-in-steel-shell (CISS) concrete piles. The permanent shell terminates 30 m below the pile cap with a cast-in-drilled-hole (CIDH) pile continuing to a depth of approximately 75 m below the water line and into the bedrock of the Franciscan Formation. The 0.78 m diameter main cable is anchored to the deck at the east bent (Pier E2) and looped around the west bent (Pier W2) through deviation saddles. At Pier E2 the cable is parallel to the deck, inducing a horizontal compressive force in the deck only.

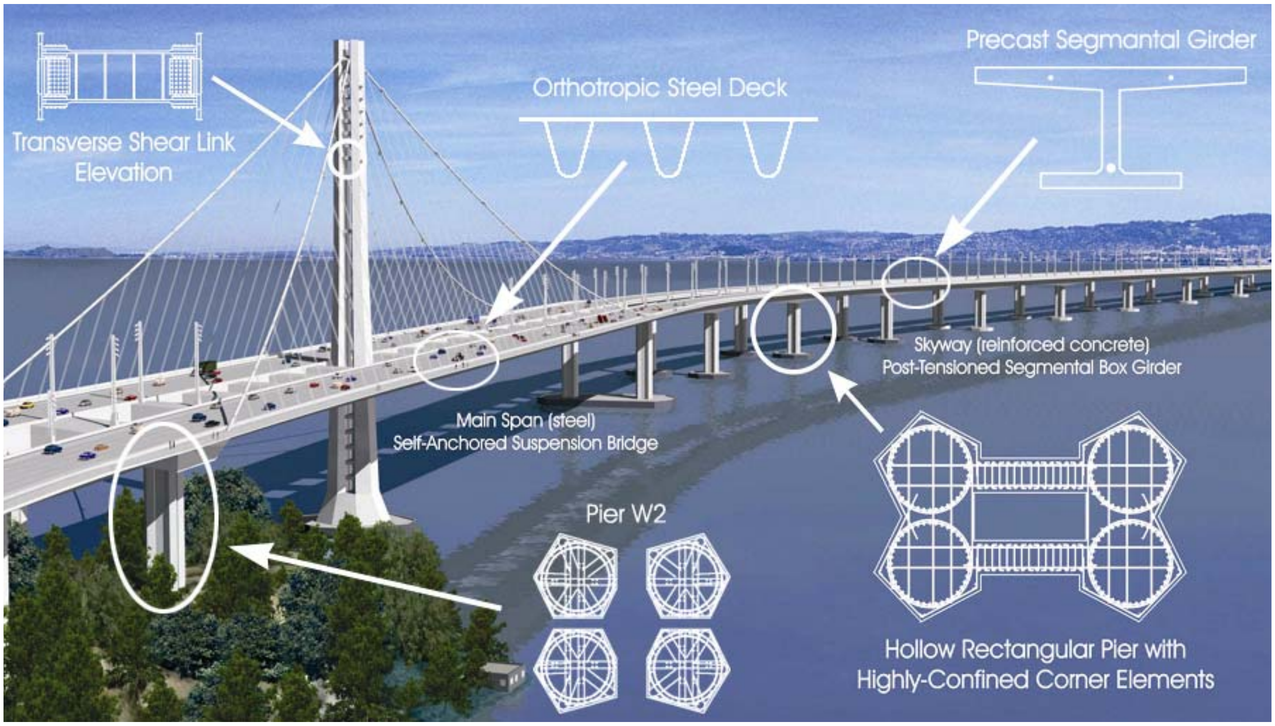


Figure 4-5. *Rendering of the new East Spans of the San Francisco-Oakland Bay Bridge and of the key structural components tested in the Charles Lee Powell Structural Research Laboratories at the University of California, San Diego (original rendering: T.Y. Lin International).*

On the Oakland end, at Pier W2, the inclined main cable induces a horizontal compressive force into the deck and a vertical tensile force that has to be resisted by the pier. The uplift at W2, due to the self-weight of the front span, is balanced partly by the self-weight of the massive cap beam of Pier W2. The additional seismic uplift is resisted by a tiedown system consisting of 28 tendons (each with 61-15 mm diameter strands) anchored in the cap beam and in the foundation blocks. The tension forces are resisted by the weight of the foundation blocks encased in the bedrock and by eight 2.5 m diameter CIDH concrete piles.

Pier W2 consists of a north and south pier fixed at the base and tied together at the top by a stiff cap beam. Each pier is made up of four 3.5 m diameter circular concrete columns, with pentagonal-shaped architectural concrete to ensure visual consistency with the other piers of the bridge. The columns are fixed to the foundation block and to the cap beam and are not interconnected (Figure 4-5). Pier E2 consists of a north and south pier fixed to the foundation at the base and tied together at the top by a stiff cap beam, and is similar in design to the Skyway piers.

Superstructure

The superstructure of the SAS bridge consists of two 25m wide dual, hollow orthotropic steel boxes, accommodating five lanes of traffic and two shoulder lanes each. The superstructure is under a permanent compression load of 200MN per each girder, corresponding to about 30 percent of the nominal yield strength of the longitudinal girders, to balance the cable tension forces. The box girders are connected by crossbeams spaced 30 m apart. The crossbeams carry the transverse load between the suspenders and ensure that the dual boxes act compositely during wind and seismic loads. The suspenders are splayed to the exterior side of the box girder and are spaced 10 m apart.

4.3 Seismic Design Philosophy Employed in Development of the East Spans

4.3.1 Performance Limit States

The seismic risk to the new East Spans of the San Francisco-Oakland Bay Bridge comes mainly from the Hayward fault, located 12 km east and capable of generating a 7.5 Richter magnitude earthquake, and from the San Andreas fault, located 25 km to the west and

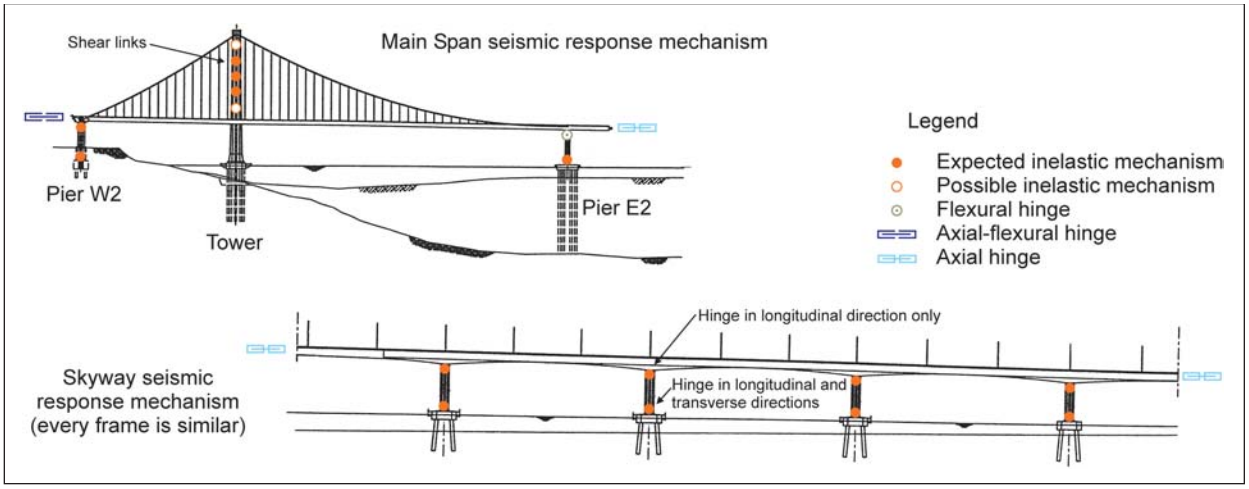


Figure 4-6. *Seismic response mechanism of the new East Spans of the San Francisco-Oakland Bay Bridge.*

capable of generating an 8.1 Richter magnitude earthquake.

The new East Spans are designed to resist two levels of earthquake, namely:

- The **Safety Evaluation Earthquake** (SEE), addressing an approximately 1,500-year event on either fault.
- A **Functional Evaluation Earthquake** (FEE) corresponding to a shorter return period (~300-year) event.

The FEE performance criteria require full service almost immediately following the earthquake with only minimal damage to the structure. Minimal damage implies essentially elastic performance, and is characterized by minor inelastic response, narrow cracking in concrete, no apparent permanent deformations and only limited damage to expansions joints that can temporarily be bridged with steel plates.

After an SEE event, the new East Spans would provide service with no more than repairable damage to the structure. Repairable damage is damage that can be repaired with minimum risk, such as minimal damage to superstructure and tower shafts, limited damage to piers (including yielding of reinforcement and spalling of concrete cover) and tower shear links, small permanent deformations that do not interfere with the serviceability of the bridge, and damage to expansion joints that can temporarily be bridged with steel plates. To ensure the ability of the bridge to carry traffic across expansion joints after the SEE event, the allowable average permanent deformation is limited to 300 mm.

4.3.2 General Seismic Design Concept

The design philosophy for the new East Spans is aimed at providing an inherent toughness in the pier-foundation system, defining a clear sequence of yielding damage in overload conditions, and avoiding the need for inspection or repair at inaccessible locations.

These requirements and the high seismicity in the Bay Area required state-of-the-art capacity design of the new bridge, allowing plastic deformation in clearly designated structural components that were specially designed for this purpose. For example, in the Skyway, plastic hinges are allowed to form at the top and bottom of all concrete piers, protecting the foundations and the superstructure against overload. In the SAS bridge, shear hinges are allowed to form in the steel shear links connecting the four steel shafts of the main tower, protecting the tower legs against yielding (Figure 4-5). The shear links were designed to be replaceable after a seismic event. Expansion joints reduce the constraints between the four distinct structures of the bridge and within the Skyway (Figure 4-6).

To implement this design philosophy and to meet the expected performance requirements, the East Spans were designed based on capacity design principles for limited-ductility structures. Detailing and proportioning requirements for full-ductility structures were implemented, delivering a structure with excellent serviceability characteristics and a high degree of inherent safety and reliability. To demonstrate that all performance requirements can be safely met by the proposed design, components in the bridge that were expected to see any inelastic action under the SEE, had to be proof-tested at large or full scale to clearly establish all performance limit states.

4.3.3 Proof Test Program

The design of the bridge was carried out based on nonlinear time-history analyses with multiple support input. The ground motions used for the analyses were specially developed for this project, taking source-to-site effects, near-source effects, as well as site-specific geological and geotechnical characteristics into account.

Local element capacities were established from first principles using section analyses or through detailed nonlinear finite element modeling. The performance limit states (capacities at predetermined damage/performance levels) for all components with expected inelastic actions or plastic hinges (Figure 4-6) were required to be verified by means of full or large-scale proof testing.

This proof-testing program was conducted at the Charles Lee Powell Structural Research Laboratories at the University of California, San Diego. In the framework of this program, two steel shear links at 100 percent scale were tested (McDaniel et al. 2001), two concrete piers of the Skyway at 25 percent scale were tested by Hines et al. (2002) and a 25 percent scale model of the West Anchor Pier W2 was tested by Dazio and Seible (2002). Two steel shear links at 50 percent scale were tested by Dusicka et al. (2002) at the University of Nevada at Reno. The latter tests complemented the full-scale shear link tests performed at U.C. San Diego, since proper boundary conditions for the link in the form of half-scale tower leg sections were introduced in the Nevada tests.

Two segmental box girder sections of the Skyway at approximately 20 percent scale, were tested by Megally et al. (2001) to investigate different design philosophies for the segment-to-segment construction joint in order to optimize the erection procedure of the Skyway, yet ensure satisfactory seismic performance. Two panels of the SAS bridge steel box girder were tested at 45 percent scale by Chou et al. (2002) to investigate the ability of the deck to sustain seismic-induced compressive stresses near yielding without buckling. These proof-of-concept tests for the SFOBB East Spans are discussed below.

4.4 Peer Review and Construction

Upon award of the design contract, Caltrans appointed a Seismic Safety Peer Review Panel (SSPR) comprised of Gerald Fox, bridge design engineer; Ben Gerwick, construction engineer and professor emeritus at U.C. Berkeley; I.M. Idriss, geotechnical engineer and professor at U.C. Davis; Joseph Nicoletti, consulting structural engineer; and Frieder Seible, structural engineer and professor at U.C. San Diego. An initial act by the Seismic Safety Peer Review Panel was the appointment of an ad hoc committee to establish the ground motion parameters for the design of the new bridge. The SSPR met periodically with the design team throughout the development of the design to provide assurance to Caltrans that appropriate design criteria had been developed and properly implemented, that appropriate analytical procedures were being applied, and that all other quality assurance procedures prescribed by Caltrans were being followed. It should be noted that three of the SSPR are also members of the Caltrans Seismic Advisory Board, which facilitated keeping the Board apprised of significant issues during design.

4.5 Problems and Delays

This important seismic safety project as been plagued with a number of unfortunate problems and delays. Although the City of San Francisco was on record as approving the design and alignment of the new bridge, about six months after award of the design contract, it requested Caltrans to change the alignment or to reconsider retrofit of the existing bridge. The U.S. Navy, in the process of transferring ownership of Yerba Buena Island to San Francisco, refused access to Caltrans for the necessary geotechnical investigations on the island. After about two years of delay and several hundred million dollars of estimated construction costs, the stalemate was resolved by the Federal Highway Administration (FHWA) claiming federal priority for the bridge right-of-way on the island.

The advertising and award of the Skyway contract, under construction as of this writing (December 2003), resulted in limited competition and costs significantly in excess of conventional estimates. The limited availability of qualified contractors and the heavy and complex construction equipment required for this bridge, plus the fact that a number of contractors and their equipment were already committed to similar work on several other toll bridges in the Bay Area, combined to create the adverse bids. Additionally, the large size and cost of the proposed construction increments (the Skyway cost is approximately 1.3 billion dollars) require proportionately large bid and performance bonds and, in the current economic recession and the risk of terrorist activity, banks are reluctant to commit such large amounts of money to one project.

4.6 Potential Future Problems and Concerns

Caltrans had initially planned to combine the SAS Signature Span and the transition structures on Yerba Buena Island in one contract in order to avoid potential conflicts with two or more contractors occupying the limited work area on the island. However, in view of the experience gained with Skyway contract, dividing the increment into two or more contracts became a desirable alternative and Caltrans decided to separate the SAS and the transition structures into separate contracts.

Other California Toll Bridge Seismic Safety Programs are funded entirely with state funds but, after the overrun on the Skyway contract, the state requested federal assistance with the remainder of the East Spans project. Federal funding requires compliance with the Federal Procurement Regulations that impose “Buy American” provisions on certain construction materials such as structural steel. In view of the fact that very few U.S. steel suppliers currently have the necessary facilities, experience, and access to waterways to fabricate, ship, and erect the large steel sections required for the SAS, Caltrans was understandably concerned regarding the SAS contract. Fortunately, the “Buy American" provisions contain an alternative clause that permits buying from international sources if the U.S. costs are 25 percent or more in excess of the international costs.

In order to facilitate procurement of bid and performance bonds and also in response to pressure from small business groups, Caltrans is attempting to break up the remaining work into smaller contracts. For example, Pier W-2 at the west end of the SAS has been awarded as a separate contract and the temporary bypass structure on the island will be awarded as separate contract.

The current schedule envisions the award of the SAS superstructure contract and the temporary detour structures in December 2003 and the YBI transition structures in February 2005. Because of the current and potential cost overruns on this project and the present budgetary crisis in California, the Seismic Advisory Board is extremely concerned that this very important seismic safety project may experience significant delays and/or work stoppages. The Board wishes to remind the Director that the existing East Span, vital to the economic wellbeing of the Bay Area, is living on borrowed time with respect to the seismic hazard. The timely completion of the bridge truly embodies *The Race to Seismic Safety.*

Section 5

Review of Seismic Hazards in California

5.1 Nature of the Earthquake Hazard

Knowledge of seismic hazards in California, their assessment, and methods by which their effects can be mitigated has grown substantially in the decade since the Loma Prieta and Northridge earthquakes. Special intensive studies by Caltrans for the retrofit of San Francisco Bay Area toll bridges and substantial research contributions from other state agencies, university engineering groups, and private consultants have brought two notable advances.

First, the seismic hazard map for California by Caltrans has been updated (Figure 1-2). Second, the special nature of the strong ground shaking near the fault source of large earthquakes has now been documented by instrumental observations, including the 1995 Kobe earthquake (M6.9), the 1999 Izmit earthquake in Turkey (M7.6), the 1999 Chi-Chi earthquake in Taiwan (M7.6), and the November 2002 Denali earthquake in Alaska (M7.9).

These recent time histories and spectra and their critical applications to structural response of bridges have provided the basis for more reliable and economical retrofit schemes.

5.1.1 Updates in Ground Motion Attenuation

Recently published studies have found significant differences in attenuation between various tectonic regions as well as for various geologic conditions and seismic sources. The last decade has seen the number of recordings of strong ground motion increase significantly, allowing the development of region-specific attenuation relations. In particular, data close to the fault source have increased the constraints on the behavior of the attenuation relation at short distances, and the uncertainty in attenuation relations has been better characterized by considering alternative models. More attention is now being given in seismic hazard analyses to the variability of the seismic wave attenuation in California, as well as the uncertainty. In a current research program, Caltrans is providing substantial support for this work through a number of university-based research investigations.

5.1.2 Near-Fault Ground Motions

Knowledge of near-fault ground motions, which often contain large long-period pulses, has improved dramatically since 1989. There are two causes for these long period pulses in near-fault ground motions. One is the constructive interference of the dynamic shaking due to fault rupture directivity (Figure 5-1). The other is due to the permanent offset, or "fling," of the ground along the fault.

Rupture directivity effects are especially severe when the rupture is toward the site and the slip direction (on the fault plane) is aligned with the rupture direction. Rupture directivity is strongest on the component of motion perpendicular to the strike of the fault (fault normal component). Additionally, permanent fling effects occur parallel to the slip direction when the site is located close to a fault with significant surface rupture. For strike-slip earthquakes, rupture directivity is dominant on the fault-normal component and static displacement effects are observed on the

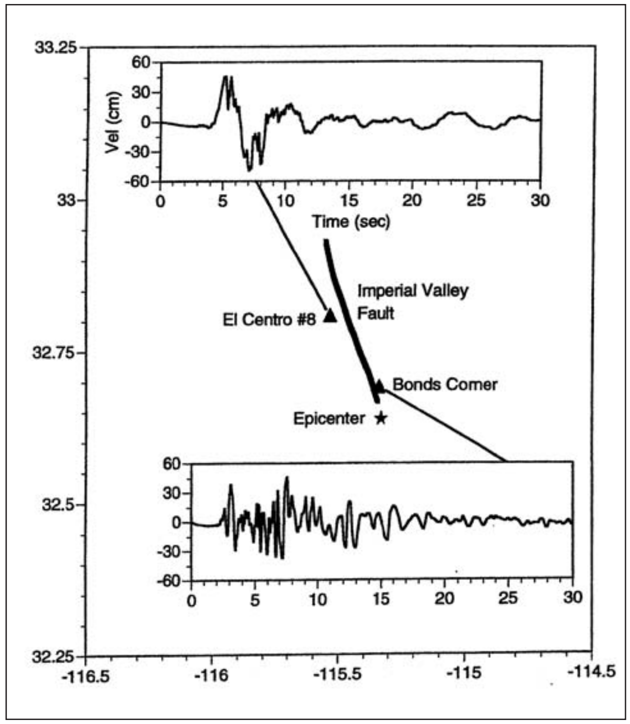


Figure 5-1. *Recorded ground velocities show the effect of fault rupture directivity. 1979 Imperial Valley, California earthquake (Bolt and Abrahamson 2003).*

fault-parallel component. Thus, for strike-slip earthquakes, like those caused by rupture of the San Andreas and Hayward faults, the rupture directivity pulse and the fling-step pulse will naturally separate themselves on the two horizontal orthogonal components.

What is important for large Caltrans structures is that the directivity of the fault source rupture causes spatial variations in ground motion amplitude and duration around faults, with systematic differences between the strike-normal and strike-parallel components of horizontal ground motion amplitudes (Bolt and Abrahamson 2003). These variations start to become significant at a period of 0.6 seconds and generally grow in size with increasing period. Modifications to empirical strong ground motion attenuation relations have been considered in studies of Caltrans toll bridges to account for the effects of rupture directivity on these strong motion amplitudes and durations. Instrumental measurements show that such directivity can modify the amplitude velocity pulses by a factor of up to 10, while reducing the duration by a factor of 2. A clear illustration is the recorded ground velocity of the 15 October 1979 Imperial Valley, California earthquake generated by a strike-slip fault source (Figure 5-1). The main rupture front on the fault moved toward El Centro and away from Bonds Corner.

5.1.3 Seismic Strong Motion, Seismic Hazard, and Design Ground Motions

There are two basic approaches to developing design spectra: deterministic and probabilistic. Both have been used in studies of large structures in California, but hazard estimates for Caltrans Important bridges have used the probabilistic method.

The deterministic approach uses selected individual earthquake scenarios (magnitude, distance, directivity, etc.). The ground motion is then computed using appropriate attenuation relations with a specified probability of the ground motion parameters not being exceeded in a specified scenario earthquake. Because there is scatter in the ground shaking measurements from earthquake to earthquake, averages must be used. A design spectrum is developed by scaling a standard spectral shape. Typically, a probability of nonexceedance of either 0.5 (median) or 0.84 (median plus one standard deviation—a measure of scatter) ground motion is used. The specific implementation of this approach by Caltrans in the design of Standard bridges is described in Section 5.3.1.

The probabilistic approach differs from the deterministic approach in that it considers the rate of occurrence of local earthquakes and also the variability of the ground motion (number of standard deviations above or below the median) and its associated probability distribution. The hazard curve gives the probability that any of the scenarios (ground motions) will produce a ground motion exceeding a selected value. For probabilistic analyses, the design ground motion is typically given by an equal hazard spectrum. Equal hazard spectra are constructed by first computing the hazard at each spectral period independently.

The equal hazard spectrum for a bridge site may not be physically achieved in a single event, but is meant to represent design criteria for all reasonable cases. An equal hazard spectrum gives at each spectral period the response spectral value that has the specified return period of the earthquake motion.

5.1.4 Design Time Histories

The construction of strong motion time histories has become an essential part of the definition of hazard for the design and testing of critical Caltrans structures. There are two main methods used to develop design seismic ground motions: (a) scaling observed ground motions from past earthquakes and (b) adjusting observed ground motions to match a selected design spectrum. The second method has been adopted in recent Caltrans work.

Spectrum-compatible time histories are time histories that have been modified in terms of the amplitude of their frequency content to match the entire design spectrum. The selection of the initial time histories for use in either scaling or spectral matching has turned out to be critical in testing the nonlinear response of the soil and structure. Potential starting ("seed") motions are based on their duration, site characteristics, event magnitude and recording distance, and the general character of the displacement history. In particular, for near-fault time histories, the character of the displacement pulse as one-sided, two-sided, or multi-sided is chosen so that the selected motions will have distinctly different time signatures (uncorrelated) to test thoroughly the structural design.

For most engineering applications, the ground motion is defined at a single point. In reality, for viaducts, large bridges, and dams, out-of-phase wave motions over inter-support distances cause differential ground accelerations and differential rotations along the base of the structure. Studies of the spatial variation of strong ground motions from observations of instrumental arrays of strong motion instruments are now available and the results have been incorporated into structural response analyses for some large Important Caltrans structures.

5.1.5 Strong Ground Motion Estimation

As an example of the contemporary analysis of a Caltrans structure, consider the estimation of the strong motion parameters for the East Spans of the San Francisco-Oakland Bay Bridge. The bridge lies between two major active faults: the Hayward fault 8 km to the east and the San Andreas fault 15 km to the west.

A design spectrum was derived based on probabilistic seismic hazard. A response spectrum with a return period of 1,500 years was selected as a "Safety Evaluation Earthquake." The hazard analysis included the effects of rupture directivity by modifying the attenuation relations to include fault rupture effects

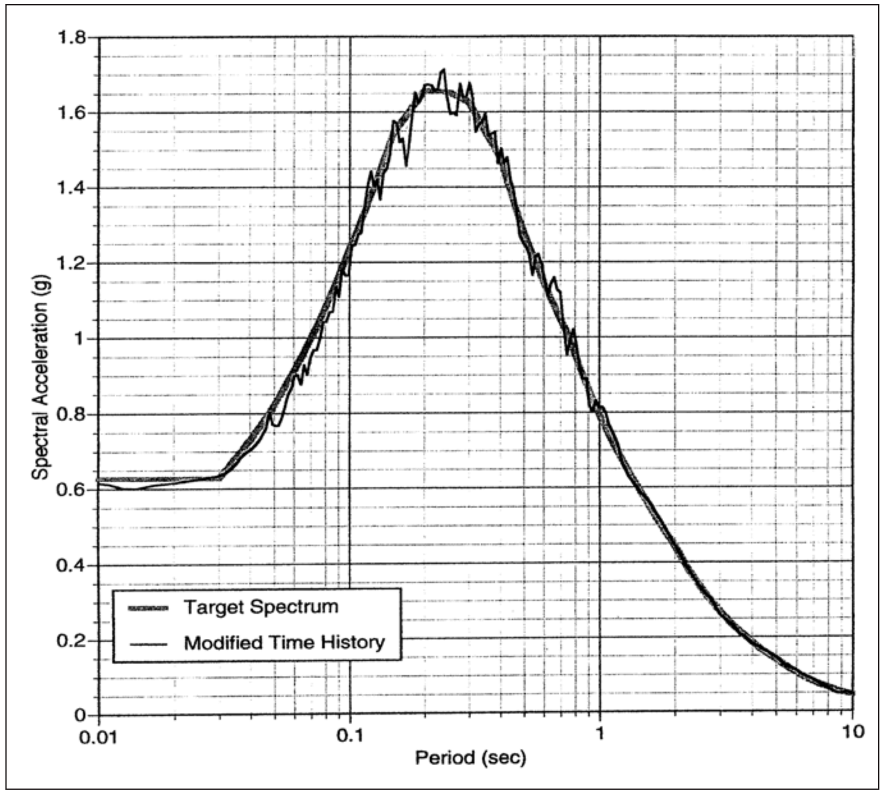


Figure 5-2. Comparison of the design spectrum and the spectrum of a modified time history (horizontal fault-normal component) for the East Spans, site of the San Francisco-Oakland Bay Bridge.

(Figure 5-2). Differences between the two horizontal components (fault-normal and fault-parallel) arise from the inclusion of the direction of the fault rupture.

The hazard for the 1,500-year return period was separated (deaggregated) into the contributions from each active fault to determine which earthquakes are most important. In this case, the deaggregation indicated that the seismic hazard at the Bay Bridge is dominated by a *M*7.8 earthquake at about 20 km distance on the San Andreas fault and an *M*7.0 earthquake at about 10 km distance on the Hayward fault. The deaggregation also showed that at a spectral period of 3 seconds, the 1,500-year return period ground motion is dominated by forward rupture directivity.

For the Hayward fault source, suitable observed time histories were available for direct use as initial time histories; however, for the San Andreas source scenarios, there are as yet no recorded time histories that satisfy the required magnitude and distance range, so some appropriate time histories were constructed as "seed" motions. Design spectrum-compatible time histories are shown in Figure 5-3.

The seismological problems described in the summary description above are expected to persist. First, greater sampling of strong ground motions at all distances from fault sources of various mechanisms and magnitudes will inevitably become available. An excellent example is the very recent *M*7.9 earthquake generated by the rupture in 2002 of the Denali fault in Alaska, from which valuable and relevant recordings of ground motion were obtained.

5.2 Geologic Hazards

Bridge performance is dependent on its site, and it is through the structure's interaction with its site that earthquakes impacts occur. The largest proportion of earthquake impacts to bridges are through vibratory ground motions. The others sources are through earthquake-induced site failure by faulting, liquefaction, densification and landsliding, or through water waves caused by tsunamis or dam ruptures.

5.2.1 Fault Rupture

Caltrans recognizes the importance of considering the effects of fault rupture offset, along with ground shaking, on the seismic

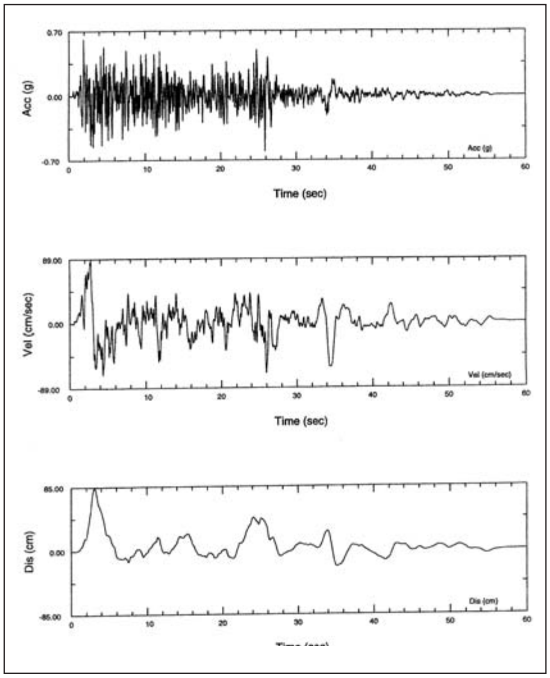


Figure 5-3. *Spectrum-compatible fault-normal time histories of the San Andreas fault source for one component of acceleration, velocity, and displacement. These were mathematically constructed from recordings of the 1940 and 1979 Imperial Valley, California earthquakes for the San Francisco-Oakland Bay Bridge (Fugro/Earthmechanics 1998).*

Figure 5-4. *The Chelungpu fault scarp as it crosses the Tachaihsi River, producing a 6-meter-high waterfall and destroying the road bridge. The damaged Shihkang dam is in the distance (from Bolt 2003; photo courtesy of Jack Un, Flying Tiger Photographic, Inc.).*

response of bridge and tunnel structures crossing active faults. Both deterministic and probabilistic methods are being used to evaluate expected horizontal and vertical offsets on designated active faults (Wells and Coppersmith 1994; Bechtel/HNTB 2002).

The impact of fault displacements can be very dramatic. The cause of the Chi-Chi, Taiwan earthquake was a south-to-north rupture of a well-mapped fault called the Chelungpu thrust, which runs along the west margin of the central mountains in Taiwan. The surface fault rupture was approximately 100 km long, with segmented offsets and a striking northeast-trending jog at its northern termination (Figure 5-4). The maximum fault slip was about 10 m, located 30 km north of the hypocenter near the towns of Wufeng and Nantou. Vertical ground displacements ranged up to 7 m along the northern part of the rupture, and as much as 5 m in the southern part.

Various design concepts are adapted to accommodate the possible occurrence of large horizontal and vertical components of fault offset underneath a bridge structure. These design concepts provide flexibility in the superstructure. through the use of a long simply-supported span crossing the fault zone, extending the width of support seats, and enhancing the ductility in columns and foundations.

An example where Caltrans went to great effort to develop such concepts was in designing bridges for the I-210/I-215 Interchange located in San Bernardino (Gloyd et al. 2002). This interchange is located between two major faults, namely the San Jacinto fault located about 3 km to the west and the San Andreas fault located about 6.5 km to the east. Excavating trenches within the area of the planned bridge structures provided evidence that fault ruptures had taken place. This discovery motivated Caltrans to set up a Technical Advisory Panel (TAP) to review the information and provide expert recommendations for structural design.

Caltrans also gave consideration of the effects of fault rupture on bridge seismic response in its retrofit program for the major toll bridges. Two of these bridges have a fault passing underneath the structure: the Palos Verdes fault passes underneath the Vincent Thomas Bridge in Los Angeles (Clarke and

Table 5-1. *Earthquakes that produced damaging tsunamis.*

Earthquake	Date	Magnitude	California Maximum Observed Wave Ht-ft.
San Francisco	1906	8.3	2
Lompoc	1927	7.3	6
Santa Monica	1930	5.25	10
Aleutians	1946	7.8	5
Kamchatka	1952	8.2	2
Aleutians	1957	8.3	3
Chile	1960	8.6	5
Alaska	1964	8.4	21

Kennedy 1997) and the Rose Canyon fault has strands that pass underneath the Coronado Bay Bridge in San Diego (Kennedy and Clarke 1997).

Fault offsets up to 2.7 meters in the horizontal direction and up to approximately 0.35 meters in the vertical direction were considered for the Vincent Thomas Bridge (Fugro/Earth Mechanics 1996). Due to the relatively high flexibility of this suspension bridge, it was found that fault rupture effects are negligible compared to corresponding effects caused by dynamic ground shaking. However, due to the much lower flexibility of the San Diego-Coronado Bridge, the estimated fault-offset displacements, which range from 0.5 to 0.8 meter under the various spans (2 through 21), did influence retrofit design concepts for the structure (Fugro/Earth Mechanics 1997). Specifically, rubber bearing base-isolation devices were placed between the bridge's towers and its deck to provide the needed flexibility to safely accommodate the design fault offset displacements.

In the future, Caltrans should complete its development of seismic design criteria related to controlling the effects of expected fault offsets on the seismic response of bridge and tunnel structures.

5.2.2 Tsunamis

Of all natural hazards affecting the safety of bridges and highways, tsunamis have received the least attention. Yet, records show that destructive tsunamis have occurred in the past and have caused damage to California highways and bridges located along the coast.

The most destructive tsunami to hit California was the result of the 1964 Great Alaska earthquake. This tsunami caused damage along all West coast states. Crescent City, California was hardest hit where, due to the shape of its bay, it was struck by a 21-foot wave. This tsunami damaged the breakwater and washed away embankments of the Elk River Bridge and destroyed a timber bridge at Dutton docks. Farther down the coast from Crescent City, wave heights reached 10 feet at Half Moon Bay, 10 feet at Santa Cruz, 8.5 feet in Monterey, and 6.5 feet in San Diego.

Table 5-1 lists those earthquakes that have caused tsunami damage along the California coastline, along with their dates of occurrence, magnitudes, and wave heights. Although the amount of bridge and highway damage from the tsunamis produced by these past earthquakes has been relatively small, expanding population centers and their associated infrastructures (including highways and bridges) along the California coastline could significantly increase the risks associated with tsunamis unless plans are developed to control the risk.

Caltrans has begun to identify the risks to its highway system associated with tsunamis. Corresponding design criteria, developed by federal and other state agencies, are being reviewed. Preliminary screening of bridges located within one-half mile of the coastline is being conducted to identify those bridge structures at risk. The objective of this effort is to develop a sound policy related to the characterization and mitigation of tsunami damage.

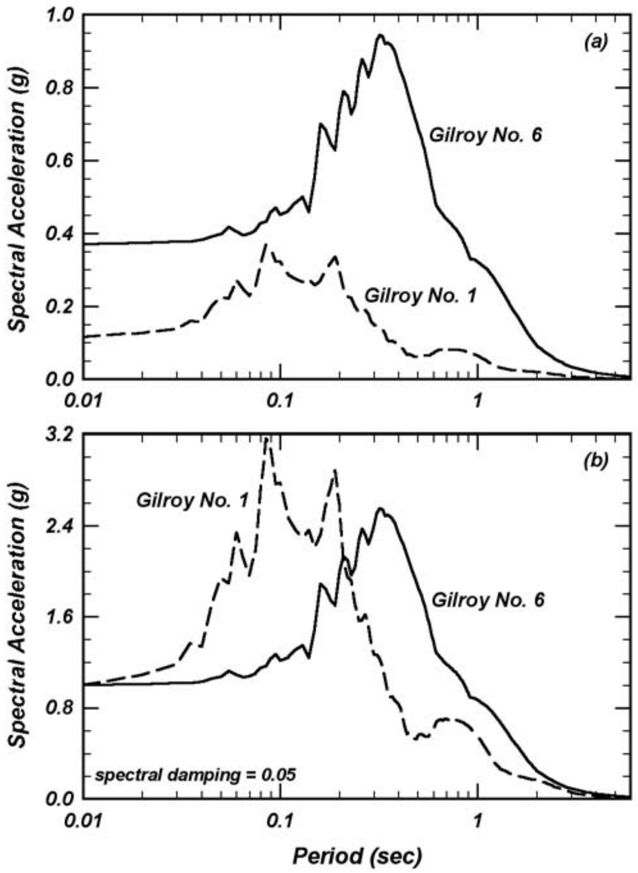


Figure 5-5. *Spectral ordinates of horizontal of earthquake ground motions recorded at a rock site (Gilroy No. 1) and at a deep soil site (Gilroy No. 6) during the 1979 Coyote Lake earthquake; (a) pseudo-absolute spectral accelerations; (b) spectral magnification ratios, i.e., pseudo-absolute spectral accelerations divided by peak ground acceleration.*

There is recent strong evidence that very large magnitude earthquakes have been generated by slip on the tectonic plate along the Cascadia subduction zone (northern California, Oregon, and Washington). These earthquakes have been dated from soil layers formed along the cost due to large tsunamis.

Caltrans should conduct a tsunami risk assessment for sites along the coast and should develop design criteria for preventing structural damage due to tsunami-induced wave actions.

5.2.3 Site Response

Recorded ground motions collected over the years have indicated the strong effects of local site conditions. Figure 5-5 shows the spectral ordinates of the horizontal motions recorded at a rock site (Gilroy No. 1) and a nearby soil site (Gilroy No. 6) during the 1979 Coyote Lake earthquake.

Quantifications of the effects of local site conditions have been made based on recorded data in addition to the use of calibrated dynamic response analyses. The need for analytical procedures is necessitated by the sparsity of recorded motions at all possible local site conditions.

Procedures to calculate the effects of local site conditions on earthquake ground motions were initiated in the U.S. by the pioneering work of Penzien et al. (1964) and Penzien (1970), who evaluated the site response at Elkhorn Slough as part of a research project sponsored by Caltrans. This was followed by development of the equivalent linear method of analysis (Idriss and Seed 1968), which became a mainstay for conducting site response studies for the past 35 years. The computer program SHAKE (Schnabel et al. 1972), which incorporated the equivalent linear method of analysis, has been and continues to be about the most widely used program for calculating site response of level ground.

More detailed procedures, incorporating nonlinear soil properties, have also been developed or are being developed. Of particular interest is the development of the computer program "Open Seas," which is being completed at the PEER Center at Berkeley. This program will be tested and calibrated by

Figure 5-6. *Liquefaction leading to lateral movement of river banks and resulting in the collapse of the Showa Bridge in Niigata during the 1964 Niigata earthquake.*

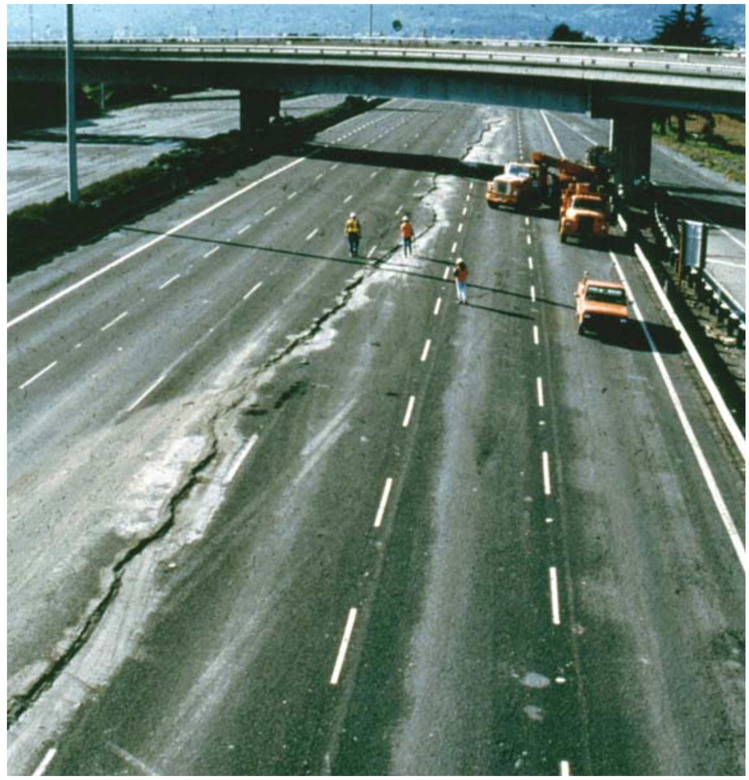


Figure 5-7. *Liquefaction leading to lateral movement and cracking of pavement at the approach to the San Francisco-Oakland Bay Bridge during the 1989 Loma Prieta earthquake.*

calculating the site response at a Caltrans bridge site in northern California. This effort is supported in part by Caltrans.

It is important to note that equivalent linear procedures are adequate for calculating site response parameters if there is no soil failure caused by the earthquake ground motions. When soil failure is likely, nonlinear analyses are preferable to estimate the amount and directions of deformations that could be caused by the earthquake ground motions.

The procedures currently used by Caltrans to conduct site response evaluations and to develop earthquake ground motion parameters for design purposes are summarized in the report *Seismic Soil-Foundation-Structure Interaction* (SAB 1999a), which was completed in 1999 by the Ad Hoc Committee on Soil-Foundation-Structure Interaction, a committee appointed by the Seismic Advisory Board.

5.2.4 Liquefaction

One of the most dramatic causes of damage to structures has been liquefaction in saturated sand deposits. Liquefaction has been manifested by the formation of boils and mudspouts at the ground surface, by seepage of water through ground cracks or in some cases by the development of quicksand-like conditions over substantial areas. Where the latter phenomenon occurs, buildings may sink substantially into the ground or tilt excessively, lightweight structures may float upwards to the ground surface and foundations may displace laterally, causing structural failures. Liquefaction can also lead to massive lateral movements (or lateral flow). These aspects are illustrated in Figures 5-6 and 5-7.

While liquefaction has been reported in numerous earthquakes, nowhere has the phenomenon been more dramatically illustrated in modern times than in the Niigata, Japan earthquake of 1964 and in the Alaska earthquake the same year. These two earthquakes helped identify liquefaction as a major problem in earthquake engineering and much was learned from examination of soil behavior in these two events. More recently, liquefaction was one of the dominant causes of damage in the 1989 Loma Prieta earthquake in California, the 1995 Kobe earthquake in Japan, the 1999 Kocaeli earthquake in Turkey, and the 1999 Chi-Chi earthquake in Taiwan.

The current approaches for evaluating the potential for triggering liquefaction and for developing defensive measures were instigated by the results of field studies following the occurrence of the 1964 Anchorage and Niigata earthquakes. These were followed by extensive laboratory studies on small and large samples tested on shaking tables to evaluate the factors that influence the onset of liquefaction, the consequences of liquefaction, and the means by which to mitigate these consequences. Field-based procedures for assessing the liquefaction potential were developed to overcome many of the difficulties of obtaining representative samples of cohesionless soils.

Much progress has been made regarding soil liquefaction over the past 49 years, since the occurrence of the 1964 Alaska and the 1964 Niigata earthquakes. Many of the issues associated with the triggering, the consequences, and the mitigation of liquefaction have been raised and addressed over this time span.

The triggering of liquefaction is currently assessed for most projects using field-based procedures, which include the use of the standard penetration test (SPT), the cone penetration test (CPT), shear wave velocity measurements (Vs), and the Becker penetration test (BPT). The mostly commonly used procedures are the SPT-based and the CPT-based with sampling. These procedures are summarized in the proceedings of the recent NCEER/NS Workshop (1997), and most recently updated by Seed et al. (2001) and by Idriss and Boulanger (2003).

The consequences of liquefaction could include one or more of the following:

- Settlements, which can be of the order of 5 percent of the thickness of the liquefied soil layer(s)—such settlements can be uniform in some cases, but are mostly abrupt and nonuniform.
- Loss of lateral support; e.g., piles extending to or through the liquefied soil layer(s).
- Loss of bearing support.
- Flotation of buried structures such as underground tanks.
- Increased lateral pressures against retaining structures such as quaywalls.
- Lateral spreads (limited lateral movements).
- Lateral flows (extensive lateral movements).

The options available include:

1. Accept the risk.
2. Modify the design to accommodate the consequences.
3. Remediate to decrease or eliminate the consequences.

Occasionally, an optimum solution might consist of a combination of remediation to decrease the consequences and modification of design to accommodate the decreased consequences. Mitchell (1998) provides an excellent summary of the various techniques available for mitigating liquefaction, including the performance of a number of remediated sites during recent earthquakes.

Caltrans conducts liquefaction assessments using the current field-based procedures, as appropriate, and has completed several remediation projects, such as the 805/8 Interchange in San Diego and the Oakland approach to the San Francisco-Oakland Bay Bridge.

5.3 Structural Performance of Highway Bridges

The structural performance of highway bridges during earthquakes depends on the characteristics of the free-field ground motions produced, soil-foundation-structure interaction (SFSI; see Section 5.4), fluid-structure interaction (FSI; see Section 5.5), and the force/deformation properties of structural components and members.

Prior to the 1971 San Fernando earthquake, the specified earthquake loading used by Caltrans in the design of highway bridges consisted of an equivalent horizontal static loading equal to 2, 4, or 6 percent of the dead weight of the structure, depending on foundation type. When using this loading in combination with other static loadings, working stress design at that time allowed a 33-1/3 percent increase in allowable stresses. The heavy damage to bridges during the San Fernando earthquake made it very clear that much higher seismic input must be specified for design if life-safety performance requirements are to be met. This realization initiated the Highway Bridge Seismic Retrofit Program by Caltrans, begun in 1971 and continuing to date.

In 1973, Caltrans issued new seismic design criteria for highway bridges, which greatly increased the specified seismic loading. For the first time, such loadings were specified in terms of response spectra, which depend on expected peak ground acceleration at bedrock level, site soil conditions, fundamental period of vibration of the structure, and a response modification factor to control inelastic deformations (damage level). The criteria allowed the response of Standard bridges to be designed to this response spectrum, using equivalent static loadings. The forces in internal components could be determined through linear static analyses. However, the response of complex or Nonstandard bridges was to be obtained using the response spectrum method of linear dynamic analysis. These Seismic Design Criteria for Bridges, first issued by Caltrans in 1973, were still in effect at the time of the 1989 Loma Prieta earthquake.

Since the 1989 Loma Prieta earthquake, Caltrans has continued to upgrade its Seismic Design Criteria for Bridges as new knowledge is gained through Caltrans seismic research efforts and strong motion measurements obtained during recent earthquakes: 1994 Northridge, California; 1995 Kobe, Japan; 1999 Chi-Chi, Taiwan; and 2002 Ismit, Turkey. Design deficiencies revealed by the damage produced in these earthquakes have led to improved analytical seismic performance assessments and both analytical and experimental research.

5.3.1 Ground Motion at Bridge Site

Assessing the seismic performance of a particular bridge structure requires characterization of the seismic excitation specified, whether it is for a functional level event (FEE) or a safety evaluation event (SEE) . Such characterization is represented by an acceleration response spectrum for each component (*x*, *y*, and *z*) of free-field ground motion expected at its site. The individual spectral values represent the seismic inputs to

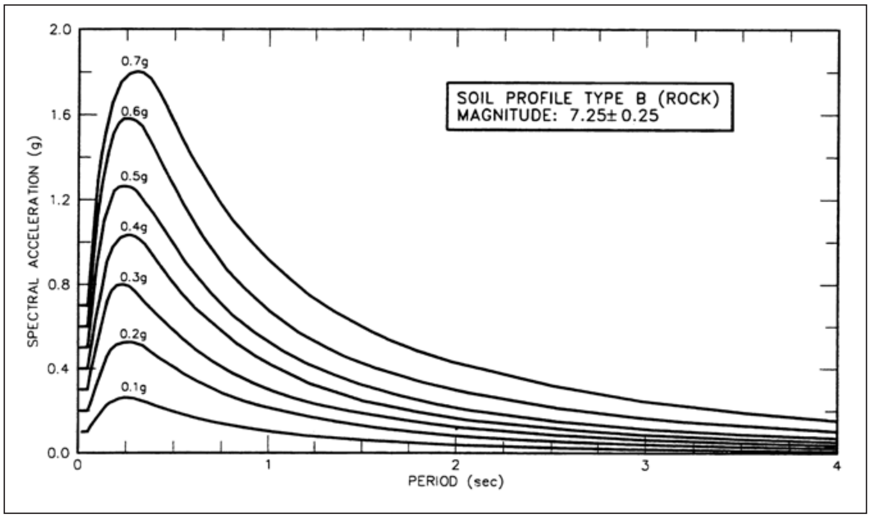


Figure 5-8. *Acceleration response spectrum curves for discrete values of peak ground acceleration.*

the response spectrum method of linear dynamic analyses mentioned above.

Currently, the acceleration response spectra used by Caltrans to represent horizontal (*x* and *y*) components of free-field motion are those developed by the Applied Technology Council (ATC) under Caltrans support. Twelve sets of these spectra, representing four different Site Classes (B, C, D, and E) and three different earthquake magnitude ranges (M = 6.5 ± 0.25, 7.25 ± 0.25, and 8.5 ± 0.25) were developed, as shown in the 1996 ATC report entitled *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations.* Site Classes B, C, D, and E (ATC-32 1996) are those conditions as defined in the National Earthquake Hazard Reduction Program (NEHRP) report entitled *Recommended Provisions for the Development of Seismic Regulations for New Buildings and Other Structures*, Vol. I (NEHRP 1997).

One of the 12 sets of spectra (Figure 5-8) represents California-type rock condition (B) and the magnitude range M = 7.25 ± 0.25. The individual curves in this set represent different peak ground acceleration levels ranging from 0.1g to 0.7g (g = acceleration of gravity = 32.2 ft/sec^2 = 981 cm/sec^2). The peak acceleration level used to assess the performance of a particular bridge structure to the SEE earthquake condition is that value shown on the 1996 California Seismic Hazard Map at the bridge's site location. All spectral values represent reasonable upper-bound mean values. For some Important bridges, SEE earthquake ground motionsare characterized by deterministically developed site-specific acceleration response spectra representing mean plus one standard deviation (m + 1σ) values.

For the Caltrans Toll Bridge Seismic Safety Program, probabilistically developed site-specific uniform hazard response spectra have been used to assess seismic performance. The 1,500-year return period spectrum has been used to represent the SEE earthquake condition; and, a lower level return period spectrum has been used to represent the FEE earthquake condition. For these Important bridges, nonlinear time history analyses have been carried out to assess seismic performance using three components (*x*, *y*, and *z*) of response spectrum-compatible time histories of motion as input at each pier location.

5.3.2 Performance Objectives

The design of bridge structures to perform satisfactorily under expected seismic conditions requires that realistic earthquake inputs be specified and that structural components be proportioned and detailed to resist these and other combined loadings within the limits of specified performance criteria.

Shortly after the 1989 Loma Prieta earthquake, Caltrans, with advice from the Seismic Advisory Board, established two performance levels and two groundmotion intensity levels for the design and evaluation of bridges (Table 5-2).

The definitions of terms incorporated in Caltrans Memo to Designers 20-1, January 1999 in Table 5-2 are:

Immediate Service Level. Full access to normal traffic available almost immediately.

Limited Service Level. Limited access (reduced lanes, light emergency traffic) possi-

Table 5-2. *Performance levels and ground motion intensities (Caltrans Memo to Designers 20-1, January 1999).*

Ground Motion at Site	Standard Performance Level	Important Bridge Performance Level
Functional Evaluation	Not Applicable	Immediate Service Level Minimal Damage
Safety Evaluation	Limited Service Level Significant Damage	Immediate Service Level Repairable Damage

See Table 10-3 for toll bridges.

ble within days. Full service restorable within months.
Minimal Damage. Essentially elastic performance.
Repairable Damage. Damage that can be repaired with a minimum risk of losing functionality.
Significant Damage. A minimum risk of collapse, but damage that would require closure for repair.
Important Bridge. One or more of the following items present. Important and Nonstandard bridges

- Bridge is required to provide secondary life safety (e.g., access to an emergency facility)
- Time for restoration of functionality after closure creates a major economic impact.
- Bridge is formally designated as critical by a local emergency plan.

Safety Evaluation Ground Motion. Up to two methods of defining ground motions may be used.

- Deterministically assessed ground motions from the maximum possible earthquake as defined by the Division of Mines and Geology Open-File Report 92-1 (1992).
- Probabilistically assessed ground motions with a long return period (approx. 1,000-2,000 years).

For Important bridges, both methods shall be given consideration. However, the probabilistic evaluation shall be reviewed by a Caltrans-approved consensus group, that is, an assigned group of Caltrans engineers who are independent of the design team for the specific bridge. For all other bridges, ground motions shall be based only on the deterministic evaluation. In the future, the role of the two methods for other bridges should be reviewed by a Caltrans-approved consensus group.

Functional Evaluation Ground Motion. Probabilistically assessed ground motions that have a 40 percent probability of occurring during the useful life of the bridge. The determination of this event shall be reviewed by a Caltrans-approved consensus group. A separate Functional Evaluation is required only for Important bridges. All other bridges are only required to meet specified design requirements to ensure Minimum Performance Safety Evaluation Level compliance.

The above performance criteria have not changed since Caltrans implemented them in the early 1990s.

5.3.3 Assessment of Seismic Performance

The procedure used by Caltrans to assess the seismic performance of a particular bridge structure depends on the type of structure—Standard or Nonstandard, Ordinary or Important—and the level of seismic excitation, Functional Evaluation Earthquake (FEE), or Safety Evaluation Earthquake (SEE).

Standard Bridges
Standard new bridges in California are designed in accordance with the Caltrans Bridge Design Specifications (BDS), supplemented by the Caltrans Seismic Design Crite-

ria (SDC), and standard design aids and details. Standard bridges are defined by Caltrans as meeting all of the following requirements:

1. Span lengths less than 300 feet.
2. Constructed with normal weight concrete girders and column or pier elements.
3. Horizontal members either rigidly connected, pin connected, or supported on conventional bearings (isolation bearings and dampers are considered nonstandard components).
4. Dropped bent caps or integral bent caps that terminate inside the exterior girder. C-bents, outrigger bents, and offset columns are considered nonstandard components.
5. Foundations supported on spread footings, pile caps, or pile shafts.
6. Founded on soil that is not susceptible to liquefaction, lateral spreading, or scour.

Initial design of the bridge is performed to establish member sizes based on the gravity load demands and experience regarding anticipated lateral loads. The ground motion at bedrock level is obtained from the Caltrans deterministic hazard map and a standard spectral shape (ARS plot) for the ground motion. The soil class is selected from the BDS. Ground motion effects on the initial design are determined from a mathematical model of the design subjected to a global linear spectral response analysis (simple, Standard bridges may be analyzed using equivalent static analysis). These effects are combined with gravity load effects to establish displacement demands for the vertical lateral load resisting elements (e.g. columns, piers, and pier walls). The displacement demands are compared with the available displacement capacity of the elements, the design is revised as required, and the procedure is iterated as necessary. The remaining structural components are designed to remain elastic based on the yield capacity of the vertical resisting elements.

Nonstandard Bridges

When Caltrans assesses the performance of a Nonstandard bridge structure, i.e., one having abrupt changes in mass, stiffness, and/or geometry, a separate linear response spectrum multi-modal analysis of a multi-degree-of-freedom (MDOF) system is used at the discretion of the engineer, but is not mandatory. Modeling is often carried out for each of three (*x*, *y*, and *z*) rigid boundary inputs, as defined by their corresponding acceleration response spectra. Displacement demand/capacity ratios are determined for each element as the basis for evaluation. Force reduction factors and force demand/capacity ratios are no longer used.

The robustness of the design depends very much on the amount of redundancy in the structural system. If the system is highly redundant, the distribution of internal forces will change each time an individual component undergoes yielding, which will continue until an analytical collapse mechanism is reached. Nevertheless, the results of the linear response spectrum analysis will provide guidance toward making effective modifications to the initial design, leading to an improved design in terms of meeting the specified FEE or SEE performance criteria. The structure can continue to displace well beyond yield until one of its ductile components reaches deformation capacity.

As in the case of a regular structure, assessing the performance of the preliminary design of a Nonstandard structure under the SEE condition should focus primarily on evaluating global displacements and deformations in those individual components that experience yielding. For an Important bridge, a response spectrum modal analysis, along with response modification factors, is not recommended at this stage of the design process. Rather, nonlinear finite-element modeling of the overall system, including foundations, should be established for use in carrying out nonlinear time history analyses with the objective of determining maximum values of component deformations, which can be compared with their corresponding deformation capacities. Deformation capacity of a member is defined as that deformation level at which the member's resistance starts to decrease with increasing deformation.

In carrying out these nonlinear time history analyses, simultaneous three-dimensional (*x*, *y* and *z*) response spectrum-compatible time histories of seismic input should be used, since superposition of separate solutions is no longer valid due to the nonlinear character of response. Further, for a long structure strongly coupled along its alignment, multiple-span segments of the total structure should be modeled; and, simultaneous three-component time histories of seismic input should be applied at each pier location. From pier to pier, these inputs should possess appropriate spatial characteristics reflecting realistic wave passage, wave scattering, and local site response effects; and, as mentioned previously, if located in the near-field to a controlling seismic source, each input should possess an appropriate velocity pulse (or pulses). Such velocity pulses will most likely dominate the critical nonlinear response of a bridge structure in such a location.

In assessing the final design performance of an Important Nonstandard bridge structure under the SEE condition, it is recommended that a minimum of three independent sets of three-component seismic inputs be applied to the nonlinear model separately, and that the largest of the resulting maximum values of critical response be used in assessing performance. This recommendation is made because of the large variations in critical response that usually occur due to nonlinear effects.

Nonstandard and Important bridges are often peer-reviewed.

5.3.4 Future Improvements Needed

Significant changes in seismic design criteria and performance assessment capabilities have taken place since the 1971 San Fernando earthquake. These changes have primarily been a result of advances in:

- Predicting the characteristics of expected free-field ground motions during future seismic events.
- Advancing linear and nonlinear modeling and dynamic analysis capabilities.
- Changing design detailing to satisfy strength/ductility requirements and to avoid brittle failures.

- Applying statistical and probabilistic methods to characterizing expected ground motions and structural behavior.
- Recognizing and quantifying uncertainties in all aspects of bridge engineering.

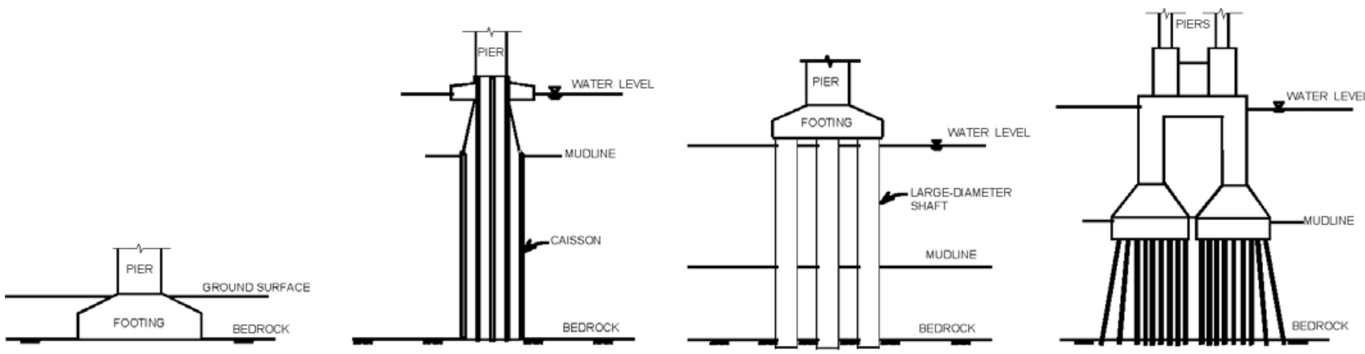
However, much remains to be done. As indicated above (Section 5.3.2), performance levels implied in the various performance objectives require an assessment of the structural and nonstructural damage that might affect the function of the bridge under specified ground motions. For Important bridges located close to an active fault, near-fault ground motions (Section 5.1.2) should be specified. The large long-period pulses in these motions are critical in evaluating expected damage. Improved guidelines should be developed for assessing seismic response of bridges to near-fault ground motions.

To satisfy performance objectives during the design phase, appropriate acceptance criteria must be developed to define the structural and nonstructural limit states for the desired levels of performance. Current practice relies primarily on strain limits for structural components (e.g. strain limits for reinforcing steel and concrete) but more needs to be done through testing and analytical studies to define the realistic limit states that control the functional performance of a bridge.

5.4 Soil-Foundation-Structure Interaction

In designing Standard bridges supported on pile foundations, or single shafts where the foundation soils are competent (standard penetration blow count N > 20), full fixity is normally assumed for the structure at the pile cap level or at about 3 pile diameters below that for structures supported on pile groups. Free-field ground motions are then specified as seismic inputs at the points of full fixity. When foundation soils are not competent (soft soils), soil-foundation-structure interaction effects are considered by evaluating foundation stiffness at the pile cap level using the empirical p-y method, as discussed below.

For large massive bridges, such as California's toll bridges, soil-structure-interaction can have a major effect on their seismic response to major earthquakes. Recognizing this fact at the beginning of the Caltrans Toll Bridge Seismic Safety Program, and recognizing the lack of adequate guidelines within Caltrans for treating SFSI, the Caltrans Seismic Advisory Board (SAB) recommended at its meeting on December 19, 1995 that a special committee be appointed to develop such guidelines. As a result of this recommendation, the SAB Ad Hoc Committee on SFSI was established in January 1996 consisting of: Joseph Penzien (Chair), International Civil Engineering Consultants, Inc.; Abbas Abghari, Caltrans; John F. Hall, California Institute of Technology; I.M. Idriss, University of California at Davis; Ignatius Po Lam, Earth Mechanics, Inc.; Brian H. Maroney, Caltrans; Joseph P. Nicoletti, URS Consult-



(a) Spread footing (b) Caisson (c) Large diameter shafts (d) Slender pile group

Figure 5-9. *Bridge foundation types.*

ants; Frieder Seible, University of California at San Diego; and Wen S. Tseng, International Civil Engineering Consultants, Inc. The final report of the SAB Ad Hoc Committee on SFSI entitled *Seismic Soil-Foundation-Structure Interaction* was submitted to Caltrans early in 1999 (SAB 1999a).

5.4.1 What is SFSI?

The 3-D (*x*, *y*, and *z*) motions produced by an earthquake at ground surface and at depth, in the absence of any structure and supporting foundation at a site, are referred to as "free-field" ground motions. If, for example, a heavy rigid mat foundation should be resting on the ground surface at the time of an earthquake, without the presence of a structure, it will be put into motion as a result of the forces that develop at the interface of the soil and the mat. The resulting inertia forces in the mat will feed back into the soil at the location of the soil/mat interface, causing the ground motions to be modified from the free-field motions. The modified motions will, in turn, alter the motion of the mat and the corresponding feedback forces into the soil. This back and forth coupled effect between soil and foundation is known as "soil-foundation interaction" (SFI).

If a structure should be resting on the mat foundation at the time of an earthquake, it will be put into motion as a result of the forces that develop at the interface of the foundation and the structure. The resulting inertia forces in the structure will feed back into the mat foundation, causing its motions to be different from the motions of the coupled soil/mat system at this same location. These modified motions of the mat will, in turn, alter the motions of the structure and the corresponding feedback forces into the mat. This back and forth coupled effect between foundation and structure is known as foundation-structure interaction (FSI). The combined coupled soil-foundation and foundation-structure interactions, which are taking place simultaneously, are referred to as soil-foundation-structure interaction (SFSI). While a mat foundation is used above to illustrate SFSI, the same interaction behavior applies to other types of foundations as well.

5.4.2 Types of Bridge Foundations

There are four general types of bridge foundations: spread footing, caisson, large-diameter shafts, and slender-pile group (Figure 5-9).

Spread Footings

Spread footings bearing directly on soil or rock are used to distribute the concentrated forces and moments at the base of bridge piers and/or abutments over sufficient areas to allow the underlying soil strata to support such loads within allowable soil bearing pressure limits. Of these loads, the lateral forces are resisted by a combination of friction on the foundation bottom surface and passive soil pressure on its embedded vertical face. Spread footings are usually used on competent soils or rock, which have high allowable bearing pressures. These foundations may be of several forms, such as 1.) isolated footings, each supporting a single pier column or wall pier, 2.) combined footings, each supporting two or more closely spaced bridge columns, and 3.) pedestals, which are commonly used for supporting steel bridge columns where it is desirable to terminate the structural steel abovegrade for corrosion protection. Spread footings are generally designed to support the superimposed forces and moments without uplifting or sliding. As such, inelastic action of the soils supporting the footings is usually not significant.

Caissons

Caissons are large structural foundations, usually constructed in water in a manner that will permit dewatering to provide a dry condition for excavation and construction of the bridge foundations. Caissons can take many forms to suit specific site conditions and can be constructed of reinforced concrete, steel, or composite steel and concrete. Most caissons are in the form of a large cellular rectangular box or cylindrical shell structure with a sealed base. They extend up from deep firm soil or rock bearing strata to above mudline, where they support the bridge piers. The cellular spaces within the caissons are usually flooded and filled with sand to some depth for greater stability. Caisson foundations are commonly used at deepwater sites that have deep soft soils. Resistance to the imposed forces and moments from a single pier takes place by direct bearing of the caisson base on its supporting soil or rock stratum and by passive resistance of the side soils over the embedded vertical face of the caisson. Since both the soil bearing area and the structural rigidity of a caisson are very large, the transfer of forces from the caisson into the surrounding soil usually involves negligible inelastic action at the soil/caisson interface. In some cases, the seismic retrofits were designed to include foundation rocking where soil interactions become an important aspect of the design process. As a note, new bridge designs typically do not rely on rocking.

Large Diameter Shafts

These foundations consist of one or more reinforced concrete cast-in-drilled-hole (CIDH) or concrete cast-in-steel-shell (CISS) piles of large diameter, usually in the range of 4 to 12 ft (1.22 to 3.66 m). Such shafts are embedded in the soils to sufficient depths to reach firm soil strata or rock where a high degree of fixity can be achieved, thus allowing the forces and moments imposed on the shafts to be safely transferred to the embedment soils within allowable soil bearing pressure limits and/or allowable foundation displacement limits. The development of large diameter drilling equipment has made this type of foundation economically feasible; thus, its use has become increasingly popular. In actual application, the shafts often extend above the ground surface or mudline to form a single pier or a multiple-shaft pier foundation. Because of their larger expected lateral displacements, compared to those of a large caisson, a moderate level of local soil nonlinearity is expected to occur at the soil/shaft interfaces, especially near the ground surface or mudline. Such nonlinearities should be considered in design.

Slender Pile Groups

Slender piles refer to those piles having a diameter or cross-section dimension of less than 2 ft (0.61 m). These piles are usually installed in a group and provided with a rigid cap to form the foundation of a bridge pier.

Piles are used to extend the supporting foundations (pile caps) of a bridge down through poor soils to more competent soil or rock. A pile's resistance to vertical load may be essentially by point bearing when it is placed through very poor soils to a firm soil stratum or rock, or by friction in the case of piles that do not achieve point bearing. In real situations, the resistance to vertical load is usually achieved by a combination of point bearing and friction. Resistance to lateral loads is achieved by a combination of soil passive pressure on the pile cap, soil resistance around the piles, and bending of the piles. The uplift capacity of a pile is generally governed by the soil friction or cohesion acting on the perimeter of the pile. Piles may be installed by driving or by casting in drilled holes. Driven piles may be timber piles, concrete piles with or without prestress, steel piles in the form of pipe sections, or steel piles in the form of structural shapes (e.g., H shape). Cast-in-drilled-hole piles are reinforced concrete piles installed with or without steel casings. Because of their relatively small cross-section dimensions, soil resistance to large pile loads usually develops large local soil nonlinearities that must be considered in design. Furthermore, since slender piles are normally installed in a group, mutual interactions among piles will reduce overall group stiffness and capacity. The amounts of these reductions depend on the pile-to-pile spacing and the degree of soil nonlinearity developed in resisting the loads.

5.4.3 Modeling of Soil-Foundation Interaction (SFI)

Depending on foundation type and its soil support condition, the modeling of SFI effects for bridge foundations can be classified into two categories: 1.) the so-called "elasto-dynamic" method developed and practiced in the nuclear power industry for large foundations and 2.) the so-called "empirical p-y" method developed and practiced in the off-shore industry for pile foundations

The fundamental elements of the elasto-dynamic method are the constitutive relations between an applied harmonic point load and the corresponding dynamic response displacements that take place within a uniform or layered half-space medium. These constitutive relations, called dynamic Green's functions, allow the formulation of a complex frequency-dependent impedance (dynamic stiffness) matrix relating a set of discrete interaction forces at the soil/foundation interface to a corresponding set of discrete relative motions between the foundation and free-field soil. The real parts of the coefficients in this matrix represent stiffness and inertia effects in the soil; while the imaginary parts represent radiation and material damping effects. Using standard finite element modeling of stiffness, mass, and damping within the foundation, along with specified 3-D free-field soil motions, the seismically-induced dynamic motions at discrete locations in the foundation can be evaluated. Of particular interest are the time histories of motion of the foundation at its intended interface location with a structure once constructed. These particular motions are referred to as foundation "scattered" motions. In addition to evaluating dynamic motions at discrete locations, corresponding stresses and deformations can be evaluated consistent with the foundation's finite element model.

The fundamental elements of the empirical p-y method are discrete sets of nonlinear lateral "p-y," axial "t-z," and axial pile-tip "Q-d" springs, which are attached to piles, and characterize the local soil resistances to pile motions. Construction of the nonlinear p-y, t-z, and Q-d relations at each specified depth location depends mainly on soil material strength parameters, i.e., the friction angle, *f*, for sands and the cohesion, *c*, for clays. Since these relations are nonlinear, the equivalent linear procedure using secant moduli is normally used to establish a matrix relating a set of discrete interaction forces at the soil/pile interfaces to a corresponding set of discrete relative motions between piles and free-field soil. Since the p-y, t-z, and Q-d relations are based on static or pseudo-static test results, they cannot represent inertia and damping effects in the soil. Thus, the equivalent linear soil stiffness matrix, so obtained, is a real valued constant coefficient matrix applicable at zero frequency.

The coefficients are, however, functions of the amplitudes of the relative displacements between piles and free-field soil. Using the soil stiffness matrix for assumed values of these relative displacements, along with a finite element stiffness matrix of the pile group, including pile cap, and specified 3-D free-field soil motions, seismically induced motions within the entire pile foundation can be evaluated. Should the amplitudes of the relative displacements between piles and free-field soil be significantly different from their assumed values, the equivalent linear coefficients in the soil stiffness matrix should be modified accordingly and be used in a reevaluation of the seismically-induced motions within the entire foundation. By such iterations, motions are finally obtained that are compatible with the coefficients in the soil stiffness matrix. The final pile cap motions obtained by this iterative procedure are referred to as foundation "kinematic" motions, since inertia and damping effects in the entire soil/pile-group system have not been included.

5.4.4 Modeling of Soil-Foundation-Structure Interaction (SFSI)

Soil-foundation-structure interaction is the integration of these three elements to evaluate the overall performance of the structural system. Consider a large bridge structure supported on caisson foundations. The soil/caisson system can be modeled as described in Section 5.4.3, making use of the elasto-dynamic method in treating soil-foundation interaction. Subjecting this system to specified seismically-induced free-field soil motions, the resulting caisson stresses, deformations, and scattered motions can be obtained. Further, subjecting the model of this soil/caisson system to a harmonic force component (force or moment) in each degree of freedom (DOF) permitted by the model at the intended foundation/structure interface location, the corresponding harmonic displacement components (translation and rotation) in all the interface DOF can be obtained.

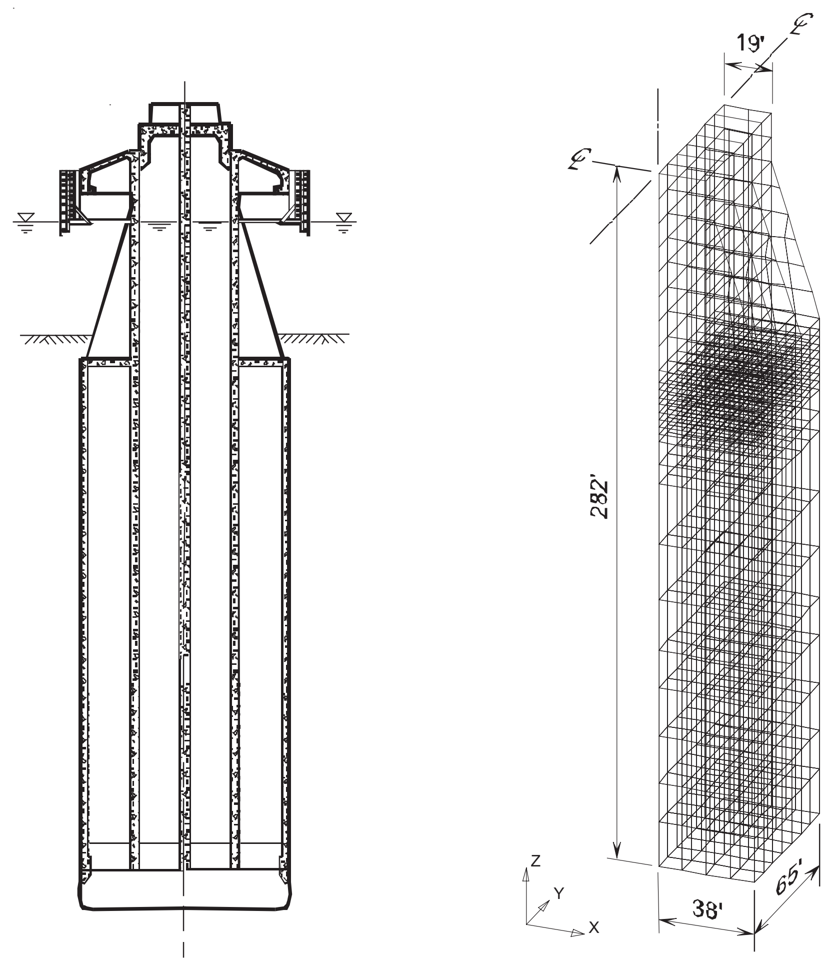


Figure 5-10.
Caisson foundation and its model for one quadrant.

By doing so, a foundation impedance matrix that is complex and frequency-dependent is obtained. The real parts of the coefficients in this matrix represent mass and stiffness effects in the soil-foundation system; while, the imaginary parts represent corresponding material and radiation damping effects.

When nonlinearities develop in the bridge's superstructure under the specified seismic condition, the equations of motion of the entire structure above the foundations must be expressed in the time domain in order to permit a solution. The interaction forces can then be evaluated and applied to the isolated foundation system to evaluate stresses and deformations within the separate foundations. Of course, when nonlinearities are developed under the specified seismic input, these terms must be changed to their appropriate nonlinear hysteretic forms. The most common types of nonlinearities that occur in structures have been modeled and implemented into computer programs. The resulting nonlinear equations of motion can then be solved by numerical step-by-step methods yielding displacement interaction force vectors for elements of the bridge.

If, instead of caisson foundations as described above, the bridge structure is supported on groups of slender piles, the empirical p-y method would be used in treating soil-foundation interaction. Subjecting each soil/pile group system to specified seismically-induced free-field soil motions, the resulting pile stresses and deformations and the pile cap scattered motions can be obtained. Further, subjecting the model of each soil/pile group system to a harmonic force component (force or moment) in each DOF permitted by the model at the intended pile cap/structure interface location, the corresponding harmonic displacement components (translation and rotation) in all of the interface DOF can be obtained. By doing so, a foundation stiffness matrix that has constant coefficients is obtained.

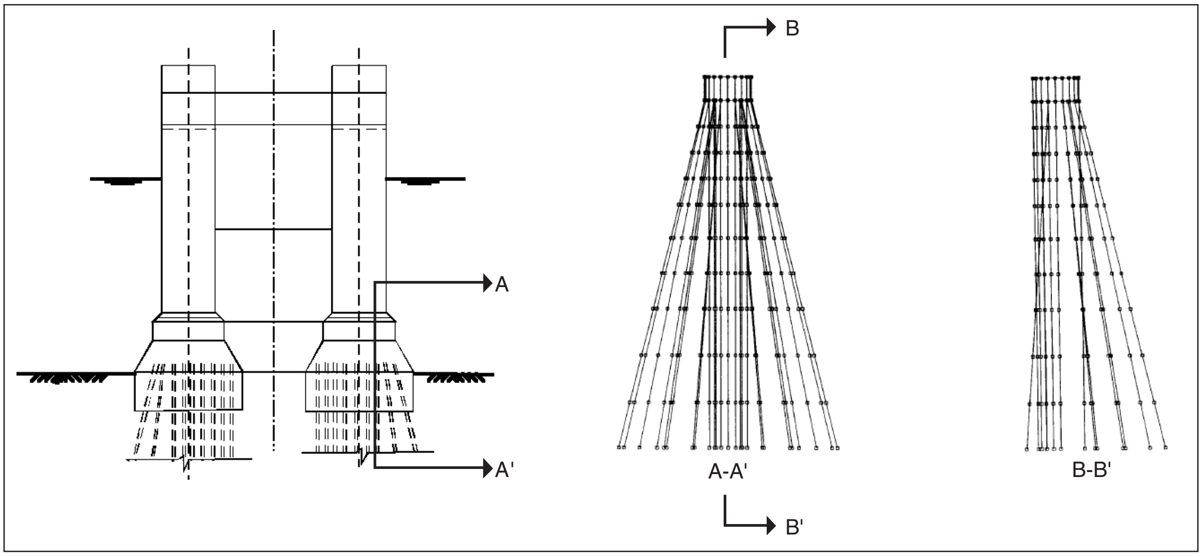


Figure 5-11. *Slender pile group foundation and a portion of the beam-element half-model.*

The finite-element model of the entire bridge structure above its pile-group foundation is combined with the foundation stiffness matrix of assembled individual foundation matrices, and the combined vector of individual kinematic foundation-motion vectors can be determined.

Since the p-y method of treating SFI makes use of the equivalent linear procedure to obtain secant moduli (Section 5.4.3), iteration must be followed in treating SFSI effects on the seismic response of the complete coupled soil-foundation-structure system so that the final secant moduli are compatible with the relative displacements produced in the p-y, t-z, and Q-d springs.

The procedure described above for treating SFSI, using either the elasto-dynamic or p-y method of modeling SFI, makes use of the substructure method of developing the equations of motions for each soil-foundation system isolated from the structure, and then couples these equations of motions for all foundations to a set of separately developed equations of motion for the structure. This coupling of equations of motion is carried out in such a manner to satisfy compatibility conditions at the interfaces between structure and foundation. The advantage of such substructuring is to reduce the number of degrees of freedom when treating the SFSI of the complete bridge system. Some engineers prefer, however, to initially formulate the equations of motion of the coupled soil-foundation-structure system, thus avoiding the substructuring procedure.

A deeply embedded caisson foundation (Figure 5-10) of a large bridge on San Francisco Bay, along with its quadrant model, was analyzed for SFI effects using the elasto-dynamic method. A slender pile group foundation of another Bay-crossing bridge (Figure 5-11), along with its beam-element half-model, was analyzed for SFI effects using the p-y method.

5.4.5 Demand Versus Capacity Analyses

Evaluation of the seismic performance of a bridge involves two parts: 1.) determing seismic demands using the methodologies described above (Section 5.4.4), and 2.) evaluating the corresponding seismic resistance capacities. The objective of the capacity evaluation is to determine the most probable levels of seismic resistance of the various elements, components, and subsystems of the bridge. The resistance capacities provided by this evaluation, along with the corresponding demands, provide the basis for judging seismic performance of the complete bridge system during future earthquakes. In the context of SFSI discussed herein, the capacity evaluation focuses on soil-foundation systems.

For a bridge subjected to static loadings, the soil-foundation capacities of interest are the load resistances and the associated foundation deflections and settlements. Their evaluation constitutes the bulk of the traditional foundation design problem. When the bridge is subjected to oscillatory dynamic loadings, including seismic, the static capacities mentioned above are, alone, insufficient input to the complex process of judging soil-foundation performance. In this case, it is necessary to assess entire load-deflection relationships, including their cyclic energy dissipation characteristics, up to load and/or deformation limits approaching failure conditions in the soil-foundation system. Because of the complexity of this assessment, the capacity evaluation must be simplified in order to make it practical. This is usually done by treating each soil-foundation system independently and by subjecting it to simplified pseudo-static monotonic and/or cyclic deformation-controlled step-by-step patterns of loading, referred to as "pushover" loading.

5.4.6 Future Development

From the discussions in Sections 5.4.3 and 5.4.4, characterization of the soil-foundation interaction forces for seismic demand analysis purposes can be achieved using either the elasto-dynamic model or the empirical p-y model for the soil medium, each of which has its own merits and deficiencies. The elasto-dynamic model is capable of incorporating soil inertia, damping (material and radiation), and stiffness characteristics; and, it can incorporate the effects of global soil nonlinearities induced by the free-field soil motions in an equivalent linearized manner. However, it suffers from the deficiency that it does not allow for easy incorporation of the effects of local soil nonlinearities. On the other hand, the empirical p-y model can appropriately capture the effects of local soil nonlinearities, but its deficiency is not being able to properly simulate soil inertia and damping effects, and it cannot treat the effects of global soil nonlinearities. Since the capabilities of the two models are mutually complimentary, it is logical to combine the elasto-dynamic model with the empirical p-y model by connecting them in series such that the combined model has the desired capabilities of both models. This combined model is referred to as the "hybrid model" (SAB 1999a).

The elasto-dynamic method of treating SFSI is valid for foundations having large horizontal dimensions, such as large spread footings and caissons, while the empirical p-y method is valid only for slender-pile foundations subjected to large-amplitude deflections. For foundations intermediate between these two classes, e.g., those using large diameter shafts, both of these methods are deficient in predicting SFSI behavior. In this case, the hybrid method of modeling has definitive advantages, and further, it has the potential to treat all classes of foundations with reasonable validity. The hybrid method does, however, need further development, refinement, and validation to make it fully acceptable for bridge applications.

5.4.7 SFSI Conclusions

Computer programs are now available to Caltrans for treating SFSI effects on the seismic response of large massive bridges using the methodologies described in Section 5.4. They also have available complete models of each existing toll bridge, which will allow nonlinear time-history analyses to be carried out assessing its dynamic response to future seismic events. While the methodologies have been greatly advanced in recent years, further improvements can be achieved as suggested in Section 5.4.6.

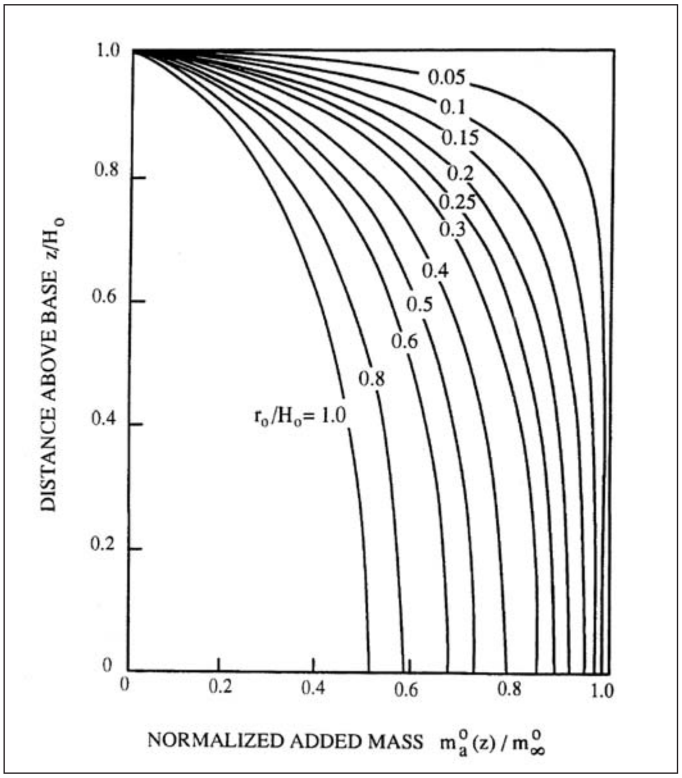


Figure 5-12. *Normalized hydrodynamic added mass curves. (Goyal and Chopra 1989a and 1989b)*

5.5 Fluid-Structure Interaction

As pointed out in Section 5.4, soil-structure interaction can have a major effect on the seismic response of large massive bridges such as California's toll bridges. For these bridges, large portions of the substructures (piers and/or foundations) extend downward through relatively deep water before reaching the mudline (Figure 5-9). In these cases, fluid-structure interaction can also have a major effect on seismic response. Thus, modeling of both types of interaction should be included when conducting seismic performance evaluations.

In treating fluid-structure-interaction, the fluid (water) is assumed to be incompressible and inviscid, leading to irrotational flow of the water around the structure. The mathematical formulation of this flow is expressed by Laplace's equation in terms of a three-dimensional time-dependent velocity potential $\phi(x, y, z, t)$. Having solved for this velocity potential, the time-dependent water pressure distribution over the surface of the structure can be evaluated.

Integrating this distribution over specified segments of the structure yields two resultant time-dependent forces. One of these forces is proportional to the relative acceleration between the free-field water and the structure; the other being dependent on relative velocity. The proportionality constant to the relative acceleration has units of mass; thus, it is referred to as the hydrodynamic added mass. This added mass is combined with structural mass in formulating the inertia term in the equations of motion. The force, which depends upon relative velocity, represents hydrodynamic damping caused by surface waves moving away from the structure.

For bridge structures, it is usually sufficient to consider only the hydrodynamic added mass and neglect the hydrodynamic damping. For a cylinder (e.g., the leg of a pier or the shaft of a foundation with its axis positioned vertically in the water), the added mass is easily obtained using the set of curves in Figure 5-12 published by Goyal and Chopra (1989a and 1989b). In this figure:

$m_a^o(Z)$	is the hydrodynamic added mass per unit of height
m_∞^0	is the mass per unit height of the water displaced by the cylinder
H_o	is depth of the surrounding water
z	is distance above the base of the cylinder
r_o	is the radius of the cylinder

Note that the added mass decays to zero at the water surface.

For a vertically positioned structural member having an elliptical cross-section instead of the circular cross-section mentioned above, a set of curves similar to those in Figure 5-12 were published by Goyal and Chopra making it easy to obtain the hydrodynamic added mass to be used for motion in each principal direction. This set of curves for the elliptical cross-section, as well as those shown in Figure 5-12 for the circular cross-section, were obtained by closed-form solutions of Laplace's equations. An approximate procedure is given by Goyal and Chopra for finding the hydrodynamic added mass for other cross-sections having symmetry about both axes making use of the set of curves for the elliptical cross-section.

The hydrodynamic added masses for bodies of arbitrary shape can be obtained through Laplace-type solutions, however not in closed form. They can however be obtained through numerical computer solutions (Yeung 1982).

5.6 Retrofit Versus New Design

5.6.1 Ordinary Standard Bridges

Design of new or retrofitted elements is governed by the same standards as new design to meet the performance objectives indicated in Sections 5.3.1 and 5.3.2. While the same ground motion is used for retrofit design as is used for new design, it is recognized that the design and detailing of an existing bridge cannot comply in all aspects with that for a new bridge, and some noncomplying features can be accepted without significantly compromising the seismic safety of the bridge. In the past, faced with limited funding for complete seismic retrofits and competing needs for action, Caltrans has sometimes only addressed the retrofit of only the most vulnerable components (e.g. installing restrainer cables and jacketing columns) of those governing the bridge's seismic response.

While this has generally been beneficial, it must be recognized that the partial retrofitting of selected components may have adverse effects on the seismic vulnerability of the remaining deficient elements. For example, the strengthening of a column by jacketing may not have yielded a bridge superstructure or foundation whose expected seismic performance is rated as acceptable. In some cases, the modification may have had an adverse effect on the other members of the superstructure and the foundations by transferring higher loads to column-to-deck connections that are already inadequately seismically reinforced. The Seismic Advisory Board recommends that, when adequate funds are not available for complete retrofits, consideration should be given to incremental retrofits that do not significantly adversely affect the remaining components and are compatible with the complete retrofit scheme.

The design of Ordinary Standard new bridges is generally not independently peer reviewed, but is reviewed by Caltrans engineer(s) not specifically part of the design team.

5.6.2 Important and Nonstandard Bridges

Important and Nonstandard bridges include all those bridges that do not meet the definition of Ordinary Standard bridges. The retrofit design of such bridges is generally independently peer reviewed, often internally within Caltrans. The representation of the ground motion, the performance objectives, and the analyses are the same as those for new bridges, but as indicated above, some noncomplying design and details may be accepted if there is significant cost saving without serious compromise to the seismic safety. Since many of these bridges contain unique or archaic components, simulated load testing and other investigations may be required to define their structural properties.

5.7 Special Problems: Wharves, Quaywalls, Tunnels, and Soundwalls

5.7.1 Wharves and Quaywalls

Quaywalls and marginal wharves are coastal structures constructed parallel to the shoreline to provide access and mooring of vessels and/or to support roadways for vehicle traffic.

Marginal Wharves

Marginal wharves consist of a pile-supported deck with a short cut-off wall at the inboard edge. Under Caltrans specifications, the slope of the soil material under the deck is governed by the width of the deck and the required dredged depth at the face of the wharf, but usually not steeper than 1.5 horizontal to 1 vertical. Rock rip-rap with a filter layer may be provided to prevent raveling of the slope. Wharves are usually constructed of reinforced concrete with vertical precast concrete piles. Batter piles are usually avoided in wharves because the greater stiffness of inclined piles as compared with that of vertical piles under lateral loads makes it difficult for them to effectively share lateral loads and also because it is difficult to achieve an adequate joint at the deck level.

In addition to the hazard associated with ground shaking, marginal wharves are also vulnerable to lateral spreading if the soils below the wharf are susceptible to liquefaction. In either case, the sloping soil below the deck, in which the piles are embedded, results in the inboard piles having a shorter effective length, and therefore greater lateral stiffness, than the outboard piles. These stiffer piles will attract a greater proportion of the lateral loads from ground shaking or lateral spreading and, because of their short effective length, will tend to be “shear critical,” rather than yielding in flexure. This condition can lead to brittle shear failure of these piles if the lateral loads have been underestimated.

Quaywalls

Quaywalls usually consist of a vertical bulkhead with an anchor system. The anchor system may rely on passive pressure of a concrete block against the soil, or it could consist of a pair of batter piles if the anchor cannot be reasonably constructed in competent soil. Alternative bulkhead systems could consist of cellular sheetpile cofferdams or, if the underlying soils are competent, gravity-type retaining walls may be used.

The seismic vulnerability of quaywalls is similar to that of marginal wharves. Inertia effects create lateral forces on the bulkhead that can be further increased if liquefaction occurs in the retained soil. Liquefaction could also reduce the passive pressures on the anchor block allowing lateral spreading and potential collapse of the bulkhead system.

5.7.2 Tunnels

Tunnel performance during an earthquake is related to the behavior of the lining and maintenance of vehicle access at portals. The linings of tunnels generally perform well compared to aboveground structures, since their deformations tend to conform closely to the deformations of the ground surrounding them. However, some damage to tunnel linings has occurred during earthquakes due to fault rupture through the lining and liquefaction of soils surrounding them. Some structural damage has also occurred at tunnel portals as a result of landslides, which blocked vehicle access (FHWA 2003).

Different types of tunnel linings are used, depending on the type and condition of surrounding ground materials. In the case of tunneling through rock, rock bolts are used to secure and prevent the fallout of rock blocks from between fractures. Shotcrete is then applied to fill crevices and smooth the surface. Further buildup of shotcrete with wire mesh can serve as a liner to protect against the fallout of small rocks. An alternative system of steel ribs and poured concrete lining is also used in tunnel construction. In some cases of railroad tunnels through rock, no linings have been installed.

Tunnels constructed in soft ground are of two types—bored, and cut-and-cover. Bored tunnels are lined by placing segmental linings, either steel or precast concrete, directly behind the tunneling shield as it moves forward. Grouting is then inserted into the void space between the lining and its surrounding soil. Cut-and-cover tunnels are constructed by excavating a wide trench, constructing the tunnel structure (usually a rectangular cast-in-place reinforced concrete box section with single or multiple cells) in the trench, and then backfilling along the sides and above the structure up to the initial grade level.

A subaqueous tunnel is constructed by floating single or multi-cell segments of prefabricated reinforced concrete tube sections into position, sinking them by introducing water into their cells, connecting the segments together in a prepared trench at the bottom, backfilling around and over all segments, and finally removing water and end diaphragms from all segments.

While the potential for seismic damage to tunnels is generally small compared to that for aboveground structures, damage can occur, especially when located near the contributing seismic source (source-to-site distances < 10 km). Such damage can be caused by:

- Fallout of a large block of rock due to inadequate rock bolting, resulting in a crushed lining.
- Landslide that has a failure surface through a tunnel, causing excessive shear distortion and/or partial collapse of the lining.

- Landslide above a portal that causes damage to its structure and/or blockage of the tunnel entrance.
- Fault rupture intersecting a tunnel, causing large shear distortions within a narrow zone at the fault line.
- Liquefaction of soils surrounding a tube or box section, causing flotation or sinking of the structure depending on relative weights of tube and surrounding soils.
- Racking of bored tunnel lining or cut-and-cover box section, causing excessive localized flexure deformations.
- Relative displacements across seismic joints at the ends of a tunnel that exceeds their corresponding capacities.

It is important that the risks associated with possible future damages to tunnels in California be assessed. In those cases of high risk, retrofit measures should be considered where feasible and cost-effective. When effective retrofit measures cannot be developed, e.g., in the case of potential damage due to fault rupture through a tunnel, contingency plans should be in place to repair the damages following a major seismic event.

5.7.3 Soundwalls

Soundwalls employed by Caltrans are typically constructed with 8-inch reinforced concrete block masonry. Walls at the edge of a roadway may be freestanding or, in some cases, such as adjacent to a cut slope, the lower portion of the wall may function as a retaining wall. Soundwalls have also been installed on the safety rail of highway bridges.

Information received from Caltrans indicates that, in the past, the walls were designed in accordance with the 1979 UBC, but current designs are based on the 1997 UBC. Design in accordance with these provisions apparently has raised some questions:

- The UBC ground motion in many cases is substantially lower than that obtained from the Caltrans seismic hazard map used in bridge design.
- Soundwalls on bridges are generally added to an existing bridge to reduce noise and are not addressed in the UBC.

Since the seismic risk associated with the failure of the soundwalls along the edge of a roadway is similar to that for the adjacent buildings, the Seismic Advisory Board considers compliance with the UBC for these walls appropriate. The use of the Mononobe-Okabe equations to derive the seismic forces on these walls from the retained soil is also considered appropriate.

The failure of soundwalls installed on bridges during an earthquake could affect the time required to re-establish service and become a critical factor in meeting the desired performance level of the bridge. The appropriate ground motionfor the design of these walls is considered to be that used in the design of the bridge. If the soundwalls are to be part of the original construction, they should be incorporated in the analytical model of the bridge and be designed for the indicated dynamic response.

Section 6

Evolution of Bridge Seismic Design Criteria

6.1 AASHTO Standard Specifications for Highway Bridges

The past 50 years have seen revolutionary changes in earthquake engineering as applied to transportation structures. This becomes apparent when one reviews the changes in seismic design criteria specified by the American Association of State Highway and Transportation Officials (AASHTO)* in its *Standard Specifications for Highway Bridges*, first (1931) through eleventh (1973) editions; and by AASHTO in its *1973 Interim Specifications for Highway Bridges* and the subsequent *Standard Specifications for Highway Bridges*, twelfth (1975) through sixteenth (1996) editions; and in AASHTO's *LRFD Bridge Design Specifications*, first (1994) and second (1999) editions. All of the above-mentioned specifications apply to Standard bridges having span lengths less than 500 feet.

6.1.1 AASHTO Standard Specifications for Highway Bridges, 1949-1961

The first reference to considering earthquake effects on bridges came in the fifth (1949) edition of the *Standard Specifications for Highway Bridges*, which stated that earthquake stresses should be considered. However, no guidelines for doing so were given. This same reference was stated again in the sixth (1953) and seventh (1957) editions.

* Prior to the early 1970s, the association was known as AASHO, the American Association of State Highway Officials.

6.1.2 AASHTO Standard Specifications for Highway Bridges, 1961-1975

The eighth edition (1961) of the *Standard Specifications for Highway Bridges* was the first to specify an earthquake loading for design (EQ), namely:

$$EQ = CD \qquad (6\text{-}1)$$

which was to be applied statically in any horizontal direction as part of a Group VII load combination given by:

$$\text{Group VII} = D + E + B + SF + EQ \qquad (6\text{-}2)$$

in which D, E, B, and SF denote dead load, earth pressure, buoyancy, and stream flow, respectively. The numerical values of C were specified to be 0.02 for structures supported on spread footings where the soil bearing capacity was rated to be greater than 4 tons/ft^2, 0.04 for structures supported on spread footings where the soil bearing capacity was rated to be less than 4 tons/ft^2, and 0.06 for structures founded on piles. The Group VII load combination was to be used in the working stress design (WSD) with a 1/3 increase in allowable stress because of the presence of the earthquake loading EQ. No seismic zone factors were provided in the 1961 specifications.

The above seismic loading provisions of the eighth edition (1961) of Standard Specifications for Highway Bridges were repeated, without modification, in the ninth (1965), tenth (1969), and eleventh (1973) editions. It should be noted that these seismic loading provisions were based mainly on the lateral

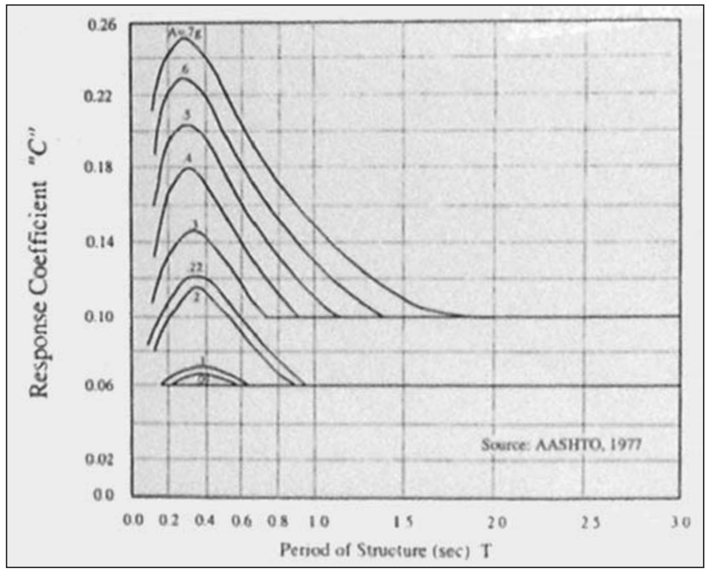


Figure 6-1. *Response coefficient "C" for values of peak acceleration "A." Depth of alluvium to rock-like material is 11-80 feet (3-24 m).*

force requirements for buildings developed prior to 1961 by the Structural Engineers Association of California (SEAOC).

6.1.3 AASHTO Standard Specifications for Highway Bridges, 1975-1992

As a result of the 1971 San Fernando, California, earthquake, during which several highway bridges were severely damaged and some even collapsed, the California Department of Transportation (Caltrans) issued new seismic design criteria for bridges in 1973, which formed the basis of the 1975 AASHTO *Interim Specifications for Highway Bridges*. The equivalent static lateral force loading specified in this document for bridges that have supporting members of approximately equal stiffness was of the form:

$$EQ = CFW \qquad (6\text{-}3)$$

which was to be applied in any horizontal direction as part of the same Group VII load combination given by Equation (6-2) in a working stress design with a 1/3 increase in allowable stress. In this equation, W represents dead load, F is a framing factor assigned the values 1.0 for single columns and 0.8 for continuous frames, and C is a combined response coefficient as expressed by:

$$C = ARS/Z \qquad (6\text{-}4)$$

in which A denotes maximum expected peak ground acceleration (PGA) as shown in a seismic risk map of the United States, R is a normalized (PGA = 1g) acceleration response spectral value for a rock site, S is a soil amplification factor, and Z is a force reduction factor that depends on structural-component type, and which accounts for the allowance of inelastic deformations. The numerical values specified for A were 0.09g, 0.22g, and 0.50g in seismic zones numbered I, II, and III, respectively.

Numerical values for R, S, and Z were not provided in the 1975 Interim Specifications; rather, four plots of C as functions of period T were given for discrete values of A. Each of these plots represents a different depth range of alluvium to rock-like material, namely 0-10 feet, 11-80 feet, 81-150 feet, or >150 feet. Figure 6-1 shows the AASHTO plot for the depth range 11-80 feet. Period T was evaluated using the single degree of freedom (SDOF) relation:

$$T = 0.32\sqrt{\frac{W}{P}} \qquad (6\text{-}5)$$

in which P equals the total uniform static loading required to cause a 1-inch horizontal deflection of the whole structure.

For complex or irregular structures, the AASHTO *Interim Specifications* required use of the modal response spectrum analysis method to generate design loads. Where fundamental periods were greater than 3 seconds, the AASHTO *Interim Specifications* required that they be designed using "current seismicity, soil response, and dynamic analysis techniques."

The same seismic design criteria in the 1975 *Interim Specifications* were repeated in the twelfth (1977), thirteenth (1983), and fourteenth (1989) editions of AASHTO's *Standard Specifications for Highway Bridges*,

Figure 6-2. *Peak rock acceleration map.*

however in these editions, the designer was given, for the first time, the choice of working stress design (WSD) or load factor design (LFD). When using WSD, the same Group VII load combination given by Equation (6-2) was specified, along with a 1/3 increase in allowable stress; however, when using LFD, the Group VII load combination was changed to the form:

$$\text{Group VII} = \gamma\,[\beta_D D + \beta_E E + B + SF + EQ] \tag{6-6}$$

in which load factor γ was assigned the value 1.3, β_D was assigned the values 0.75, 1.0, and 1.0 when checking columns for minimum axial load and maximum moment or eccentricity, when checking columns for maximum axial load and minimum moment, and for flexure and tension members, respectively, and β_E was assigned the value 1.3 for lateral earth pressure and 0.5 for checking positive moments in rigid frames.

6.1.4 AASHTO LRFD Specifications—First (1994) and Second (1999) Editions

The working stress design (WSD) philosophy, which requires that calculated design stresses not exceed specified levels, underwent adjustment in the 1970s through the introduction of load factors reflecting the variable predictabilities of different load types, a philosophy referred to as load factor design (LFD). During the period 1988 to 1993, the AASHTO *LRFD Bridge Design Specifications* were developed using statistically-based probability methods. The load and resistance factor design (LRFD) philoso-

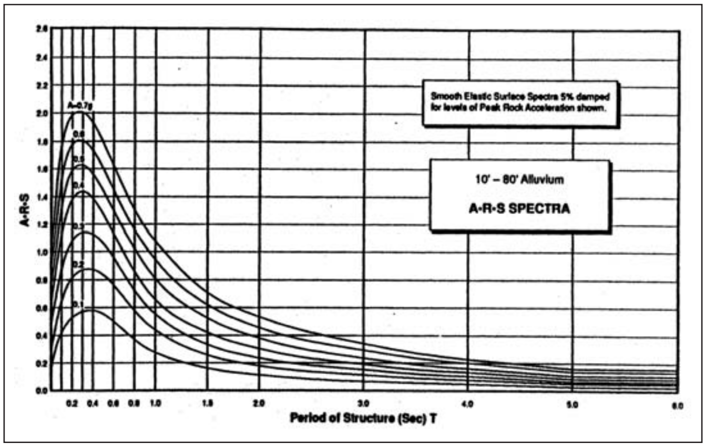


Figure 6-3. *Caltrans ARS spectra for 10-80' layer of alluvium.*

Table 6-1. *Response modification factors (R).*

Structure	**R**
Wall-type pier	2
Reinforced concrete pile bents	
Vertical piles only	3
One or more battered piles	2
Steel or composite & steel concrete bridge pile bents	
Vertical piles only	5
One or more battered piles	3
Multiple-column bent	5
Connections	
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundation	1.0

phy makes use of load and resistance factors developed through statistical analyses (Kulicki 1999).

The AASHTO *LRFD Bridge Design Specifications*, first (1994) and second (1999) editions, requires that each bridge component and connection satisfy all limit states in accordance with the relation:

$$\eta \Sigma \gamma_i Q_i \leq \phi R_n \qquad (6\text{-}7)$$

in which η is a factor related to a ductility factor η_d, a redundancy factor η_R, and an operational importance factor η_i in accordance with $\eta = \eta_D \eta_R \eta_i$, γ_i is a statistically-based load factor applied to force effect Q_i, and ϕ is a statistically-based resistance factor applied to the nominal resistance R_n. The numerical values to be used for these factors can be found in the *LRFD Specifications* (AASHTO 1994, 1999).

6.2 Caltrans Seismic Specifications for Highway Bridges

6.2.1 Caltrans Modifications to Standard Specifications, 1971-1975

Immediately after the San Fernando earthquake of January 1971, Caltrans modified the numerical values of C used in Equation (6-1) by a factor of 2.5 based on damage observed in that earthquake. At the same time, Caltrans also revised the transverse reinforcing in bridge columns, from stirrups to continuous spiral reinforcing, and increased the minimum transverse bar reinforcement required. Bar spacing was revised from 12 inches to less than 6 inches. At the same, time Caltrans eliminated lap splices in column reinforcing, including the splice between the column steel and the footing steel, and discontinued the use of splices or couplers in plastic hinge regions on columns.

In 1973, Caltrans, working with the California Division of Mines and Geology, developed a statewide seismic hazard map, which provided bridge designers with site-specific peak rock acceleration data (A) (Figure 6-2). All known seismic faults were digitized and mapped. This was the first use of site-specific ground motions in seismic design of bridges. Figure 6-3 shows plots of spectral acceleration (ARS) as a function of period T for discrete values of A, which apply to the site condition 10-80' of alluvium. Research was conducted to develop foundation soil response spectra so the seismic hazard and soil-structure interaction could be more accurately predicted.

Starting with the new seismic design criteria in 1973, Caltrans required combining the two orthogonal seismic coefficients using the 30 percent rule as expressed by $EQ_x + 0.3EQ_y$ and $EQ_y + 0.3EQ_x$, with the larger of the two used for design.

6.2.2 Caltrans Adoption of ATC-6

In 1981, the Applied Technology Council (ATC) issued *ATC-6 Seismic Design Guidelines*

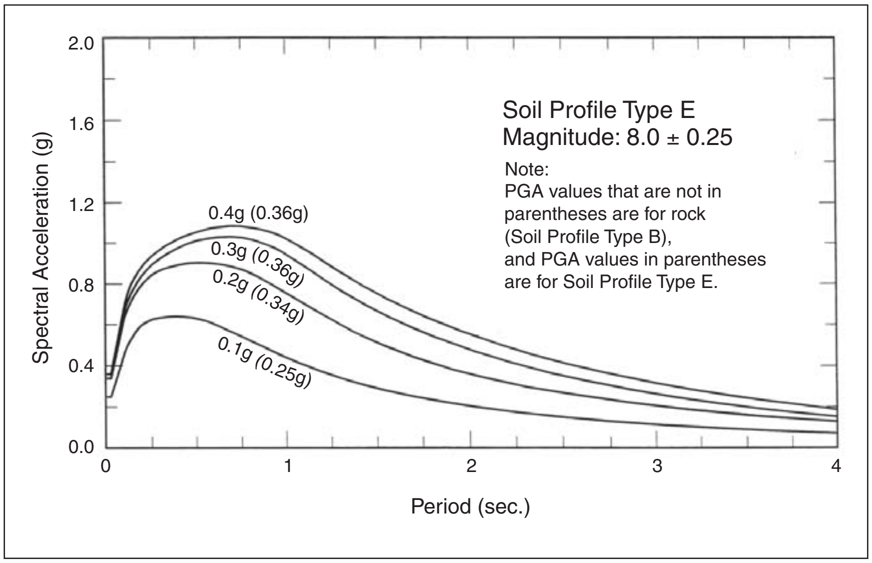


Figure 6-4. *Typical ARS plot used by Caltrans for deep mud.*

for Bridges under the sponsorship of the Federal Highway Administration, Department of Transportation. These guidelines were adopted by Caltrans immediately and formed the basis for the Caltrans Seismic Design specifications. Although the national guidelines provided four categories of seismic performance, based on the levels of acceleration coefficients, and four different analysis procedures, Caltrans adopted the site-specific procedure and only one analysis procedure for all bridges.

Since inelastic deformations are allowed in ductile bridge elements, the combined EQ_x and EQ_y elastic force components are then divided by appropriate response modification factors to obtain modified values of EQ. Caltrans used the modifications factors shown in Table 6-1.

6.2.3 Modifications Following the 1989 Loma Prieta Earthquake

Immediately following the Loma Prieta earthquake of October 1989, Caltrans engaged researchers to develop a series of acceleration response spectra for deep mud (Figure 6-4). An additional bridge classification of “Important” was added to the LFD specifications, which was intended to provide a higher performance level after an earthquake.

6.2.4 ATC-32, Improved Seismic Design Criteria for California Bridges

Shortly after the 1989 Loma Prieta earthquake, Caltrans contracted with the Applied Technology Council (ATC) to review the *Standard Specifications for Highway Bridges* and update them where necessary. The refinement of this effort resulted in ATC-32, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations*, was published in 1996 and formed the basis for the latest Caltrans Bridge Seismic Design Specifications.

6.3 Dual Level Design

The design of transportation structures to perform satisfactorily under expected seismic conditions requires that realistic earthquake loadings during their lifetimes be specified and that the structural components be designed to resist these and other combined loadings within the limits of certain expected performance requirements. In regions of high seismicity, earthquake loading is often critical among the types of loading that must be considered because a great earthquake will usually cause greater stresses and deformations in the various critical components of a structure than will all other loadings combined. Yet, the probability of such an earthquake occurring within the life of the structure is very low. On the other hand, a moderate earthquake is very likely to occur during the same period of time and also has the potential to produce damage unless controlled. Considering both types of earthquakes, a dual criterion strategy of two-level design is usually adopted for Important bridges as follows:

1. **Functional Evaluation Earthquake (FEE).** A functional evaluation earthquake is defined as one that has a relatively high probability of occurrence during the lifetime of a structure. The structure should be proportioned to resist the intensity of ground motion produced by this event without significant damage to the basic system, thus allowing it to remain functional immediately following the FEE event.
2. **Safety Evaluation Earthquake (SEE)**. A safety evaluation earthquake is defined as the most severe event that can reasonably be expected to ever occur at the site. Because this earthquake has a very low probability of occurrence during the life of a structure, significant structural damage is permitted; however, collapse and serious personal injury or loss of life should be avoided.

The challenge is to set seismic design criteria that will satisfy this dual criterion strategy in a cost-effective manner.

Important bridges located on major heavily traveled routes, where no convenient alternative routes exist, are now being designated as Important bridges on lifeline routes. These bridges are expected to remain functional immediately following an SEE event; therefore, they must be proportioned to resist the intensity of this event without experiencing significant damage. Because of this specified high level of performance during an SEE event, response under the FEE condition is of minor concern.

6.4 Displacement Control Versus Strength Control

Because of the philosophy of accepting nonlinear yielding in structural supporting elements, design control is now based on allowable displacement of columns rather than the strength of these columns.

6.5 Caltrans Seismic Design Criteria

Caltrans seismic design practice began to change soon after the 1989 Loma Prieta earthquake. Seismic design criteria utilizing displacement ductility design methods were developed in 1991 and early 1992, with oversight from external peer review panels, for the Terminal Separation Viaduct Replacement project in San Francisco, the Santa Monica Viaduct retrofit in southern California, and the I-880 Cypress Viaduct reconstruction project in Oakland. The project-specific seismic design criteria created for these projects served as a catalyst in the rapid evolution of Caltrans seismic design practice from force-based to state of the art displacement ductility-based analysis procedures.

While commonly practiced on individual projects, in September 1993, Caltrans published Memo-to-Designers 20-7, formally adopting displacement ductility analysis as an approved alternative method to seismically retrofit bridges. Interim memos and design guidelines were issued as new information was developed. In July 1999, *Caltrans Seismic*

Design Criteria (SDC) was published, documenting the extensive change in practice that had occurred since the 1989 Loma Prieta earthquake in the seismic design of new bridges.

The Caltrans SDC is based on the results of ATC-32, research findings, experience gained during the Seismic Retrofit Program, information exchanged with external peer review panels, input by the Seismic Advisory Board, and engineering judgment. This performance-based criteria utilizes displacement ductility principles to avoid brittle failures, capacity design principles to target inelastic response at predetermined locations while protecting the rest of the structure from damage, and places an emphasis on redundancy, member proportioning and balanced geometry to encourage predictable behavior of the structure. As new knowledge is gained and research results are assessed and implemented, the SDC continues to be updated. The current version of the SDC was published in December 2001, with another update pending at the time of publication of this document.

Section 7

Past Performance of Transportation Structures and Ensuing Changes in Caltrans Practices

7.1 Introduction and Background

It has been over 30 years since the 1971 San Fernando earthquake in California. This event was the genesis of modern bridge seismic design specifications and construction details in the United States. Thirteen years have passed since the Governor's Board of Inquiry into the cause of structural failures during the Loma Prieta earthquake of October 1989. The Board issued its final report with the warning title *Competing Against Time*.

The California Department of Transportation staff engineers, consulting firms, independent peer review teams, and university researchers have cooperated in an unprecedented, accelerated research-based program of bridge seismic design and retrofit strengthening to meet the challenge presented in that report.

Performance of highway bridges in the January 1994 Northridge earthquake provided reasonable assurance that those bridges designed or retrofitted to the post-1973 design criteria, and which have improved structural detailing, can withstand expected earthquakes without collapse or serious damage. The major causes of damage in the earlier earthquakes have been separation of deck expansion joints, causing deck systems to collapse, and the failure of older nonductile columns. Bridges constructed prior to the 1971 San Fernando earthquake have suffered the most damage in recent California earthquakes. Those bridges were designed for ground accelerations of 0.06g with no consideration for the spectral response of the structure, for the performance of the foundation material in a seismic event, or for structural ductility. Damage to bridges in Kobe (1995), Turkey (1999) and Taiwan (1999) followed similar patterns because the bridges were generally designed under older codes and had not been retrofitted to meet the latest codes. Bridges designed for the latest codes performed well and showed that modern design codes and details produce bridges that can withstand major earthquakes.

Since 1971, Caltrans and the California Geological Survey (CGS), formerly known as the California Division of Mines and Geology (CDMG), have developed a comprehensive seismic hazard map of the state that allows bridge designers to design for site-specific peak rock acceleration. Caltrans and CGS have conducted research to develop foundation-soil response spectra so that the seismic hazard and soil-structure interaction can be more accurately predicted. Based on the results of this research, Caltrans has adopted more stringent performance criteria to prevent collapse or serious damage in major earthquakes. Soil liquefaction effects have been researched and appropriate mitigation techniques have been developed and are currently being implemented. The required confinement details have been developed to ensure ductile performance in a seismic event, tested in half-size laboratory models for confirmation of ductile performance. These confinement details have been used in newer and retrofitted bridges, and field-tested in three moderate earthquakes (the 1992 Landers and Cape Mendocino earthquakes and the 1994 Northridge earthquake). At the national level,

Figure 7-1. *Spans slipped off narrow support seats in the 1971 San Fernando earthquake.*

the American Association of State Highway and Transportation Officials (AASHTO) has adopted seismic design specifications patterned after the West coast specifications. The excellent performance of bridges utilizing these newer design criteria and details gives bridge designers an indication that these structures can withstand a larger earthquake without collapse. Damage should be expected, but it can be repaired in many cases while traffic continues to use the bridge.

The California State Department of Transportation (Caltrans) owns and maintains over 12,000 bridges with spans over 20 feet (6.7 meters). There are an equal number in the city and county systems. Caltrans maintains the condition data for all these bridges and some 6,000 other highway structures such as culverts (spans under 20 feet/6.7 m), pumping plants, tunnels, tubes, Highway Patrol inspection facilities, maintenance stations, toll plazas and other transportation-related structures. Structural details and current condition data are maintained in the Department Bridge Maintenance files as part of the National Bridge Inventory System (NBIS) required by Congress and administered by the Federal Highway Administration (FHWA).

These data are updated annually to the FHWA and are the basis upon which some of the federal gas tax funds are allocated and returned to the states. The maintenance, rehabilitation, and replacement needs for bridges are prorated against the total national needs. Not until 1993 was seismic retrofitting accepted as an eligible item for use of federal funds because it was assumed by most other states to be only a California problem. After much lobbying by Caltrans, the Federal Intermodal Surface Transportation Efficiency Act (ISTEA) of 1992 provided for seismic retrofit to be eligible for federal bridge funds.

Prior to the 1933 Long Beach earthquake, there was no special consideration for seismic design of buildings or bridges in California. The severe damage to schools that resulted from that seismic event resulted in creation of the Structural Engineer license and a requirement for special consideration of seismic forces in the design of public schools in California. After the 1940 El Centro earthquake, the bridge design office of the California Division of Highways developed minimal seismic design factors for bridges. The 1940 El Centro record was digitized and used for buildings as the seismic design spectra for over 30 years before an earthquake of greater magnitude occurred in California. The 1971 San Fernando earthquake caused severe damage to hospitals, public utilities, and freeway bridges, recording a peak ground acceleration of 1.0g and large ground displacements. The 1971 earthquake caused both building and bridge designers to revise their design criteria and structural details to provide better resistance to the forces and displacements of major seismic events.

AASHTO has typically adopted seismic design criteria modeled after those in California, initially as guideline specifications only. Until recent years, most other states in the United States have not been concerned with seismic design for bridges. For example, the 1940 California seismic design criteria were not adopted by AASHTO until 1961, and the 1973 California seismic design criteria were

Figure 7-2. *Separation of thermal expansion joints.*

Figure 7-3. *Pullout of column reinforcing steel.*

not adopted nationally until publication of the AASHTO *Interim Specifications* in 1975.

The 1989 Loma Prieta and the 1994 Northridge earthquakes are the most significant in recent California history and produced the best information for bridge designers available. While experts consider the Loma Prieta and the Northridge earthquakes of only moderate magnitude, many bridges in the affected areas that had been retrofitted with pre-San Fernando seismic retrofit details performed well. This reasonable performance of older bridges in a moderate earthquake is significant for the rest of the United States because that knowledge can assist engineers in designing appropriate seismic retrofit programs.

Figure 7-4. *Single-column shear failure.*

7.2 Performance of Concrete Bridges

7.2.1 1971 San Fernando Earthquake

While we have learned something new from nearly every earthquake in California and other locations throughout the world, the major causes of bridge damage and collapse were made clear by the San Fernando event. These failure modes will be repeated again and again until bridges constructed prior to 1971 are seismically retrofitted to current seismic safety standards. It is important to compare bridge failures from the 1971 San Fernando event with those of the most recent events in Northridge, California (1994) and Kobe, Japan (1995). The major causes of bridge failures in 1971 were:

1. Collapse of superstructures due to support seats that were too narrow (Figure 7-1).
2. Separation of thermal expansion joints in bridge deck systems, resulting in the loss of support for suspended sections (Figure 7-2).
3. Loss of bond between column reinforcing steel and footing concrete, causing pullout and column collapse (Figure 7-3).
4. Horizontal shear failure of supporting columns due to insufficient lateral reinforcement (Figure 7-4).

Because there is no redundancy in a single-column supported bent, these failures were more critical than similar shear failures in multiple-column supports, and generally resulted in total column failure and collapse of the supported structure.

The opposite has been true for multiple column-supported structures, which have often sustained severe damage, but no structural collapse (Figure 7-5). The Seismic Advisory Board believes that this is due to the inherent redundancy in the framing systems of multiple column bents, even though they were not designed or reinforced to perform as ductile members.

Immediately after the February 9, 1971 San Fernando earthquake, Caltrans began a comprehensive upgrading of the *Bridge Seismic Design Specifications and Seismic Construction Details*. Caltrans modified the specifications to correct the identified deficiencies so that new bridge designs would incorporate them. After Caltrans completed this work, the Applied Technology Council (ATC) took the process a step further with ATC-6, *Seismic Design Guidelines for Highway Bridges*, which became the basis for a similar bridge seismic retrofit design specification that was adopted in 1992 by the American Association of State Highway and Transportation Officials (AASHTO) as the U.S. standard.

Shortly after the 1971 San Fernando earthquake, Caltrans adopted a site-specific seismic design philosophy. The California Geological Survey (CGS) was engaged for the development of an earthquake ground fault map. The maximum credible events on seismic faults throughout the state define peak bedrock acceleration levels, shown on CGS Map Sheet 45 (Figure 6-2). This map shows the 275 known faults in California and includes contours of various levels of expected peak rock acceleration determined from average attenuation relationships developed by various seismologists. This approach has been criticized as too conservative, but

Figure 7-5. *Multiple column shear failure.*

Caltrans cost analyses show that the additional cost for an average bridge is minimal compared to the cost of a design for a lower probability event.

The Seismic Advisory Board believes that the maximum credible approach is reasonable for Standard bridges. Since the 1989 Loma Prieta earthquake, Caltrans has used a site-specific hazard analysis to determine the most probable design earthquake for major structures. This most probable event is then incorporated into the seismic design procedure along with the maximum credible event for Important bridges.

7.2.2 Whittier Narrows Earthquake, 1987

Hinge joint restrainers performed well during the moderate 6.0 Whittier Narrows earthquake of October 1, 1987. However, the I-605/I-5 separation bridge in Los Angeles sustained shear failure, and reemphasized the inadequacies of pre-1971 column designs (Figure 7-6). Even though there was no collapse, the extensive damage resulted in Caltrans accelerating basic research into practical methods of retrofitting bridge columns on the existing pre-1971 nonductile bridges. That research program began in August 1986

Figure 7-6. Column shear failure in the 1987 Whittier Narrows earthquake.

at the University of California at San Diego, and the Whittier Narrows earthquake speeded its approval and execution. The research is ongoing today at the University of California at San Diego, the University of California at Berkeley, and other university and private research facilities. The Loma Prieta earthquake two years later provided more impetus to the program.

The Whittier Narrows earthquake proved the Caltrans post-1973 *Bridge Seismic Design Specifications and Seismic Construction Details* to be adequate, and relatively new structures performed well in the event. Older existing bridge structures, however, proved to be a substantially more challenging problem.

Research was undertaken in the United States, New Zealand, and Japan to improve analytical techniques, and to develop basic data on the strengths and deformation characteristics of lateral load resisting systems for bridges. The National Science Foundation began supporting bridge seismic research in 1971 and Caltrans supported research on selected issues as well. The latter focused on cable restrainers and related issues needed to implement the initial efforts at bridge vulnerability assessment and retrofit. In 1986, just before the Whittier Narrows earthquake, Caltrans identified the vulnerable elements of existing bridges and began a statewide Highway Bridge Seismic Retrofit Program to systematically reinforce the older, nonductile bridges. The first focus was on superstructure retrofit and involved installation of hinge joint restrainers to prevent deck joints from separating. This was the major cause of bridge collapse during the San Fernando earthquake and was judged by Caltrans engineers and other investigators to be the highest risk to the traveling public. Included in this initial phase was the installation of devices to fasten the superstructure elements to the substructure in order to prevent those superstructure elements from falling off their supports (Figure 7-7). This phase was essentially completed in 1989, after approximately 1,260 bridges on the state highway system had been retrofitted at a cost of over $55 million.

7.2.3 Loma Prieta Earthquake, 1989

The Loma Prieta earthquake of October 17, 1989 again proved the reliability of hinge joint restrainers, but the tragic loss of life at the Cypress Street Viaduct on I-880 in Oakland emphasized the necessity to immediately accelerate the Column Retrofit Phase of the Caltrans Bridge Seismic Retrofit Program with a higher funding level for both research and implementation.

Other bridge structures in the earthquake-affected counties performed well, suffering the expected column damage without collapse. With the exception of a single outrigger column-cap joint confinement detail, those bridges using the Caltrans post-1973

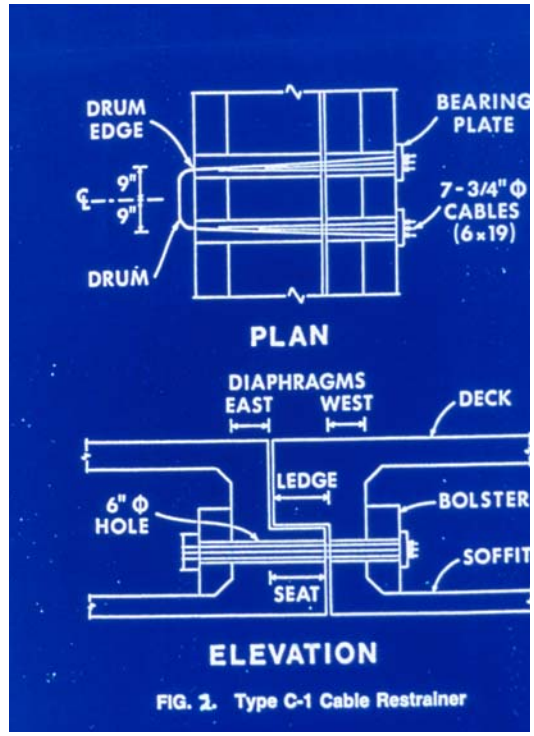


Figure 7-7. *Early hinge restrainer detail.*

design specifications and confinement details performed well.

Damage to long, multiple-level bridges, such as the Cypress Street Viaduct, showed the need to more carefully consider longitudinal resisting systems because earthquake forces cannot be carried into abutments and approach embankments as they can on shorter bridges. After the Loma Prieta earthquake caused 44 fatalities on the state highway system, capital funding for seismic retrofitting was increased to $300 million per year. At the same time, bridge seismic research funding was increased from $0.5 million annually to $5.0 million annually, with an initial $8.0 million allocation from the special State Emergency Earthquake Recovery legislation of November 1989. Using the special research funding provided, Caltrans engaged additional research teams and facilities to assist in this massive program.

Both U.S. and foreign researchers have conducted much research into the causes of bridge damage in the Loma Prieta earthquake. Most of these research papers can be obtained from the National Information Service for Earthquake Engineering, Earthquake Engineering Research Center (EERC) of the University of California at Berkeley. The EERC, located at the Richmond Field Station of the University of California at Berkeley, has been designated as the national repository for information on the 1989 Loma Prieta earthquake. There are over 175 documents on file at the repository relating to bridge aspects of the Loma Prieta earthquake. Additional research papers and project reports can be obtained from Caltrans Division of Structures; the Department of Applied Mechanics and Engineering Science, University of California at San Diego; and the Multidisciplinary Center for Earthquake Engineering Research (MCEER), State University of New York at Buffalo (SUNY). MCEER has a database search service known as QUAKLINE.

7.2.4 Northridge Earthquake, 1994

The Northridge earthquake of January 17, 1994 reinforced the prior actions of Caltrans for practice improvement, research, and retrofit of bridges. Efforts were increased—in some cases significantly. Most notably, the seismic evaluation of the toll bridges was completed.

7.3 Prior Research Results

Much had been learned about bridge performance in previous earthquakes (1971 San Fernando and 1987 Whittier Narrows), and only budgetary constraints prevented Caltrans from executing seismic retrofit of older

Figure 7-8. *Good performance of new hinge restrainer detail.*

bridges at a more rapid pace. It is important, however, to observe and discuss the performance of the new seismic design criteria that was used on bridges designed after 1972, and those seismic retrofit devices that had been installed prior to Loma Prieta.

7.3.1 Hinge Joint Restraining Devices

The initial phase of the Caltrans Highway Bridge Seismic Retrofit Program involved installation of hinge joint restrainers to prevent deck joints from separating (Figure 7-7). Research and testing of the restrainers was conducted at the University of California at Los Angeles. These joint restrainer systems have performed well (Figure 7-8) in subsequent earthquakes, including the Whittier Narrows (1987), Loma Prieta (1989), Cape Mendocino (1992), and the three southern California events of 1992.

In the eight counties declared disaster areas after the Loma Prieta earthquake, there were approximately 350 bridges that had already been retrofitted with hinge joint restrainers. There was no observed failure of any of these restrainers. It is agreed by Caltrans staff engineers that bridge spans would have fallen off their supports without the installation of these restrainers. The University of Nevada at Reno was awarded a Caltrans research project to test the performance of bridge hinge joint cable restrainers under dynamic loading. Saiidi and Maragakis (1995) of the University of Nevada, Reno, and Yashinsky (1994) of the Caltrans Office of Earthquake Engineering have published papers evaluating the performance of these restrainer details.

7.3.2 Properly Confined Column Reinforcement

Most columns designed by Caltrans since 1971 contain a slight decrease in the main column vertical reinforcing steel and a major increase in confinement and shear reinforcing steel over pre-1971 designs.

All new columns, regardless of geometric shape, are reinforced with one or a series of spiral-wound interlocking circular cages. While hoop size and spacing vary, the typical transverse reinforcement detail consists of #6 (3/4 inch/18 mm diameter) hoops or continuous spiral at approximately 3-inch (75 mm) pitch over the full column height. This provides approximately eight times the confinement and shear reinforcing steel in columns than was used in the pre-1971 nonductile designs for highway bridges. After modification of the *Bridge Seismic Design Specifications and Seismic Construction Details* in 1973, all main column reinforcing was continuous into the footings and superstructure. Splices are now mostly welded or mechanical, both in main and transverse reinforcing. Transverse reinforcing steel is designed to produce a ductile column by providing shear capacity in excess of the flexural capacity and by confining the plastic hinge areas at the top and bottom of columns (Figure 7-9). The use of grade 60A 706 reinforcing, a more ductile steel, in bridges has been specified on all projects since the early 1990s. In the eight counties declared a disaster area after Loma Prieta, there are approximately 800 bridges designed after 1972 and which were designed using the newly developed *Bridge Seismic Design Specifications and Seismic Construction Details*. With

Figure 7-9. *New column reinforcing.*

Figure 7-10. *Good performance of new column retrofit detail.*

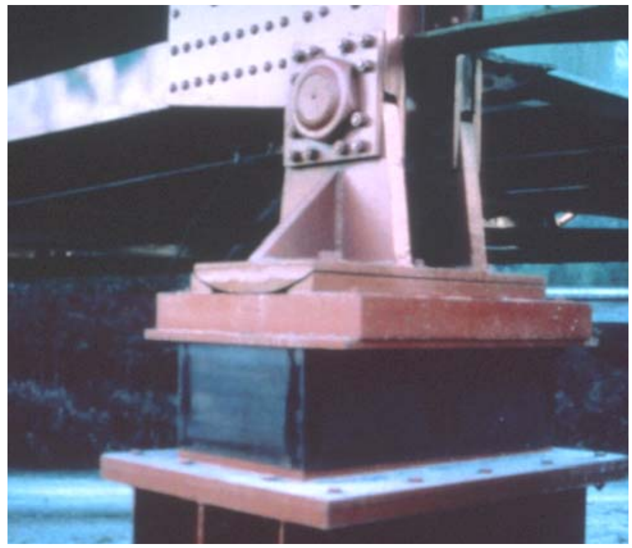


Figure 7-11. *Typical rubber base isolation bearing.*

the exception of damage to one outrigger beam-column joint on the I-980 southbound connector in Oakland, there was no documented damage to any of these 800 bridges designed to post-1973 design specifications (Figure 7-10).

7.3.3 ARS for Alluvium and Dense Foundation Materials

Caltrans developed a series of acceleration response spectra (ARS) for alluvium and average harder soils after the 1971 San Fernando earthquake. These spectra were representative for prediction of the dynamic response of those types of foundation materials. Professor Harry Bolton Seed, of the University of California at Berkeley, was instrumental in the development of these design spectra. Those bridges situated on average foundation materials and designed using these spectra and the post-1973 *Bridge Seismic Design Specifications and Seismic Construction Details* performed well in the 1989 Loma Prieta event.

7.3.4 Base Isolated Girder Systems

Although only one bridge in the eight affected counties was base-isolated, it did perform well during the Loma Prieta event. The bridge was the Sierra Point Overhead in San Francisco, which was designed prior to 1972 for lateral force requirements of only 0.06g. It was subjected to horizontal peak ground motions of approximately 0.18g during the Loma Prieta earthquake and showed no signs of distress. It should be noted, however, that the Caltrans design procedure is to force seismic loads into the abutments, so the back wall must fail prior to the base isolation bearings being totally engaged (Figure 7-11).

Figure 7-12. Cypress Street Viaduct collapse in Oakland during the 1989 Loma Prieta earthquake.

7.4 Problems Associated With Existing Bridge Criteria, Details, and Practice

A discussion of the problems encountered in highway bridge performance during the Loma Prieta and Northridge earthquakes will illustrate the need for continual research in the area of structural response in moderate and major earthquakes.

7.4.1 Older Bridges Designed for Pre-1972 Seismic Forces and Design Criteria

The major causes of bridge damage in the Loma Prieta earthquake were the criteria and details for which they were originally designed. There were over 4,000 bridges in the combined state, county and city systems in the eight counties that were declared a disaster area after the earthquake. Only 100 of those bridges were damaged in the earthquake and only 25 sustained what can be termed major damage, as reported in the Post-Earthquake Investigation Team (PEQIT) Report. Only one of the 800 bridges in the counties that had been designed after 1972, and which used the newer seismic forces and details, suffered damage. While Loma Prieta was, admittedly, a moderate earthquake, the bridge performance was generally what bridge designers expected. Most of the research that has been commissioned since Loma Prieta is aimed at developing better assurance that bridges will withstand a major earthquake without collapse or major damage, and ensure that Important transportation structures can remain essentially functional after a major seismic event.

7.4.2 Seismic Performance Criteria Required

The Governor's Board of Inquiry hearings brought out the fact that there was no formal documented policy on the required seismic performance of bridges in the Caltrans *Standard Specifications for Highway Bridges*. These specifications are utilized by many other public agencies and other states; therefore, it was critical that formal performance criteria be developed. Performance criteria were adopted in 1993.

7.4.3 Dynamic Response of Deep Soft Foundations

The effect of the dynamic response of deep soft soils in the structure foundations also proved to be a contributing factor to the collapse of the Cypress Street Viaduct (Figure 7-12) and must be analyzed and included in future Caltrans design procedures. This is especially critical for structures with relatively long periods of vibration, for instance long, tall, or flexible bridges. The effect of incoherence in the foundation response is an important factor in the design of very long structures such as the San Francisco-Oakland Bay Bridge and mile-long freeway viaducts. Abrahamson developed the most widely used spatial coherence functions for ground motions, and Hao, Oliviera and Penzien (1989) the most widely used approach to developing spatial ground motion time histories. Caltrans has developed a comprehensive set of acceleration response spectra for various depths of soft mud. These

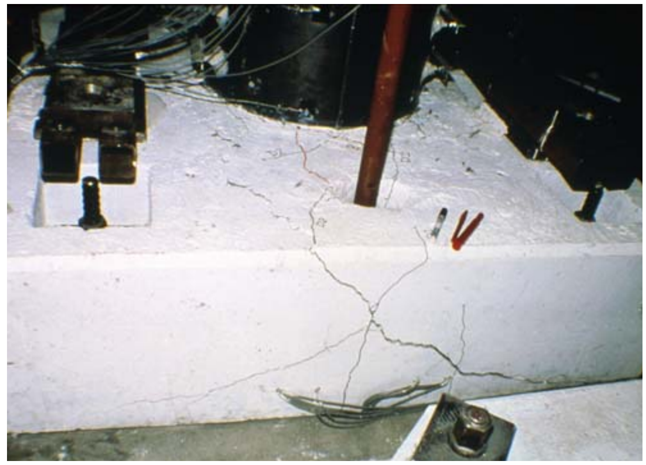


Figure 7-13. *Damage pattern for column-footing model with no top mat in testing lab.*

Figure 7-14. *Left. Additional reinforcing being added to existing column and footing.*

Figure 7-15. *Right. Column damage on Embarcadero Freeway Viaduct, San Francisco.*

spectra have been in use since the Loma Prieta earthquake.

7.4.4 Column-Footing Interaction

Investigation of damage at the Cypress Street Viaduct in Oakland subsequent to the Loma Prieta event revealed a deficiency in many pre-1973 designed bridge footings. Some of these footings suffered joint shear failures, which caused structural settlement. These footings were typically designed for vertical loads and only a 0.06g lateral force. Investigation and research by Chai, Priestley and Seible (1991) of the University of California, San Diego revealed a potential for failures due to lack of reinforcing steel in the top of the footing to resist lateral moments. Their analysis and laboratory tests did show a need for a top mat (Figure 7-13) of reinforcing steel in footings, which substantiated retrofit details implemented in designs prior to the Loma Prieta event (Figure 7-14).

7.4.5 Inadequate Column Confinement Reinforcement

Other than the Cypress Street Viaduct failure, column damage was limited to a few critical bents on the Embarcadero Freeway Viaduct (Figure 7-15) , the Terminal Separation Structure, the Central Freeway Viaduct, and the Southern Freeway Viaduct (Interstate 280). All these structures were in San Francisco and all were multi-level box girder decked structures supported on framed bents. Generally, those damaged bents were located in areas over deep soft bay mud.

The damage on the Central Freeway Viaduct was located in a few bents on the northern end between Oak and Turk Streets. This was the only damage to portions of a structure that was not constructed over deep soft soils. These structures were closed almost immediately, with the exception of that portion of the Central Freeway Viaduct south of Oak Street where there was no sign of damage. Temporary splice beams were installed on those columns of the Central

Figure 7-16. *Severe column damage to the Southern Freeway Viaduct, San Francisco, in the Loma Prieta earthquake.*

Figure 7-17. *Close-up of column damage in Figure 7-16.*

Freeway Viaduct where column hinge joints had been located in the original design. This splice was intended to keep the joint from separating in a future seismic event until a more permanent retrofit detail with new columns could be installed.

The most spectacular bridge damage, and that which was closest to collapse, occurred in the vicinity of Innis Street on the Southern Freeway Viaduct, Interstate 280, a pre-1971 design. The shorter of two columns supporting long outrigger bents failed in joint shear near the lower deck level (Figures 7-16 and 7-17). This freeway was damaged at only four bent locations, however. While the damage was minimal, there was obvious concern for the integrity of this pre-1972 design, non-ductile, reinforced concrete structure.

All these viaducts had been designed in the late 1950s to early 1960s for lateral forces of 0.06g, and used standard details of the period, which we now know were weak and provided insufficient confinement, especially at beam-column joints. All the damaged areas were shored up with heavy timber falsework to reinforce them during aftershocks and possible future seismic events until permanent repairs could be made. Since both the duration and magnitude of the Loma Prieta earthquake were high, the Seismic Advisory Board believes it was prudent to close the structures to public traffic until they could be retrofitted to current seismic standards.

Figure 7-18. *Collapsed end spans over Interstate 5 in the 1994 Northridge earthquake.*

Figure 7-19. *Reconstructed I-5 interchange.*

7.4.6 State Route 14/Interstate 5 Connectors

Two miles (3km) south of Gavin Canyon, approximately 8.2 miles (12km) from the epicenter of the Northridge earthquake, the major interchange at State Route 14 and Interstate Route 5 suffered the most significant and expensive damage. While the news media reported only the collapsed end spans of two connector ramps, there was significant internal hinge damage to other connector ramp structures, including the collapsed end spans on the southbound Route 14 connector to southbound Interstate 5 (Figure 7-18). The three end spans collapsed onto both the southbound and northbound lanes of Interstate 5.

All the bridges in this interchange are long, curved connector structures with several frames and intermediate thermal expansion deck hinges. During the Northridge earthquake, a peak ground acceleration of at least 0.70g was estimated for this location. Measurement of aftershocks at this site indicates a large variation in ground motion. There is evidence of relative movement between the longer columns and the short end columns. It is apparent from analysis that the short end columns at both these collapsed spans caused the failure because they were unable to displace elastically with the remainder of the structure, which was supported on much longer columns.

It is important that bridge designers analyze these failures, determine the causes of collapse, develop newer and better details, and fully implement those details in all new designs and in retrofitting older, nonductile structures. Many of the details have been developed and tested, and are now in use in the California bridge seismic retrofitting program.

These connector bridges were under construction during the 1971 San Fernando earthquake, during which only one column failed. That column, 180 feet (59 m) high, was reconstructed with multiple spiral-reinforced column cages, and it survived the Northridge earthquake with no damage. However, the designers were concerned about the foundation supports for the ramps due to the apparent ground movement and the fact that this was the second recent earthquake to cause ground disturbance in this canyon. The cost estimate for the already-designed seismic retrofit of this interchange was close to replacement cost, so it was decided to remove and replace all but one of the ramps.

The reconstructed I-5 interchange has two significant differences from the original structures (Figure 7-19). There is only one

__Figure 7-20.__ Special hinge for large displacements.

__Figure 7-21.__ Isolation casing for short columns.

deck hinge joint in each ramp (versus 3 or 4 on the original structures), and the short end columns are constructed in steel shell isolation casings. These casings are between 20 feet (6.5 m) and about 48 feet (16 m) deep, providing for an increased elastic column length. On the shortest columns adjacent to the abutment, the elastic length is doubled by the use of the casings. This detail results in all the columns having similar stiffness and sharing seismic demands nearly equally.

One connector ramp is 1,532 feet (510 m) long with columns ranging from 25 to 75 feet (8 to 24 m) in length, so it was not possible to eliminate all the deck hinge and expansion joints. However, a new detail was developed for these hinges (Figure 7-20). The hinge joint is centered between two columns approximately 40 feet (13m) apart, and the two deck elements cantilever from the adjacent bents. Since neither side supports the other, there is no risk of a deck collapse, even in a major seismic event with large ground movement. The joint may separate, but a steel plate can be placed over that joint and the bridge can remain in service during repair. This detail has been utilized extensively in this interchange on three of the longest connector ramps

The South Connector carries southbound Interstate 5 traffic to northbound Route 14 and has the highest column, at 180 feet (59m) long. In addition to the special hinge detail, this ramp has several shorter columns built with the isolation casing. Those columns to the left of the hinge in Figure 7-20 are short and those to the right are increasingly longer as the structure crosses over the deepest part of the canyon, a Southern Pacific mainline railroad, three of the other connector ramp structures, and several freeway lanes. Figure 7-21 shows the detail of the isolation casing.

7.4.7 Boundary Conditions, Architectural Details, Column Stiffness

The remainder of the damaged structures in the Northridge earthquake were older, conventional reinforced box girders on the Santa Monica Freeway and post-tensioned cast-in-place prestressed box girders on State Route 118. These were all replaced with modern post-tensioned cast-in-place box girders without deck expansion joints or hinges, and with improved seismic details. Caltrans completed the reconstruction in record time with the use of accelerated contracts and incentive clauses for early completion.

Figure 7-22. *Changed column stiffness caused by post-construction change in boundary conditions.*

Figure 7-23. *Excellent column performance on SR-118 in the 1989 Northridge earthquake.*

Change in Boundary Conditions
The design details that caused damage and failures need to be analyzed, however. Figure 7-22 shows a column failure that was caused by a change in boundary conditions after the bridge was constructed. A concrete channel lining was installed close to a row of columns, forming a bond beam. The point of fixity on the column shifted from the top of footing, as assumed in the design, to the top of the channel lining. The shorter elastic length caused the column to be stiffer than assumed, and it therefore attracted excessive lateral load and failed.

Analysis techniques available today provide the bridge designer the ability to analyze the various boundary conditions, which would be known at time of design, and those that could be anticipated by future modifications, and detail the column for those conditions.

Good Column Performance
On the positive side, there were instances of acceptable column performance in a seismic event. The bridge depicted in Figure 7-23 is on State Route 118 in Northridge within two miles (3 km) of the epicenter of the 1994 Northridge earthquake. Ground accelerations were in excess of 0.6g at this location. This kind of column damage poses no immediate threat to safety or performance of the bridge structure. The well-confined column core has remained intact and can sustain the dead and live loads as well as more severe lateral loads from aftershocks. The only repair necessary is cosmetic to replace the surface concrete. It is advisable to design so the hinge area can be easily inspected after a seismic event.

Architectural Details
Figure 7-24 shows an architectural feature of a flared column capital typical of those used in California for aesthetic enhancement of the bridge prior to the 1994 Northridge earthquake. Current details place a physical 2" to 4" gap at the top of the column, isolating it from the bent cap. The flared sections are typically designed to be lightly reinforced and assumed to break off in an earthquake so that only the central core of the column will carry the seismic forces. However, on this bridge, the designer designed the flares to be a part of the structural column. In the crude

Figure 7-24. *Expected performance of column architectural features.*

Figure 7-25. *Actual performance of flared column.*

computer modeling available in 1973, the nonprismatic section was not modeled accurately, and plastic shear demands were not investigated. This resulted in the column elastic length being shorter than assumed, causing it to take more force than it was designed to handle. In the design, the total column length was assumed for the elastic length; but in the earthquake, the stiff flare forced the plastic hinge to form just below the flare (Figure 7-25), essentially shortening the column elastic length to one-half of that assumed in the design analysis. This is another detail that can be modeled more accurately with the computer technology available today.

Many engineers feel that these architectural details should not be used on columns. That is not a progressive attitude; the same detail at the top of a longer column, such as those in interchanges or in deep ravines, will not appreciably affect the elastic length or ductile performance of the column. Even on the column shown in Figure 7-24, the design details for the flared portion have now been improved so the flare will not act as a structural element. Much smaller reinforcing steel bars are used now, and a cold joint at the top of the flared section introduces a failure plane to isolate the flare from the central core. Only the confined core inside the spiral reinforcing acts as a structural element. Better analysis and appropriate detailing can allow the use of these architectural features and still result in satisfactory structural performance in an earthquake.

7.5 Alaska Earthquake, Denali Fault, 2002

On November 3, 2002, a magnitude 7.9 earthquake ruptured the Denali fault in the rugged Alaska Range, about 90 miles south of Fairbanks. Called the Denali Fault earthquake, this shock was the strongest ever recorded in the interior of Alaska. Although comparable in size and type to the earthquake that devastated San Francisco in 1906, it caused no deaths because it struck a sparsely populated region of south-central Alaska.

The Denali Fault earthquake ruptured the earth's surface for 209 miles, crossing beneath the vital Trans-Alaska Oil Pipeline, which carries 17 percent of the U.S. domestic oil supply. Although slightly damaged by movement on the fault and by intense shaking, the pipeline did not break, averting a major economic and environmental disaster. This success is a major earthquake engineering achievement.

Approximately 300 of Alaska's 1,000 highway bridges are located in the area of strong ground shaking—that is, where the ground acceleration exceeded 0.10g. The longer spans are through-trusses; the shorter ones are composed of steel or precast-concrete girders. These bridges were built from the 1940s to the 1990s in accordance with bridge design specifications set forth at that time by the American Association of State Highway and Transportation Officials (AASHTO). Although many bridges were damaged and several will have to be replaced, all bridges continued to carry traffic after the earthquake. A few required load restrictions.

One of the damaged bridges was the Tanana River Bridge on the Alaska Highway. Built in 1944, it is a 946 foot-long (288 m), three-span cantilever truss bridge on pier walls and seat-type abutments. The superstructure is supported by two movable bearings at the abutments and by two fixed bearings at each pier. The bridge is nearly parallel to the Denali Fault. During the earthquake, the superstructure moved transversely, breaking the steel pin bearings at the west pier.

Four major railroad bridges and many smaller bridges exist in the part of the track closest to the fault. The major structures include two through-truss bridges with spans of 700 feet (213 m) and 504 feet (154 m), a deck arch with a 334-foot (102 m) long main span, and a deck girder structure that spans the Riley Creek fault at the entrance to Denali National Park. None of these bridges suffered any damage during the earthquake.

Of particular importance was the performance of the Trans-Alaska Oil Pipeline, which was constructed in the 1970s to transport crude oil from the North Slope of Alaska to the ice-free port of Valdez. The 48-inch-diameter (1,219.2 mm) pipeline was built at a cost of approximately $8 billion by the Anchorage-based Alyeska Pipeline Service Company, which still operates the facility. The pipeline passes through permafrost along approximately 300 mi (483 km) of its 798 mi (1,284 km) length.

During the 2002 Denali Fault earthquake, the ground was offset beneath the pipeline, and violent shaking damaged a few of the pipeline's supports near the fault, but the pipeline did not break (Figure 7-26). Design studies located the Denali Fault within a 1,900-foot corridor crossing the pipeline route and estimated that the pipeline could be subjected to a magnitude 8.0 earthquake, in which the ground might slip 20 feet horizontally and 5 feet vertically. These estimates proved to be remarkably accurate for the 2002 Denali Fault magnitude 7.9 earthquake, in which the rupture crossed the pipeline within the 1,900-foot corridor, and the fault shifted about 14 feet horizontally and 2.5 feet vertically. To accommodate the projected fault movement and intense earthquake shaking from a magnitude 8.0 quake, the zigzagging Trans-Alaska Oil Pipeline, where it crosses the Denali Fault, is supported on Teflon shoes that are free to slide on long horizontal steel beams.

During the shutdown, crews inspected the structure and initiated repairs. The pipeline had moved laterally, axially, and vertically during the earthquake. Five sections were found to be damaged, although none fell or were so severely damaged that they had to be replaced. In eight locations, the shoes that connect the pipeline to the horizontal cross members had fallen to the ground, leaving the pipeline unsupported for one or two spans. The pipe, however, was able to span the unsupported length elastically.

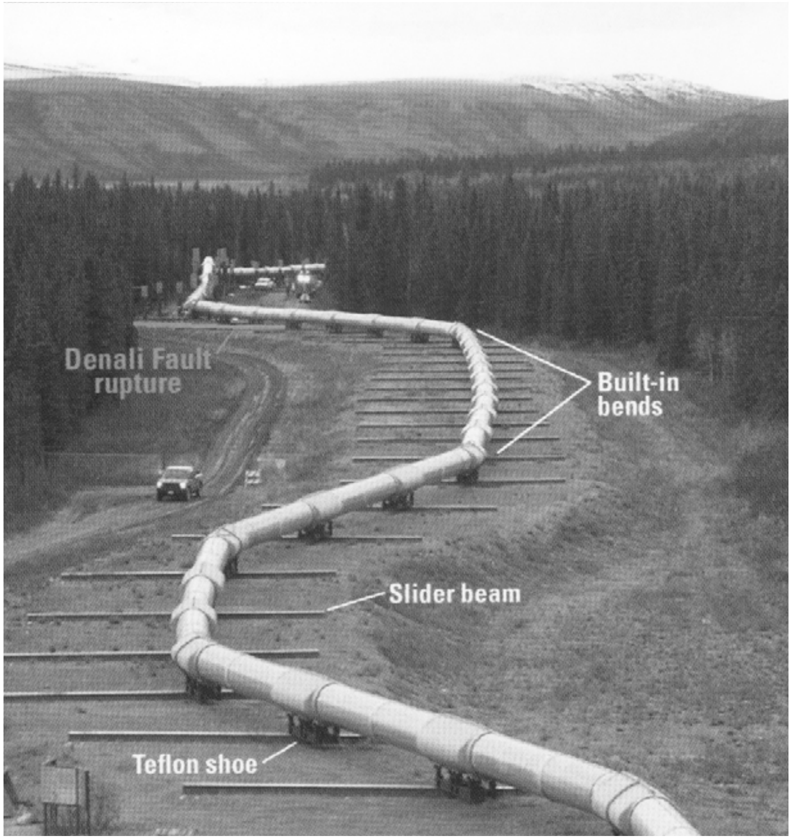


Figure 7-26. Alaska pipeline where it crosses the Denali fault.

The Denali Fault earthquake caused approximately $90 million in direct damage and losses to lifelines. Damage to buried lifelines was generally nonexistent or slight, largely because they were located some distance away from the earthquake. Bridge damage in the highest seismic intensity region was limited and mostly associated with embankments. Still, it is unusual to see single-span bridges damaged so severely that they need to be replaced. Such damage can be avoided in future earthquakes by designing for the appropriate seismic forces.

7.6 Performance of Steel Bridges

Major damage to steel bridges occurred in the 1989 Loma Prieta earthquake. Most of this damage was caused by bearing failures and drop-off of girders due to narrow bearing seats. The repair and retrofit strengthening of steel bridges is covered in Sections 7 and 8 of this report.

Prior to the Loma Prieta earthquake, most of the damage to steel bridges had been restricted to substructure failures. Span drop-off caused by narrow bearing seats occurred during the San Fernando earthquake in 1971. Very little damage to steel superstructures was observed.

During the 1994 Northridge earthquake, there was minor damage to bearings and end diaphragms on several steel bridges. None of these bridges had to be closed while repairs were accomplished.

7.7 Summary and Conclusions

It is clear from the performance of concrete bridges in major earthquakes over the past 30 years that there are two important lessons. First, bridges that have been designed using the latest seismic codes and construction details perform well, usually exhibiting the predicted minor damage. Second, it is very clear that until existing bridges are strengthened, particularly older ones, to current design specifications, earthquakes will continue to cause bridge failures. While future failures cannot be precluded, since there is much we do not yet know, better design will significantly reduce the likelihood of bridge failures.

Over the past 30 years, the structural engineering community, working with seismologists, geotechnical engineers, and other experts in the earthquake engineering field, has learned much from each major earthquake. From these observations, Caltrans has developed vastly improved seismic design specifications for bridges and new details have been developed and tested to better ensure ductile performance in a seismic event.

What remains to be accomplished is a continuing evaluation of older bridges to determine their seismic vulnerability, and a concerted effort to retrofit these bridges where needed.

The Seismic Advisory Board believes there are several important lessons to be derived from evaluation of the response of bridges:

1. It is extremely difficult to design bridges to survive large fault displacements, especially displacements greater than 1 meter. The details of bridges damaged in Turkey and Taiwan must be studied and structural details improved. Fault crossings should be avoided when possible, but it is not always possible. The Seismic Advisory Board believes that we have a good chance of success in handling large movements, up to 3 meters, with careful detailing.
2. Structures constructed within 5 km of the fault line must be designed for the so-called velocity impulse, which has the potential to produce a larger displacement of the structure than the acceleration effects. This is commonly called the fling, or directivity, effect and which can often produce larger than normal vertical effects, even for slip faults (Section 5.1.2).
3. Structures constructed over deep soft mud must be given careful attention due to the lack of lateral resistance in these muds in an earthquake. The net effect is generally an enhancement of the rock accelerations rather than the energy dissipation provided by harder soils and alluvium. However, the soft soil will not always enhance the rock acceleration excessively, especially if the rock has low shear wave characteristics. In those cases, the foundations (piles) perform like column extensions. Displacements and p-delta effects can become structurally critical if they are not sized correctly.
4. The performance of base isolation systems and energy dissipators must be carefully evaluated to have sufficient capacity before they can be depended on to perform satisfactorily during a seismic event. Furthermore, structural members joined by these devices should be tuned for optimum performance.
5. From the amount of damage to shear keys, there needs to be much more conservatism in the design of these important elements. They are a miniscule cost item in a bridge, but they play an important role during an earthquake. Many bridges have been protected from major damage or collapse by shear keys. Better design of these connection details would provide damage protection and yield better performance. Such design requires careful attention to displacement.
6. Vertical accelerations near faults, especially subduction faults, are significant, as shown in recent seismic events. Bent cap and superstructure details need to recognize vertical demands.

7. Bridge performance following an earthquake is a very subjective measure of performance. It ranges from repairable damage to immediate full use. The cost premium to provide post-earthquake serviceability can be staggering. Also, building stronger members to prevent premature yielding can be dangerous in the sense that the structure then borders on shear failure that can be catastrophic.
8. Shear keys are designed to limit displacement in moderate earthquakes, but to fuse (fail) when large earthquake displacements occur in order to protect other members. When shear keys fail, they are performing their intended purpose.

Section 8

Caltrans Problem-Focused Seismic Investigation Program

Following the 1989 Loma Prieta earthquake, the Governor's Executive Order D-86-90 of 1990 required the Director of Caltrans to assign high priority to a program of problem-focused investigations that require resolution for effective engineering and assessment of Caltrans structures. The term "research" is used below for such problem-focused investigations with the understanding that this research effort is goal-oriented and determined based upon both the nature of the problems to be resolved and the expected applicability of the results. The Governor's Executive Order called for a comprehensive earthquake vulnerability evaluation of transportation structures and a program of earthquake monitoring of bridge structures with appropriate seismic strong motion instruments.

There is no doubt that the Caltrans seismic research program has contributed significantly to improvements in the safety of California's highway structures. Some of the major retrofit research accomplishments are:

- Column retrofit program at U.C. San Diego, which led directly to the methods used to retrofit columns following the 1987 Whittier Narrows and 1989 Loma Prieta earthquakes.
- Joint retrofit program at U.C. Berkeley, which also led to procedures used after the Loma Prieta earthquake.
- Restrainer retrofit program at UCLA, which developed procedures used following the 1971 San Fernando earthquake.
- Pier wall program at U.C. Irvine.
- Abutment research at U.C. Davis, which rationalized design procedures for bridge systems.

The Caltrans Seismic Advisory Board (SAB) has been strongly in favor of these comprehensive studies supported by Caltrans, which have been directed toward improving the seismic response of California's transportation system. The SAB has also pointed to limitations of the Caltrans seismic research plan, such as overlapping and uncoordinated procedures and policies.

There are at least two major parts to the current Caltrans research program. One part is under the general direction of the Caltrans Division of Engineering Services (DES). From an initial investment of $500,000 per year, this critical support for focused seismic research has continued at an annual expenditure of $5 million since the 1989 Loma Prieta earthquake. The research associated with Caltrans DES involves both in-house problem-oriented work and external contracts. It spans narrowly based, project-derived studies to broad generic problems on earthquake hazards and seismic performance. Over the past two decades, results of the Caltrans Bridge Seismic Research Program have provided California with state-of-the-art bridge design and retrofit technology. It is the position of the SAB that this Caltrans leadership role in seismic bridge research needs to be maintained to minimize losses from future California earthquakes.

The Caltrans Division of Research and Innovation is the other focus of Caltrans research support. Through the efforts of this division, Caltrans has been a major partner with other groups in a research program carried out at the Pacific Earthquake Engineering Research Center (PEER) at the University of California at Berkeley. The goal of this program, known under the acronym PEARL (Program of Earthquake Applied Research for Lifelines), is to provide a scientific and engineering basis for the improvement of the design and operation of lifeline infrastructures. Many PEARL projects can be regarded as basic earthquake engineering hazard studies. The Caltrans Division of Research and Innovation has supported a total commitment of about $4.5 million over several years. Caltrans has also provided critically needed research facilities, particularly at the University of California at San Diego, that provide proof-testing of construction components for Caltrans projects as well as for research and development.

Of special note, Caltrans has sponsored essential research by outside entities since 1985. For example, the experimental work and testing of a specific type structure led to an aggressive retrofit program in which over 2,000 bridge structures have been seismically upgraded. In addition, Caltrans has placed seismic strong motion instrumentation that has already yielded a large design and response bonus, which will be multiplied many times when future California earthquakes occur. The results of the strong motion instrumentation measurements will narrow the uncertainties in ground motion shaking estimates (Section 5.3.1) and provide real-time earthquake intensity maps for postearthquake recovery and repair.

There remain many pressing research matters that have been identified by Caltrans practitioners as needing resolution. These needs include:

- Assessments of earthquake hazards, including ground shaking and liquefaction.
- Additional bridge fragility estimates for loads caused by liquefaction and fault rupture.
- Soil-structure interaction of bridge foundations.
- Improved methods of estimating losses.
- Direct and indirect cost values to enable sounder decisionmaking for highway performance.
- Better strategies for providing rapid field damage inspections.
- Better utilization of the "shake maps" developed in California.
- Assessment of seismic response modification devices (SRMDs) in bridge design and retrofit.

The need for resolution of these very practical needs has caused the Seismic Advisory Board to place a top priority on maintaining a high quality, productive seismic research program within Caltrans. There are several ways to make the present program more productive. For example, the SAB questions the efficiency of dividing seismic research in Caltrans between separate entities, specifically the Division of Research and Innovation and the Division of Engineering Services. Under a recent Caltrans seismic research plan, all contracts, including research contracts, will go through a competitive awards process, which will open the work to the most competent research entities, including state and private universities and private companies. The SAB has reservations about the effectiveness of the results of competitively selected awards where the quality of the research team and work do not dominate the cost of the proposed work in the evaluation criteria.

The Seismic Advisory Board's assessment of seismic-related research in Caltrans suggests that there is a need for this research in both the Division of Engineering Services and the Division of Research and Innovation, and that all research should be coordinated by a single Caltrans engineer reporting directly to the Director. This engineer should have responsibility for monitoring and developing priorities for Caltrans research efforts, both in-house and through grant and contract procedures. The engineer, perhaps a member of the Division of Engineering Services, should be a member of the Caltrans research committee, and have responsibility to suggest research policy and stimulate in-house formulation of research needs. The position would also be responsible for ensuring the implementation of research results within Caltrans.

Caltrans designers need to be encouraged to identify specific needs for research and have their requests screened and assigned priority by the head of their design group so that the in-house research committee can develop research programs with optimal usefulness and cost-effectiveness. Research goals and results in the past have been disseminated at very successful workshops, open to both Caltrans engineers and interested members of the profession.

The SAB favors a research program driven by Caltrans in-house research needs. This policy should enable the line item for research in the Governor's budget for this essential problem-solving program to be justified more readily in terms of necessity and cost-effectiveness. In this recommended process, attention has to be given to the need to avoid cumbersome administrative procedures and maintain continuity of purpose over time so that research projects respond to a consistent requirement during performance.

The research needs of Caltrans should be addressed without preventing other innovative research ideas from being identified and presented for consideration. The research administration process requires that Caltrans maintain a panel of design engineers to monitor technical progress and coordinate research. In addition, the research program must have a strong dissemination component, incorporate its findings into Caltrans practice, and be available for incorporation into general engineering bridge design practice.

The SAB can support, in an advisory capacity, the research functions of Caltrans and its technical advisory panels. Agendas of these meetings and minutes of substantial decisions made on research issues should be made available to the SAB. The Division of Engineering Services and the advisory panels should respond to specific requests from the SAB for information on the research program. The SAB should regularly schedule discussion on research needs at its meetings,

Figure 8-1. *Half-scale proof-test model in support of the San Francisco double-deck viaduct retrofits.*

Figure 8-2. *Carbon fiber jacketing of two-column bridge bent on the I-10, Santa Monica Viaduct.*

following appropriate presentations by Caltrans staff.

For practical reasons, Caltrans, as the major transportation agency of the State of California should, wherever feasible, support the state universities. California's universities depend crucially on contracts to further their engineering education and research programs. Their faculty has historically provided significant advice to Caltrans, and many former university students are now Caltrans professional staff. As a next resource, other universities and commercial and nonprofit organizations should be considered for research that cannot be appropriately performed by the state universities.

In conclusion, design-oriented research and subsequent implementation of results is essential to the continued effort to achieve the seismic safety of California's transportation structures. Consequently, the adequacy of research funding in the Caltrans budget must be reviewed annually. As mandated in 1990, whenever justified by Caltrans needs, research contracts should extend beyond in-house project-specific problems to more general engineering questions of a basic kind. While research for specific projects can be appropriately funded from within the Caltrans project funds, some significant general research funds should be available for the latter basic studies. Because of the importance of setting budgets and expenditure authority, the Director of the Department of Transportation must have responsibility for overall priorities and the role of the seismic research programs within the overall Caltrans mission.

The balance of this section describes examples of some of the problem-focused research completed under Caltrans sponsorship. Caltrans research has made use of many academic institutions to conduct these investigations, including the Universities of California at San Diego, Berkeley, Davis, Irvine and Los Angeles, the University of Southern California, and the University of Nevada at Reno. These and other academic institutions, professional societies—particularly the Applied Technology Council—and private organizations have provided a basis for advancing Caltrans seismic engineering practice and improving the prospects for reliable, safe, cost-effective highway bridges.

8.1 Development and Validation

8.1.1 Large-Scale Structural Testing and Retrofit Concept Development

Even before the damage to the Interstate-5 bridge column occurred in the 1987 Whittier Narrows earthquake, there was work going on at the University of California at San Diego (UCSD) to develop effective retrofit procedures for inadequately reinforced concrete columns. The first seismic retrofit technology development and validation programs were conducted at UCSD in direct support of the Caltrans Phase I and Phase II Bridge Column Retrofit Program. Over 200 one-third to one-half scale columns with different geometries and retrofit measures were tested to understand basic performance characteristics and develop reliable design models and

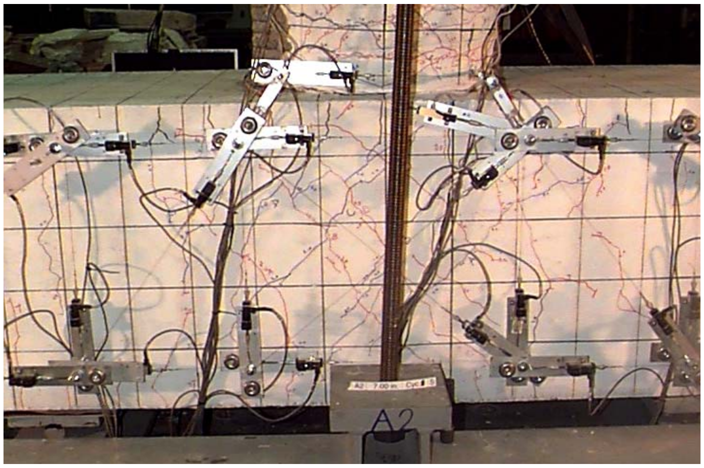


Figure 8-3. *Joint during test to design load levels.*

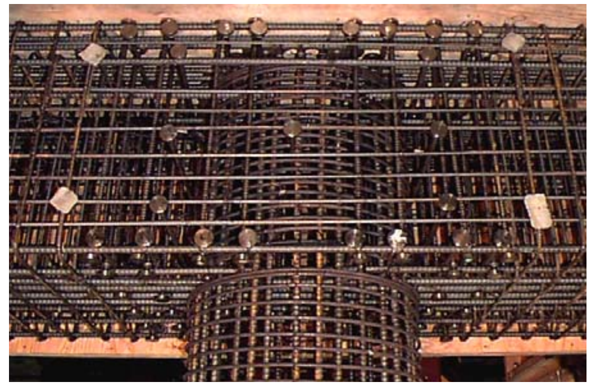


Figure 8-4. *Headed reinforcement in bridge joint.*

guidelines. Half- or full-scale systems tests were used for final retrofit technology validation. For example, half-scale models of the generic San Francisco double-deck viaduct retrofit concept (Figure 8-1) were tested at UCSD and at U.C. Berkeley, and a full scale proof-test of a carbon fiber jacket retrofitted two-column bridge bent from the I-10 Santa Monica Viaduct was conducted at the UCSD facilities (Figure 8-2).

The 1989 Loma Prieta earthquake demonstrated the vulnerability of older bridge construction under earthquake loading. Engineers at U.C. Berkeley responded by conducting field tests on one portion of the Cypress Street Viaduct (I-880) left standing. A subsequent series of laboratory experiments led to the development of new retrofit details. Caltrans has since used those new details to retrofit the remaining double-deck viaducts in the San Francisco Bay Area and for new double-deck joints such as those used in the San Francisco airport modifications.

Research on beam-column connections has continued with several experiments on design and construction of single-level viaducts. One key phase of the work studied new reinforcement geometries (Figure 8-3), including headed reinforcement that significantly reduces joint reinforcement congestion, and reduces bridge construction time and cost while not sacrificing seismic performance (Figure 8-4). The reinforcement details developed in the U.C. Berkeley and U.C. San Diego laboratories are now widely used in Caltrans bridge construction.

8.1.2 Special Facilities Development

Full-scale proof of concept testing and performance validation is particularly important for the special seismic response modification devices (SRMDs) described in Section 9.2, since devices of this size have never been manufactured or tested anywhere in the world prior to the California Toll Bridge Seismic Safety Program.

The size and dynamic performance range of these devices required the design and construction of a test system capable of full 6-DOF (degree of freedom) real-time seismic testing. In the case of isolation bearings, the full bridge self-weight and any time-dependent overturning seismic loads need to be applied through the device.

Caltrans commissioned the design and construction of these proof-testing facilities at UCSD on a very short time line (2 years from design to completion). The SRMD testing system, together with a friction pendulum bearing, was completed for the Benicia-Martinez Bridge (Figure 8-5). Performance characteristics of the SRMD testing system are provided in Table 8-1. The SRMD testing system has been used to prototype test all SRMDs for the Caltrans Toll Bridge Seismic Safety Program and to conduct many performance tests of the actual devices installed in the toll bridges.

The seismic response modification device (SRMD) testing system at UCSD will be used to recharacterize SRMDs installed in the toll bridges after five years. This was recommended so that aging and performance characteristics (such as friction and energy absorption) following prolonged exposure to marine environments can be determined.

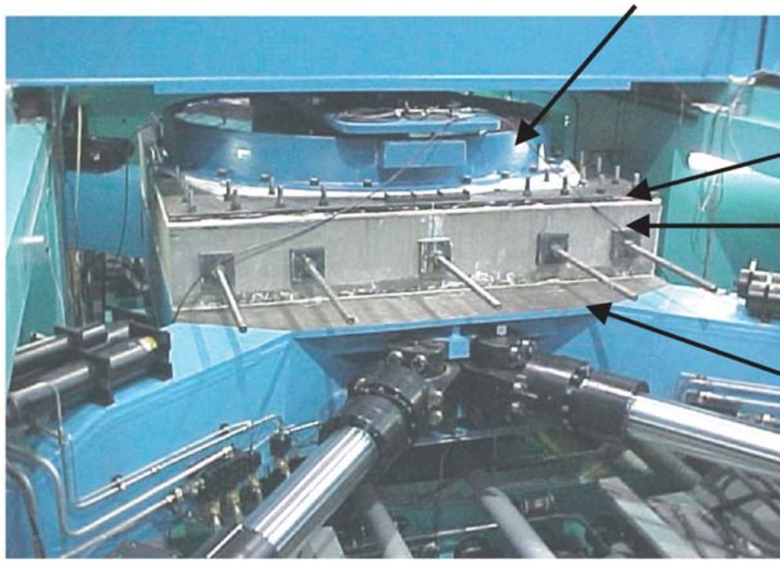


Figure 8-5. *Dynamic prototype and proof test of friction pendulum isolation bearing for the Benicia-Martinez Bridge.*

Table 8-1. *SRMD technical specifications*.

	Vertical	Longitudinal	Transverse
Force	53,400 kN (12,000 kips)	8,900 kN (2,000 kips)	4,450 kN (1,000 kips)
Displacement	±0.127 m (5 inches)	±1.22 m (48 inches)	±0.61 m (24 inches)
Velocity	±254 mm/sec (10 in./sec)	±1,778 mm/sec (70 in./sec)	±762 mm/sec (30 in./sec)
Clearance	Up to 1.52 m (5 ft)		~4 m (13 ft)
Relative rotation	±2°	±2°	±2°

The $15 million price tag for the SRMD testing system has been more than justified with the discovery of several design and manufacturing flaws in these unique devices that would have jeopardized seismic protection of toll bridges during a major earthquake.

Soil-Foundation-Structure Interaction

One of the least understood aspects of bridge seismic response is still the soil-foundation-structure interaction (SFSI), and Caltrans is currently developing a full-scale SFST testing facility at UCSD. The new SFSI facility will have two soil pits for exchangeable soil conditions and be capable of full-scale construction and testing of pile foundations and spread footings. The soil pits are directly adjacent to the world's first outdoor shake table (Figure 8-6), currently under construction for the National Science Foundation (NSF). This 25x40 foot outdoor shake table will have a 2,200-ton payload and will accommodate a large laminar soils box for real time SFSI experiments, including liquefaction.

8.1.3 New Seismic Design Details

Other new bridge design seismic details currently under development at UCSD are sacrificial abutment shear keys, designed to fuse and protect abutments and their pile foundations from difficult-to-repair damage. One concern is that current sacrificial shear key details result

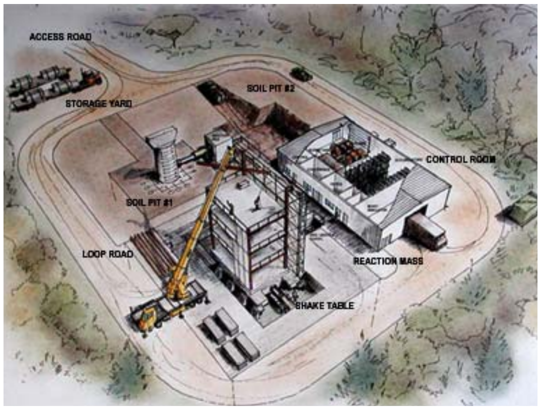


Figure 8-6. *Outdoor shake table and soil-foundation structure interaction test facility with two soil pits at U.C. San Diego.*

Figure 8-7. *Expected damage to current external abutment sacrificial shear keys.*

in significant unwanted overstrength and abutment damage (Figure 8-7). Development of new design details will allow a more predictable fuse (with pile protection) and perhaps enable design of abutments that will suffer less damage in earthquakes and be easier to repair if damaged (Megalli, Seible 2002).

For more economical construction over water or in congested urban areas, precast segmental bridge construction can offer significant advantages in construction/erection speed, concrete quality, and seismic response. Precast segmental construction has not generally been used in California due, in part, to uncertainties about the seismic performance of the segment-to-segment joints. Recent large-scale tests at UCSD have shown that not only do precast segment-to-segment joints have high and ductile rotation capacity, but that externally post-tensioned joints show improved seismic response following a large seismic event, with significant damage reduction and less permanent deformation (Figure 8-8; Seible et al. 2001).

All three new toll bridges currently under construction in California—namely the San Francisco-Oakland Bay Bridge East Spans, the third Carquinez Strait Bridge, and the second Benicia-Martinez Bridge—feature a new pier design that consists of hollow piers with highly confined corner elements for increased seismic deformation capacity (ductility). This concept of hollow piers with highly confined corner elements was developed and proof-tested at UCSD (Hines et al. 2002; Seible 2002) on ¼-scale models of a typical skyway pier of the new San Francisco-Oakland East Spans (Figure 8-9).

Figure 8-8. *Controlled damage in sacrificial external shear keys in proposed abutment design.*

Figure 8-9. *Quarter-scale San Francisco-Oakland Bay Bridge longitudinal pier proof test.*

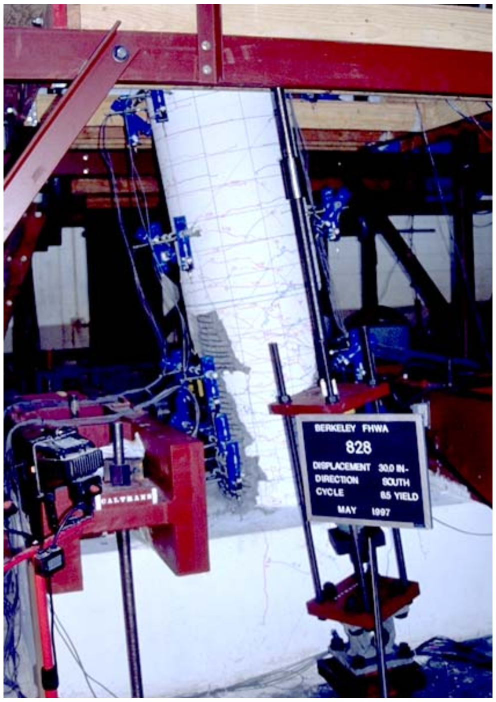


Figure 8-10. *Laboratory test on a bridge column proves its toughness and its ability to undergo extreme earthquake movements.*

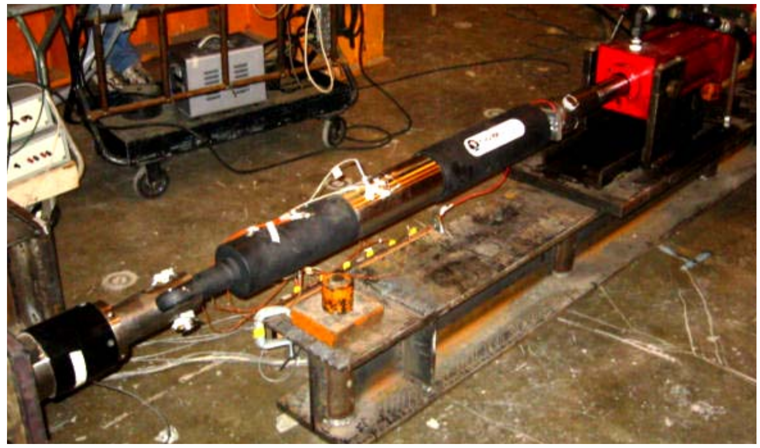


Figure 8-11. *Tests of large-scale damping devices used to absorb earthquake energy in major bridges.*

Figure 8-12. *Large-scale field test to determine stiffness and capacity of end diaphragm abutments.*

Bridge Pier Design Studies

Research results from studies on bridge pier design conducted at U.C. Berkeley have improved the ability to predict the response of reinforced concrete bridge columns, and thereby improve bridge designs. Early studies subjected large-scale bridge columns to simulated earthquake loading to identify how strong they were and how much movement they could sustain during an earthquake without failing (Figure 8-10). These studies have led not only to safer designs, but have enable Caltrans engineers to achieve safety with greater economy. Data from these and other tests have been made available in a web-accessible database.

Models of bridge columns are being shaken under simulated earthquake motions on the U.C. Berkeley earthquake simulator. These studies are providing basic information on the dynamic response of bridges and are the building blocks for effective bridge design computer programs that can simulate the complete dynamic response of a bridge under earthquake loading. The latest in computer technology for bridge seismic analysis can be found on the Internet.

The most recent phase of work is to devise innovative bridge design and construction solutions that will improve bridge performance so that bridges can continue to function following major earthquakes. One technology showing great promise incorporates post-tensioning reinforcement and special details to achieve tough bridge column designs that are "self-centering" so that they return to their original, undeformed position following earthquake shaking.

Other techniques in which U.C. Berkeley engineers are taking the lead include seismic isolation and energy dissipating devices for high-performance bridges, including applications to the major toll bridges (Figure 8-11).

End Diaphragm Abutment Walls

The seismic stiffness and capacity of end diaphragm abutment walls was a primary source of uncertainty for designers of bridges. Abutment walls play a major role in the seismic response of many bridges, and there was no experimental data to confirm the accuracy of

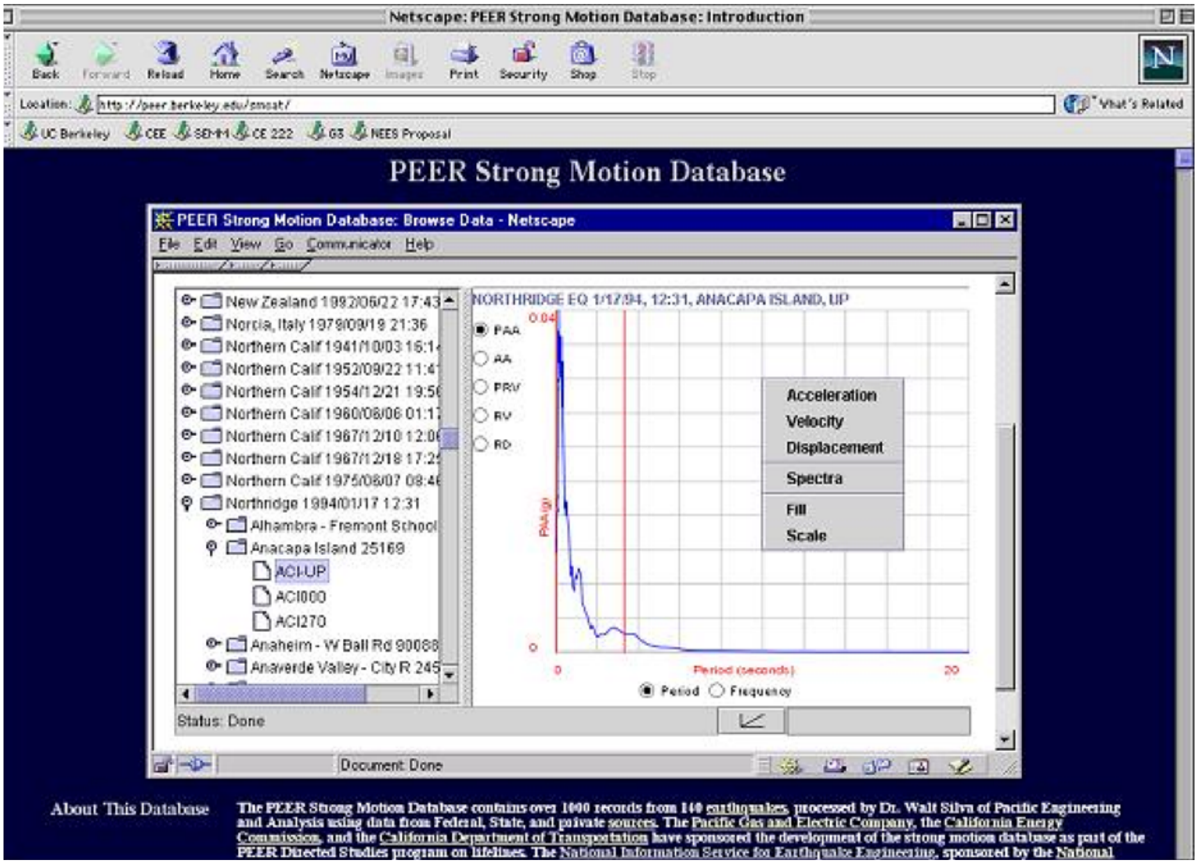


Figure 8-13. PEER strong ground motion database.

design values routinely used by Caltrans. Researchers at U.C. Davis combined a series of centrifuge model tests and a pair of near-full scale abutment tests (Figure 8-12) to obtain measurements of stiffness and capacity for different conditions. It was found that the design values being used for capacity were reasonable, but the design values being used for stiffness were too large.

8.2 Strong Ground Motion

8.2.1 Database Development

The University of California at Berkeley has a coordinated development and distribution program of a web-based, searchable database of strong ground motion data. This database is being developed with support from Caltrans and leveraged funding from other lifelines organizations working within PEER. The strong motion database (Figure 8-13) brings together over 1,500 strong ground motion records from 143 different earthquakes in a web-accessible format. The database has become increasingly popular with practicing engineers who use the ground motion data to shake computer models of bridges to better understand bridge performance in earthquakes. Engineering seismologists use the database for advanced computer simulations to study how earthquake waves travel from the earthquake source to a bridge site.

8.2.2 Rapid Estimation of Ground Motions

Immediately after a major earthquake, emergency responders and operators of lifeline systems in the affected area need guidance as to the likely distribution of damage. In areas that are densely instrumented with a network of seismometers, the measured distribution of strong ground shaking can be rapidly assembled and broadcast as an indirect measure of likely damage. In sparsely instrumented locations, however, there may be insufficient empirical data available. To supplement such data, new methods developed at U.C. Berkeley make it possible to automatically determine finite-source parameters of earthquakes, such as the causative fault plane characteristics and rupture velocity. These source parameters are then used to simulate near-fault ground

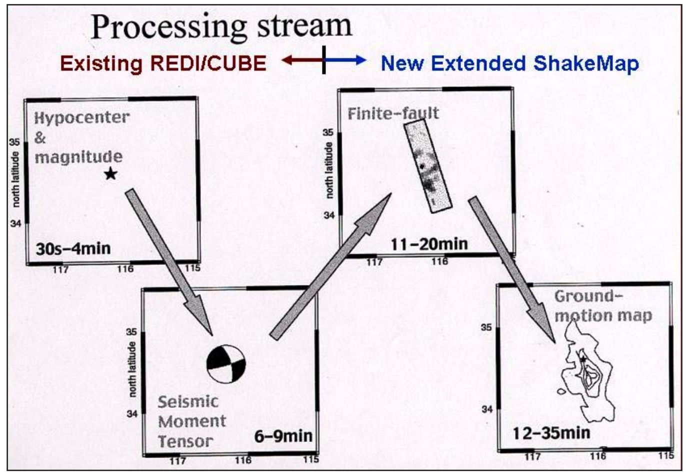


Figure 8-14. *Rapid estimation of ground motions.*

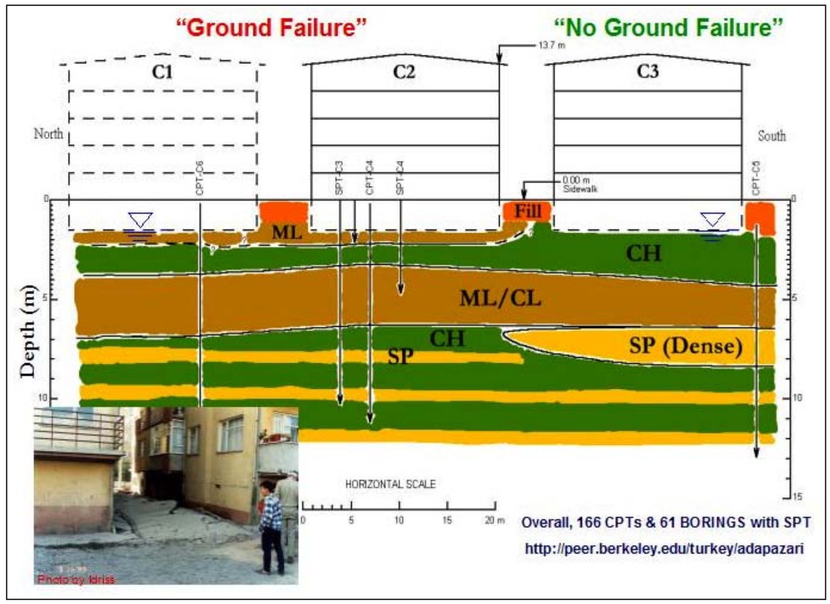


Figure 8-15. *A comprehensive subsurface investigation effort at this site uncovered the problem that led to extensive ground failure—the low plasticity silt (ML) directly below the structures that were damaged. This investigation shows that current liquefaction susceptibility criteria must be revised based on these case histories because it failed to categorize these silts as liquefiable.*

motions for areas where there are no nearby recording instruments (Figure 8-14). This process can be carried out automatically to produce and distribute estimates of shaking within 30 minutes of the event, and can then be reviewed and updated by seismologists in real time.

8.3 Geotechnical Research

8.3.1 Ground Failure

Scientists at U.C. Berkeley have been researching three main areas of ground failure:

- Identify when and where ground failures can occur.
- How to improve ground to prevent this failure.
- How to design foundations to survive these failures if they occur.

To identify when and where ground failure has occurred, researchers have traveled to major earthquake sites around the world, including Japan, Taiwan, and Turkey where they have been able to gather real-world data on what site characteristics lead to ground failure (Figure 8-15). Observations of ground failure from earthquakes elsewhere in the world are directly transferable to California. Field observations then guide carefully controlled laboratory experiments to understand in detail how ground failure occurs. Results of this research are being built into Caltrans design methods for bridge construction, as well as being incor-

Figure 8-16. Tests of bridge foundation embedded in a large "bucket of mud" shaken by the earthquake simulator help engineers understand how to better design bridge foundations.

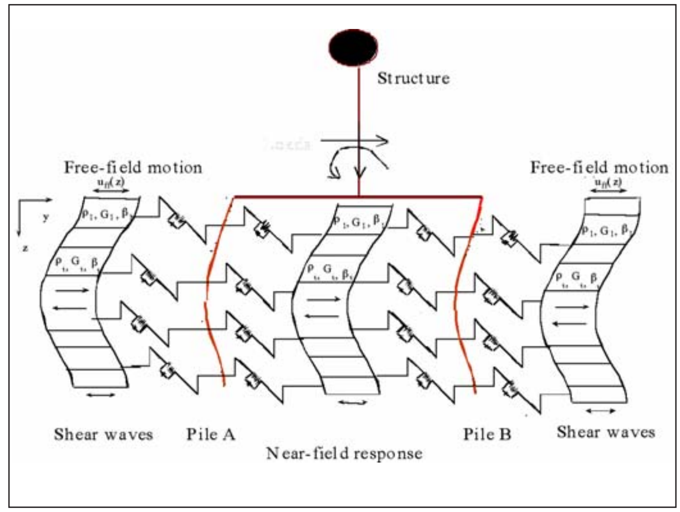


Figure 8-17. Symbolic engineering analytical model to describe the response of a bridge pier supported on very soft soils in near-field earthquake ground motions comparable to the experiment depicted in Figure 8-16.

Figure 8-18. The U.C. Davis 9-meter-radius centrifuge. A flexible shear beam container is loaded on the shaker at the end of the arm to the right. A technician is inspecting the rigid glass-walled container of another experimental model.

porated into computer programs for bridge performance simulations.

When ground failure does occur, its impact on bridge construction can be reduced by effective design of the foundation system. Foundations have been the subject of extensive research by U.C. Berkeley engineers. In one study, a giant "bucket of mud" was supported on the U.C. Berkeley earthquake simulator and subjected to massive earthquake movements (Figures 8-16 and 8-17). By studying the response of bridge foundations embedded in the bucket, Berkeley engineers have been able to better characterize how foundations can stabilize the otherwise unstable response of the soil. These findings are being incorporated into computer simulations that enable Caltrans engineers to fully understand the response of bridge structures in unstable soils subjected to earthquakes.

8.3.2 Geotechnical Modeling

Researchers at the University of California at Davis are investigating effects of earthquakes via laboratory testing, large-scale testing, centrifuge model testing, and theoretical work including finite element modeling and constitutive modeling. Theoretical work includes practical application of advanced tools to investigate soil-structure interaction, and development of rigorous constitutive models for soil-pile interaction and behavior of soils during earthquake loading. U.C. Davis is home to one of the largest (9-meter-radius) geotechnical centrifuges in the world. It is equipped for leading edge seismic research (Figure 8-18), and over the past 15 years has produced research with direct application to transportation structures.

Numerous research projects that addressed pressing seismic problems for California have been performed at U.C. Davis over the past 15 years:

- Performance of pile foundations for bridges in areas of liquefaction and lateral spreading.
- Seismic response of pile foundations for bridges in areas of soft clay deposits.

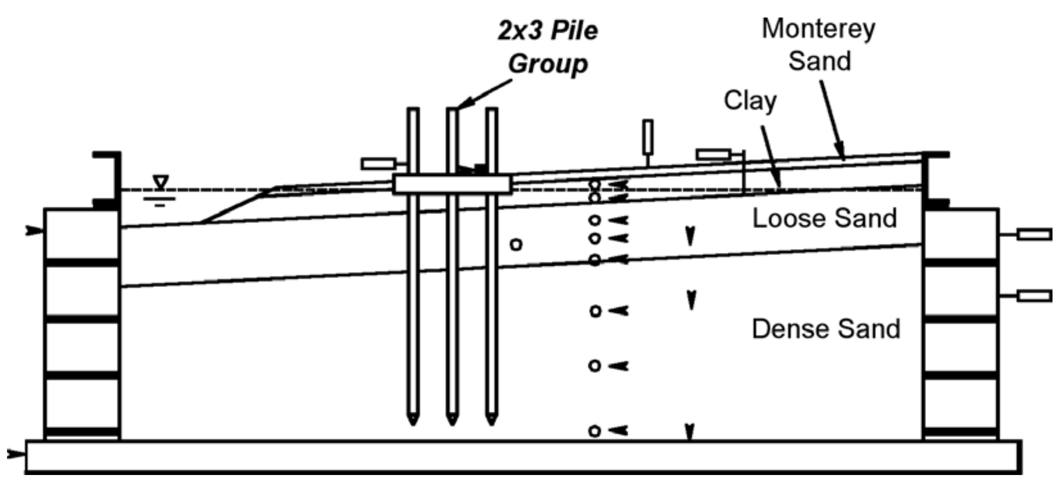


Figure 8-19. *Cross-section of a centrifuge model of a pile foundation in liquefied and laterally spreading ground during earthquake shaking.*

- Seismic site response of soft soil deposits.
- Performance of reinforced soil walls and reinforced soil abutments for bridges.
- Soil-structure interaction for sound walls on retaining walls and reinforced soil walls.
- Seismic stiffness and capacity of diaphragm abutment walls for bridges.
- Ground improvement for the remediation of liquefaction hazards at bridge sites.
- Lateral spreading of liquefied soils.
- Stability of slopes with liquefied soil layers.

In addition, numerous other projects have been performed that addressed seismic problems faced by other organizations (ports, harbors, industrial facilities), and the findings from such studies have been beneficial to Caltrans projects.

8.3.3 Centrifuge Modeling

Centrifuge modeling has emerged in the past 15 years as an essential tool for advancing the science and practice of geotechnical earthquake engineering. The U.C. Davis centrifuge provides engineers and researchers with the ability to use small-scale physical models to realistically represent field conditions. Since the behavior of soil depends on confining stress, a 2-foot-deep model of a soil deposit will not behave the same during an earthquake as a 100-foot-deep soil deposit. However, placing the 2-foot-deep model on a centrifuge and subjecting it to a centrifugal acceleration of 50 times earth's gravity produces stresses that are equal to a 100-foot-deep deposit, enabling much more accurate scale modeling. Developing a more accurate estimate of stresses is the key purpose behind centrifuge modeling.

The other big advantage of centrifuge modeling is that multiple soil and soil-structure systems can be constructed, highly instrumented, and shaken with many different earthquake motions to expeditiously obtain data that cannot be obtained from even the most ambitious field instrumentation efforts. These data are essential to the development of reliable methods for designing safe and economical systems.

The performance of pile foundations for bridges in areas of liquefaction and lateral spreading is a good example of U.C. Davis research for Caltrans. Extensive damage to pile-supported bridges and other structures in areas of liquefaction and lateral spreading has been observed in many large earthquakes, but the basic mechanisms of soil-pile interaction in liquefied soil and their effects on superstructure performance were very poorly understood only 10 years ago. There was an urgent need to develop design methods for these problems because of the implications for public safety, their effects on transportation networks, and the large cost of constructing foundations to resist the estimated effects of liquefaction and lateral spreading.

One series of centrifuge tests involved pile group foundations embedded in a soil profile of stiff clay over loose sand (Figure 8-19). During earthquake shaking, the loose sand liquefied and the overlying clay layer spread laterally downslope. Extensive ground cracking occurred in the clay layer as it deformed down slope (Figures 8-20 and 8-21). These ground failure patterns are very similar to

Figure 8-20. *Side view of the structure (its supported piles extend to near the bottom of the soil) after the top layer of sand has been removed. The exposed clay surface shows the mounding of soil on the uphill (right) side of the pile cap, and the opening of a gap on the downhill (left) side.*

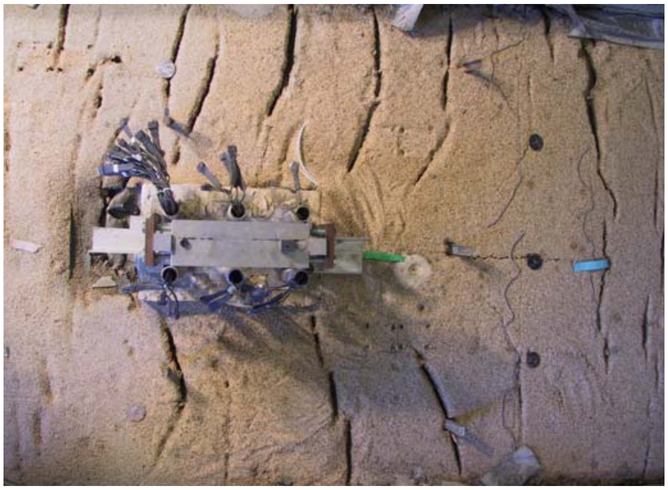


Figure 8-21. *View looking down on a structure after shaking. Notice the ground cracking as the ground spread laterally (from the top of the photo toward the bottom) due to liquefaction of the underlying layers.*

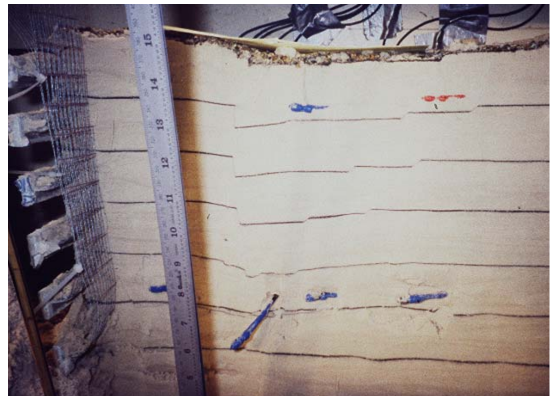


Figure 8-22. *Excavation of a model reinforced soil retaining wall showing failure wedges in backfill (note the offsets in the horizontal black sand marker layers). This model that suffered significant deformations during very intense shaking on the centrifuge.*

patterns observed in the field after large earthquakes. Each of these centrifuge models had more than 100 sensors that recorded detailed features of each test specimen's behavior during earthquake shaking, and these data identified load-transfer mechanisms that had not been previously anticipated. Subsequently, the data provided the basis for new, improved guidelines for analyzing and designing pile foundations under the effects of such lateral spreading loads.

Reinforced soil walls have increasingly been used as an economical choice for retaining systems along roadways and bridge abutments, but there was insufficient experimental data or field experience for the calibration or verification of seismic design methodologies. Over several projects, a broad set of experimental data covering a range of reinforced soil wall types and earthquake shaking characteristics was obtained through dynamic centrifuge modeling (Figure 8-22). Again, these data have provided the experimental basis against which seismic design methodologies have been revised and refined, thereby increasing the confidence with which designers can predict the performance of built structures.

Section 9

Technology Development and Application in Seismic Retrofit and New Bridge Design/Construction

Following the 1971 San Fernando earthquake, Caltrans embarked on a seismic safety program of unprecedented magnitude and complexity for bridges. Not only was the extent of the seismic vulnerability assessment and the Highway Bridge Seismic Retrofit Program enormous (over 75 percent of California's bridges were designed or built prior to the 1971 San Fernando earthquake), but earthquake engineering as a science was still very young and countermeasures to mitigate the seismic vulnerability of bridges needed to be developed and validated before they could be broadly implemented.

Thus, delivery of the Caltrans seismic safety program for bridges required significant research and technology development to provide the basic scientific and engineering tools to address bridge vulnerabilities. Both the technology development and application started immediately following the 1971 San Fernando earthquake. However, it was not until the 1989 Loma Prieta earthquake that sufficient resources were provided to accelerate seismic design and retrofit technology development and deliver bridge seismic retrofit in a holistic or systems approach, and not as a single-element fix.

The Highway Bridge Seismic Retrofit Program, while still not fully completed to date (over half of the locally-owned bridges still need to be addressed), has improved new bridge seismic design and detailing and has accelerated the development of new technologies, procedures, and tools to extend the seismic safety concept from the structure to the underlying geology and earthquake source, and from the individual bridge to entire transportation routes and systems.

The following subsections review some of the opportunities for adaptation of new technologies in bridge design. The SAB recommends that Caltrans continue to be receptive to new and innovative technologies, especially where they may allow improvements that cannot be accomplished using current practices.

9.1 Retrofit Technologies

While it is impossible to cover all retrofit technology developments and applications by and for Caltrans since the 1971 San Fernando earthquake, a few of the key technologies are highlighted here to illustrate their extent and impact.

Following the 1971 San Fernando earthquake, Caltrans faced the question of where to put limited financial resources. Limited resources could have been used to retrofit one or a few of the most vulnerable bridges, or to address a critical design issue with severe consequences for all bridges. Superstructure unseating at thermal expansion joints caused by short seat widths (Figure 9-1), and large relative seismic movements of adjacent bridge frames contributed to numerous bridge failures in the 1971 San Fernando earthquake. Cable restrainers across the movement joints (high strength wires or bars) or seat extenders (internal extra-strong steel pipes or external structural steel sections) were inexpensive and easy-to-install retrofit measures (Figures 9-2 and 9-3).

Due to the unknown location, magnitude, and intensity of the next major earth-

Figure 9-1. *Seat width problems in pre-1971 bridge construction.*

Figure 9-2. *External cable restrainer.*

Figure 9-3. *Seat width problems and seat extensions.*

quake, Caltrans opted for a first-phase retrofit program that would address the unseating issue at all critical bridge movement joints across the state, thus, effectively eliminating this common vulnerability with severe life safety consequences for all bridges. This comprehensive Restrainer and Seat Extender Retrofit Program was completed by Caltrans in 1989 and has largely prevented the unseating failure of bridge superstructures in subsequent earthquakes—with a few exceptions where bridge and/or restrainer geometry were not properly addressed.

In 1986, towards the end of the Restrainer and Seat Extender Retrofit Program, Caltrans started a comprehensive research and technology development program at the University of California, San Diego, to address another important bridge vulnerability with the potential for a significant life safety impact—namely that of bridge column failures. The 1971 San Fernando earthquake had shown numerous column failures due to shear flexural hinge failures, and rebar/lap splice debonding, all resulting from insufficient horizontal or transverse column reinforcement (Figures 9-4 through 9-6).

The retrofit solution for this deficiency was rather simple—the addition of horizontal reinforcement on the outside of the column through a jacket of concrete, steel, or other material. The wide variety of column geometries, reinforcement ratios, axial load ratios, and expected seismic deformation demands and required systematic understanding of the underlying mechanics and development of design guidelines and retrofit technology (Priestley et al. 1996). It was then appropriate for broad application to hundreds of thousands of bridge columns.

The most common type of column retrofit in California, namely steel jacketing (Figure 9-7), can be seen on virtually every major multi-level freeway interchange or overcrossing in California. Since single-column bents were considered particularly vulnerable due to lack of redundancy, Caltrans embarked on the Single-Column Bent Retrofit Program following the 1987 Whittier Narrows earthquake primarily to address confinement and lap splice clamping issues in the end regions of tall single-column bents.

This retrofit research and implementation program was significantly accelerated following the 1989 Loma Prieta earthquake and Governor Deukmejian's Executive Order

Figure 9-4. *Flexural confinement problems.*

Figure 9-5. *Reinforcement development problems.*

Figure 9-6. *Shear failures of short columns in the 1971 San Fernando earthquake.*

Figure 9-7. *Column jacketing.*

Figure 9-8. *Shear failures of short columns in the 1994 Northridge earthquake.*

D-86-90 to implement the Board of Inquiry recommendations (Housner et al. 1990). The benefit of the Highway Bridge Seismic Retrofit Program was proven during the 1994 Northridge earthquake. Close to the epicenter in the region where ground accelerations exceeded 0.25g, all 60 bridges retrofitted with Highway Bridge Seismic Retrofit Program technology survived the earthquake with little or no damage (Housner et al. 1994).

The same retrofit technology (column jacketing) with modified design guidelines has been extensively applied to shorter columns in multi-column bents under the Multi-Column Bent Retrofit Program. The shorter columns in multi-column bents are primarily prone to shear failures (Figure 9-8), and external jacketing (added horizontal reinforcement) can provide the additional required shear strength/capacity.

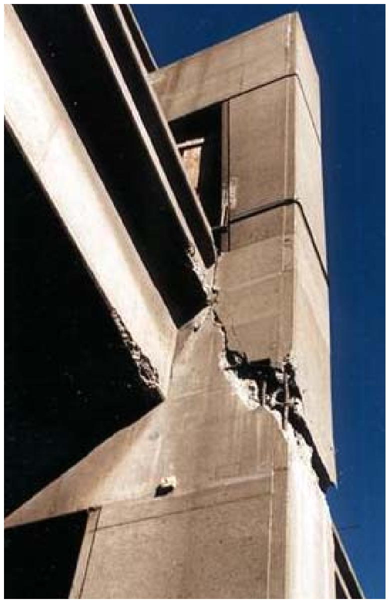


Figure 9-9. *Joint failures, I-880 Cypress Street Viaduct, Oakland, in the 1989 Loma Prieta earthquake.*

Figure 9-10. *San Francisco double-deck viaduct retrofit.*

The retrofit and strengthening of bridge columns in most cases also requires an assessment and strengthening of the adjacent member, namely the column/footing joint and the column/cap beam joint region. These joint regions are typically deficient in shear force transfer, a mechanism that is much more difficult to assess and to retrofit. In particular, the 1989 Loma Prieta earthquake re-emphasized the need to address beam/column joint problems with the damage caused to numerous double-deck viaducts (Figure 9-9), and in many cases only complete joint replacement (Figure 9-10) can result in reliable retrofit performance. To date, research is ongoing to develop retrofit technologies for joint regions that are not only effective, but also constructible and economical.

Increased awareness of the importance of soil-foundation-structure interaction (SFSI) during earthquakes has not only resulted in a better understanding of geotechnical aspects in bridge retrofit, but also to better characterization of soils and technologies to improve soils for better performance under strong ground shaking. For example, liquefiable soils can be stabilized through special drainage systems (e.g. stone columns) or mixing with cementitious materials (e.g. soil/cement mixing or jet grouting).

9.2 Seismic Response Modification Devices (SRMDs)

The Caltrans Toll Bridge Seismic Safety Program addresses California's long-span toll bridges and requires a different set of assessment tools and retrofit technologies. Most toll bridge superstructures and many of their substructures are made of steel, and their height and long spans often result in large spectral structural displacements. These large structural displacements can either be addressed by providing special elements to accommodate these displacements in the form of isolation bearings (Figure 9-11), and/or by providing special devices to limit these deformations through energy dissipation in the form of discrete damping devices, primarily viscous dampers (Figure 9-12). In other cases, lock-up devices or fuses (Figures 9-13 through 9-15) are used to engage or disengage portions of the structure in the event of large seismic motions. All of these special devices are introduced in a bridge structure to modify the dynamic structural response and are therefore referred to as seismic response modification devices (SRMDs). While Caltrans typically approaches the use of SRMDs in new bridge design cautiously, due to life cycle and maintenance issues, the retrofit of long-span toll bridges with high traffic volume does not allow for prolonged lane closures/traffic interruptions and SRMDs are frequently the only retrofit measure that allows implementation under full traffic.

Figure 9-11. *Friction pendulum bearing dish (3.8 m diameter) for the Benicia-Martinez Bridge retrofit.*

Figure 9-12. *Viscous dampers installed at the Vincent Thomas Bridge in Los Angeles.*

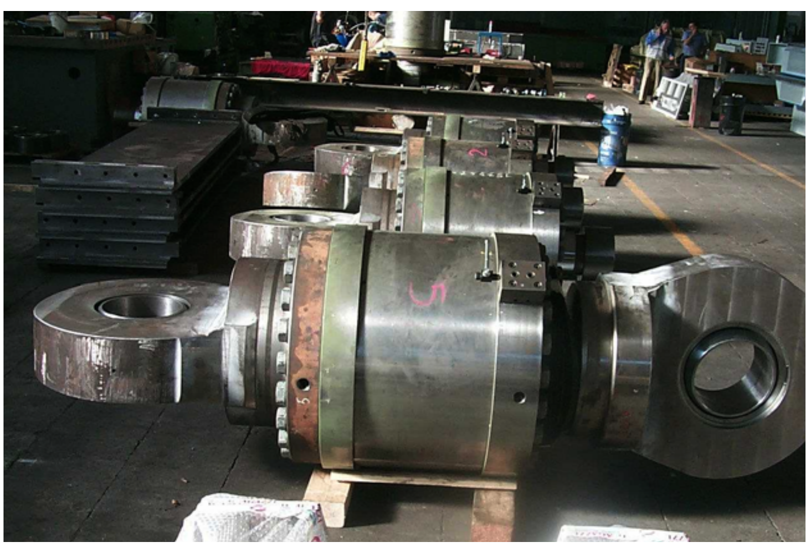


Figure 9-13. *Shock transmission units for the Carquinez Strait Bridge retrofit.*

Figure 9-14. *Shock transmission unit installed, Carquinez Strait Bridge retrofit.*

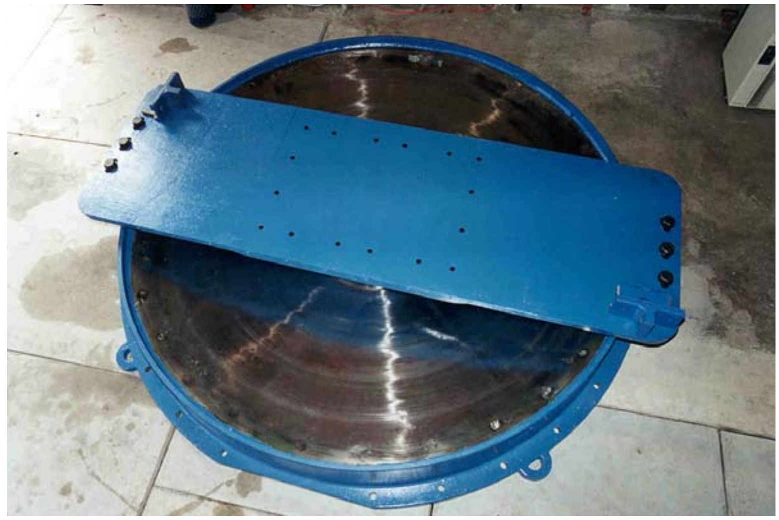


Figure 9-15. *Friction pendulum isolation bearing with windlocks.*

Figure 9-16. *Installation of friction pendulum bearing on the Benicia-Martinez Bridge.*

Figure 9-17. *Seismic retrofit of a masonry building using wet lay-up of fiber reinforced polymer (FRP) jackets for column retrofit.*

Figure 9-18. *Carbon fiber jacketing of bridge columns.*

The benefit of SRMDs in the retrofit of toll bridges can best be demonstrated by the example of the Benicia-Martinez Bridge. Isolation of the superstructure was accomplished by using large diameter friction pendulum sliding bearings installed on top of the piers and below the superstructure (Figure 9-16). This resulted in significantly reduced seismic demands in the superstructure steel truss members. Without isolation, over 75 percent of all members in the superstructure were deficient and in need of replacement or strengthening, a retrofit concept very difficult and costly to implement under traffic.

The California Toll Bridge Seismic Safety Program required SRMDs of sizes and capacities not commercially available at the time and the sheer size of these devices required in many cases the development of new construction/manufacturing techniques as well as the performance validation of these mechanical devices (Section 8.1).

9.3 Advanced Composite Materials

The past decade has seen a significant increase worldwide in the use of advanced composite materials for bridge rehabilitation to repair damaged structures, to strengthen structures for increased demand, and to retrofit structures for seismic action. In the United States, advanced composites were first used for seismic retrofit of buildings and bridges in the mid 1990s. However, uncertainties about durability and maintenance issues have limited installation on bridges to specially-funded demonstration projects.

Advanced composite materials consist of glass, carbon, or aramid fibers embedded in a polymer matrix and are referred to as fiber reinforced polymers (FRPs). FRPs can be applied to existing structures either in the form of a wet lay-up (fabric saturated with polymer resin; Figure 9-17), or in the form of surface bonded cured (typically protruded)

Figure 9-19. Rectangular column with carbon fiber jacket in half-scale laboratory test.

FRP strips (Figure 9-18). FRP overlays have been used successfully to strengthen columns or girders for shear, reinforced concrete slabs in flexure, and even on joints for external cap/column connections.

Some of the early applications of FRPs were developed and implemented during the Caltrans Phase I and Phase II Bridge Column Retrofit Program (Figure 9-19), and demonstrated significant benefits over the more conventional steel jacketing approach due to easy handling and speed of installation.

While a steel jacket installation on a typical single-column bridge bent for a high-level approach or overcrossing can take up to 3 or 4 days due to the extensive welding and grouting operations required, a CFRP (carbon fiber reinforce polymer) jacket can be installed and cured in a few hours, minimizing the need for lane closures and traffic interruptions.

Other key benefits of FRPs are their high mechanical characteristics and their light weight (up to 5 times stronger and 5 times lighter than mild steel). These benefits are offset by higher material costs and by uncertainties about the durability of the resin matrix in terms of UV degradation with time and of glass fibers due to their reactivity over alkaline environments.

Other issues that the Seismic Advisory Board believes need to be better addressed for broad based applications concern fire resistance and quality control/inspection measures during and after the retrofit installation.

9.4 Technology Development and Validation

Most of the retrofit technologies described above required special technology development specifically for bridge applications, since the bridge scale typically does not allow off-the-shelf technology. The bridge environment requires special manufacturing and construction considerations. Since, in many cases, life safety performance of a bridge depends on these retrofit applications, Caltrans required both proof-of-concept (proof testing) and performance validation (performance testing) of this bridge retrofit technology. While in many cases the basic retrofit concept was developed and tested on scaled model components of a bridge, final proof testing is often performed on full-scale prototype elements.

Since the 1989 Loma Prieta earthquake, Caltrans has a long and successful record of working with industry and academia to develop new seismic retrofit technologies and validate performance through large or full-scale laboratory testing under simulated seismic loads (see Section 8 for research results).

9.5 New Seismic Design Tools and Details

Caltrans seismic technology development is not limited to bridges, but also extends to other critical transportation components such as tunnels, embankments, approach slabs, and soundwalls. Design tools, seismic design details, and construction technology for new bridges in regions of high seismicity

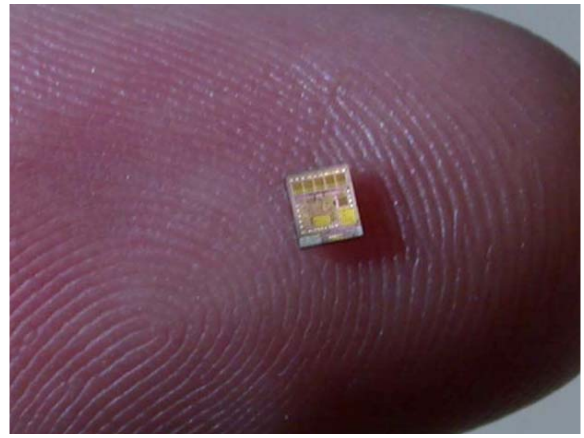


Figure 9-20. *"Smart dust" sensor (NEVIS).*

are also being improved through Caltrans-funded research.

More and more, seismic design tools for new bridges consist of time history analyses in direct support of the design process. These tools allow the bridge seismic designer to capture time-dependent demand aspects such as near-source velocity pulses and flings, as well as internal force redistributions through modeling of inelastic response. The Toll Bridge Seismic Safety Program's nonlinear time history analyses were used to validate final retrofit performance. Major new bridge designs for toll bridges routinely use three or more sets of 3-component time histories with wave passage and kinematic soil-structure-interaction effects as the design basis.

Foundations for new large bridges rely more and more on large diameter pile foundations, either in the form of CIDH (cast-in-drilled-hole) piles or CISS (cast-in-steel-shell) piles due to ease of installation and economy. Caltrans is still investigating the full soil-foundation-structure interaction characteristics of these large diameter pile foundation systems. Better quality assurance/quality control procedures to monitor construction and installation are still needed. With larger diameter piles, new pile-to-pile-cap connection details are also required to ensure capacity design protection of inaccessible, belowground components.

9.6 Sensors and Sensor Networks

New wireless smart sensors are becoming smaller and smaller—already the size of a penny including RAM, radio, battery pack and GPS (Figure 9-20), and less and less expensive (less than $100, depending on the sensor itself) so that deployment of hundreds or thousands of these sensors in a distributed fashion is possible in a very short time. The GPS equipped sensors can determine and report their position and data (e.g. accelerations) either continuously or triggered remotely by a threshold event, and the data can be transmitted wireless in near real time with only a few seconds latency to allow on-line data processing and information/knowledge generation. These sensors can also be calibrated to identify bridge traffic by type (truck, sedan, bicycle, pedestrian) and can thus be used also for bridge security on catwalks under major toll bridges. Digital video images can also be used to monitor critical or important bridge components, for example the lead rubber isolation bearings on the San Diego-Coronado Bridge (Figure 9-21). At the same time, the digital video image can be used to monitor the main shipping channel under the bridge for the U.S. Coast Guard, and the Navy assets in San Diego Bay for the U.S. Navy by establishing virtual security perimeters (a geo fence) around the asset or bridge pier and through pattern recognition (comparison of subsequent picture frames) alert the responsible agency of an intrusion in real time.

Finally, data can be transmitted wireless (Figure 9-21), with bandwidth currently up to 45Gbps through the NSF HPWREN (High Performance Research and Education Network) Internet backbone, which is already equivalent to the simultaneous transmission of images from 6 digital video cameras and

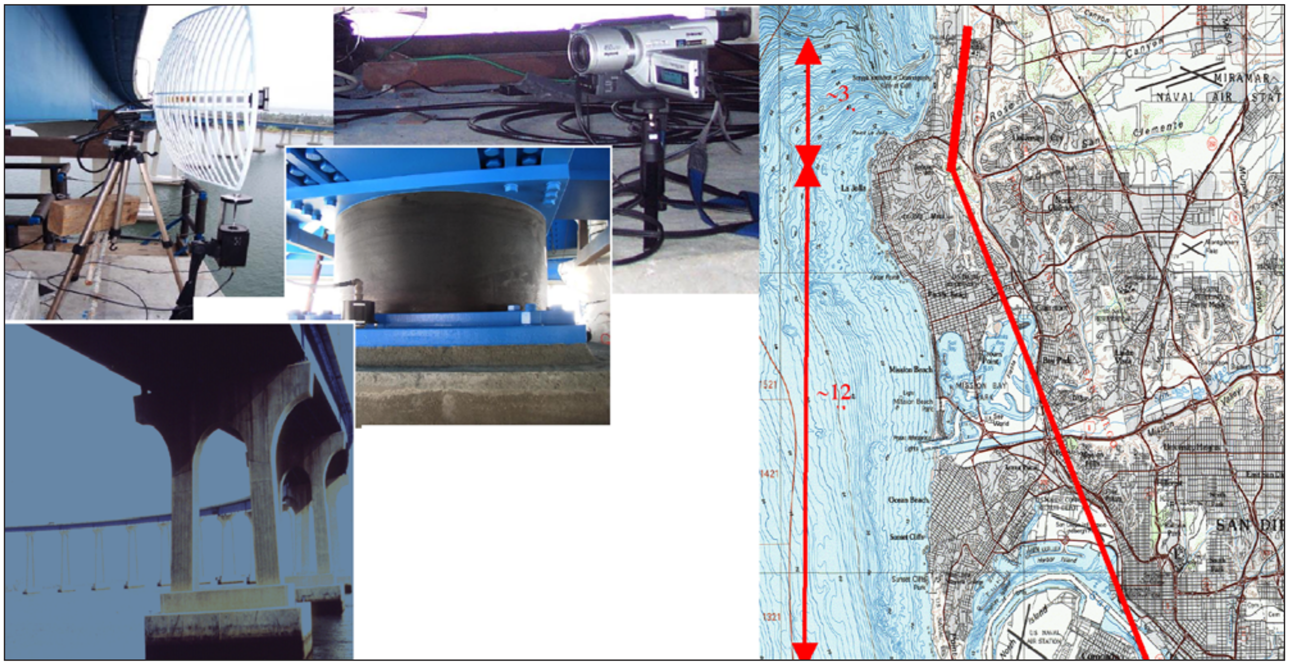


Figure 9-21. Coronado Bridge digital video installation for structure and security monitoring.

400 data channels in real time. These capabilities are increasing almost daily.

9.7 MEMS (Micro Electro-Mechanical Systems)

The next foreseeable sensor development will be in devices that cannot only sense, but also take certain action based on the data measured or received. These devices are referred to as MEMS (micro electro-mechanical systems). Combined with smart distributed systems and wireless communications technology, MEMS can be part of, or be used as, structural materials to create self-adjusting or self-correcting structures.

Recent advances in silicon chip design and manufacturing have resulted in this new class of devices. A MEMS device includes sensors, computer processor, memory, power source, and wireless communication on a single, miniature board. In addition to sensors for acceleration, other sensors are available for strain, weather parameters, chemical detectors, and light and infrared monitoring.

Newer MEMS are integrated with GPS receivers to sense location. Current MEMS have a package size on the order of centimeters, but in the future, much smaller packages are expected. The small devices require relatively low electrical power, and research is underway to develop robust power sources based on solar, wind, and power scavenging. MEMS devices have programmable computers, so they can be upgraded remotely as needs change. Since standard silicon chip manufacturing methods are used, the price of MEMS is expected to decrease significantly as the demand for the devices increases.

Studies are now underway to investigate the sensitivity, accuracy, and dynamic range of MEMS devices for earthquake engineering applications. The low cost will allow much denser instrumentation of ground motion. With MEMS devices, it is possible to contemplate instrumenting northern and southern California on a grid of 1 kilometer, which would require on the order of 100,000 devices. Dense instrumentation would significantly increase knowledge about earthquake source mechanisms, path effects, and the influence of site conditions and topography on ground motion. This ambitious goal can only be achieved with inexpensive MEMS that use wireless communication.

MEMS devices also allow monitoring of the structural health of bridges and other structures. Current instrumentation procedures for bridges capture the global dynamic response of a structure during an earthquake, but they are not sufficient to identify damage in a structure, which is a local phenomenon. Current seismic instruments do not record ambient motion, so data are unavailable for monitoring structural health. In contrast, inexpensive MEMS devices can be deployed in critical locations of a bridge such as foundations, plastic hinge regions, movement joints, bearings, and fatigue-rated components. They can be turned on to collect data on ambient motion, which can be processed to detect global and local changes in behavior of the bridge. During an earthquake, a dense structural array will provide data to identify damage and determine inspection priorities. Using forward simulation, rapid estimates could be made about the safety of a bridge in an expected aftershock. Integrating the data from individual bridges in the transportation network will provide critical information for emergency responders making critical decisions about road closures and alternative routing and for prioritizing inspection across a region.

Earthquake and structural applications for MEMS are one component in a possible (or potential) information architecture for Caltrans. Other information applications are loop detectors, traffic signal operations, and video surveillance, some of which will use other types of MEMS devices. As with earthquake and structural monitoring, these applications can share a common wireless communication systems, databases, and maintenance and operations. A number of applications for the transportation system will benefit by the shared information architecture.

Section 10

Summary and Review of Recommendations in *Competing Against Time* and Actions Taken

10.1 *Competing Against Time* Report

The report to Governor Deukmejian by the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake was entitled *Competing Against Time*. The Board of Inquiry identified three challenges that must be addressed by the citizens of California if they expect a future adequately safe from earthquakes:

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

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

1. Determine why the Cypress Street Viaduct of Interstate 880 and one span of the San Francisco-Oakland Bay Bridge failed in the earthquake.
2. Determine whether these failures were or could have been foreseen.
3. Advise on how to accurately predict possible future bridge and structure failures.
4. 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. Make recommendations 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 of Inquiry added responses to many of the questions that it felt the people of California would want to know the answers to, including the foremost question: *California's freeways—are they earthquake safe?*

The *Competing Against Time* report goes into great detail in responding to those questions and the three challenges by making specific recommendations to the Governor. The Board of Inquiry interpreted its Charter broadly and made recommendations that were directed both at seismic issues for bridges and some of the larger issues of seismic safety facing the state. As a result of its findings, the Board developed eight recommendations for implementation quoted below from *Competing Against Time* (1990).

Board of Inquiry 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 shall be that all transportation structures be seismically safe and that important transportation structures maintain their function after earthquakes.
2. Take the following actions with regard to public and private buildings and facilities.
    a. Propose legislation to ensure no new structure is exempt from adequate seismic safety standards.

b. Set goals that all state-owned structures be seismically safe and important ones maintain function after earthquakes.

c. Initiate and fund a research program to allow goals to be achieved.

3. Direct the Seismic Safety Commission to review and advise the Governor and legislature periodically on state agencies' actions in response to these Recommendations.

Board of Inquiry Recommendations for Action by the Director of the Department of Transportation

4. Prepare a plan to meet the goals established by the Governor.
5. Form a permanent Earthquake Advisory Board.
6. Ensure that Caltrans seismic design policies and construction practices meet the seismic safety policy and goals established by the Governor by considering the following:
   a. Review and revise as necessary standards, performance criteria, specifications and practices of current design practice.
   b. Institute independent seismic safety review of important structures.
   c. Conduct professional development in earthquake engineering.
   d. Fund a continuing program of basic and problem-focused research on earthquake engineering issues pertinent to Caltrans responsibilities.
7. Take the following specific actions for specific structures.
   a. Continue the work of the Independent Review Committee on the San Francisco Freeway Viaducts.
   b. Develop a long-term program for strengthening existing structures.
   c. Have seismic vulnerability analysis performed for important transportation structures.
   d. Implement a comprehensive program of seismic instrumentation.

Recommendations for Action by Transportation Agencies and Districts

8. Independent districts 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 analysis and evaluations of important transportation structures 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.

10.2 Executive Order D-86-90

The Governor of California signed Executive Order D-86-90 on June 2, 1990. That Order may prove to be the most significant step to improve seismic safety taken by the State of California in the last several decades. Executive Order D-86-90 sets for the first time the policy that all state owned and operated structures are to be seismically safe and that important structures are to maintain their function after earthquakes (Caltrans 1998). Simply put, the Governor required the Department of Transportation, in particular, and all other state agencies in general, to use generally-accepted earthquake resistant codes and to seek external evaluations of compliance.

Simpy put, the Governor required the Department of Transportation, in particular, and all other state agencies in general, to use generally-accepted earthquake resistant codes and to seek external evaluations of compliance. The full text of Executive Order D-86-90 is reproduced in Attachment 2.

10.3 How Well Has the Peer Review Process Worked?

The peer review process for individual bridges initiated by Caltrans in accordance with Executive Order D-86-90 is not "peer review" by a group, but often one or two knowledgeable bridge engineers (who may be Caltrans engineers), but who are not part of the design team. The majority of peer reviewers have been structural engineers, academics, and researchers knowledgeable in seismic design and analysis. They bring an independent perspective to review of the design and analysis of bridges. In this process, the scope of the peer review is directed more to the state-of-the-art than to the prior state-of-the-practice. At least three peer-reviewed projects are worth noting.

10.3.1 San Francisco Double-Deck Viaducts

These bridge structures, damaged by the Loma Prieta earthquake, were segregated into a number of design contracts for retrofit and awarded to engineering consultants. The scope of the Highway Bridge Seismic Retrofit Program was essentially to restore the pre-existing structural capacity. Some of the design and construction had been accomplished by the time a peer review panel was appointed (toll bridges were handled differently, see Section 10.4). The panel determined that the pre-existing capacity did not provide the level of performance mandated by the Governor's proclamation and that retrofit design should include strengthening of the viaducts. The panel's recommendation was reviewed and approved by Caltrans and the consultants' scope of services was amended.

The consultants were required to make periodic presentations as they developed the retrofit analysis and design. The presentations were critiqued and Caltrans representatives responded. As various analysis and design issues were identified, they were discussed and resolved with the design group. The retrofit of an existing structure is always more complex than new design, and this project was made even more complex by the introduction of performance objectives and

Table 10-1. Consultant-designed Caltrans Toll Bridge Seismic Safety Program projects.

Project	Project No.	Project Starts	Final PS&E	Retrofit Complete (expected)*	Estimated Project Cost $ (m)
1. Benicia-Martinez	59x473	06/95	02/97	09/01	98.4
2. Carquinez	59x476	06/95	05/97	08/03	88/53.2*
3. Richmond-San Rafael	59x475	06/95	12/98	12/04	282.7
4. San Mateo-Hayward	59x474	06/95	05/97	01/00	106.1
5. Vincent Thomas	59x477	06/95	12/96	07/99	28.0
6. San Diego-Coronado	59x478	06/95	09/98	02/02	55.7
				Total	**713.1**

* Retrofit of the 1958 span is complete and construction of a new span to replace the 1927 bridge opened to traffic November 11, 2003.

the need to maintain traffic during construction. A number of innovative analysis and design procedures developed during this peer review process were documented by consensus in ATC-32, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations*, and many of the procedures were subsequently adopted as Caltrans bridge design criteria. Some of the procedures introduced by the ATC project include:

- Displacement-based ductility for bridge piers.
- Joint shear design.
- Capacity design for foundations.
- Strut and tie models.
- Minimum stand-alone criteria for individual frames.
- Performance-based design.

Two of the damaged viaducts (Embarcadero and Beale Street offramps) were demolished, but not replaced. A third viaduct (Central Freeway) was partially retrofitted—only the double-deck sections were eventually demolished. Only the Southern Freeway Viaduct (I-280) was completely retrofitted. The peer review panel was kept in force throughout construction, but only nominal services were required at that point.

10.3.2 Replacement of Cypress Street Viaduct

This project was divided into four segments. Three of the segments were awarded to consultants, and Caltrans designed the fourth in-house. Caltrans provided the oversight for the design and prepared analysis and design issues to be discussed and resolved with the panel and the design consultants. Many of these issues were similar to those encountered in the San Francisco viaducts and similar resolutions were recommended. The participation of the peer review panel was limited to the resolution of the pre-design issues.

10.4 Toll Bridge Seismic Safety Program

California's toll bridges are unique, long-span, complex structures, and their continued functionality is of significant importance to the social and economical vitality of the state. As a result of the Governor's Executive Order D-86-90, Caltrans is required to institute independent seismic safety reviews for the retrofit of Important bridge structures. Consequently, California's state-owned toll bridges and their seismic retrofit designs were designated to be peer reviewed by an independent seismic safety Peer Review Panel (PRP).

The seismic retrofit of California's toll bridges, the Toll Bridge Seismic Safety Program, has presented unique engineering challenges due to the vintage and complexity of these long-span bridge structures and their socioeconomic importance to the state in terms of life safety and transportation network functionality (Table 10-1).

The retrofit design for one of California's ten toll bridges, namely the San Francisco-Oakland Bay Bridge (SFOBB) West Span, was handled in-house by Caltrans. Six of the Caltrans-controlled designs for the Toll Bridge Seismic Safety Program were performed by independent engineering consulting firms or joint ventures with close contractual control by Caltrans Office of Structure Contract Management (OSCM), and technical supervision and general input from Caltrans Translab and Division of Structures, the Caltrans Seismic Advisory Board (SAB), the Value Analysis (VA) team, and the seismic safety Peer Review Panel. Two other Caltrans toll bridges, namely Antioch and Dumbarton, are of newer vintage and not considered among the top priority bridges for retrofit (Table 10-2). The Golden Gate Bridge is owned and operated by a separate bridge district with an independent retrofit program.

This section provides a summary of the Caltrans Toll Bridge Seismic Safety Program and process, and summarizes the Peer Review Panel's assessment (Caltrans 1999) for the six consultant-designed retrofit projects.

10.4.1 Performance Expectations

The seismic performance expectations of the toll bridges vary for the different bridge structures. While all bridge retrofits have to be designed to meet the "no-collapse" criterion under the Safety Evaluation Earthquake (SEE), the expected performance following the SEE depends on the bridge designation and its importance in the state's transportation highway system. For example, the Benicia-Martinez Bridge is on a state lifeline route and is designated an Important bridge that requires return to service within a reasonably short time following the SEE.

However, significant damage with prolonged closure can be accepted at other bridge locations. In addition to the response under the SEE, the Toll Bridge Seismic Safety Program designs also addressed a more frequent moderate seismic design event, namely the Functional Evaluation Earthquake (FEE) to ensure prescribed levels of service, depending on the bridge designation. (Note: For some of the toll bridges, e.g., the Richmond-San Rafael, the FEE was relaxed or eliminated by Caltrans as a design event). Finally, two of six toll bridges, namely the Vincent Thomas and the San Diego-Coronado, are crossing potentially active faults with the possibility of ground surface fault offsets within the bridge domain, which required a third design level, namely the Fault Rupture (FR), to be considered in conjunction with the "no-collapse" performance requirement. A summary of the Caltrans multi-level design approach used for the Toll Bridge Seismic Safety Program, with descriptive performance levels for different bridge designations is provided in Table 10-3, together with general definitions of the SEE, FEE and FR. The specific performance requirements will be discussed separately for each of the six bridge retrofit projects.

Table 10-2. California's toll bridges.

Bridge	Const. Comple-tion (Year)	Avg. Daily Traffic (# of Vehicles)	Total Length (m)	Main Span Type	Main Span Length (m)	Estimated Retrofit/New Const. Cost ($million)
Existing (Retrofit)						
Golden Gate	1937	120,000	2,790	Steel stiffening truss suspension	1,280	200
Antioch	1978	30,000	2,880	Twin corten-steel girder composite with lightweight concrete deck	140	—
Benicia-Martinez	1962	100,000	1,896	Steel truss	161	100
Carquinez Strait (1927 WB)	1927/ 1958	111,000	1,022	Steel cantilever truss	336	62
Richmond-San Rafael	1956	35,000	6,309	Steel cantilever truss	326	348
San Francisco-Oakland West Bay Spans	1936	280,000	6,100	Steel stiffening truss suspension	705	214
San Mateo-Hayward	1967	75,000	11,273	Steel box girder with orthotropic steel deck	229	112
Dumbarton	1981	45,000	2,600	Steel box girder lightweight con-crete composite	104	—
Vincent Thomas	1964	90,000	1,849	Steel stiffening truss suspension	450	26
Coronado	1969	63,000	3,440	Steel box girder with orthotropic steel deck	201	64
New						
Third Car-quinez Strait	2003*	111,000	1,028	Orthotropic steel box suspension	728	188
Second Benicia-Martinez		100,000	1,653	Segmental PS lightweight con-crete cantilever	161	184
New San Fran-cisco-Oakland East Bay Replacement		280,000	3,100	Self-anchored orthotropic steel box suspension; segmental PS	565 total	1,500

*Now in use.

Table 10-3. *Caltrans seismic performance criteria for toll bridges.*

Ground Motion	Minimum Performance Level	Limited Performance Level	Full Performance Level
Functional Evaluation Earthquake (FEE)	Immediate Full Service (I) Repairable damage within 90 days. Allow lane closures outside peak hours. Minor concrete spalling, joint damage and limited buckling of secondary steel members.	Immediate Full Service (II) Repairable damage within 30 days. Repairs will require minimum interference with the flow of traffic. Minor concrete spalling, joint damage and limited buckling of secondary steel members.	Immediate Full Service (III) Minimal damage. Essentially elastic. Minor concrete cracking. Minor buckling in secondary steel members.
Safety Evaluation Earthquake (SEE)	No Collapse (I) Significant damage with a high probability of repair. Maintain vertical load carrying capacity and a minimum lateral system capacity. Damage may require full closure for public traffic. Repairs will require complete evaluation.	Limited Service Intermediate repairable damage. Light emergency vehicles within hours. Reduced public traffic lanes within days. Lateral system capacity is relatively reduced. Repairs within a year.	Immediate Full Service (I) Minor repairable damage. Lateral system capacity slightly affected. Minor concrete spalling, joint damage and limited buckling of secondary steel members. Lane closure outside peak hours only. Repairs within 90 days.
Fault Rupture	No Collapse (II) Extensive damage with low probability of repair. Maintain residual capacity for probable vertical and lateral service loads only.	Not applicable	Not applicable

See Table 5-2 for Standard bridges.

Definitions:

FEE: Functional Evaluation Earthquake. The Functional Evaluation Earthquake (FEE) shall be based on the spectra for a 285-300 year return equal hazard (probabilistic) event. This (FEE) corresponds to 60 percent probability that the ground motion is not exceeded during the useful life of these toll bridges, considered to be around 150 years.

SEE: Safety Evaluation Earthquake. The Safety Evaluation Earthquake (SEE) shall be based on the target response spectra. For the San Francisco Bay Area, the 85th percentile rock spectra for the maximum credible event corresponds approximately to the 1000-2000 year return period equal hazard spectra and was selected as a target spectra. For the San Diego and Long Beach areas, motions a little below the 84th percentile deterministic rock motion spectra were selected for target spectra. This corresponds approximately to a 950-2,000 year return period equal hazard spectra.

FR: Fault Rupture. For Fault Rupture (FR) assessment, consult a geotechnical engineer. The findings are subject to approval by a Caltrans consensus group.

10.4.2 Toll Bridge Performance Assessment and Retrofit Design

Each of the state's toll bridges is unique in geometry, bridge type, vintage, design details, and seismic reliability and performance. This did not allow predefined assessment and retrofit concepts to be applied, but rather required independently developing for each bridge performance and design criteria, hazard/vulnerability assessment and retrofit strategy development, analytical modeling and analysis applications, and retrofit design and details. In each of the design teams, significant progress was made in all of the above areas with a strong commitment by both Caltrans and the consultants to ensure the highest degree of reliable seismic performance of these vital bridges.

Retrofit concepts and details varied. A more conventional approach of strengthening and displacement control was employed for the Vincent Thomas, San Mateo-Hayward, and Carquinez Strait Bridges. Strategic seismic response modification through EBFs (eccentrically braced frames) was employed for the Richmond-San Rafael Bridge. The San Diego-Coronado and Benicia-Martinez Bridges used base isolation strategies. In all cases, the resulting retrofit designs are tailored to the specific bridge and site condition and are expected to meet or exceed the specified performance criteria.

The Toll Bridge Seismic Safety Program for the six consultant-designed toll bridge retrofits referenced in Table 10-1 commenced in 1995 with design efforts, and construction is expected to be complete in 2010. The importance of the toll bridges in terms of life-safety and socioeconomic function for the state resulted in seismic assessments and retrofit designs that significantly expand the state-of-the-art in several key areas.

Hazard Definition

Site-specific hazard assessments were performed for each of the six toll bridges based on controlling source mechanisms, source-to-site parameters and, where applicable, near-source motion characteristics. A consistent methodology and approach were developed by the SAB's Ad Hoc Committee on Soil-Structure-Foundation Interaction (SFSI) defining the generation of representative rock motion spectra and time histories, site-specific motion development along the entire bridge length, and soil-foundation-structure interaction models for different soil conditions. In particular, the systematic introduction of near-source effects through adjusted long period fault normal and fault parallel target spectra, as well as consideration of dynamic and kinematic SFSI effects at each pier location, were performed for the first time in a consistent approach for bridge retrofit design.

Nonlinear Analytical Modeling

The significant inelastic response of the retrofitted bridges under the Safety Evaluation Earthquake (SEE) required nonlinear analytical models that captured all inelastic actions in elements, sliding joints, hinges, and seismic response modification devices to estimate the most likely response of the bridge. Thus, the most probable deformations and forces in the bridge under the SEE design event in the form of time history analyses were employed as the design basis. The differences in the individual bridges required that each design team independently had to deal with these nonlinear analysis/design issues. This approach represents a significant advancement in the state-of-the-art in seismic response assessment.

Retrofit Design

In order to meet the required performance specifications, in many cases conventional retrofit approaches of strengthening and displacement control did not suffice. Thus, new strategies and concepts needed to be developed, which in some cases far exceeded current engineering practice. For example, full isolation of the Benicia-Martinez superstructure resulted in a bridge that is expected to be fully operational after the SEE. Using this approach requires accommodating superstructure displacements of ± 4 feet at structural response periods between 3.5 to 5 seconds, a range in which reliable input information becomes sparse and questionable. Many of the toll bridge retrofits resorted to seismic response modification devices such as isolation bearings and dampers for displacement control and energy dissipation, in conjunction with conventional strengthening techniques for capacity designed components.

Plan Check and Performance Validation

Caltrans required an independent plan check and performance validation of the retrofitted toll bridges. These were required to be performed by a separate team utilizing a completely independent global nonlinear analysis model. In most cases, even completely different analytical tools and programs were employed. This independent check and analytical validation of the retrofit measures was in parallel to a complete technical review by Caltrans of all retrofit designs, an independent evaluation of retrofit measures and their constructability by a Value Analysis (VA) team, and the independent seismic safety review of the Peer Review Panel. These measures make bridges under the Caltrans Toll Bridge Seismic Safety Program some of the most heavily checked and scrutinized seismic design projects anywhere.

Caltrans not only assisted in pursuing new ideas and concepts, but supported the validation of these ideas and concepts with large or full-scale experiments and detailed local inelastic analysis models. The list is long and includes the testing and evaluation of the San Diego-Coronado precast prestressed piles, the H-piles for the Carquinez Strait, Richmond-San Rafael, San Mateo-Hayward-and Vincent Thomas Bridges, the restrainer beam system for the Carquinez Bridge, the eccentrically braced frames (EBFs) and existing tower leg retrofits for the Richmond-San Rafael Bridge, and finally, all seismic response modification devices (SRMDs) such as isolation bearings, dampers, and shock transmission units present in one form or another in all of the toll bridges.

10.4.3 Limitations and Conclusions of Toll Bridge Retrofit Design

Areas where the retrofit schedule and cost constraints limited the design effort for the toll bridges are in: 1.) a better definition and assessment of the Functional Evaluation Earthquake (FEE) design level, and 2.) the rigorous assessment of the final retrofit design with multiple sets of input motion time histories.

While at the start of the Highway Bridge Seismic Retrofit Program, a clear dual-level design approach was specified in the form of a SEE and FEE. The FEE was never fully defined in the hazard development and in the structural performance assessment phases. Typically, the FEE hazard was taken directly as 60 percent scaled SEE, with the same spectral shape and site parameter definitions, and did not really provide a separate design evaluation. On the other hand, the performance definitions for both SEE and FEE did not differ significantly, so design to SEE level automatically ensured compliance with FEE requirement at the 60 percent reduced input.

While for the Toll Bridge Seismic Safety Program, the chosen design approach results in adequate functionality both for moderate and strong earthquakes, future Caltrans bridge design and retrofit should focus on a better definition of the FEE. This should be both in terms of a probabilistic hazard development for this reduced design input and in terms of better quantification of performance parameters in engineering and socioeconomic terms for a meaningful performance-based design for the more frequent low to moderate seismic events.

Finally, the retrofit assessment and design of five of the six consultant-designed toll bridges was strictly based on a single set of ground motion time histories, even though it was made spectrum-compatible at the rock motion level. The studies for the Benicia-Martinez Bridge and the San Francisco-Oakland Bay Bridge East Spans, with multiple sets of spectrum-compatible time history input, have shown that significant response variations can be observed, particularly in structures with nonlinear inelastic response, and it is not clear that response, amplification factors established for the Benicia-Martinez Bridge to account for these response variations are met in the other five Toll Bridge Seismic Safety Program designs.

While it is questionable whether or not these factors readily apply to other bridges, the Peer Review Panel strongly suggested to Caltrans to follow through with the initial intent—namely to perform retrofit design validations with at least three sets of time histories and to ensure that the retrofit designs are adequate for the maximum response of the three input motions. With the exception of this one issue of multiple sets of time histories for overall validation of the seismic retrofit designs for the toll bridges, the Peer Review Panel was satisfied with the design process. Caltrans has not only met, but significantly expanded, the state-of-the-art in seismic hazard definition, bridge response assessment, and retrofit design. The SAB expects that the retrofitted toll bridges will meet the defined performance requirements with a high degree of structural reliability.

Table 10-4. Program status summary by lead agency.

Lead Agency	Number of Bridges	Pre-Strategy	In Design	Under Construction	Retrofit Complete	No Retrofit Required	% Delivered
Los Angeles County	293	15	76	20	129	53	62
Santa Clara County	38	0	4	0	25	9	90
Caltrans	903	282	177	101	230	113	38
Total	1234	297	257	121	384	175	45

10.5 Actions Taken by Other Transportation Jurisdictions

10.5.1 Local Bridges

The California Seismic Safety Retrofit Program, which was established by emergency legislation (SB 36X) following the 1989 Loma Prieta earthquake, includes the Local Bridge Seismic Retrofit Program.* The program provides funding assistance to local agencies for remedying structural design deficiencies of public bridges on local streets and roads in California. Following 1989 Loma Prieta, all of California's approximately 24,000 bridges were screened by Caltrans for risk of collapse during future seismic events. Of these, approximately one-half (12,000) are publicly-owned local agency bridges. The screening process carried out by Caltrans indicated that about 1,150 of these local agency bridges may pose a threat to life safety and should be reviewed for needed retrofit. The local agencies themselves identified an additional number of their bridges that needed consideration for seismic retrofit, bringing the total number of potentially high-risk local bridges to 1,234.

Three lead agencies have been designated to carry out the Local Bridge Seismic Retrofit Program in California. These agencies are Los Angeles County for the 293 potentially high-risk bridges located in that county, Santa Clara County for the 38 bridges of similar classification located in that county, and Caltrans for the remaining 903 potentially high risk bridges located elsewhere in the state. These 903 bridges include 227 Bay Area Rapid Transit (BART) bridges and 25 Department of Water Resources (DWR) bridges.

The status of the Local Bridge Seismic Retrofit Program as of July 1, 2003 is summarized by lead agency in Table 10-4, which shows a number of bridges in five categories:

1. *Pre-Strategy*
2. *In Design*
3. *Under Construction*
4. *Retrofit Complete*
5. *No Retrofit Required*

The *Pre-Strategy* category includes all bridges for which either the final strategy report has not been received or final strategy decision has not been reported to the lead agency. The meaning of each of the other four categories is self-evident. Note that approximately one-fourth (297) of the 1,234 potentially high-risk bridges are in the *Pre-Strategy* category. Those in this category include 227 BART bridges and 25 DWR bridges, leaving 45 bridges in other local jurisdictions. The local agency having jurisdiction over each bridge in this category is responsible for developing the associated retrofit strategy and reporting it to the designated lead agency. Failing to do so, this agency must accept full responsibility for any unsatisfactory structural performance during future earthquakes.

* The information in this Section 10.5.1 was obtained from the Caltrans "Local Bridge Seismic Retrofit Program Progress Report" dated July 1, 2003. This progress report is updated every three months.

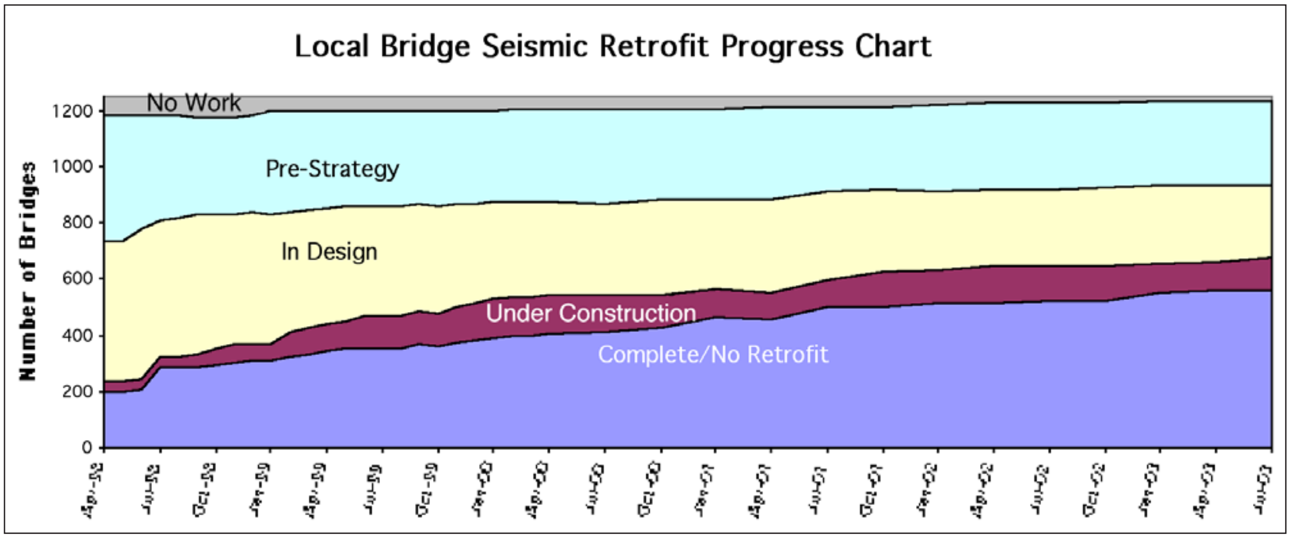


Figure 10-1. *Local Bridge Seismic Retrofit Program progress chart (BART and DWR bridges not included).*

The retrofit progress chart (Figure 10-1) indicates the number of potentially high-risk local bridges, in each of the five categories as a function of time starting April 1998 and extending to July 2003. The number of bridges in the *Pre-Strategy* category decreased rapidly for a couple of months after May 1998, but has leveled off, leaving 297 in this category as of July 2003.

In the interest of public safety, the Seismic Advisory Board recommends that every effort be made to advance these 297 bridges (included are 227 BART and 25 DWR bridges), a total of 297 bridges, into the next *In Design* category and further to advance all of the potentially high-risk local bridges into the *Retrofit Complete* category as soon as possible.

Critical to accomplishing the latter goal of moving all of the potentially high-risk local bridges into the *Retrofit Complete* or *No Retrofit Required* categories is securing the required funds for design and construction. Under plans developed following the 1989 Loma Prieta earthquake, the Local Bridge Seismic Retrofit Program was to be funded 20 percent by the State of California through Caltrans and 80 percent by the Federal Highway Administration (FHWA) through its Highway Bridge Replacement and Rehabilitation (HBRR) program.

This historic division of funding for design and construction has led to the progress shown in Figure 10-1. At the present time, the 80 percent FHWA funding for the local program appears to be reasonably assured, but the state is no longer providing the 20 percent matching funds. Unless the state again provides these design funds, it is unlikely that the Local Bridge Seismic Retrofit Program can be completed in a timely manner.

The Governor's proposed 2003-4 budget for Caltrans reduces the local assistance budget for the state matching funds by $10.5 million for the 2003-2004 fiscal year and $13 million in 2003-2004. With these proposed reductions, no additional funding is being provided for the state match for seismic retrofit projects. A letter dated January 22, 2003 was sent to all local agencies that have bridges funded under the Local Bridge Seismic Retrofit Program informing them of the funding changes relating to the state match provided in this program. In the letter, local agencies were reminded that they are still responsible for the maintenance and safety of their bridges and the lack of state funds for this program does not release them from these responsibilities.

In view of increases in retrofit design ground motion requirements for bridges as described in Section 5.1 and improvements in the methodologies of predicting bridge performance during seismic events, the SAB recommends that Caltrans re-screen all of California's approximately 24,000 bridges for risk of collapse during future seismic events using updated screening algorithms. Any additional local agency bridges showing a potential high risk should be added to the Local Bridge Seismic Retrofit Program.

10.5.2 Bay Area Rapid Transit District

Construction of the original Bay Area Rapid Transit (BART) system was completed in 1976 (Bechtel/HNTB 2002). It consists of 72 miles of rapid transit lines, which include 34 stations. Of this total, there are 21miles of subway and twin-bore tunnels (including the 3.6-mile Transbay Tube, and, the 3.2-mile tunnel through the Berkeley Hills), 24 miles of aerial line, and 27 miles of at-grade track. In the 1990s, BART initiated its program to add 30 miles of extensions and nine stations to the original system. Of this total, the 7.8-mile Pittsburg-Antioch Extension opened in 1996, the 14-mile Dublin-Pleasanton Extension opened in 1997, and the 1.7-mile Colma Extension opened in 1996. The San Francisco International Airport Extension, which included four new stations opened in 2003.*

The development of seismic design criteria for the BART East Bay Extensions started in 1988, before the 1989 Loma Prieta earthquake. The goal at that time was to ensure life safety in accordance with the FTA seismic design guidelines. In July 1990, Bay Area Transit Consultants (BATC), under contract to BART, issued its "Preliminary Summary Report: Proposed Seismic Design Criteria for Structural Design of BART Extensions." This report showed that under maximum credible earthquake conditions, aerial structures designed to meet such criteria might suffer substantial damage, but would not collapse.

After the Loma Prieta earthquake, the design criteria for the East Bay Extensions was upgraded to that of functionality, as recommended by BART staff and its Board of Consultants (G. Housner, B. Bolt, and J. Penzien). This upgraded criteria was published in May 1991 in a report prepared by BATC and BART Engineering entitled "Extensions Seismic Criteria." Further upgraded criteria reflecting the latest seismic design requirements was published in February 1998 in a report prepared by BATC and BART Engineering entitled "SFO Extensions Seismic Criteria for Use in Design of the SFO Extension from Daly City to Millbrae."

Several investigations related to the seismic performance of the original BART system were initiated as a result of the 1989 Loma Prieta earthquake. The first of these investigations was reported in May 1991 by International Civil Engineering Consultants, Inc. in its report entitled "Seismic Performance Investigation of the Hayward-BART Elevated Section Instrumented under CSMIP." This investigation studied the seismic performance of a portion of the aerial structure north of the Hayward BART Station, which was instrumented by the then-named California Division of Mines and Geology under its Strong Motion Instrumentation Program (CSMIP). The various findings included: 1.) the girders are strongly coupled by rails in the longitudinal direction, causing them to behave essentially as a unit with almost no relative motions across the joints, 2.) the structural responses in both longitudinal and transverse directions are significantly influenced by soil-structure interaction effects, and 3.) pier base moments predicted by elastic modeling under maximum credible earthquake conditions would

* Information provided in Section 10.5.2 has been taken from a publication entitled "BART System-wide Seismic Vulnerability Study" prepared by Bechtel Infrastructure Corporation, the Bechtel/HNTB team, and issued in June 2002.

be significantly greater than their corresponding capacities under maximum credible earthquake conditions.

The second investigation BART initiated as a result of the Loma Prieta earthquake was conducted by Parsons Brinckerhoff Quade & Douglas, Inc. on the seismic performance of the Transbay Tube. The results of this investigation were published by PBQ&D in November 1991 in its report "Transbay Tube Seismic Joints Post-Earthquake Evaluation." A major finding stated in this report was "the joints will likely remain intact and functional after the next earthquake." However, rehabilitation of the joints was recommended to restore their capacities as close as possible back to their original design capacities.

The Caltrans Office of Earthquake Engineering investigated the seismic performance of the BART aerial structure located at 29th Avenue in Oakland. Its January 2000 report on this investigation, "Caltrans Analysis of the Aerial Structure at 29th Avenue," indicated that the pier footing was adequate. However, the findings of Sverdrup Civil, Inc. and MGE Engineering, Inc., as published in the March 2000 report entitled "Seismic Retrofit Strategy Report—Aerial Structure at 29th Avenue," indicated that to satisfy the functionality requirement, the footings needed to be strengthened.

Caltrans, in cooperation with BART, has performed seismic retrofits on BART aerial structures as follows:

- In 1996, three pier footings were enlarged and thickened with top mat reinforcing, and cast-in-drilled hole (CIDH) piles were added to the structure over Melrose Avenue on the A-Line in Oakland. Also in 1996, two pier footings were enlarged and thickened with top mat reinforcing and CIDH piles were added to the structure over Martinez Avenue on the A-Line in San Leandro.
- In 1997, restrainers were added to girders of the structure over Peralta Blvd. on the A-Line in Fremont; and that same year, five pier footings were enlarged and thickened with top mat reinforcing and CIDH piles were added to the structure over I-680/24 on the C-Line. In addition, a shear key was added to one abutment of this same I-680/24 structure, and its girder diaphragms were strengthened by adding cross-bracing and top and bottom chords.
- In 1998, five pier columns of the structure over I-880 (Cypress Street Viaduct) on the M-Line were replaced with four new support structures to allow the specified clearance for the new freeway lanes.

In September 2000, BART launched a comprehensive BART Seismic Retrofit Program with the goal of strengthening the BART system ahead of a highly probable future earthquake. The seismic vulnerability study of this program, including a seismic risk analysis, represents extensive and detailed engineering and statistical analyses and review by the BART management and staff, BART's general engineering consultant Bechtel/HNTB, the Bechtel/HNTB Design Review Board, G&E Engineering Systems, Inc., an independent Peer Review Panel, Caltrans, and the California Seismic Safety

Commission. While the complete vulnerability study had not been completed at the time this report was prepared, preliminary results indicate elements of the original BART system most susceptible to earthquake damage as follows:

- Aerial structures, including 24 miles of aerial guideway and 15 aerial stations, which, based on computer models, have potential for collapse of the bent. However, such failures have not been observed in past earthquake damage investigations, and the extent of likely damage is uncertain; less extensive damage states would result in limited operability, so that trains could traverse a damaged location at slow speeds.
- The backfill surrounding the Transbay Tube is prone to liquefaction. Assuming a worst case, liquefaction could cause excessive movement of the seismic joints and structural stress that could cause the tube to fail. However, due to the mix of different soils originally used to backfill the tube and changes from sedimentation over the last 30 years, it is impossible to predict definitively how these soils will react. It is possible that, if hydraulic pressure were to be relieved through the backfill, no damage to the tube would result. The criticality of the tube and the uncertainty of the consequences of liquefaction require that the worst-case scenario be considered for this study.
- The Berkeley Hills Tunnel crosses the Hayward fault and would be seriously damaged by any significant offset of the fault at that location.
- Administrative buildings, yard buildings, parking structures, and other buildings are likely to be damaged and possibly unusable following the earthquake.
- Various kinds of equipment (substations, ventilation equipment, etc.), some of which could cause functional outages to train operations if dislodged from their anchorages.
- At-grade and underground trackways and stations could be damaged, but most are not expected to become critical to safety or BART operability.

Various retrofit concepts have been under development with the intent that their implementation would remove, or reduce to acceptable levels, the element deficiencies listed above. Unfortunately, due to lack of funds, this effort had to be nearly terminated in January 2003. Due to the importance of providing seismic safety for BART passengers and preventing a damaging postearthquake impact on the San Francisco region's economy, the Seismic Advisory Board recommends that every effort be made to secure the funds needed to complete the comprehensive BART Seismic Retrofit Program.

10.5.3 Golden Gate Bridge, Highway, and Transportation District

The famous Golden Gate Bridge, which serves as a critical link between San Francisco and Marin County to the north, was constructed during the period 1933-1937.* Since then, a number of projects have been undertaken to preserve, protect, and extend the life of this structure, including the following:

- Because of a great windstorm on December 1, 1951, which threatened the safety of the bridge, a lower lateral bracing system was added to the deck structure during the period 1953-1954 to increase the aerodynamic stability of the bridge.
- During the period 1973-1976, the suspender ropes were replaced due to the discovery of corrosion on the original ropes.
- Following the 1971 San Fernando earthquake, Caltrans issued new retrofit design standards for existing bridges. This led to retrofitting both the San Francisco and Marin approaches to the bridge during the period 1980-1982 to increase their seismic resistance.
- Because of deterioration due to weather exposure, the original reinforced concrete roadway deck and its supporting steel stringers were replaced with a lighter and stronger steel deck during the period 1982-1986.

As a result of the Governor's Board of Inquiry report and Executive Order D-86-90 which followed, the Golden Gate Bridge, Highway, and Transportation District initiated its seismic retrofit program of the Golden Gate Bridge, which has proceeded as follows:

- Immediately following the 1989 Loma Prieta earthquake, the District contracted with T.Y. Lin International to undertake a seismic vulnerability evaluation of the Golden Gate Bridge. The evaluation, completed in November 1990, determined that a major earthquake on a nearby segment of the San Andreas or Hayward fault would cause severe damage to the bridge and could require significant repairs and interruption of traffic.
- Subsequently, the District contracted with T.Y. Lin International and Imbsen Associates, Joint Venture, to perform evaluation studies of seismic retrofit alternatives. The evaluation report, completed in July 1991, provided development and recommendation of specific retrofit measures that are both necessary and sufficient to meet the performance requirements established by the Governor's Board of Inquiry.
- The final seismic retrofit design of the bridge structures started in 1992 and was divided into two parts. The retrofit design of the suspension bridge, north approach viaduct, north anchorage housing, north pylon, and south pier fender and retrofit design for wind stabilization of the suspension bridge was contracted with T.Y. Lin International and Imbsen Associates, Joint Venture. The retrofit design of south approach viaduct, south anchorage housing, arch, and south pylons was contracted with Sverdrup Civil.

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* Information in Section 10.5.3 has been taken from "Highlights, Facts, and Figures," Golden Gate Bridge Highway and Transportation District (GGBHTD,) and a communication from its engineering department.

- The seismic retrofit construction project was divided into three phases. The Seismic Retrofit of North Approach Viaduct started in September 1997 and was completed in September 2001, with a total construction cost of $59.3 million in District funds. Seismic Retrofit of South Approach Structures, with a contract amount of $122.3 million in federal funds, started in June 2001 and is scheduled to be completed in February 2005. Phase III Seismic Retrofit of Suspension Bridge, North Anchorage Housing, North Pylon, and South Pier Fender and Retrofit for Wind Stabilization, with an estimated cost of $160 million, will start when funding is secured.

It is critical to public safety and the economy of the San Francisco Bay Area that the retrofit program for the Golden Gate Bridge be completed as soon as possible.

Section 11

Summary and Review of *The Continuing Challenge* and Actions Taken

The Northridge earthquake of January 17, 1994 provided an opportunity for the Seismic Advisory Board to evaluate the performance of Caltrans bridges, retrofit programs, peer review programs, and technical procedures. The SAB report *The Continuing Challenge* was prepared in 1994 by the Caltrans Seismic Advisory Board as a report to the Director of the California Department of Transportation and covered the following three concerns:

1. Evaluation of changes and developments in Caltrans seismic design criteria and the Highway Bridge Seismic Retrofit Program since Loma Prieta earthquake four years earlier.
2. Findings on the performance of highway bridges in the Northridge earthquake.
3. Improvements needed for Caltrans bridge seismic design and retrofit programs and procedures.

11.1 Damage to State and Interstate Highway Bridges

There were a total of 2,523 state and interstate highway bridges in Los Angeles County that Caltrans has responsibility for maintaining.

The Northridge earthquake caused the collapse of seven highway bridge structures and the consequent disruption of a large portion of the northwest Los Angeles freeway system. Of the seven bridges that collapsed, five had been scheduled for retrofit. Two bridges, the Mission-Gothic Undercrossing and Bull Creek Canyon Channel, both on State Route 118, had been identified as not requiring retrofit. Three of the bridges were designed and built prior to the 1971 San Fernando earthquake; two bridges were designed before 1971, but construction completed after 1971; and two bridges were designed and built a few years after San Fernando, but before Loma Prieta, and not to current standards. Many other bridges in the region of strongest ground shaking sustained damage, but did not collapse and some had to be closed temporarily while repairs were made.

11.2 Damage to Local Government Bridges

11.2.1 Los Angeles City and County

There were a total of about 1,500 bridges maintained by Los Angeles County and about 800 bridges maintained by the City of Los Angeles. Most of these are small, single-span bridges and most were remote from the area of strong ground motion. Only a few of the city and county bridges were significantly damaged and were back in service relatively soon. This report does not cover in detail the damage or repair required for the local government bridges, because Caltrans prime responsibility is for state owned and maintained bridges.

11.2.2 Toll Bridges

The Continuing Challenge report stressed the fact that as of spring, 1994, no construction projects were underway for toll bridges, either in southern California or northern California. The size and complexity of toll bridges makes progress slower, but their importance puts a premium on completion before they are damaged in an earthquake.

This was the case for the East Spans of the San Francisco-Oakland Bay Bridge. At the time of the October 1994 report, hazard analyses were complete for all 11 toll bridges, and vulnerability analyses had been completed for a few. Preparation of retrofit designs had not started for most, and no construction was expected for some time.

11.3 Research and Confirmation Testing for Toll Bridges

In the nine years since *The Continuing Challenge* was published, significant research and confirmation testing has been sponsored by Caltrans that directly affects design of toll bridges seismic retrofit.

Section 12

Review of Improvements in Seismic Design Practice Since 1989

12.1 Improvements for Seismic Design Practice

The following improvements in technical knowledge have contributed greatly to the economy and efficiency of retrofit technology.

- Nonlinear time history analysis tools
- Probabilistic hazard assessment
- Fault surface rupture design considerations
- Constructability review/value engineering
- Seismic safety peer review
- Performance-based design

In the aftermath of the Loma Prieta earthquake, the reliance of Caltrans on independent engineering peer review has been emphasized, often with the addition of external experts both in bridge design and other earthquake engineering topics.

12.2 Review of Technical Improvements in Seismic Design Practice Since 1989

12.2.1 Vision 2000, Performance-Based Seismic Engineering for Buildings (SEAOC, 1995)

This document discusses the inadequacies of the global reduction factor, R, in the seismic design of buildings. A completely rigorous approach to seismic design is described and the assumptions associated with less rigorous approaches are identified. Alternative seismic design criteria are proposed for consideration in future building codes with the most rigorous criteria applicable to large and complex structures and simpler criteria proposed for smaller and regular structures. The Vision 2000 document introduces the concept of performance-based design with acceptance criteria based on the nonlinear response of individual structural components. Approximate and/or simplified procedures (e.g. push-over analyses, capacity spectra, and inelastic demand ratios) are described to represent the nonlinear response.

12.2.2 NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273 and 274, 1997)

These guidelines, which were prepared concurrently with Vision 2000 (and with some of the same personnel), provided guidelines for the seismic rehabilitation of existing buildings using some of the concepts for performance-based design introduced in Vision 2000. Acceptance criteria, based on existing research, are provided for linear static and dynamic analyses as well as for nonlinear dynamic analyses.

The above documents, although developed for the seismic design of buildings, have also provided useful approaches for the seismic design of bridges and have benefitted from advances in bridge design and analysis. The basic premise that the nonlinear (elastic plus inelastic) displacement of a structure can be determined from a linear elastic analysis was introduced by the documents and has subsequently been adopted by Caltrans in the design of Standard bridges. Although this premise has been confirmed for regular integral structures, additional research is required to determine whether it is applicable to non-standard bridges with expansion joints and complex alignments. Additionally, the application of performance-based design in bridges is complicated by the fact that the desired performance objective may be controlled by nonstructural components (e.g., expansion joints or abutment fills). The development of appropriate acceptance criteria relating the structural response to the performance of these critical nonstructural components also requires additional research.

12.2.3 ATC-32, Improved Seismic Design Criteria for California Bridges: Provisional Recommendations

This document was prepared by ATC, under a Caltrans contract, and was intended to present the state-of-the-art in the seismic design of bridges. ATC selected experts in the various disciplines of seismic analysis and design and appointed a Project Engineering Panel to provide guidance and oversight. ATC- 32 documented some of the design and analysis procedures that had been developed during the retrofit of the San Francisco double-deck viaducts and presented recommended revisions to the Caltrans *Bridge Design Specifications* and the related seismic criteria documents. ATC-32 was completed in 1996 and Caltrans has subsequently integrated many of the recommendations into its design provisions.

The National Center for Earthquake Engineering Research (NCEER), established by the National Science Foundation and headquartered at the State University of New York at Buffalo, is a consortium of researchers from numerous disciplines and institutions throughout the U.S. Under two multi-year contracts from the Federal Highway Administration, NCEER sponsored research dedicated to the definition of seismic hazards for new and existing highway bridges (NCEER Projects 112 and 106). Additionally, Project 106 provided for the revision of the FHWA manual for the retrofitting of existing bridges. The research conducted under these contracts contributed to the state-of-the-art in earthquake engineering, particularly in the geotechnical disciplines, regarding the seismic effects on piling and other deep foundations, topographical effects on ground motion, and identifying and mitigating the effects of liquefaction and lateral spreading.

12.2.4 MCEER Highway Project 094

This ongoing project, also funded by FHWA as an extension of Projects 106 and 112, includes the development of a formal loss estimation methodology for highway bridges and the development of seismic design and retrofitting manuals for special bridges. The project is currently at about the midpoint of its 6-year term, and recent presentations indicate that the project researchers need to develop a better definition of what constitutes a "special" bridge and the loss estimation methodology needs to consider directivity in the ground motion and develop more realistic fragility functions for the bridges.

12.2.5 NCHRP Project 12-49

This project, as part of the National Cooperative Highway Research Program, funded a study to develop recommended load and resistance factor design (LRFD) specifications to be incorporated by AASHTO for the seismic design of highway bridges. The study was performed under the joint auspices of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER, formerly NCEER). The recommended design specifications are similar in many respects to those currently in use by Caltrans and provide the basis for eventually incorporating the latest seismic practices into a new AASHTO guide "Specification for Seismic Design of Highway Bridges."

The above projects provide guidelines for the evaluation of structural response and design to resist seismic forces. Since the Loma Prieta earthquake of 1989, analytical tools have been developed to provide a better understanding of the nonlinear response of structures. These developments, supplemented by experimental testing and research, have allowed performance-based design to be implemented in the seismic design of Important bridges in the Toll Bridge Seismic Safety Program. Default acceptance criteria (e.g., allowable strains in concrete or reinforcement) based on available experimental data have been provided for Standard bridges. The default data presumably provide some margin of safety against failure, but additional experimental and analytical research is needed to define the acceptance criteria for the desired performance level for a specified performance objective. As indicated above, defining a desired performance level for a highway bridge is further complicated by the fact that the performance level (e.g., access by emergency vehicles) may be governed by nonstructural components (i.e., expansion joints or settlement of abutment fills).

Section 13

Issues Yet to be Addressed from the State's Perspective

The Governor's Executive Order D-86-90, signed on June 2, 1990 was issued in response to the Governor's Board of Inquiry recommendations on the 1989 Loma Prieta Earthquake in its 1990 report, *Competing Against Time* (Housner et al. 1990). Among other things, the Executive Order encouraged local transportation agencies to review the findings and recommendations of the Board of Inquiry and to adopt policies, goals, and actions similar to those proposed for Caltrans.

13.1 Caltrans Responses to Recommendations of the Board of Inquiry

On January 26, 1994 and again on July 31, 2001 Caltrans submitted a progress report that responded to recommendations contained in *Competing Against Time*.

Item 6 of those Caltrans reports responded to three recommendations concerning issues other than state highway bridges and freeways. The Board of Inquiry had called for action by agencies and independent districts that are responsible for transportation systems, rail systems, highway structures, airports, ports and harbors. It fell short of issuing a mandate to those agencies, but used the admonition they "should" be encouraged. The responses by Caltrans (Caltrans 2001) were as follows:

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

Status on January 26, 1994: It is difficult for Caltrans to determine whether any of these agencies adopted policies and goals or whether they implemented practices to meet them. Caltrans has no authority to comply with the directive.

Status on July 31, 2001: It is difficult for Caltrans to determine whether any of these agencies did adopt the policies and goals or whether they implemented practices to met them. Caltrans has no authority to require these agencies to comply with this directive. We do know that BART finally adopted similar seismic policies as Caltrans.

Recommendation B
Perform comprehensive earthquake vulnerability analysis and evaluation of important transportation structures (e.g., the BART Transbay Tube and Golden Gate Bridge) using state-of-the-art methods in earthquake engineering, and install seismic instrumentation.

Status on January 26, 1994: It is a known fact that the Golden Gate Bridge and Highway Transportation District has conducted a seismic vulnerability analysis of the Bridge and has a consultant preparing vulnerability analysis of the Bridge and has a consultant preparing seismic retrofit plans for the Bridge. Caltrans has no information on seismic instrumentation, however.

Status on July 31, 2001: The Golden Gate Bridge and Highway Transportation District has conducted a seismic vulnerability analysis of the Bridge and had consultants prepare seismic retrofit plans for the Bridge. Construction is underway at the present time.

Caltrans has no information on seismic instrumentation, however.

Recommendation C
Institute independent seismic safety reviews for important structures.

Status on January 26, 1994: It is known that the Golden Gate Bridge and Highway Transportation District has conducted a seismic safety review of the Bridge.

Status on July 31, 2001: It is known that the Golden Gate Bridge and Highway Transportation District conducted a seismic safety review of the bridge. A retrofit design was completed and is in construction.

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

Status on January 26, 1994: It is not known to Caltrans whether any of the agencies has conducted such a program.

Status on July 31, 2001: It is not known to Caltrans whether any of the agencies has conducted such a program.

It is apparent from the above that little has been done since 1994 for non-highway bridges.

13.2 Seismic Safety Commission Report

The Seismic Safety Commission in its report *California at Risk: 1994 Status Report* (SSC 94-01) outlined 42 initiatives that encourage hazard reduction in five different categories and set goals for agencies to act on in the next five years (SSC 91-08). This report (in advisory capacity to the Governor) was issued in 1991 in response to the California Earthquake Hazards Reduction Act of 1986. Category I of that report listed 19 initiatives under Existing Vulnerable Facilities.

Of these, Initiative 1.16 called on Caltrans to lead an effort entitled, "Improve Earthquake Performance of Transportation Structures" and pointed out that hundreds of older highway and railroad bridges in California are in need of seismic retrofit. Initiative 1.17 was entitled "Improve Earthquake Performance of Offshore Oil Facilities," and called on the State Lands Commission to lead this effort and proposed extending the service life of offshore hydrocarbon production facilities in state and federal waters.

Initiative 1.18 called for the Seismic Safety Commission to lead and was entitled, "Improve Performance of Transportation Systems and Infrastructure." It declared that commerce depends on a number of transportation facilities at risk during earthquakes, including port facilities, airports, railroads, pipelines, and producers. Acceptable performance of these systems should be determined.

In the SSC's *California at Risk: 1994 Status Report*, Caltrans was praised for the substantial progress it had made in decreasing earthquake risk to the state's highways. Caltrans has decreased the earthquake vulnerability of the state's vital transportation corridors by retrofitting several hundred bridges.

No real progress was noted for Initiative No. 1.18 to "Improve Earthquake Performance of Transportation Infrastructure." The lead agency was then listed as Caltrans. Three items were described in the summary:

- Item 1. Identify responsible entities.
- Item 2. Describe appropriate performance levels.
- Item 3. Recommend policies and actions.

The lack of progress is understandable because *California at Risk* only pointed out the risks and made recommendations to mitigate them without suggestions of how it should be accomplished. Very little action on their recommendations has occurred during the 10 years since the SSC report was issued.

In 1995 the Seismic Safety Commission published an exhaustive Report to Governor Pete Wilson entitled, *Turning Loss to Gain* describing the damage to the state's infrastructure and economy from the Northridge earthquake of 1994 (SSC 95-01). The commission offered some 168 initiatives in the form of recommendations to the Governor on how to reduce the earthquake risk in California. Chapter IV, "Achieving, Seismic Safety in Lifelines" included recommendations on freeway bridges, railroads, natural-gas supplies, electrical utilities, water supply, communications, and dams.

Chapter IV of the report opened with the following *statement:*

> Lifeline systems are, in many ways more vulnerable than buildings or other structures. Because a system is typically spread over a large area, it is susceptible to a wide range of earthquake hazards. Different parts of the system can experience very different levels of shaking or ground deformation in the same event. The performance of a system is tied to the weakest of its hundreds or thousands of components. In addition, because many lifelines are buried, damage is difficult to detect and repair, particularly when several components of the same system are damaged.
>
> The Commission believes that the state and local agencies and the private sector can take a number of additional actions to improve reliability and reduce the vulnerability of lifelines to earthquakes.

The report goes on to itemize the damages to lifelines in the Northridge earthquake beginning with freeway bridges. Finally, in Chapter VI, the report states:

> The previous 167 recommendations cannot make California safer from earthquakes unless state and local governments, businesses, and individuals start to pay more attention to reducing earthquake risks.

The *Turning Loss Into Gain* report provides a prescription for giving seismic safety a level of priority consistent with the enormity of California's earthquake threat and a list of items "for immediate action" by the Governor and Legislature to achieve improvement in reducing earthquake risk in California.

13.3 Current Actions by Seismic Safety Commission

The Seismic Safety Commission still states its goal as "To significantly reduce statewide seismic hazards by the end of the century." But now that the century has changed with only some of the issues addressed, the SSC has embarked on a new program entitled *California Earthquake Loss Reduction Plan: 2002-2006* (SSC 02-02). The Utilities and Transportation Element (there are eleven elements) has the following stated objective:

> To ensure that all public and private utilities and transportation systems can withstand earthquakes to the degree that they will be able to: 1) provide protection of life; 2) limit damage to property, and 3) provide for the resumption of system functions as soon as practicable. The intent of this objective is to limit the impact to only short-term interruptions, with minimal life loss and economic disruption to the affected regions.

Under this program element, the Seismic Safety Commission monitors state agencies. The SSC also sponsors and closely follows legislation reflecting on or improving the state's loss reduction plan.

As an example, in its meeting minutes of September 2002, the SSC reported on the BART Seismic Upgrade Peer Review (SSC 2002). It was noted that BART approached the SSC in 2000 for help and support in planning its major system-wide retrofit project. In response, the SSC recommended that BART perform a detailed vulnerability study and develop a risk-based plan to strengthen the system. The Commission suggested using a peer review team from the Pacific Earthquake Engineering Research (PEER) Center for that purpose. BART welcomed the Peer Review Team's recommendations and it is seeking funding to implement the plan. (A bond issue was defeated by the state's voters in the November 2002 election.)

There are approximately 50 state agencies that could be or should be involved in the plan to reduce earthquake loss. Unfortunately, each agency has current needs that are perceived as more urgent than the earthquake loss plan by its administrators; therefore action gets deferred. The Seismic Safety Commission in its advisory role can only recommend and monitor.

Caltrans has a record of liaison with the Seismic Safety Commission and reporting routinely on its progress. The Chief of the Office of Earthquake Engineering has represented Caltrans at meetings of the SSC.

13.4 Other Transportation Structures

In *California at Risk: 1994 Status Report*, the Seismic Safety Commission lists seven major issues: 1.12 through 1.18. It addressed each issue in some detail. However, only one of this issues (Issue 1.16) addressed earthquake performance of transportation structures, for which Caltrans has direct responsibility. The other issues addressed were dams, electric and gas utilities, river delta levees, sewage and water treatment systems, offshore oil facilities, seaports, airports, and railroad systems.

Table 13-1. *Major transportation facility operators (excerpted from Table II, page 13 of SSC 94-01).*

Airports	Ports	Bridges	Rail
Burbank	Humboldt Bay Harbor	Golden Gate Bridge, Highway, Transportation District	Rail–Freight
Eureka/Arcata	Port of Long Beach		Burlington Northern
Fresno	Port of Los Angeles		Santa Fe Transportation Co.
Long Beach	Port of Oakland		Southern Pacific
Los Angeles International	Port of Hueneme		Union Pacific
Monterey	Port of Redwood City		
Oakland	Port of Richmond		Rail–Passenger
Ontario International	Port of Sacramento		Amtrak
Orange County	Port of San Diego		BART
Sacramento Metropolitan	Port of San Francisco		Cal Train
San Diego	Port of Stockton		L.A. County Trans.
San Francisco International	Encinal Terminals (Alameda)		Sacramento Regional
San Jose			San Diego Metro
Santa Barbara			San Francisco Municipal Railway
			Santa Clara County Trans.

Under Initiative 1.18, "Other Transportation Systems," the SSC report states that these systems are also critical to the state's interest, since they may be seismically vulnerable. The organizations listed are largely independent of state regulations on earthquake performance and their standards and practice are largely unknown to the state government. Caltrans should take the lead in determining whether these facilities can withstand earthquakes by contacting the non-state entities, such as those listed in Table 13-1, to determine what actions they have taken to reduce and manage seismic risk. This would place Caltrans in an entirely new and complicated role as the state overseer on all non-state transportation systems.

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Attachments

1. **Governors Executive Order D-86-90**
2. **Recommendations and Findings from Past Earthquake Reports**
3. **Membership of Board with Brief Resumes**

Attachment 1

Governor's Executive Order D-86-90

The following is the text of Executive Department State of California Executive Order D-86-90 signed on June 2, 1990 in response to the Report of the Board of Inquiry on the Loma Prieta earthquake of 1989 report and recommendations.

WHEREAS, on October 17, 1989 a major earthquake occurred in Northern California, causing deaths, injuries, and widespread damage to transportation facilities and other structures; and

WHEREAS, an independent Board of Inquiry was formed in November 1989 to investigate the reasons for the collapse of transportation structures and to recommend actions to reduce the danger of tragic structural failures in future earthquakes; and

WHEREAS, the Board of Inquiry found that there is a high probability that one or more major earthquakes will strike heavily populated areas in Northern and Southern California in the future; and

WHEREAS, California's state of earthquake readiness needs improvement to better protect the public safety and our economy from potentially serious impacts of future earthquakes;

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 Statues of the State of California, do hereby issue this order, to become effective immediately:

1. It is the policy of the State of California that seismic safety shall be given priority consideration in the allocation of resources for transportation construction projects, and in the design and construction of all state structures, including transportation structures and public buildings.
2. The Director of the Department of Transportation shall prepare a detailed action plan to ensure that all transportation structures maintained by the State are safe from collapse in the event of an earthquake and that vital transportation links are designed to maintain their function following an earthquake. The plan should include a priority listing of transportation structures that will be scheduled for seismic retrofit. The Director shall transmit this action plan to the Governor by August 31, 1990.
3. The Director of the Department of Transportation shall establish a formal process whereby the Department seeks and obtains the advise of external experts in establishing seismic safety policies, standards, and technical practices; and for seismic safety reviews of plans for construction or retrofit of complex structures. The Director shall transmit a summary of this process to the Governor by August 31, 1990.

4. The Director of the Department of Transportation shall assign a high priority to development of a program of basic and problem-focused research on earthquake engineering issues, to include comprehensive earthquake vulnerability evaluations of important transportation structures and a program for placing seismic activity monitoring instruments on transportation structures. The Director shall transmit a description of the research program to the Governor by August 31, 1990.
5. Local transportation agencies and districts are encouraged to review the findings and recommendations of the Board of Inquiry on the 1989 Loma Prieta earthquake and to adopt policies, goals, and actions similar to those proposed for Caltrans.
6. The Director of the Department of General Services shall prepare a detailed action plan to ensure that all facilities maintained or operated by the State are safe from significant failure in the event of an earthquake and that important structures are designed to maintain their function following an earthquake. The plan should include a priority listing of facilities that will be scheduled for seismic retrofit. The plan shall further propose measures by which the state agencies construction new facilities or retrofitting existing facilities would:
    a. be governed by the provisions of a generally accepted earthquake resistant code for new construction;
    b. secure structural safety review and approval from the Office of the State Architect;
    c. seek independent review of structural and engineering plans and details for those projects which employ new or unique construction technologies; and
    d. have independent inspections of construction to insure compliance with plans and specifications.

The Director shall transmit the plan to the Governor by August 31, 1990.

7. The Department of General Services shall, when negotiating leases of facilities for use by state employees or the public, consider the seismic condition of the facilities and shall initiate leases only for those facilities that demonstrate adequate seismic safety.
8. The Seismic Safety Commission shall review state agencies' actions in response to this executive order and the recommendations of the final report of the Board of Inquiry and provide a report to the Governor on the adequacy and status of actions taken by December 1, 1990.

9. The University of California and the California State University shall give priority consideration to seismic safety in the allocation of resources available for construction projects. The University of California and the California State University shall prepare and transmit to the Governor by August 31, 1990 a description of their plans to increase seismic safety at facilities they maintain or operate.

IN WITNESS WHEREOF I have hereunto set my hand and caused the Great Seal of the State of California to be affixed this 2nd day of June 1990.

SEAL	George Deukmejian Governor of California
ATTEST:	March Fong Eu Secretary of State

Attachment 2

Recommendations and Findings From Past Earthquake Reports

Competing Against Time

In November 1989 Governor George Deukmejian of California 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 also 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 (Executive Order D-83-89). The Board issued its report *Competing Against Time* on May 31, 1990 (Housner et al. 1990). This section reviews the findings and recommendations of the Board.

George W. Housner was appointed by the Governor as Chairman of the Board, Joseph Penzien served as Vice Chairman, and its members were Mihran S. Agbabian, Christopher Arnold, Lemoine V. Dickerson, Jr., Eric Elsesser, I. M. Idriss, Paul C. Jennings, Walter Podolny, Jr., Alexander C. Scordelis, and Robert E. Wallace. John F. Hall served as Technical Secretary, and Ben Williams served as Administrative Officer in support of the Board; Charles C. Thiel Jr. served as the Technical Writer and Editor of the Board's Report.

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 testimony at those meetings. In addition to these seven public meetings, Board members toured the Cypress test structure and several of the damaged San Francisco structures. An Annotated Bibliography in the Report contains a comprehensive listing of the materials that were sent or made available to all of the Board members for their use.

Central to the Board's process was determination of what occurred during the earthquake and why. These findings formed the basis for the eight recommendations made by the Board of Inquiry (see Section 10 for list of eight Recommendations). The 52 specific findings of the Board are organized under the following general headings and reproduced below. The full report, *Competing Against Time*, (Housner et al. 1990), presents a discussion that gives the rationale for each finding and the technical information on which they were based.

1. 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.

2. The Loma Prieta earthquake was anticipated by the Working Group on California Earthquake Probabilities, but not with high enough probability of occurrence (30 percent in the next 30 years) or confidence in the forecast (low) to have caused Caltrans or others to respond directly to this forecast.
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.
4. Soft ground on the border of the San Francisco Bay amplified ground motions more than anticipated by current codes.
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.

2. 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.
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.
8. Historically, the fiscal environment at Caltrans has inhibited giving the level of attention to seismic problems at the level required.
9. The Board finds that Caltrans has the reputation of being the best transportation agency among the States and a leader in bridge design.
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.
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.
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.

3. 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.

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.
15. Caltrans bridge seismic design codes have improved substantially since the 1971 San Fernando earthquake, but have not been subjected to independent review.
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.
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.
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 earthquake failure.
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.
20. Caltrans has implemented a number of actions to improve seismic design, including an independent review process for major projects.

4. Findings on Bay Bridge 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.
22. The Bay Bridge was designed for 10 percent g earthquake accelerations, comparable to the levels specified in the 1930 Uniform Building Code for buildings.
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.
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.
25. There is no evidence that foundation failure contributed to the failure of the Bay Bridge span.

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.
27. The structural steps taken to repair the Bay Bridge appear to be appropriate for the short-term.
28. No engineering assessment of the dynamic, seismic performance of the Bay Bridge has ever been made.
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.

5. 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.
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.
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.
33. No evidence was presented to the Board that the foundation failed or that foundation problems contributed to the Cypress Viaduct 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.

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

6. Findings on San Francisco Freeway Viaducts

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

7. 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.
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. Such restrainers are not generally sufficient to prevent collapse under strong earthquake shaking.
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.
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.
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.

45. An evaluation of the current Caltrans seismic retrofit program indicated 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 State-wide, 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.
46. There are no widely accepted technical standards for seismic retrofit of bridges.

8. 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.
48. Independent, technical review is essential to achieve consistent excellence in civil engineering design and construction.
49. The registration of professional engineers with a specialty in bridge design is not warranted.
50. Loss of life from and damage to currently existing substandard structures will dominate the impacts suffered in future California earthquakes.
51. Many structures are not subject to seismic codes or to review by an independent third party before construction.
52. Many State-owned structures are seismically substandard and many known hazardous conditions have not been addressed.

The Continuing Challenge

The January 17, 1994 Northridge earthquake and its damage to freeway structures provided an opportunity for the Seismic Advisory Board (SAB) to evaluate the performance of Caltrans bridges, retrofit programs, peer review programs, and technical procedures. The SAB undertook this as consistent with its responsibility to provide continued, focused evaluations of Caltrans seismic policy and technical procedures. At the time of this assessment the Board was composed of George. W. Housner, Chairman, Joseph Penzien, Vice-Chairman, Bruce A. Bolt, Nicholas F. Forell, I. M. Idriss, Joseph P. Nicoletti, Alexander C. Scordelis, and Frieder Seible; Charles Thiel served as editor for the report. The report, *The Continuing Challenge: The Northridge Earthquake of January 17, 1994*, was published in October 1994 (SAB 1994)

The Seismic Advisory Board:

1. Evaluated the past four years of changes and developments in seismic design criteria and the highway bridge retrofit program.
2. Summarized Board findings on the performance of highway bridges in the Northridge earthquake.
3. Recommended improvements to Caltrans bridge seismic design and retrofit programs and procedures.

The Seismic Advisory Board bases the following findings and recommendations on its analysis and review of:

- Northridge earthquake's impacts on transportation structures.
- Caltrans retrofit program.
- Caltrans response to the recommendations contained in Competing Against Time.
- Directions given by Governor Deukmejian's Executive Order D-86-90, dated June 2, 1990.
- Requirements of Senate Bills 36X and 2104.
- The SAB's report The Continuing Challenge [Housner et al. 1994] gives detailed findings and recommendations.

The Seismic Advisory Board recommended that the indicated actions be undertaken on a priority basis in the following areas:

- Bridge performance in the Northridge earthquake
- Retrofit program
- Design
- Caltrans management actions
- State action

Bridge Performance in the Northridge Earthquake

1. ***Finding:*** Caltrans has 12,176 state bridges and of these 9,206 were designed prior to the engineering impact of the 1971 San Fernando earthquake. At this time, knowledge of destructive earthquakes and the seismic performance of structures was in an undeveloped state so that bridges designed prior to the San Fernando earthquake were not up to current standards of seismic design and it was known since the San Fernando, Whittier, and Loma Prieta earthquakes that some of these structures could not survive intense ground shaking. Examples are the Nimitz Freeway double-deck viaduct that collapsed in the 1989 Loma Prieta earthquake and the bridges that collapsed in the Northridge earthquake.
2. ***Finding:*** Damages observed in the Northridge earthquake are, in the main, consistent with those observed in the 1989

Loma Prieta, 1987 Whittier Narrows, and 1971 San Fernando earthquakes.

3. ***Finding:*** The Northridge earthquake provided a valuable test for Caltrans design procedures in high-intensity, moderate magnitude earthquakes, but did not constitute a test of their behavior in the larger, long-duration earthquakes that are expected to occur in the future.

4. ***Finding:*** Of the seven bridges that collapsed, five had been identified and scheduled for seismic retrofit. Two, the Mission & Gothic Undercrossing and the Bull Creek Canyon Channel Undercrossing on State Route 118, had been evaluated as not high-risk and were not scheduled for retrofit.

   Recommendation: Caltrans should evaluate those bridges that were not included in the first retrofit group to determine if they require retrofitting. The evaluation should be performed with the essential objective of collapse avoidance in all earthquakes.

5. ***Finding:*** The performance of recently retrofitted bridges in the Northridge earthquake appears to be acceptable. The evolving post-Loma Prieta earthquake design and retrofitting practices used by Caltrans appear to be sound. No significant damage has been reported to the 60 bridges retrofitted by Caltrans in the region of strong shaking since the start of the post-1987 retrofit program. Prior to 1987, the retrofit approach was to use expansion joint restrainers only. Performance of joint restrainers in the Northridge earthquake was mixed. While retrofitted bridge performance in this event was acceptable, evaluation of the expected performance of these bridges in other earthquakes with greater durations may reveal opportunities for improvement.

   Recommendation: A thorough study of the performance of bridges in the Northridge earthquake should be conducted to determine if changes in Caltrans design practices and priority setting procedures are needed. This should be completed through in-house and independent, external studies, as appropriate. Bridges of both concrete and steel should be studied.

6. ***Finding:*** The public can have confidence in the seismic safety of the Northridge earthquake replacement structures because they are being well designed and peer reviewed.

Retrofit Program

7. ***Finding:*** Caltrans has made acceptable progress in implementing the retrofit program of single-column-bent bridges, with construction either begun or completed on 100 percent of identified bridges. In addition to retrofitting the single-column-bents the program includes retrofitting the abutments and footings as needed. For the multiple-column-bents, bridges, the retrofit program has been completed for only about 7 percent of projects. It has made slower progress on toll bridges,

where vulnerability studies are only now being initiated on some, and construction is not underway on any.

Recommendation: Caltrans should identify the most hazardous highway bridges in the State and fully retrofit them as quickly as practical, instead of approaching the retrofit programs by category of structures.

Recommendation: More emphasis must be given to starting toll bridge retrofit construction projects on as rapid a schedule as practical.

8. ***Finding:*** The priority setting process used by Caltrans, and as reviewed by the Seismic Advisory Board, involves classifying structures by vulnerability, seismic hazard, and impact on the community. Each category has several elements, some of which do not now appear to be weighted appropriately (for example, soil conditions at the site and the system response of interconnected bridges, such as the sequence of bridges on the Santa Monica Freeway). The present process yields priority lists determined by calculations that do not take into account all important factors affecting seismic safety.

   Recommendation: The Caltrans prioritizing procedure should be reviewed and modified based on current understanding. Attention should be given to the quality of information used in the process, including the presence of nonductile columns, variable soil conditions, and the effect that a series of bridges has on the vulnerability of a freeway as an interconnected system. Other characteristics and their weightings should also be re-examined.

Design

9. ***Finding:*** Caltrans design procedures have two performance categories: important and common. The performance objective for important bridges is to have full access available to normal traffic almost immediately following a major earthquake. The performance criteria for all common bridges in a major earthquake are to avoid collapse, but to allow significant damage and limited service. While any of three characteristics—secondary safety, economic impact or emergency use—can lead to classification as "important," there is some ambiguity in the specific characteristics that make a bridge important. The public's response to the Northridge earthquake suggests that more bridges should be classified as important than the current procedure yields.

   Recommendation: Caltrans should reconsider and broaden the definition of an *important* structure and the appropriate performance objectives for both important and common bridge categories. Concurrently, the acceptance criteria, or limit states, leading to each performance objective should also be defined.

10. ***Finding:*** The Northridge earthquake occurred on a previously unidentified blind thrust fault, a type of fault that does not have a surface trace. The possibility of blind-thrust earthquakes was well recognized by both the technical community

and Caltrans. The Northridge earthquake produced ground motions that were high, but within the range considered possible. With few exceptions, vertical accelerations were not unusually high compared with horizontal accelerations.

Recommendation: Future seismic hazard assessments should consider the likelihood of blind thrust faults.

11. ***Finding:*** The duration of the strong velocity pulse observed in near field time history recordings during the Northridge earthquake once again affirms its importance to design. It occurs at sites near fault ruptures and above thrust faults. The possibility of a velocity pulse at a site should be given consideration for near field sites in the design of bridges, especially when assessing nonlinear response.

    Recommendation: The Caltrans bridge design procedures should be assessed, and revised as required, to determine if they adequately reflect the structural demands caused by velocity pulses.

12. ***Finding:*** The seismic hazard used in the design of common bridges is based only on deterministic evaluations for the maximum earthquakes that can occur throughout the state as prepared by the California Geological Survey (CGS). There is some debate as to how these earthquakes and the faults on which they occur should be selected and what attenuation relationship should be used to determine the best estimate of ground motions at a site. The current map only reflects mean peak ground motion estimates; it does not include duration effects or velocity pulses, both of which may be important for common bridge design.

    Recommendation: Caltrans should reconsider the technical assumptions leading to the deterministic map and prepare a new one to reflect current understanding of both seismic hazard and the way in which these values are used in bridge design.

13. ***Finding:*** Caltrans has several hundred steel girder bridges in California. A number of these in the San Fernando Valley area were subjected to strong shaking and sustained severe damage to the end bearings and to the bearing supports. None of these bridges collapsed but at the end of the earthquake they were in a potentially hazardous condition.

    Recommendation: Caltrans should investigate the support systems for steel girder bridges and strengthen them as required.

14. ***Finding:*** Unusual damage was reported to some steel girder bridges. At this writing, two skew bridges have been identified in the region of strong shaking as having cracking in girder webs near welded stiffener plates.

    Recommendation: Caltrans should very carefully check all steel bridges and elements in the region of strong shaking to determine if there has been damage. Bridges outside this area throughout the state should be checked for the possibility of having cracks caused by fatigue.

Caltrans Management Actions

15. ***Finding:*** Caltrans has followed the directions of the Governor based on *Competing Against Time* and the directions of the Governor's Executive Order D-86-90. Administratively, and in practice, Caltrans is committed to producing seismically safe transportation structures.

    Recommendation: Caltrans should continue its commitment to improving the seismic safety of the state's highway bridges.

16. ***Finding:*** Peer review of the design of new and retrofit bridges has been implemented for complex structures. Peer review is not being conducted for the more prevalent common types.

    Recommendation: The scope of projects that are peer reviewed should be extended to include a few representative projects for the more common, prevalent types of structures to validate the design and/or retrofit approach.

17. ***Finding:*** There is considerable variation in how peer review has been implemented for different structures.

    Recommendation: Peer review should be standardized in terms of: 1) which bridges are to be scrutinized; 2) the scheduling of the review to allow designers time to modify the design in response to reviewer comments; and, 3) how complete the peer review should be, ranging from the initial strategy and type selection to the final seismic design detailing. The specific terms of content and format should *not* be standardized—they must be project-specific.

18. ***Finding:*** Strong motion records were obtained from only six bridges located 14 to 115 miles from the epicenter. None of the bridges that collapsed or had substantial damage were instrumented, thus denying the opportunity to evaluate the effectiveness of design and analysis procedures by comparison with actual response.

    Recommendation: Both Caltrans and the California Strong Motion Instrumentation Program must make a greater commitment to installing instruments on bridges, especially toll bridges. Engineers must have recordings from bridges and their sites subjected to high-level ground motions to advance the state of the art in bridge design and analysis.

State Action

19. ***Finding:*** The basic and applied research findings and knowledge that have allowed the development of improved seismic design procedures and practices for bridges have come from research on all types of structure and conditions. The continued development of effective seismic design and retrofit procedures for bridges will depend on knowledge generated in many areas of earthquake engineering.

    Recommendation: Caltrans should continue its vigorous program of research and development for bridges.

20. ***Finding:*** Budgetary, administrative and personnel constraints are the primary reasons why the Caltrans hazardous-bridge retrofit program had not accomplished as much as desirable prior to the Northridge earthquake. In the past, limitations on budget and personnel were the principal drawbacks. Now the issues are: 1) the number of people assigned and their skill levels; 2) the ability of management to contract with qualified engineers to develop designs; and, 3) the ability to initiate construction contracts. Caltrans is working near the limit of what can be realistically done with their current personnel levels and procurement limitations.

    Recommendation: If the public wants safer bridges faster than at the current pace, then it will have to provide greater resources, including both administrative and personnel needs, and resolve the legislative, legal, and administrative impediments to implementing retrofit projects quickly.

21. ***Finding:*** The two collapsed bridges on the Santa Monica Freeway (I-10) were removed, new spans were constructed, and normal traffic flow was established by May 20. This rapid replacement of the damaged bridges was accomplished by means of special contractual arrangements that provided incentives for completion ahead of schedule and disincentives for completion behind schedule.

Attachment 3

Membership of Board with Brief Resumes

The Caltrans Seismic Review Board (SAB) prepared this report. The individuals who serve on the SAB were appointed by the Director of Caltrans for their technical knowledge and expertise in earthquake engineering and allied sciences. The SAB was established in 1990. The founding chairman was Professor George Housner of the California Institute of Technology, who also chaired the Governor's Board of Inquiry on the Loma Prieta Earthquake. He served as the chairperson for the development of the two previous reports in this series: *Competing Against Time* and *The Continuing Challenge*.

The SAB has seven distinguished members. The following abbreviated resumes are intended to indicate the qualifications and experience they bring to the task of preparing this report.

Joseph Penzien, Chairman SAB

Frieder Seible, Vice-Chairman SAB

Bruce A. Bolt

I.M. Idriss

Joseph P. Nicoletti

F. Robert Preece

James E. Roberts

Dr. Charles Thiel assisted the SAB in the formulation and completion of this report. He undertook the task of organizing and editing the contributions of Board members into a coherent form and contributed portions of the text.

Joseph Penzien

Dr. Joseph Penzien is Chair of the Caltrans Seismic Advisory Board. Dr. Penzien received his Bachelor of Science degree from the University of Washington in 1945 and his Doctor of Science degree from the Massachusetts Institute of Technology in 1950, where his research concerned blast effects on structures. Dr. Penzien joined the civil engineering faculty at the University of California at Berkeley where he continued his research on the dynamics of structures, and where his principal focus shifted from blast effects to seismic and wind effects. In 1968, he became the founding director of the U.C. Berkeley Earthquake Engineering Research Center (EERC) with responsibility for its research and laboratory development programs, including design of the earthquake simulator (shaking table) facility at the U.C. Berkeley Richmond Field Station. Dr. Penzien retired from U.C. Berkeley in 1988, but continued his professional activities with Eastern International Engineers in Taipei, Taiwan. In 1990, along with two partners, he founded International Civil Engineering Consultants, Inc. in Berkeley, California. His work in private practice has focused on setting seismic design criteria and guiding seismic response and performance evaluations of important engineered facilities including nuclear power plants, highrise buildings, large arch dams, transportation structures, and tunnels. Dr. Penzien served as Vice-Chairman of Governor George Deukmejian's Board of Inquiry on the Loma Prieta Earthquake. He is the author of many papers on earthquake engineering topics, and a co-author of a widely referenced text on structural dynamics. He has been a member of numerous peer review panels and consulting boards on projects including U.S.-Japan cooperative research projects, Golden Gate Bridge retrofit, Bay Area Rapid Transit (BART) extension projects, BART retrofit, MUNI Metro Turnaround project, Applied Technology Council ATC-6 and ATC-32 projects, and FHWA-sponsored bridge research projects at the National Center for Earthquake Engineering Research (NCEER) and Multidisciplinary Center for Earthquake Engineering Research (MCEER) at the State University of New York at Buffalo. Dr. Penzien has been a member of the National Academy of Engineering for 26 years.

Frieder Seible

Dr. Freider Seible is vice-chair of the Caltrans Seismic Advisory Board. Dr. Seible is the Dean of the Jacobs School of Engineering, University of California, San Diego, and holds the Eric and Johanna Reissner Chair in Applied Mechanics and Structural Engineering and the Walter J. Zable Chair in the Jacobs School of Engineering. He developed and directs the Charles Lee Powell Structural Research Laboratories, which serve as a worldwide resource for full-scale testing and analysis of structures. He has published more than 500 papers and technical reports primarily related to seismic design of bridges and buildings, and has served on or led many committees on security, reconstruction and retrofit of buildings, bridges, and transportation infrastructure. He is a member of a federal Blue Ribbon Panel on Bridge and Tunnel Security; a member of the National Research Council Panel for Building and Fire Research Laboratory, National Institute of Standards and Technology (NIST); and is chair of the Governing Board of the California Institute of Telecommunications and Information Technology. Seible received a Dpl. Ing. from the University of Stuttgart, a Masters of Science from the University of Calgary, and a Ph.D. from the University of California, Berkeley, all in civil engineering. Seible is a member of the National Academy of Engineering, and a recipient of the CERF Charles Pankow Award for Innovation.

Bruce A. Bolt

Dr. Bruce Bolt was born in Australia in 1930. He was schooled at the University of Sydney for an honors degree in applied mathematics, and subsequently appointed to the faculty of the Mathematics Department. After completion of a Ph.D. in elastic wave theory, he won a Fulbright scholarship to Lamont Geological Observatory at Columbia University in 1960 and to Cambridge University (U.K.) in 1961. There he met Perry Byerly, Professor of Seismology at the University of California at Berkeley, which led to an invitation to a chair in seismology at U.C. Berkeley in 1963. At U.C. Berkeley Dr. Bolt was the Director of the University of California Seismographic Stations for 28 years; Chairman of the U.C. Berkeley Academic Senate in 1992-1993; and recipient of the University Citation in 1992. He has been Chairman of the California Seismic Safety Commission, President of the California Academy of Sciences (its medalist in 1989) and President of the Seismological Society of America. He was elected to: the National Academy of Engineering in 1978, Overseas Fellow of Churchill College, Cambridge University in 1980, and Associate of the Royal Astronomical Society of London in 1987. Dr. Bolt received the Alfred Alquist Medal of the California Earthquake Safety Foundation in 1995. He has made many post-earthquake investigations and has written six and edited eight textbooks on earthquakes, geology and computers, and other topics. He has published over 200 research papers (1955-2003). Dr. Bolt is now a Professor Emeritus and does engineering consulting.

I.M. Idriss

Dr. I.M. Idriss is a Professor in the Department of Civil Engineering and Environmental Engineering at the University of California at Davis (UCD). He completed his Ph.D. degree in 1966 at the University of California at Berkeley. His areas of teaching, research and practice are: geotechnical earthquake engineering; soil mechanics and foundation engineering; earthfill and rockfill dam engineering; and numerical modeling. He has been involved in many post-earthquake geotechnical investigations beginning with the 1964 Alaska and Niigata earthquakes. He has developed or co-developed many of the currently used procedures for evaluating the behavior of soil sites and soil structures during earthquakes. Dr. Idriss served as a member of Governor George Deukmejian's Board of Inquiry on the Loma Prieta Earthquake, and has been a Member of the SAB since its inception, was Chairman of Caltrans External Research Committee from 1991-1999, and was a member of the Caltrans Peer Review Panel for Toll Bridges in Northern and Southern California. Since 1998, Dr. Idriss has been a member of Caltrans Peer Review Panel for the design and construction of the New East Spans of the San Francisco-Oakland Bay Bridge. Dr. Idriss has received many awards and honors over the past 35 years, including election to the U.S. National Academy of Engineering in 1989, receipt of the first H. Bolton Seed Medal from ASCE in 1995, and the distinguished scholarly public service award from the University of California at Davis in 1999.

Joseph Nicoletti

Mr. Joseph Nicoletti graduated from the University of California with a B.S. degree in Civil Engineering in 1943. Following service in World War II, he joined the structural engineering staff of John A. Blume in 1947 and became an officer in the firm when it incorporated in 1957. He was the Senior Project Engineer for the firm from 1957 to 1971, when it merged with the URS Corporation. He was the Chief Engineer from 1971 to 1983, when he retired as President. He was an independent consultant until the Loma Prieta earthquake of 1989, when he rejoined URS as a Senior Consultant. Mr. Nicoletti worked on diverse engineering projects including the design of piers, wharves, and other waterfront structures in the Bay Area, Pacific islands, South America, and Saudi Arabia; a commercial port for the government of Guam; aircraft hangars for the Navy and Air Force; the Embarcadero Center office buildings and the Hyatt Regency Hotel in San Francisco; the Bonaventure Hotel in Los Angeles, and the Diablo Canyon Nuclear Power Plant. He was designer of record for the retrofit of the California State Capitol building. Mr. Nicoletti has served on several peer review panels for the repair and/or replacement of double deck viaducts damaged in the Loma Prieta earthquake. He served as chair of the Engineering and Design Advisory Panel (EDAP) of the Metropolitan Transportation Commission to select the design of the replacement bridge for the East Spans of the San Francisco Bay Bridge and later chaired the seismic safety peer review panel for design of the replacement bridge.

F. Robert Preece

Mr. F. Robert Preece received his Bachelors degree for the University of Nevada, Reno and a Masters degree from Stanford University, both in civil engineering. Mr. Preece spent nine years with Bethlehem Steel Company, rising to District Engineer for Northern California. He entered private practice 1956 with the engineering/architectural firm of Simpson, Stratta & Associates in San Francisco, and became licensed as a structural engineer. He began to actively incorporate earthquake engineering into design of buildings and industrial structures. In 1962, Mr. Preece became Vice President of Testing Engineers, Inc. and supervised inspection and testing of construction materials and processes on major projects throughout the Bay Area. In 1978, he started his own consulting practice, Preece, Goudie & Associates. Mr. Preece retired from active practice in 2000. Throughout his career he has been active in visiting foreign and domestic sites of major earthquakes and analyzing performance of structures. He served as president of the Structural Engineering Association of California, the Consulting Engineers Association, the Applied Technology Council, and was vice-president of the Earthquake Engineering Research Institute. He served six years on the board of the Building Seismic Safety Council. He has written and published extensively in technical publications on the design and performance of materials and what we have learned from building performance in earthquakes.

James E. Roberts

Mr. James E. Roberts is a 50-year veteran of the California Department of Transportation with a comprehensive background in planning, budgeting, design, construction, administration, as well as equipment acquisition, maintenance and management. He received a Bachelor of Science in Civil Engineering from the University of California (Berkeley) in 1953 and a Master of Science in Structural Engineering from the University of Southern California in 1966. He has been responsible for management of major statewide programs. Mr. Roberts was the Principal in Charge of the $4.5 billion Caltrans Bridge Seismic Retrofit Strengthening Program and has implemented a comprehensive "problem focused" research program in support of seismic design and retrofit for California's 12,000 state highway bridges. He has conducted research in bridge technology, written and published several technical papers on bridge design and maintenance (especially seismic design and retrofit) and delivered hundreds of professional talks on various phases of bridge and transportation engineering to audiences ranging from professional societies to civic groups. He is an active member of many technical and professional organizations such as ACI, ASCE, SEAOC, TRB and IABSE. Mr. Roberts chaired the AASHTO-AGC-ARTBA Task Force 32, which produced the *Manual For Corrosion Protection of Concrete Bridges*, a handbook for bridge managers and owners, published in 1993. Mr. Roberts has received many professional awards for his achievements in bridge engineering and engineering management, and was inducted into the National Academy of Engineers in February 1996. He was the 1997 Golden Beaver Award recipient for Engineering, awarded by the construction industry, and the 2001 Roebling Medal for "A Lifetime of Contributions to the Advancement of Bridge Engineering."

Charles C. Thiel Jr., Ph.D.

Dr. Charles C. Thiel Jr. is President of Telesis Engineers in Berkeley, California. Dr. Thiel holds a Ph.D. in Engineering Sciences from Purdue University. He served as Director of the Division of Applied Research at the National Science Foundation, and was the founding head of the National Earthquake Hazards Reduction Program, first in the President's Office of Science and Technology and then in the Federal Emergency Management Agency. In 1976 he received the Arthur S. Flemming Award as one of the 10 Outstanding Young Men and Women in the Federal Service. Following his federal service, he was affiliated with Woodward Clyde Consultants, Forell-Elsessor Structural Engineers, and Stanford University, where he was a consulting Professor of Structural Engineering for 15 years. Dr. Thiel was founding editor of the Journal *Earthquake Spectra*. He has served since its formation as Chairman of the California State University Seismic Review Board that is responsible to the Trustees for seismic and all State University construction. He was the editor of and a contributor to *Competing Against Time* and *The Continuing Challenge*. He has been active in the development of the past four editions of the SEAOC Blue Book, and chaired the group that prepared the state building code sections on seismic retrofit of existing state buildings, and for acute care hospitals. He has authored many technical papers on earthquake engineering topics.

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