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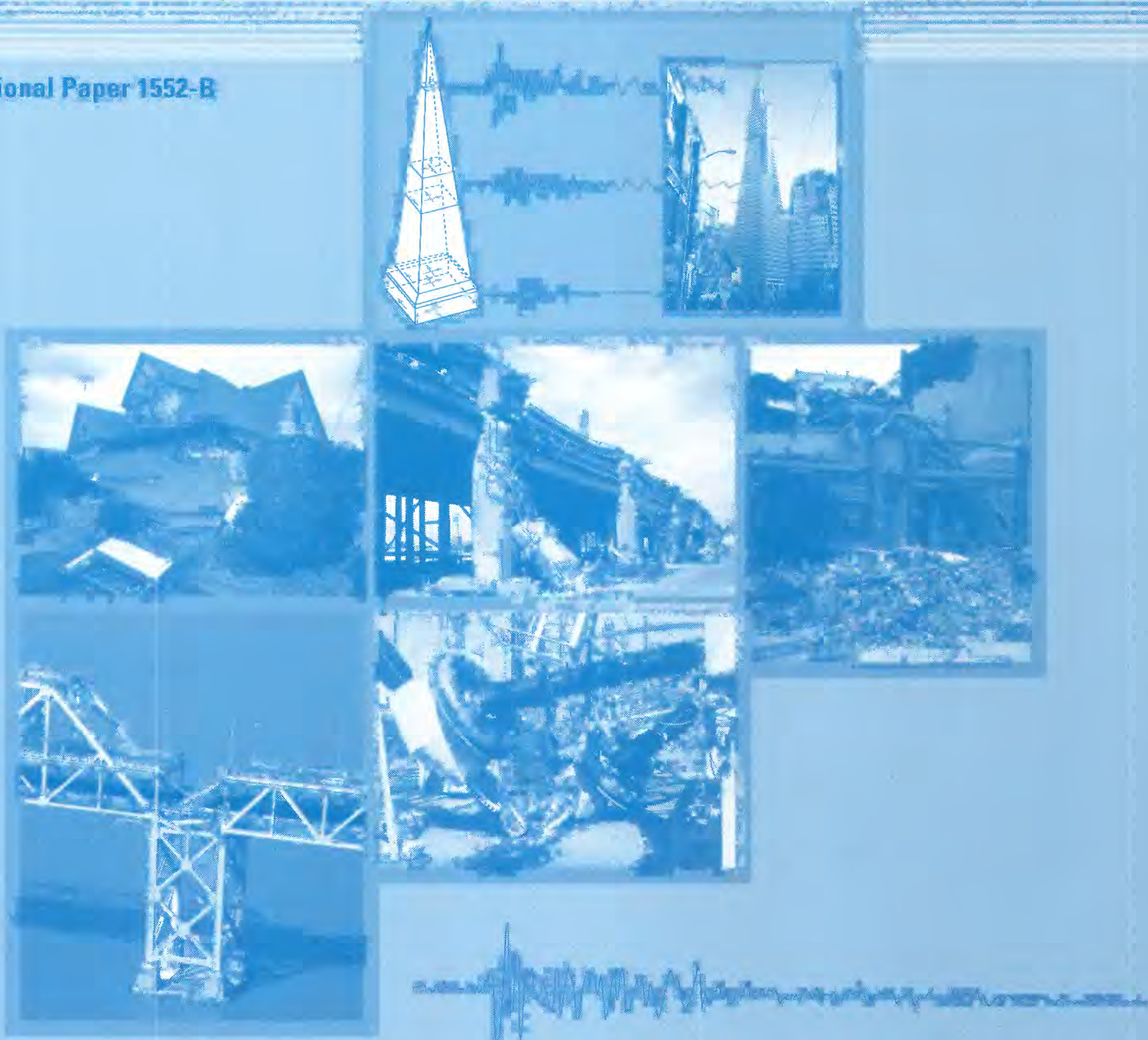
The Loma Prieta, California, Earthquake of October 17, 1989—Highway Systems

Professional Paper 1552-B



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The Loma Prieta, California, Earthquake of October 17, 1989—Highway Systems

By MARK YASHINSKY

PERFORMANCE OF THE BUILT ENVIRONMENT
THOMAS L. HOLZER, *Coordinator*

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CONTENTS

THE LOMA PRIETA, CALIFORNIA, EARTHQUAKE OF OCTOBER 17, 1989: PERFORMANCE OF THE BUILT ENVIRONMENT

HIGHWAY SYSTEMS

By Mark Yashinsky,
California Department of Transportation (Caltrans)

ABSTRACT

This paper summarizes the impact of the Loma Prieta earthquake on highway systems. City streets, urban freeways, county roads, state routes, and the national highway system were all affected. There was damage to bridges, roads, tunnels, and other highway structures. The most serious damage occurred in the cities of San Francisco and Oakland, 60 miles from the fault rupture. The cost to repair and replace highways damaged by this earthquake was $2 billion. About half of this cost was to replace the Cypress Viaduct, a long, elevated double-deck expressway that had a devastating collapse which resulted in 42 deaths and 108 injuries.

The earthquake also resulted in some positive changes for highway systems. Research on bridges and earthquakes began to be funded at a much higher level. Retrofit programs were started to upgrade the seismic performance of the nation's highways. The Loma Prieta earthquake changed earthquake policy and engineering practice for highway departments not only in California, but all over the world.

INTRODUCTION

The Loma Prieta earthquake occurred on the San Andreas fault in the Santa Cruz Mountains of Northern California. The earthquake caused damage to roads and bridges within about 100 miles of the epicenter including major damage to bridges in the cities of San Francisco and Oakland. On two bridges, this damage resulted in a tragic loss of life. On the Cypress Viaduct in the City of Oakland, 42 people died and 108 people were injured. One person died and 13 people were injured on the nearby East Bay Crossing of the San Francisco-Oakland Bay Bridge. Approximately 100 bridges suffered some damage from the earthquake. Eleven bridges and several roads were closed, resulting in traffic problems in the weeks and years following the quake.

This report is a summary of how roads and bridges were impacted by the earthquake. The main impact was damage to roads and bridges. However, there were other effects to bridges as well. These include changes to bridge seismic design codes, acceleration of California's bridge retrofit program, initiation of bridge retrofit programs in other states and countries, changes to emergency response procedures, improved methods of analysis, higher levels of bridge research funding, legislation affecting bridges, and changes to society's attitudes concerning bridges and earthquakes.

The Loma Prieta earthquake occurred in a remote location and was much smaller than a maximum credible event for the area. However, because it interrupted a World Series baseball game being played in San Francisco, it caught the world's attention. Moreover, although overall damage to the state highway system was minimal, there was major damage to some important bridges. These facts, as well as the unfortunate loss of life on two bridges, made the California Department of Transportation (Caltrans) a target for criticism after the earthquake. Some felt that Caltrans was negligent in allowing seismically deficient state bridges to be used by the public. Should Caltrans have been aware of any bridges that couldn't withstand a large earthquake? Should Caltrans have replaced all bridges that were seismically deficient? These questions led Governor Deukmejian to create a Board of Inquiry to determine why the bridge damage occurred. The board spent several months holding hearings to determine what Caltrans' seismic policies were before the earthquake. On May 31, 1990, the board published its report "Competing Against Time" (Thiel, 1990). They found that Caltrans had been doing a good job of improving their seismic design procedures for new bridges. They felt that the major cause of the bridge damage was the low level of funding for Caltrans' seismic retrofit program. They recommended that Caltrans increase funding for the seismic retrofit program, fund additional seismic research, utilize more state-of-the-art solutions to protect bridges from future earthquakes, and open Caltrans up to external review of its seismic policies.

The State of California has about 24,000 state and local bridges, many of which were designed before a rigorous seismic design code was developed. Before the 1971 San Fernando earthquake, there was minimal seismic criteria for bridges. The San Fernando earthquake was the genesis of Caltrans' education into the effects of earthquakes on bridges. One of the lessons from that earthquake was that bridge superstructures could fall off hinge and abutment seats. Thus Caltrans began the first seismic retrofit program to provide cable restrainers and shear keys on existing bridges to prevent them from separating at the thermal expansion joints during earthquakes. This program cost $54 million and resulted in retrofits to about 1,265 bridges. The earthquake also showed that columns with #4 ties at 12 inches were incapable of handling the large displacements that occurred. Caltrans changed its design criteria to include more tightly spaced transverse reinforcement and better reinforcing details between columns and footings and bent caps. The Whittier-Narrows earthquake of 1987 again pointed out the need to retrofit older unconfined columns. Caltrans began a testing program at the University of California at San Diego to evaluate the effectiveness of encasing columns in steel shells. Caltrans learns from every earthquake and uses that knowledge to improve design procedures.

Table 1.—*Chronology of significant events related to the Loma Prieta earthquake*

DATE	EVENT
9 February 1971	6.6 magnitude San Fernando earthquake occurs (major bridge damage). Causes major changes to Caltrans' seismic design procedures. Initiates Phase 1 retrofit program (providing restraint at hinges).
1 October 1987	6.1 magnitude Whittier Narrows earthquake occurs (minor bridge damage). Initiates Phase 2 retrofit program (providing confinement for single column bents).
17 October 1989	**7.1 magnitude Loma Prieta earthquake occurs at 5:04 P.M. (P.S.T.). (12 state bridges and several highways closed).**
21 October 1989	Rescue effort ends on the Cypress Viaduct.
23 October 1989	California Legislature holds special session to address Loma Prieta.
2 November 1989	Route 92/101 Interchange reopens. Mora Drive Overcrossing reopens (10 state bridges still closed).
3 November 1989	State Route 129 reopens in Watsonville.
6 November 1989	State Legislature appropriates $1 million for seismic engineering research. State Bill 36X and Assembly Bill 36X signed into law mandating a seismic retrofit program for all publicly owned bridges.
17 November 1989	East Bay Bridge reopens. West Grand Avenue Viaduct reopens. The Distribution Structure reopens (7 state bridges still closed).
18 November 1989	Highway 17 landslide cleared and the road is reopened to traffic.
25 November 1989	China Basin Viaduct reopens (6 state bridges still closed).
28 November 1989	Testimony begins before the Governor's Board of Inquiry (first of seven public hearings).
30 November 1989	Full scale test of portion of Cypress Street Viaduct begins.
13 December 1989	Governor's Board of Inquiry tours damage.
14 December 1989	Testimony continues before the Governor's Board of Inquiry (second public hearing).
4 January 1990	Testimony continues before the Governor's Board of Inquiry (third public hearing).
17-18 January 1990	Testimony continues before the Governor's Board of Inquiry (fourth public hearing).
26 January 1990	Struve Slough Bridge reopens to traffic. 5 state bridges remain closed - Cypress Viaduct, Central Viaduct (partial), Embarcadero Viaduct, Terminal Separation, and the Southern Viaduct.
8 February 1990	Testimony continues before the Governors Board of Inquiry (fifth public hearing).
10 February 1990	Cypress demolition completed
1-2 March 1990	Testimony continues before the Governors Board of Inquiry (sixth public hearing).
15 March 1990	Testimony concludes before the Governors Board of Inquiry (seventh public hearing).
31 May 1990	Governor's Board of Inquiry publishes its report, "Competing Against Time."
17 January 1994	6.7 magnitude Northridge earthquake occurs at 4:30 A.M. (P.S.T.).
17 December 1995	Southern Freeway Viaduct completely reopened to traffic, however retrofit is still continuing.
15 November 1996	Top deck for Central Viaduct removed. No decision made for retrofit or replacement of northern (concrete) portion of structure.
15 November 1998	Rte 880 (Cypress) Viaduct replacement completed (anticipated).
31 December 2000	Anticipated completion of Phase II bridge retrofit program including San Francisco Bay Toll Bridges.

The seismic criteria for bridge design in place at the time of the Loma Prieta earthquake was far in advance of that being used by the rest of the country. However, the speed at which Caltrans retrofits older bridges as new knowledge revealed their vulnerability is highly dependent on budgetary constraints. Before Loma Prieta, it was hard to justify spending a great deal on seismic retrofits when there were so many other pressing problems. Before the San Fernando earthquake, there was very little bridge damage from any California earthquake. Until Loma Prieta, only two people had ever died in California from bridge damage during earthquakes. Many more people die every year from traffic accidents on the highway system than from bridges collapsing during earthquakes and yet both of these problems are competing for the same scarce tax dollars. Since Loma Prieta and the recommendations of the Governor's Board of Inquiry, Caltrans has made protecting bridges from earthquakes a high priority. Before Loma Prieta such a commitment was almost impossible in spite of the fact that living where earthquakes occur is dangerous. The people of California do not have enough money to remove all dangers to society. However, Caltrans is working to reduce the risk of bridges collapsing during earthquakes.

The California State Legislature also played an active role after the earthquake. They wanted to ensure that the public would be adequately protected against a recurrence of events like the Cypress Viaduct collapse. A special 2-week session was called by the Governor beginning on October 23, 1989, to write specific legislation to address seismic safety issues. Simultaneously, the Senate Transportation Committee held a hearing in San Francisco to gather facts about the bridge damage. Burch Bachtold, Director of District 4 (where most of the damage occurred) and James Roberts, Chief of the Division of Structures testified at the hearing. At the end of the 2-week session, on Wednesday, November 6, 1989, the Governor signed 24 bills that made seismic safety a much higher priority in California. The Senate and Assembly passed identical bills to speed the legislative process. The most significant legislation for highways was Assembly Bill 36X. It exempted earthquake repair work from having to meet the usual time-consuming permit processes. This enabled repairs of the damage to begin quickly. It also allocated a quarter-cent sales tax that helped raise money to seismically retrofit vulnerable bridges. The most significant events related to highway systems after the earthquake are listed in table 1.

After the earthquake, Caltrans' first task was to identify the damaged state and local bridges and to determine what repairs were needed before they could be reopened. To understand how Caltrans' emergency response worked after Loma Prieta requires an explanation of how Caltrans is organized. Caltrans divides the State of California into 12 Transportation Districts. Most of the earthquake damage occurred in District 4 (fig. 1). District transportation engineers work out of an office located in each District. District 4's office was located in San Francisco at the time of the earthquake. The District Director is in charge of district personnel and signs all the contracts for work done in their District.

Bridge maintenance engineers have three offices. Bridge Maintenance South is located in Los Angeles, Bridge Maintenance North is located in Sacramento, and Toll Bridge Maintenance is located at the Bay Bridge Toll Plaza. Bridge maintenance engineers are responsible for the inspection and evaluation of all state and local bridges, and the maintenance of state bridges once construction is completed. Each bridge maintenance engineer is assigned responsibility for the bridges in their transportation district. If the maintenance engineer requires traffic control or a manlift, the engineer will usually ask a district maintenance crew for help.

Bridge construction engineers have a main office in Sacramento; however, they typically work from a field office next to their construction site. They are responsible for overseeing the work of contractors on bridge projects. If the work involves roads and bridges, a district construction engineer will have overall responsibility for the project and the bridge construction engineer will be responsible for any structures on the project. Bridge design engineers work from an office in Sacramento and design structural facilities on state roads. All bridge maintenance engineers, bridge construction engineers, and bridge design engineers work for the Division of Structures. The Chief of the Division of Structures at the time of the earthquake was James E. Roberts.

District transportation engineers have general responsibility to ensure that all transportation systems are functioning in their district. Most of the responsibility for bridges comes from the Division of Structures in Sacramento. Consequently, coordination between District and Headquarters staff was required after the earthquake. This coordination required good communication, especially between field and office personnel. Immediately after the earthquake, most of the damaged bridges were identified by District maintenance crews, California Highway Patrol officers, and bridge construction engineers. This was because they were in the area and could quickly examine the bridges nearby. They closed the structure if it was deemed hazardous and contacted District office personnel who would then contact bridge maintenance. Most field personnel were equipped with mobile two-way radios.

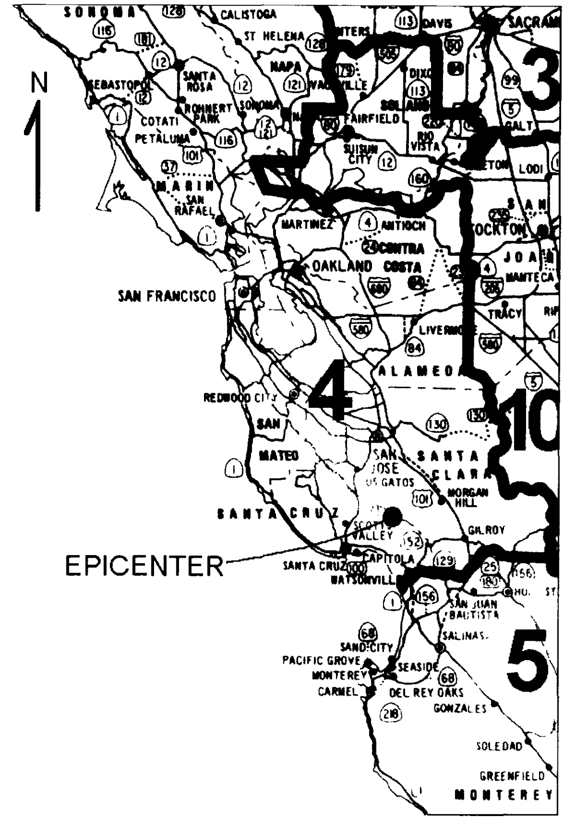

Figure 1.—Map of state highway districts (numbered) impacted by the earthquake.

Immediately after the earthquake, electricity, phone service, and the radio transmitter were all inoperative at District 4's office. One of the lessons learned from the earthquake was how effective cellular phones and pagers were for communication with field personnel. Unfortunately, they were a scarce commodity in the days immediately following the earthquake. To improve communication, an emergency command center was set up in the office at the Caldecott Tunnel. This center had superior radio equipment for communicating with field personnel. Even so, the mobile radio band was so jammed after the earthquake that a new band had to be found for Caltrans' staff. After a few days, bridge maintenance set up headquarters at the Cypress Viaduct to manage the inspection and repair effort. However, due to the distractions imposed by the rescue effort at Cypress, the bridge maintenance command center eventually moved to the Bay Bridge Toll Plaza. All of the staff from bridge maintenance north as well as six engineers from bridge maintenance south participated in the inspection effort. Every morning all bridge maintenance engineers were given state routes to inspect. They drove along the route with a log book and checked off each bridge as they completed an inspection report. They also inspected all county and city bridges. Maintenance engineers also completed inspections for all of the Bay Toll Bridge Crossings.

After the bridge maintenance engineer determined what was needed to temporarily or permanently repair a damaged bridge, an emergency contract would be created by the District Director. The ability to quickly write contracts and obtain money to pay contractors was essential to expedite bridge repair and reopen damaged bridges after the earthquake. California's legislature passed a bill to allow Caltrans to forego (for a limited time) the lengthy process of obtaining environmental documents so they could quickly make repairs and reopen bridges. Moreover, the District Director could hire contractors and write emergency force account contracts that would bypass the process of advertising contracts and evaluating bids, which can take many weeks. A force account pays a contractor for labor, equipment, and materials at the direction of a Caltrans' engineer, usually at a higher rate than would normally be allowed. The repair of most bridges immediately after the earthquake was made with force account contracts. The Bay Bridge was repaired in 30 days with a force account contract. A special contract with an incentive clause for early completion was written to replace the Struve Slough Bridges in Watsonville. Contractors from the Bay Area immediately began shoring damaged bridges, on occasion, even before an inspector had a chance to examine the bridge. Hard work on the part of many individuals helped the Bay Area recover so quickly. Several interesting reports describe how the Bay Area turned to trains, ferries, and other forms of mass transit after the earthquake (Bennett, 1991; Fahey, 1991).

Money for emergency repairs was available on federal aid roads from the Federal Highway Administration (FHWA) (table 2). The United States Congress appropriated $1.37 billion of emergency relief funds for the repair of roads and bridges. On those roads not eligible for FHWA funding, the State Office for Emergency Services (OES) and the Federal Emergency Management Agency (FEMA) provided funds for earthquake repairs. These funds are appropriated by the Congress of the United States after an emergency and are used not only for road repairs but also to repair public buildings and other public facilities. FEMA and FHWA money was available to both state and local bridges. However, the vast majority of state roads and bridges are repaired with FHWA funds, while the majority of local agency repairs are made with FEMA money. The cost to repair or replace damaged bridge structures after the earthquake was about $1.7 billion. However, the replacement of the Embarcadero Viaduct, Terminal Separation, and a portion of the Central Viaduct may never occur due to opposition from the city of San Francisco. The cost to repair roads was an additional $0.3 billion. Table 2 shows how the federal emergency relief funds were allocated to repair the damage. An additional $0.33 billion came from the state of California. These costs do not include retrofitting or replacing seismically deficient structures in the Bay Area after the earthquake. To put these costs in perspective, we can compare the costs to the highway system from other recent earthquakes as shown in table 3. What is particularly worrisome about the Loma Prieta earthquake is that most of the highway damage occurred so far from the fault rupture.

Most of the bridges with major damage were closed at least temporarily after the earthquake. All of the double-deck viaducts in San Francisco with the exception of the Alemany Interchange were closed due to column cracks. Some bridges, like the eastern portion of the San Francisco-Oakland Bay Bridge and the Struve Slough Bridge were reopened in record time. Other bridges, like the Cypress Street Viaduct and the Southern Freeway Viaduct, were still undergoing repair or replacement 6 years after the earthquake. Finally, structures such as the Embarcadero Viaduct and Terminal Separation were demolished with little chance of ever being rebuilt. A combination of politics, economics, and the state of bridge research were powerful forces affecting the fate of each damaged bridge.

After any earthquake, two groups from Caltrans are responsible for making a thorough investigation of bridge damage. Bridge maintenance engineers record the damage so that they can maintain their bridges and finance repairs. Caltrans' Post Earthquake Investiga-

Table 2.—*Cost to the Federal government of repair to highway system*

Type of Repair	**Federal Highway Administration funds, in millions of dollars**
Replace Embarcadero and Terminal Separation.	120
Replace Central Viaduct.	40
Other Bridge Repair and Replacement.	910
State Highway Repair.	280
City and County Roads and Bridges.	21

Table 3.—*Bridge repair costs after recent earthquakes*

Earthquake	Cost
1989 Loma Prieta	$1.7 billion
1994 Northridge	$0.3 billion
1995 Kobe, Japan	$6.5 billion

tion Team (PEQIT) records bridge damage as a way of evaluating current seismic design procedures and as a record for future research. A report of the PEQIT investigation after the Loma Prieta earthquake is available in Zelinski (1994). A third group, from the Federal Highway Administration (FHWA), accompanies bridge maintenance engineers and writes their own report to justify federal emergency funding.

Other groups from around the world rushed to California to make a record of earthquake damage. The National Institute of Standards and Testing (NIST) produced an excellent report on bridge damage (Lew, 1990). The Earthquake Engineering Research Institute (EERI) also has an excellent report (Benuska, 1990). Countries from Asia and Europe also recorded the damage (Kawashima, 1990).

Another interesting report, put together by Caltrans Transportation District 4, was an oral history of the earthquake. This report is the transcribed record of hundreds of recorded interviews taken in the days following the earthquake. The interviews offer a look at the experiences of many of the people involved in the emergency response after the earthquake. Secretaries and district directors, maintenance workers, and ferry boat operators are all included. The transcribed record, many thousands of pages long, is currently being summarized under a contract with the University of California and will be a great resource for a variety of researchers.

Not only was Caltrans busy repairing damaged bridges after the earthquake, but the seismic retrofit of existing bridges became Caltrans' number one priority. In fact, the line between repair and retrofit became somewhat vague. For instance, Caltrans expedited the retrofit and repair of several of the double-deck bridges in San Francisco in an attempt to get these structures back into operation as soon as possible. Several contracts were written for consultants to make Plans, Specifications, and Estimates (PS&E) to retrofit China Basin, Southern Freeway, and others. However, those retrofits were eventually rejected by a special Peer Review Panel chosen to review retrofits to the double-deck bridges in San Francisco. The cost to the state of California was several million dollars. Robert Cassano of the Peer Review Board and former Chief of the Division of Structures writes (Cassano, personal commun., 1993):

> Although it is very understandable, considering the transportation crises created by the loss of these viaducts, it is none-the-less unfortunate that considerable money was spent developing detailed plans and performing construction work that was subsequently discarded. This happened because of concerns that performance objectives would not be met using the original retrofit schemes or because of political factors. (I consider the demolition of the Embarcadero Viaduct and portions of the Central Viaduct to be political decisions.) With the perfect vision afforded by hindsight, it seems that more time should have been spent on conceptual studies and in building a consensus with local jurisdictions rather than rushing headlong into the final design phase.

The politics that Mr. Cassano refers to was the opposition to elevated expressways in the city of San Francisco. Caltrans spent much time and effort redesigning the Terminal Separation, now with little chance for its reconstruction. Thus, Mr. Cassano felt that more time should have been spent testing the political waters before proceeding with design.

James Roberts, Chief of the Division of Structures at Caltrans along with Deputy Director William Schaefer and Public Affairs Officer James Drago became spokespersons for Caltrans after the earthquake. In the weeks and months that followed the earthquake, they presented to reporters, state legislators, and the Governor's Board of Inquiry how Caltrans had been in the forefront of seismic related research and design since the 1971 San Fernando earthquake had shown the damage a large earthquake could inflict on transportation facilities. It soon became apparent that Caltrans' seismic design code was more than adequate and that the major problem had been a low level of funding for seismic retrofits of older bridges. However, that situation soon changed, as the State Legislature held hearings, passed laws, and appropriated funds to retrofit bridges deemed vulnerable to collapse or major damage from a large earthquake.

The Loma Prieta earthquake alerted the world to the effects a large earthquake can have on a modern metropolitan area. It changed seismic policy for cities, states, and countries around the world. The Federal Highway Administration made changes to its bridge specifications that all states would follow. Many countries sent investigation teams to write reports and make recommendations based on what they learned. Cities examined their emergency procedures, retrofitted vulnerable structures, and changed their design codes. Caltrans began an ambitious research program that has advanced the state of knowledge of earthquake engineering.

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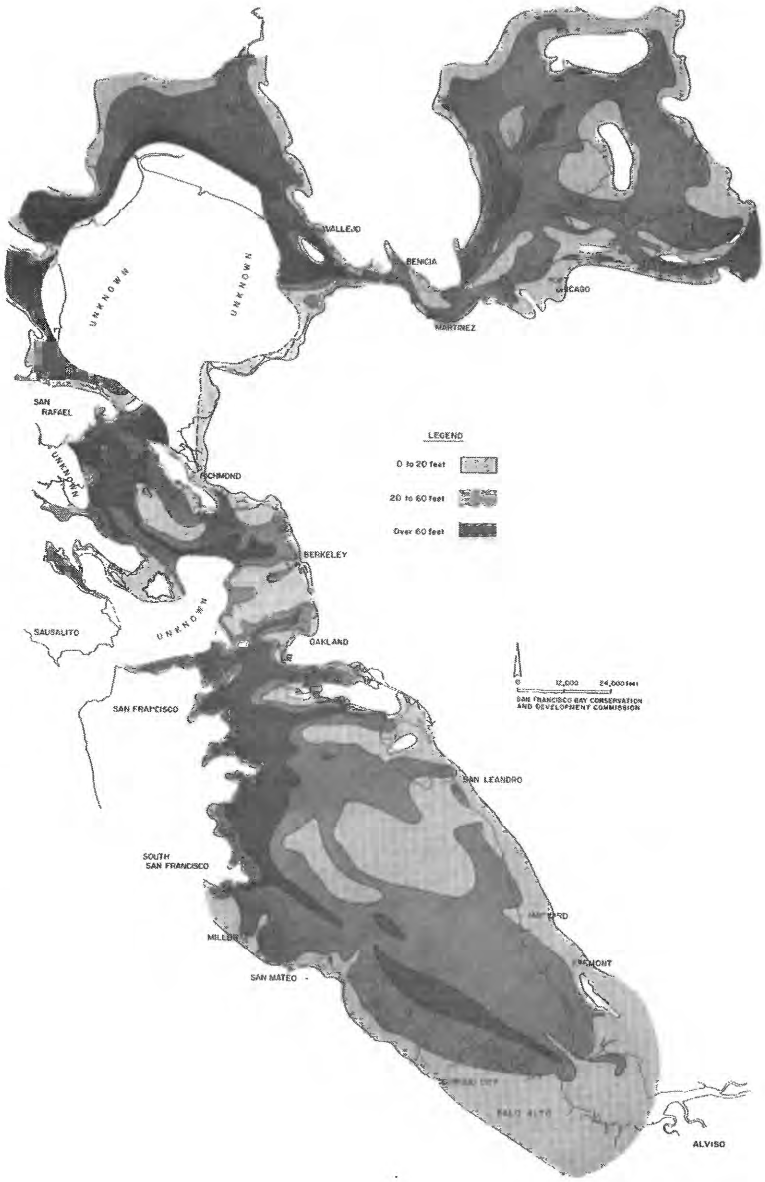

Figure 2.—Thickness of San Francisco Bay mud in feet (Goldman, 1969).

BRIDGE DAMAGE

Bridge damage from the Loma Prieta earthquake occurred over a large area and more than 100 miles from the earthquake epicenter. Some of the contributing factors to this damage were as follows:

Most of the Bridge Damage Occurred Where There Was Soft Soil

The Cypress Viaduct collapse occurred where the structure was supported on loose fill over Bay mud (fig. 2). There was no damage to portions of the freeway on firmer material. The damage to the San Francisco-Oakland Bay Bridge was a result of a combination of soft soil and flexible piles. All of the doubled-deck viaducts (fig. 3) except for the Central Freeway Viaduct were on soft soil. This includes the Embarcadero, Terminal Separation, China Basin, and Southern Freeway Viaducts, all of which suffered major damage during Loma Prieta. These bridges were over 60 miles from the epicenter, but the soft soils amplified the earthquake motions causing damage to these long, flexible structures.

The collapse of Struve Slough, near Watsonville, occurred in what is essentially a swamp. The deep soft soil pushed the pile extensions away from the bent caps. The Napa River Bridge, although over 100 miles from the epi-

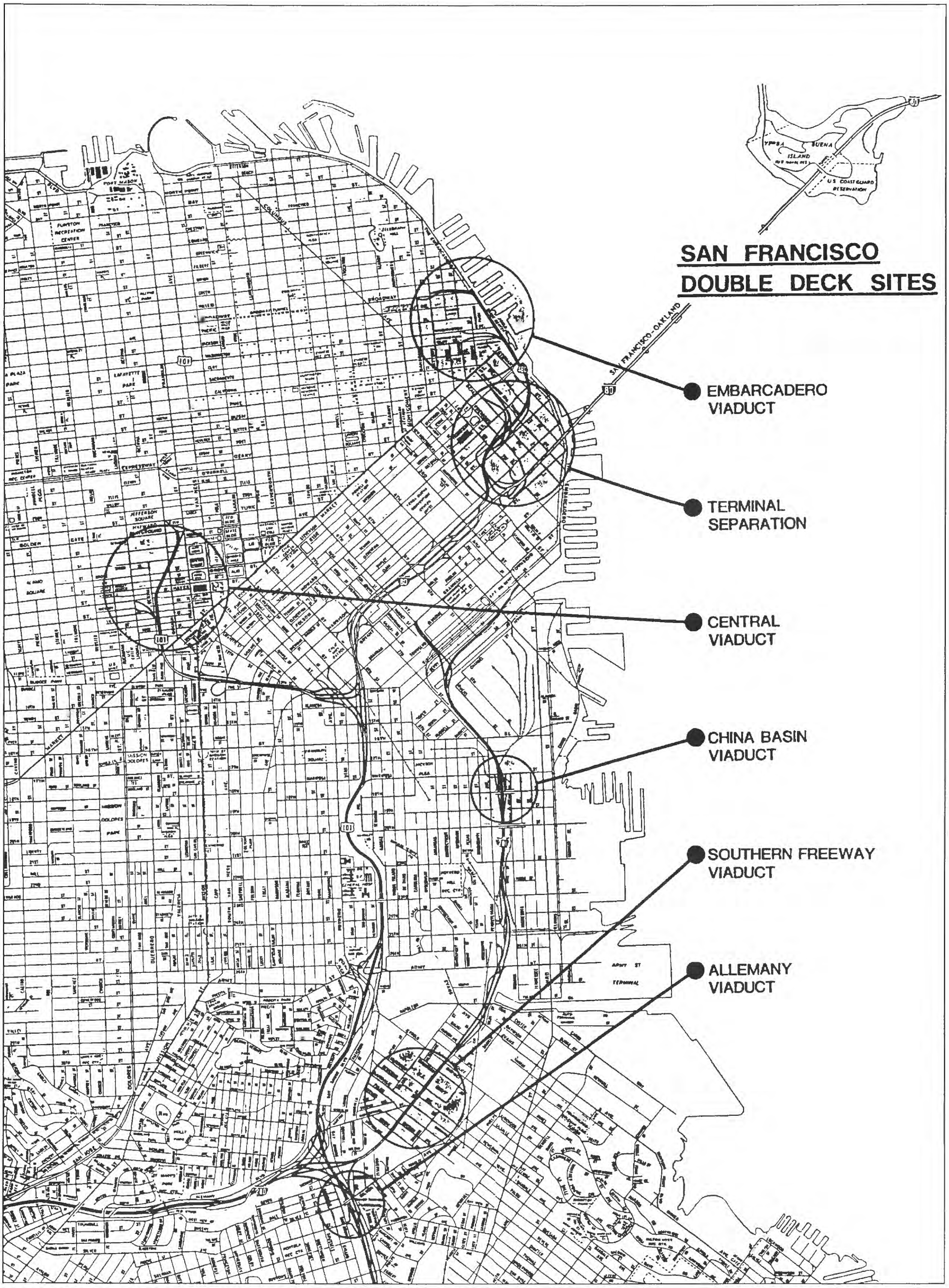

Figure 3.—Location of San Francisco double-deck viaducts.

center, sustained substantial damage due to the long-period motion which was amplified by soft soil. Of the 2,000 state bridges affected by the earthquake, only 20 suffered significant damage and 17 of those were on poor soils. This means that 85 percent of the heavily damaged bridges were on poor soils.

Most of the Damage Occurred to Bridges Built Before the 1971 San Fernando Earthquake

With the exception of the Route 980 Southbound Connector OC and the Route 92/101 Interchange, all of the state bridges with over $100,000 in damage as a result of the Loma Prieta earthquake were older bridges, without post-San Fernando seismic details. The San Francisco-Oakland Bay Bridge sustained its most devastating damage when the portion of the bridge east of Pier E-9 moved away from the pier, shearing all the bolts connecting it to the pier and dropping the top and bottom spans onto a platform on Pier E-9. Almost all the other major damage was the result of insufficient concrete reinforcement (particularly confinement reinforcement) that allowed columns, joints, and pins to suffer damage, pile extensions and precast girders to pull out of bent caps, and shear keys and hinge diaphragms to shatter. Research and new concrete details were developed in the 1970's that prevented serious damage to newer bridges.

Most of the Serious Bridge Damage Occurred in the Cities of San Francisco and Oakland

Of the 20 bridges that suffered major damage, six were in Oakland and six were in San Francisco, both 60 miles from the rupture, illustrating how site amplification and focusing of ground motion can damage bridges far from the earthquake hypocenter. Because cities are often the location of elevated freeways, complicated interchanges, and unengineered fills, urban areas are particularly vulnerable to highway damage during large earthquakes.

Most of the Damage Was the Result of Inadequately Designed Connection Details

Almost all of the serious bridge damage was the result of problems in transferring forces between vertical and horizontal elements. This was a problem for both steel and reinforced concrete members. Outrigger joints cracked, pinned connections shattered, moment connections sheared, bolted connections broke, and development rebar pulled out (fig.

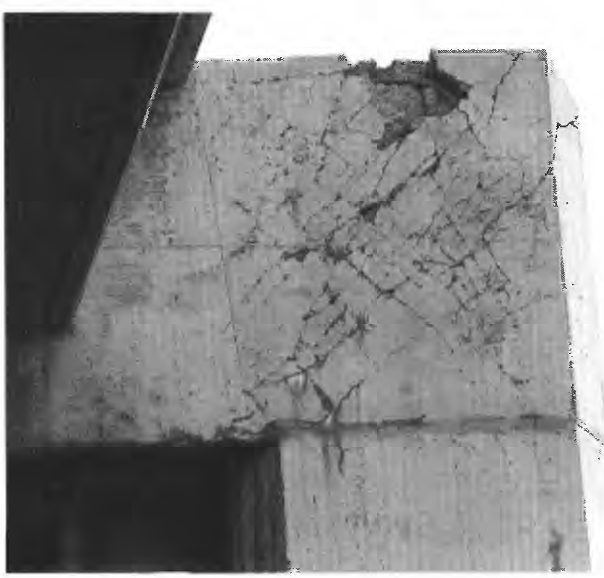

Figure 4.—Damage to outrigger knee-joint on Route 980 Southbound Connector.

Figure 5.—Location of bridges with over $100,000 in damage.

4). Much of the research after the earthquake focused on improving these connections.

There were 20 state-owned bridges that suffered over $100,000 in damage during the Loma Prieta earthquake. This damage varied from complete collapse to cracks and spalls that were expensive to repair but had no impact on system performance. Table 4 and figure 5 provide geographical and other information about these bridges.

There were 40 state-owned bridges that suffered between $5,000 and $100,000 in damage during the earthquake. Table 5 and figure 6 provide details on these bridges. A few of these bridges had serious enough damage to require temporary closure.

There were also many state-owned bridges that had damage of less than $5,000 to repair. Most of this minor damage was a result of bridge movement, particularly at expansion joints and bridge approaches that resulted in bearing damage and minor spalls. The occurrence of minor bridge damage was more general and covered a wider area than major bridge damage. Although most of the damage was confined to older bridges,

Table 4.—*State bridges with over $100,000 of damage*

BRIDGE LOCATION # (see fig. 5)	COUNTY	ROUTE	POST MILE	BRIDGE NAME	STATE BRIDGE #	EPICENTRAL DISTANCE (miles)	SOIL	YEAR BUILT	DAMAGE
1	SOL	037	R07.39	Napa River Bridge	23-064	78.6	Poor	66	Superstructure shifted longitudinally. Restrainers damaged.
2	MRN	101	04.03	Richardson Bay Bridge	27-010	68.7	Poor	57	Bearings, restrainers, bent caps, and columns damaged.
3	CC	680	24.26	Mococo OH	28-171	70.0	Poor	62	Hinges and earthquake restrainers damaged. Cracks in piles.
4	ALA	080	01.15	East Bay Bridge	33-025	60.0	Poor	36	Top and bottom decks fell at pier e9. Anchor bolt and bearing damage. Large bridge movement.
5	ALA	880	30.38	Fifth Avenue OH	33-027	56.4	Poor	48	Columns, bent caps, bearings, and substructure damaged.
6	ALA	580	46.09	Distribution Structure	33-0611	58.1	Poor	35	Bent caps and columns damaged.
7	ALA	080	02.41	Port of Oakland OC	33-126I	60.0	Poor	66	Bents and columns damaged.
8	ALA	880	32.39	Cypress Street Viaduct	33-178	58.9	Poor	57	Upper deck collapsed.
9	ALA	980	00.01	Route 980 South Connector	33-483f	57.6	Good	85	Two outrigger bents damaged.
10	SF	280	R04.40	Southern Freeway Viaduct	34-046	56.9	Poor	64	Bent and column damage.
11	SF	480	L00.01	Terminal Separation	34-054	59.0	Poor	55	Bearings on steel spans damaged. Bridge shored and closed.
12	SF	480	00.54	Embarcadero Viaduct	34-055	59.4	Poor	55	Column damage to bents 72 through 74. Bridge shored and closed.
13	SF	280	R04.07	Alemany Interchange	34-070	56.6	Poor	60	Minor cracks.
14	SF	101	R04.25	Central Viaduct	34-077	59.0	Good	55	Bent and column damage. Bridge closed.
15	SF	280	R06.61	China Basin Viaduct	34-0100	57.7	Poor	71	Bent and column damage. Bridge closed.
16	SM	092	R14.44	San Mateo-Hayward Bndge	35-054	43.2	Poor	67	Spalls on concrete trestle and misc. Steel damage.
17	SM	092	R11.78	Route 92/101 Separation	35-252I	42.8	Poor	86	Bearing and expansion joint damage.
18	SCR	001	17.19	Grant UC	36-0751	8.1	Poor	56	Damage to bearings and curtain walls.
19	SCR	001	R01.59	Struve Slough	36-0881	10.8	Poor	65	Complete collapse.
20	SCL	280	13.12	Mora Drive OC	37-235	23.9	good	67	Superstructure moved 4 inches longitudinally. Restrainers damaged. Column cracked.

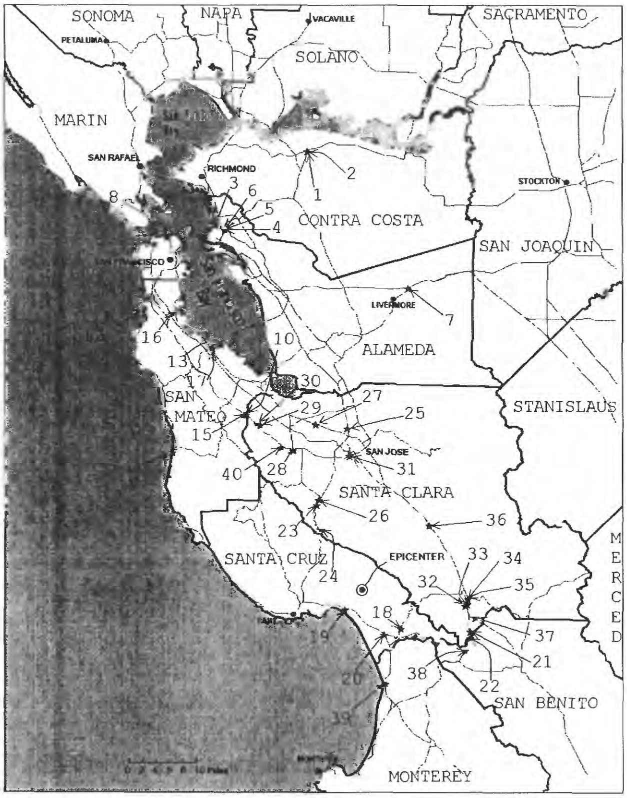

Figure 6.—Location of state bridges with between $5,000 and $10,000 in damage.

several newer bridges also sustained minor damage. This type of damage is considered inevitable for bridges in seismic areas as expansion joints bang together and approach fills settle. There was also some damage to city, county, and privately owned bridges. Table 6 and figure 7 provide details on these bridges. This was generally minor damage with the exception of a few bridges near the epicenter.

In addition, railroad bridges owned by the Southern Pacific Transportation Company and by the Atchison, Topeka, and Santa Fe Railway Company sustained some minor damage (fig. 8). Three timber pedestrian bridges owned by the city of Palo Alto were damaged and closed after the earthquake. There was even some bridge damage in state parks and recreation areas. The San Lorenzo River Bridge (Big Tree Bridge) in Henry Cowell State Park sustained some minor damage. Abutment #4 moved backward several inches and had some spalls. The Forest of Nisene Marks and Mt. Diablo State Park had some minor damage to trail bridges. Surprisingly, there was no damage to bridges owned by some local and private agencies. Of particular note was the absence of damage to the Bay Area Rapid Transit (BART). This absence was particularly fortunate because it allowed commuters an alternative method of transit across the Bay and into San Francisco, while the San Francisco-Oakland Bay Bridge and other important structures were closed after the earthquake. San Francisco's Municipal Railway (which includes the cable car system) was also back in operation once electricity was restored. Finally, the Golden Gate Bridge, owned and operated by the Golden Gate Bridge Highway and Transportation District, went through the earthquake unscathed.

On the following pages, each state-owned bridge that sustained major damage from the Loma Prieta earthquake is described in some detail. The term "bridge" is used as a generic title for structures used to carry a variety of things over a variety of obstacles. These structures actually have specific names depending on what is crossing them and what is below them. Table 7 provides some of the more common names and abbreviations for the different bridge structures.

There is a large amount of information available about many of the state-owned bridges that suffered significant damage during the earthquake. Only the most essential data is provided in this report. For those seeking more information, a brief list of reports that describes the bridge damage in more detail is given at the end of each main section.

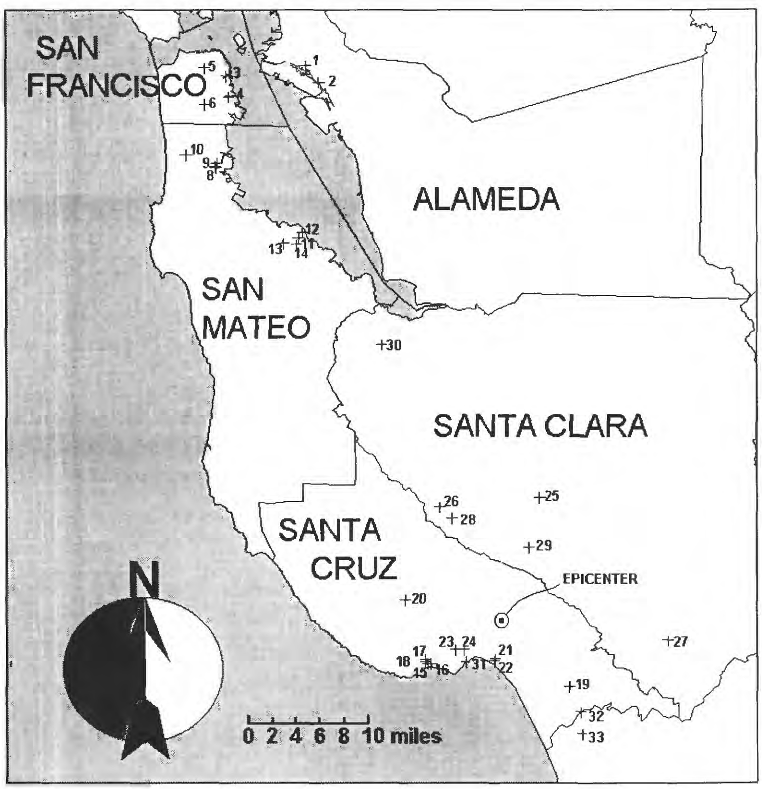

Figure 7.—Location of city and county bridges damaged during the earthquake.

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Table 5.—*State bridges with $5,000 to $100,000 of damage*

MAP LOCATION # (see fig 6)	COUNTY	ROUTE	POST MILE	BRIDGE NAME	STATE BRIDGE #	EPICENTRAL DISTANCE	YEAR BUILT	DAMAGE
1	CC	004	R14.61	Route 4/242 Separation	28-243s	68	78	Abut. 1 joint damage
2	CC	242	R03.24	West Connector OC	28-249g	68	78	Joint seals damaged
3	ALA	080	03.79	Powell Street UC	33-020	60	54	Approach slab damage
4	ALA	580	45.23	North Connector Viaduct	33-302h	59	70	Hinge diaphragm damage
5	ALA	580	45.14	580/w980 South Connector	33-303h	59	70	Crack in C-bent
6	ALA	980	01.98	East Connector Viaduct	33-304g	59	70	Crack in C-bent
7	ALA	084	R29.68	First Street Separation	33-398	47	65	Cracks in Bent 2 column
8	SF	480	04.51	Marina Viaduct	34-014	62	36	Hinge cracks Bents 37-39
9	SF	001	06.18	West Pacific Avenue UC	34-015	62	39	Gunite soffit fell Bent 4
10	SM	280	R0.01	Alpine Avenue UC	35-009l	31	69	Abut. & soffit cracks
11	SM	001	17.90	San Gregorio Creek Bridge	35-030	35	31	Wingwall cracks
12	SM	001	R44.88	Milagra Drive POC	35-188	54	65	Minor crack at Column 2
13	SM	092	R13.83	Foster City Lagoon	35-189l	43	67	Column cracks
14	SM	001	R46.65	South Connector OC	35-204f	55	72	Spalling at Bent 3
15	SM	280	R00.30	San Francisquito Creek	35-234l	31	69	Backwall, soffit cracks
16	SM	380	05.73	Huntington Avenue OH	35-253	51	71	Curtain wall damage
17	SM	092	T12.78	Mariners Island OC	35-284	43	77	Spalls at Pier 2
18	SCR	152	01.94	Corralitos Creek	36-001	11	36	Cracks in piles
19	SCR	001	10.01	Aptos Creek Bridge	36-011	4	48	Curtain wall cracks
20	SCR	001	R02.68	Connector Separation	36-084f	10	67	Abutment 1 shifted
21	SCL	101	R00.70	Sargent Bridge and Overhead	37-006l	20	70	Column & girder cracks
22	SCL	101	00.01	Pajaro River Bridge	37-007	21	41	Expansion joint damage
23	SCL	017	05.43	Sidehill Viaduct	37-029	14	40	Bearing seats damaged
24	SCL	017	01.14	Madrone Drive UC	37-059	10	38	Abutments rotated
25	SCL	880	05.34	Coyote Creek Bridge	37-065r	24	52	Abut. 4 approach settled
26	SCL	017	06.55	Main Street OC	37-117	14	55	Abut. 3 & bin wall cracks
27	SCL	101	43.85	Lawrence Expressway OC	37-152	26	61	Cracks and spalls
28	SCL	085	R18.49	West Connector OC	37-228f	23	67	Cracks and spalls
29	SCL	280	17.78	Arastradero Road UC	37-251r	29	67	Column cracks
30	SCL	280	18.38	Page Mill Road UC	37-252k	29	69	Column cracks
31	SCL	280	R02.87	Bird Avenue OC	37-267	20	69	Abut approach buckled
32	SCL	101	R04.94	Monterey Road UC	37-304l	18	73	Barrier rail, approach slab, & jt seal damage
33	SCL	101	R04.94	Monterey Road UC	37-304r	18	73	Appr. Slab & joint damage
34	SCL	101	R05.08	South Gilroy OH	37-305r	18	73	Approach slab damage
35	SCL	152	R09.91	10th Street Separation	37-325	18	72	Slope paving damage
36	SCL	101	R19.21	Coyote Creek Bridge	37-349r	15	80	Abutment 4 approach settled & misc. Cracking
37	SCL	101	03.25	Route 101/25 Connector	37-475 g	20	89	Hinge damage
38	SBT	101	05.21	San Benito River Bridge	43-004l	21	32	Pile & girder cracks
39	MON	001	96.44	Elkhorn Slough	44-074	17	85	Concrete column spalls
40	SCL	280	11.50	Foothill Expressway	37-239l	24	67	Slope paving damaged

Table 6.—*City and county bridges damaged during earthquake*

Local Bridge	Owner	Map Location # (see fig. 7)	State Bridge #	Epicentral Distance (mi)	Damage or repair cost
7th street bridge	City of Oakland	1	33C-149L/R	56 7	Approach settlement
Fruitridge Avenue Bridge	Alameda County	2	33C-147	54 4	$8,778
3rd Street Bridge	City of San Francisco	3	34C-027	58 7	$5,500
3rd Street Bridge	City of San Francisco	4	34C-024	56 6	$33,783
Geary Street P O C	City of San Francisco	5	34C-043	60 3	Minor column cracks
San Jose Street P O C	City of San Francisco	6	34C-14	57 2	Movement (very minor)
East Grand Avenue OH	South San Francisco	7	35C-148L/R	51 3	Spalls at pier & abutment
Produce Avenue Bridge	South San Francisco	8	35C-021	51 3	Wingwall damage
San Mateo Road Bridge	South San Francisco	9	35C-048	51 3	Sidewalk crack @ abut.
Hickey Blvd OC	South San Francisco	10	35C-032	54 2	Damage at abut 1
Foster City Blvd Bridge	Foster City	11	35C-070L/R	41 4	Bridge movement
Shell Blvd Bridge	Foster City	12	35C-070	41 5	Bridge movement
Hillsdale Blvd Bridge	Foster City	13	35C-068	41 6	Spalling at shear keys
Beach Park Blvd Bridge	Foster City	14	35C-062L/R	41 6	Settlement and spalls
Riverside Bridge	City of Santa Cruz	15	36C-099	8.7	Major damage - closed
Ocean Village Bridge	City of Santa Cruz	16	36C-100	8 7	Major damage to piers
Soquel Bridge	City of Santa Cruz	17	36C-105	8 7	Cracks to second pier
San Lorenzo River Bridge	City of Santa Cruz	18	36C-102	8 7	Abutment damage
Corralitos Creek Bridge	Santa Cruz County	19	36C-081	9 0	Minor cracks
Zayante OH	Santa Cruz County	20	36C-092	10 0	Settlement and hinge spall
Aptos Creek Bridge	Santa Cruz County	21	36C-075	4 0	Cracks above arch
Spreckles Road Bridge	Santa Cruz County	22	36C-113	4 2	Approach settlement
Rodeo Gulch Bridge	Santa Cruz County	23	36C-042	5.8	Cracks in abutments
Soquel Creek Bridge	Santa Cruz County	24	36C-078	5 0	Approach settlement
Almitos Creek Bridge	City of San Jose	25	37C-808	12 2	Abut. Shear key damage
Rundell Creek Bridge	Santa Clara County	26	37C-174	12.7	Abut backwall cracks
Uvas Creek Bridge	Santa Clara County	27	37C-177	15 7	Hinge and abut damage
Los Gatos Creek	Santa Clara County	28	37C-584	11 0	Pier & abut damage
Llagos Creek Bridge	Santa Clara County	29	37C-167	10.0	Pier & abut. damage
San Antonio Road Overhead	City of Mountainview	30	37C-120	28 9	Sheared hinge bolts
Cliff Drive Bridge	City of Capitola	31	36C-110	5 7	Approach settlement
Pajaro Creek Bridge	City of Watsonville	32	44C-55	11.5	Pier and abut. damage
Elkhorn Road OH	Monterey County	33	44C-33	13.4	Approach settlement, spalls at abutments & girders

Table 7.—*Functional classification of bridge structures*

Bridge Type	Description
Bridge (BR)	Usually reserved for structures over water.
Overhead (OH)	A structure carrying a highway over a railroad.
Underpass (UP)	A structure carrying a railroad over a highway.
Overcrossing (OC)	A structure carrying a local road over a highway.
Undercrossing (UC)	A structure carrying the highway over a local road.
Separation (SEP)	A separation in grade between two highways.
Interchange	Structures connecting intersecting roadways.
Causeway	A low structure over a body of water.
Viaduct	A long structure carrying a highway over many obstacles.
Aqueduct	A structure carrying water over many obstacles.
Pipeline Overcrossing	A structure carrying a pipe over a highway.
Pedestrian Overcrossing (POC)	A structure for carrying people.

Figure 8.—Location of railroad bridge damage (courtesy of American Railway Engineering Association).

MAJOR BRIDGE DAMAGE IN THE CITY OF OAKLAND

SAN FRANCISCO-OAKLAND BAY BRIDGE: EAST BAY CROSSING

DESCRIPTION OF BRIDGE

The San Francisco-Oakland Bay Bridge (figs. 9 and 10) is actually two separate bridges: a suspension bridge on the west side of Yerba Buena Island and a truss bridge on the east side of the island (there are also two approach structures and a tunnel at Yerba Buena Island). Both bridges have an upper and lower five lane deck. Construction was completed on these bridges in 1936. The East Bay Crossing is composed of five different segments. From Yerba Buena Island there are four 288-foot-long simple span trusses (Pier YB1 to E1), then a 2,400-foot-long cantilever truss (Pier E1 to E4), five 504-foot-long simple span trusses (Pier E4 to E9), fourteen 288-foot-long simple span trusses (Pier E9 to E23), and 16 simple span steel and concrete girder spans for the eastern approach (Pier E23 to E39). Concrete Piers YB1 and E1 are founded on sandstone; steel diagonally braced bents are on spread footings at YB2 to YB4 and on concrete caissons from Piers E2 to E5. Steel diagonally braced bents from Piers E6 to E16 are on timber piles, and concrete piers from E17 to E23 are on timber piles. Pier E9 is a very stiff four-legged tower that acts as an anchor for the more flexible piers east of the cantilever. Fixed and expansion truss shoes at-

Bridge #33-25 / Route 80 / Post Mile 1.15

Approximate Latitude & Longitude
N. Lat. 37° 48.7' W. Long. 122°21.7'

Epicentral Distance
60 miles

Peak Ground Accel.	N/S	U/D	E/W
Sandstone on west end	0.06	0.03	0.03
Bay mud at east end	0.29	0.07	0.27

Length	Width	Skew	Year Built
22,654'	63.3'	0.00°	1936

Main Span Type
Double deck through steel truss

Average Daily Traffic = 253,400

Figure 9.—The San Francisco-Oakland Bay Bridge (East Bay Crossing in background).

tach the superstructure to the substructure. Expansion joints for temperature movement are located above the double towers at YB3, E4, and E11. The ground varies from Franciscan sandstone at the surface of Yerba Buena Island to over 500 feet of Bay mud at the eastern approach.

The East Bay Crossing was seismically retrofitted in 1976. Rod and tie-down restrainers were installed at the east approach from Pier E23 to E38. Steel girder restrainers were installed at the expansion truss shoes at Piers E17 to E22. Figure 11 shows the geology that supports this structure. This drawing is based on borings that were done in San Francisco Bay in 1995 as part of the effort to retrofit the bridge in 1996.

The bridge motion during the earthquake was highly influenced by the change in geology from rock at the west end of the bridge to 500 feet of alluvium at the east side of the bridge. This dramatic change in stiffness meant that the east side of the bridge moved much more during the earthquake. This is reflected in the ground motions recorded near the site. The ground on the west side of the bridge had a peak acceleration of 0.06 *g* while the east side had a PGA of 0.29 *g*. Apparently, the high-frequency motion was filtered out by the time it reached the bridge and only the long frequency motion arrived at the bridge to accelerate the Bay mud over the deep alluvium.

BRIDGE DAMAGE

Except for a few minor problems on the West Bay Crossing, all of the earthquake damage occurred on the East Bay Crossing. The most significant damage was at Pier E9 where the top and bottom decks were pulled off their seats, causing them to fall onto a platform and electrical building near the top of the pier (fig. 12). The rest of the damage occurred from Piers E17 to E23 where anchor bolts failed, earthquake restrainers were damaged, stringers moved on their seats, the concrete pedestal at E17 rocked, and expansion joints spalled.

The movement of the decks at Pier E9 was the result of all 20 high-strength bolts at each of the east truss shoes breaking (fig. 13). This allowed the superstructure trusses on the east side of Pier E9 to move approximately 10 inches to the east and 5 inches to the north. The decks at Pier E9 are supported on steel stringers that rest on 6-inch steel angles bolted to large floor beams. When the trusses moved to the east, the stringers were pulled off the angles on the west end. The stringers remained on the bearing seats at the east end, which were bent as the west side fell.

The north and south fixed truss shoes at Piers E18 to E22 had all 12 one-inch-diameter bolts broken at each pier connection to the concrete pedestals. When the fixed shoes broke free, the expansion side of the trusses slammed into the steel girder restrainers (fig. 14). These restrainers apparently prevented the trusses from falling off the concrete pedestals, at least for the moderate forces generated from this earthquake. Elastomeric pads absorbed the impact when the truss shoes slammed into the restrainer beams. These damaged restrainers were easily replaced. At Pier E17, instead of the fixed truss shoes breaking, as occurred at Piers E18 to E22, the bottom of the concrete pedestals rocked. The concrete at the bottom corners of the pedestals spalled off and some vertical reinforcement buckled.

At Pier E23 the rivets connecting the bottom flange of the floor beam (fig. 15) to the concrete pier were sheared off, allowing the span between Pier E23 and E24 to move. A 9-foot drop-in span for the top deck at Pier E23 (figs. 16 and 17) rested on seats attached to this floor beam in a manner similar to that described for Pier E9 (fig. 18). When the floor beam moved, the steel stringer webs were pulled out of the guide angles at Pier E23 (similar to Pier E9) and to within one-quarter inch of the edge of the bearing seat.

ANALYSIS OF DAMAGE

There were several factors that influenced this bridge's performance during the earthquake. The large distance from the hypocenter meant that only longer frequency accelerations were arriving at the bridge. Since the west end of the bridge sits on bedrock, it was the east end of the bridge (on deep Bay mud) that was excited by the earthquake. The bridge is designed as a series of very flexible steel piers anchored to stiffer anchor piers. Pier E9 is designed to resist all the inertia force from Pier E9 to the expansion joint at Pier E11. This force was much larger than the 0.1 *g* that the bridge had originally been designed to handle during an earthquake. Therefore, the bolts designed to hold the trusses in place broke. The bearing seat at Pier E9 was too short to handle the subsequent movement of 10 inches longitudinally and 5 inches transversely during the earthquake. If these bearing seats had been longer, the trusses could have slid around without dropping the bridge decks. Stiff, concrete Pier E17 was designed to take the inertia force all the way from the expansion joint at Pier E11. Pier E17 could not handle this force and rocked at the base of the pier. The reason the bolts on the fixed bearing did not break (as they did from Pier E18 to Pier E23) was because there was a steel jack surrounding the bearings at the top of Pier E17 that prevented them from moving. The weakest area of Pier E17 was at the base of the pedestals, which rocked and spalled during the earthquake.

The spans from Pier E18 to Pier E23 are simple steel trusses with one end fixed and the other end on rollers. At every pier the fixed end broke, allowing the truss spans to slam into the steel girder restrainers. Similarly, the rivets on the floor beam at Pier E23 broke, allowing the span to the east of Pier E23 to move, almost pulling a short, 9-foot drop-in span off its bearings.

Basically, the inertia force was greater than the design force on the east end of the bridge, causing damage at anchorages and points of fixity. Fortunately, this damage generally consisted of broken bearings that resulted in spans sliding on their supports. The only serious dam-

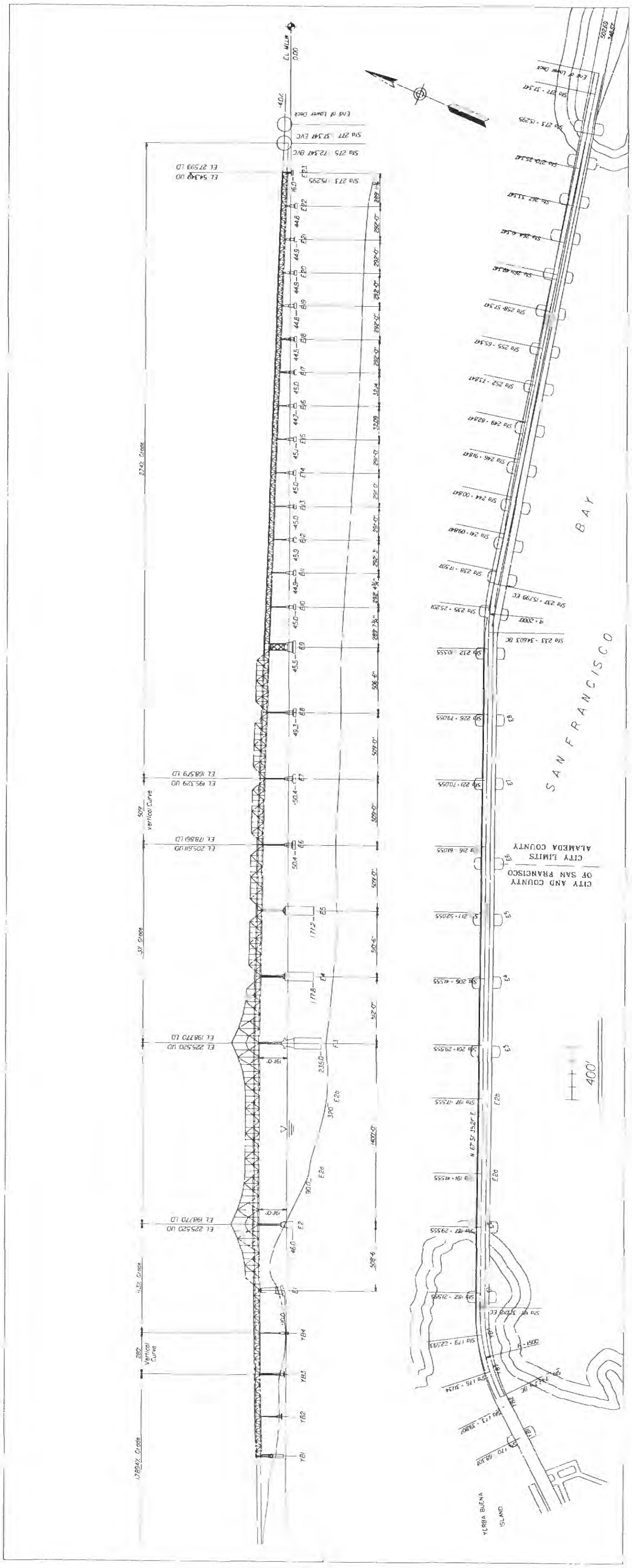

Figure 10.—Plan and elevation drawings of the East Bay Crossing.

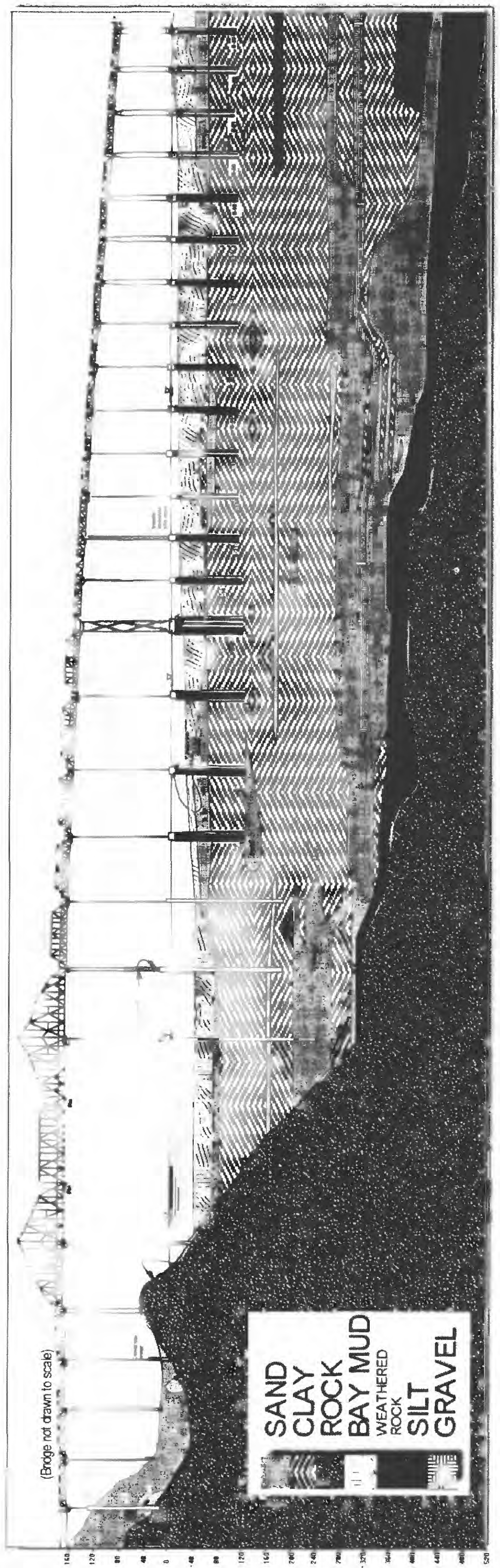

Figure 11.—Soil profile for the East Bay Crossing.

Figure 12.—Fallen decks at Pier E9 of the East Bay Crossing.

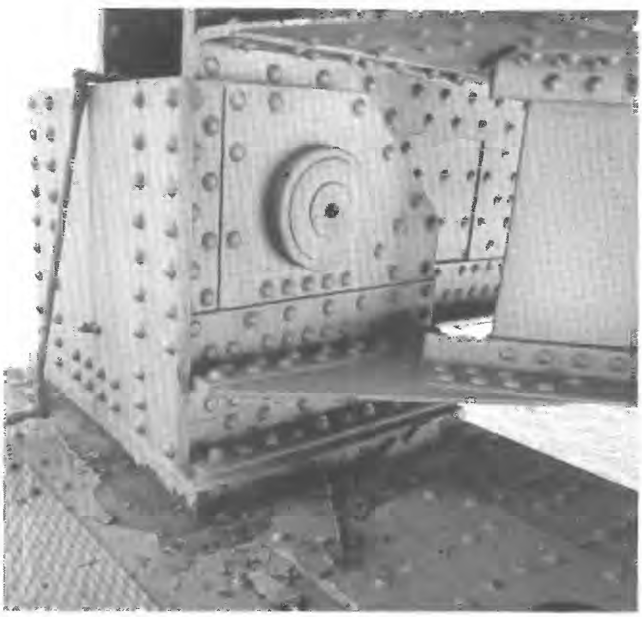

Figure 13.—Truss shoe at Pier E9 of the East Bay Crossing showing broken bolts and shoe movement after earthquake.

Figure 14.—Steel girder restrainer on East Bay Crossing with damage to rods and elastomeric pads after the earthquake.

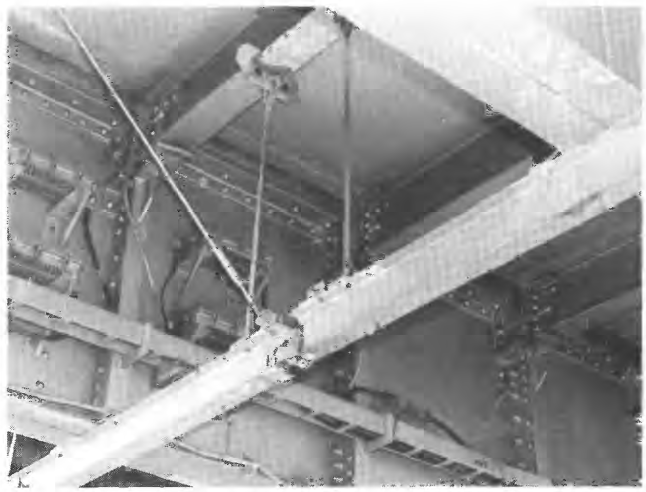

Figure 15.—Pier E23 floor beam with small girder seat that almost dropped this span during the earthquake (photograph by Erik Zechlin).

age was the dropping of the 50-foot span at Pier E9, which resulted in an unfortunate loss of life. Since the peak ground acceleration from the maximum credible earthquake at this bridge is at least 0.5 g, a substantial retrofit is required to prevent severe bridge damage from a rupture on the San Andreas or Hayward faults, which are near this site.

BRIDGE REPAIR

After the earthquake, an emergency contract was given to the Smith-Rice Company, who had the biggest derrick barge in the area to repair the bridge. Rigging International was the prime subcontractor. The fallen decks at Pier E9 were removed, new girders put in place, and precast concrete deck sections (fig. 19)

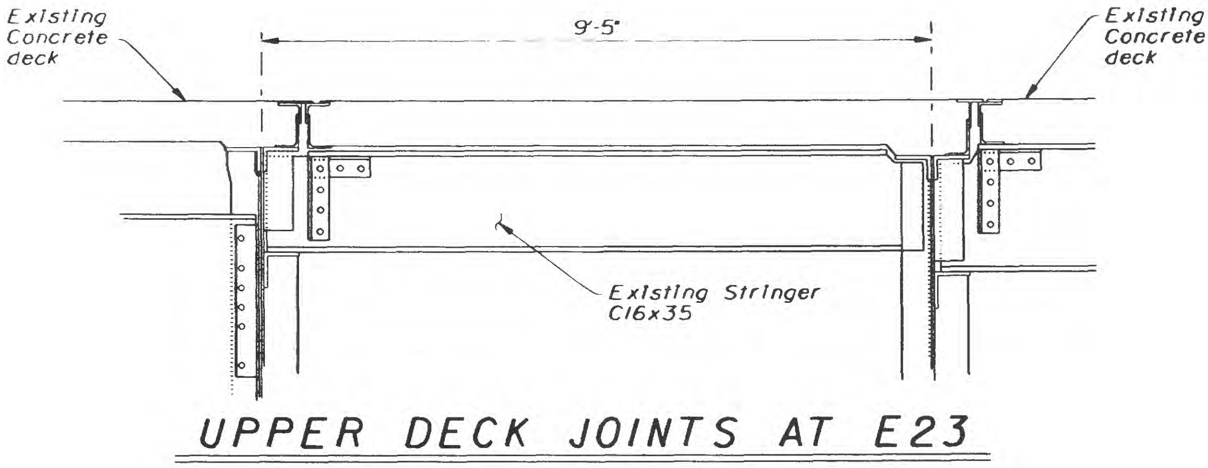

Figure 16.—Plan drawing of steel girders, floor beams, and girder seats at Pier E23 of the East Bay Crossing.

laid on top. All the truss sections were jacked back into place, new high-strength bolts were used, and new joint seals were laid.

On Friday, November 17, 1989, one month after the bridge was closed, it was reopened. On the first day, only pedestrians were allowed on the bridge. Politicians stood on the new span and gave speeches. People were allowed to look around and assure themselves that the bridge was safe. On Monday morning, the bridge was opened to commute traffic.

Since that time, Caltrans has been busy assessing the vulnerabilities of this bridge and devising a retrofit strategy to prevent major damage from the maximum credible earthquake. The retrofit strategy (fig. 20) that eventually emerged was to use reinforced concrete bents embedded around the existing steel piers with special bearings to isolate and dampen the earthquake motions from the superstructure. The cantilever portion was to have two additional piers and an exterior frame to strengthen the structure. However, the cost of this retrofit was so high that a replacement structure is now being planned instead.

Figure 17.—Top deck at Pier E23 of East Bay Crossing showing 9 foot drop-in section between floor beams (photograph by Erik Zechlin).

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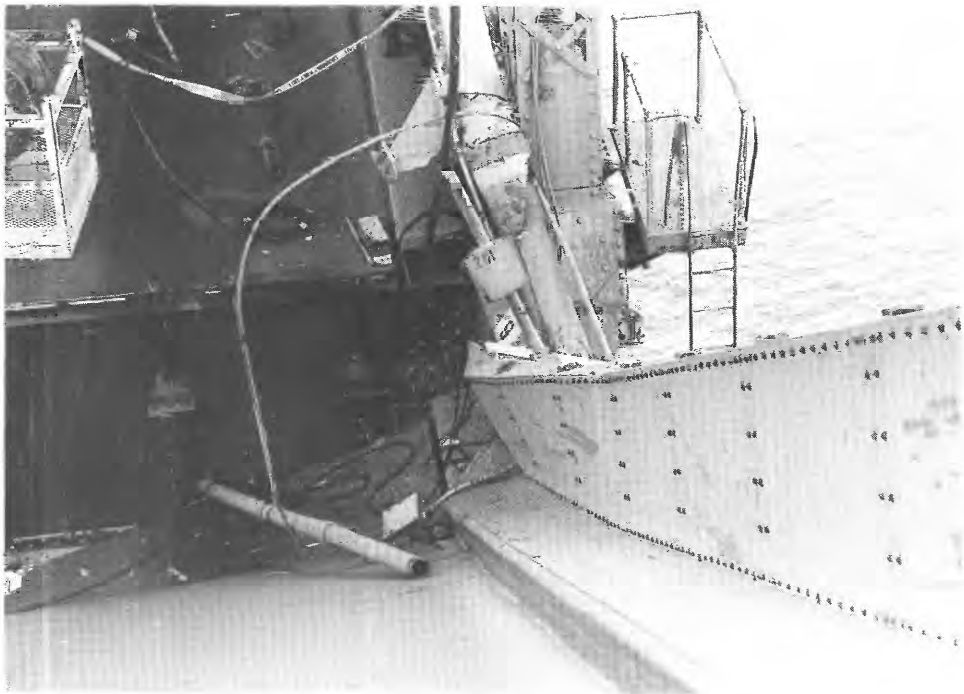

Figure 18.—Collapsed lower deck at Pier E9 of the East Bay Crossing showing short girder seats attached to floor beam.

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Figure 19.—Installing new, precast deck panels at Pier E9 on East Bay Crossing after the earthquake.

Figure 20.—Proposed retrofit of encasing steel towers in concrete for East Bay Crossing (currently (1997) superseded by plans to replace this structure).

CYPRESS STREET VIADUCT

DESCRIPTION OF BRIDGE

The Cypress Street Viaduct (figs. 21 and 22) was a long connector that took traffic from the Distribution Structure on I-80 south onto I-880 in the City of Oakland. This structure was a two-level, cast-in-place reinforced concrete, box girder bridge. Four lanes of traffic on the top and bottom decks were supported by two column bents. These bents had 11 different configurations as shown in figure 23 and described in table 8. Note that some of the top bent caps were prestressed as denoted by the dashed line. Many of the bents had pins (shear keys) at the top or bottom of the top columns, as indicated by circles in figure 23. All the bents were pinned above the pile caps as well. These column pins were four 59-inch-long #10 bars with fiberboard placed around them at the joint (fig. 28). There was a superstructure hinge at every third span on both superstructures. Design began on the Cypress Viaduct in 1949, and construction was completed in 1957. All these pins and hinges were used to simplify the analysis for this long, complicated structure.

Prestressing was just beginning to be used after World War II and was considered an innovation on this bridge (fig. 24). Caltrans' seismic criteria used at that time was to design for a lateral force of 0.06 times the dead load for structures on pile footings. This structure was seismically retrofit in 1977 with Type C1 cable restrainer units installed at all the hinges.

BRIDGE DAMAGE

Like the nearby East Bay Crossing, portions of the Cypress Street Viaduct were supported by soft Bay mud; bedrock was more than 500 feet below the ground surface. Figures 25 and 26 show that the portion of the viaduct that suffered damage was supported by this weak soil.

The Cypress Street Viaduct suffered the most catastrophic damage of any structure during the Loma Prieta earthquake. A large portion of the upper deck collapsed (fig. 27). This was from Bent 63 in the south all the way to Bent 112 in the north. Only Bents 96 and 97 remained standing. This collapse was a result of the weak pin connections at the base of the columns of the upper frame. Figure 28 clearly shows that there was inadequate confinement around the four #10 bars to restrain them during the earthquake. Also, the strong ground motion was probably influenced by the site conditions, as shown in figure 25. The sequence of collapse was as shown in figure 29. Although this was the most common collapse mechanism, there were several other types of failure as well. At some bents, the top and bottom decks both collapsed. Just south of Bent 113, the roadway sheared off, and at some bents where columns were supported by a cantilevered girder, the girder collapsed. Figures 30 shows an aerial view of that portion of the structure where the top deck collapsed. Figures 31 to 33 provide details of the damage. Table 9 lists all the significant damage that occurred. References to several papers that provide a complete description of earthquake damage for the Cypress Street Viaduct can be found at the end of this section.

RESCUE EFFORTS

The most important task after the earthquake was an intense effort to rescue any survivors on the collapsed freeway. Immediately after the quake, residents provided ladders and assistance to help people from the collapsed top deck (fig. 34). After the Oakland Fire Department arrived, everyone helped to extinguish burning vehicles and locate and rescue

Bridge #33-178 / Route 880 / Post Mile 32.4

Approximate Latitude & Longitude
N. Lat. 37° 49.1' W. Long. 121°17.2'

Epicentral Distance
60 miles

Peak Ground Acceleration	N/S	U/D	E/W
At Oakland Wharf	0.29	0.07	0.27

Length	Width	Skew	Year Built
19,330'	52.0'	varies	1957

Main Span Type
Two level, cast-in-place reinforced concrete, box girder bridge.

Average Daily Traffic = 137,700

Figure 21.—Cypress Street Viaduct.

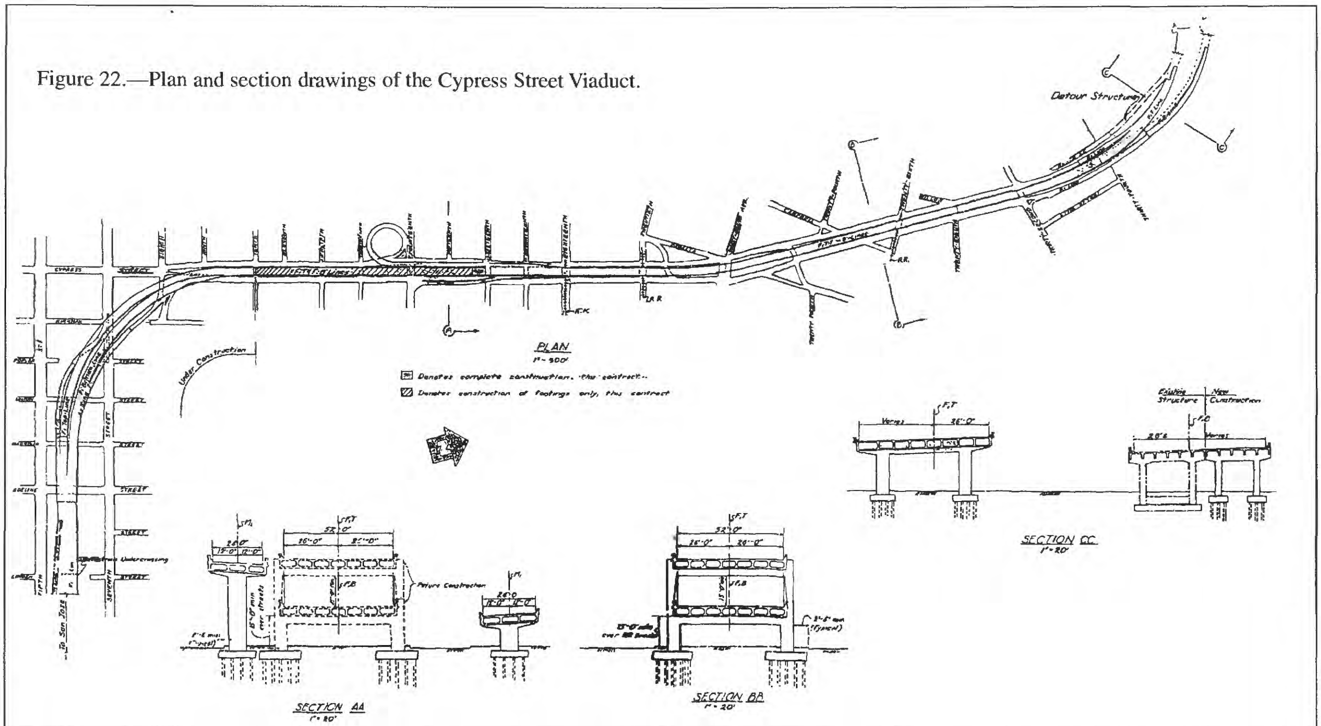

Figure 22.—Plan and section drawings of the Cypress Street Viaduct.

Table 8.—*Bent types used on the Cypress Viaduct (see figure 23 to identify types*

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

survivors. The exterior barrier rail was removed to allow workers to crawl under the collapsed deck. Everyone worked around the clock in an effort to rescue any survivors. Shoring was installed by contractors using contractor and state-furnished timber. Contractors excavated through the fallen top deck to reach the victims and vehicles trapped below. By Saturday morning, October 21, the last survivor, Mr. Buck Helm, was gently taken off the bridge. The entire operation had become a coordinated effort between Caltrans, the California Highway Patrol (CHP), local police, fire, and Sheriff/Coroner personnel. The Sheriff/Coroner's people had responsibility for identification and removal of victims, and the removal of vehicles was handled by the CHP and police. Caltrans' job was to provide safe access for search and rescue teams by inspecting the damaged structure and by providing shoring. Initially, the Scene Commander was the Oakland Fire Department. However, they quickly had their hands full coordinating the efforts of all the other fire departments that came to help. The Oakland Police took over command until at a meeting of all the participants there was an agreement to have the CHP take over as Scene Commander. Figures 35 and 36 show the Cypress Street Viaduct in the days following the earthquake.

To obtain a complete record of people injured or killed at the site required constant shoring and excavation of the bridge, the careful handling of remains, laborious contacts with area hospitals, and a computerized system to organize the accumulated information. There was pressure from the families of victims, as well as from politicians and the media to expedite this effort. An effort was made to keep victim's families as well as the media constantly informed. An infor-

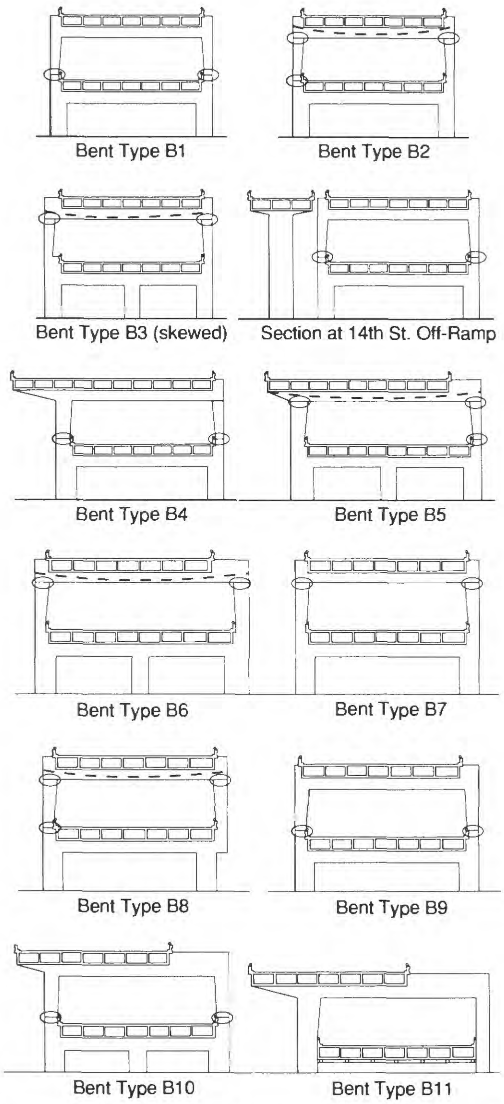

Figure 23.—Cypress Viaduct bent types as described in table 8 (from Thiel, 1990).

Figure 24.—Cypress Street Viaduct (1957).

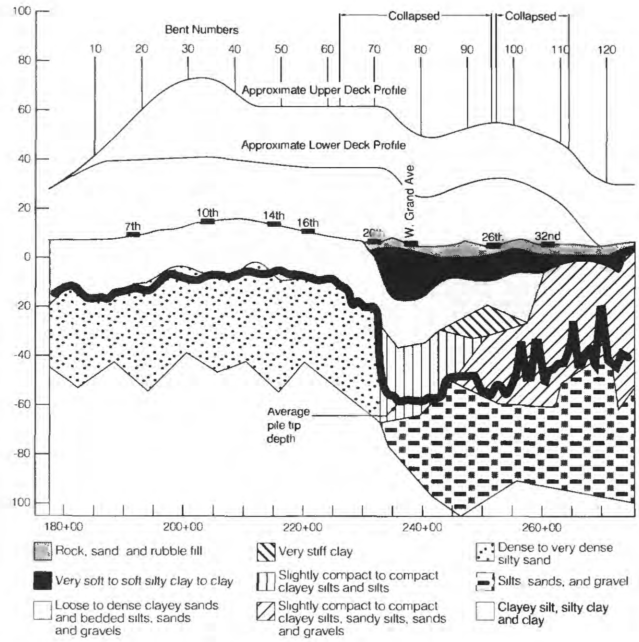

Figure 25.—Soil profile at Cypress Viaduct (Thiel, 1990).

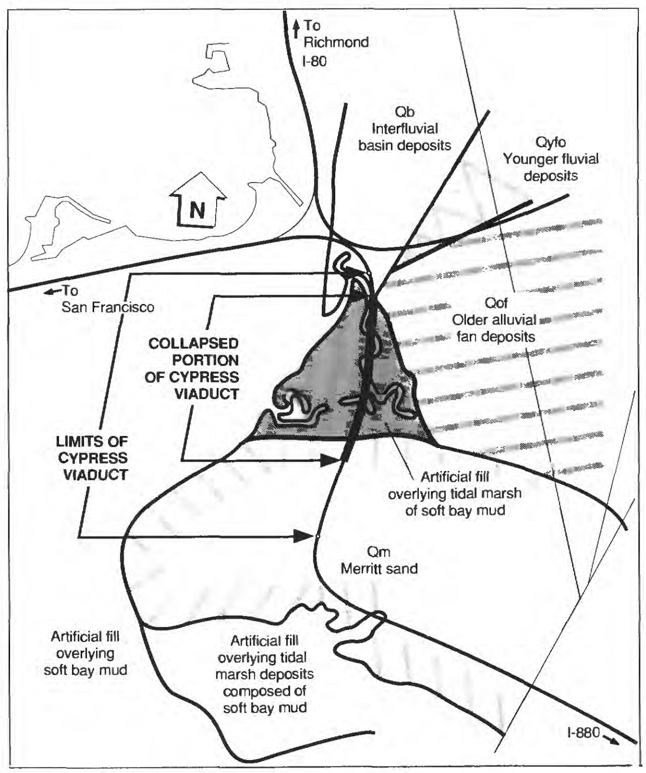

Figure 26.—Geological map of Cypress Viaduct area (Thiel, 1990).

mation center was set up a block from the command center. A Public Information Officer was assigned to provide interviews and press releases for the media.

TESTING AND DEMOLITION

After the last human remains and vehicles were removed from the site, the task switched to the expeditious removal of the freeway (fig. 37). The biggest problem was finding a place to dispose of the debris. The city of Oakland passed an ordinance prohibiting disposal of the rubble in the city. It was hoped that the debris could be recycled and used as construction material. However, there were difficulties in removing the reinforcement from the concrete and obtaining the necessary permits; in the end, almost all the material was hauled by trucks and disposed of along the Route 580 right of way in Castro Valley.

Another issue was making sure that minority contractors were obtaining a fair share of the work. This can sometimes be a problem when working with force account contracts. Caltrans made a special effort to include WBE/DBE contractors in all the work at Cypress. It was also necessary to shore and repair the offramp at I-980 to allow vehicles to take I-980 to I-80 (see section "Route 980 Southbound Connector").

Finally, the portion of this viaduct that was still standing afforded researchers a rare opportunity to do (1) destructive testing to help understand the collapse, the behavior of pile groups to failure, and new retrofit schemes and (2) testing to compare computed deflection and damage to actual bridge behavior. These tests were carried out by the University of California at Berkeley and Caltrans' Division of Structures. Steel jacking platforms were built (fig. 38). The bents were modified with different retrofits to provide confinement to vulnerable members. Then vibration tests were done as well as transverse loading to failure. However, due to time constraints to complete the testing, most of these confinement retrofit schemes were of limited value. Of more importance was the pipe seat extender testing. High-strength pipes were inserted in hinges to pre-

Figure 27.—Pin connection failure at Cypress Viaduct.

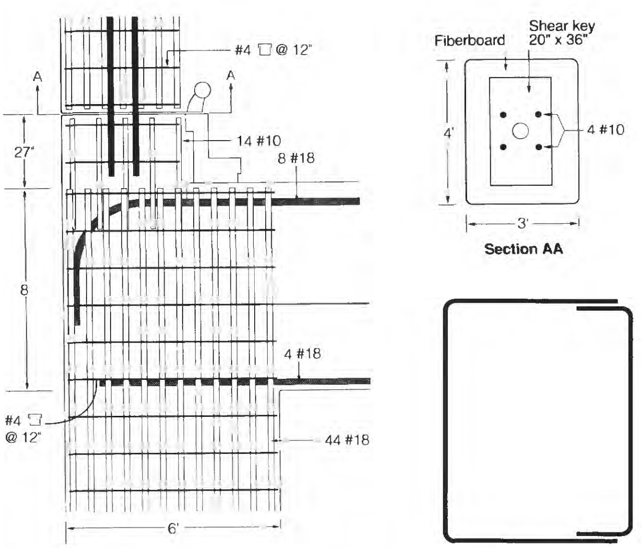

Figure 28.—Pin reinforcement details for Cypress Viaduct.

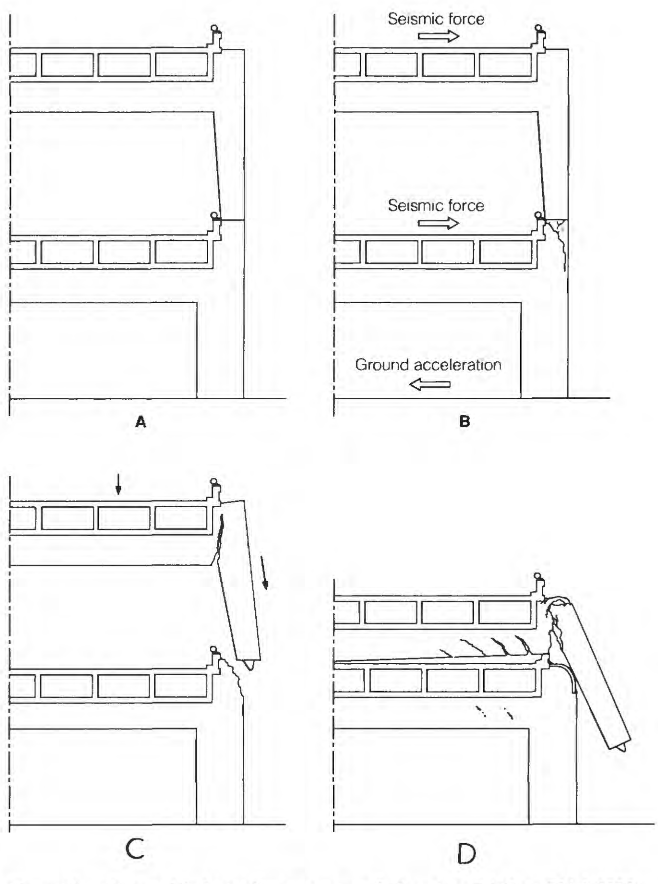

Figure 29.—Cypress Viaduct collapse sequence for bent type B1 (from Thiel, 1990).

Figure 30.—Collapse of Cypress Viaduct between Bent 62 and Bent 113.

Figure 31.—Hinge damage at Cypress Viaduct.

Figure 32.—Rotation at Bent 76 of Cypress Viaduct (note prestressing tendons).

Table 9.—*Summary of damage to Cypress Viaduct*

Bent Nos.	Bent Type	Bent Condition
32-45	B1	Standing, little observed damage.
46-47	B1	Standing, little observed damage, shear cracks in east and west faces of upper deck, location of ambient vibration test
48	B1	Standing, little observed damage.
49	B1	Standing, cracking in the critical region.
50	B1	Standing, cracking in the critical region, pounding with exit ramp.
51-54	B1	Standing, cracking in the critical region.
55	B1	Standing, cracking in the critical region.
56	B4	Standing, extensive cracking in the critical region.
57-61	B5	Standing, extensive cracking in the critical region.
62	B6	Standing, southernmost standing bent, extensive cracking in the critical region.
63-69	B1	Upper level collapsed, upper girder flat on lower girder, typical B1 failure.
70	B7	Upper level collapsed, upper girder flat on lower girder.
71-72	B2	Upper level collapsed, upper girder flat on lower girder.
73	B8	Upper level collapsed, cantilever failed, upper girder flat on lower girder.
74	B8	Upper level partially collapsed, cantilever tailed, transition between upper girder flat and upper girder tilted.
75-80	B2	Upper level partially collapsed, upper girder tilted, pin-ended column remained in place.
81-94	B1	Upper level collapsed, upper girder flat on lower girder, typical B1 failure.
95	B3	Upper level collapsed, upper girder flat on lower girder, east column displaced to the north.
96-97	B3	Standing, extensive cracking in lower-girder lower-column joint, and evidence of transverse cracking.
98	B3	Upper level collapsed, upper girder flat on lower girder, east column displaced to the north.
99-103	B1	Upper level collapsed, upper girder flat on lower girder, typical B1 failure.
104	B1	Upper level collapsed, extensive damage to lower girder, upper girder flat on lower girder, both upper and lower decks are on the ground at the expansion joint north of 104, typical B1 failure.
105	B1	Upper and lower levels collapsed, upper girder flat on lower girder, lower girder on ground.
106	B1	Upper level collapsed, extensive damage to lower girder, upper girder flat on lower girder, typical B1 failure.
107	B9	Upper level collapsed, upper girder flat on lower girder, similar to a B1 failure.
108	B9	Upper level collapsed, upper girder flat on lower girder, shear failure in lower east side column, similar to a 81 failure.
109-111	B10	Upper level collapsed, upper girder flat on lower girder, similar to a 81 failure.
112	B11	Collapsed, shear in girder. Abutment showed evidence of transverse motion.
113	B11	Standing, little observed damage, deck sheared completely south of this bent.
114	B11	Standing, little observed damage.

Figure 33.—Bent 70 of the Cypress Viaduct (view to south).

Figure 34.—Impromptu rescue efforts by people living near the Cypress Viaduct (copyright by Roy Williams, Oakland Tribune).

Figure 35.—Rescue efforts at the Cypress Viaduct (by trained personnel) continued for 5 days following the earthquake.

Figure 36.—Command Center at Cypress Viaduct.

vent unseating (fig. 39). This retrofit scheme has since become very popular.

By early February 1990 the Cypress Street Viaduct was completely removed and resurfacing and repairs to Cypress Street (now Nelson Mandela Parkway) were completed.

NEW BRIDGE CONSTRUCTION

The alignment for the Cypress Street Viaduct replacement as well as the design criteria for this project presented many challenges. The new route runs beside the Southern Pacific and Bay Area Rapid Transit right-of-ways (fig. 40), which has caused many logistical and negotiating headaches. Hazardous materials were found along the new alignment, which required an expensive cleanup. Also, the new alignment runs through several areas of soft Bay mud, which required developing site-specific response spectra. The weak soil also required designing large, stiff foundations with 42-inch-diameter steel pipe piles. Problems with outrigger bent caps handling the large torsional moments for longitudinal earthquakes necessitated using pinned bent cap connections. Lack of knowledge on joint shear transfer between reinforced concrete columns and bent caps required a testing program at the University of California at San Diego and a new conservative design for bent cap joints.

This project was divided into seven different sections and each section was designed by a different consultant or by Caltrans using the same design criteria. At five locations, the new bridge is Caltran's typical cast in place, prestressed concrete, box girder design. At the horseshoe fly-over at the toll plaza, a steel orthotropic box girder was used to eliminate the need for falsework and to handle the tight radius curve. On the portion of the project that goes over the existing Distribution Structure (on the northeast end of the project), steel welded plate girders were used. The project is scheduled for completion by the end of 1998. The cost of this replacement is almost a billion dollars, due to the City of Oakland's insistence on a new alignment and due to removal of toxic waste at the new site.

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Figure 37.—Removal of the Cypress Viaduct.

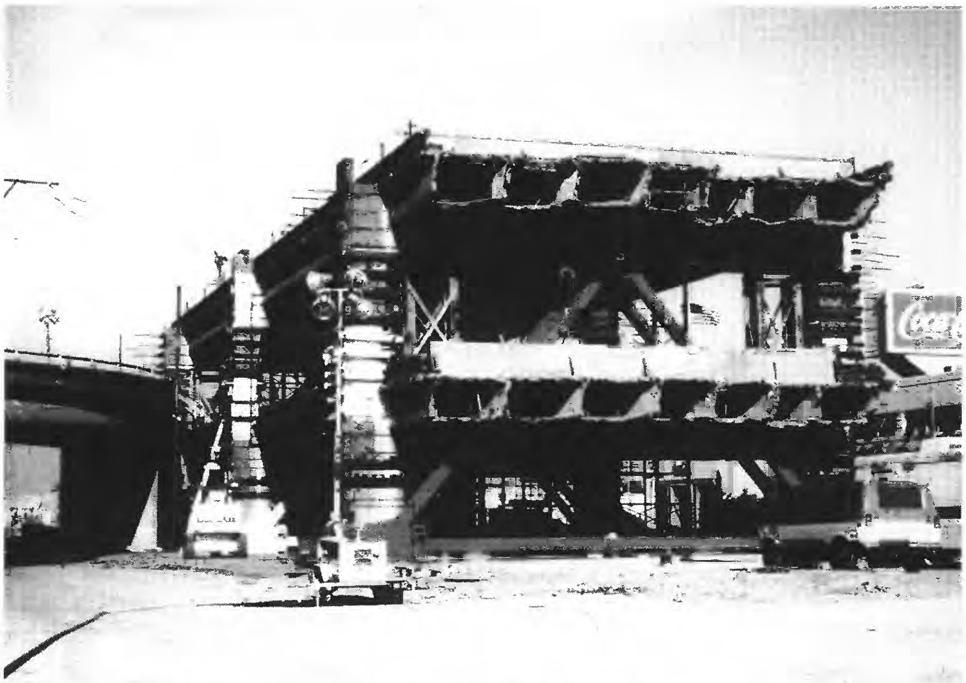


Figure 38.—Retrofit strategy being tested with jacking frame on section of the Cypress Viaduct.

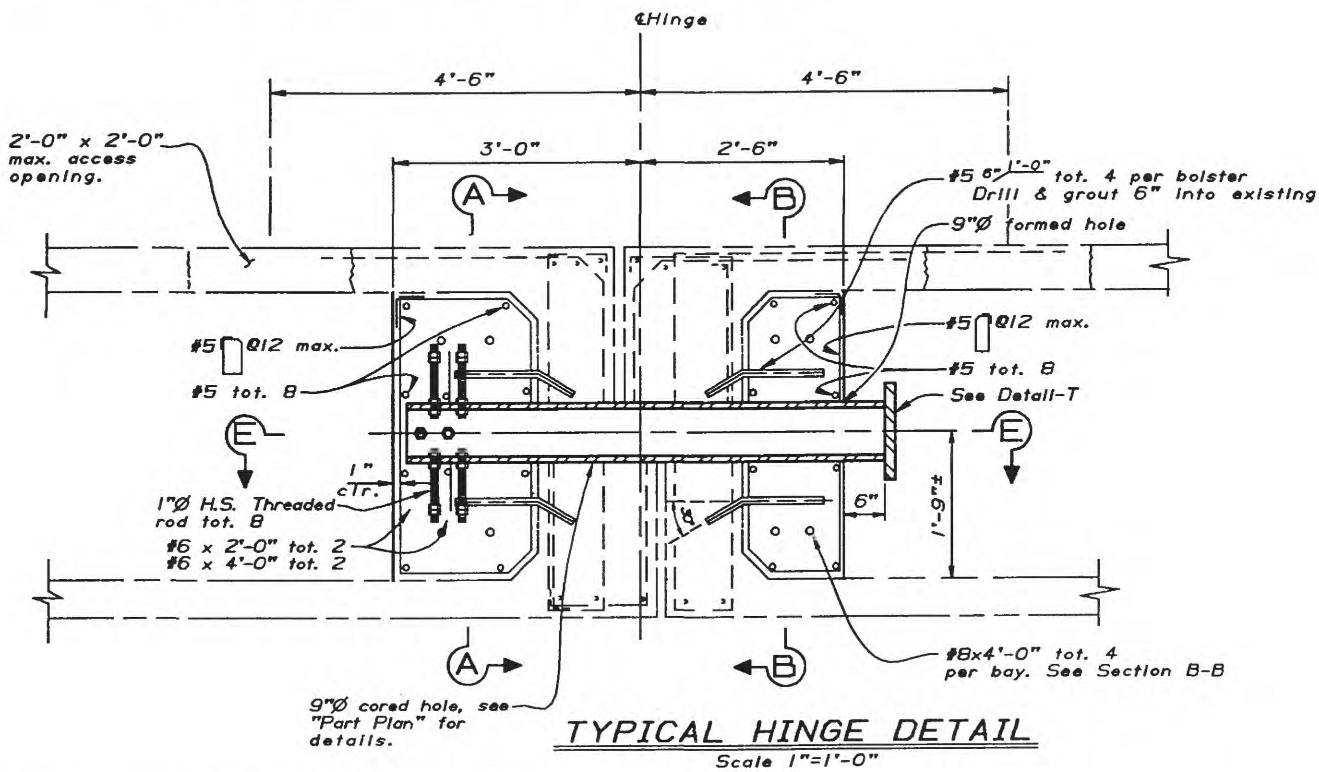


Figure 39.—Pipe seat extender test for the Cypress Viaduct

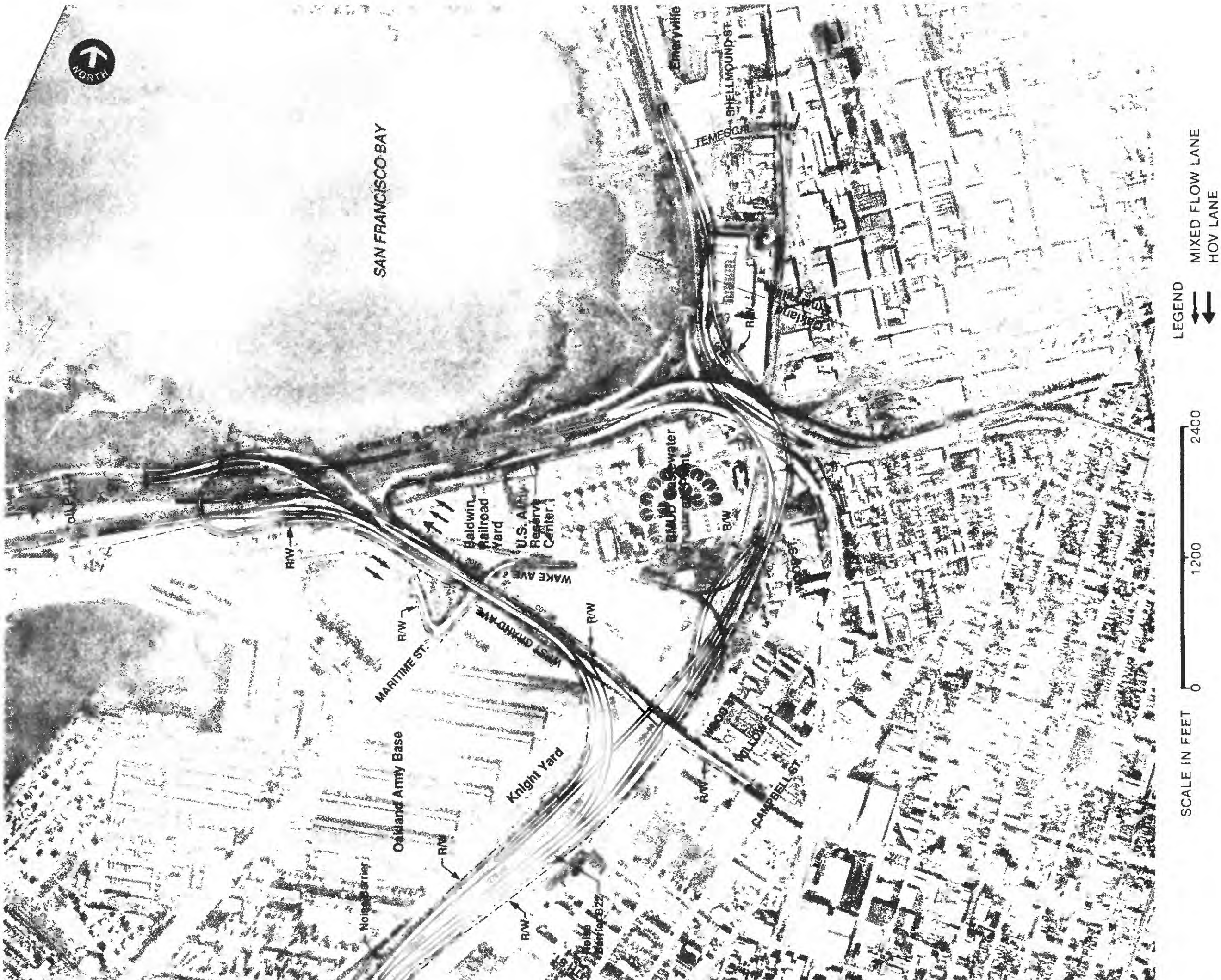
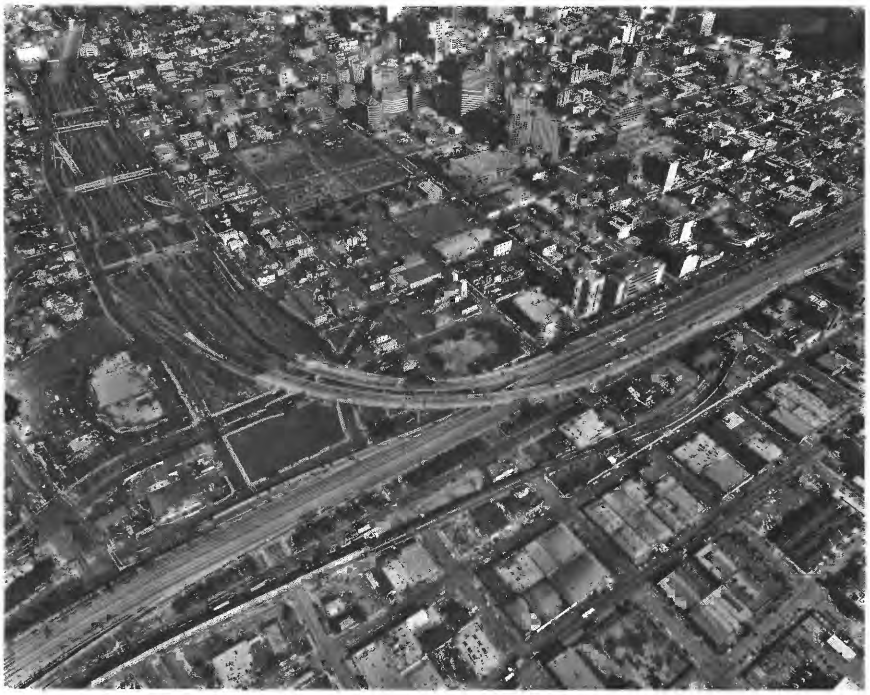


Figure 40.—Aerial view of alignment for Cypress Viaduct replacement.



Bridge #33-483 / Route 980 / Post Mile 0.01

Approximate Latitude & Longitude
N. Lat. 37° 48.1' W. Long. 122°16.9'
Epicentral Distance
58 miles

Peak Ground Acceleration	N/S	U/D	E/W
At Oakland Wharf	0.29	0.07	0.27

Length	Width	Skew	Year Built
2,265'	54' & varies	varies	1980

Main Span Type
Reinforced Concrete & Prestressed Concrete Box Girders.

Average Daily Traffic= 67,500 in 1990

Figure 41.—Route 980 Southbound Connector.

ROUTE 980 SOUTHBOUND CONNECTOR

DESCRIPTION OF BRIDGE

The Route 980 Southbound Connector is a 19-span, 1,800-foot-long, cast-in-place, concrete, box girder bridge (figs. 41 to 45). It carries traffic from southbound I-980 onto southbound I-880 in the city of Oakland. Starting at the south end, from the hinge at span 24 to the hinge at span 33 the superstructure is nine 75-foot conventionally reinforced spans. Then the superstructure changes to ten 120-foot prestressed concrete spans. The bridge is supported on multicolumn bents, outrigger bents, and one "C" bent as it steps gingerly around surface streets and expressways. All the columns except on the "C" bent are pinned at the base. The bent foundations are on spread footings (the log of test borings shows dense to compact sands) and the single abutment on the north end is a seat type abutment on steel "H" piles.

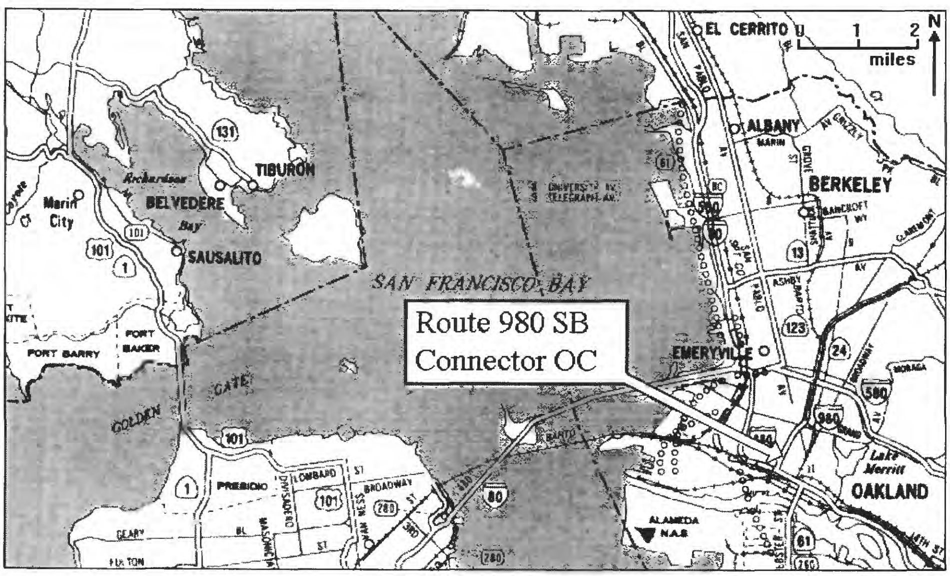


Figure 42.—Location of Route 980 Southbound Connector.

BRIDGE DAMAGE

There was quite a bit of superficial damage and a surprising amount of movement to this bridge, which was on good soil and 60 miles from the fault rupture. It is the only bridge built in the 1980's that suffered any significant damage during the earthquake. The serious damage was to the west outrigger knee joint on Bent 38 (figs. 45 to 49). This bent is very close to the hinge at span 37, which meant the superstructure could not provide much transverse stiffness for this bent. When this portion of the bridge

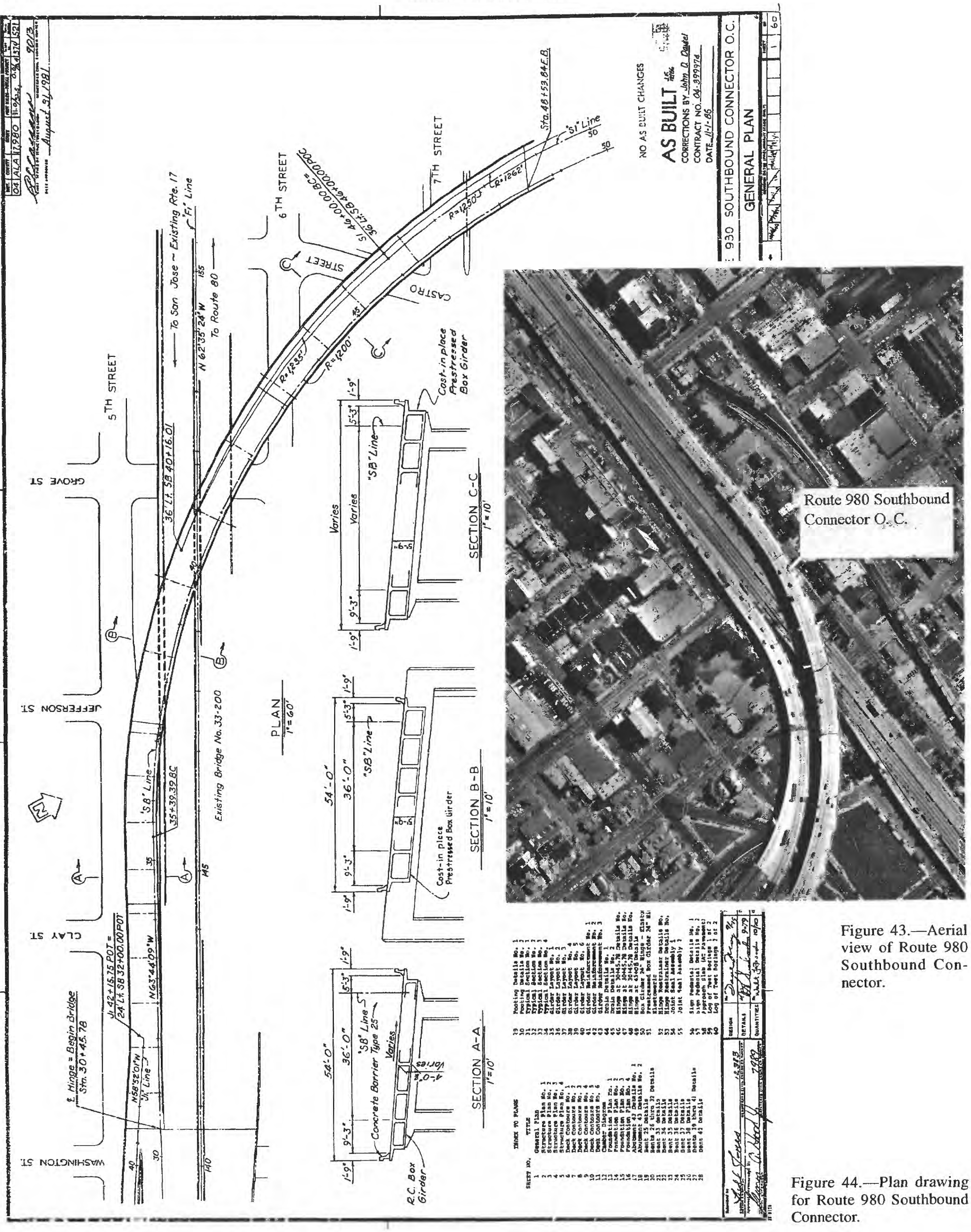


Figure 43.—Aerial view of Route 980 Southbound Connector.

Figure 44.—Plan drawing for Route 980 Southbound Connector.

moved transversely, the main restraints were the moment-resisting cap/column joints at Bent 38. Both joints were cracked, the west joint most severely, with the cracks, more severe on the hinge side of the bent. Also, the bridge deck over the bent sustained some cracks suggesting high stresses to the extreme fibers of the bent cap in bending.

The vulnerability of knee joints from opening and closing moments had been studied before this bridge was built (Park and Paulay, 1975). Caltrans' philosophy has been to design very strong joints and bent caps that force the damage to occur in the ductile columns. This bridge had very ductile columns, but the joints were underdesigned. Joint damage to this and other bridges has resulted in a major research and testing effort at the University of California, San Diego, to come up with better joint details and a coherent philosophy for joint design.

Bent 38's west outrigger knee joint was confined with W5 spiral wire which broke along with a #18 vertical reinforcing bar in the joint. Whatever moment capacity this joint had was severely reduced by cracking and spalling of the concrete after the confinement reinforcement broke. The #18 "X" reinforcement in this joint was tested in the materials lab after the quake. It was found that the broken rebar was of grade 75 steel, while the other bars were grade 60 steel. Moreover, the broken rebar had a curve radius of 11 3/8 inches, while the plans showed a bend radius of 18 inches. It is believed that the difference in grade and the sharp bend in the bar caused the failure. The difference in grade made the rebar much stronger, causing most of the load to be transferred to the stiffer bar. Other bridge damage including spalling of concrete at the wingwall and shear keys of the north abutment. There were column cracks at Bents 35 to 37 that appear to have occurred prior to but were widened by the earthquake. There was damage to the sidewalk, asphalt, and soil around the base of these columns, indicating that they experienced a lot of movement during the earthquake. The expansion joints at the abutment and hinges all showed signs of banging and large movements.

BRIDGE REPAIR

As mentioned previously, this connector was essential to move traffic from I-880 to I-80 after the collapse of the Cypress Street Viaduct. The connector was closed immediately after the earthquake. Subsequently, structural personnel examined the bridge and determined a temporary and permanent repair to allow traffic back on the roadway. Emergency repairs began on the bridge on October 20, 1989. They consisted of timber cribbing under the cap, supported by 24-inch-diameter steel pipe shoring capped with steel plates and braced with diagonal cables for lateral support. This allowed traffic back on the bridge. Then heavy-duty structural steel falsework frames were installed, and the pipe shoring was removed. Traffic was shifted to the east half of the roadway, and work was begun to strengthen the cap/column joints on the west side by adding reinforcement and increasing the size of the joints. After the concrete was cured, traffic was shifted to the west side, and the same procedure was done to the east side of the bridge. Then the falsework was removed, the surface streets below the connector were given minor repairs, and traffic was allowed back on the streets below the bridge. All the work was completed as of November 7, 1989. The total cost of this repair was just under $100,000. Since the only deficiencies to this modern structure were the weak cap/column joints, this repair was considered sufficient to protect the bridge from the maximum credible earthquake.

Figure 45.—Bent 38 of Route 980 Southbound Connector after the earthquake.

Figure 46.—Closer view of knee joint damage to Bent 38 of Route 980 Southbound Connector.

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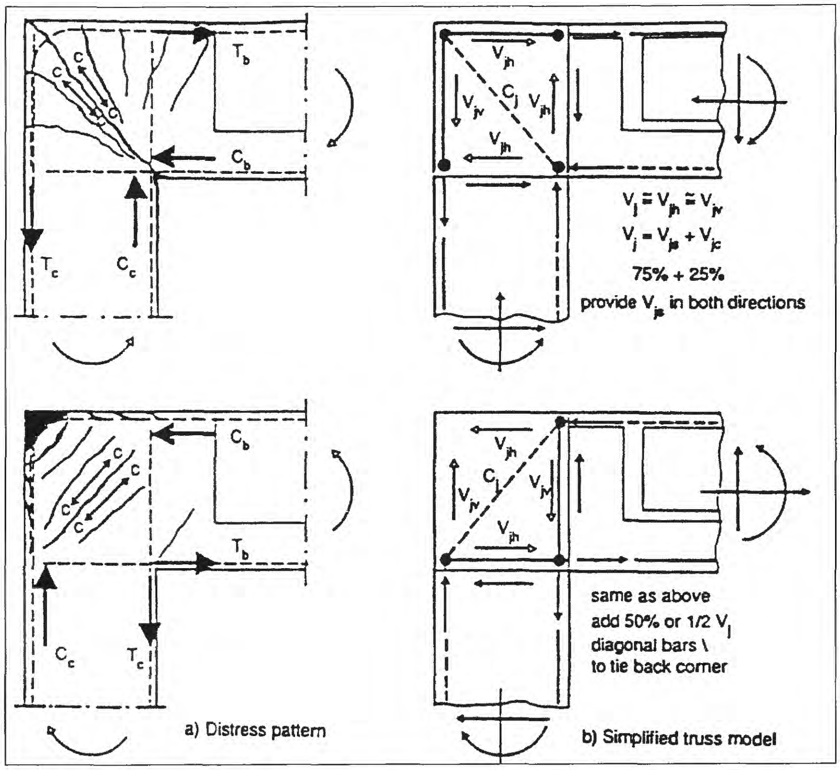

Figure 47.—Distress pattern for closing and opening of knee joints (Seible and Priestley, 1991).

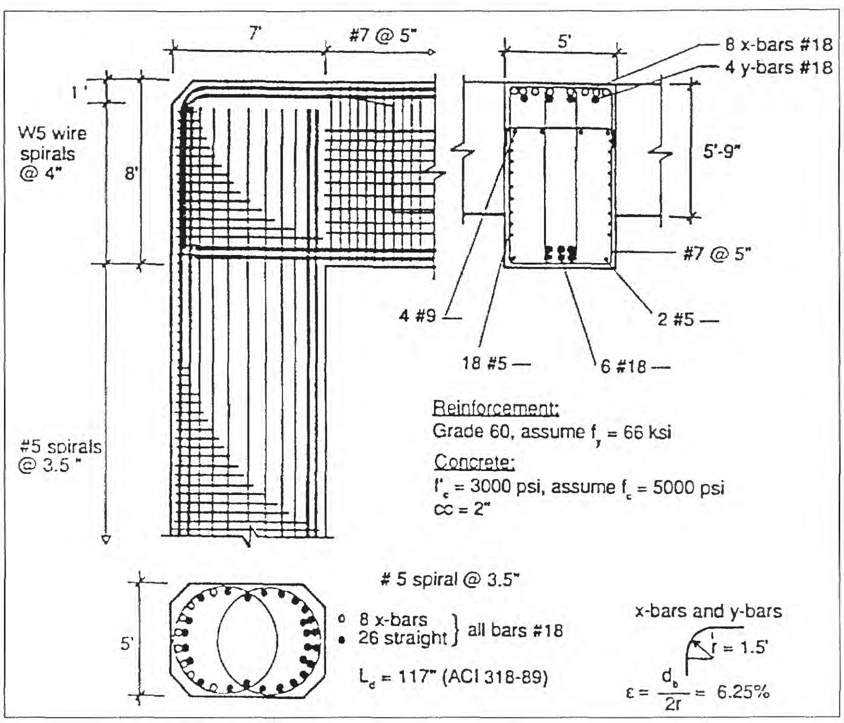

Figure 48.—Drawing of knee joint reinforcement for Bent 38 of Route 980 Southbound Connector.

PORT OF OAKLAND OVERCROSSING

DESCRIPTION OF BRIDGE

This long viaduct crosses southeasterly over I-80 (by the Bay Bridge Toll Plaza), the Baldwin Railroad Yard, and the Oakland Army Base until it meets West Grand Avenue at grade. It has 10 alignments that include several on and off ramps (figs. 50 to 52; table 10). The superstructure comprises simple and cantilever steel stringer spans supported on a variety of bents (including an unusual two-column bent supporting a single column bent). The variety of member types on this structure is due to its having been built in the 1930's and then substantially extended in the 1950's. The original 1930's-designed structure is mostly two-column steel bents framed into steel floor beams spanning over the railroad yard. The 1950's structure is mostly reinforced concrete single column bents with steel and concrete hammer head bent caps. The foundations are typically pedestal type concrete footings on untreated timber piles. The log of test borings shows soft clays at I-80 with denser mate-

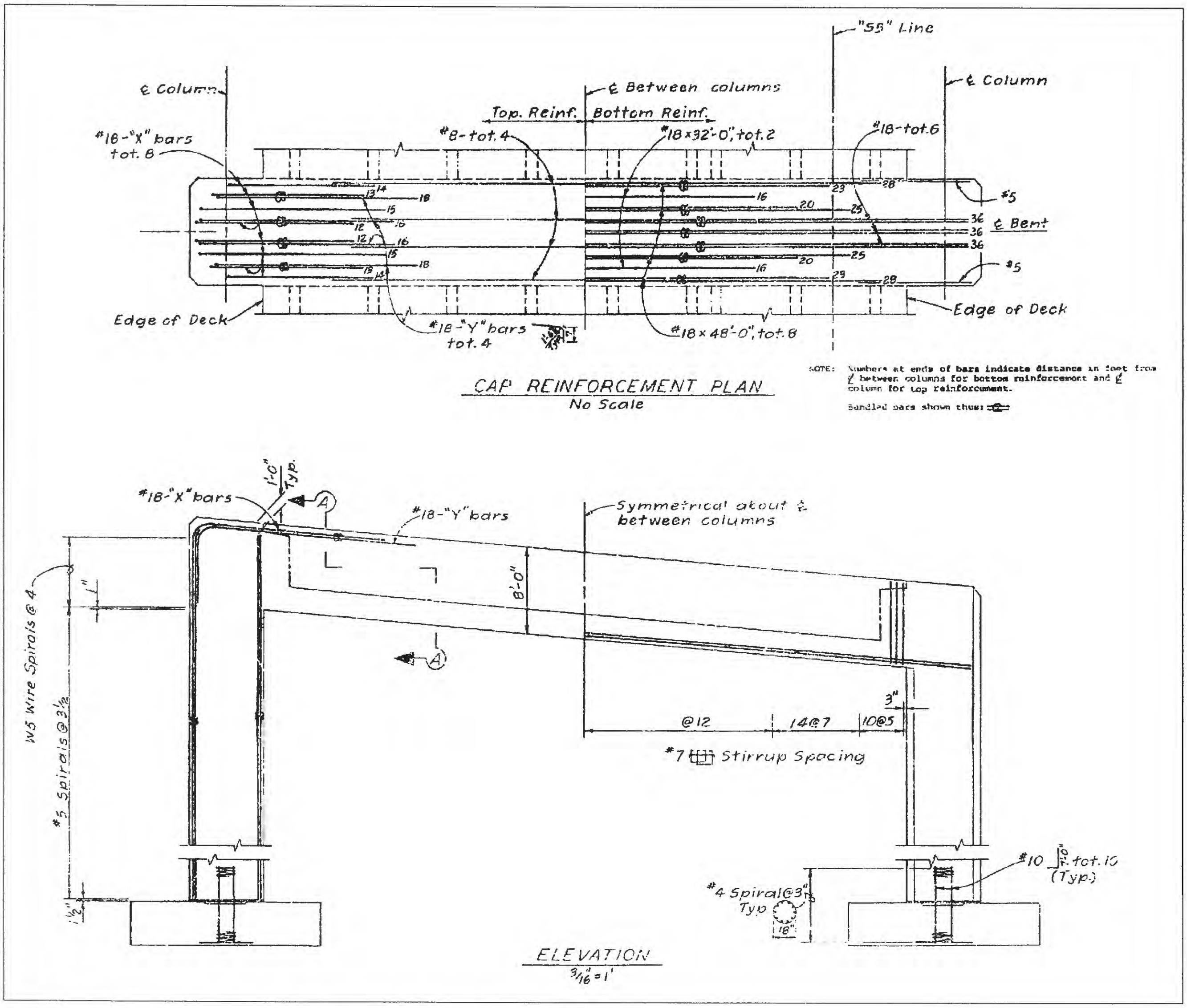

Figure 49.—Reinforcement details for Bent 38 of Route 980 Southbound Connector.

Bridge #33-126L/R / Rte 80 / Post Mile 2.41

Approximate Latitude & Longitude
N. Lat. 37° 49.6' W. Long. 122°18.4'

Epicentral Distance
58 miles

Peak Ground Acceleration	N/S	U/D	E/W
At Oakland Wharf	0.29	0.07	0.27

Length	Width	Skew	Year Built
5,445'	28.0'	varies	1937 & 1966

Main Span Type
Steel stringers (rolled sections)

Average Daily Traffic = 9,500 in 1984

Figure 50.—Port of Oakland (West Grand Avenue Viaduct) in 1992.

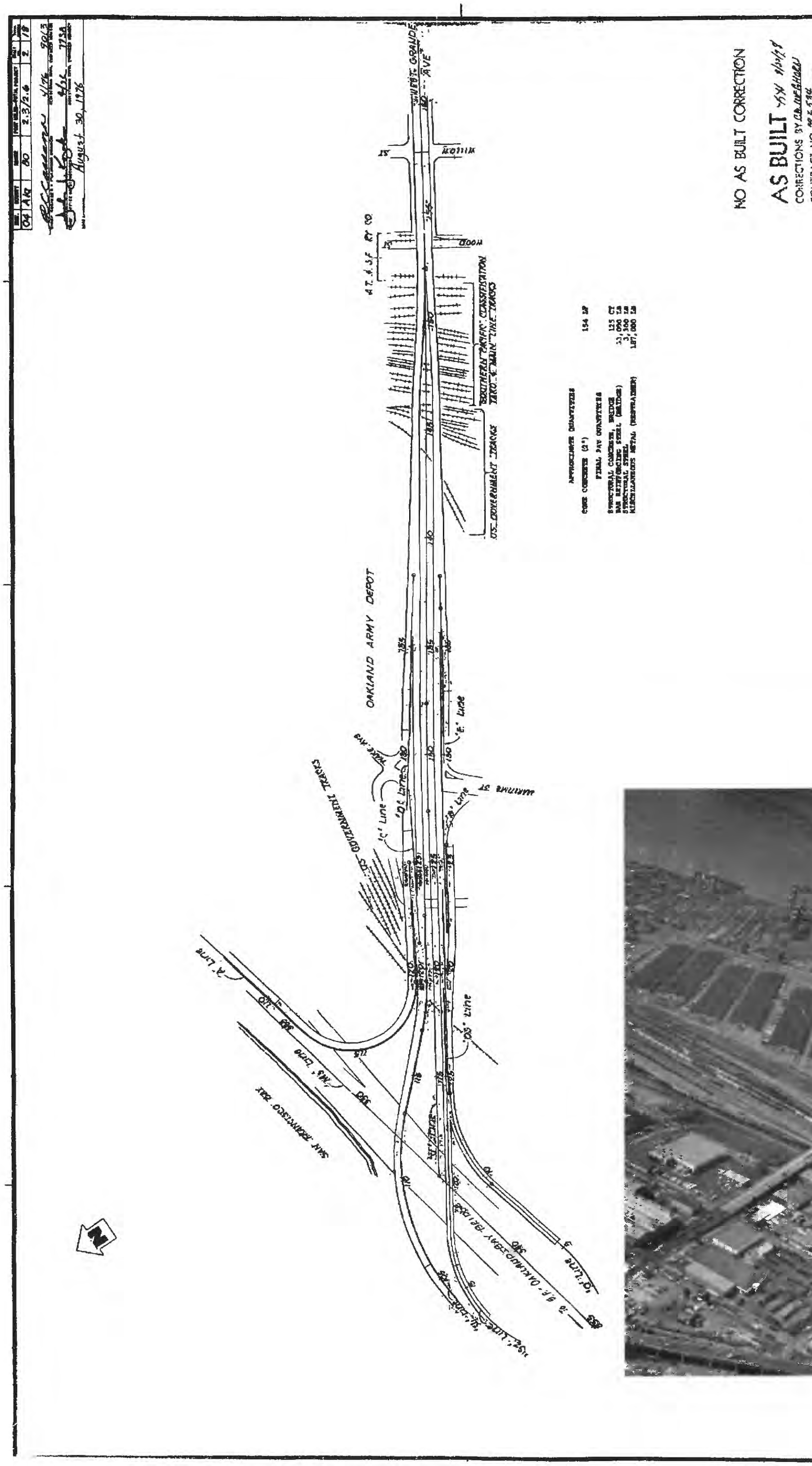

Figure 51.—Location of Port of Oakland Overcrossing.

Figure 52.—Plan drawing of the alignment for the Port of Oakland Overcrossing.

rial to the south. The bridge was retrofitted in the 1970's with cable restrainers and catcher blocks.

BRIDGE DAMAGE

The most noticeable evidence that a strong earthquake had occurred was the quantity of ejected sand around the columns at the north end of the bridge (fig. 53). There was substantial minor damage to this bridge during the earthquake. Anchor bolts that held the bottom steel girder flange to the bent cap were broken at a few locations. Also, some anchor bolts that held the steel bent caps to the columns were damaged. Mortar pads under girders had been broken. Keeper plates that held the steel bearings in place had been knocked loose at many locations. Some girders had moved sideways up to 14 inches, buckling the crossbracing. Surprisingly, cable restrainers had yielded at several locations (fig. 54). Steel girder flanges had buckled at two locations. There was also some concrete damage. The deck overhang had spalled from banging at expansion joints. There was some shear cracking at bent caps. The most serious damage was spalling at the base of some of the reinforced concrete single column bents (fig. 55). Table 11 summarizes the visual inspection of damage for each alignment. All the evidence suggests that the bridge moved primarily in an east-west (transverse) direction during the earthquake. After the earthquake, this bridge was used to haul heavy timber for shoring at the nearby Cypress Street Viaduct. However, it was later decided to close the structure for a few days while some repairs were made.

BRIDGE REPAIRS

Because of the damage to anchor bolts, welds, etc., the first order of business was to bring in a snooper (a truck with an articulated arm and large basket) and do a thorough ultrasonic inspection of steel girders and steel connecting elements. After the extent of damage was ascertained, repairs were initiated. Concrete cracks were repaired by epoxy injection. Concrete spalls were repaired with dry pack (fig. 56). For the spalling at some column bases, additional reinforcement was added to the repair. Because of the decision to replace this structure as part of the I-880 replacement project (fig. 40), repairs were done only to bring this structure back to its condition before the earthquake but not to retrofit it to survive a stronger event. However, all the existing bolts at the steel bent caps were replaced with high strength bolts, all the cable restrainers were replaced, and the other extensive repairs to this structure probably have improved its seismic performance. Because of the extent of minor damage to this long structure, the cost of repairs was almost $1 million.

REFERENCES

Bridwell, R.R., 1990, The San Francisco Oakland Bay Bridge and the Port of Oakland Overcrossing, Loma Prieta Earthquake Damage/Repair Report: Caltrans, 33 p.

Caltrans, Feb. 1937, July 1956. etc., As-built bridge plans for the Port of Oakland OC.

Stolarski, P., 1990, Inspection of structures damaged in the Loma Prieta Earthquake: Caltrans internal memo, 2 p.

Figure 53.—Sand boil at Bent 3 of the O line at Port of Oakland Overcrossing (view to south) (photograph by Li Hong Sheng).

Figure 54.—Restrainer damage at Port of Oakland Overcrossing (Photograph by Philip Stolarski).

Table 10.—*Alignment for Port of Oakland Overcrossing*

Alignment	Location
O line	Northbound I-80 to Port of Oakland.
S line	Southbound I-80 to Port of Oakland.
J line	Port of Oakland to southbound I-80.
A line	Port of Oakland to northbound I-80.
B line	Southbound J line to Maritime Street.
C line	Maritime Street to northbound J line.
D line	Northbound JL line to Maritime Street.
E line	Maritime Street to southbound JR line.
JL line	West Grand Avenue to northbound J line.
JR line	Southbound J line to West Grand Avenue.

Table 11.—*Summary of damage to Port of Oakland Overcrossing*

Alignment	Damage
O line	West approach settled about 4 inches. Sand was ejected from around the foundations of Bent 3 to Bent 10.
S line	Abutment 1 moved about 1 inch in both directions. Pavement around Bent 4 was cracked and sand was ejected around the column. Bent #6 has some cracks along top west corner.
J line	Approach settled 2 inches. Abutment 1 moved about a half inch transversely and 2 inches longitudinally. The steel girder bearings at Abutment 1 moved, spalling some concrete. Bents 2, 3, and 9 had concrete spalls at their connections with superstructure. Sand was ejected under Spans 2, 3, and 4.
A line	West approach settled about 5 inches. Pavement crack along north wingwall indicated soil movement. Abutment showed signs of movement and had spalls under bearings.
D line	Abutment 30 had its two northernmost bearing keeper plates broken.
JL line	Bent JL30 and JL35 had soil cracks around footings, indicating rocking. Bent JL35 had sand ejected around the footing. Some keeper plates at hinge were broken.
JR line	Bents JR28 to JR34 have soil cracks around footings and concrete spalls at column bottoms, indicative of flexural yielding in east-west direction.

Figure 55.—Spalling at southeast face of Bent 30 on JR line of Port of Oakland Overcrossing (photograph by Li Hong Sheng).

Figure 56.—Column damage and repair at Port of Oakland Overcrossing.

Figure 57.—The Distribution Structure in mid-1950's.

Br #33-61 L/R / Rte 580 / Post Mile 46.09

Approximate Latitude & Longitude
N. Lat. 37° 49.2' W. Long. 122°15.1'

Epicentral Distance
58 miles

Peak Ground Acceleration	**N/S**	**U/D**	**E/W**
At Oakland Wharf	0.29	0.07	0.27

Length	**Width**	**Skew**	**Year Built**
3,,996'	45.0'	Varies	1935

Main Span Type
Steel girders

Average Daily Traffic = 141.850

DISTRIBUTION STRUCTURE

DESCRIPTION OF BRIDGE

The Distribution Structure (figs. 57 and 58) is the first major interchange on I-80 after coming off the Bay Bridge. It allows traffic to switch between I-80, I-580, and the Cypress Viaduct. It is similar to the nearby Port of Oakland OC in its history and structure, as well as in the damage it sustained during the earthquake. The structure, built in 1935, is simple span steel girders on single-column and two-column bents supported on timber piles. The 1953 and 1958 structures are continuous reinforced concrete "T" girders on multicolumn bents with steel piles, concrete piles, and some spread footings. The log of test borings shows some areas of soft, black clay. The alignment for this structure is shown in figures 59 and 60 and table 12. A typical mid-1970's seismic retrofit was done connecting all the expansion joints with cable restrainers.

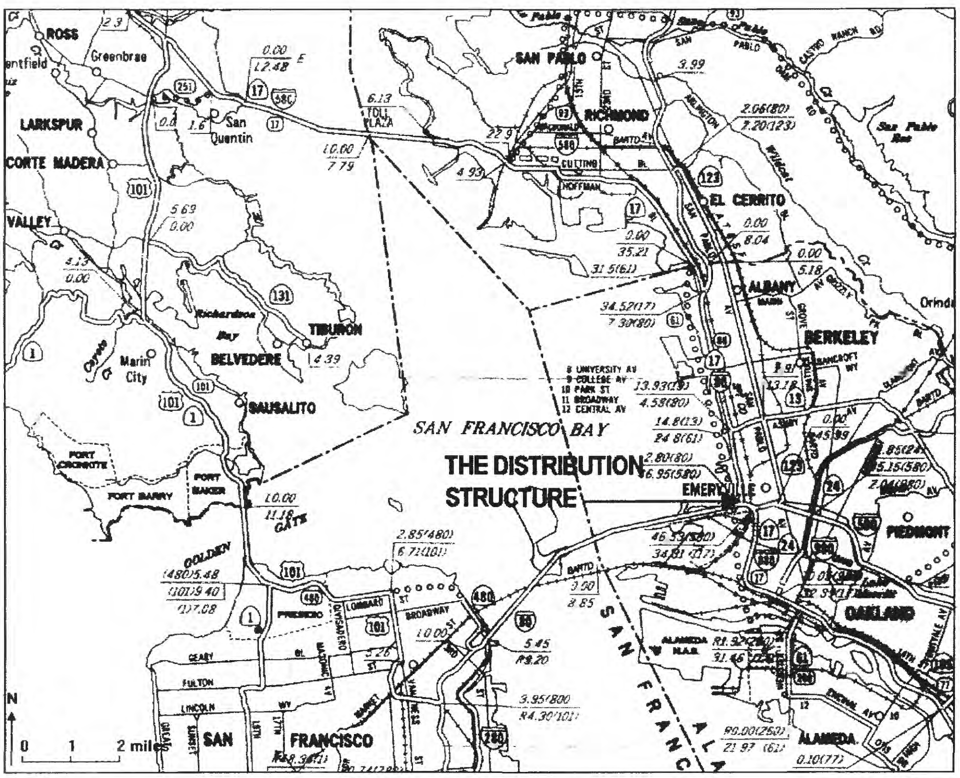

Figure 58.—Location of Distribution Structure.

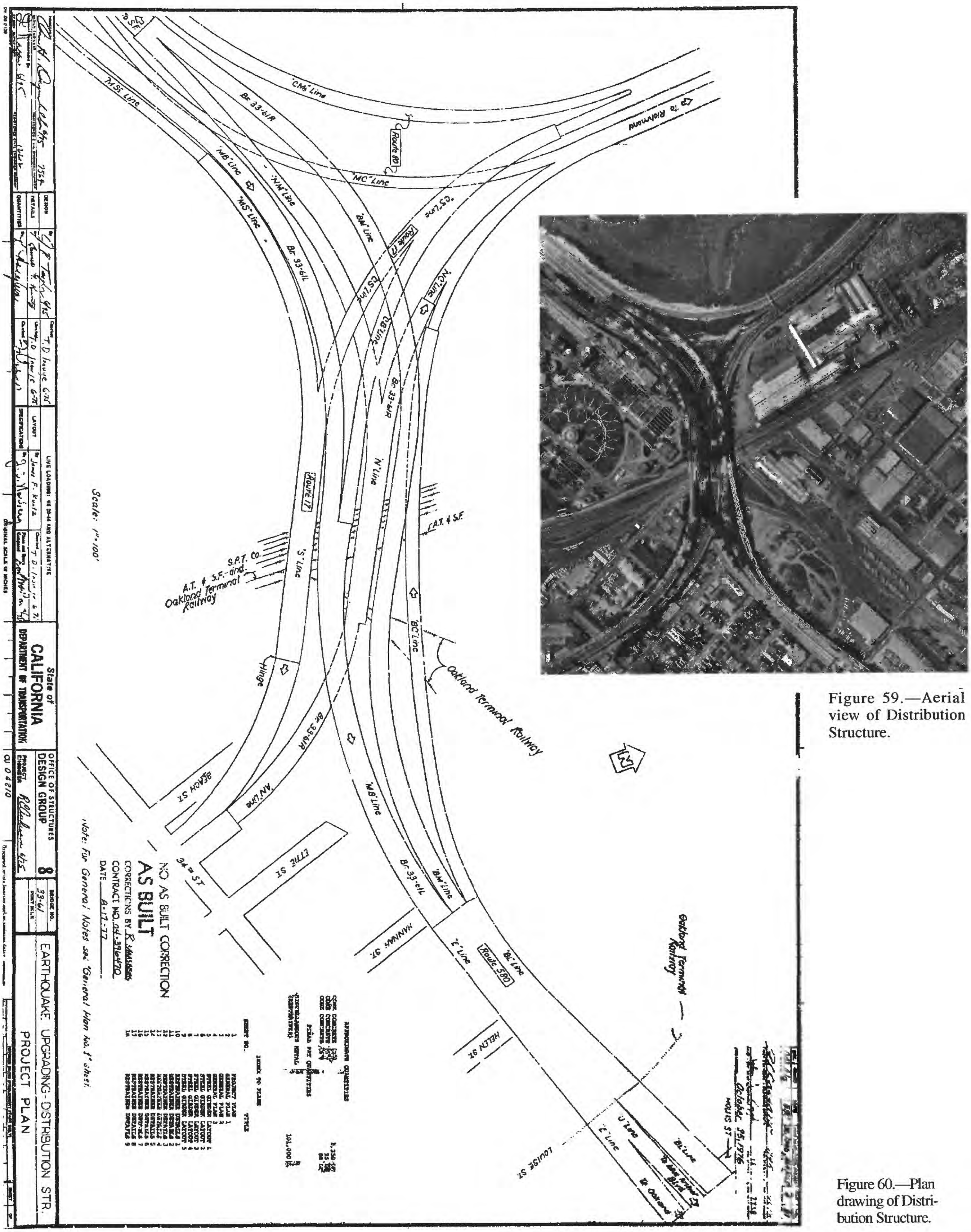

Figure 59.—Aerial view of Distribution Structure.

Figure 60.—Plan drawing of Distribution Structure.

BRIDGE DAMAGE

After the earthquake, this bridge was inspected by the Office of Structural Maintenance (Ng, 1989), the Post Earthquake Investigation Team (Zelinski, 1994), and by the Welding and Metals Technology Unit (Stolarski, 1990). Reports summarizing these investigations can be obtained from Caltrans. These reports (particularly the Structural Maintenance report) provide much more detailed information about bridge damage.

There was a great deal of minor and moderate damage to this structure. There was substantial damage to keeper plates, anchor bolts, and other steel hardware that kept the bearings in place. There was some minor cracking at the bottom of columns. Figures 61 and 62 show the results of transverse rocking of bents which caused concrete to spall and anchor bolts to deform at the column-to steel-bent-cap connection. The MB alignment had the most damage. The backwall for Abutment MB1 was severely cracked. The most significant damage was at Bent MB25, where there were many cracks at the bottom of the column, the bearing keeper plates were knocked off, the girder ends were bent, the bent cap web had a bow, and the superstructure shifted 3 inches to the north. Figure 63 shows the keeper plate damage. Table 13 summarizes the structural damage.

BRIDGE REPAIR

Emergency repairs were done on this bridge shortly after the earthquake (fig. 64). In 1990, a seismic retrofit was done as part of the phase 2 retrofit program. This retrofit was for an earthquake with a shorter return period since it was thought at the time that a new structure would replace this bridge in a couple of years. However, as of 1996 a new retrofit and widening is being designed for this structure. Some of this structure is being replaced as part of the new I-880 (Cypress Street Viaduct).

The 1990 retrofit consisted of bearing and restrainer replacements, steel shells that provided confinement for single column bents, and an elaborate footing retrofit that required the contractor to go under the existing footing and drill holes and attach high-strength rods to the existing piles so they could support the footing in tension. Figure 65 shows some of the typical retrofit details for this structure.

REFERENCES

Ng, Steve, 1989, Supplementary bridge report for the Distribution Structure: Caltrans.

Stolarski, P., 1990, Inspection Report: Caltrans internal memo.

Zelinski, Ray, 1994, Loma Prieta earthquake PEQIT Report: California Department of Transportation.

Figure 61.—Spalls and anchor bolt damage at Bent BC8 of the Distribution Structure.

Figure 62.—Closer view of damage at top of Bent BC8 of the Distribution Structure.

Table 12.—*Distribution Structure alignments*

Alignment	Location
CM	Route 80 SIB from Berkeley to Toll Plaza.
MC	Route 80 NIB from Toll Plaza to Berkeley.
NM	Route 880 NIB to Toll Plaza.
NC	Route 880 NB to Berkeley.
B M	Route 580 NB to Toll Plaza.
MB	Route 80 NB from Toll Plaza to Route 580 SB.
MS	Route 80 NB from Toll Plaza to Route 880 SB.
SA	Route 880 SB.
AN	Route 880 NB to Route 80.
CS	Route 80 SB from Berkeley to Route 880 SB.
CB	Route 80 SB from Berkeley to Route 580 SB.

Table 13.—*Summary of damage to the Distribution Structure (Zelinski, 1993)*

Alignment	Damage
CB	Bents CB2 and CB3 have the two interior steel girder keeper plates sheared off. Bent CB4 has at least one interior steel girder keeper plate sheared off.
CS	At Abut 1, Bent CS2, and Bent CS3, the structure has moved 2 inches in the east-west direction. Soil dropped 4 inches around the column at Bent CS2. Signs of foundation movement (soil cracks) from Bent CS2 to Bent CS5. Gap at Abutment 1 is about 4 inches wide and more than 3 feet deep at west side. Pavement crack along west side curb of approach.
BM	Approach has settled 6 inches. Pavement buckled at end of approach slab. Roadway shifted 2 inches to north.
MB	Footing rocking can be seen from the soil cracks around the footing. Series of soil cracks follow structure alignment. Concrete spalled at bottom of pinned bearing pad and above hole of restrainers at Bent MB9. Several bearing plates at Bent MB25 moved and fell.
CM	Roadway crack follows alignment of CM line toward Toll Plaza. Liquefaction can be seen along roadside and cracks. North side of roadway shifted 4 inches to north and has 6 inches settlement.

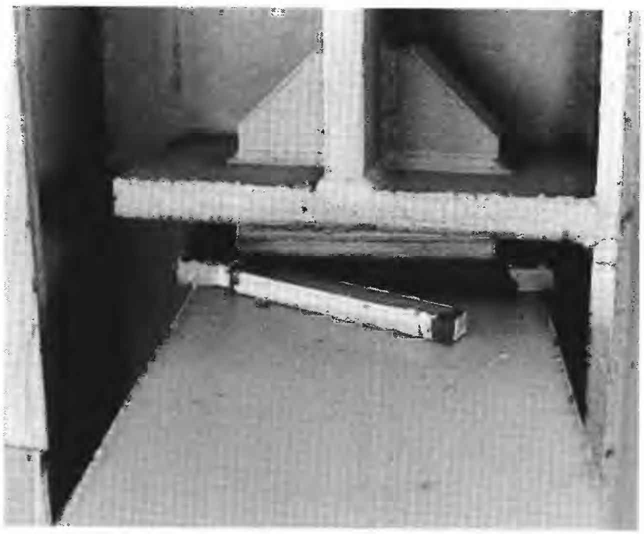

Figure 63.—Keeper Plate Damage on the Distribution Structure (photograph by Li Hong Sheng).

Figure 64.—Emergency Repairs to the Distribution Structure.

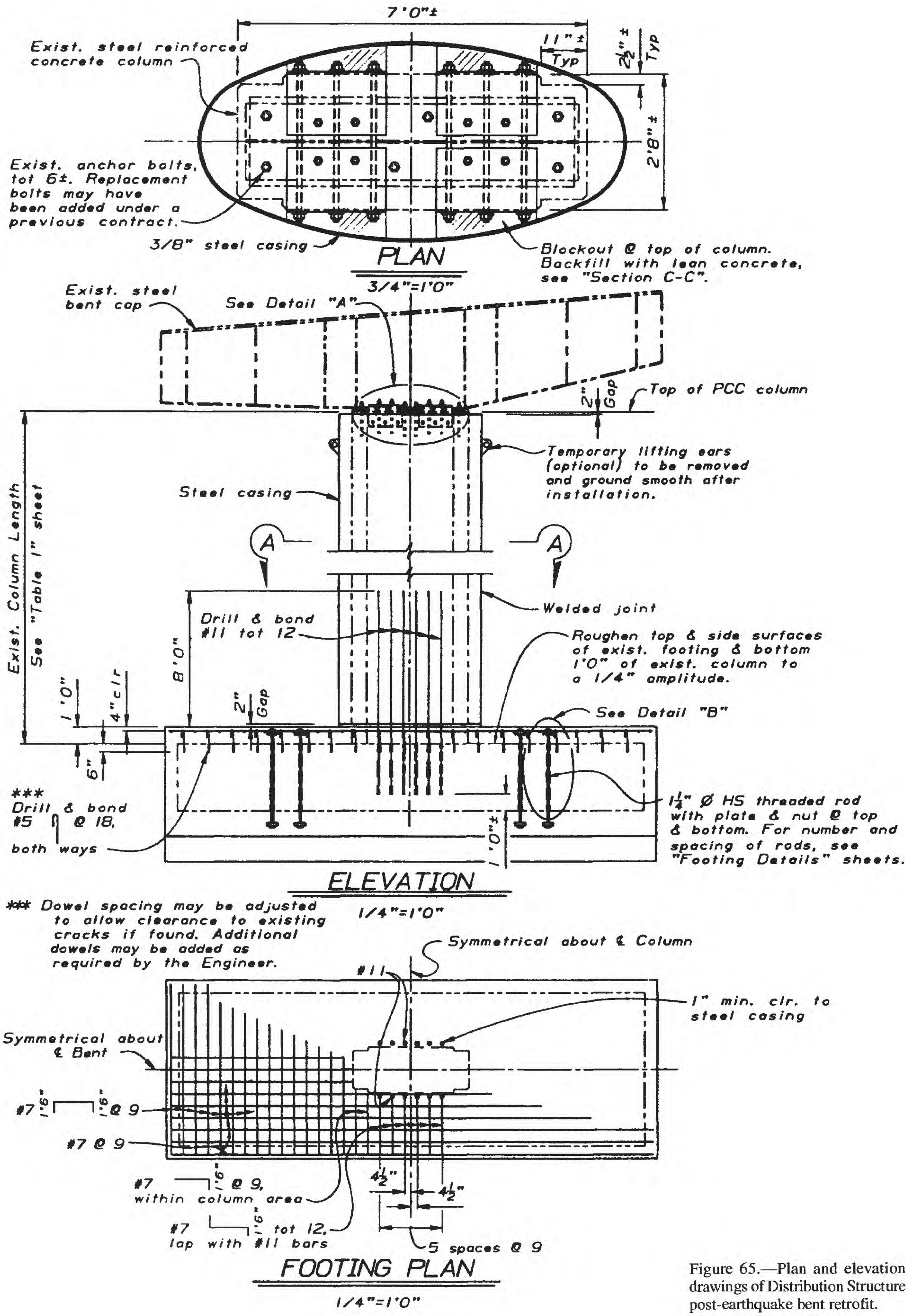

Figure 65.—Plan and elevation drawings of Distribution Structure post-earthquake bent retrofit.

Figure 66.—Fifth Avenue Overhead (before 1963 widening).

Br #33-027 / Rte 880 / Post Mile 30.38

Approximate Latitude & Longitude
N. Lat. 37° 47.5' W. Long. 122° 15.6'

Epicentral Distance
56.4 miles

PGA	N/S	U/D	E/W
Oakland 24-story bldg.	0.18	0.04	0.14

Length	Width	Skew	Year Built
2,554'	109'	Varies	1948

Main Span Type
Continuous steel 'I' girders

Average Daily Traffic = 176,300

FIFTH AVENUE OVERHEAD

DESCRIPTION OF STRUCTURE

The Fifth Avenue Overhead (figs. 66 to 68) is a steel girder bridge with a concrete deck on mostly concrete (and a few steel) multicolumn bents supported on pile foundations in the City of Oakland. It was built in 1947, widened in 1963, and seismically retrofit in 1985 (fig. 69). The consultant responsible for the retrofit recommended that the bridge be replaced because of soil problems, an inadequate substructure, fatigue, etc. However, a superstructure retrofit was done instead that provided cable restrainers at the steel girder hinges.

BRIDGE DAMAGE

This bridge was inspected by Structural Maintenance on November 16 and 17, 1989, and again on December 12 and 13, 1989. There was a walk-through examination of the substructure and a random inspection of the bearings and hinges using a ladder, a pair of binoculars, or at some locations a personnel lift. There was a cursory drive-by inspection of the deck surface and a binocular inspection of overhangs at the transverse joints and at the underside of longitudinal joints. Significant damage was limited to the interior supports from Bent 30 to Bent 40, with some minor damage elsewhere (figs 70 to 74). Table 14 summarizes the damage. The most unusual damage was to the steel wide-flange columns at Bents 36 to 39 (figs. 75, 76). These columns showed indications of one or two cycles of inelastic deformation and plastic hinging above the pedestals. This damage may have been exacerbated by the pedestals and railroad collision walls that made the bases very rigid.

BRIDGE REPAIR AND REPLACEMENT

After the earthquake, Caltrans drew up plans, and repairs were made (figs. 77, 78). Because of vulnerabilities on this structure, consultants were hired to design a replacement structure. However, a variety of problems has pushed this project back to the year 2002 or beyond.

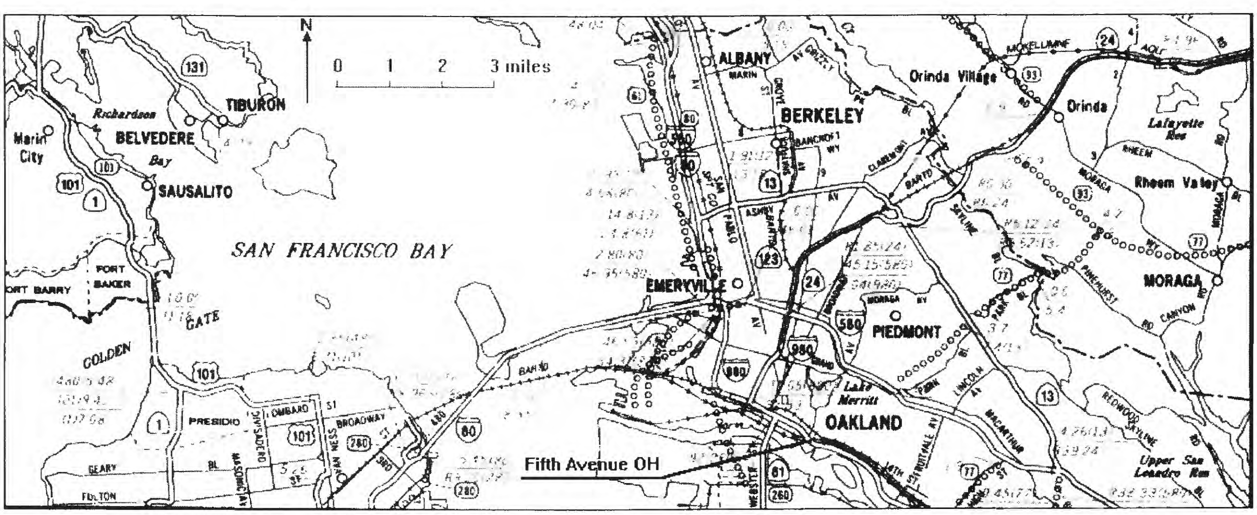

Figure 67.—Location of Fifth Avenue Overhead.

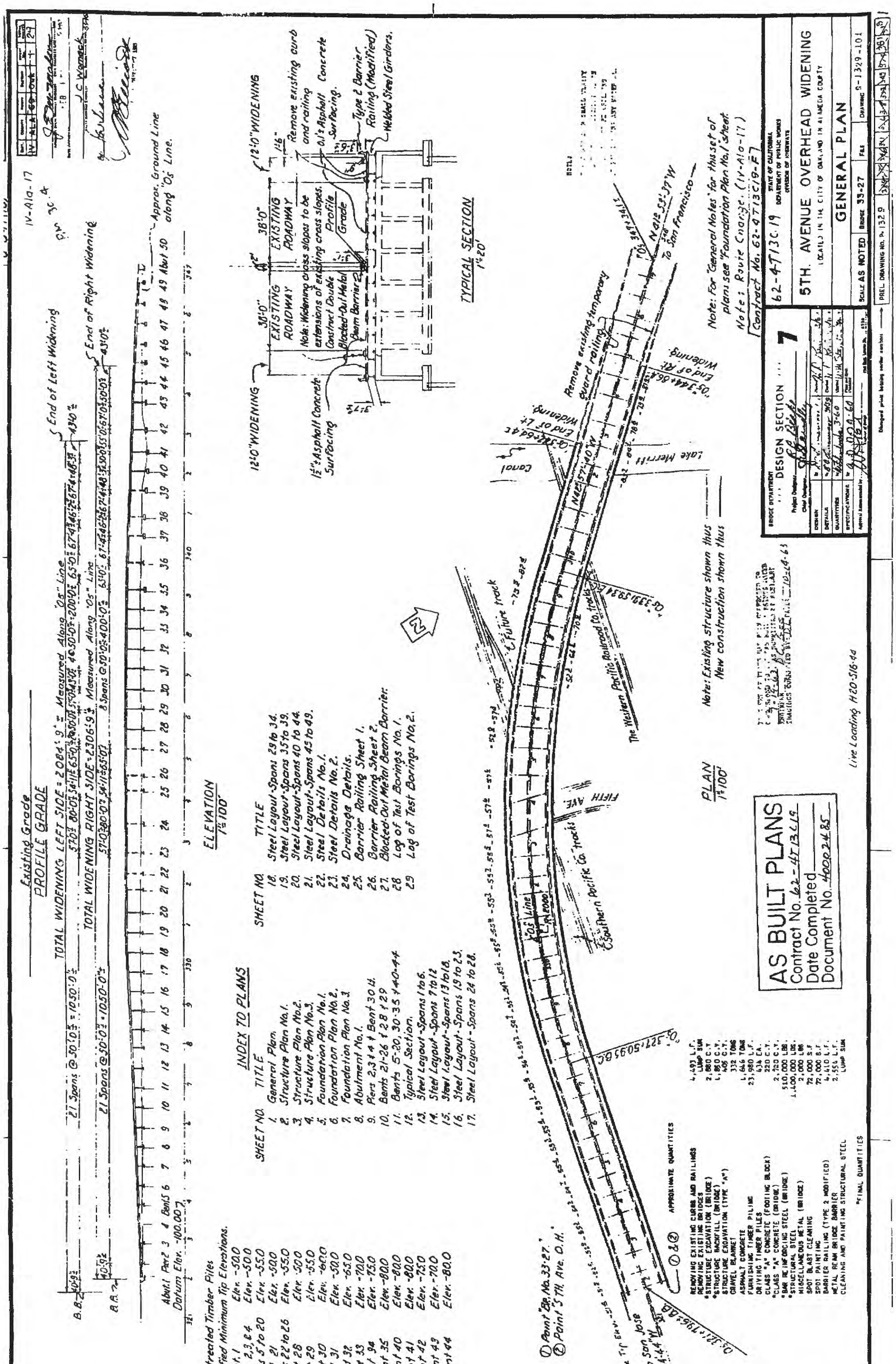

Figure 68.—General Plan drawing for 1963 Fifth Avenue Overhead widening.

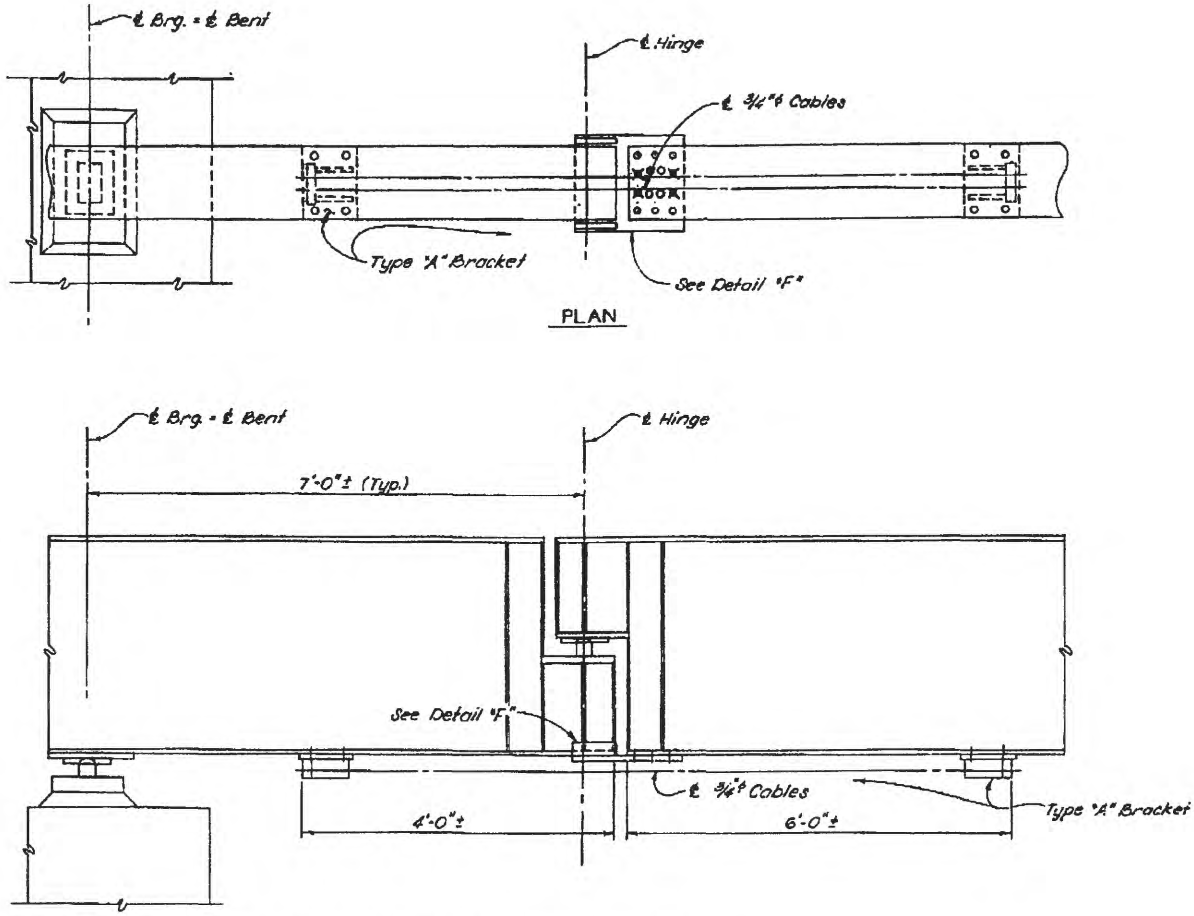

Figure 69.—Elevation drawing for the 1985 retrofit of Fifth Avenue Overhead.

Figure 70.—Fifth Avenue Overhead at time of the earthquake (courtesy of Earthquake Engineering Research Institute).

Figure 71.—Spall at column base for Bent 35 of Fifth Avenue Overhead (courtesy of Earthquake Engineering Research Institute).

Table 14.—*Summary of damage to Fifth Avenue Overhead*

Location	Damage
Abut 1 to Bent 24	No damage.
Bent 25	Cracks to railroad collision wall. Deck spall at hinge in Span 24.
Bent 26	Cracked collision wall. Hairline crack of Column 3 at top of wall.
Bent 27	Hairline crack in Column 1 similar to Bent 26.
Bent 30	1/4" opening in soil around Column 7 and diagonal crack in concrete pad at bottom of column.
Bent 31	Spalls on all faces of Column 8 about 6" above ground line.
Bent 32	Column 8 cracked and spalled near ground and at bent cap.
Bent 33	Columns have cracks near the ground and near the cap.
Bent 34	Similar to Bent 33 but more severe.
Bent 35	Similar to Bent 34 but even more severe.
Bent 36 to 39	At this location there are steel three-column bents embedded in concrete pedestals and connected to skewed railroad collision walls. There was damage at the column to cap connections, and plastic hinging of some of the steel columns above the pedestals.
Bent 40 to 42	These columns sit in Merritt Lake Canal. The earthquake spalled concrete previously loosened by corrosion damage.
Bent 46	Damage to superstructure hinge in Span 46.
Bent 47	Slight damage to keeper rings at top of rockers at hinge.

Figure 74.—Damaged hinge at Bent 36 of Fifth Avenue Overhead (courtesy of Earthquake Engineering Research Institute).

Figure 72.—Damage to railroad collision wall of Fifth Avenue Overhead (courtesy of Earthquake Engineering Research Institute).

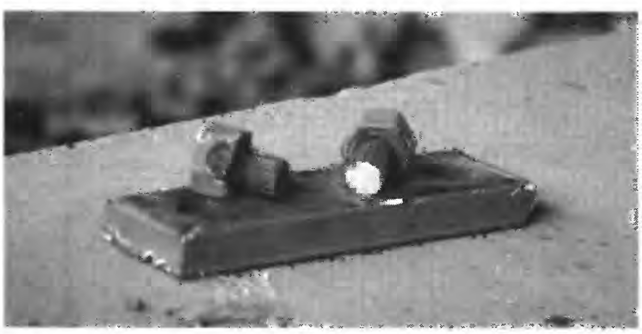

Figure 73.—Broken keeper plate at Bent 35 of Fifth Avenue Overhead (courtesy of Earthquake Engineering Research Institute).

Figure 75.—Location of steel columns at Fifth Avenue Overhead (courtesy of Abolhassan Astaneh-Asl).

Figure 76.—Steel column damage at Fifth Avenue Overhead (courtesy of Abolhassan Astaneh-Asl).

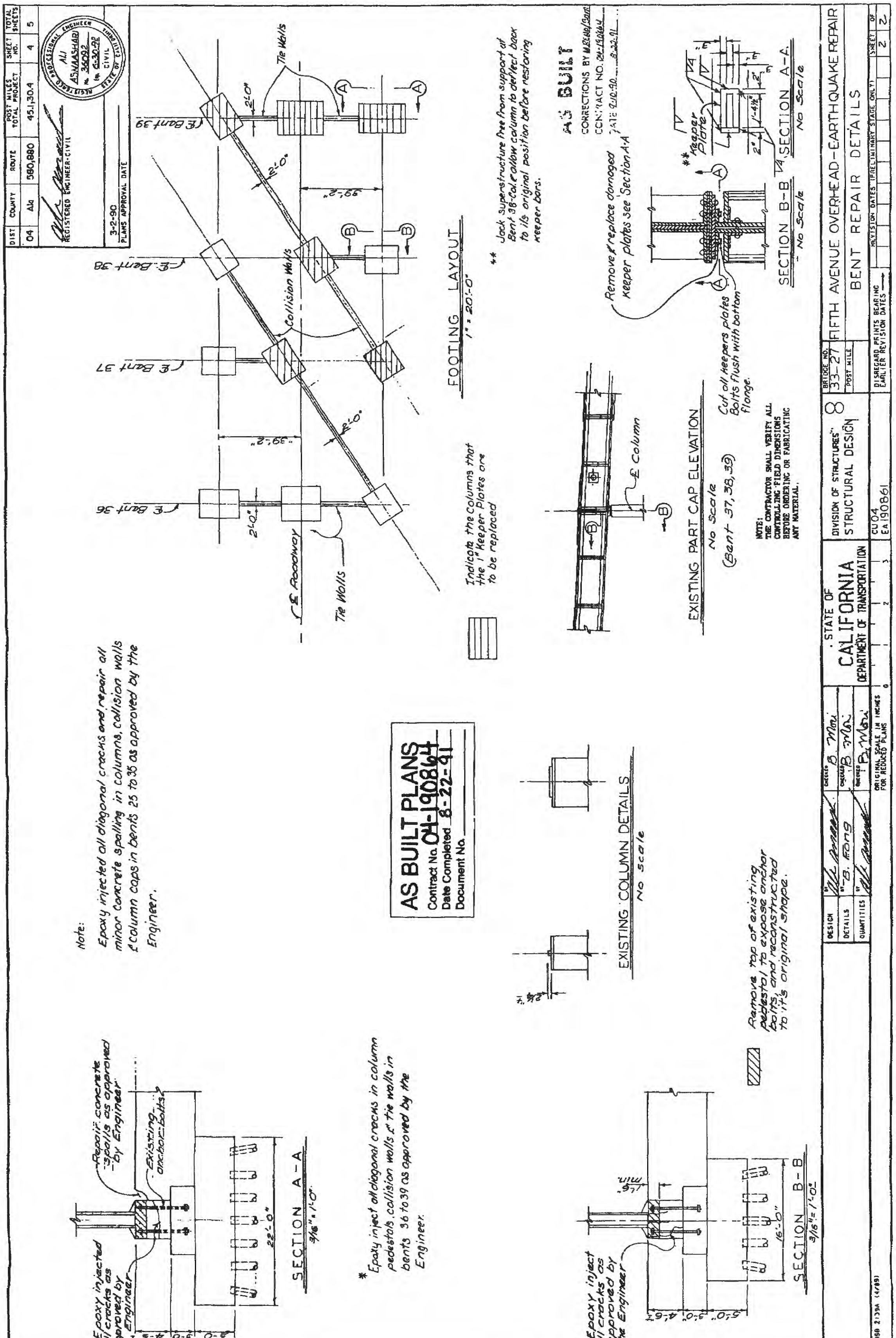

Figure 77.—Bent repair detail illustrating how the railroad collision walls are attached to Fifth Avenue Overhead.

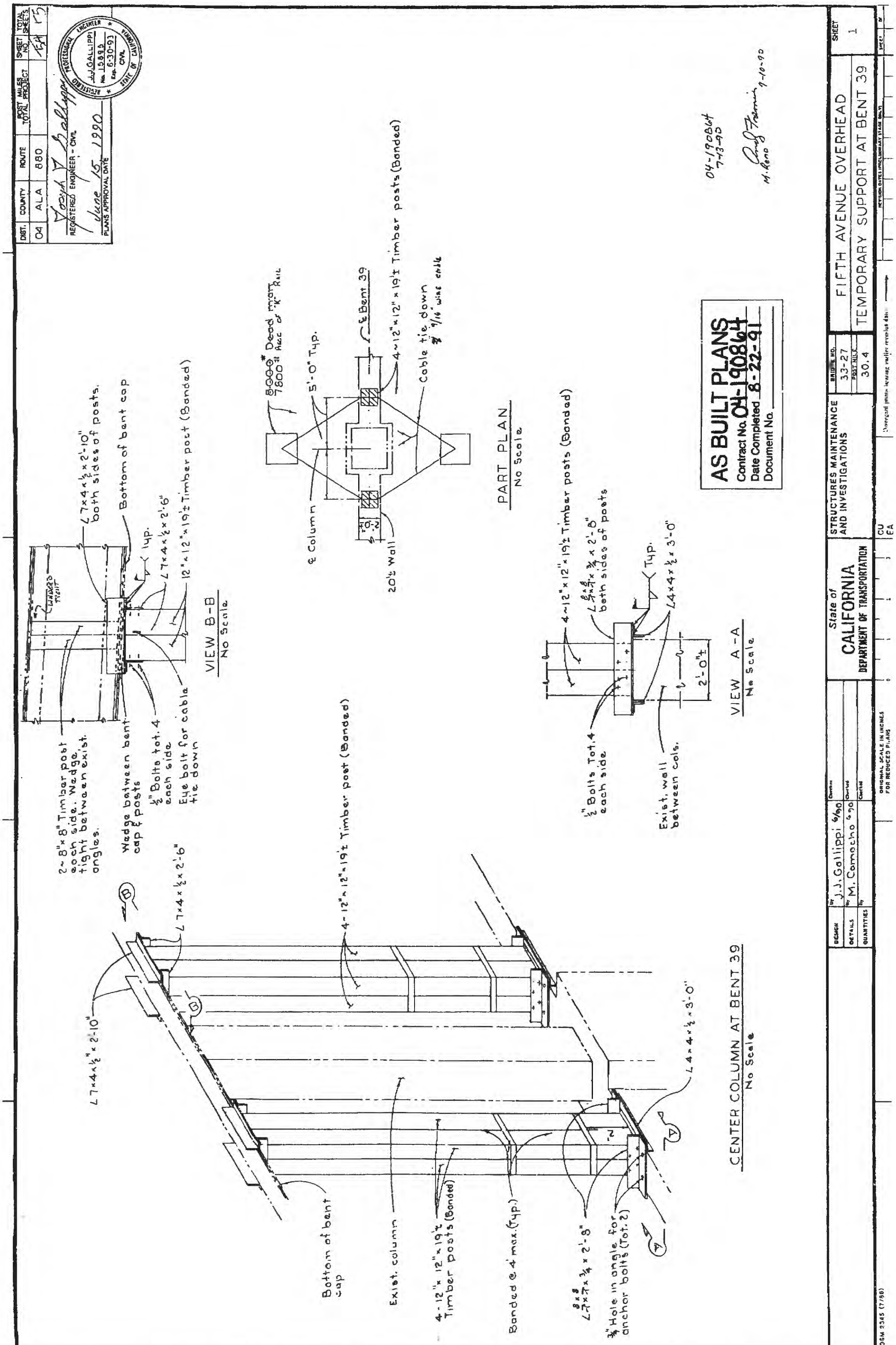

Figure 78.—Bent repair detail illustrating the use of falsework to support superstructure on Fifth Avenue Overhead.

Figure 79.—Location of damaged viaducts in eastern San Francisco.

MAJOR BRIDGE DAMAGE IN THE CITY OF SAN FRANCISCO

SAN FRANCISCO VIADUCTS

One of the most significant characteristics of the earthquake was that much of the bridge damage occurred 60 miles away in the cities of Oakland and San Francisco. Figure 40 provides a view of that area of Oakland that suffered severe bridge damage. The six damaged San Francisco viaducts can be located on figure 2. Figure 79 is a photograph of these six San Francisco viaducts. Caltrans has a contract with several aerial photography firms to survey for highway damage after every large earthquake. Many of the aerial photographs in this report come from those surveys, which provide vertical and oblique aerial photos of all the highways impacted by the Loma Prieta earthquake.

The following pages provide information on damage to six San Francisco viaducts. All of these viaducts have at least a small portion or the whole structure supporting a double deck. Many of these elevated expressways were torn down after the earthquake and never replaced. Others are awaiting political decisions before they are rebuilt. This process is in sharp contrast to the 1994 Northridge earthquake, for which the city of Los Angeles spared no expense in helping Caltrans replace all the damaged structures in less than a year and reflects the different attitudes the two cities have toward their elevated expressways.

SOUTHERN FREEWAY VIADUCT

DESCRIPTION OF STRUCTURE

This long viaduct carries I-280 traffic from China Basin to the Alemany Interchange in the City of San Francisco (see figs. 2, 81). It was built in three contracts. Contract 14T13C17 was the southernmost contract which was begun in 1962. Contract 14-207034 was for the middle section, begun in 1964. Contract 14-207044 was for the north end of the viaduct, begun in 1966.

The configuration of decks and frames varies from a two-frame double-deck structure in the south to a very wide single-deck structure in the north. Very long outrigger bent caps are found where the structure changes from a double-deck to a single-deck bridge. Consequently, the bent geometry varies considerably. The double-deck bents have prestressed top bent caps and a variety of pinned and moment-resisting connections (fig. 81). Like many of the long bridges around the bay, one end of this structure is supported on sandstone (the south end), which rapidly descends, leaving most of the structure on Bay mud. The structure is supported by 14BP102, 10BP42, and CIDH piles. There are also some spread footings. The bridge was retrofitted in the 1970's with restrainer cables (fig. 82).

BRIDGE DAMAGE

All of the earthquake damage occurred to work done under Contract 14-207034, which is the middle (double deck) portion of the bridge. There was column damage from Bent 46 to Bent 58. There was also some superstructure damage including deck spalls at spans 17, 24, 42, and 43 and some transverse cracks in Span 32. Almost all the expansion joints were opened about 1.5 to 2 inches.

Bent 46 Left Column.—The column was cracked just below the upper deck reinforced concrete bent cap. A nearly vertical crack runs across both the north and south faces for about 15 feet.

Bent 47 Left Column.—There are three vertical cracks where the lower deck bent cap meets the left column. They run from the top to the bottom of the cap. There is a spall (2 ft.by 2 in.by 6 in.) at the top face of the cap where it meets the column.

Bent 48 Left Column.—A large crack occurred all the way though the top of the left column (fig. 83). The top of the column is not monolithic with the outrigger bent cap. This pin connection clearly began to fail due to movement of the top deck. The framing diagram (fig. 84, 85) shows how the left column must carry the upper deck transversely.

Bent 51 Right Column.—Diagonal cracks formed where the column frames into the bent caps (fig. 80, 86).

Bent 52 Right Column.—Similar to Bent 51, a shear transfer crack formed at the bottom bent cap.

Bent 58.—The barrier rail and overhang were damaged by banging against a column.

REPAIR AND RETROFIT

After the earthquake, temporary supports were placed at the damaged bents. Then a temporary retrofit was completed on vulnerable columns (fig. 87). However, it was rejected by the Peer Review Panel as not providing sufficient ductility, especially at the column to cap connections (the Governor's Board of Inquiry recommended that major bridge projects should be peer reviewed). The eventual retrofit (fig. 88) includes edge beams to pick up flexural moments that otherwise would be carried as torsion in the bottom bent cap, oversized joints for shear transfer, prestressed top bent caps on pinned connections, and new, very ductile columns and footings.

REFERENCES

Orsolini, Greg, 1994, Seismic retrofit of the I-280 Southern Freeway Viaduct: DeLeuw Cather, 11 p.

Liu, David, 1993, Seismic retrofit design for the Southern Freeway Viaduct: Imbsen & Assoc. 23 p.

Dameron, R.A., and Kurkchubasche, I.R., 1994, 3D F.E. analysis of joint shear in Bent A-78 of the Southern Viaduct: Anatech Research Corp, 4 p.

Br #34-46 / Rte 280 / Post Mile 4.40

Approximate Latitude & Longitude
N. Lat. 37° 44.9' W. Long. 122°23.5'

Epicentral Distance
56.9 miles

Peak Ground Acceleration	**N/S**	**U/D**	**E/W**
San Francisco Int. Airport	0.24	0.05	0.33

Length	**Width**	**Skew**	**Year Built**
21,588'	Varies	Varies	1964

Main Span Type
Concrete box girder bridge

Average Daily Traffic = 27,000

Figure 80.—Damage to Bents 51 and 52 of Southern Freeway Viaduct.

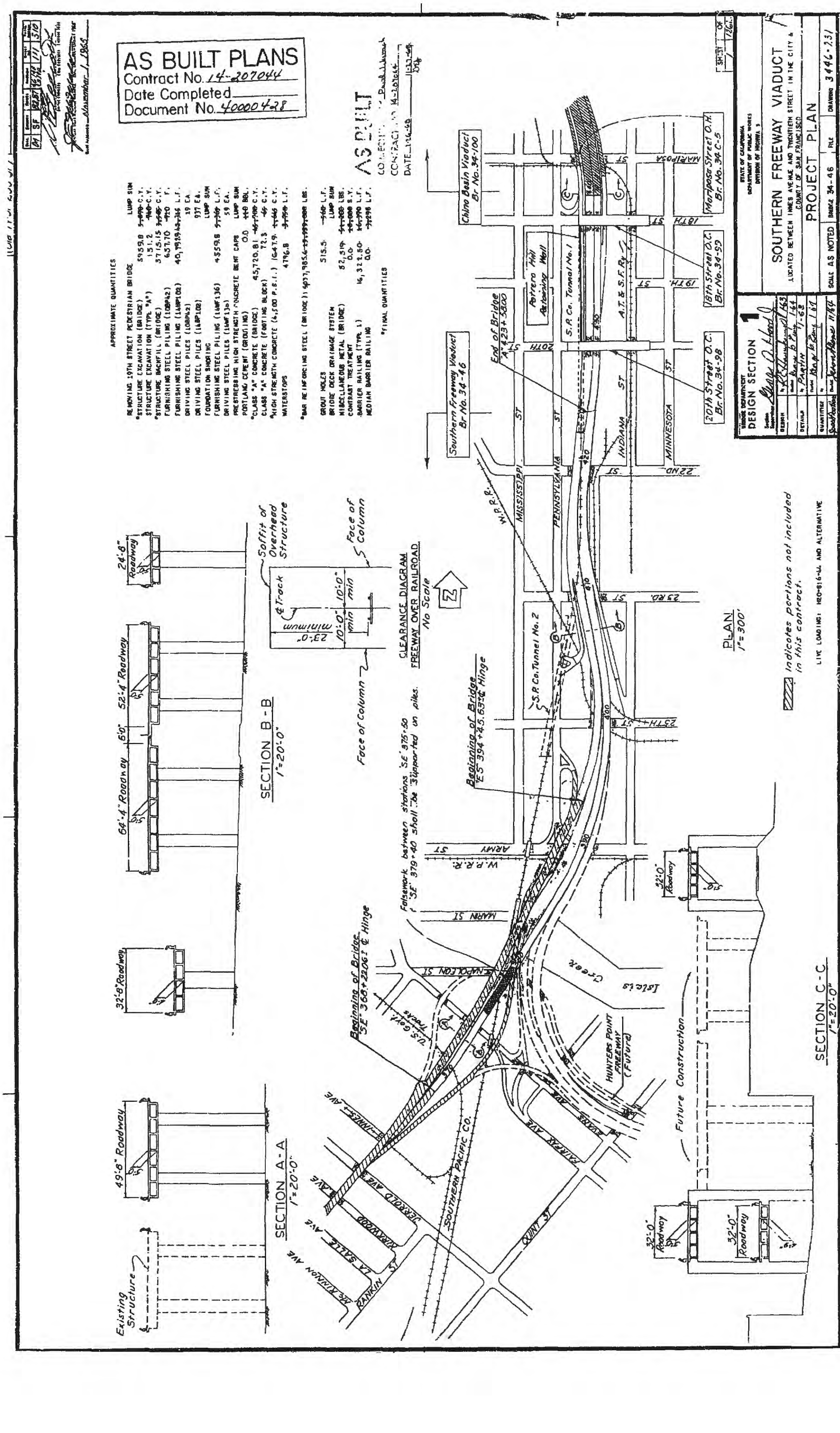

Figure 81.—General Plan drawing of Southern Freeway Viaduct showing details for north end.

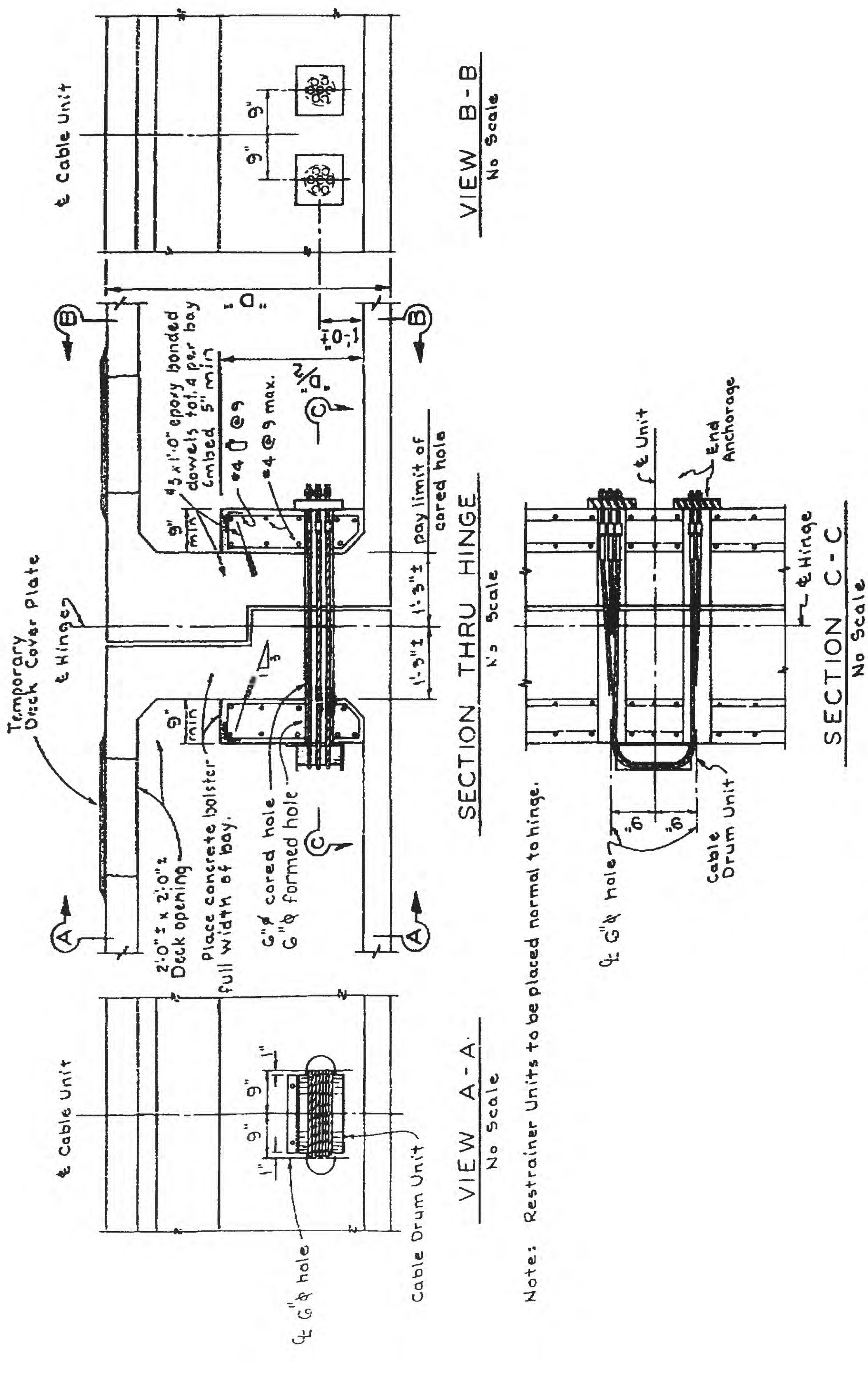

Figure 82.—Section drawing of 1972 restrainer retrofit for Southern Freeway Viaduct.

Figure 83.—Damage to Bent 48 of Southern Freeway Viaduct.

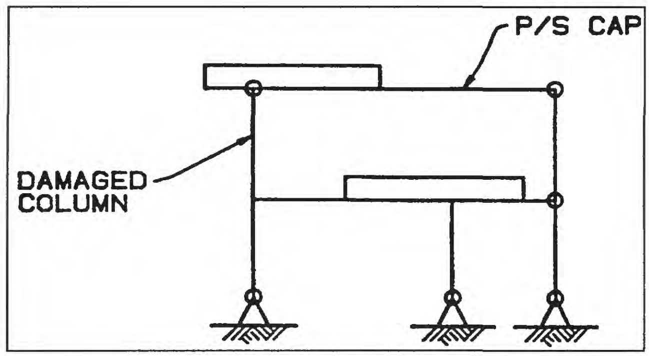

Figure 85.—Schematic drawing of Bent 48 frame at Southern Freeway Viaduct.

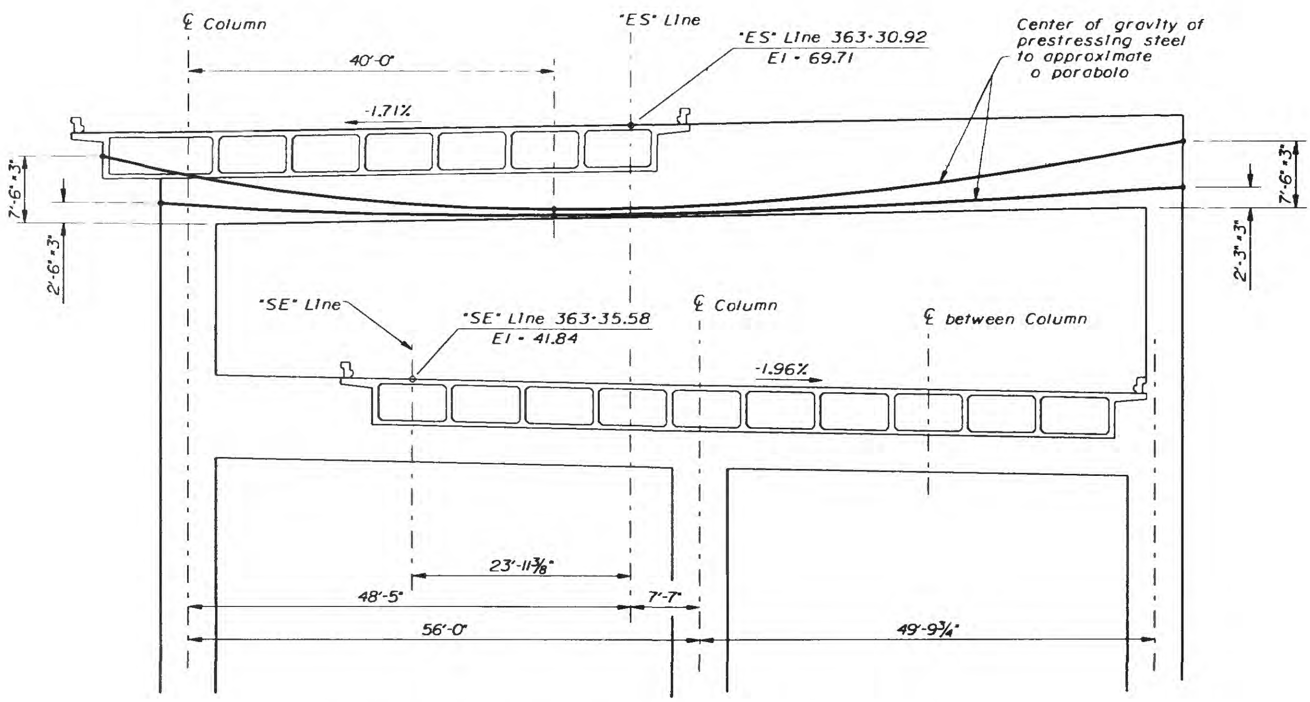

Figure 84.—Section drawing of Bent 48 at Southern Freeway Viaduct.

Figure 86.—Damage at Bent 51 of Southern Freeway Viaduct.

Figure 87.—Retrofit of the Southern Freeway Viaduct later rejected by the Peer Review Panel.

Figure 88.—Completed retrofit of the Southern Freeway Viaduct (photograph by Charles Sikorsky).

Br #34-100 /Rte 280 /Post Mile R6.61

Approximate Latitude & Longitude
N. Lat. 37° 45.8' W. Long. 122°23.5'

Epicentral Distance
57.7 miles

Peak Ground Acceleration	**N/S**	**U/D**	**E/W**
575 Market Street	0.08	0.06	0.11

Length	**Width**	**Skew**	**Year Built**
6,364'	137.7'	Varies	1971

Main Span Type
Concrete Box Girder Bridge.

Average Daily Traffic = 35,700

Figure 89.—The China Basin Viaduct before the earthquake.

CHINA BASIN VIADUCT

DESCRIPTION OF BRIDGE

China Basin Viaduct (fig. 92) is the northernmost portion of the elevated I-280 in San Francisco. It is the least typical of the six damaged San Francisco viaducts because it is single level for much of its length, only becoming two level for the tall ramps near the north end. There are three alignments used to define this structure. The "A" line is the main alignment that dead ends to the left of figure 90 (it was originally intended to connect to the Bay Bridge). The "S2" line is the shorter, southbound ramp below the "A" line in the figure. The "N1" line is the tall, northbound ramp. Figure 91 provides a plan drawing of the structure. The bridge's superstructure is a combination of cast-in-place, reinforced concrete box girders, cast-in-place prestressed box girders, and (predominantly) precast prestressed concrete I-girders (that were used to span over the many railroad tracks). The bents are multicolumn, with a few single column and outrigger bents for the ramps. The bent caps are reinforced concrete with some prestressed bent caps at the ramps. The foundations are a variety of spread footings, cast-in-drilled-hole (CIDH) pile footings, and driven steel pile footings. The soil profile (fig. 92), which was created after the earthquake, shows soft Bay mud under the entire structure.

Figure 90.—China Basin Viaduct.

Like many of the other long viaduct structures, this bridge was constructed under one contract for the northern portion of the viaduct and a later contract for the southern portion in the early 1970's and as a whole was without many seismic details. A

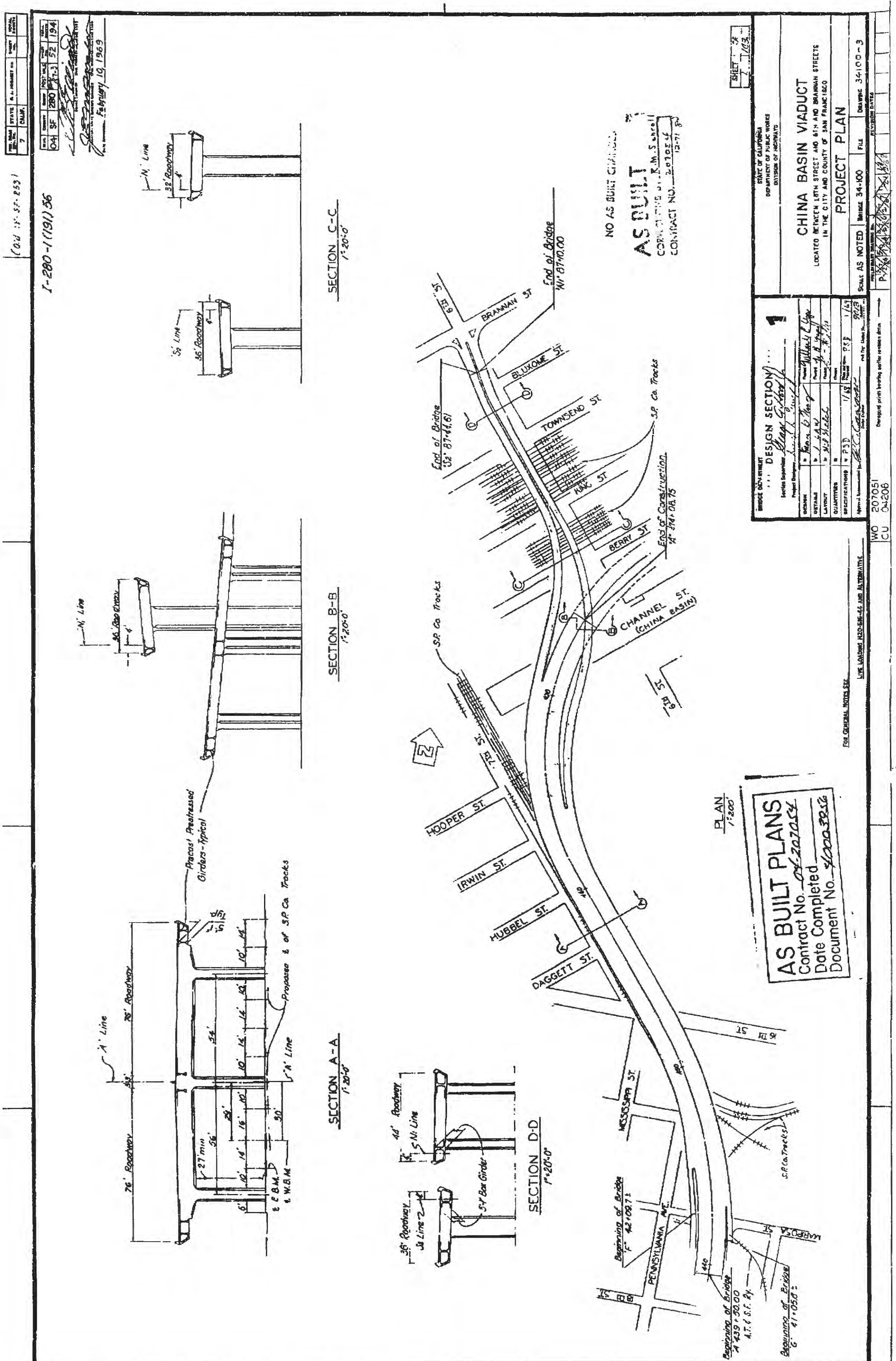

Figure 91.—General Plan drawing for China Basin Viaduct.

retrofit contract in the 1980's completed the job of restraining this structure.

BRIDGE DAMAGE

The entire structure showed signs of significant movement during the earthquake. The most serious damage was to outrigger knee joints similar to the damage at the other San Francisco viaducts. Since the ground motion was relatively small, it suggests that outrigger bents require much better joint details to survive the maximum credible event. There was also signs of large bridge motions along the entire viaduct. Table 15 provides the openings of the expansion joints measured after the earthquake.

Bent A-11 and A-14.—There was damage to the end diaphragms at Bents A-11 and A-14. These were locations where the precast I girders were joined over the bents. The hinge bolts punched through the diaphragms at these locations.

Bent A-12.—The outrigger bent cap at A-12 suffered shear cracks on both sides of the bridge deck (fig. 93).

Bent A-31.—The tops of the three columns were fixed to the bent cap, which suffered some minor cracks at the top inside faces of the exterior columns due to transverse movement.

Bent A-32.—This bent is similar to A-31 except that the right column here supports an outrigger bent cap. The outrigger cap had significant diagonal cracks that extended from the column up to bridge deck, possibly due to longitudinal movement of the superstructure (fig. 94).

Bent N-35.—This outrigger bent had significant diagonal cracks where the columns frame into the cap and where the cap frames into the superstructure. It is reported that the cracks from the superstructure to the cap had existed since construction but may have worsened due to the earthquake. The cracks around the columns were the result of transverse movement during the earthquake. There were 2 inches of longitudinal movement to the hinge just north of this bent. Also one could see the outline of the column footings in the soil and pavement around these columns (fig. 95).

Bent S2-41.—This bent has a shorter outrigger cap with two fixed columns and precast I-girders framed to the cap. Again, there was cracking that began where the bottom of the cap joined the column and extended to the bottom of the exterior girder (fig. 96).

Hinge A-44.—This hinge had pre-earthquake problems of excessive opening due to prestress shortening and thermal expansion. It appears that the earthquake further aggravated that problem. The earthquake restrainers have punched through the diaphragm at some of the bays (fig. 97).

Abutment FN-45.—The approach fill behind this abutment settled about 12 inches. There was also damage to the barrier rail above the abutment wingwalls.

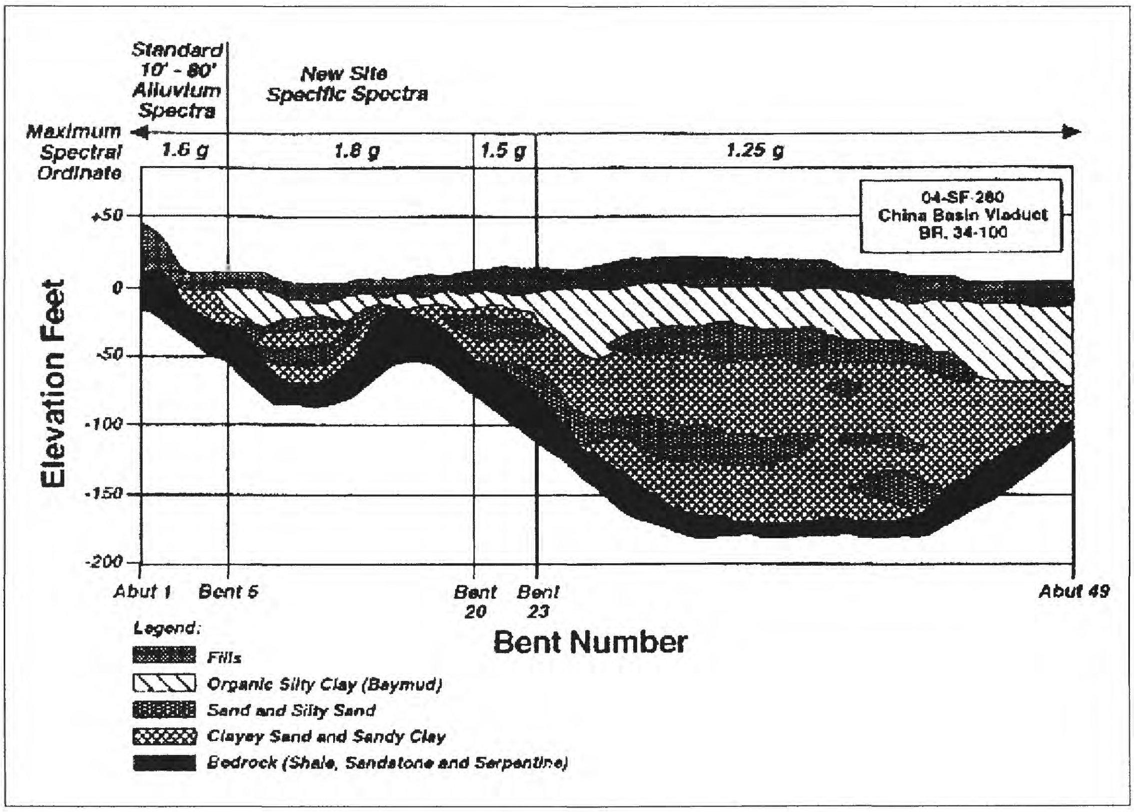

Figure 92.—Soil profile for China Basin Viaduct (courtesy of CH2M Hill).

REPAIR AND RETROFIT

There were several repair/retrofits done to this structure after the earthquake. Immediately after the earthquake, crews erected timber shoring under damaged portions of the viaduct. Repairs were designed by Caltrans for the damaged bents and to repair the hinges and abutment, which allowed the structure to reopen. In the meantime, a retrofit contract was given to consultants to strengthen the bridge to survive the maximum credible earthquake (fig. 98). Both of these early designs were later rejected by the Peer Review panel because the original intent was to prevent collapse, but the criteria from Caltrans and the Governor was that the viaducts should remain serviceable after the maximum earthquake. Caltrans developed a soft-soil spectra for this site, and the retrofit consultant modified their design to include the entire structure and to use the new spectra for the location (fig. 92). This new retrofit required enlarging existing columns and providing much larger footings.

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Stake, Sherman, 1990, China Basin Viaduct earthquake repair: 14 p.
Casey, John, and others, 1993, Seismic bridge retrofit in California—A case study: CH2M Hill, 5 p.
Dameron, R.A., Parker D.R., 1993, 3D F.E. analysis of seismic retrofitted foundations on the China Basin Viaduct: ANATECH Applications Corp, 10 p.
Zelinski, Ray, 1993, Road to recovery—San Francisco viaducts reconstruction: Caltrans, 19 p.

Figure 93.—Diagonal shear cracks at outrigger cap for Bent A-12 of China Basin Viaduct (photograph by Mike Van de Pol).

Table 15.—*Expansion joint movement at China Basin Viaduct*

Alignment and span number	Movement in inches	Total opening in inches
S2 LINE - SPAN 43	1"±	2 3/4"
S2 LINE - SPAN 41	1 1/8" ±	3 5/8"
S2 LINE - SPAN 38	3/8" ±	2 7/8"
S2 LINE - SPAN 34	1"±	2 1/2"
S2 LINE - SPAN 31	3/4"±	2 1/8"
S2 LINE - SPAN 30	1 1/2"±	2 3/4"
N1 LINE - SPAN 44	1 1/16"	3"
N1 LINE - SPAN 42	3/4"	2 3/4"
N1 LINE - SPAN 41	1/2"	3"
N1 LINE - SPAN 35	1/2"	3 1/2"
N1 LINE - SPAN 32	1"	2 1/2"
N1 LINE - SPAN 28	1/2"	2 13/16"
A LINE - SPAN 35	1 1/2"	4 5/16"
A LINE - SPAN 32	-	2 9/16"
A LINE - SPAN 26	1"±	2 1/2"
A LINE - SPAN 23	1/4"±	2 7/8"
A LINE - SPAN 20	1"±	3 1/4"
A LINE - SPAN 17	2"±	3 5/16"
A LINE - SPAN 14	1/2"±	2 1/2"
A LINE - SPAN 11	-	2 7/8"
A LINE - SPAN 8	1"	2 13/16"

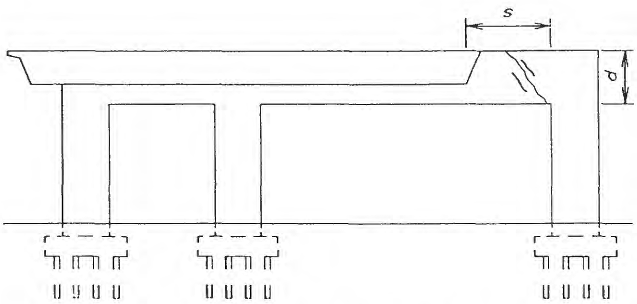


Figure 94.—Damage to Bent A-32 of China Basin Viaduct.

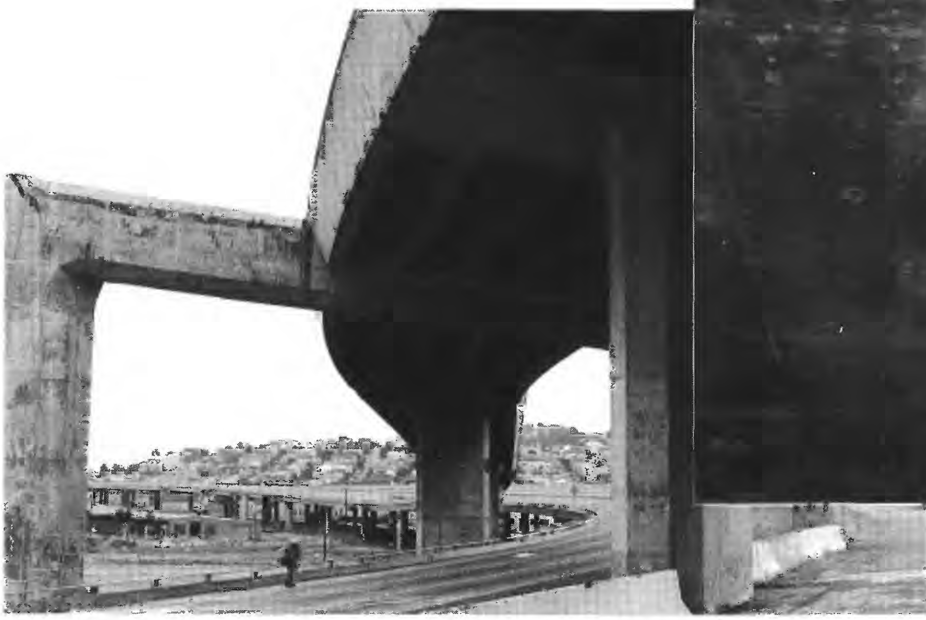


Figure 97.—Damage at Hinge A-44 of China Basin Viaduct (photograph by Mike Van de Pol).

Figure 95.—Damage to knee joints at Bent N1-35 of China Basin Viaduct.

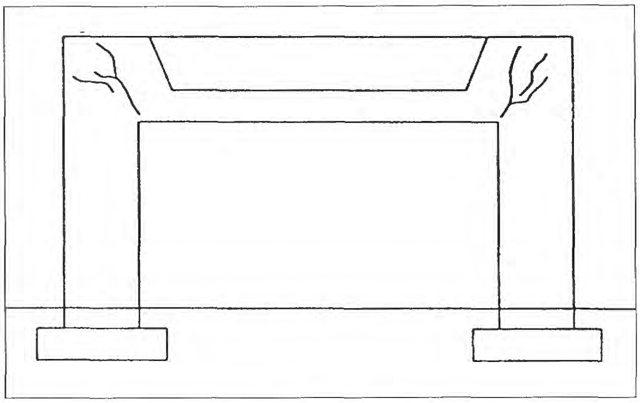


Figure 96.—Damage at Bent S2-41 of China Basin Viaduct.

Figure 98.—Similar to the retrofit that was done to Southern Freeway Viaduct immediately after the earthquake, this retrofit to China Basin Viaduct was also rejected because it could not provide a high enough level of serviceability after the maximum credible earthquake.

Br #34-55 / Rte 480 / Post Mile 0.54

Approximate Latitude & Longitude
N. Lat. 37° 47.5' W. Long. 122°23.4'

Epicentral Distance
59.4 miles

Peak Ground Acceleration	**N/S**	**U/D**	**E/W**
575 Market Street	0.08	0.06	0.11

Length	**Width**	**Skew**	**Year Built**
8,804'	52'	Varies	1959

Main Span Type
Concrete Slab and Box Girder Bridge.

Figure 99.—Embarcadero Viaduct, view to the northwest from Mission Street.

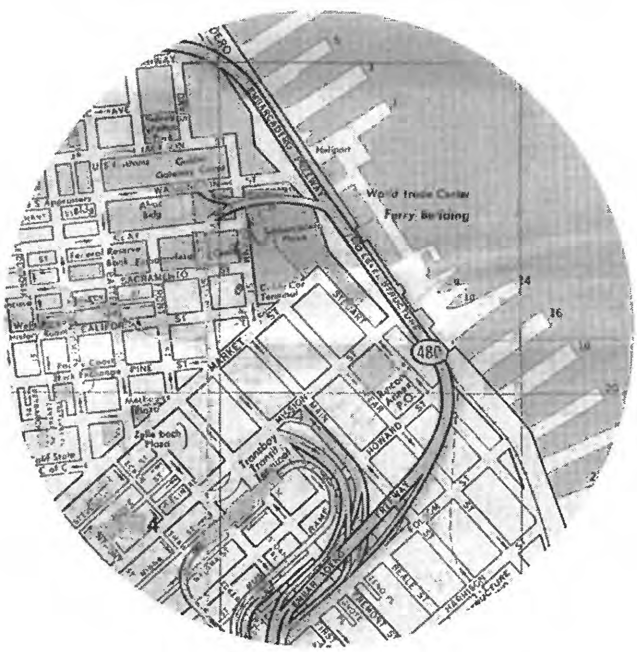


Figure 100.—Location of Embarcadero Viaduct.

EMBARCADERO VIADUCT

DESCRIPTION OF BRIDGE

The Embarcadero Viaduct was that portion of the double-deck ramps that extended from the Bay Bridge (beyond Bent 56 of Terminal Separation) as far as Broadway to the north and to Sansome Street to the west in the city of San Francisco (figs 100 to 106). This viaduct carried two reinforced concrete box girder superstructures on outrigger bents. The bottom bent cap was reinforced concrete with fixed column connections, while the top cap was prestressed with pinned column connections. The foundations were pile footings with a pinned connection to the columns. The soil was loose alluvium over Bay mud. The bridge was completed in 1959 and retrofitted in 1985 with cable restrainers that tied the pinned column to cap connection together.

BRIDGE DAMAGE

This bridge had the same inadequate reinforcement details as many of the other damaged double-deck viaducts. In fact, it was similar to the collapsed portion of the Cypress Viaduct. However, the curved alignment and the on and off ramps provided the structure with some transverse stability. Also, the top columns were not stubbed into a pedestal as in the Cypress Viaduct. A cursory inspection was made immediately after the earthquake by Caltran's Post Earthquake Inspection Team (PEQIT). This survey found that most of the damage was cracking and spalling at the interface between the lower columns and lower bent cap from Bent 75 to Bent 88. There was also considerable damage to the surface road beneath the viaduct. A more thorough inspection was later performed by maintenance engineers. Table 16 summarizes their findings. Figures 107 to 113 show the typical bent configurations for this structure. Figures 114 to 120 show typical damage.

DESCRIPTION OF REPAIRS

Immediately after the earthquake, the viaduct was closed and timber shoring was brought in to support the superstructure where the columns and bent caps were damaged (from Bents 75 to 83). Twenty policemen were needed for the closure at a cost of $14,000 a day. Caltrans created a retrofit for this structure; however, the City of San Francisco decided it had better uses for the land on which the expressway stood and Caltrans removed the viaduct.

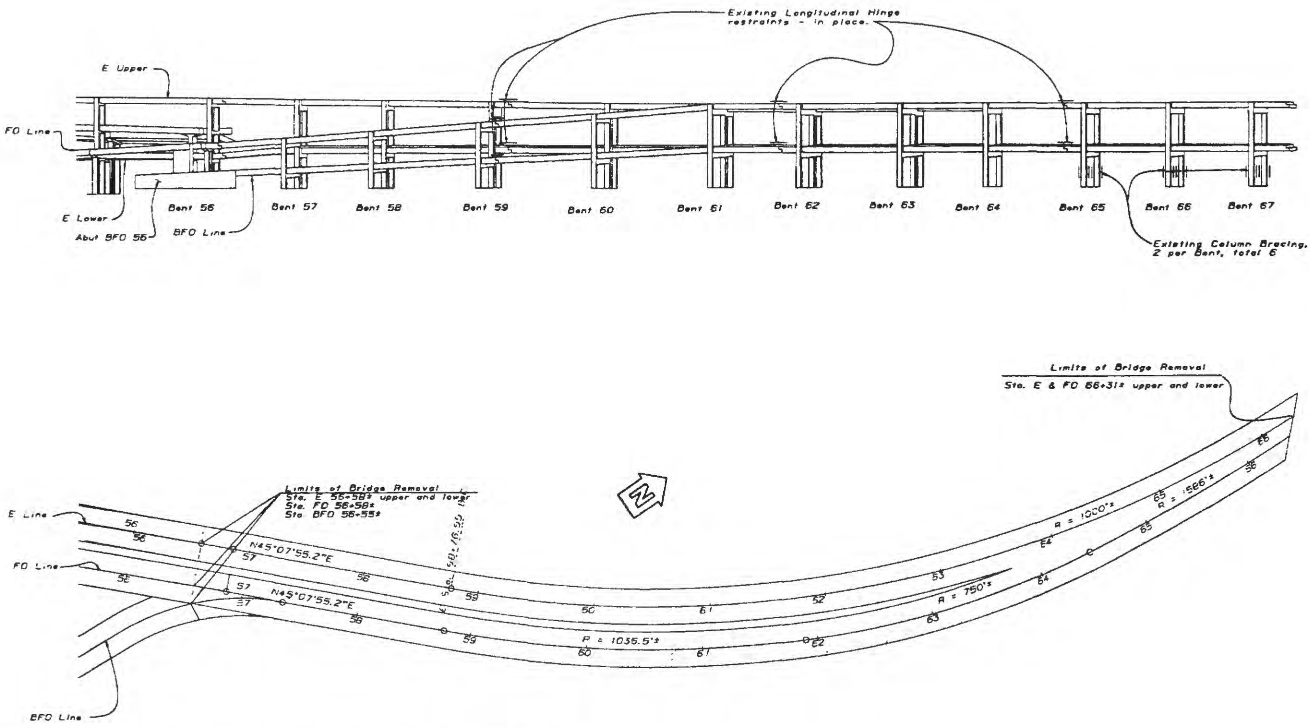


Figure 101.—Structural Plan drawing for Embarcadero Viaduct.

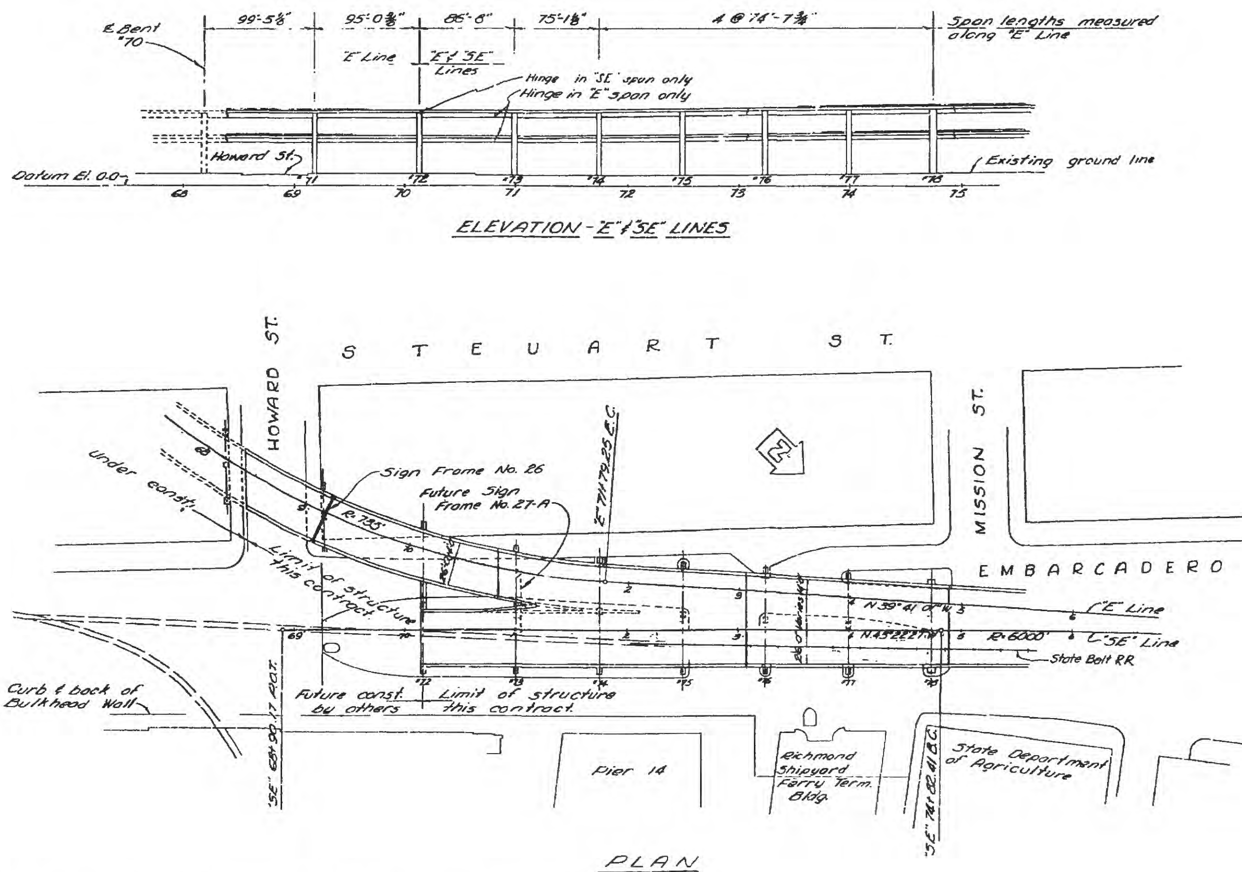


Figure 102.—Structural Plan drawing for Embarcadero Viaduct.

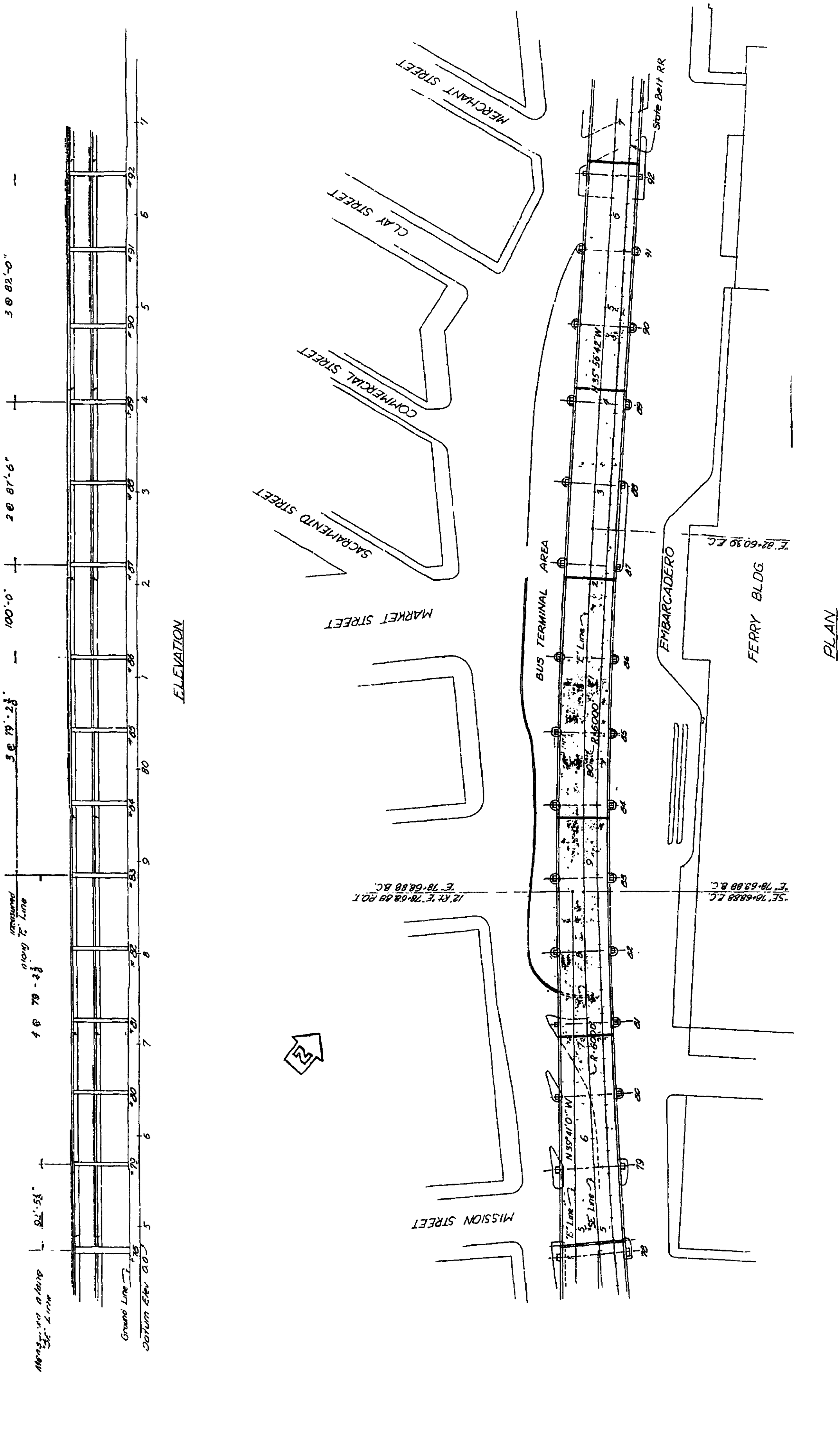


Figure 103.—Structural Plan drawing for Embarcadero Viaduct.

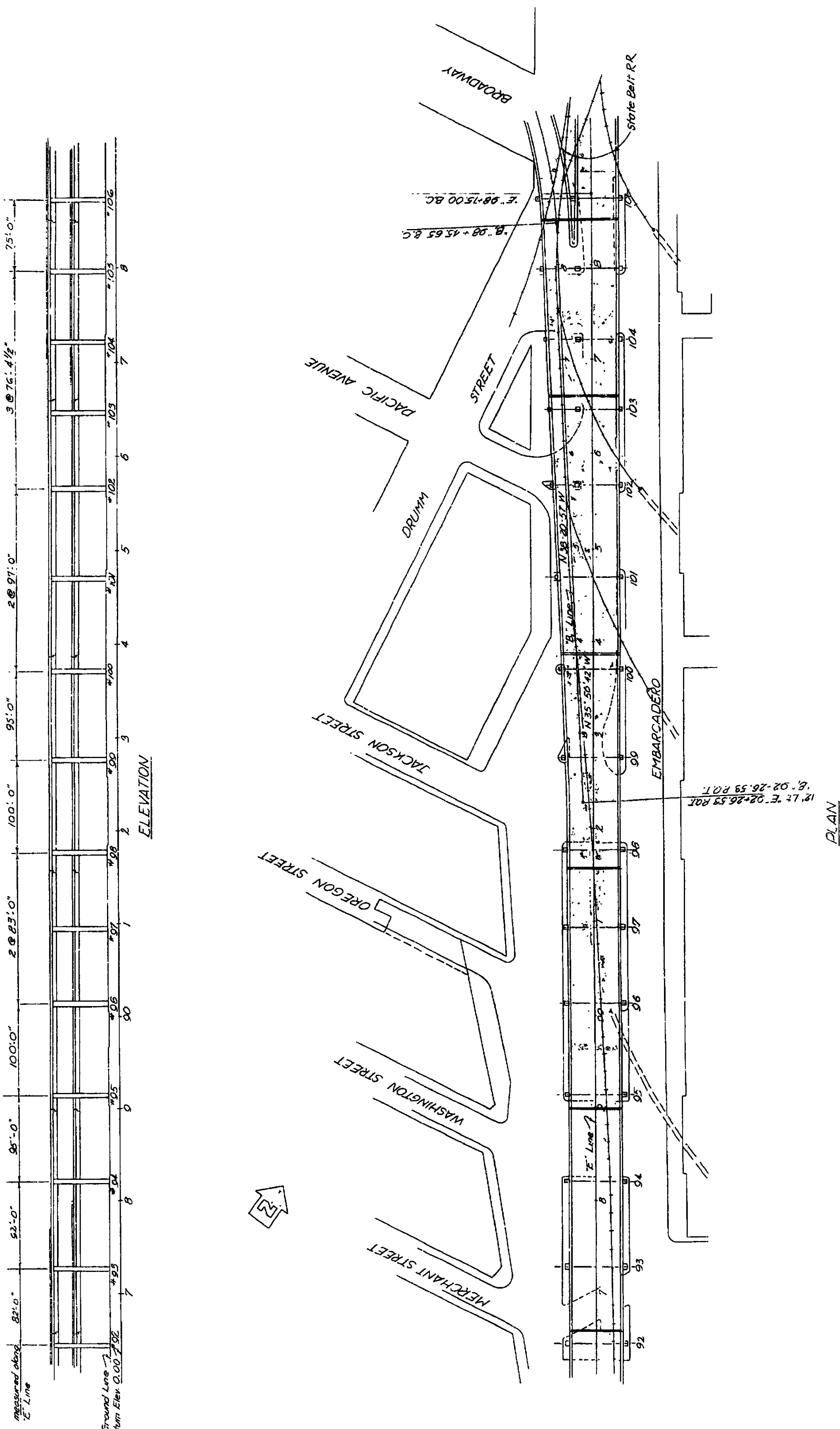


Figure 104.—Structural Plan drawing for Embarcadero Viaduct.

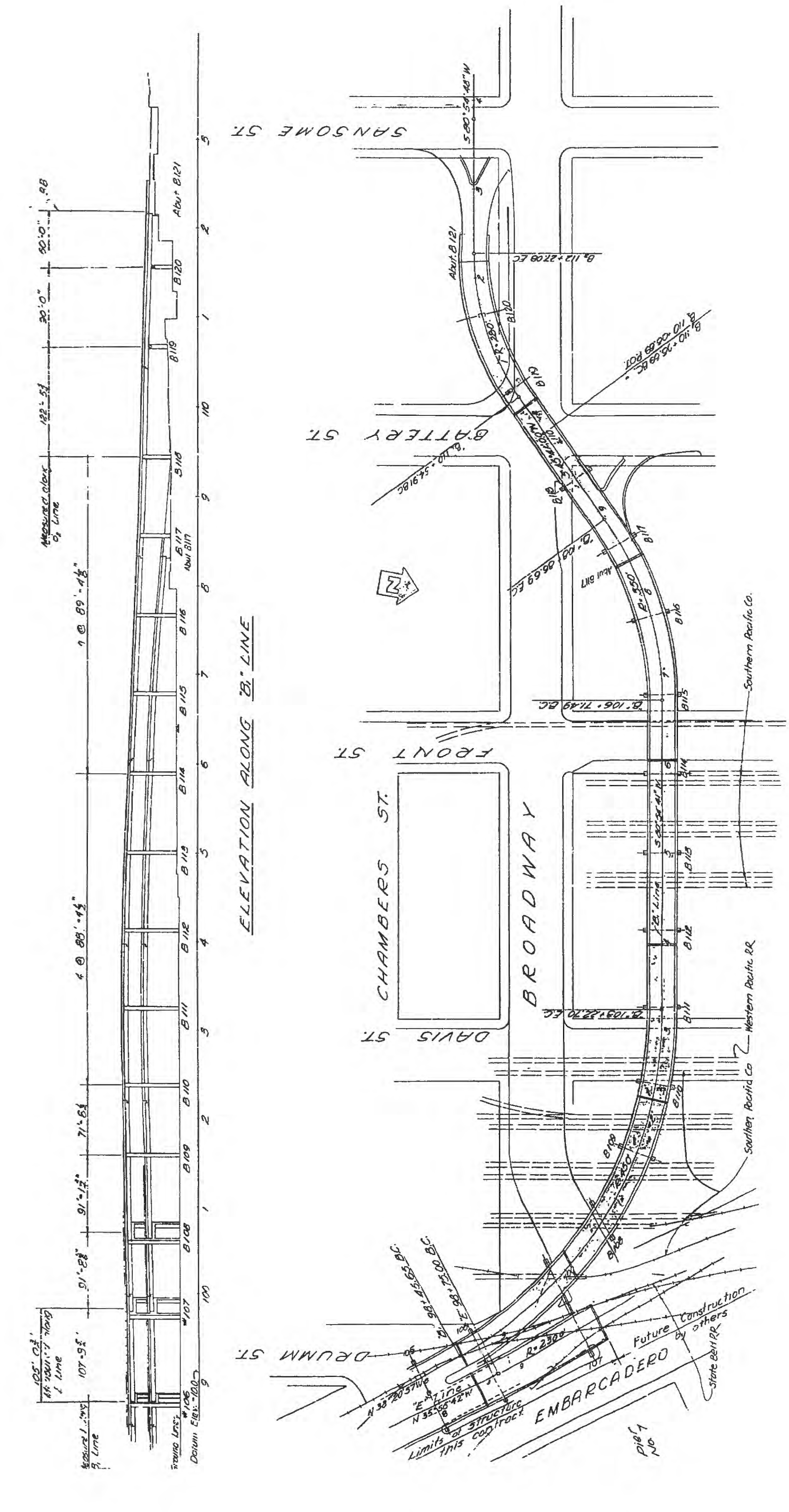


Figure 105.—Structural Plan drawing for Embarcadero Viaduct.

Table 16.—*Summary of damage to Embarcadero Viaduct*

Location	Damage
Bent 56 to 64	No damage.
Span 64	Small spall on lower left side of hinge seat.
Bent 65 to 69	No damage.
Span 67	Small spall on lower right side of hinge.
Bent 68 to 69	No earthquake damage.
Bent 70	Left column had a vertical crack where it joined the lower cap.
Bent 71	No damage.
Bent 72	Crack in lower bent cap between two superstructures. Left column cracked at cap.
Bent 73	A fine, vertical crack where left column meets lower cap.
Bent 74	Vertical cracks between all three columns and lower cap (rt col. also had diag. crack)
Bent 75	Cracks and spalls where columns met lower bent cap.
Bent 76	Large diagonal cracks under the lower cap for left and right columns.
Bent 77	Cracks and spalls for all three columns under lower cap.
Bent 78	Right column had serious damage with spalls and cracks on column and lower cap.
Bent 79	Diagonal cracks where left and right columns met lower cap (cap cracked at left col.).
Bent 80	Diagonal cracks where left and right columns met lower cap.
Bent 81	Left column and lower deck shattered at connection to lower cap.
Bent 82	Same as Bent 81. Edge of bottom deck spalled next to columns.
Bent 83	Edge of bottom deck spalled next to columns.
Bent 84	Edge of bottom deck spalled next to columns (obscured by ivy).
Bent 85	Edge of bottom deck spalled next to columns.
Bent 86	No damage.
Bent 87 to 88	Edge of bottom deck spalled next to columns.
Bent 89 to 90	No damage.
Bent 91	Edge of bottom deck spalled next to columns.
Bent 92	No damage.
Bent 93	Edge of bottom deck spalled next to columns.
Bent 94	Edge of bottom deck spalled next to columns.
Bent 95	No damage.
Bent 96 to 97	Edge of bottom deck spalled next to columns.
Bent 98 to Abut. 121	No damage.
Clay St Ramp	No damage
Wash. St Ramp	No damage

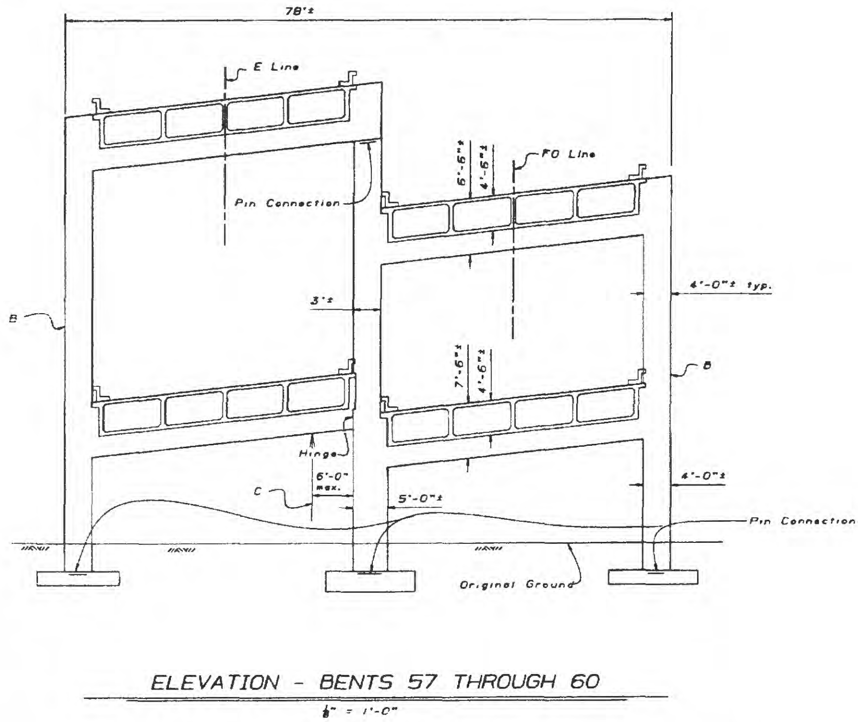


Figure 107.—Elevation drawing of Bents 57 to 60 at Embarcadero Viaduct

Figure 106.—Embarcadero Viaduct under Clay Street Ramp.

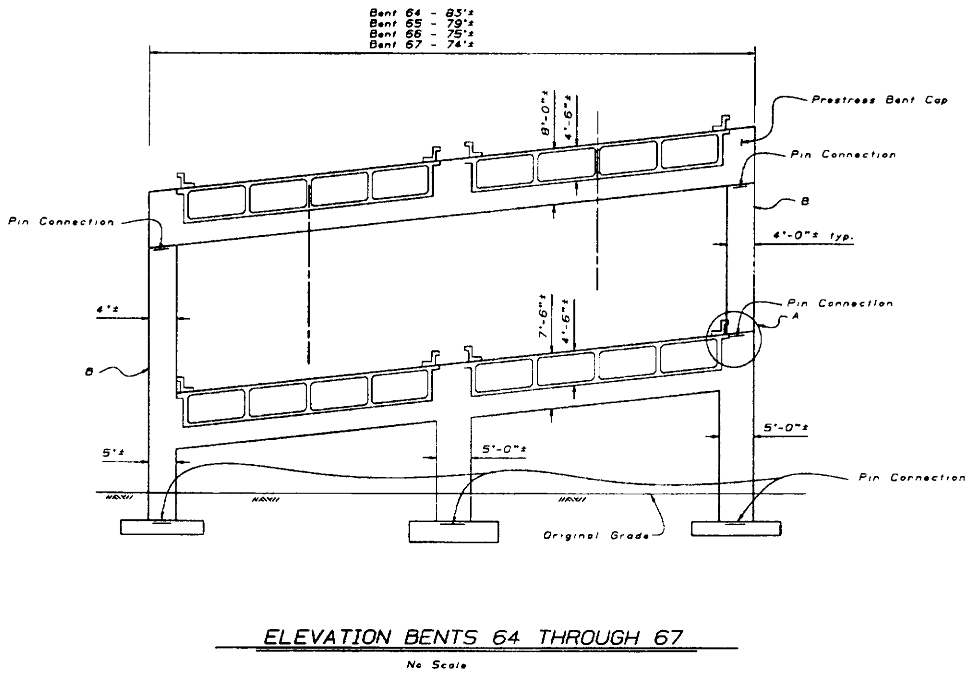


Figure 108.—Elevation drawing of Bents 64 to 67 at Embarcadero Viaduct.

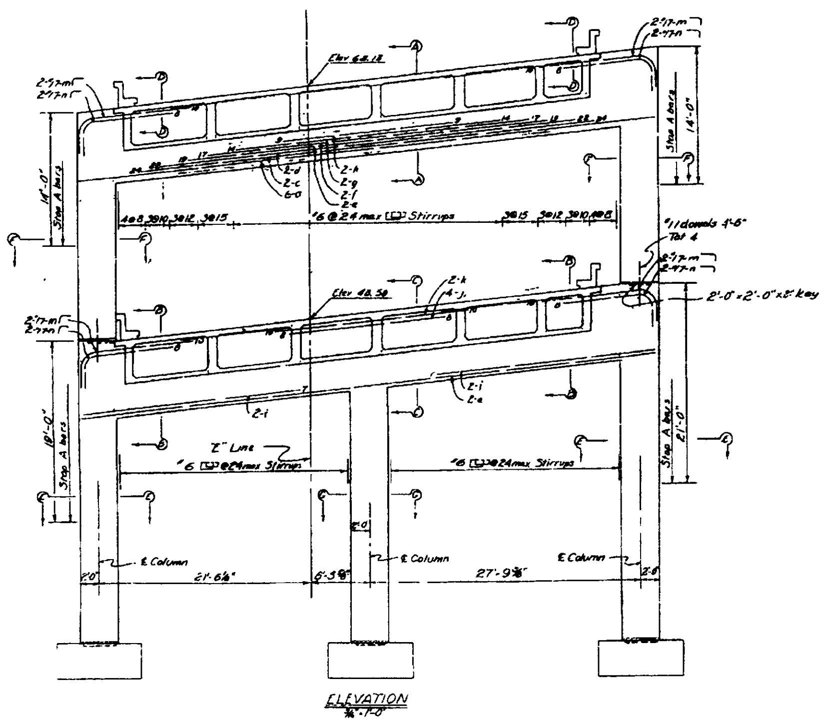


Figure 109.—Elevation drawing of Bent 71 at Embarcadero Viaduct.

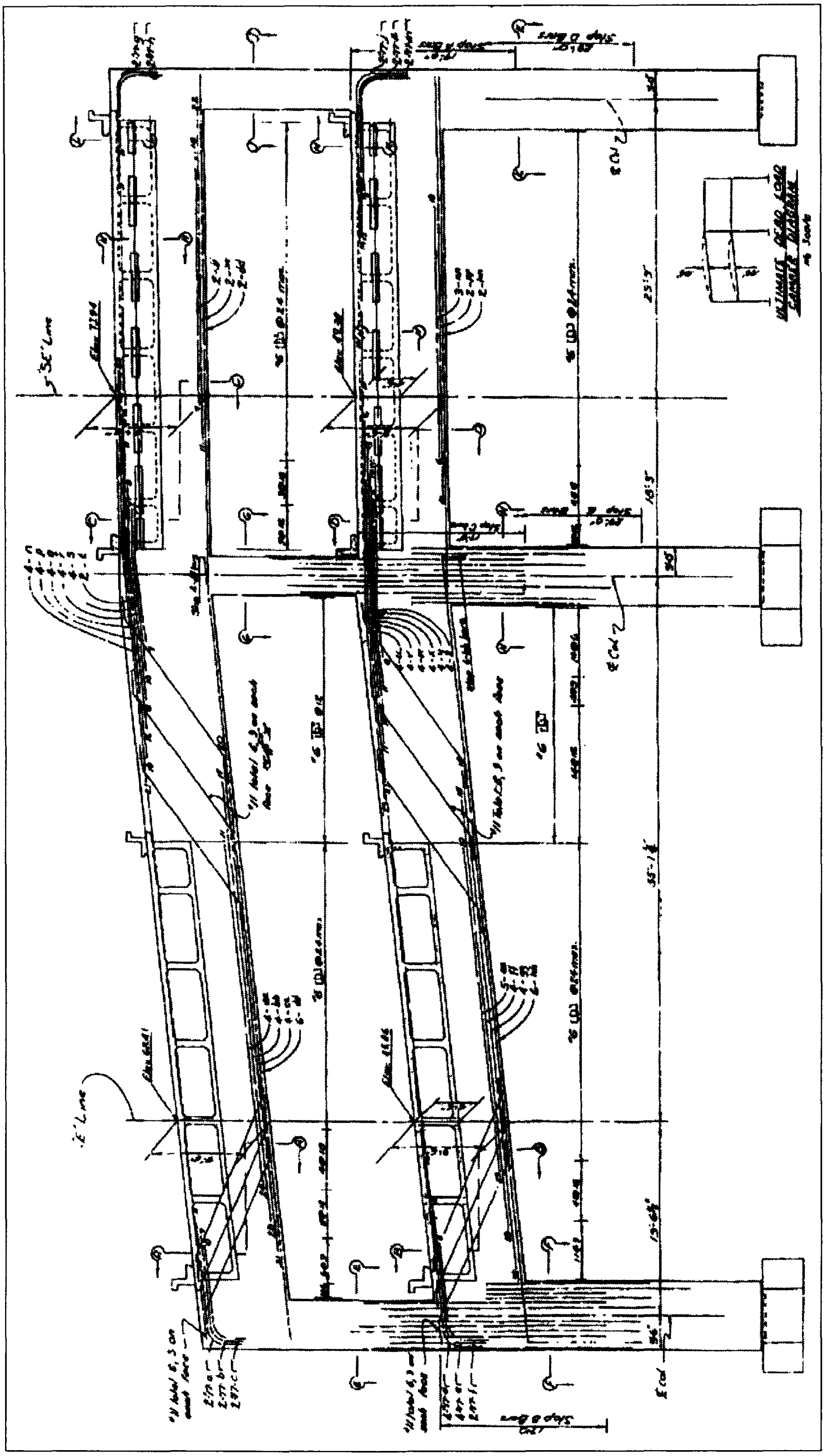


Figure 110.—Elevation drawing of Bent 72 at Embarcadero Viaduct.

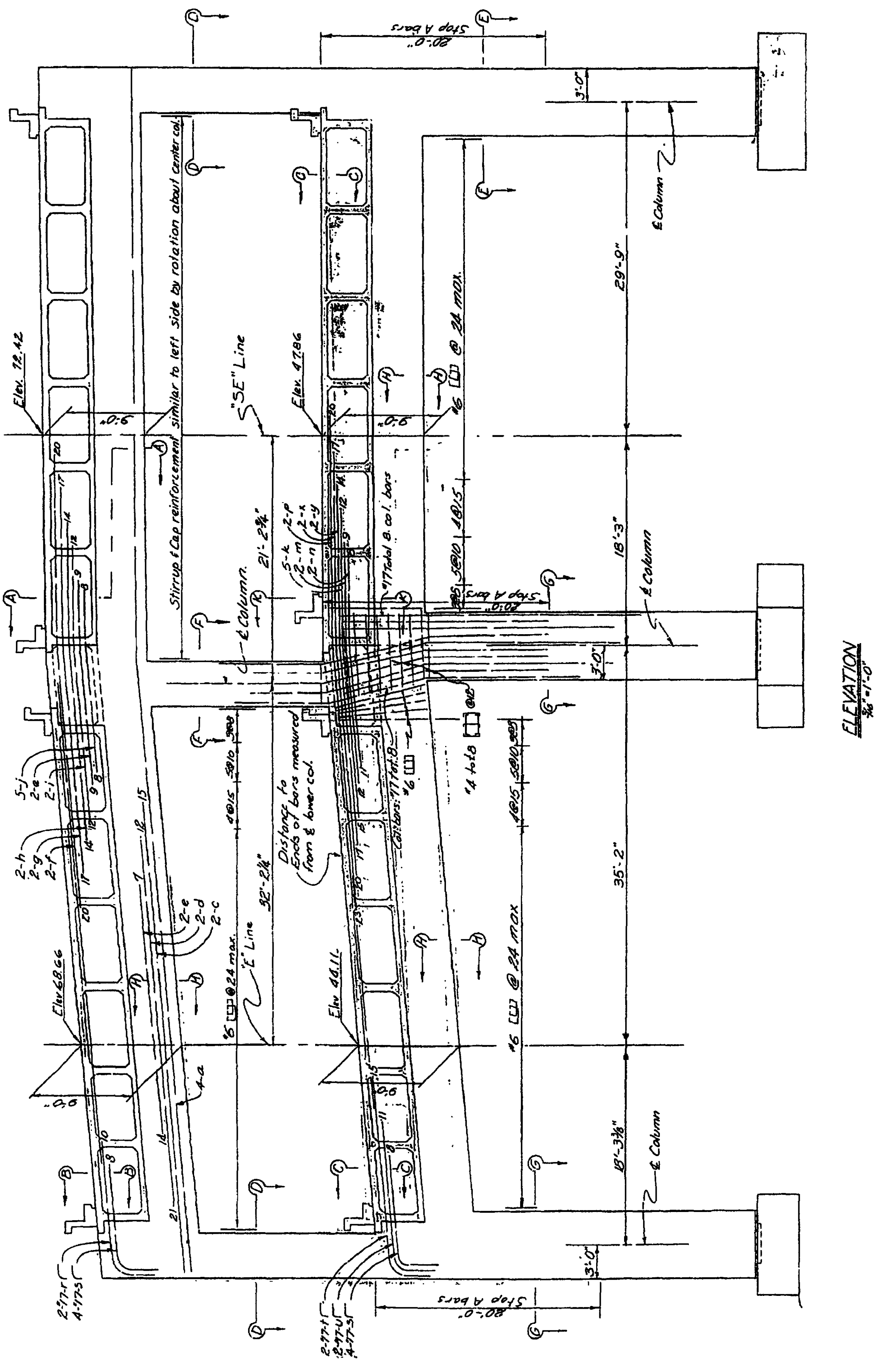


Figure 111.—Elevation drawing of Bent 73 at Embarcadero Viaduct.

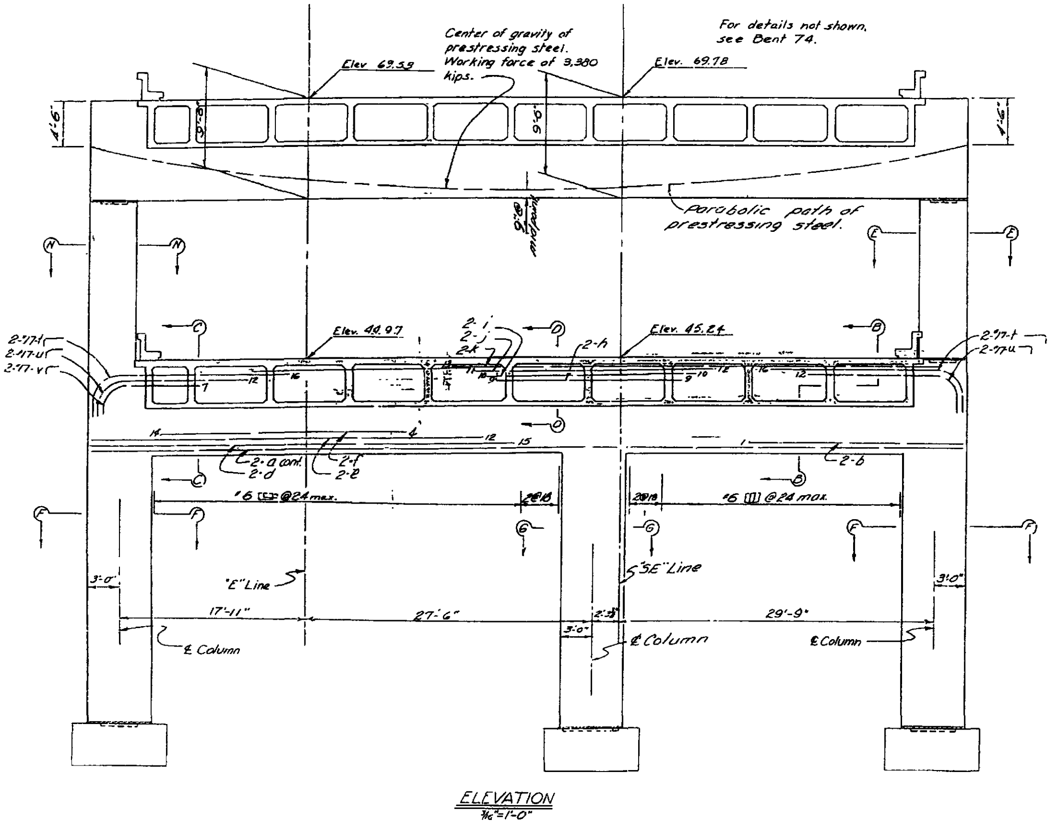


Figure 112.—Elevation drawing of Bent 77 at Embarcadero Viaduct.

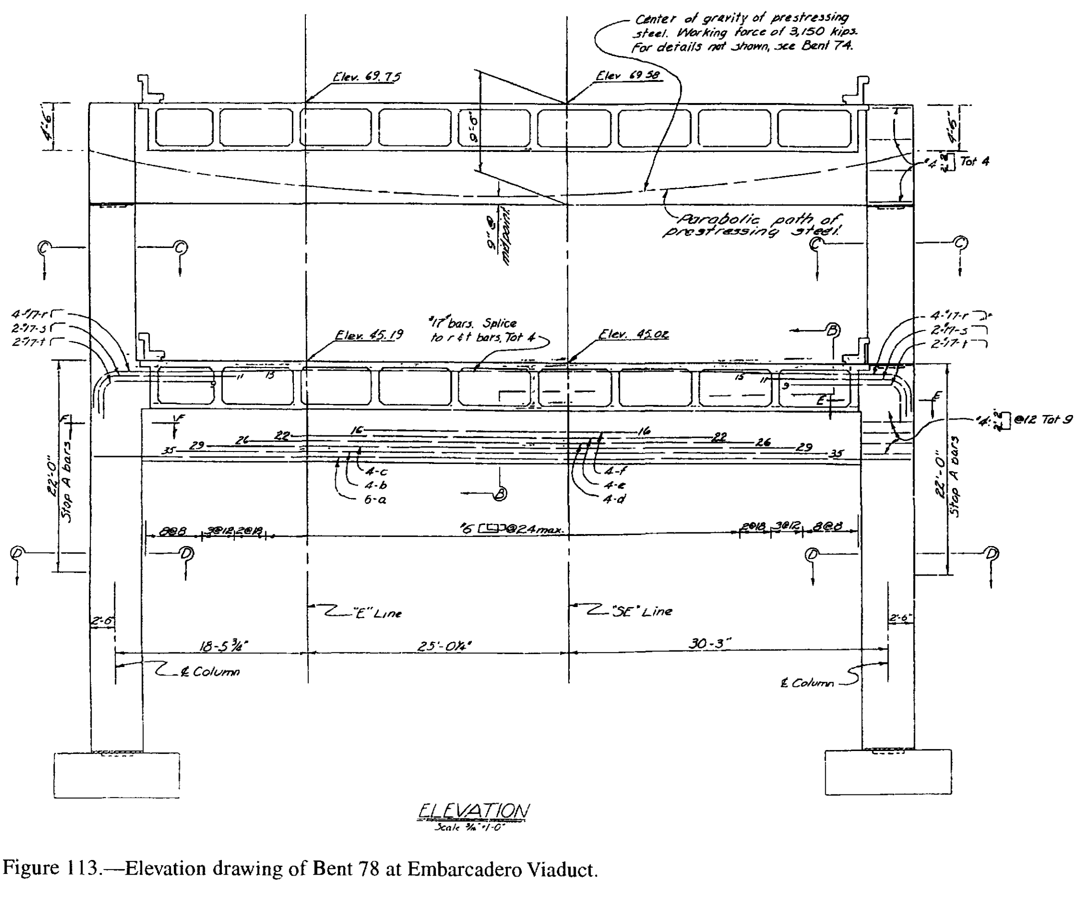


Figure 113.—Elevation drawing of Bent 78 at Embarcadero Viaduct.

Figure 114.—Column damage to west side of Bent 75 at Embarcadero Viaduct.

Figure 115.—Column damage to west side of Bent 76 at Embarcadero Viaduct.

Figure 116.—Damage to sidewalk under Bent 77 of Embarcadero Viaduct.

Figure 117.—Column damage on east side of Bent 78 at Embarcadero Viaduct.

Figure 118.—Column damage on east side of Bent 79 at Embarcadero Viaduct.

Figure 119.—Column damage to west side of Bent 79 at Embarcadero Viaduct.

Figure 120.—Damage to west side of Bent 80 at Embarcadero Viaduct.

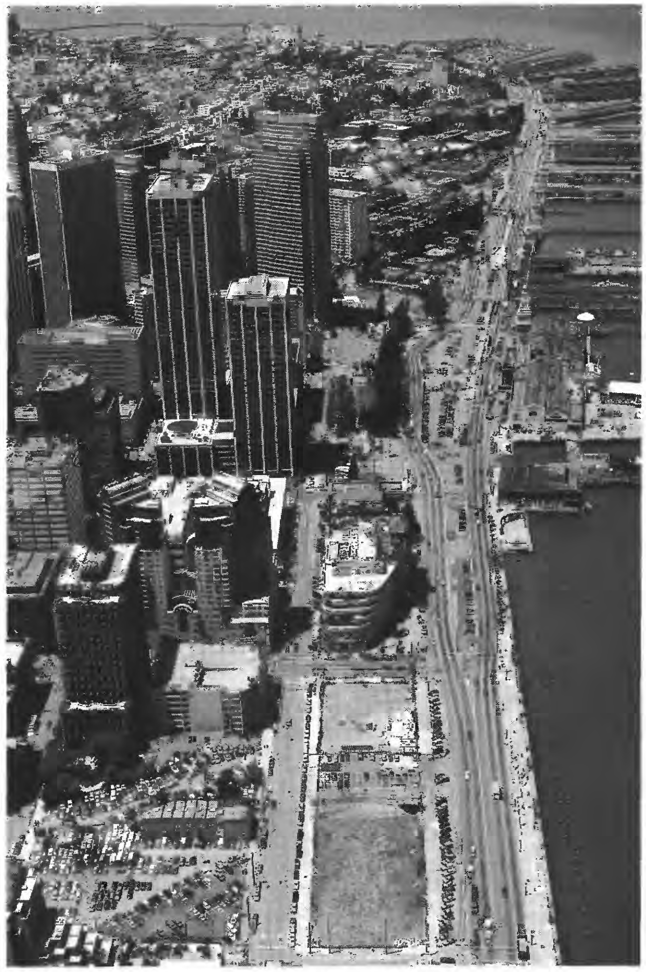


Figure 121.—View after removal of Embarcadero Viaduct.

TERMINAL SEPARATION

DESCRIPTION OF BRIDGE

The Terminal Separation was a multilevel structure built in the 1950's to connect the San Francisco Oakland Bay Bridge to the Embarcadero Viaduct and to San Francisco city streets (figs. 121 to 124). It must have been built just as the lower deck of the Bay Bridge was changed from carrying trains to carrying cars and trucks (as indicated by change orders to the lower deck of Terminal Separation during construction). This structure has a complicated alignment of circular on and off ramps and connector structures. The location of damage (on alignment "BR") was at the only location where the superstructure changed from a few steel plate girders to reinforced concrete box girders (fig. 129). The bents are single and multicolumn reinforced concrete with some prestressed bent caps. Most of the bridge was supported on 10BP42 (and 14-inch welded) steel piles. There were also some Raymond piles (which are concrete piles with reinforcement for the top 12 feet) on the north end of the structure. Much of the bridge was on soft Bay muds (fig. 125).

BRIDGE DAMAGE

Except for Bent 44, this bridge experienced only minor damage from the earthquake. A cursory inspection of the bridge by the Post Earthquake Inspection Team on October 20 did not detect any damage. An inspection done on November 3 by Structures Maintenance found minor cracks at Bent 32 and failed rocker bearing bars that dropped the steel girders onto the bent cap at Bent 36 (fig. 126). A more careful examination by Structures Maintenance dated November 9 lists the following damage.

Bent 44 (Column 2).—The lower outrigger bent cap was completely fractured with ≥1/4-inch cracks extending transversely across the top of the bent cap approximately 6 feet back from the face of the column and extending down both sides to the bottom of the bent cap at the column face (fig. 127). Concrete was removed using a hammer, which showed fracturing extending into the core of the bent cap. The condition of the fractured concrete was rubble. Concrete in this area was lightweight Portland Cement Concrete. Estimated cost to repair this damage was $250,000.

Bent 48.—The bent cap between Columns 2 and 3 (fig. 128) was cracked vertically at midspan, with the cracks extending into the soffit. At Column 3 the bent cap was cracked, extending from the bottom of the cap at the column inward and upward at 45°. The bent cap at Column 4 was cracked at the column similarly to that described for Column 3. Repair cost at this bent was estimated to be about $50,000.

Bent 49.—No earthquake damage (fig. 129).

Bent 50.—No earthquake damage (fig. 129).

Bent 51.—The bent cap at Column 1 was vertically cracked at the column face, with exposed rebar (fig. 130). Total estimated cost of repair for this bent was estimated as $50,000.

TESTING AND REMOVAL

The repair cost for the entire structure would have been $350,000. However, the structure was removed 2 years after the earthquake, mainly for political and aesthetic reasons. Before the structure was removed the foundations were tested in a repeat of the testing done on the Cypress Viaduct. This was to confirm the higher values for pile stiffness and ultimate strength that was gathered from the Cypress tests. Engineers wanted to take advantage of these higher values in their designs but needed further testing to confirm these values. In 1996, plans were being made to retrofit the west approach to the Bay Bridge and build new off-ramps to replace the existing Harrison and Terminal Street off-ramps.

Figure 122.—Terminal Separation.

Br #34-54 / Rte 80 / Post Mile 5.45

Approximate Latitude & Longitude
N. Lat. 37° 46.9' W. Long. 122°23.8'

Epicentral Distance
59 miles

Peak Ground Acceleration	N/S	U/D	E/W
575 Market Street	0.08	0.06	0.11

Length	Width	Skew	Year Built
Varies	28'	Varies	1955

Main Span Type
Reinforced concrete box girders, (with a few welded plate girders, and concrete slabs)

Figure 124.—Terminal Separation view to northeast.

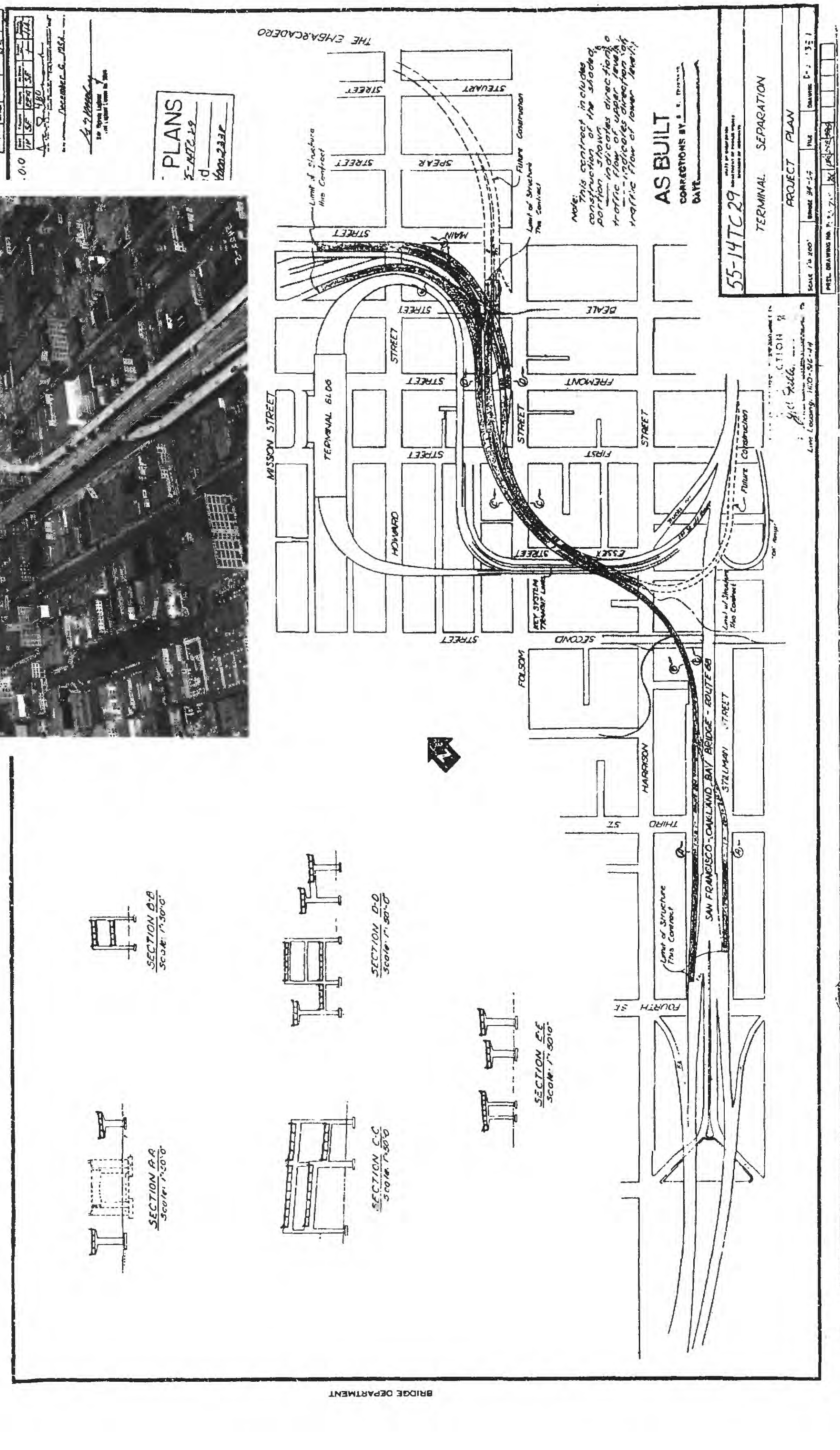


Figure 123.—Plan drawing of Terminal Separation.

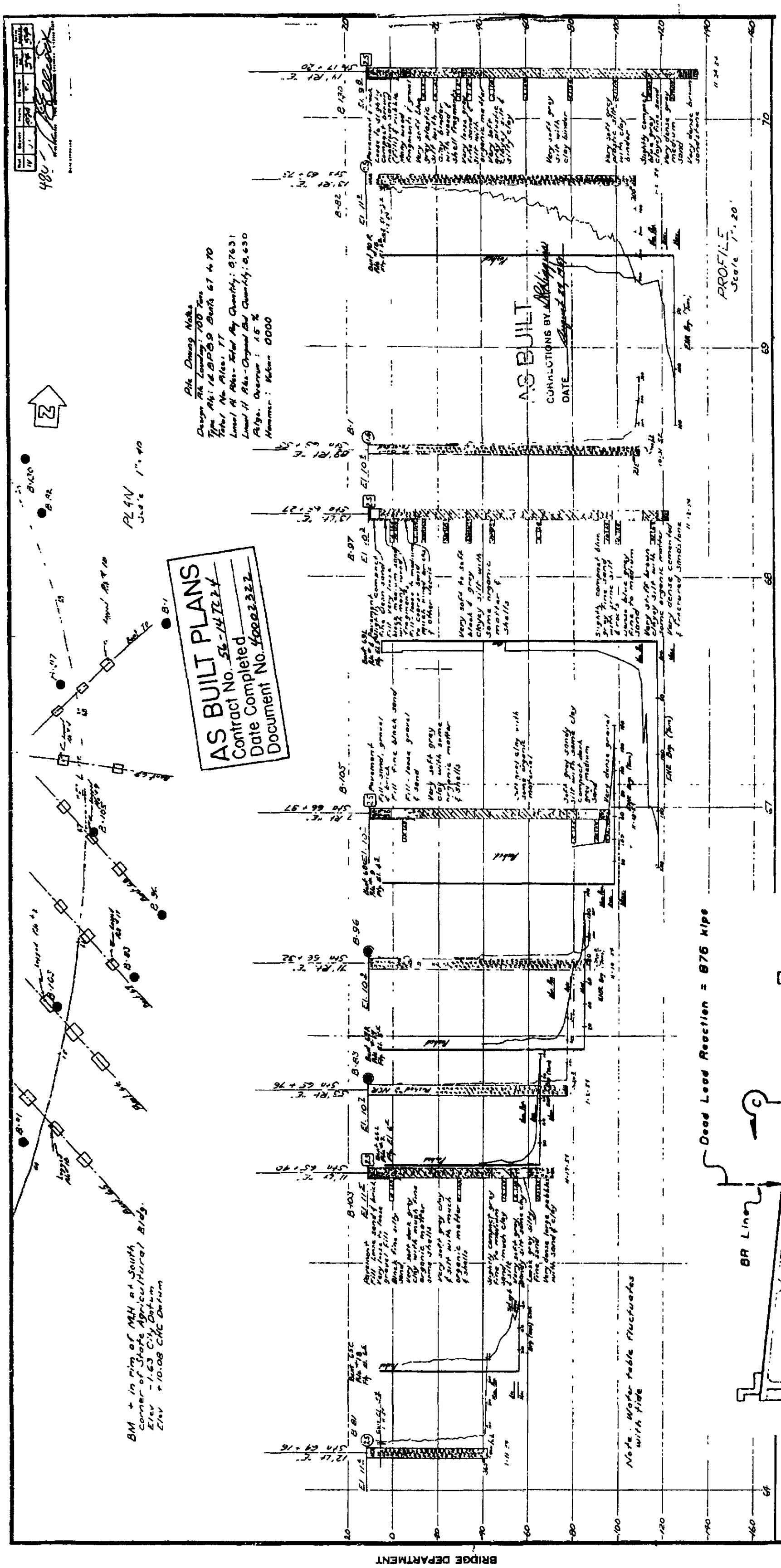


Figure 125.—Log of test borings for portion of Terminal Separation.

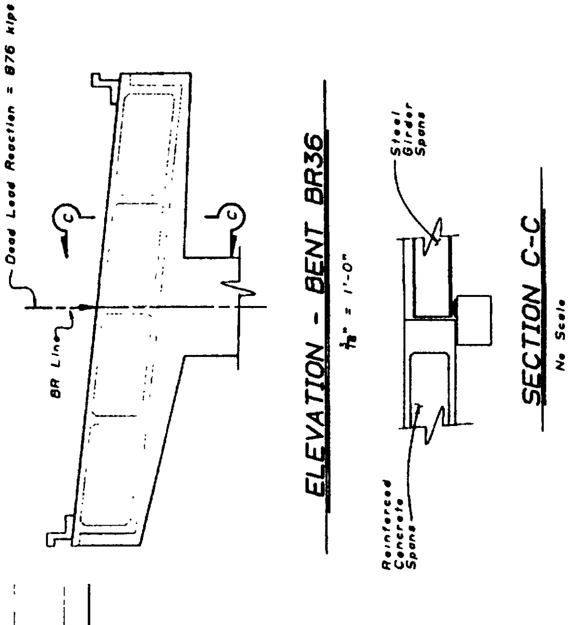


Figure 126.—Elevation drawing of Bent 36 at Terminal Separation.

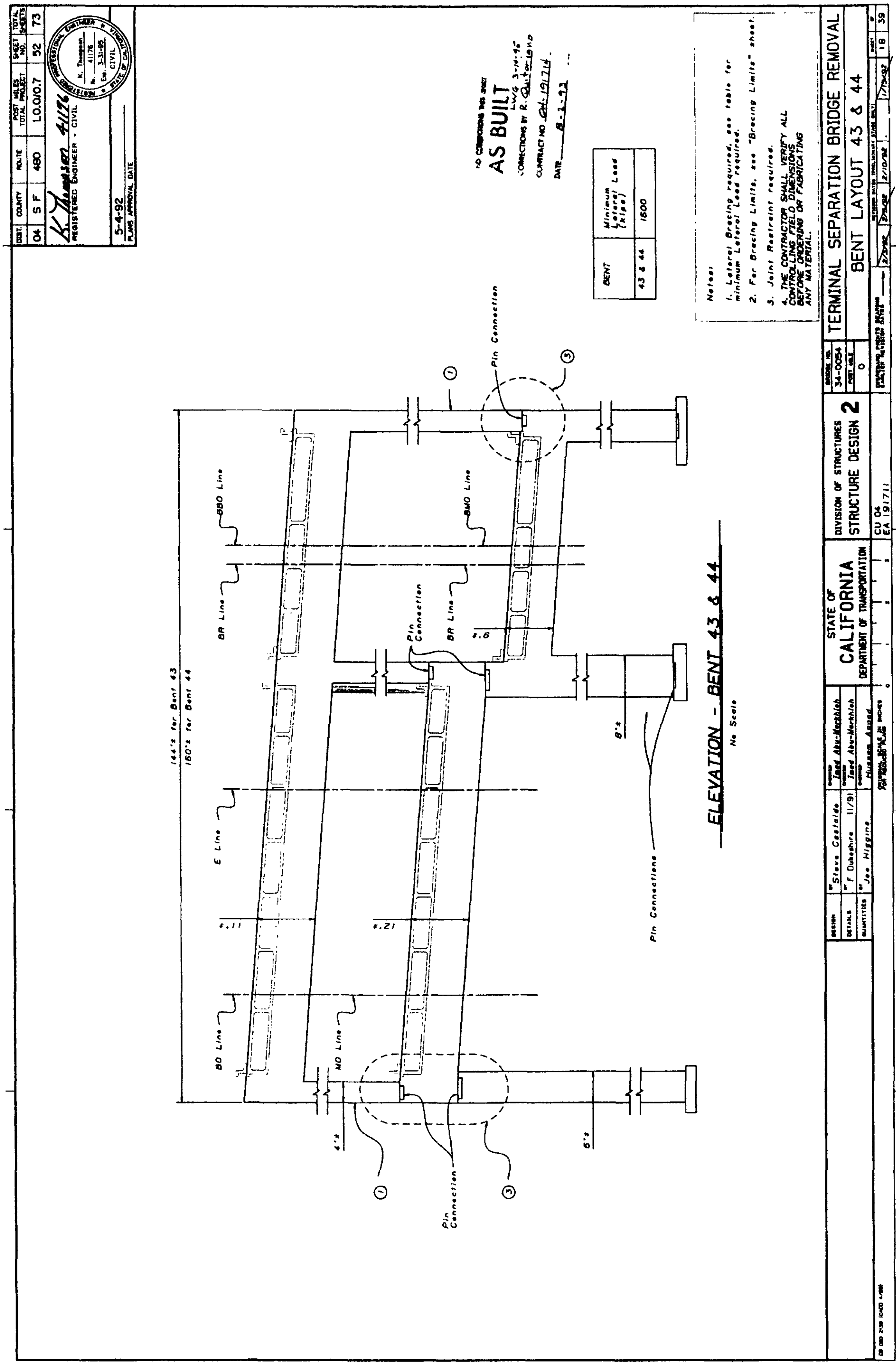


Figure 127.—Elevation drawing of Bent 43 and 44 at Terminal Separation.

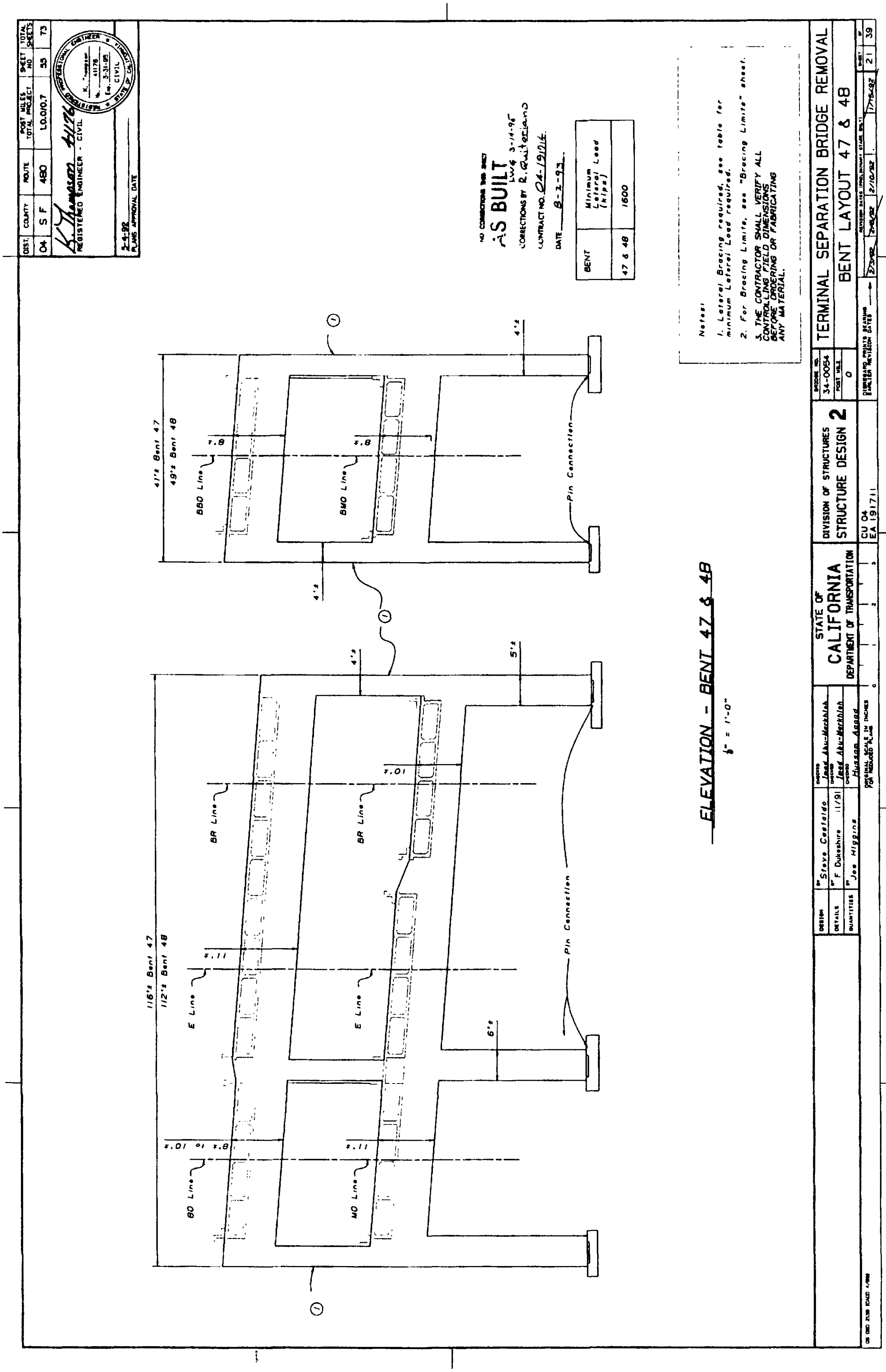


Figure 128.—Elevation drawing of Bent 47 and 48 at Terminal Separation.

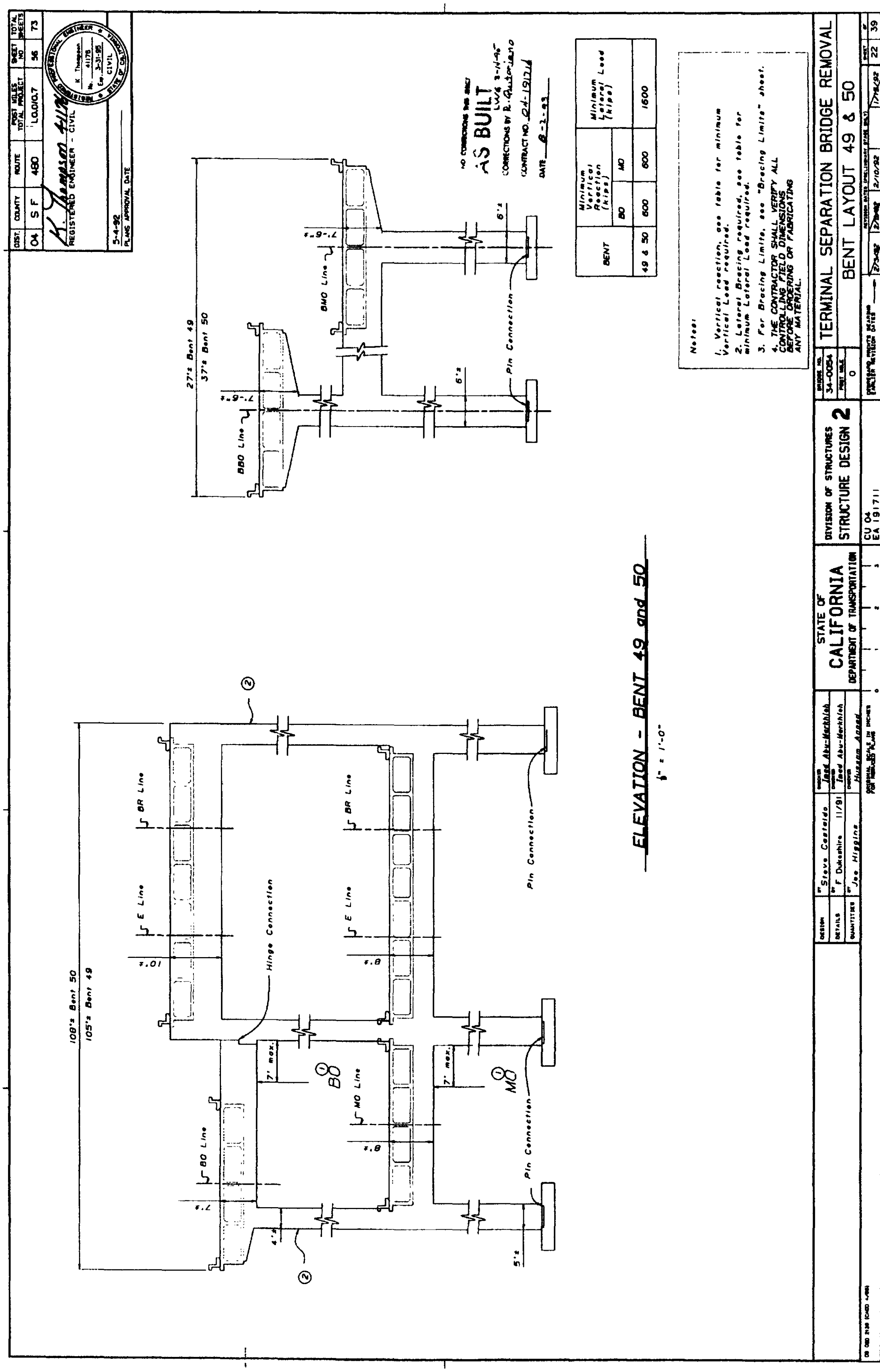


Figure 129.—Elevation drawing of Bent 49 and 50 at Terminal Separation.

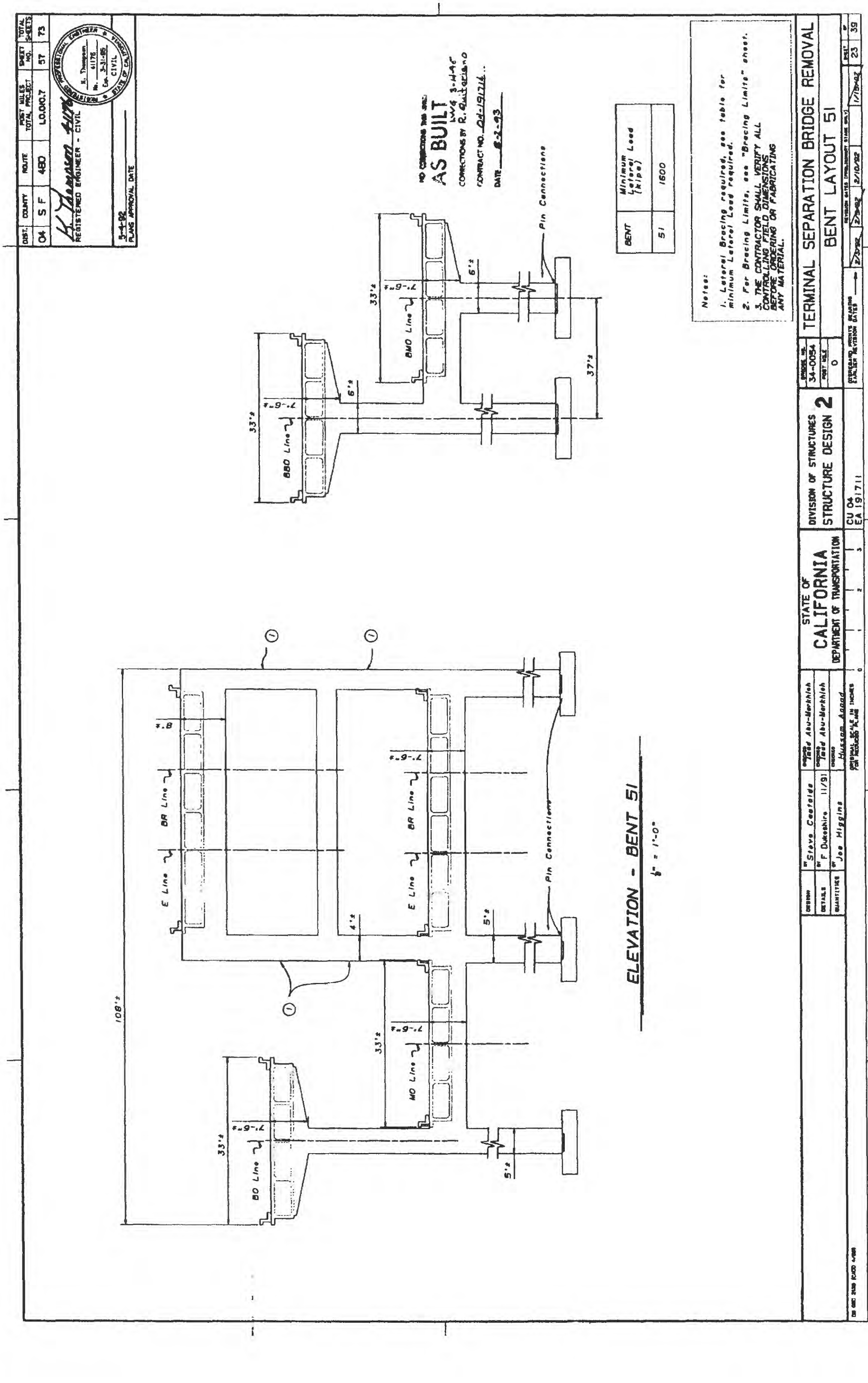


Figure 130.—Elevation drawing of Bent 51 at Terminal Separation.

CENTRAL FREEWAY VIADUCT

DESCRIPTION OF BRIDGE

The Central Freeway Viaduct was a 2.7-mile-long elevated freeway in central San Francisco. Its location is shown on the map in figure 2 and on the aerial photographs in figures 131 and 132. The structure was composed of concrete and steel sections. A double-deck reinforced concrete box girder section extended from Mission Street north to Turk Street (fig. 134). The portion north of the ramps at Oak and Fell Streets where there was earthquake damage was closed and has now been removed (fig. 136). There are a few prestressed bent caps and one prestressed superstructure (Span 21) spanning over Market Street. A steel girder section extends from Mission Street to I-80 in east San Francisco (fig. 135). There are also three sets of ramps (fig. 133). On the east end of the steel structure there are connectors that join the bridge to I-80 north to the Bay Bridge and I-101 south. On the west end of the steel structure are on and off ramps to Mission and Van Ness Streets. The double-deck structure has ramps at Oak and Fell Streets. The viaduct is supported on pile foundations on a deep, dense sand.

The Central Viaduct was completed in 1959 at a cost of $7.7 million. In 1974 it became part of the Phase I retrofit program, with restrainers tying the hinges together and with restrainers that went through the steel bent caps to tie the steel girders together. Like most of the other double-deck viaducts, the Central Viaduct was given a column retrofit in 1990 that was later rejected by the peer review panel. In 1996 a new retrofit was begun for the steel portion of the viaduct. A contract has been given to remove the top deck of the concrete portion, and a retrofit of the bottom deck is being planned. However, the city of San Francisco may eventually raze the concrete structure.

BRIDGE DAMAGE

Figures 137 and 138 show plan views of portions of this structure that sustained damage during the earthquake. Figure 142 (Thiel, 1990) is a schematic diagram of the bents that sustained damage to this structure. The bridge was inspected by Structures Maintenance on October 26, 1989, with the following findings:

Bent 8.—The bent cap was spalled below the bearing for the right exterior girder on the southbound side (upper deck). The bearing appeared to have remained in place, but there was a leveling nut exposed under the bearing plate as well as three vertical rebars and a horizontal rebar. Approximately 1 to 4 inches of the bottom of the bearing plate was exposed. At the time of this investigation the girder had not been supported off the bent cap with timbers (cost of repair, $5,000).

Bents 36 and 37.—There were some fine vertical cracks in the left columns at the lower deck bent cap. These cracks appeared to extend completely through the column since they could be seen on both sides. This condition appeared to be stable and not structurally significant.

Bents 42, 43, 44, 45, 46, and 48.—Column 2 (and Column 3 for three column bents) had a similar cracking pattern, which consisted of mostly heavy and medium cracks proceeding through the center (of the sides of the columns facing up and down station) and terminating at the opposite sides of each column after following on approximately diagonal path. These cracks, starting at the base on the lower deck, proceeded up from 10 to 14 feet toward the upper deck. They appeared to be of a compression failure nature. In Column 2 of Bent 43 and in Column 3 of Bent 48, this cracking was advanced enough to form an "X" pattern and to induce significant spalling and incipient spalling at Bent 43 with exposure of main and tie rein-

Figure 131.—Central Freeway Viaduct (in foreground).

Br #34-077 Rte 101 / P.M. R4.25

Approximate Latitude & Longitude
N. Lat. 37° 46.2' W. Long. 122°24.9'

Epicentral Distance
59 miles

Peak Ground Acceleration	N/S	U/D	E/W
Pacific Heights	0.05	0.03	0.06

Length	Width	Skew	Year Built
13,994'	Varies	Varies	1955

Main Span Type
Reinforced concrete box girders, welded plate girders, and concrete slabs

Average Daily Traffic = 133,000

forcement (figs 139 to 141). In all cases the cracks appeared to extend through the entire column cross section. At street level, Column 3 of Bent 48 and Column 2 of Bent 49 had the same damage as mentioned above, indicating it was distressed and only partially functional. Figure 142 shows all the significant damage that occurred at Central Viaduct during the earthquake.

There was also some minor damage including:

Bent 47.—Column 1 had some incipient spalling at the right corner near the base of the column at the level of the lower deck.

Bent 48.—Column 3 has a minor corner spall in left corner where the column joined the top bent cap (near the top level).

Bent 54.—Near column 3 there were old shear cracks, which may have been worsened by the earthquake. A crack proceeded from the inside corner of the base of the cap upwards to the top of the cap, roughly on a diagonal trace. Another fine crack, 6 feet farther out on the cap, was roughly parallel to the first crack. These cracks were only on the down station face of the cap. Column 1 had a 1.5- by 1.5- by 2.25-foot-deep spall at the right corner at the base of the column at street level. A 2-inch length of main reinforcing bar was exposed.

Figure 133.—Ramps onto Central Viaduct at Oak and Fell Streets.

Figure 134.—Double-deck portion of Central Freeway Viaduct.

Figure 132.—Central Freeway Viaduct.

Figure 135.—Steel girder portion of Central Freeway Viaduct.

Figure 136.—Central Viaduct removal.

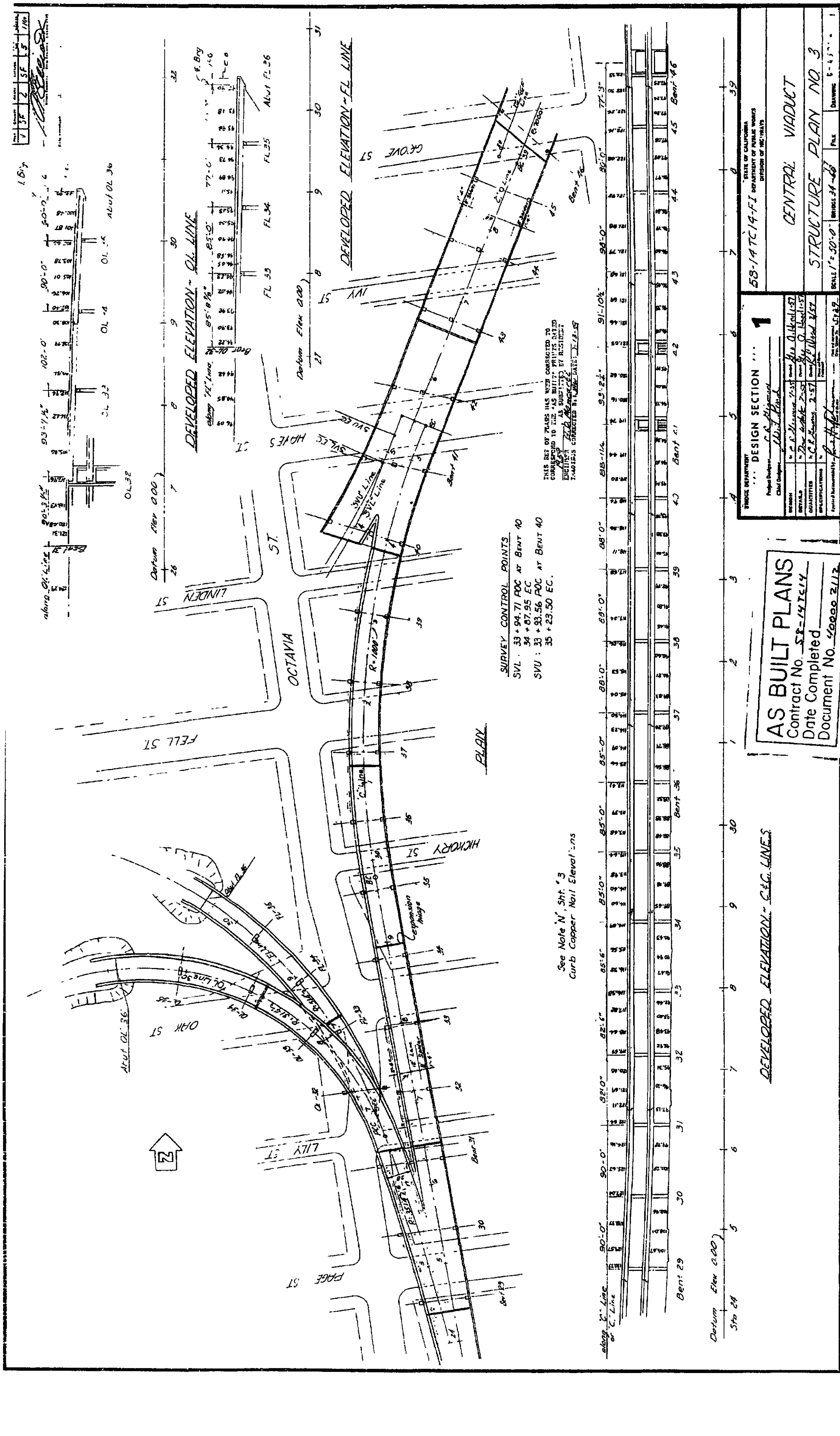


Figure 137.—Structural Plan drawing of damaged area at Central Freeway Viaduct.

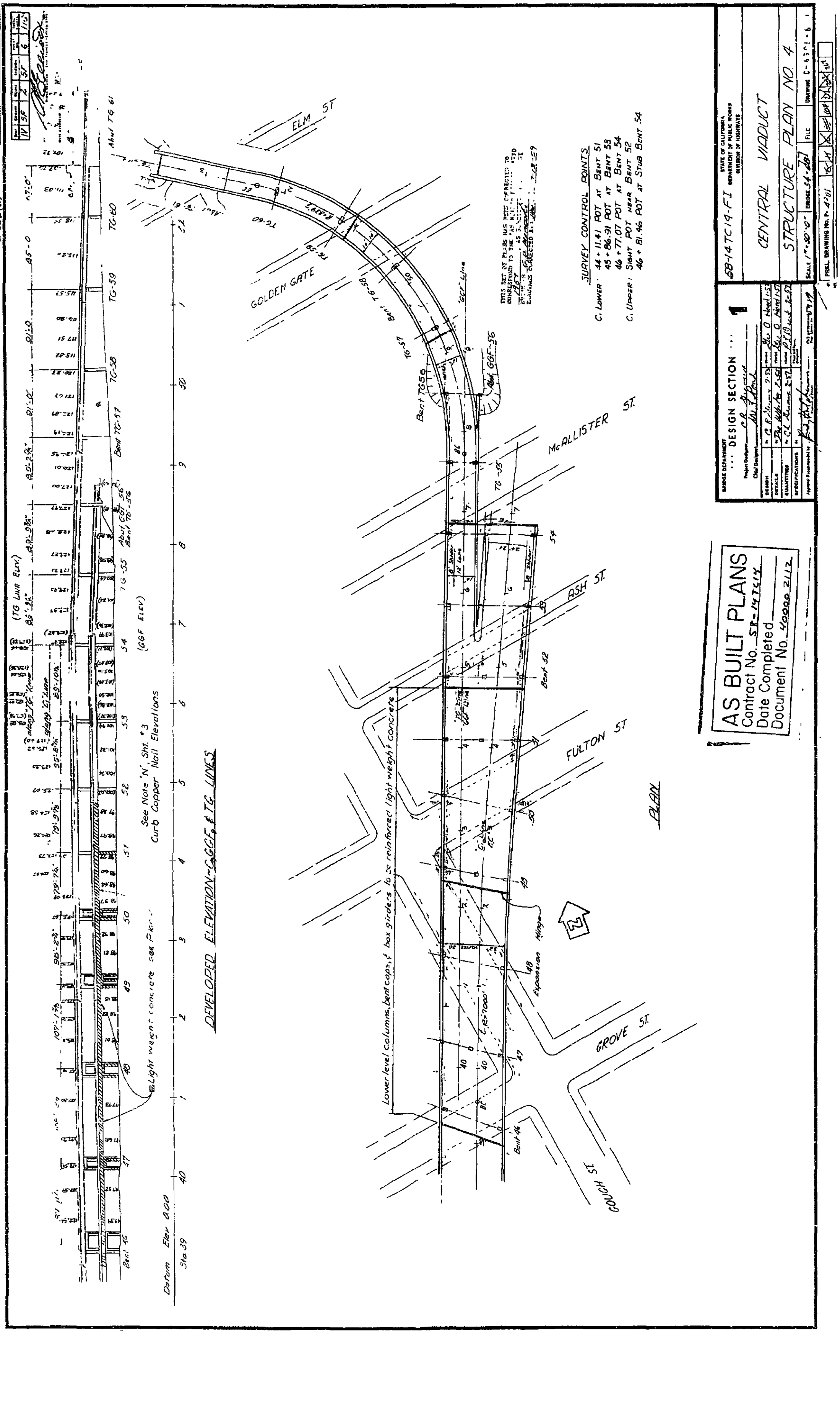


Figure 138.—Structural Plan drawing of damaged area at Central Freeway Viaduct.

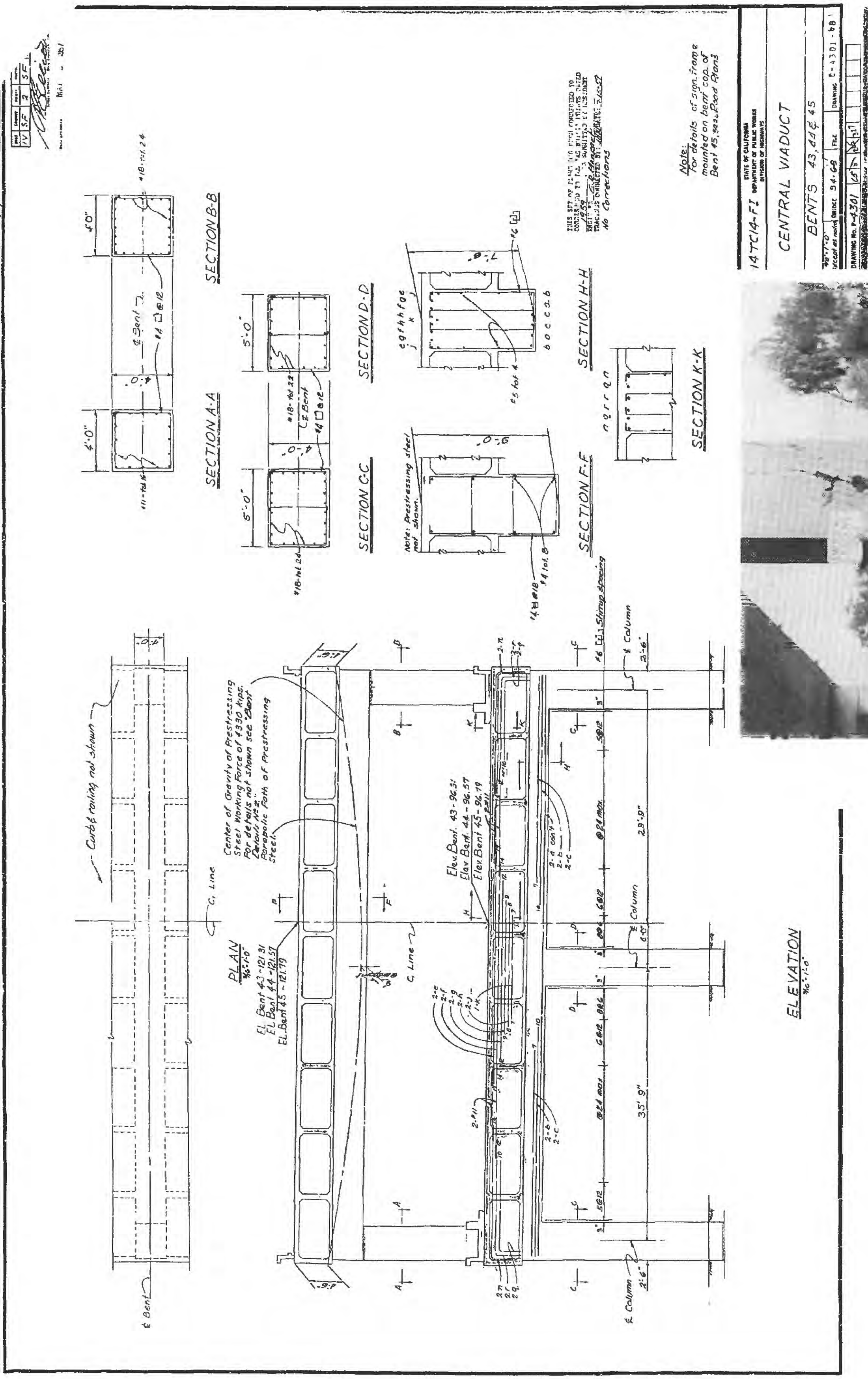


Figure 139.—Elevation drawing of Bent 43 at Central Freeway Viaduct.

Figure 140.—Damage to Column 2 of Bent 43 at Central Freeway Viaduct (photograph by Mike Van de Pol).

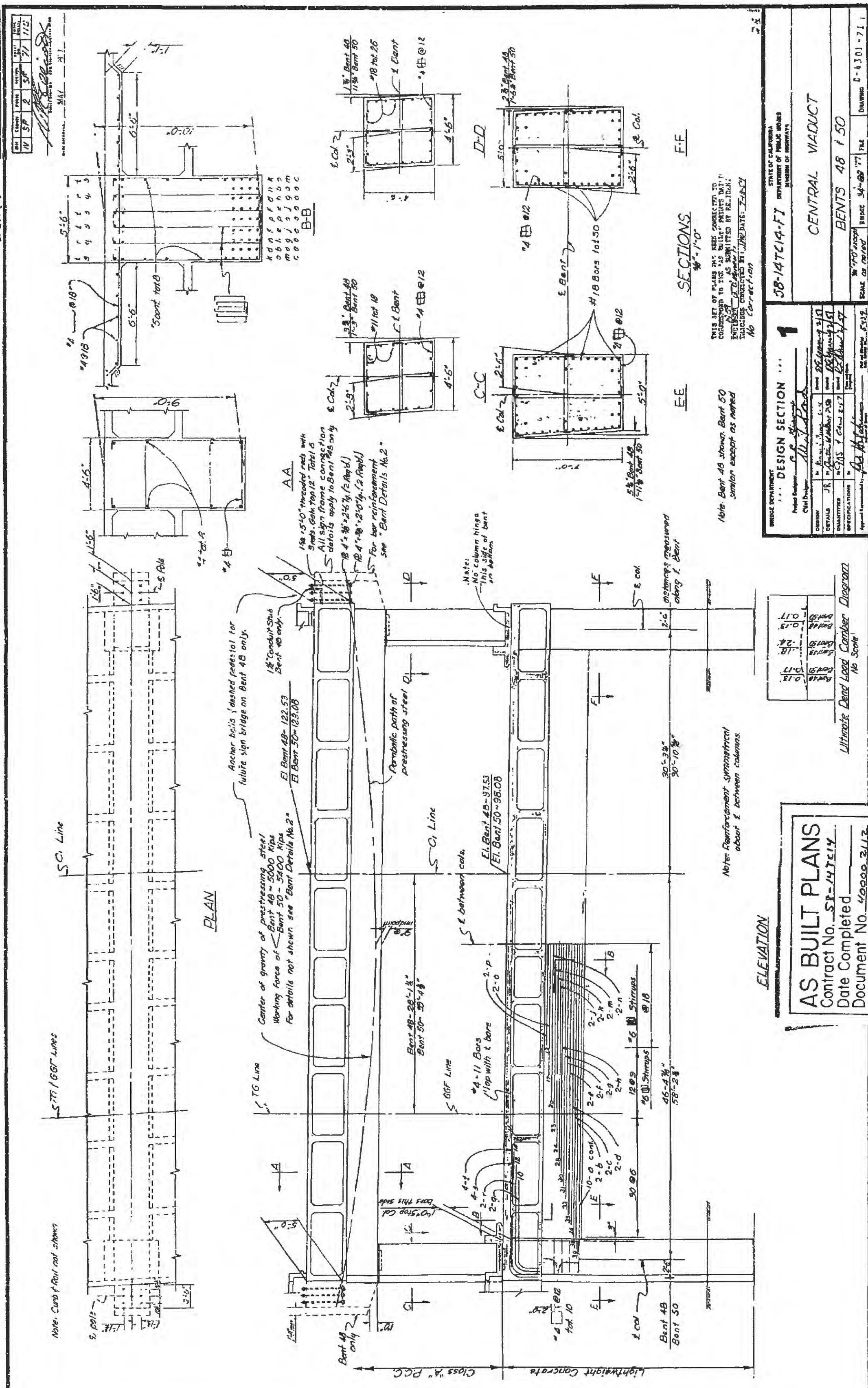


Figure 141.—Elevation drawing of Bent 48 at Central Freeway Viaduct.

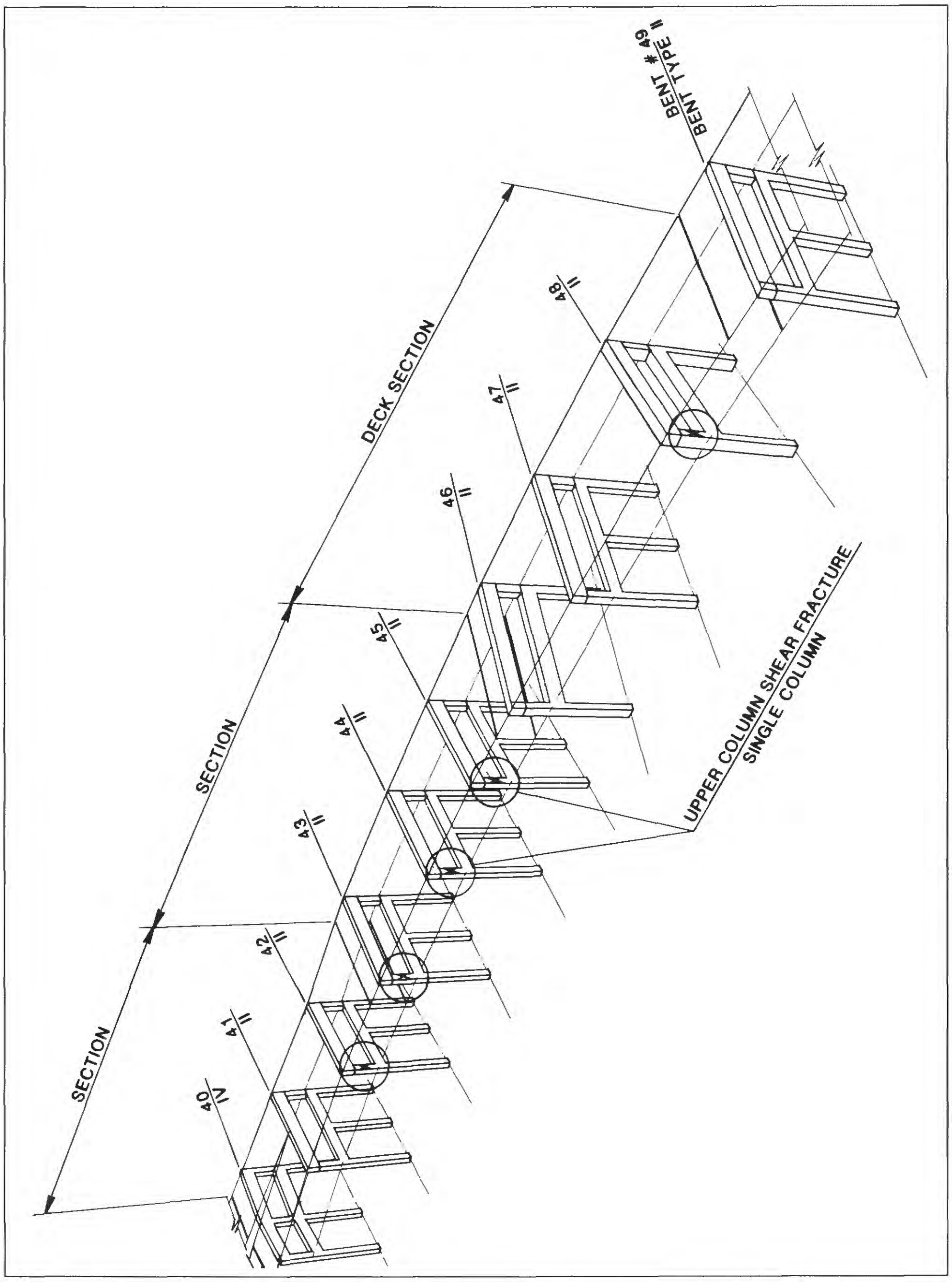


Figure 142.—Damage on Central Viaduct (Thiel, 1990).

ALEMANY INTERCHANGE

DESCRIPTION OF BRIDGE

This viaduct (fig.143) has single and double-deck reinforced concrete box girder superstructures (also a few precast girders) on single and multicolumn bents. It is the southernmost of the double-deck viaducts shown in figure 2. It connects I-101 with I-280 on six separate alignments—line RW and RE to the north, WU and WL to the south, and AL and AU to the east (fig. 144). It is a reinforced concrete structure on pile foundations in fairly solid material. It becomes a fairly dense sandy clay below the first 10 feet of soil. The bedrock (sandstone with layers of shale) is at 60 feet below the surface at Abutment WU-1 and rises to 12 feet at Bent 10. It then descends to 170 feet at Bent 17 and then rises to 65 feet at Bent 23.

BRIDGE DAMAGE

This is the least damaged of the six San Francisco viaducts. Most of the damage appears to be minor spalls from banging during the earthquake. However, because it is a double-deck viaduct and is just south of the heavily damaged Southern Viaduct, it is included here. Structural Maintenance completed their investigation on October 31, 1989, which is summarized in table 17.

WORK RECOMMENDED

Repair column at Bent 27: Cost =$95,000. Place restrainer at Span 27: Cost = $50,000.

REPAIR AND RETROFIT

This viaduct also had a proposed retrofit that was rejected by the Peer Review Panel. However, a very innovative retrofit was done in 1992. This structure, like so many bridges, begins on very short bents and gradually ascends on very tall columns. To prevent substructure damage on the short, stiff bents and to prevent superstructure damage on the tall, strong bents, isolators were installed to give the substructure a uniform response along its entire length and to prevent large plastic moments from forming on the tall columns (fig. 145).

Table 17.—*Summary of damage to Alemany Interchange*

Alignment (see fig. 144)	Damage
RE line Span 3	Hinge is open 2 1/2 in. There is a spall on the right barrier rail.
RE line Bents 4 to 6	Soil pushed away from columns. Fine diagonal cracks in bent caps.
RE line Span 6	Hinge is open 2 in. Spalls on right curb pull box and barrier rail.
RE line Span 8	Hinge is open 2 1/2 in.
RE line Bent 10	Sand boils at bottom of column. Old drain box pushed 1/2 in. from column.
RE line Span 11	Hinge is open 2 1/2 in.
RE line Span 15	Hinge is open 2 1/2 in.
RE line Span 19	Hinge is open 1 3/4 in.
AL line Span 9	Hinge is open 1 1/4 in. at bottom, closed at top right, and open 1 1/2 in. at top left.
AL line Span 14	Hinge is open 2 1/2 in.
AL line Span 19	Bearings look good.
AU line	No damage. Hinge opening 2 in. Span 4, 2 1/2 in. Span 8, and 1 1/4 in. Span 11.
WL line Bent 16	Spall at rail joint.
WL line Span 19	Skewed hinge is open 1 1/2 in. but has no offset.
WL line Bent 27	Small spall in left corner. Column has 1 in. wide crack at side walk.
WL line Span 27	Hinge opening 2 in. on left and 3 in. on right (assume no restrainers installed).
WL line Span 30	Hinge is open 3 in.
RW line Bent 2 to 5	Ground cracks around footing (exposed but no damage found).
RW line Span 3	Hinge is open 2 in.

Figure 143.—Alemany Interchange (view to south).

Br #34-070 / Rte 280 / Post Mile R4.07

Approximate Latitude & Longitude
N. Lat. 37° 44.1' W. Long. 122°24.4'

Epicentral Distance
59 miles

Peak Ground Acceleration	N/S	U/D	E/W
Diamond Heights	0.12	0.05	0.10

Length	Width	Skew	Year Built
11,541'	varies	varies	1960

Main Span Type
Continuous concrete box girders

Average Daily Traffic = **151,700**

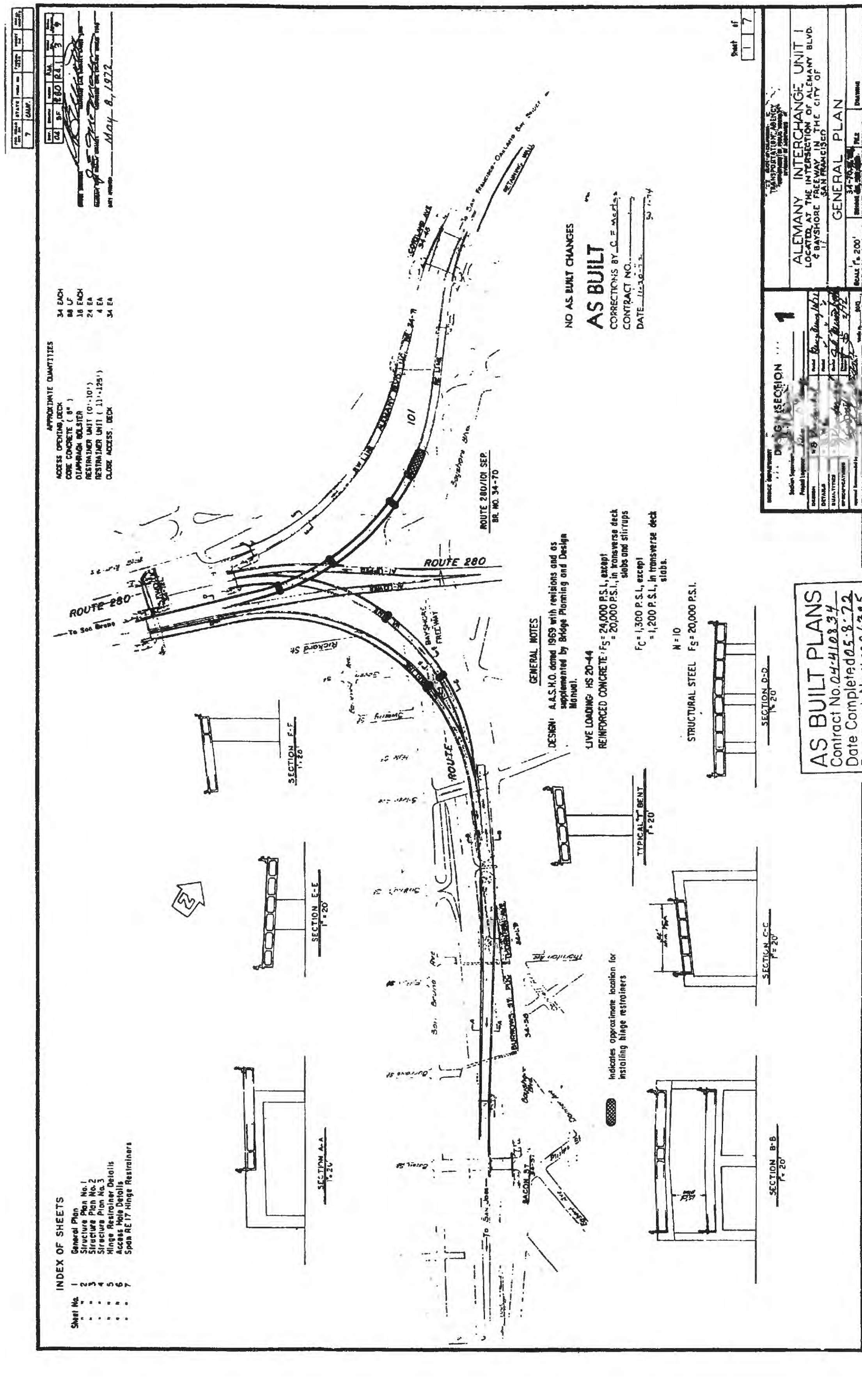


Figure 144.—General Plan drawing showing alignment of Alemany Interchange.

Figure 145.—Completed retrofit for Alemany Interchange.

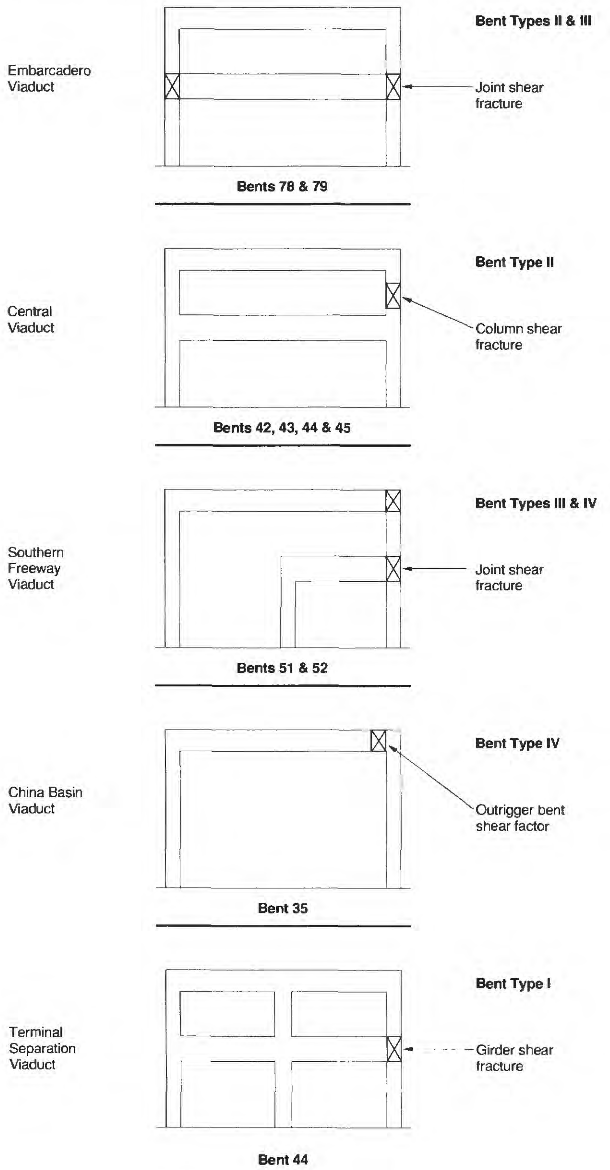


Figure 146.—Summary of damage to San Francisco Viaducts (Thiel 1990).

SUMMARY OF DAMAGE TO SAN FRANCISCO VIADUCTS

On the previous pages we have examined how six San Francisco viaducts performed during the Loma Prieta earthquake. Figure 146 (Thiel, 1990) gives a good sense of the problems that can result from using outrigger bents without good seismic details. There was shear and torsional damage to columns, bent caps, and girders. There were problems of shear transfer in joints. Pin connections performed poorly. Much of the research after the Loma Prieta earthquake looked at better reinforcement details for member or joint shear.

OTHER BRIDGES WITH MAJOR DAMAGE

ROUTE 92/101 SEPARATION

DESCRIPTION OF BRIDGE

Construction of this interchange was begun in 1971, with a portion built as a cast-in-place, prestressed concrete box girder structure on two-column reinforced concrete bents but with the bulk of the interchange made of temporary timber stringer spans on timber pile bents. It remained this way through the 1970's, probably as a result of lack of funding for highway projects. In 1986 the structure was finally completed as a cast-in-place, prestressed concrete box girder interchange with single and multicolumn bents (figs. 147 to 149). The newer portion was built as a modern seismically resistant bridge using all of the new Caltrans innovations such as spiral reinforcement and continuous longitudinal reinforcement in the columns. The structure is supported by 70-ton and 100-ton concrete piles. The log of test borings completed in 1986 indicates soft clays for the first 30 feet gradually becoming very dense at about 65 feet below sea level (fig. 150). It appears that this soft clay soil combined with the curved alignment caused large movements that overstressed areas of the structure.

BRIDGE DAMAGE

The most significant damage to this structure was failure of elastomeric pads at the hinge for Span 19 of the Northwest Connector (fig. 151; table 18). It had been assumed that the greased plates above and below the bearing would allow the hinge to move during the earthquake. Apparently the grease and neoprene pads harden over time and the pads can fail in shear since they are not designed for earthquakes. Replacing bearings at hinges is a difficult and expensive procedure. Large jacking frames had to be built on the ground that reached the superstructure soffit. A better hinge design is needed that will allow maintenance crews to reach the bearings from inside the cells of the box girder superstructures. A report was prepared by Structural Maintenance on November 17, 1989, that

Figure 147.—Route 92/101 Interchange.

Br #35-252L/R / Rte 92 / Post Mile R11.78

Approximate Latitude & Longitude
N. Lat. 37° 33.2' W. Long. 122°17.7'

Epicentral Distance
42.8 miles

Peak Ground Acceleration	**N/S**	**U/D**	**E/W**
Foster City	0.29	0.11	0.26

Length	**Width**	**Skew**	**Year Built**
4,282' & 4,362'	38'	Varies	1971

Main Span Type
Continuous concrete box girders

Average Daily Traffic = 44,350

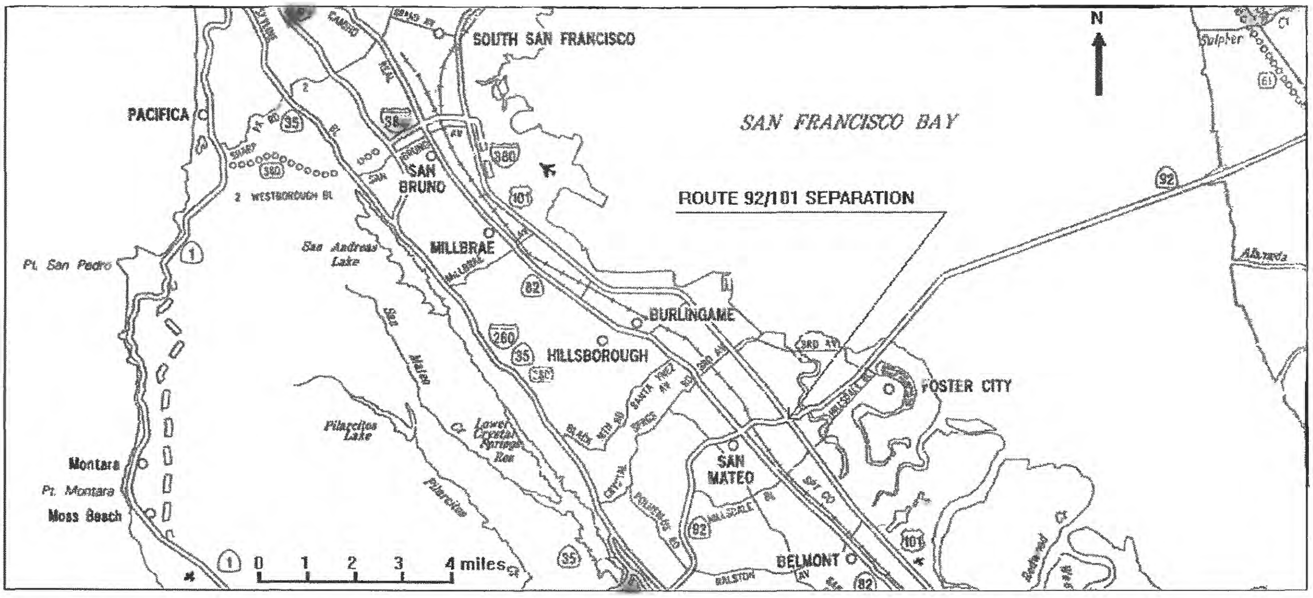


Figure 148.—Location of Route 92/101 Separation.

gives a complete description of bridge damage and repairs. The following is a brief summary based on that report.

DAMAGE AND REPAIR TO THE NORTHWEST CONNECTOR

The Northwest Connector consists of three longitudinal frames on single column bents. The superstructure is a prestressed concrete box with two vertical, interior girders and two sloping, exterior girders (figs. 152 to 154). The depth varies from 5 feet at the abutments to 8 feet in the central portion of the bridge. The traveled way is 26.25 feet on a bridge width of 29.25 feet. The northerly frame is about 600 feet long, extending from the hinge in Span 11 to the hinge at the fifth point of Span 16. Then the center frame extends about 700 feet to the hinge at the five-sixths point in Span 19. From this hinge, the southerly frame runs about 450 feet to Abutment 23.

The hinge at the center frame suffered serious damage due to relative movement between the three frames during the earthquake (fig. 155). The elastomeric bearing pads did not delaminate, but certain layers were compressed and worn down. Inspection of the joints showed that the suspended side of the Span 19 hinge was 1.5 inches lower than the support side. On October 25, 1989, work began to replace the elastomeric pads at Span 19. The superstructure was jacked up, new elastomeric pads and restrainers were placed in the hinge, and the bridge was reopened to traffic on November 2, 1989. The Span 16

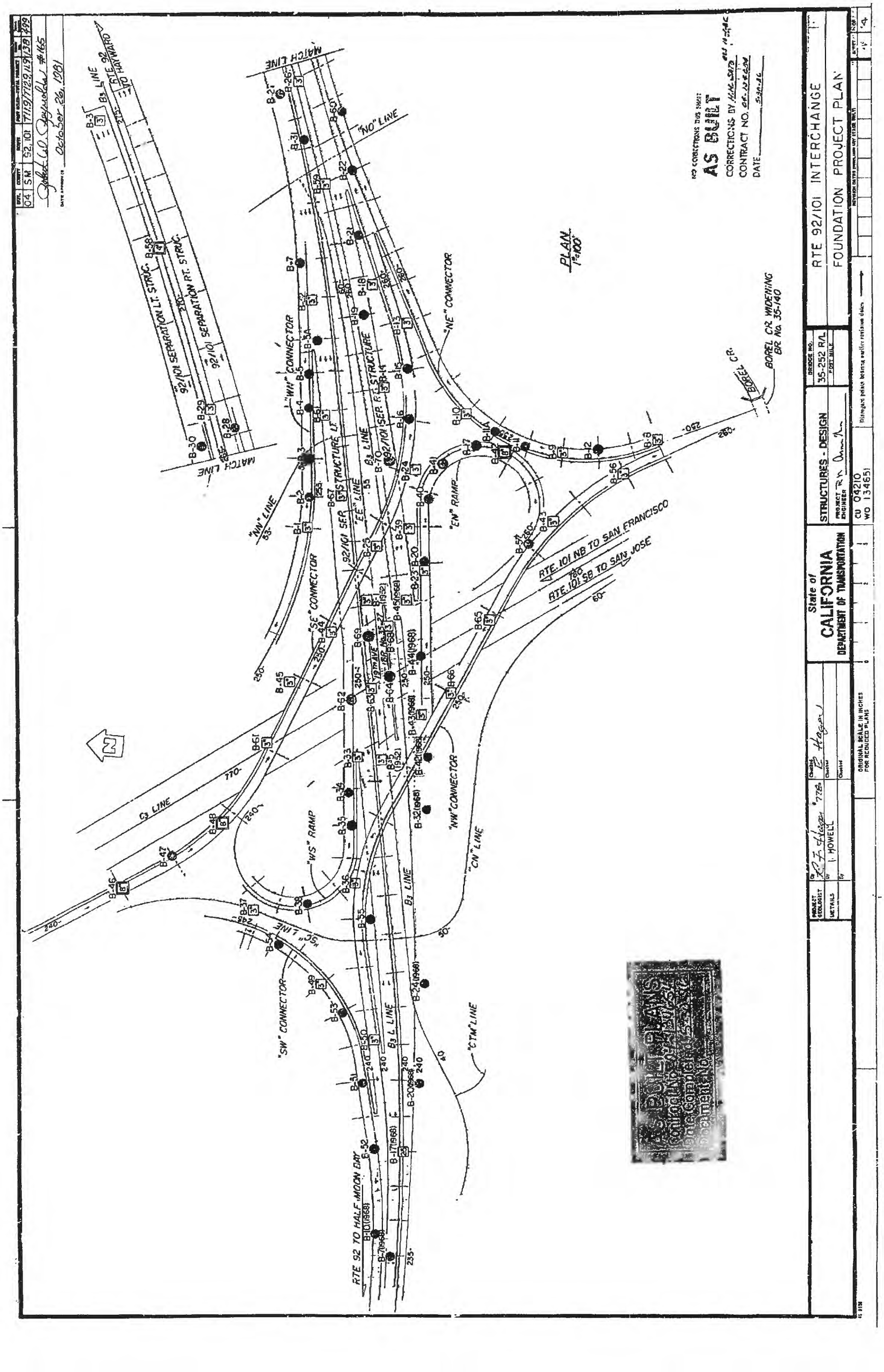


Figure 149.—Plan drawing of the Route 92/101 Interchange.

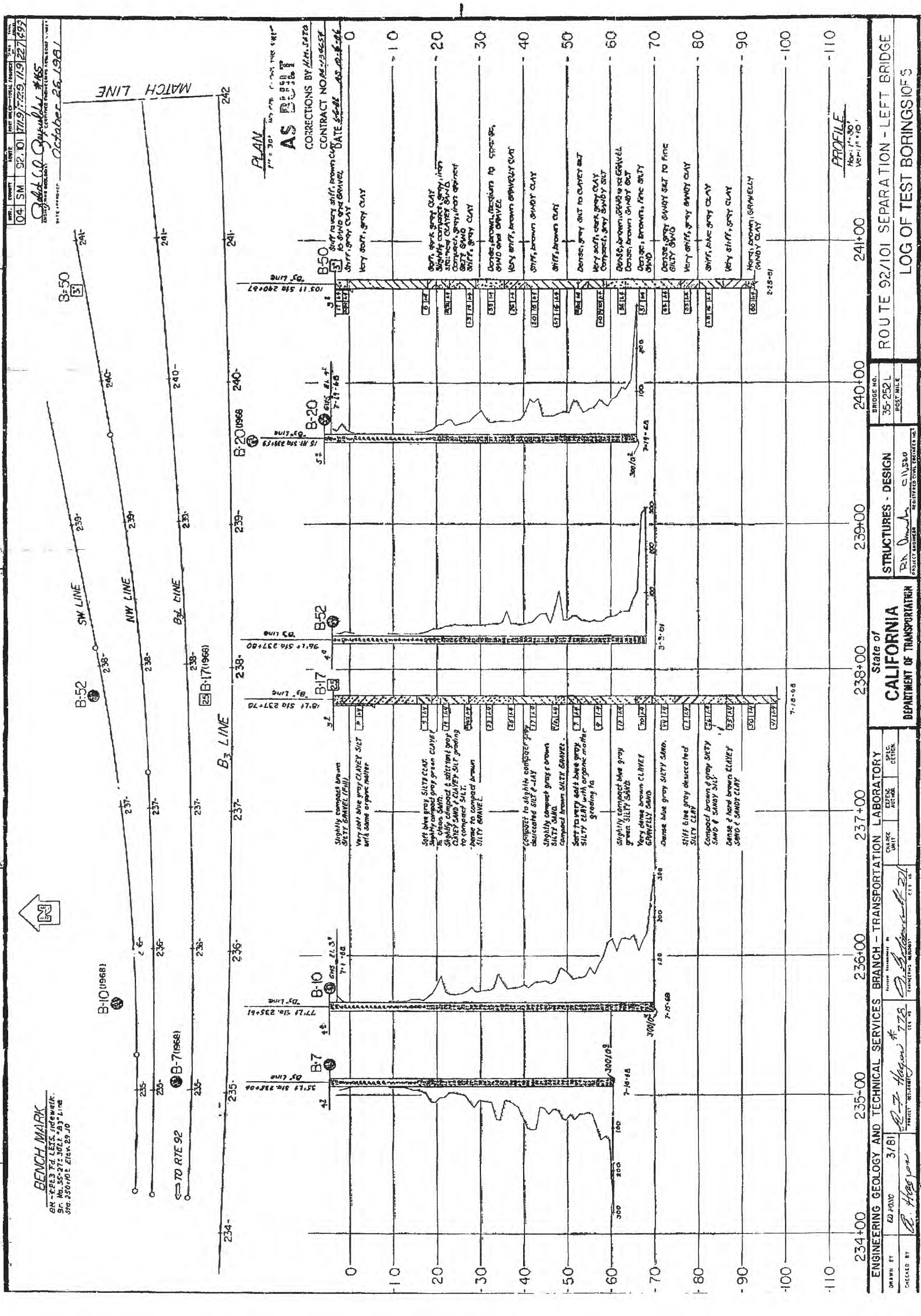


Figure 150.—Log of borings for Route 92/101 Interchange.

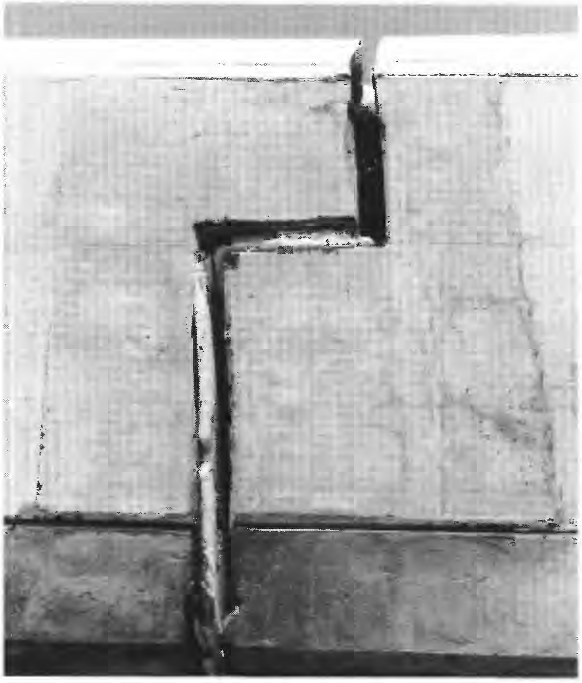


Figure 151.—Hinge opening at Northwest Connector of Route 92/101 Interchange due to earthquake.

hinge is located above a San Mateo city street. On November 9, 1989, the bridge was again closed to traffic to replace the bearings at Span 16. The Northwest Connector ramp was re-opened to traffic at 5:20 p.m., November 12, 1989. The jacking towers were dismantled on November 13. Support timbers and soil were removed and the city street was re-opened to westbound traffic on November 15, 1989.

To a casual observer the Southeast and Northwest Connectors appear to be nearly identical structures. They are only 350 feet apart at their center frames which are both supported by long, single column bents founded on prestressed concrete piles. However, the Southeast Connector has features that give it much better stability. The center frame of the Southeast Connector is supported on four single-column

Table 18.—Hinge movement for Northwest Connector of Route 92/101 Interchange [Openings on November 17, 1989 at 55°F.]

Hinge Location	On Left Side	On Right Side
Northwest Connector Span 11 hinge	3 3/4 inches	3 5/8 inches
Northwest Connector Span 16 hinge	6 1/2 inches	6 1/2 inches
Northwest Connector Span 19 hinge	5 1/8 inches	5 1/8 inches
Northwest Connector Abutment 23	2 7/8 inches	2 7/8 inches
Southwest Connector Abutment 1	1 7/8 inches	2 inches

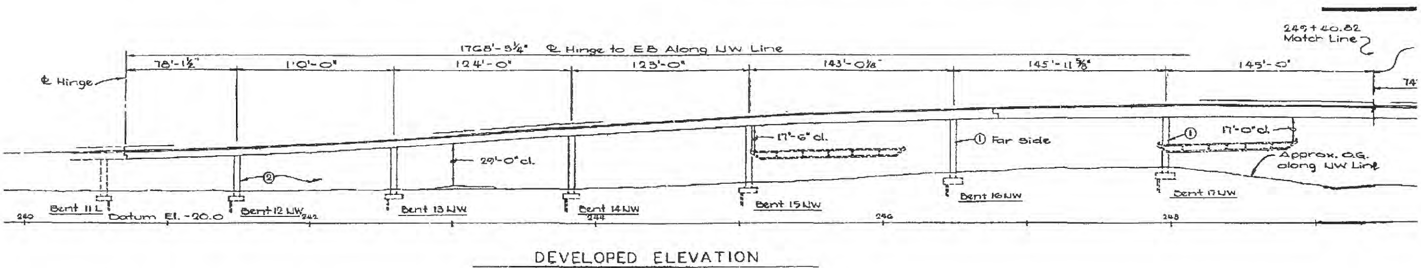


Figure 152.—Elevation drawing of the Northwest Connector of Route 92/101 Interchange.

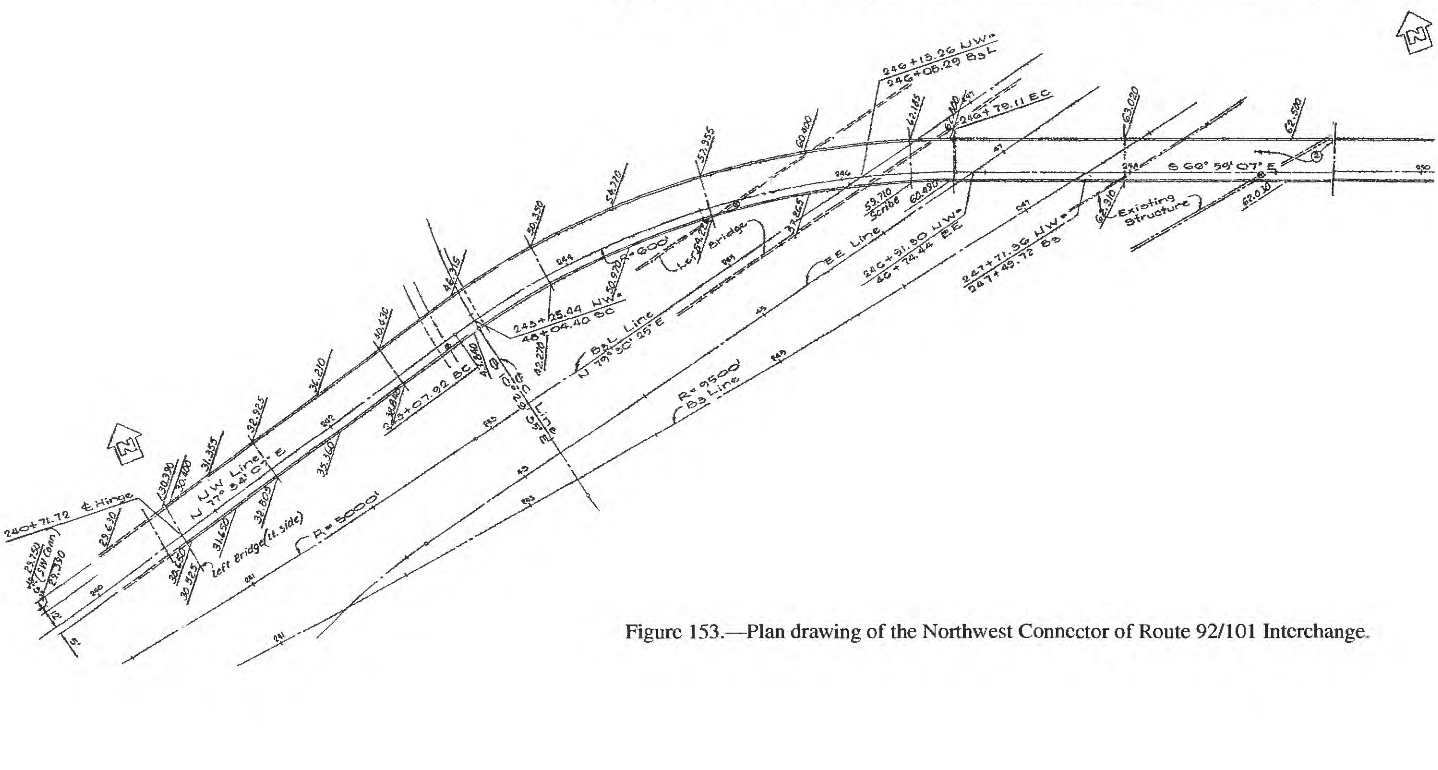
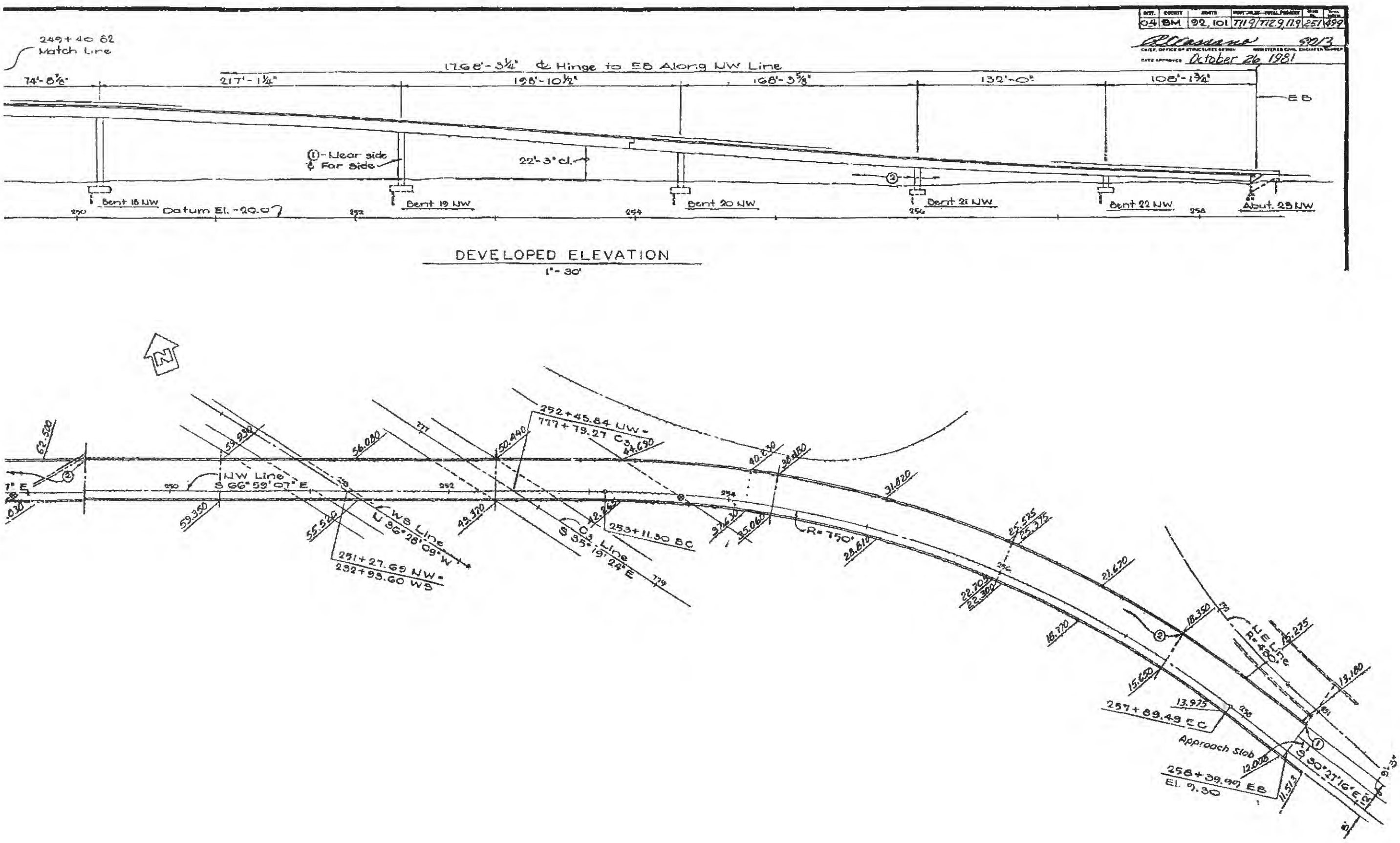


Figure 153.—Plan drawing of the Northwest Connector of Route 92/101 Interchange.

bents, while the Northwest Connector has only three single-column bents. Moreover, at the Southeast Connector, Bent 6 and Bent 9 each has a 6-foot thickness of sandy gravel material at the surface over about 28 feet of soft clay, resting upon higher density, stiff material. Bent 7 and Bent 8 have 18 feet of stiff highway embankment material over 20 feet of soft clay resting on stiff material. This condition is not present for the Northwest Connector, where there is only a dense blanket of 3 to 5 feet over the 30-foot layer of soft clay. The stiffer material at the Southeast Connector may have provided enough resistance to prevent large hinge movements.

Figure 154.—Northwest Connector on Route 92/101 Interchange.

Figure 155.—Spall at Northwest Connector of Route 92/101 Interchange.



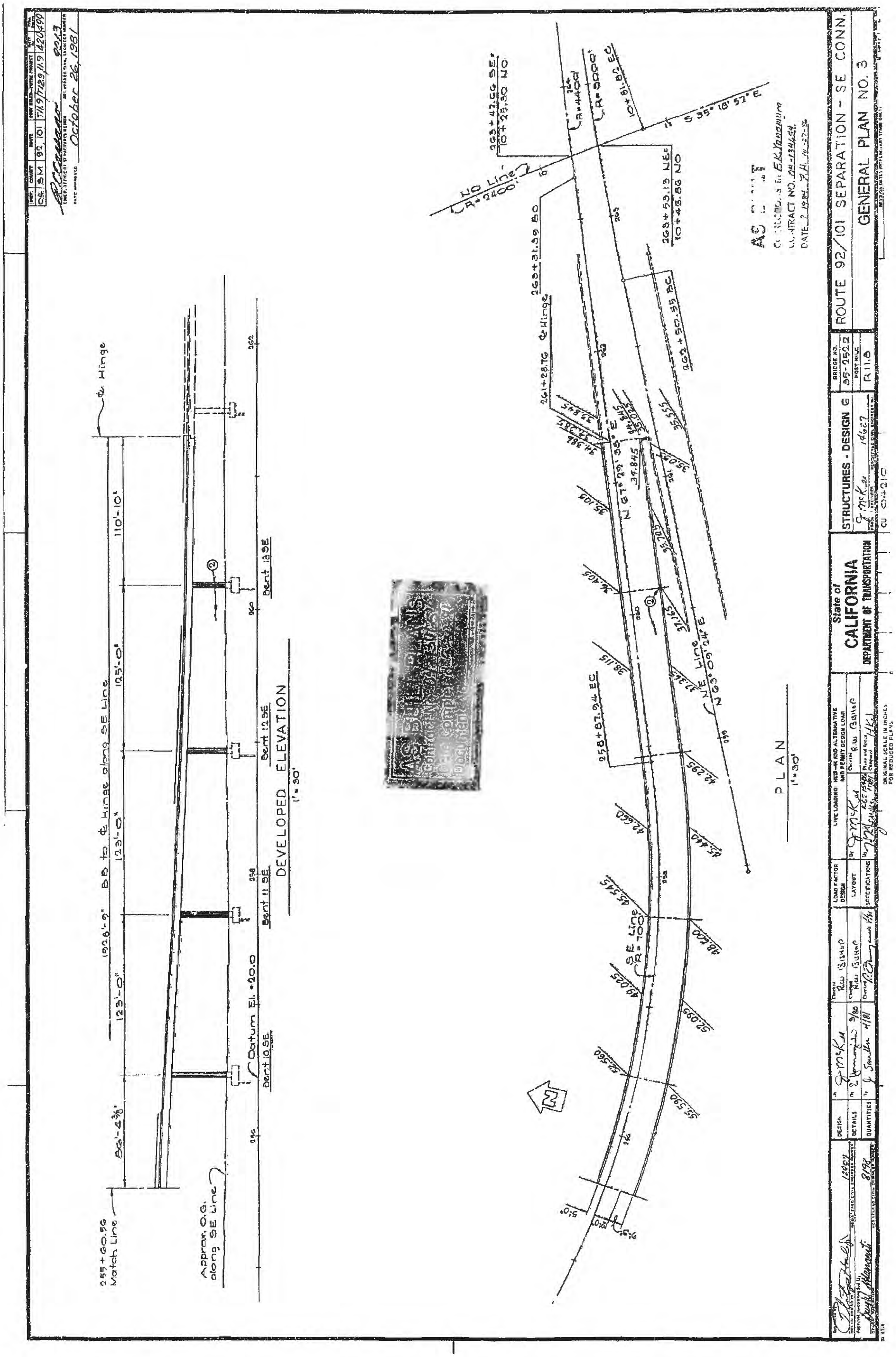


Figure 156.—General Plan drawing for the eastern portion (including Bents 12SE and 13SE) of Southeast Connector of Route 92/101 Interchange.

DAMAGE TO THE SOUTHEAST CONNECTOR

Figure 156 shows the geometry of the eastern portion of the Southeast Connector. At the 2-year inspection of this interchange (on February 1, 1990), two spalls were noticed at the column of Bent 12SE. The spall in the south side of the column is just above the collar around the base of the column. This spall is oblong in shape, measuring about 2 feet vertically, and about 1 foot horizontally. It is very shallow, only in the surface of the column concrete. The other spall is very small, located on the opposite side of the column, also at the base of the column, just above the collar.

Because the column of Bent 12SE was spalled near the bottom of the column, it was decided to investigate the condition of the footing block concrete and the supporting piling. It was decided also to investigate the condition of the footing of Bent 13SE as well, although there were no spalls at that column.

The soil around the base of most of the columns of this structure in the vicinity of Bent 12SE and 13SE had consolidated because of the October earthquake. There is a small crack in the soil around these single column bents, outlining the shape of the footing (fig. 157). The consolidation is not severe, only an inch or two in some locations. At other locations, only a small crack is visible in the soil, outlining the edges of the footing. In most locations, the cover over the footing block concrete is only 2 to 3 inches.

As part of the 1990 inspection, Dave Aro, Bridge Supervisor, and Steve Maas, Leadworker, both assigned to the Caltrans Foster City Maintenance facility, excavated two sides of the footing at two locations: Bent 12SE and Bent 13SE. A trench about 3 feet wide was dug along the north edge and the east edge of both footings. The soil side of the trench along the east edge was sloped to prevent the material from caving into the trench while Steve and I were working in the excavation. The trench was excavated to a depth of about 6 feet. (The vertical dimension of the footing block concrete is 5.00 feet.) The trench along the north side was made deeper

Figure 157.—Foundation movement at Southeast Connector of Route 92/101 Interchange.

near the northeast corner of the footing to provide a sump for ground water to drain while they studied the footing block concrete and the condition of the corner pile of the footing. In addition to excavating along the two edges of the footing, the north and east portions or the top surface of the footing were exposed and swept clean. The inspection showed the following:

1. There are no spalls at the base of the columns where the column joins the top of the footing.

2. There are no cracks in the footing block concrete either in the top surface or in the vertical faces that were exposed by the trench excavations. It was found that there are no vertical (punching or vertical shear) cracks, no horizontal (delamination) cracks, and no diagonal (horizontal shear) cracks.

3. The pile in the northeast corner of each of these two footings had no visible distress.

4. There are some fine to medium-wide cracks in the concrete collar around the base of the columns. These cracks stop at the bottom of the collar. They do not extend into the footing.

Since no earthquake damage was found in the footing block concrete or in the supporting piling of Bent 12SE and Bent 13SE, no further footing excavation and investigation is planned. The findings at these two locations indicate that the other footings of this structure also remain in good condition.

DAMAGE TO THE NORTHEAST CONNECTOR

In 1991, Professors Saidi and Maragakis from the University of Nevada at Reno made a study of several wide soffit cracks near the hinge of the Northeast Connector (figs. 158 and 159). These cracks were reported after the earthquake, but an investigation had to wait until access holes were cut into the soffit (November 21, 1991). The cracks, along with some damage to the barrier rails at the hinge, suggested that they were caused by the earthquake (figs. 160 to 162). Because the cracks were vertical along the webs, they could not have been caused by shear. Because the cracks were near the hinge, they could not be caused by flexure. Therefore, it was felt that the cracks were caused by tension. The eccentric location of the restrainers could possibly have produced a flexural moment which may have caused these cracks. Also, the role of prestressing tendons combined with cable restrainers needs more study.

There was also damage to the ramps on this interchange. Refer to figure 141 for the orientation of the ramps. Table 19 summarizes the damage.

CONCLUSION

This interchange may have had some problems during construction that were exacerbated during the earthquake. These problems could have been from using a poor quality

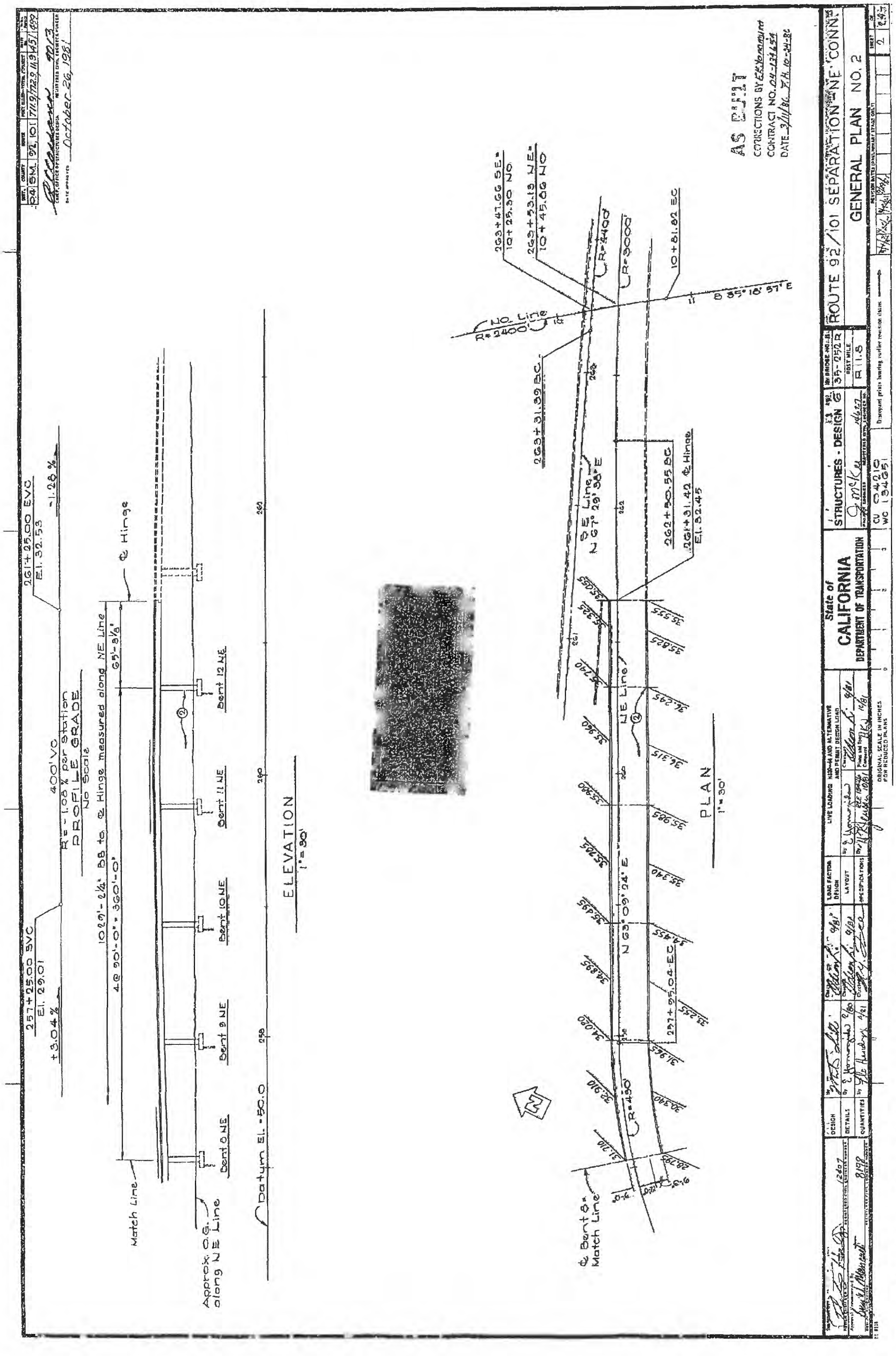


Figure 158.—General Plan drawing of east end of Northeast Connector of Route 92/101 Interchange.

concrete or from early removal of the falsework that caused the many soffit cracks and the poor performance of the bearings that is continuing long after the earthquake. Also the maintenance reports on this structure reveal a variety of problems. Since this is a recent structure with modern seismic details, the columns performed well and there was no serious damage. However, replacing bearings is expensive and requires the temporary closure of parts of the interchange. The construction of large, curved interchanges on soft Bay muds should be avoided since it causes large movements and a lot of banging of hinges and spalling of concrete.

The performance of this structure during the earthquake shows that modern seismic details can prevent major damage but cannot prevent a variety of minor damage during large earthquakes.

Figure 159.—Portion of Northeast Connector of Route 92/101 Interchange where soffit cracks were investigated.

Figure 160.—Soffit cracks on Northeast Connector of Route 92/101 Interchange.

Figure 161.—M. "Saiid" Saidi investigating cracks in interior bay of Northeast Connector of Route 92/101 Interchange.

Figure 162.—Barrier rail damage at hinge of Northeast Connector of the Route 92/101 Interchange.

Table 19.—Summary of damage to ramps on Route 92/101 Interchange

Locations on SE Ramp	Damage
Abut. 1	Inside face of rail spalled. Joint open 3 in. (wingwall rail to bridge rail open 6 1/4 in.).
Span 9	Soffit spalled at hinge in several places.
Bent 12	Column spalled at reduced section by ground. Ground cracks match footing perimeter.
Span 13	Soffit spalled at hinge and at Bent 13.
Locations on ES Ramp	**Damage**
Span 10	Railing spalled where ramp joins mainline eastbound.
Span 13	Left exterior girder spalled at hinge. Joint open 1 1/2 in.
Bent 15	Top of right column spalled on inner side.
Bent 16	Top of left column spalled on outer side.

Br #36-88R/L / Route 1 / Post Mile R1.59

Approximate Latitude & Longitude	Peak Ground Acceleration N/S U/D E/W	Main Span Type
N. Lat. 36° 54.4' W. Long. 121°46.4'	Watsonville 0.28 0.66 0.39	Continuous concrete 'T' girders

Epicentral Distance	Length	Width	Skew	Year Built	Average Daily Traffic = 15,900
10.8 miles	763' & 800'	28'	30°	1965	

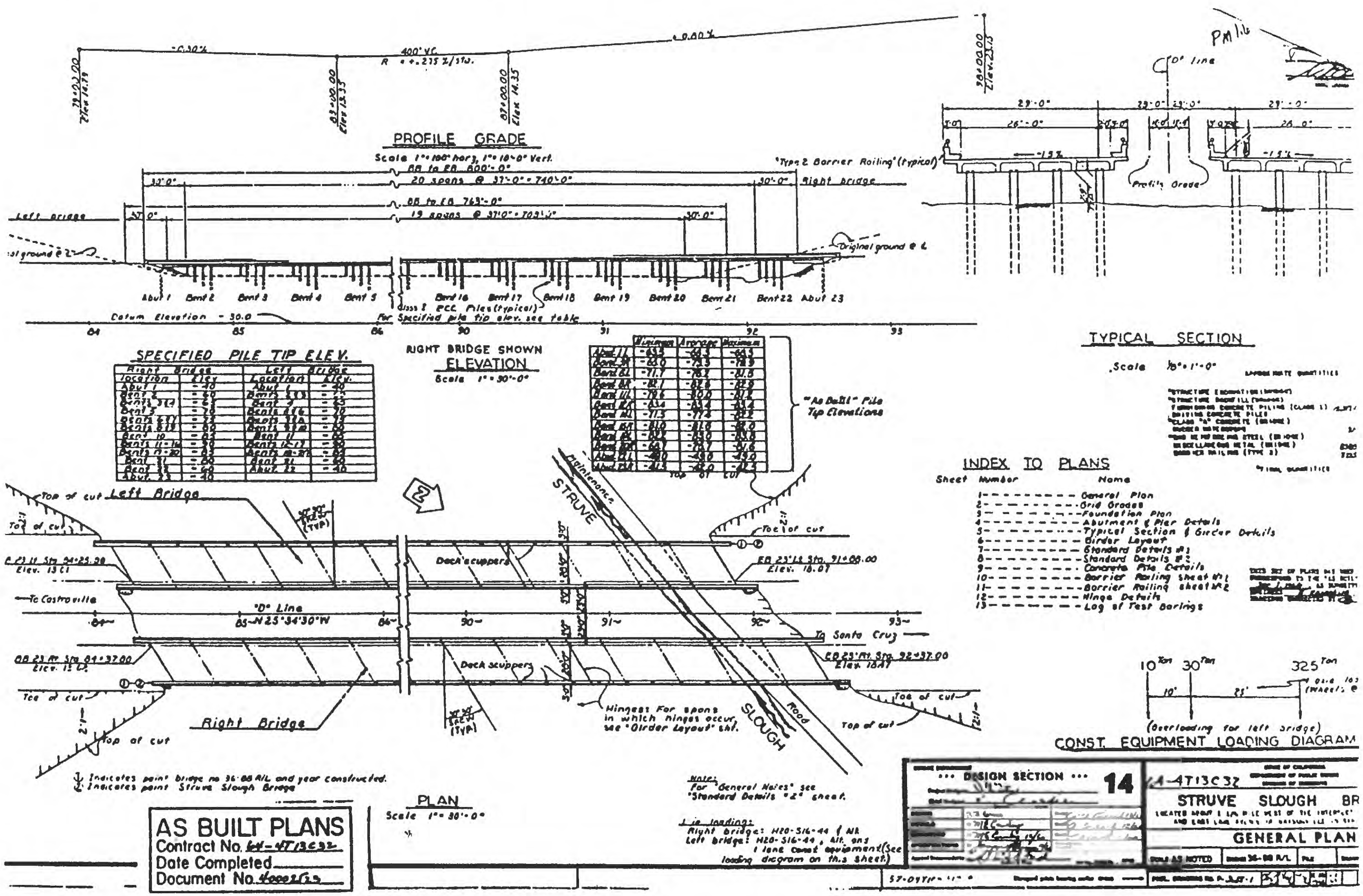


Figure 163.—General Plan drawing of Struve Slough Bridges.

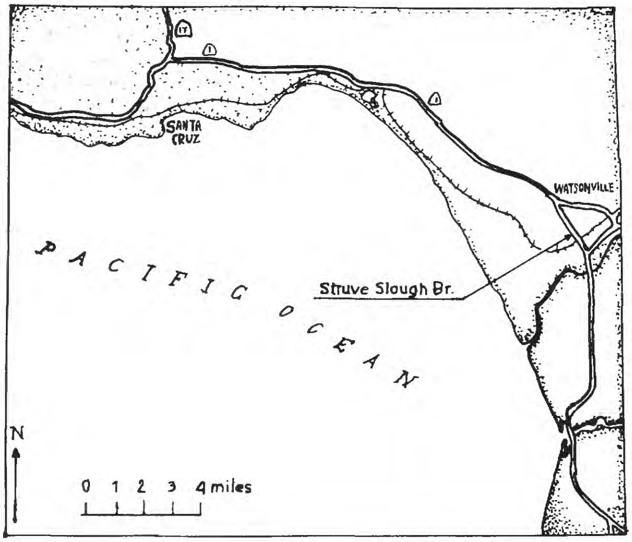


Figure 164.—Location of Struve Slough Bridges.

STRUVE SLOUGH BRIDGES

DESCRIPTION OF BRIDGE

The Struve Slough Bridges were two reinforced concrete "T" girder structures built in 1965 (figs. 163 to 165). The superstructures were continuous for several spans, with hinges located in spans 6, 11, and 17 on the right bridge and 6, 11, and 16 on the left bridge (21 total spans). The bridges were supported on four, 80-foot-long, 14-inch-diameter driven concrete pile extensions at each bent that were embedded into special diaphragms that acted like bent caps. The reason the piles were so long is illustrated in figure 166, which shows that these bridges were sitting in very soft clay. The bridges had end-diaphragm-type abutments, which meant that temperature movement was controlled at the three hinges. The bridges had a seismic retrofit in 1984 which consisted of cable restrainers that tied the hinges together.

BRIDGE DAMAGE

The main problem for these bridges was the weak soil that supported them. Once the very soft clay started moving,

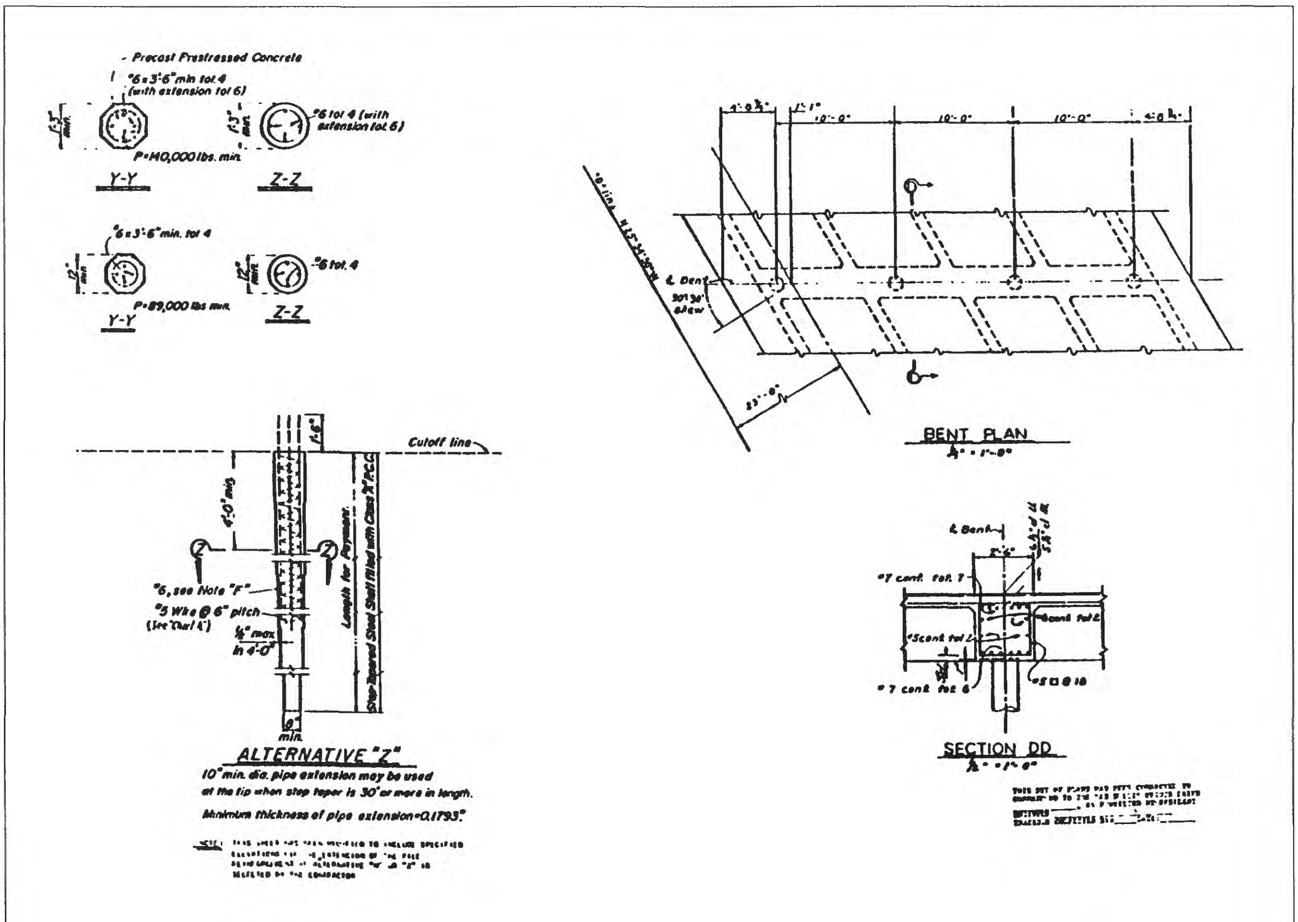


Figure 165.—Bent and pile details for Struve Slough Bridges.

it dragged the pile extensions along with it, causing severe damage to both bridges. The left bridge collapsed from spans 6 to 17. Many of the piles were crushed. Several piles in Bents #14 and #15 were displaced longitudinally, separated from the bent cap, and punched though the bridge deck (figs. 167 to 171).

There was some speculation that the high recorded vertical motion may have had an influence on the bridge damage, particularly since several pile tips were driven through the concrete deck slab. However, an analysis of this structure (Saadeghvaziri, 1990) showed that the fundamental vertical period of 0.20 seconds was far from the power component of the ground motion. Also, the vertical mode had a small participation factor.

The restrainers performed surprisingly well during the earthquake, holding the hinges together even on the collapsed portion of the bridge. There was a great deal of displacement. Abutment 1 of the left bridge moved 6 inches longitudinally, and the approach fill settled 3 inches behind the abutment. The bridge superstructure displaced up to 2 feet at midspan. Almost all the pile extensions were damaged. There were large holes hollowed out in the soil around the piles. The right bridge did not perform much better. The superstructure dropped several feet at the hinge for Span 11, causing a depression for several spans. Most of the pile extensions were severely damaged.

BRIDGE REPLACEMENT

Damage to this bridge stopped all traffic from traveling on the Pacific Coast Highway. Because of the severe damage and potential for more damage during the next earthquake, it was decided to replace the damaged bridges with new structures that could survive the large movement of the soft soil during earthquakes. Caltrans Design Section 11 was given the task of coming up with a new design in 8 days. At first they thought they could use the standard design charts for a slab bridge. However, putting in pile extensions that were basically unsupported through 70 feet of soft soil required a dynamic analysis.

The final design called for 24-inch concrete piles cast into driven steel pipe piles and socketed 10 feet into bed-

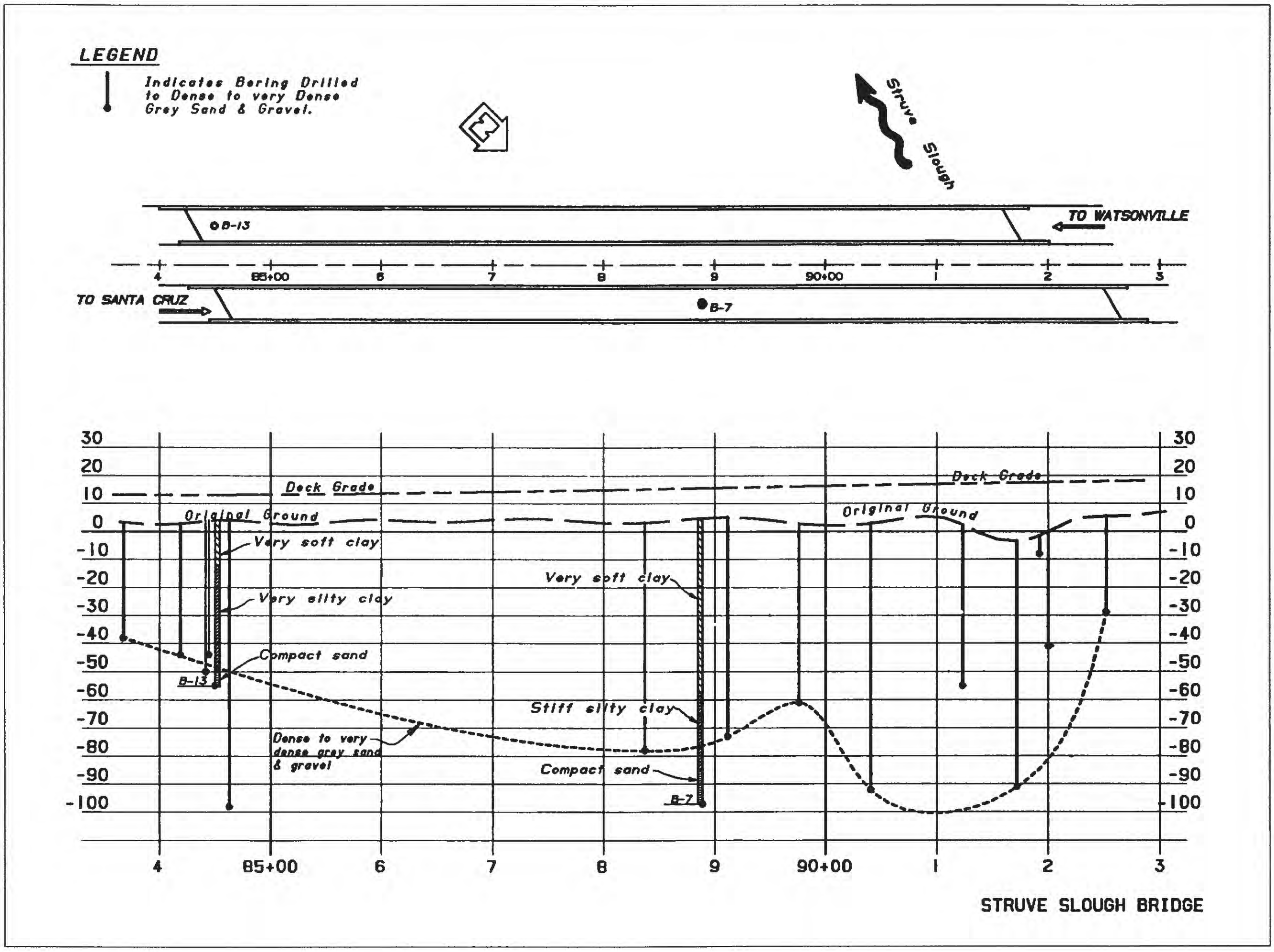


Figure 166.—Soil profile for Struve Slough Bridges.

rock. The superstructure was designed as two continuous flat slabs without joints (figs. 172 to 173. The complete package of plans, specifications, and quantities, was ready for contractors bids on November 15, 1989.

This was the first incentive/disincentive contract offered by Caltrans. These types of contracts have become very popular as a way to quickly rebuild damaged bridges after disasters. This contract offered an incentive of $30,000 a day to finish early and a disincentive for finishing late of $32,000 a day, with 90 calendar days to finish the contract. Typically, contractors will consider the incentive/disincentive when computing their bids, resulting in a bid that is below the estimated cost of construction. For this work the $4.85 million contract provided for two 830-foot-long 40-foot-wide slab bridges in a swamp. There were environmental constraints on the project, as well as problems working on extremely soft soil and having to drive 200 new piles (the new pile locations were specified to avoid the existing piles). The existing structures were removed in the first 5 days and it took another 27 days to drive the piles. The contractor asked Caltrans to allow him to support the falsework on brackets attached to the steel shells of the existing piles. Steel stringers spanned between the piles and supported the plywood formwork. Ironworkers then came in and placed the reinforcement. Then 4,115 cubic yards of concrete was poured and cured. The job was completed in 55 days (35 days before the end of the contract) on January 25, 1990. Work was done around the clock and demonstrated why cast-in-place bridge construction dominates in California. The construc-

Figure 168.—Cable restrainers holding hinge together on Struve Slough Bridges (superstructure was unsupported beyond this point).

Figure 169.—South abutment of left Struve Slough Bridge (view to north).

Figure 167.—Pile damage on Struve Slough Bridges.

tion cost for this structure with the incentive and bonus was $83 per square foot. The average cost for slab bridges is about $60 per square foot. However, the costs would have been slightly higher due to environmental constraints and working in a swamp. The original contract put a $600,000 cap on the incentive, but Caltrans amended the contract for a bonus payment if work proceeded around the clock. Apparently, the State and Federal agencies felt that opening up the Pacific Highway a month early was worth the extra cost. For more information about the environmental, economic, and construction problems on this contract, refer to the references listed below.

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Figure 170.—Piles penetrating deck of left Struve Slough Bridge (view to south).

Figure 172.—Completed new Struve Slough Bridges.

Figure 171.—Reconstruction of Struve Slough Bridges.

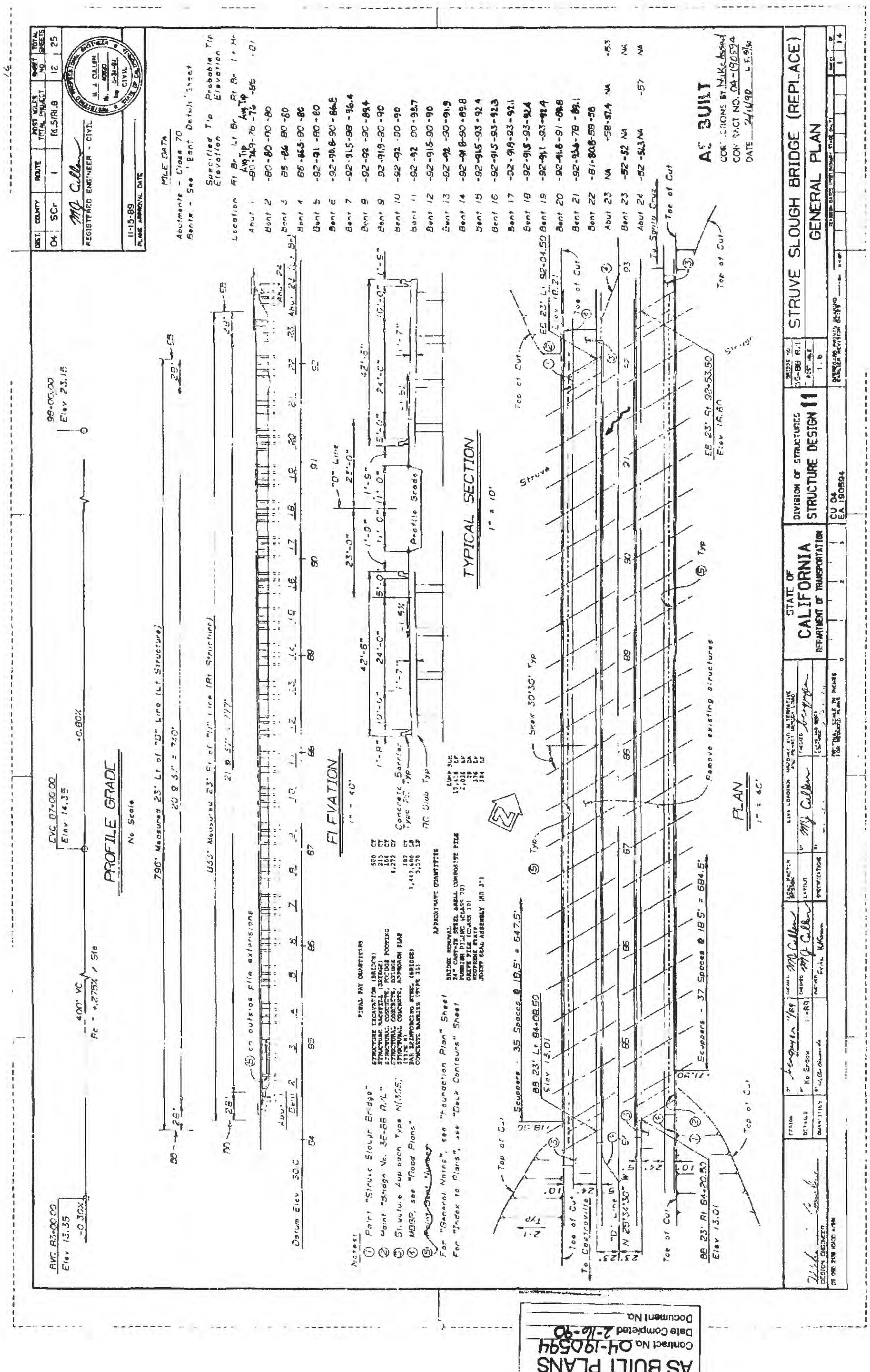
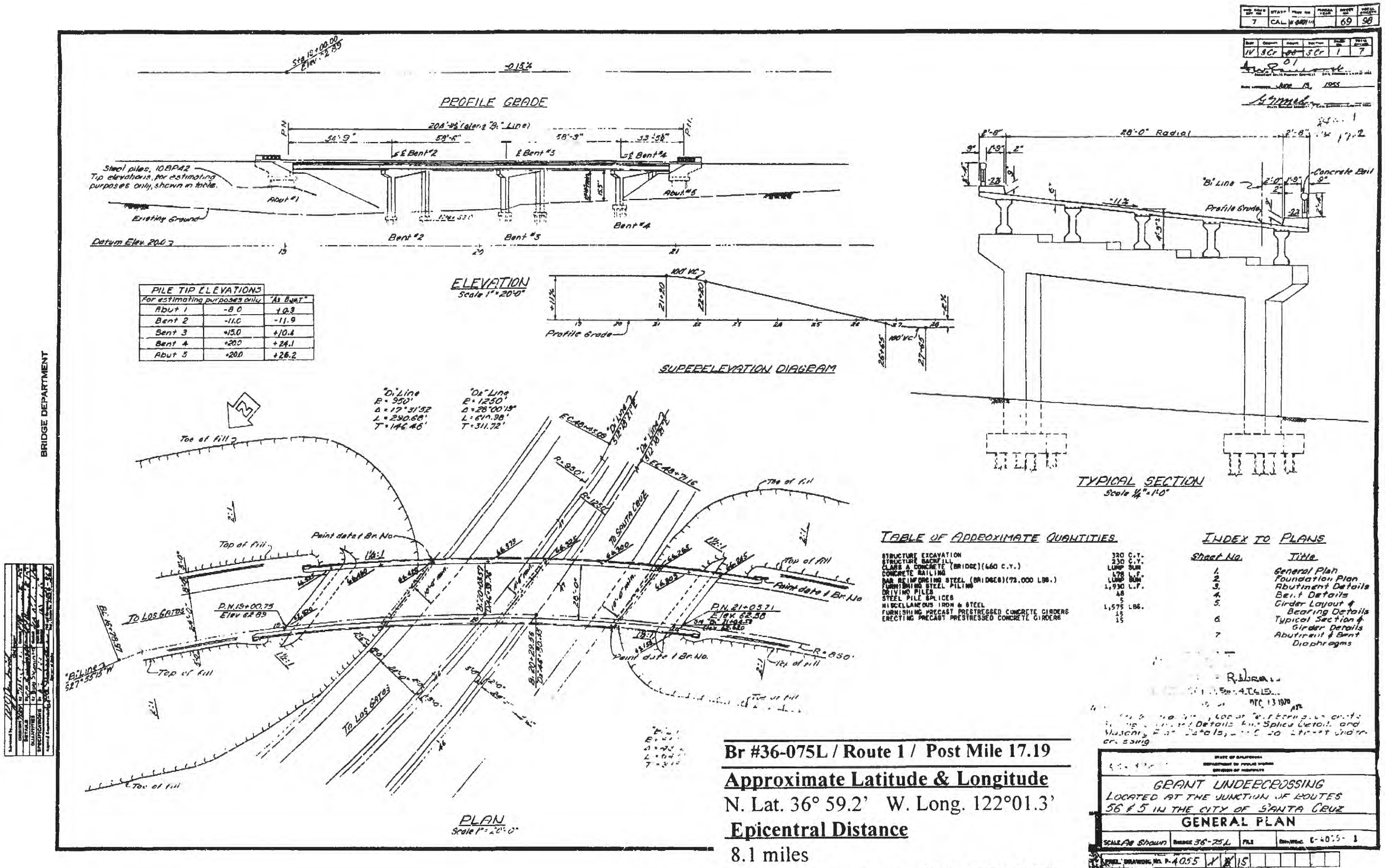


Figure 173.—General Plan drawing of new Struve Slough Bridges.



Br #36-075L / Route 1 / Post Mile 17.19

Approximate Latitude & Longitude

N. Lat. 36° 59.2' W. Long. 122°01.3'

Epicentral Distance

8.1 miles

Peak Ground Acceleration	N/S	U/D	E/W
Santa Cruz	0.44	0.40	0.47

Length	Width	Skew	Year Built
205'	28'	Varies	1956

Main Span Type

Precast Prestressed 'T' girders

Average Daily Traffic = 28,500

Figure 174.—General Plan drawing of Grant Undercrossing.

GRANT UNDERCROSSING

DESCRIPTION OF BRIDGE

This bridge has a precast, prestressed concrete "T" girder superstructure from Span 1 to Span 3 and a reinforced concrete "T" girder superstructure for Span 4 (figs. 174 and 175). The span lengths are 54.75, 58.5, 58.25, and 33.5 feet, for a total length of 205 feet. The bridge spans from east to west on a 850-foot radius curve. The superstructure is supported on two rectangular column bents, with the bottom of the columns pinned. The main column reinforcement is 1-inch-wide square bars and one-half-inch hoops at a 12-inch spacing.

Abutment 1 is a pedestal type with a backwall, while Abutment 5 is an end diaphragm type. The approaches and abutments are in fill. The foundations have pile caps with a bottom mat of reinforcement and driven steel piles.

BRIDGE DAMAGE

At Abutment 1 the right wingwall broke off. The fill had settled as much as 12 inches on the outside of the wingwalls (figs. 176 and 180). The approach also settled about 6 inches. There was curb and railing damage all along the structure (figs. 178 and 179). At Bent 4, Span 3 moved transversely about 1 inch to the right, spalling the diaphragm and shearing the bearing plate bolts and grout pads. At Abutment 5 the wingwalls moved outward, causing the fill to settle (fig. 177).

The moderate damage sustained at this bridge is typical after large earthquakes, particularly for short multispan bridges that span between two large embankment fills. These shorter structures get hammered by the soil behind the abutments, driving them back and forth longitudinally and transversely.

Figure 176.—Settlement along abutment of Grant Undercrossing (photograph by Mike Van de Pol).

Figure 175.—Location map of Grant Undercrossing.

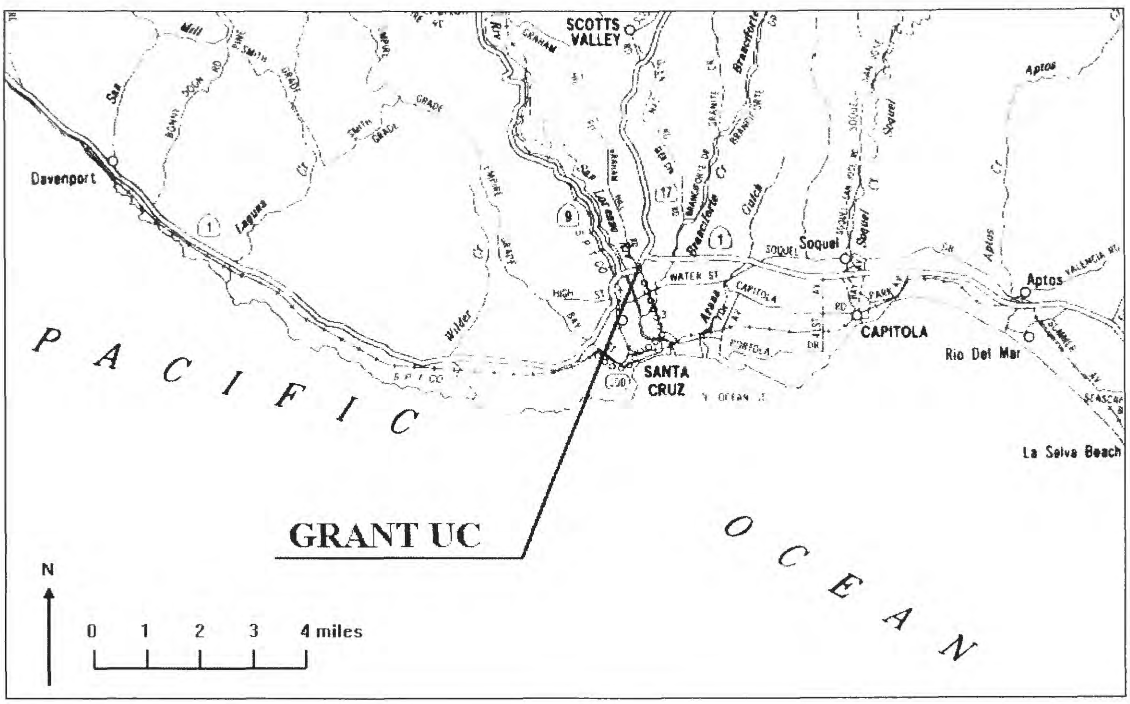


Figure 177.—Abutment 5 approach to Grant Undercrossing (view to west) (photograph by Mike Van de Pol).

Figure 179.—Exterior spall at Bent 4 of Grant Undercrossing (view to south) (photograph by Mike Van de Pol).

Figure 178.—Barrier rail on north side of Bent 3 at Grant Undercrossing (photograph by Mike Van de Pol).

Figure 180.—Broken keeper plate for Abutment 5 of Grant Undercrossing (photograph by Mike Van de Pol).

Figure 181.—Mococo Off-Ramp in 1996.

Br #28-171 / Route 680 / Post Mile 24.46

Approximate Latitude & Longitude

N. Lat. 38° 01.5' W. Long. 122° 06.7'

Epicentral Distance

70 miles

Peak Ground Acceleration	N/S	U/D	E/W
Martinez VA	0.07	0.03	0.05

Length	Width	Skew	Year Built
717.7'	29.7'	0	1962

Main Span Type

Continuous concrete slab ramp (connected to welded steel girder)

Average Daily Traffic = 83,500

MOCOCO OVERHEAD

DESCRIPTION OF BRIDGE

This structure includes two parallel steel girder bridges on hammerhead pier walls that carry Route 680 traffic over railroad tracks and also a concrete slab bridge on a tight radius curve that acts as an off-ramp to take traffic onto Escabar Street in the city of Martinez (figs. 181 to 184). Only the off-ramp had some damage during the earthquake. That structure is supported at the bents by six reinforced concrete pile extensions embedded in a very weak soil (20 to 70 feet thick) composed of very soft clay and peat above layers of sandstone and siltstone. The off-ramp has a hinge in Span 5 and another at Span 11. There was no seismic retrofit.

BRIDGE DAMAGE

The off-ramp had damage to the hinges (fig. 188) and to some pile extensions (figs. 185 to 187) during the earthquake. The soft soil did not provide enough support for the pile extensions, and they moved enough to have flexural damage at the high moment locations at the bottom and the top of the piles. Most of the piles had some damage, but the most cracks were at Bents 4, 5, and 6, particularly for the exterior piles. The pile with the most damage was on the left side of Bent 4. The bents vary in height, with the shortest bents being closest to the abutment. The taller, more flexible piles had no cracking. However, they showed a permanent deformation in single and a few in double curvature.

The movement of the pile extensions also caused hinge damage. Hinges for reinforced concrete slab superstructures are made by modifying steel wide-flange girders and embedding them into the reinforced concrete slab (fig. 189).

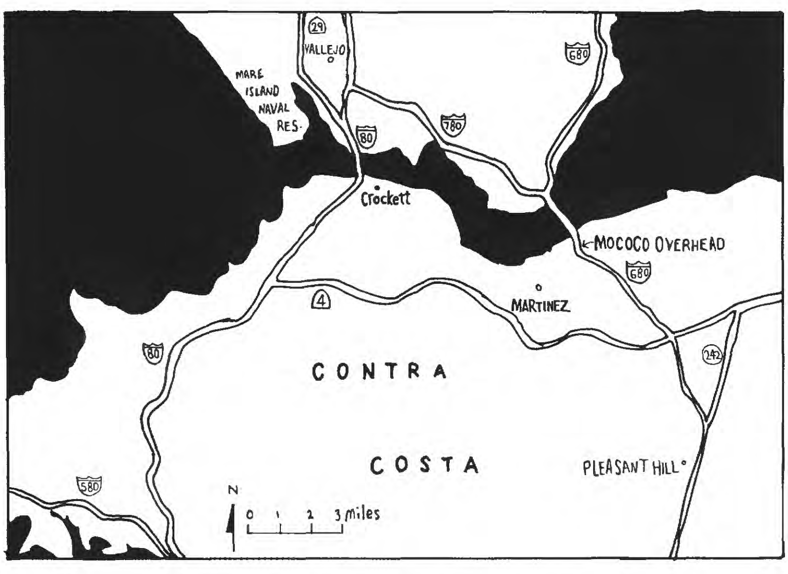


Figure 182.—Location of Mococo Off-Ramp.

Figure 183.—Location of connection between Mococo Off-Ramp and Mococo Overhead.

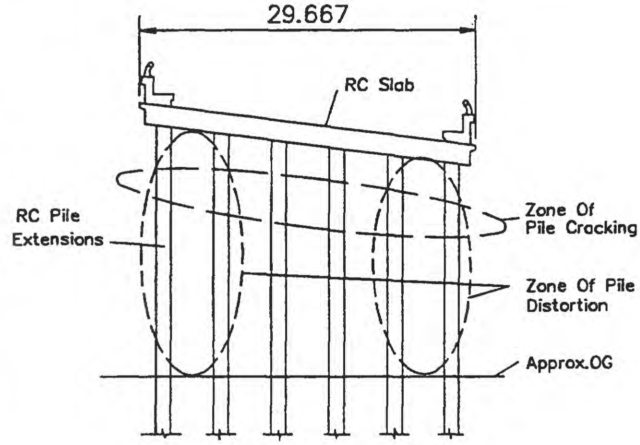


Figure 185.—Elevation drawing of damage to pile extensions of Mococo Off-Ramp.

Figure 186.—Damage to Mococo Off-Ramp concrete pile extensions at ground surface.

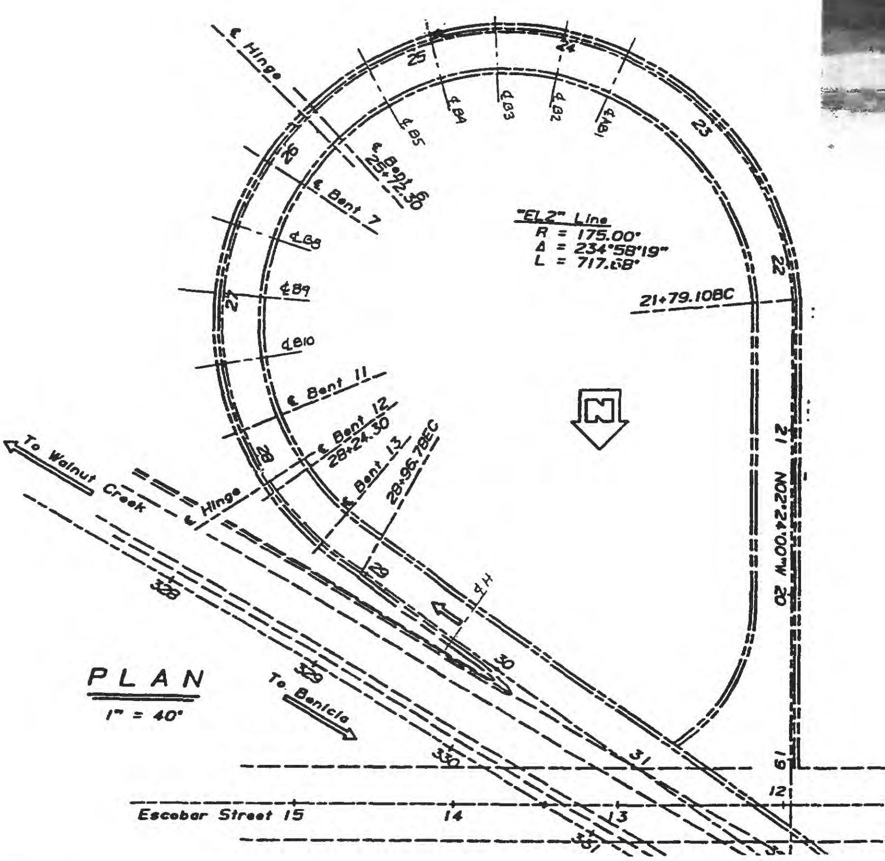


Figure 184.—Plan drawing of Mococo Off-Ramp.

Figure 187.—Cracking to top of pile extension of Mococo Off-Ramp.

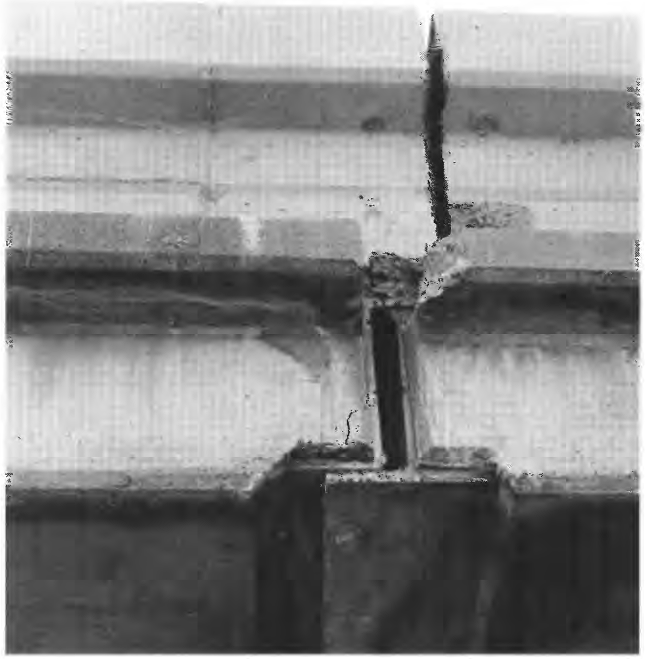


Figure 188.—Typical hinge damage to Mococo Off-Ramp.

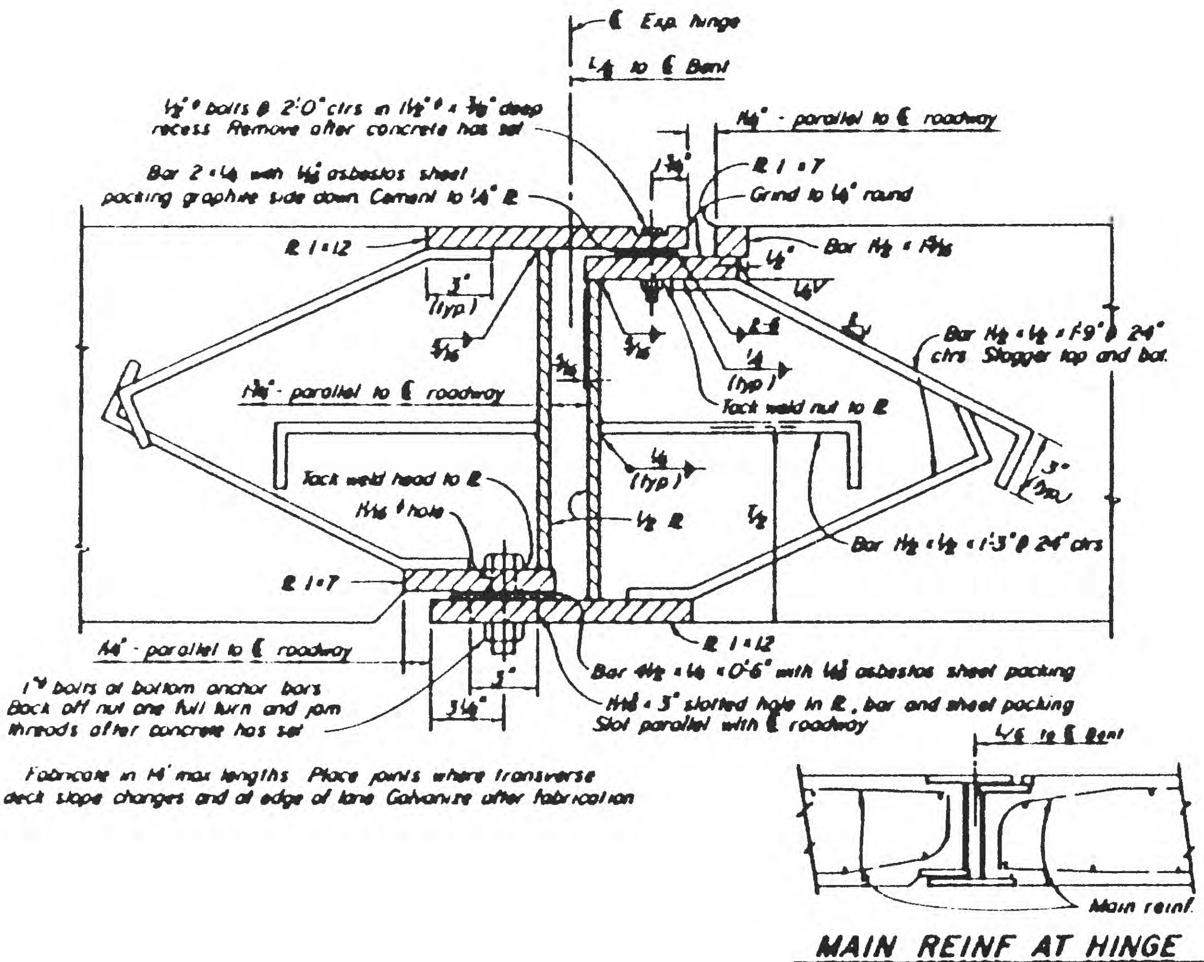


Figure 189.—Hinge detail for Mococo Off-Ramp.

They provide about 5 inches of seat width. They are located at one-sixth of the span length from a bent. Bolts go through slotted holes at the top and bottom of the hinge to allow for temperature movement while preventing unseating.

Hinge 1 by Bent 6 experienced a uniform longitudinal displacement of about 1 inch. About half of the bolt heads at the bottom of the hinge were missing, exposing fresh-looking metal on the bolt shaft. However, the bolt heads could not be found from searching the ground. No other damage was found to the hinge, and the ride over the hinge felt normal. At Hinge 2 by Bent 12 the gap was closed at the right (inside) edge of deck, and widened to about three-fourths inch on the outside edge of deck. There was some spalling on the outside barrier. Otherwise, the hinge appeared similar to Hinge 1, with some bolt heads missing but no other damage. At Hinge 3, which is at the connection between the steel girder and slab superstructures, there was a constant hinge opening and spalling on the inside concrete barrier.

REPAIR AND RETROFIT

Because of concerns about the structural integrity of this bridge, it was temporarily closed until October 20, when a Post Earthquake Investigation Team had time to inspect it. They concluded that the damage was light, that it did not threaten the ability of the structure to carry traffic, and the bridge was reopened. However, there was some concern as to how this structure would respond to a closer event. Emergency repairs included epoxy injecting the cracks in the pile extensions and the installation of restrainers at the hinges. This was accomplished by drilling holes through the deck for the bolts to hold the restrainer brackets (fig. 190). The main difficulty was in identifying and avoiding the reinforcement in the deck.

In 1991, a contract was written to design a retrofit for this structure (fig. 191). Surprisingly, only the mainline structure was retrofit.

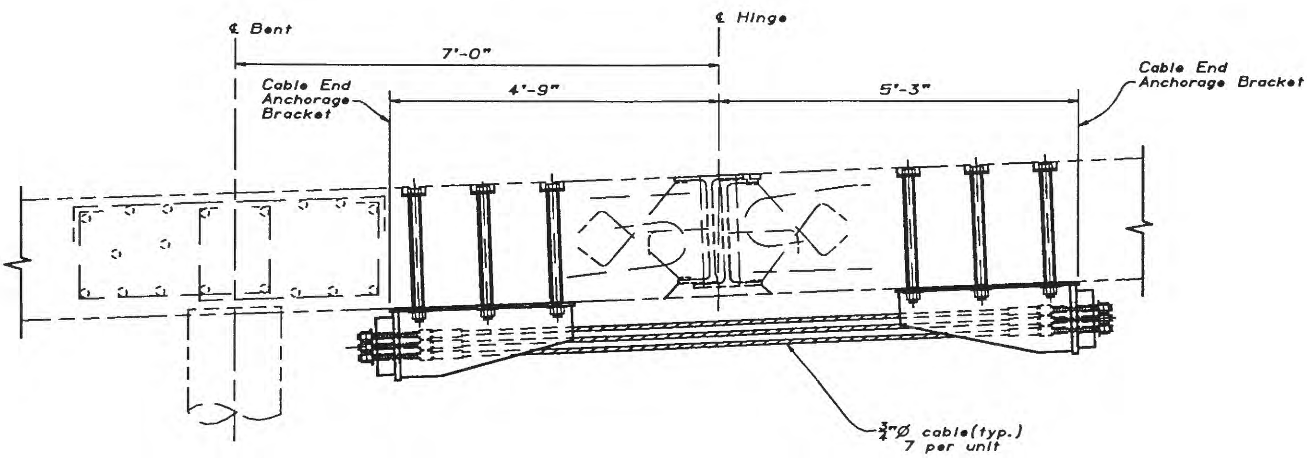


Figure 190.—Elevation drawing of emergency hinge restrainer retrofit of Mococo Off-Ramp.

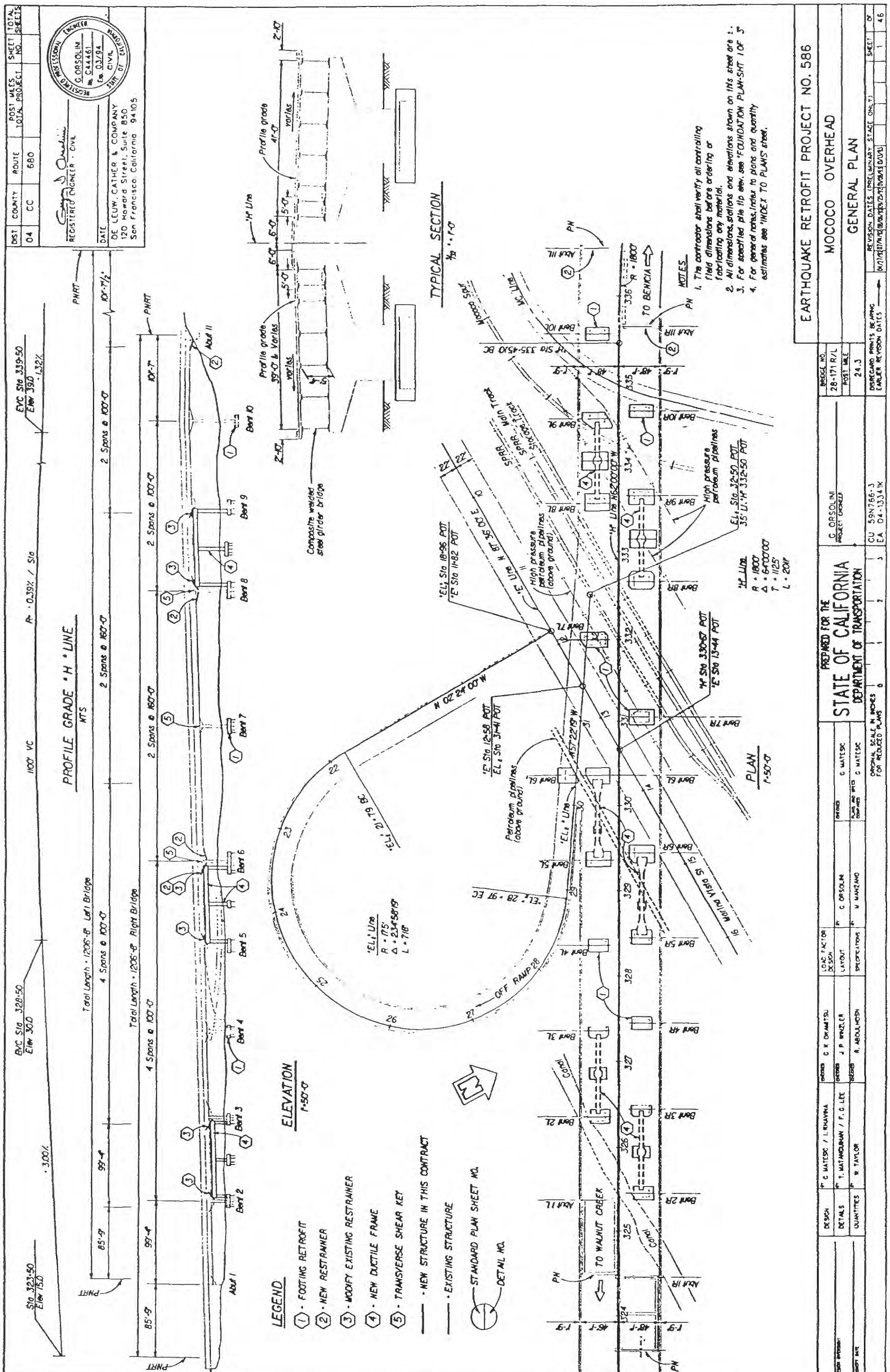


Figure 191.—General Plan drawing for seismic retrofit of Mococo Overhead.

Figure 192.—Napa River Bridge before earthquake.

Br #23-064 / Route 37 / Post Mile R7.39

Approximate Latitude & Longitude
N. Lat. 38° 07.2' W. Long. 122° 16.7'

Epicentral Distance
78.6 miles

Peak Ground Acceleration	N/S	U/D	E/W
Martinez VA	0.07	0.03	0.05

Length	Width	Skew	Year Built
3,280'	74.3	0	1962

Main Span Type
Prestressed precast concrete 'I' girders

Average Daily Traffic = 29,300

NAPA RIVER BRIDGE

DESCRIPTION OF BRIDGE

This long bridge climbs to over 120 feet as it crosses the Napa River near the city of Vallejo (figs. 192 to 195). It has a precast prestressed "I" girder superstructure with a concrete deck, except for a 188-foot steel girder span over the ship channel. Concrete diaphragms were cast at every pier to tie the ends of the precast girders together. At certain locations, two diaphragms were cast to provide expansion joints for temperature movement. These special joints are at the two abutments and at Piers 5, 9, 13, 17, and 23.

Most of the piers are flexible reinforced concrete two-legged bents, except for the stiffer four-legged towers at Piers 2, 7, 11, 16, and 20. There are 25 spans. They are typically 120 feet long, except for the span over the ship channel and the 30-foot spans over the towers. The superstructure rests on bearing pads at the fixed locations and steel rocker bearings at the expansion joints. The substructure is supported on pile caps with either 54-inch-diameter hollow precast prestressed piles or 24-inch-square precast prestressed piles. These piles are supported by layers of very soft to very stiff Bay mud (fig. 196). The poor material is about 100 feet deep at the west end of the structure and narrows to less than 20 feet at the east end. There was a seismic retrofit in 1984 that included cable restrainers at the expansion joints and transverse shear keys on top of the bent caps between all the girders.

BRIDGE DAMAGE

This structure has many characteristics that make it sensitive to long-period ground motion. At 80 miles away, the motions for this bridge were extremely small but extremely long period. This is similar to what occurred during the 1985 Mexico City earthquake, where damage occurred on very soft soils almost 200 miles from the rupture. Soft soil is very responsive to very slow motion as is demonstrated by the fact that almost all the serious bridge damage that occurred during this earthquake was many miles away on weak cohesive soil sites. Also, we have seen that bridges with a history of maintenance problems perform particularly poorly during earthquakes. Some bridges begin to spall concrete at the joints and start to damage bearings and expansion joint devices al-

Figure 193.—Napa River Bridge before earthquake.

most as soon as they are built. That was the case with several of the bridges we have studied, including the Napa River Bridge.

The first investigation of damage was done by a Post Earthquake Investigation Team; however, it is difficult to effectively examine a tall bridge over water without a catwalk or snooper truck. A snooper truck became available on October 30, 1989, and was used on this bridge for 2 days and then again on December 9 (fig. 197).

The basis for much of the damage can be seen in figure 198. Almost all the damage was the result of the precast "I" girders pulling out of the diaphragms (figs. 199 to 202). They were tied together with #6 reinforcement that ran transversely through the diaphragms. During the earthquake the girders pulled out, delaminating the concrete behind the #6 reinforcement.

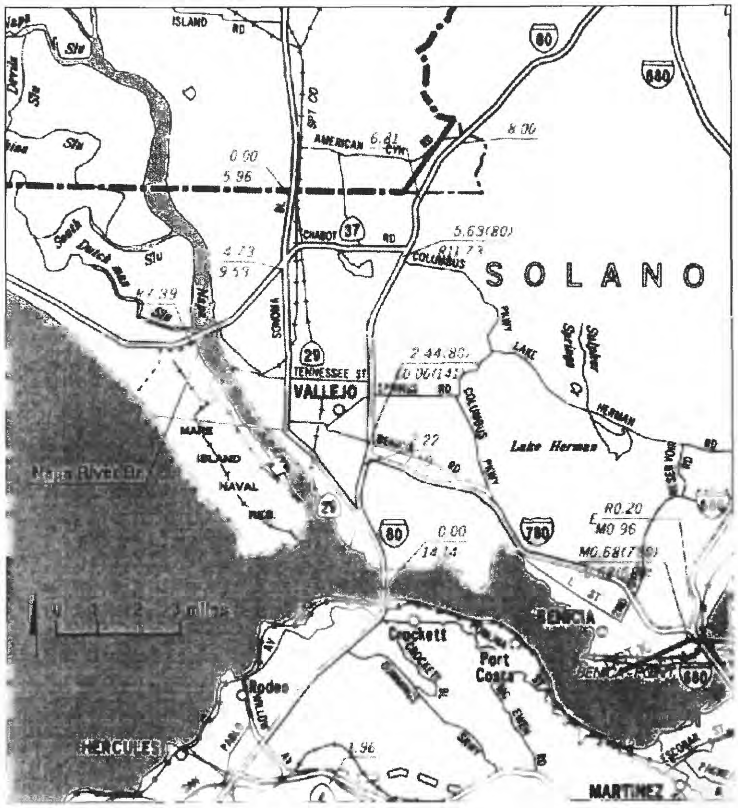


Figure 194. —Location of Napa River Bridge.

DIAPHRAGM DAMAGE

There were small to large spalls in the concrete diaphragms between the girders at the following locations. Pier 9 (Span 8 side), Pier 10 (both sides), Tower 11a (Span 10), Pier 12 (both sides), Pier 15 (both sides), Tower 16 (both sides), Pier 18 (both sides), Pier 19 (both sides), Tower 20 (both sides), Pier 21 and 22 (on the span 21 side). The diaphragms for Pier 12 at Span 11 and Pier 15 on both sides had major spalls and shear key failures.

Figure 195.—Two-and-four-legged piers on Napa River Bridge.

GIRDER DAMAGE

The girders at Piers 12 and 15 have pulled out of their diaphragms. Pullout at Pier 12 (Span 11) side was about 1 inch toward Pier 11. Pullout at Pier 11 at the Span 12 side was less severe. At Pier 15 at the Span 14 side the longitudinal displacement of the girders was at least 3.5 inches. Five inches of the girder ends sheared off along the horizontal holes for the continuous reinforcement in the diaphragm. Hence, the bearing area of the girder was reduced by 5 inches for girder loss and by 3.5 inches due to movement of the girder toward Pier 14 for a total bearing loss of 8.5 inches at the Pier 15 cap. The girder still had about 12 inches of longitudinal seat on the concrete pad.

OTHER DAMAGE

The joint at Pier 15 was designed as fixed for translation and free for rotation. After the earthquake it was free to do both. The joint width was 4.5 inches at the deck. The hinge in Span 13 near Pier 14 has opened an additional 0.325 inch. The hinge in Span 13 near Pier 13 has closed an additional 3.5 inches. The joint seal in the hinge in Span 13 near Pier 14 has failed. Pier 14 has deflected about 4 inches toward Pier 13, causing the restrainer cables at Pier 13 to hang loosely. There was flutter at the holes for the restrainers at Pier 14 (the restrainers are double wrapped through the holes in the bent caps as shown in fig. 203), and the restrainers were probably damaged. However, they held the superstructure on the bents.

There were some 0.0625-inch-wide cracks on the struts that separate the columns at Pier 9, Tower 11 (Span 10 side), Tower 16 (both sides), and Pier 17. At Pier 9 the cap has a shear crack. The restrainers and shear keys were damaged at Pier 9 and Pier 17. At Pier 17 the damage was extensive and required replacement. There was some edge spalling of the concrete bearing pads on the Span 11 side of Pier 12 under Girders 3, 4, and 6. The Pier 17 expansion joint was opened 7.125 inches between the steel angles of the finger joints. Four inches was the normal construction plan opening. The joint is in good condition.

BRIDGE REPAIRS

Although this bridge was not damaged enough to require closing after the earthquake, the repairs were very expensive. The use of a snooper and traffic control to inspect the bridge cost about $30,000 for 5 days. The cost of replacing and repairing cable restrainers was over $100,000. Epoxy injection and replacing diaphragms was over $200,000. A retrofit for this structure was done by Caltrans. Because of the weak soil, it was feared that new foundations would be required; however, some of the original piles were located near the structure and tested. It was found that the existing foundation would be adequate to support the structure for the maximum credible event.

Figure 197.—Snooper on top of Napa River Bridge.

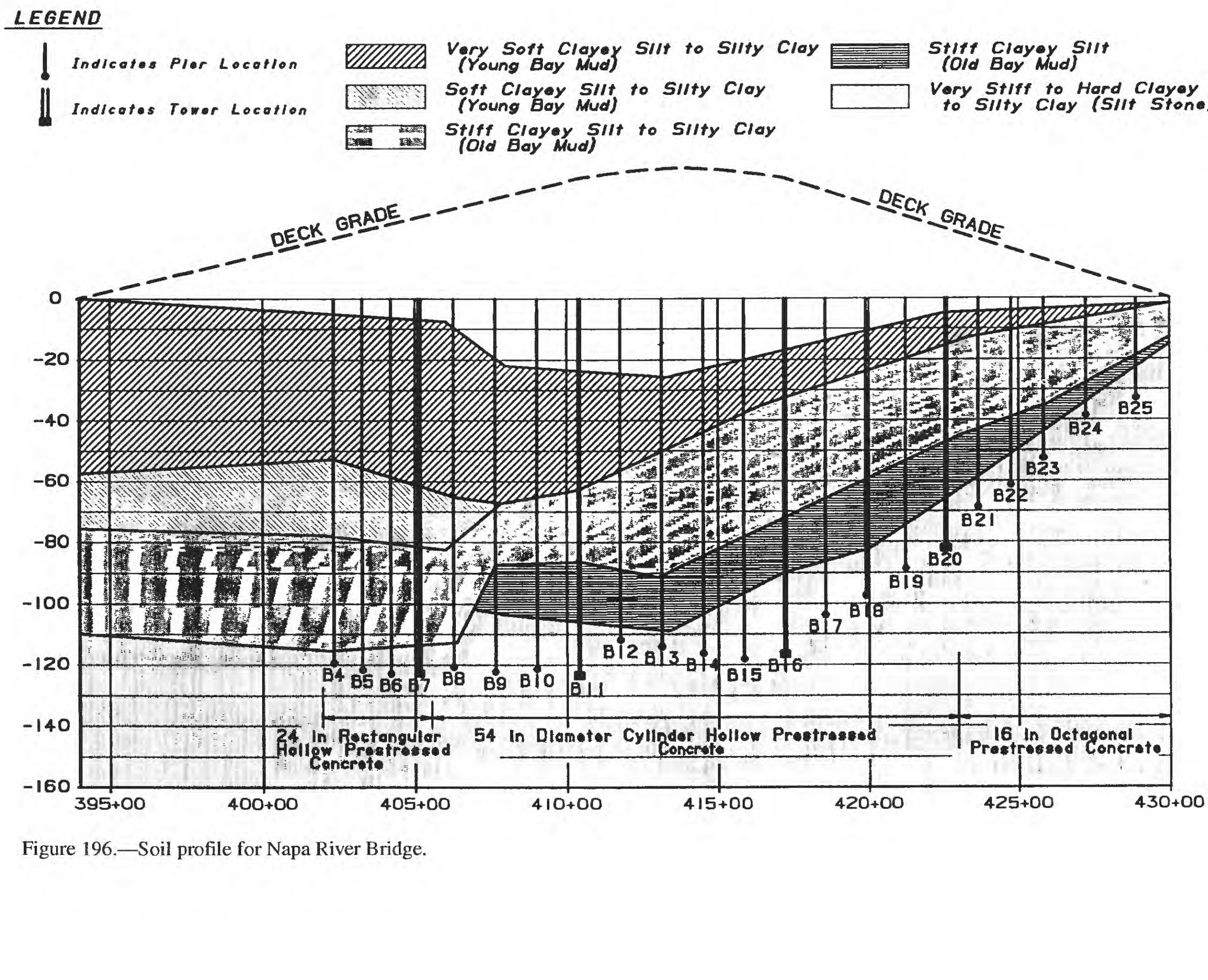


Figure 196.—Soil profile for Napa River Bridge.

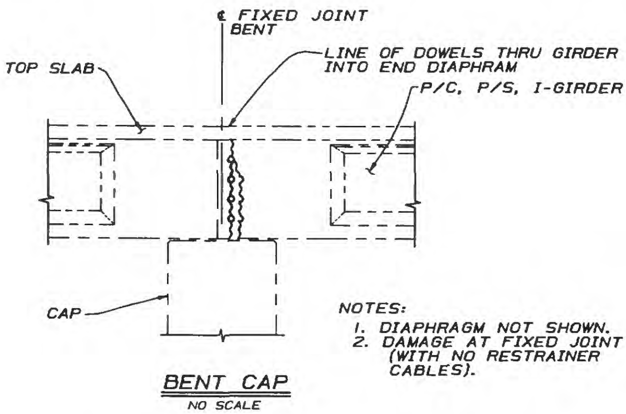


Figure 198.—Detail drawing of typical damage to Napa River Bridge.

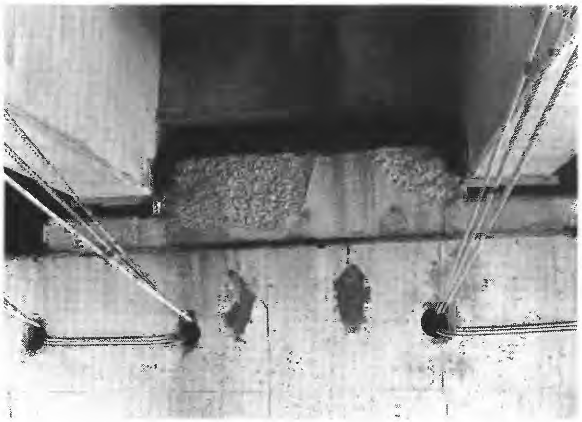


Figure 201.—Spalling on the Span 16 side of Pier 17 at Napa River Bridge.

Figure 199.—Diaphragm damage to Napa River Bridge.

Figure 202.—Spalled diaphragm at Bay 3 of Span 14 side of Pier 15 at Napa River Bridge.

Figure 200.—Spalling at diaphragm of Napa River Bridge.

Figure 203.—Bent cap damage to Pier 17 of Napa River Bridge.

Figure 204.—Richardson Bay Bridge and Separation before earthquake.

Br #27-010 / Route 101 / Post Mile 4.03

Approximate Latitude & Longitude
N. Lat. 37° 53.0' W. Long. 122° 31.0'

Epicentral Distance
68.7 miles

Peak Ground Acceleration	N/S	U/D	E/W
Point Bonita	0.07	0.03	0.07

Length	Width	Skew	Year Built
2,864'	138'	Varies	1957

Main Span Type
Prestressed precast concrete 'I'girders

Average Daily Traffic = 141,000

RICHARDSON BAY BRIDGE AND SEPARATION

DESCRIPTION OF BRIDGE

This bridge is similar to the Napa River Bridge in its components and in its performance during the earthquake (figs. 204 to 207). The superstructure has precast prestressed "I" girders for half of its length, with cast-in-place box girders for the rest of the structure. The superstructure is 138 feet wide, 4 feet 7 inches deep, and has 44 spans, with a total length of 2,864 feet. There is also a 597-foot off-ramp structure. The superstructure is supported by ten 36-inch-diameter column bents, by four towers, and by two seat-type abutments. All bents are on pile footings embedded in Bay mud (fig. 208). The bridge was built in 1957 and widened and retrofitted with cable restrainers in 1973.

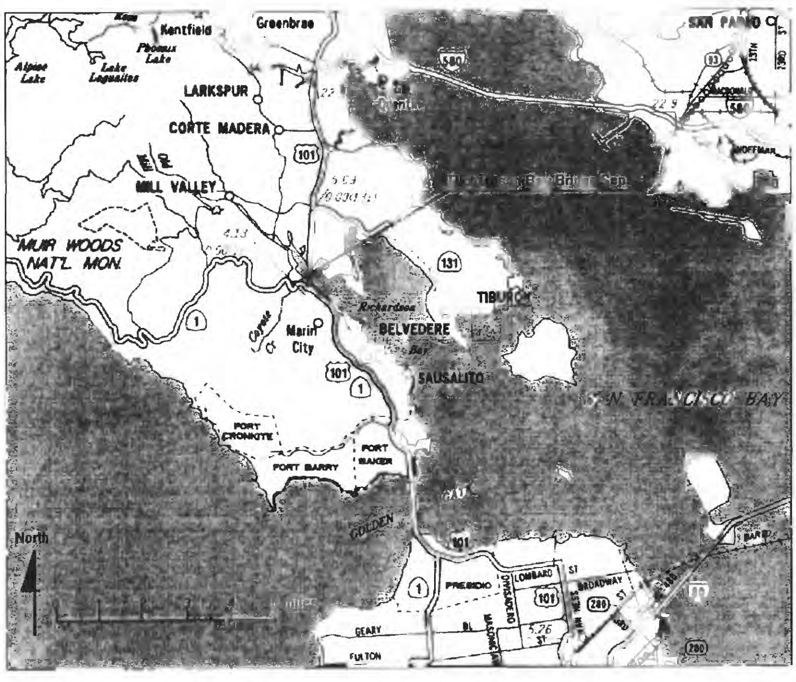


Figure 205.—Location of Richardson Bay Bridge and Separation.

BRIDGE DAMAGE

Like the Napa River Bridge, a thorough inspection of the Richardson Bay Bridge and Separation had to wait for the snooper to arrive on the structure. This bridge had moderate damage as a result of the earthquake. The deck finger expansion joints on the right side of the bridge at Bent 26 and Bent 36 failed. The deck concrete sheared at the connection with the expansion joints. The segmented rockers at Bent 26 and Bent 36 had various tilts, few were normal, and most were near their maximum rotation. There were many spalls and cracks in the diaphragms similar to

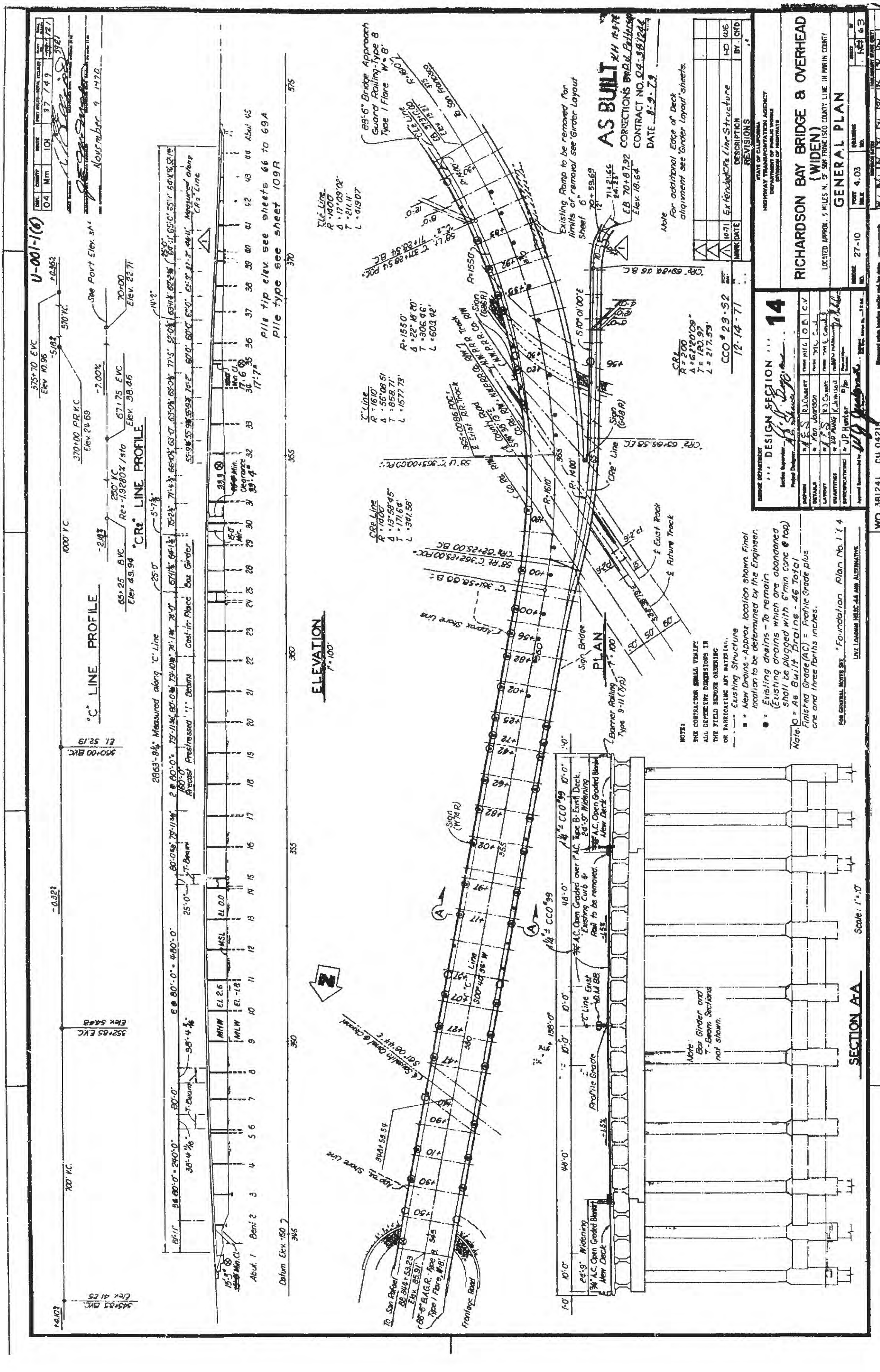


Figure 206.—General Plan drawing for Richardson Bay Bridge widening.

Figure 207.—Bents at box girder section of Richardson Bay Bridge.

the Napa River Bridge, owing to girder pull out, deflections, bent racking, and from large longitudinal forces from the cable restrainers. Many of the restrainers were damaged or stretched taut.

Some of the shear keys were damaged and spalled the vertical cap face. At Bent 31 about 50 feet of the key was sheared, at Bent 32 about 70 feet was sheared, and at Bent 35 about 28 feet was damaged.

There was quite a bit of preexisting damage that may have been increased by the earthquake. Many of the columns had minor spalls where they join the bottom strut above the footing. However, no rebar was exposed. The right exterior girder in Span 30 at Bent 31 had a spall that reduced its bearing area. The hinge in Span 14 had about a 5-inch opening on the left side.

REPAIR AND RETROFIT

This bridge had both emergency repairs and a later retrofit. Table 20 summarizes the repairs made after the earthquake. A retrofit was designed for this bridge in 1993 (figs. 209 to 211). The strategy was to take all the load at the towers by installing stiff pier walls. Although the superstructure could shear off during a large earthquake, it would effectively isolate the bridge. Rubber fenders would catch the superstructure after about a foot of movement.

Table 20.—*Summary of earthquake repairs to Richardson Bay Bridge and Separation*

Location	Repair	Cost
Bent 26 and 36	Replace expansion joint devices.	$70,000
Bent 26 and 36	Plumb or replace rockers.	$80,000
Bent 26 and 36	Replace restrainers.	$30,000
Bent 31, 32, 35, and Hinge 14	Install new restrainers	$30,000
Bent 24, 26, 29, 31, 32, and 38	Repair diaphragms	$12,000
Bent 26, 31, 32, and 36	Replace diaphragms	$42,000
Bent 23, 31, 32, and 35	Repair cap spalls	$37,000
Bent 20 to 23, 26, and 35	Epoxy inject column cracks	$20,000
Bent 26 and 27	Epoxy inject strut cracks	$18,000
Bent 22 to 28, and 43	Repair column spalls	$71,000
	TOTAL COST =	**$450,000**

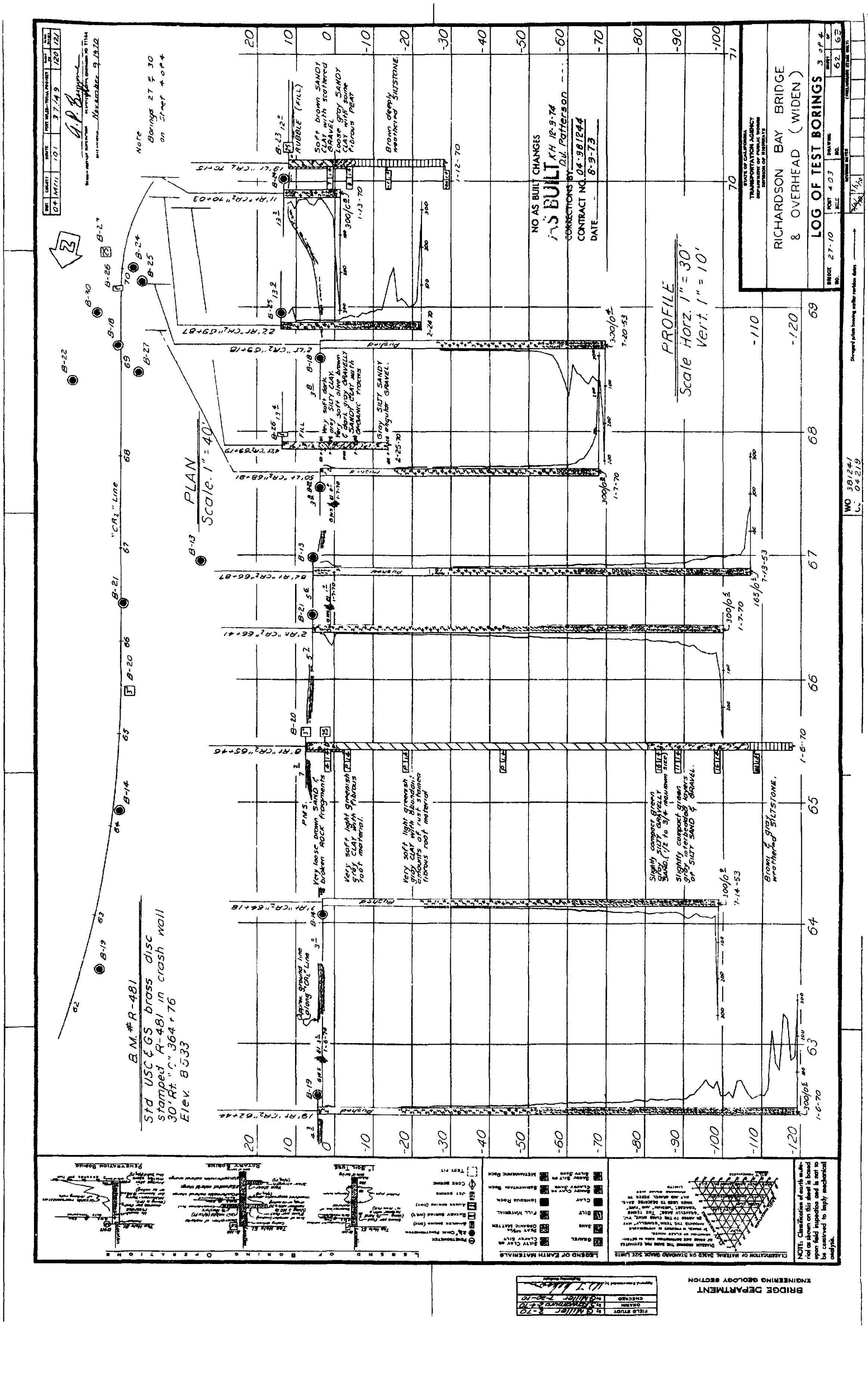


Figure 208.—Log of borings at Richardson Bay Bridge.

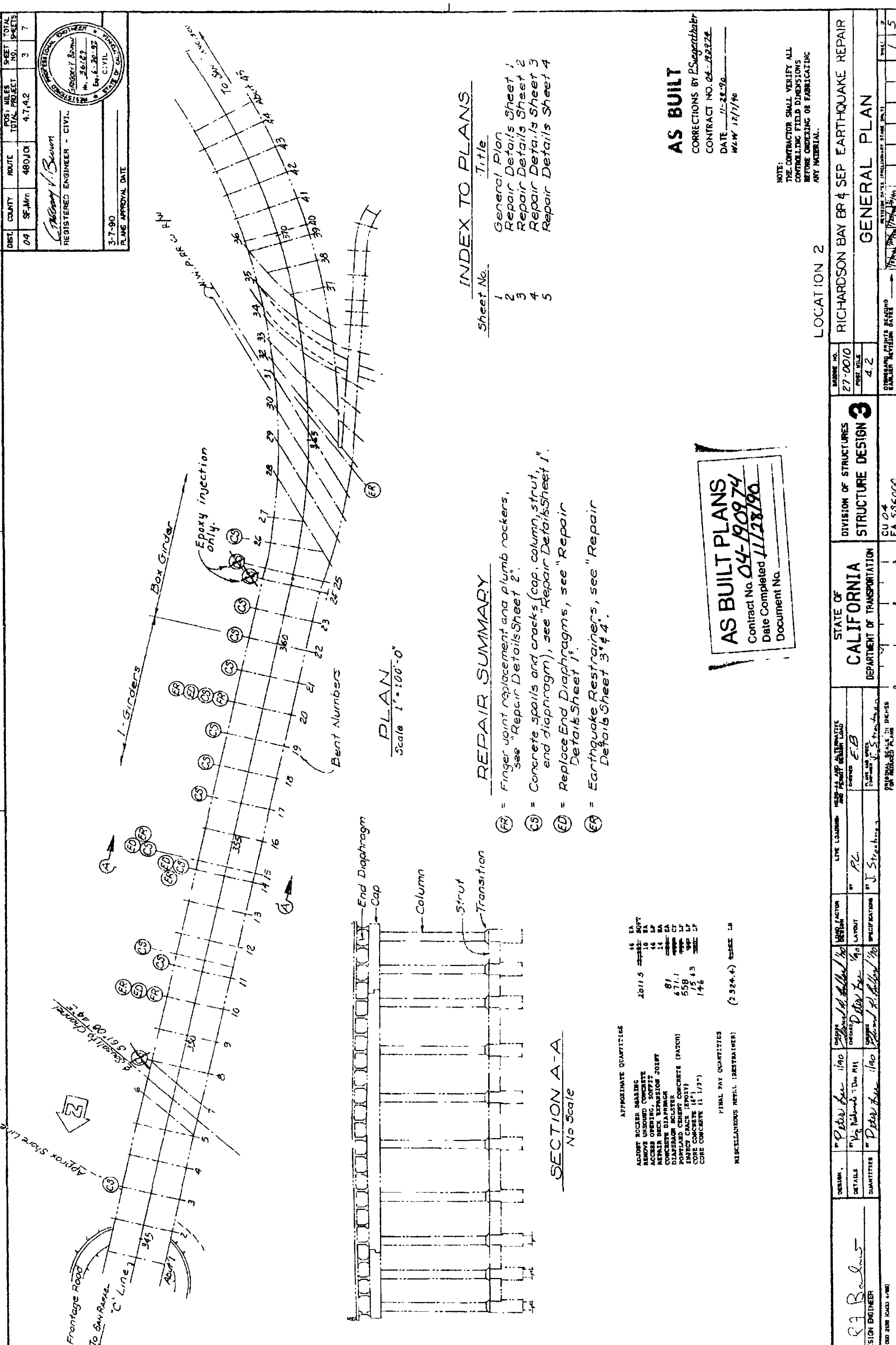


Figure 209.—General Plan drawing of repair for Richardson Bay Bridge and Separation.

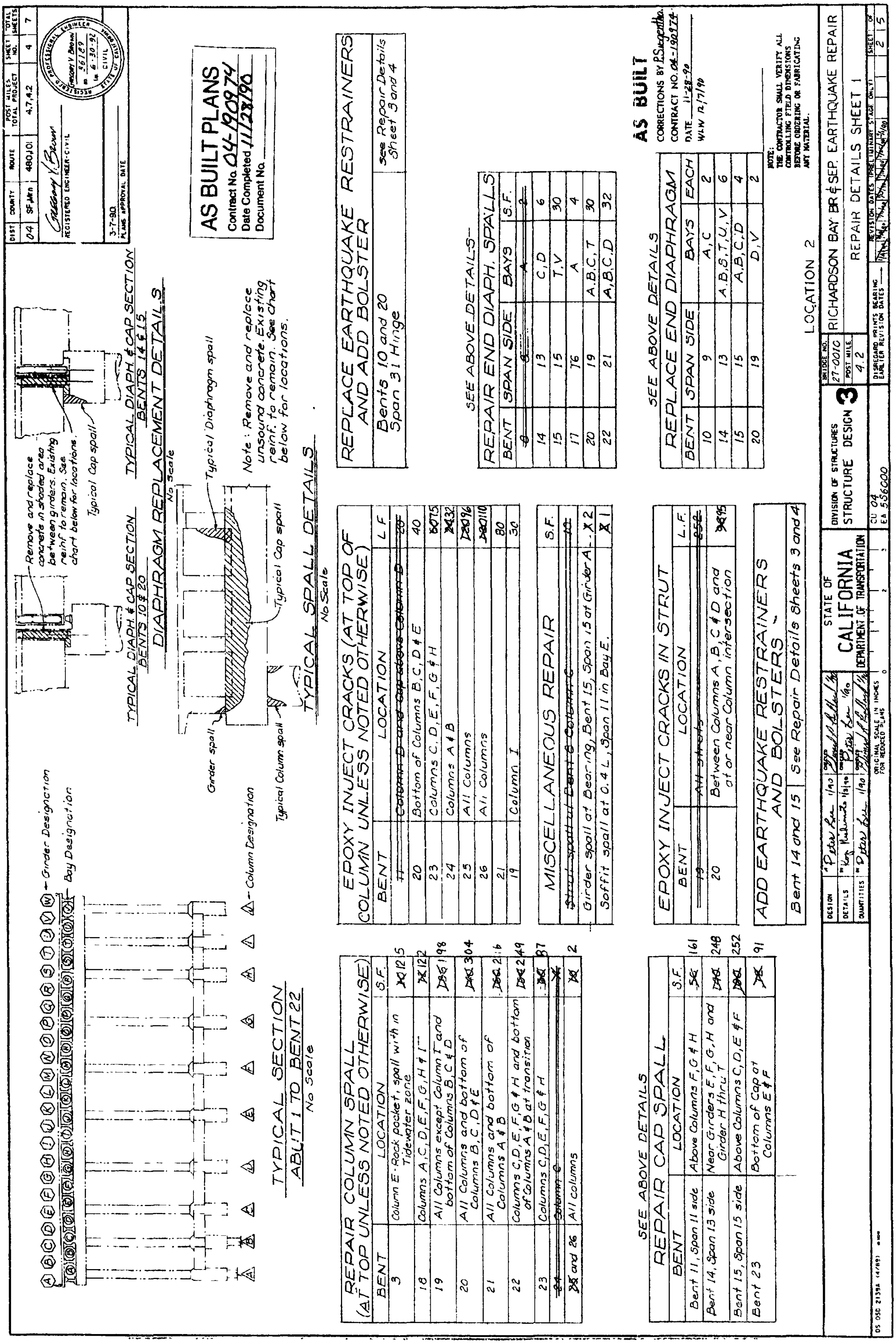


Figure 210.—Earthquake repair drawing for Richardson Bay Bridge and Separation.

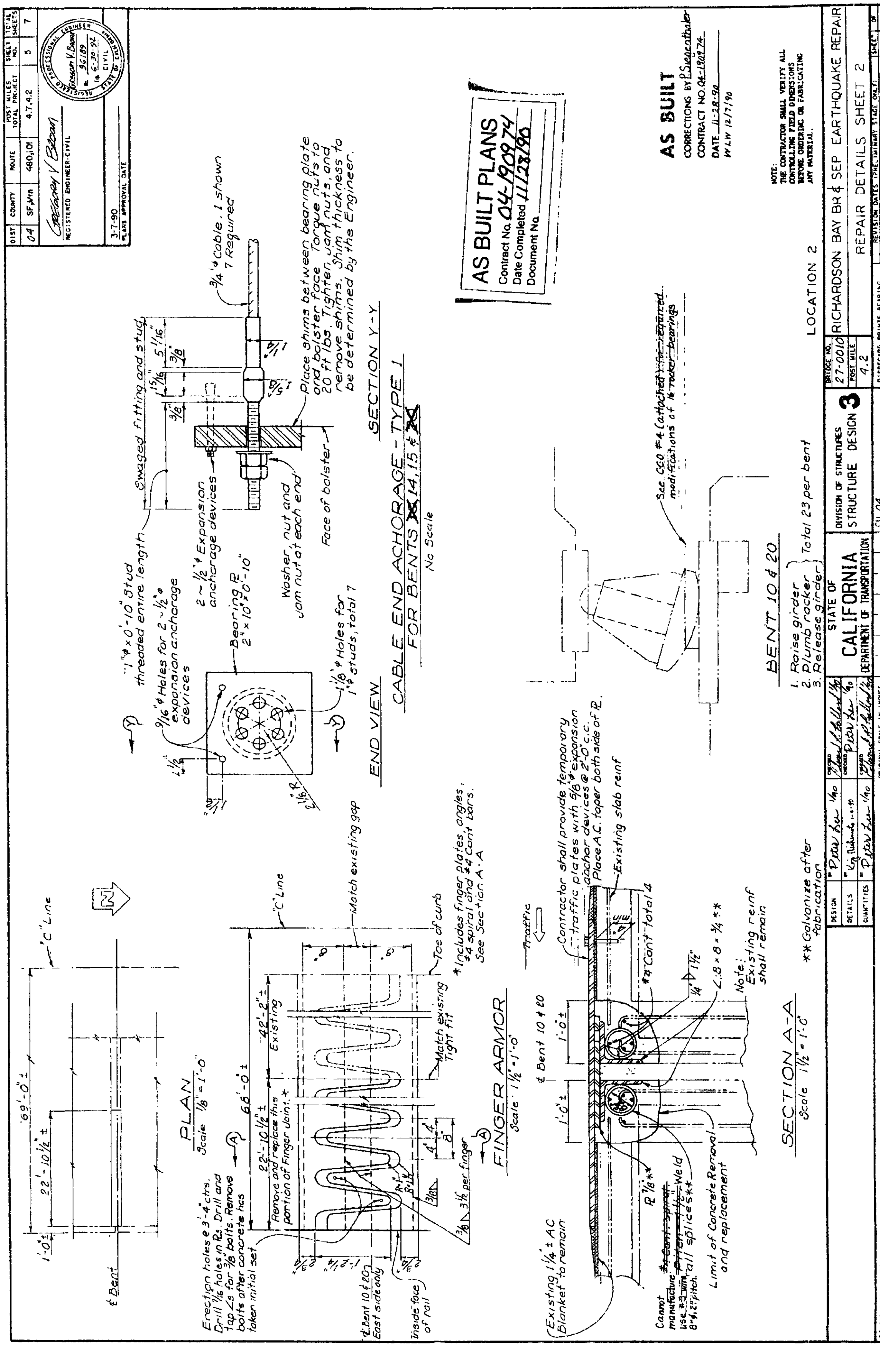


Figure 211.—Earthquake repair drawing for Richardson Bay Bridge and Separation.

Figure 212.—San Mateo-Hayward Bridge.

Br #35-054 / Route 92 / Post Mile R14.44

Approximate Latitude & Longitude

N. Lat. 37° 36.0' W. Long. 122° 12.8'

Epicentral Distance

43.2 miles

Peak Ground Acceleration	N/S	U/D	E/W
Foster City on west (alluvium)	.29	.11	.26
Hayward on east (Franciscan)	.10	.05	.12

Length	Width	Skew	Year Built
36,069'	59.2'	none	1967

Main Span Type

Steel box girder with a concrete girder approach

Average Daily Traffic = 72,000

SAN MATEO-HAYWARD BRIDGE

DESCRIPTION OF BRIDGE

The San Mateo-Hayward Bridge is one of several San Francisco Bay crossings. This structure is about 7 miles long and takes traffic westward from Hayward to Foster City, just south of San Francisco International Airport (figs. 212 and 213). The bridge is just east of the Route 92/101 Separation, which also had some damage described in this chapter. Figure 214 shows the geology of the area including the San Andreas fault, which is 6.9 miles west of the bridge, and the Hayward fault, which is 5 miles east of the bridge. This bridge has 5 miles of concrete trestle on the east end (fig. 215) and almost 2 miles of a double steel box girders on the west (figs. 216 and 225).

The concrete portion is a precast "T" girder superstructure, 54 feet wide, and with 857 simple spans. Each span ends in a corbel supported by a dropped bent cap on prestressed concrete piles and the next span sits on the corbel of the previous span. The piles extend through about 5 feet of water and then through 20 to 50 of Bay mud into stiffer material.

The steel portion is an 80-foot-wide superstructure of alternating anchor and suspension units. The anchor units sit on pin bearings, and the suspension units hang from alternating tied and expansion hinges (fig. 217). The shorter 208-foot spans are supported by concrete two-legged piers on piles, while the longer 292-foot spans and the ship channel span are on two-legged steel towers (figs. 218 and 219). The piles for the steel portion are steel "H" piles with tip elevations varying from 40 feet to 220 feet below sea level. The footings are rectangular for the shorter spans and bell shaped for the longer spans. The bridge ends on the west side with a 220-foot-long, four span concrete "T" beam superstructure on two column bents. It was designed and built in the 1960's and was opened to traffic in 1967.

BRIDGE DAMAGE

There was some minor damage to the bridge during the earthquake. However, on such an enormous structure even minor damage requires a lot of work to examine and repair. As was the case for all these tall water crossings, much of the inspection had to wait until a snooper was available. Traffic control and rental of the snooper was expensive. Moreover, divers were required to inspect the thousands of foundations and pile connections. The damage included:

Bearings at Piers 18 and 21.—Bearings supporting the steel box girders had moved 1.5 inches at the north bearing of Pier 18 and 4.5 inches at the south bearing of Pier 21. Six of the eight interior pin caps (1.75-inch screw cap) had broken off. The end keeper plates were bent (figs. 220 to 223).

Towers at Piers 16 and 18.—The towers slid on the base plates. At Pier 16 the north column slid 2 inches to the south and a half inch to the west. At Pier 18 the south column slid 2 inches to the north and a half inch to the west. However, this damage was not serious because the tower legs are connected to the base plates by very long bolts. During an earthquake, these bolts can move around without breaking, providing some isolation to the towers.

Span 19 Tied Hinges.—The south box girder tie bar located inside the south web was broken loose from the 2-inch-diameter pins. The south box girder pins at the north web were deformed.

Pier 38.—At the backwall for Pier 38 the trestle earthquake restrainer shelf bearings supporting the first trestle span were damaged. The painters stairway giving access to the walkway behind Pier 38 has fallen into the bay (fig. 224).

Concrete Trestle.—There were some spalls and cracks in the socket between the piles and pile caps.

Traveler Rail.—The painter's walkway under Span 15 had some cracking, deformation, and connection damage.

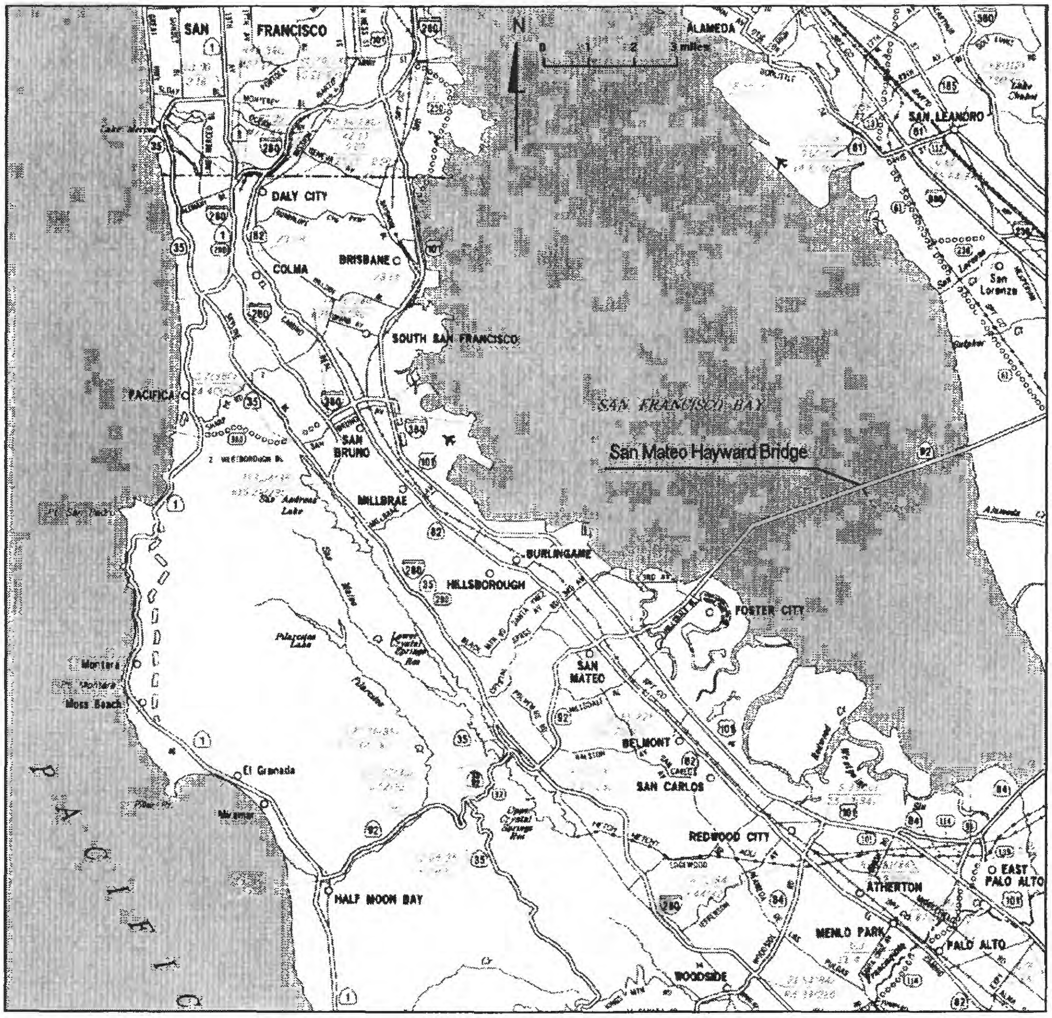


Figure 213.—Location of San Mateo-Hayward Bridge.

BRIDGE REPAIR AND RETROFIT

Immediately after the earthquake, there was some concern about the condition of this structure. The bridge was temporarily closed while Structural Maintenance personnel drove across the bridge looking for misalignments. They used a motorboat and a flashlight to inspect the concrete trestle. Then traffic was allowed to cross the structure. The cost of jacking the towers back into place and repairing spalls to the concrete trestle totaled over $1 million.

Because this bridge was built before a great deal of consideration was given to seismic loads, it is highly vulnerable to a large earthquake from the San Andreas or Hayward faults. Since the earthquake, all of the San Francisco Bay toll bridges have been studied to determine the maximum and 400-year return period earthquake demands and to identify the vulnerabilities of the structures. In 1995, consultants were hired to design seismic retrofits for these structures. Although a retrofit design has not been completed for the San Mateo-Hayward Bridge the vulnerability analysis recommended increasing the seat width for the trestle portion and providing more piles and more ductility to the piers of the steel portion.

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White, Richard W., 1990, Supplementary bridge report: Caltrans internal document.

Donikian, Roupen R., 1993, Seismic vulnerability assessment of the San Mateo-Hayward Bridge: The Cygna Group.

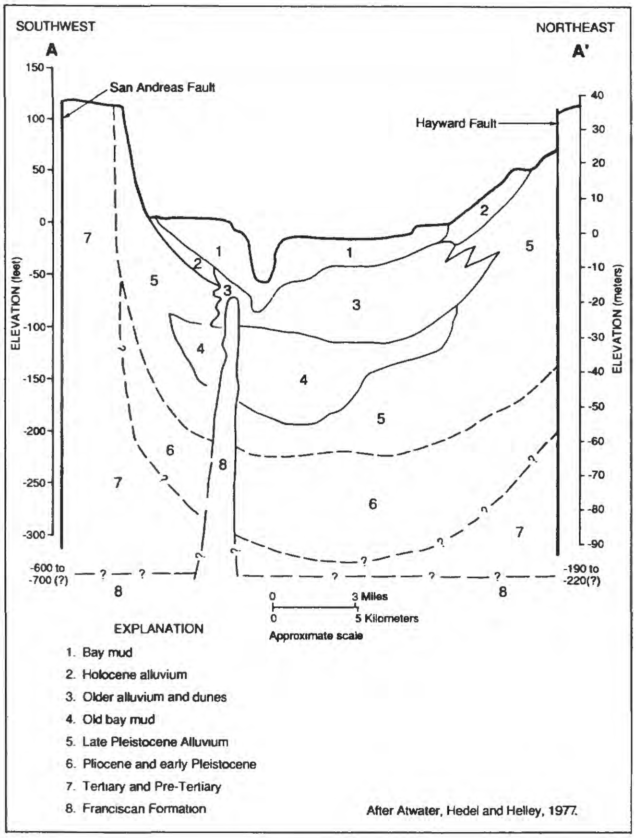
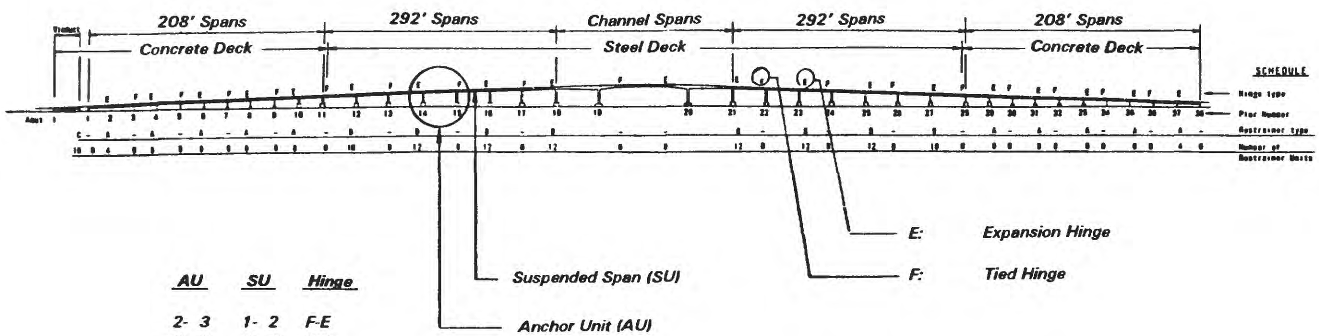


Figure 214.—Soil profile for San Francisco Bay near San Mateo-Hayward Bridge (courtesy of Geomatrix Consultants).

Figure 215.—Concrete trestle portion of San Mateo-Hayward Bridge.



AU	SU	Hinge
2- 3	1- 2	F-E
4- 5	3- 4	F-E
6- 7	5- 6	F-E
8- 9	7- 8	F-E
10-11	9-10	F-E
12-13	11-12	F-E
14-15	13-14	F-E
16-17	15-16	F-E
18-19	17-18	F-E
20-21	19-20	F-E
22-23	21-22	E-F
24-25	23-24	E-F
26-27	25-26	E-F
28-29	27-28	E-F
30-31	29-30	E-F
32-33	31-32	E-F
34-35	33-34	E-F
36-37	35-36	E-F
--	37-38	E-F

Figure 216.—Elevation drawing of the steel portion of the San Mateo-Hayward Bridge (courtesy of the Cygna Group).

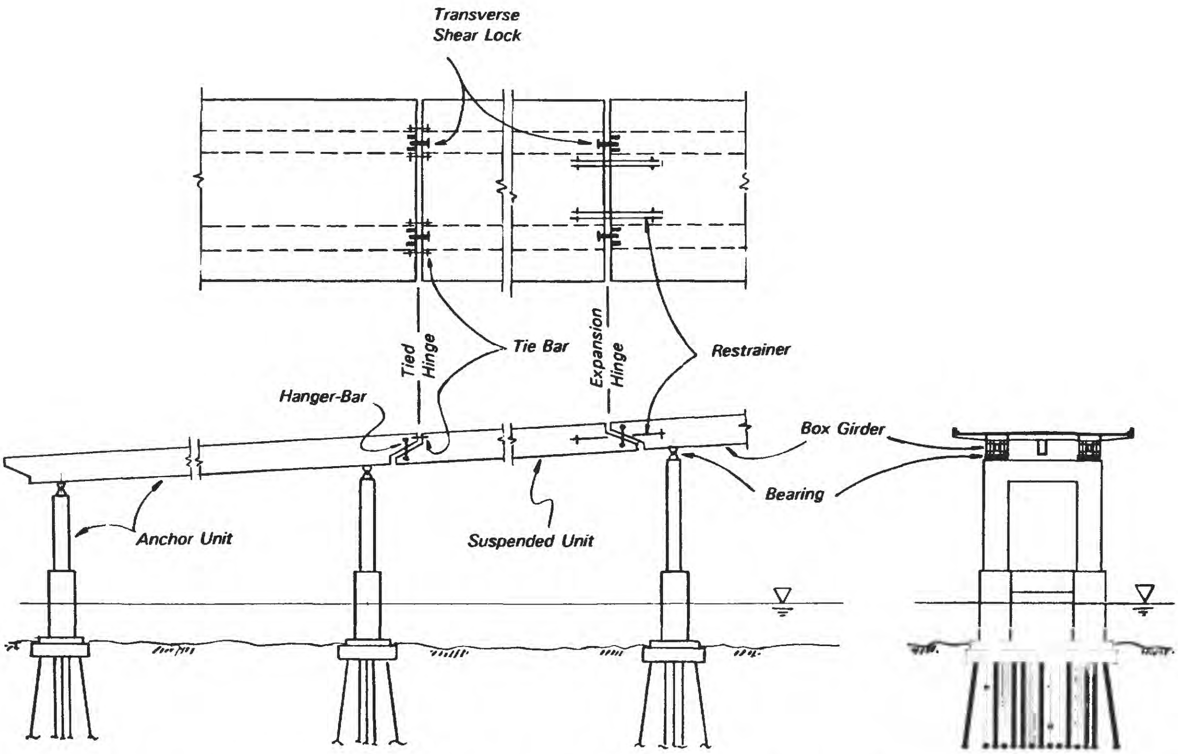


Figure 217.—Steel bridge structural system at San Mateo-Hayward Bridge (courtesy of the Cygna Group).

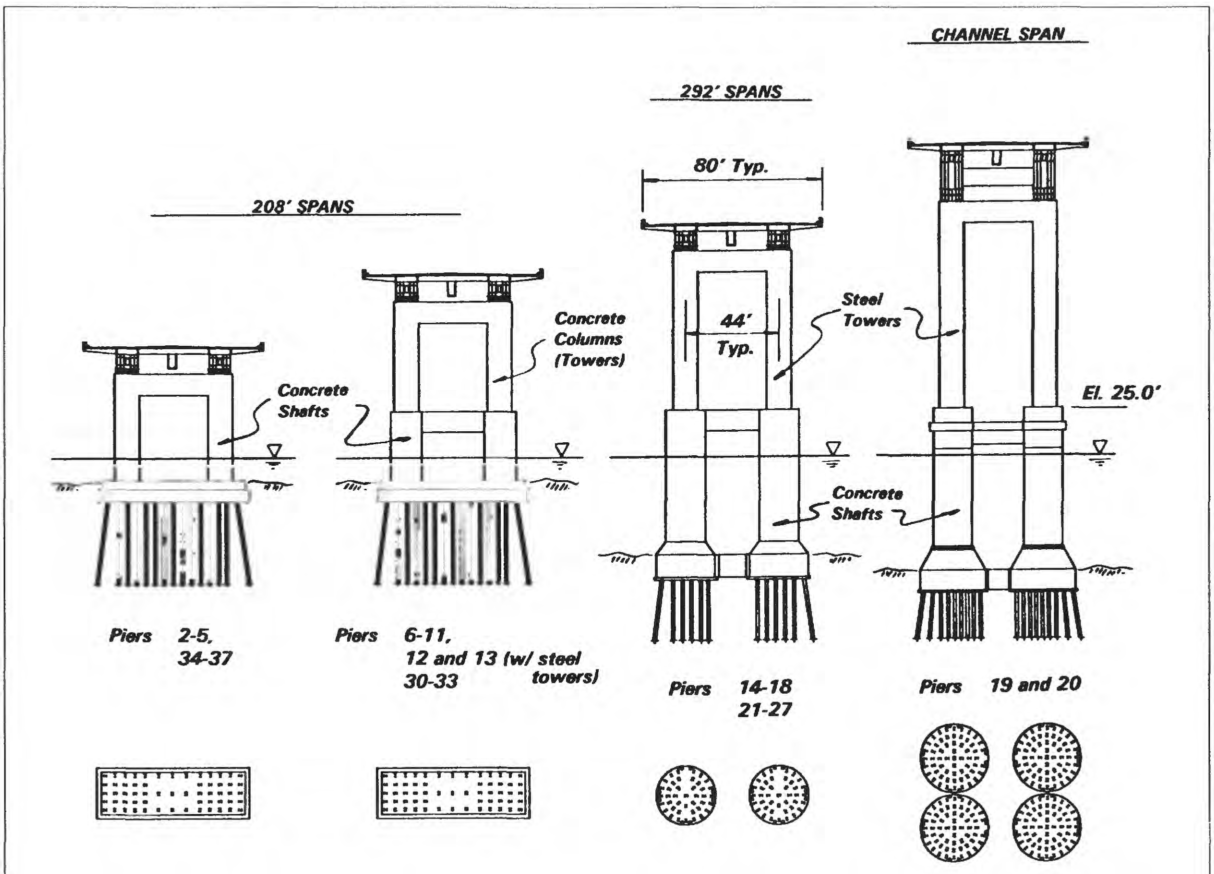


Figure 218.—Piers for steel superstructure at San Mateo-Hayward Bridge (courtesy of the Cygna Group).

Figure 219.—Piers supporting the steel superstructure of San Mateo-Hayward Bridge.

Figure 220.—Repair to damage at Pier 21 of San Mateo-Hayward Bridge.

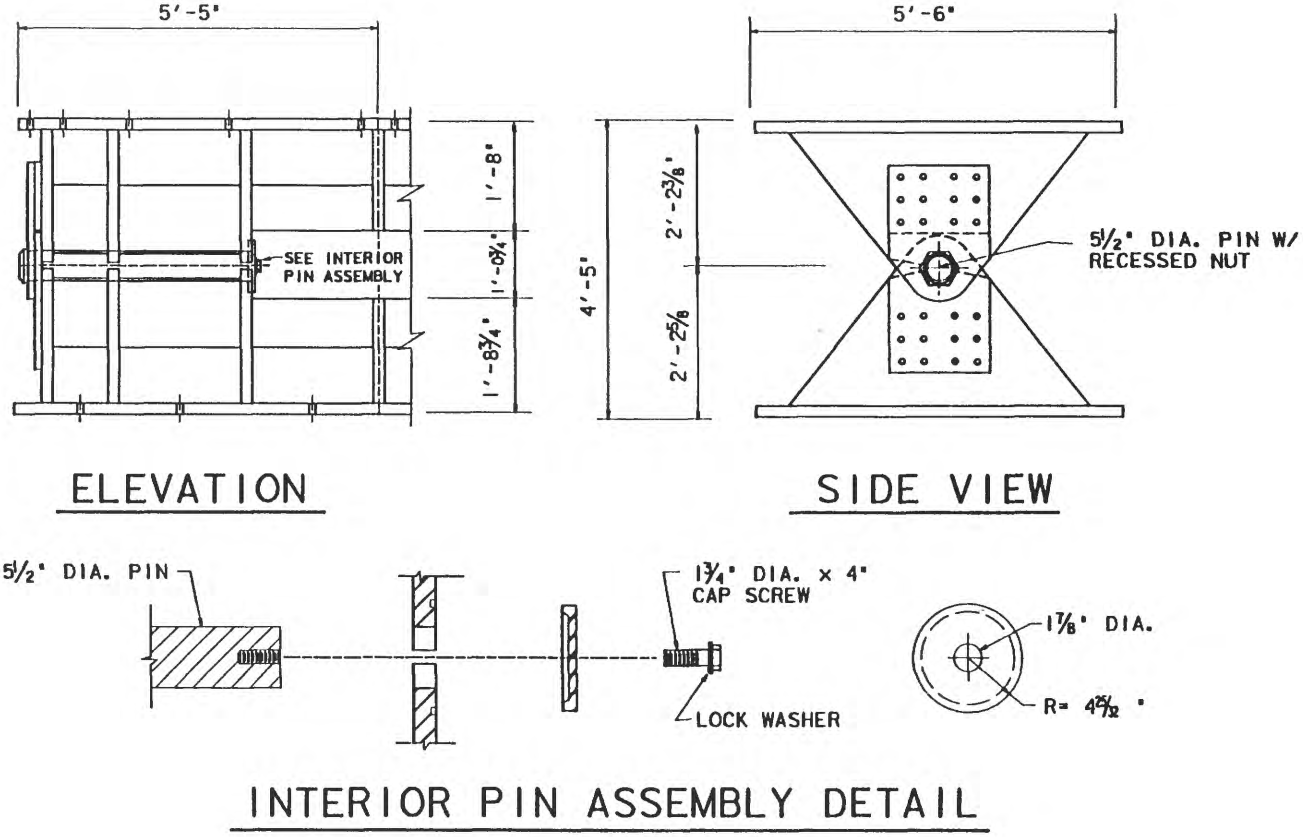


Figure 221.—Details of steel pin support used as bearings at Piers 18 to 21 of San Mateo-Hayward Bridge (courtesy of Earthquake Engineering Research Institute).

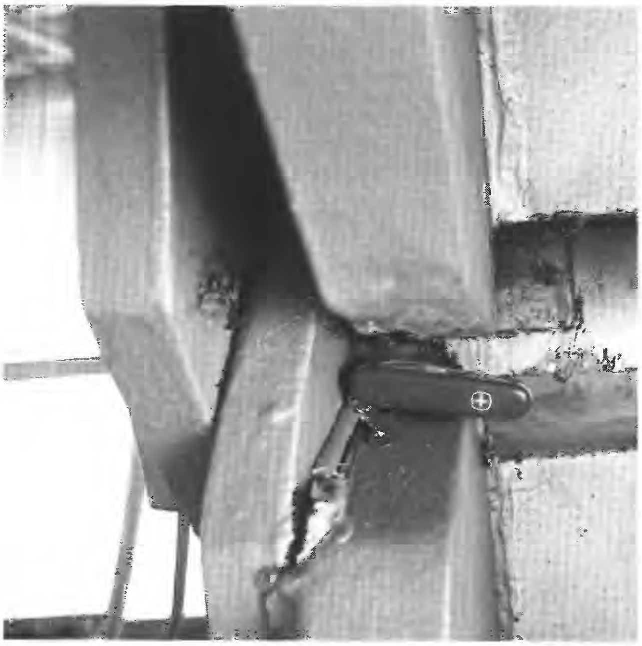


Figure 222.—Signs of movement at bearing (scraped paint) of San Mateo-Hayward Bridge (courtesy of Earthquake Engineering Research Institute).

Figure 223.—Broken cap and washer from bearing of San Mateo-Hayward Bridge (courtesy of Earthquake Engineering Research Institute).

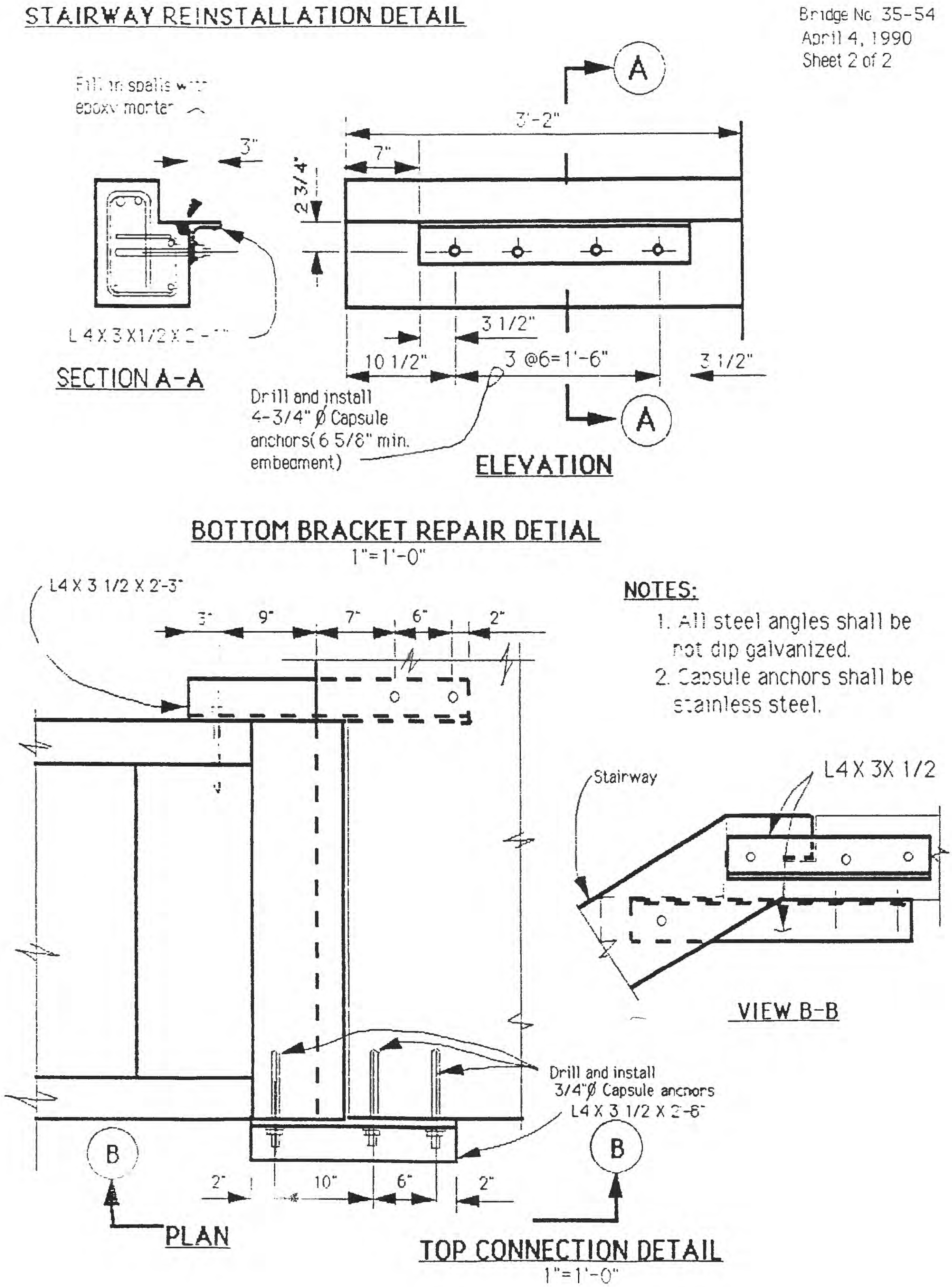


Figure 224.—Plan for stairway repair of San Mateo-Hayward Bridge (White, 1990).

Figure 225.—Ship channel under San Mateo-Hayward Bridge.

MORA DRIVE OVERCROSSING

DESCRIPTION OF BRIDGE

This lightly traveled bridge goes over Route 280 near the city of Los Altos (fig. 227). It is a two-span, continuous, six-cell, prestressed, box girder superstructure with a rounded soffit at the overhangs. It is supported on a single, reinforced concrete column and two diaphragm-type abutments (fig. 226). The column is very wide transversely, with a large flare where it joins the superstructure. The bridge is on a 600-foot radius curve and has a moderate skew with respect to the highway below. It was built in a cut section of the highway on what appears to be fairly weak material (fig. 228). The abutments are on a single row of piles and the column is supported by a spread footing.

BRIDGE DAMAGE

The most serious damage to this bridge was several wide cracks that appeared to go all the way through the column (figs. 229 and 230). These cracks began at the footing on the north side of the column and extended to 10 feet above the footing on the south side of the column. There were also some smaller cracks. Maintenance engineers chipped out about 7 square feet of the cracked portion of the column to a depth of 6 inches. They found some of the reinforcement was heavily corroded. The abutments also showed signs of movement including some concrete spalls where the abutment met the bridge soffit (figs. 231 and 232).

There are several possible explanations for the column damage. The column had only #4 rebar at 12-inch

Figure 226.—Mora Drive Overcrossing.

Br #37-235 / Route 280 / Post Mile 13.12

Approximate Latitude & Longitude
N. Lat. 37° 20.6' W. Long. 122° 04.6'

Epicentral Distance
23.9 miles

Peak Ground Acceleration	**N/S**	**U/D**	**E/W**
Saratoga Aloha Ave.	0.34	0.41	0.53

Length	**Width**	**Skew**	**Year Built**
207'	38.0	Varies	1967

Main Span Type
Concrete box girder bridge

Average Daily Traffic = **700**

spacing for transverse reinforcement. It also had a large flare that attracted more shear. The reinforcement was corroded in the area of damage. All these things probably contributed to the damage.

BRIDGE TESTING AND REPAIR

On February 22, 1990, a Portable Linear Accelerator (MINAC) was used to see if radiology could detect smaller networks of cracks in reinforced concrete at this bridge. The film was examined using a Quantex image processor, but only a few minute cracks were found propagating from the large crack. It also showed that the cracks on each face were not connected.

After the earthquake, a layer of asphalt was spread behind the abutment to smooth the approach (fig. 233). In 1992 the bridge was repaired and retrofitted with a steel casing for the column that matched the flare (fig. 234). A top layer of reinforcement and concrete was added to the existing column footing to allow it to handle the plastic moment capacity of the column. Also, the cracked soffit concrete was patched near the abutments.

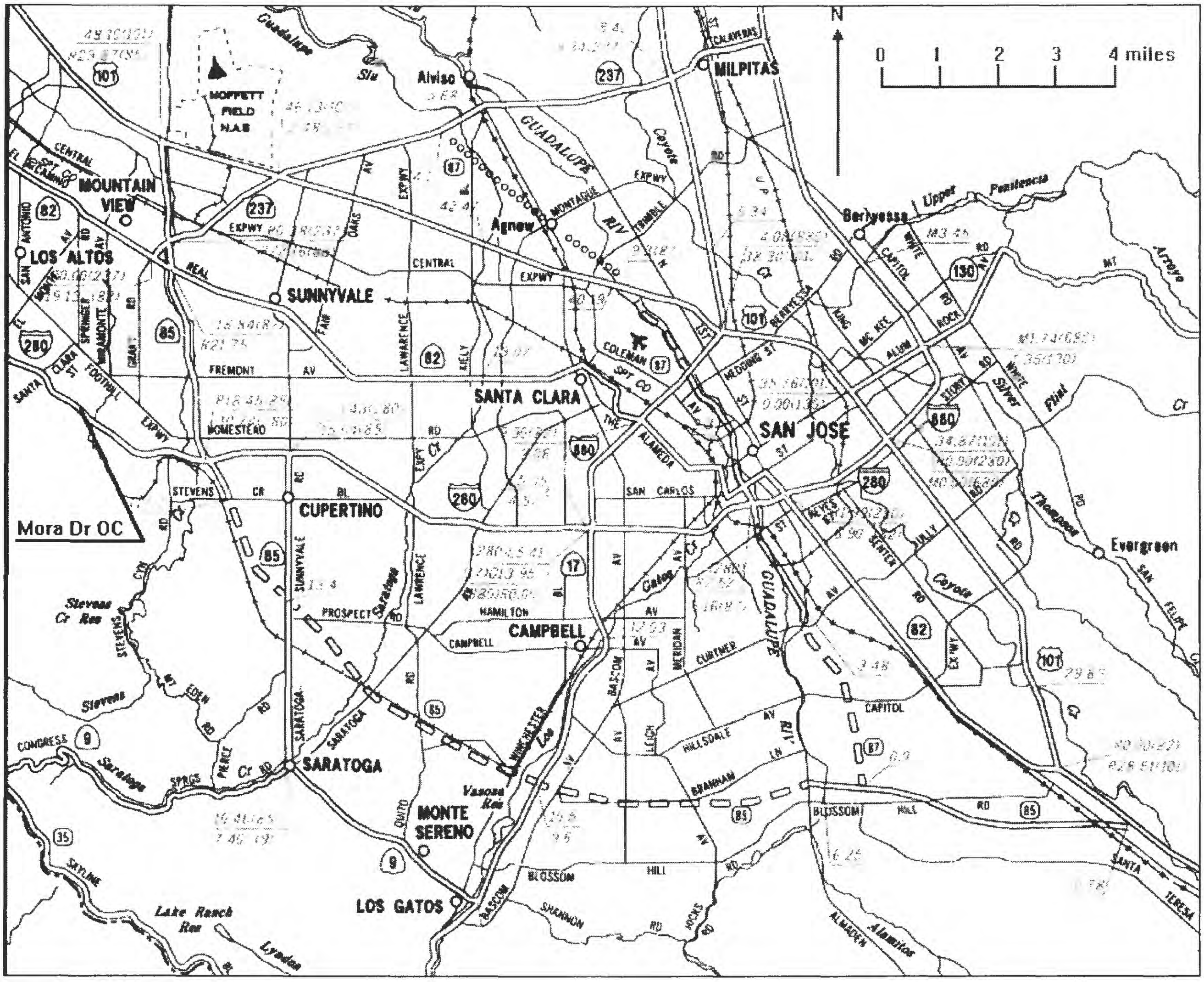


Figure 227.—Location of Mora Drive Overcrossing.

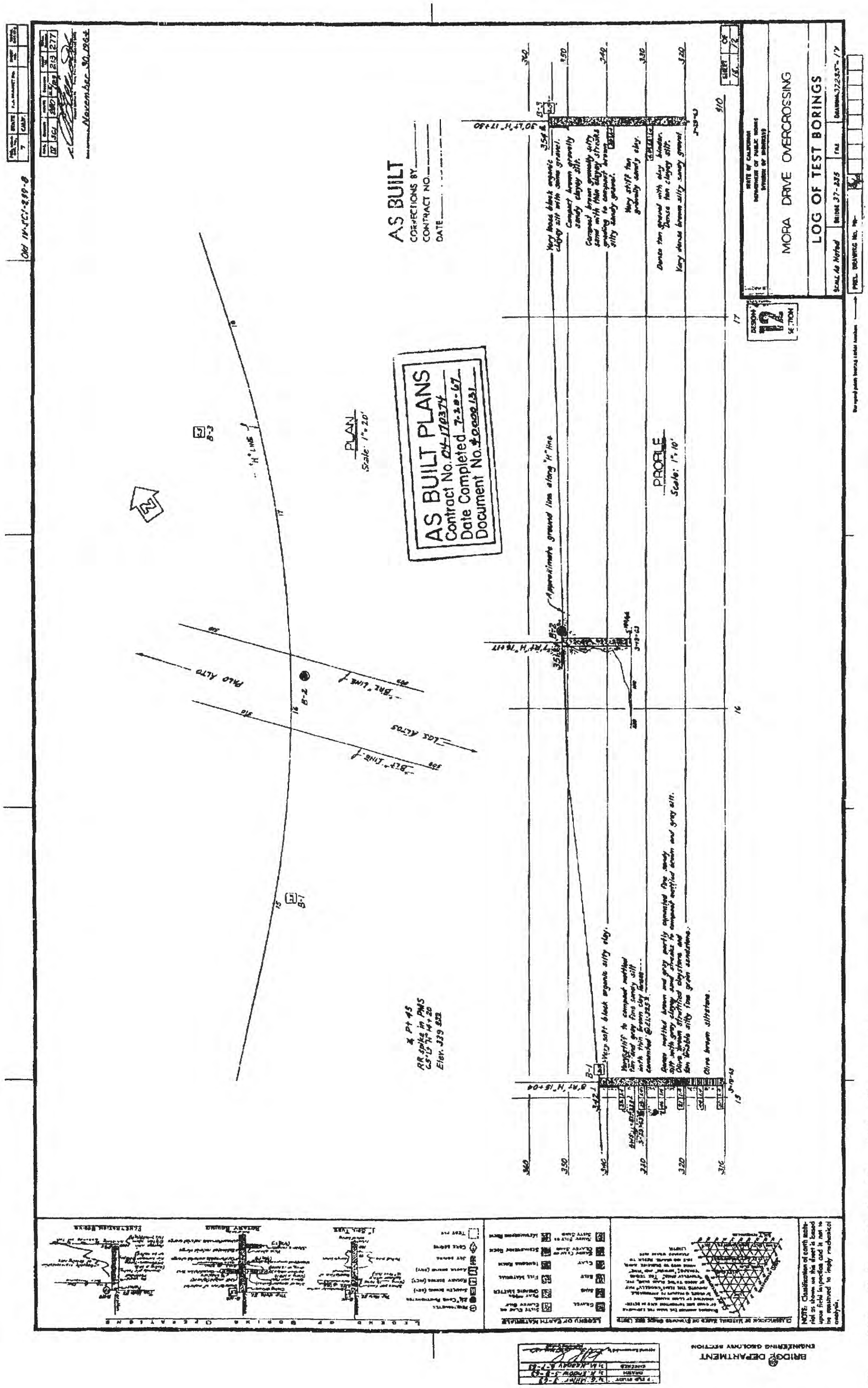


Figure 228.—Soil profile for Mora Drive Overcrossing.

Figure 229.—Damage on northwest side of column at Mora Drive Overcrossing.

Figure 230.—Damage on southwest side of column at Mora Drive Overcrossing.

Figure 231.—Vertical offset of west abutment at Mora Drive Overcrossing (courtesy of Earthquake Engineering Research Institute).

Figure 232.—Horizontal offset of west abutment at Mora Drive Overcrossing (courtesy of Earthquake Engineering Research Institute).

Figure 233.—Repair to approach of Mora Drive Overcrossing (courtesy of Earthquake Engineering Research Institute).

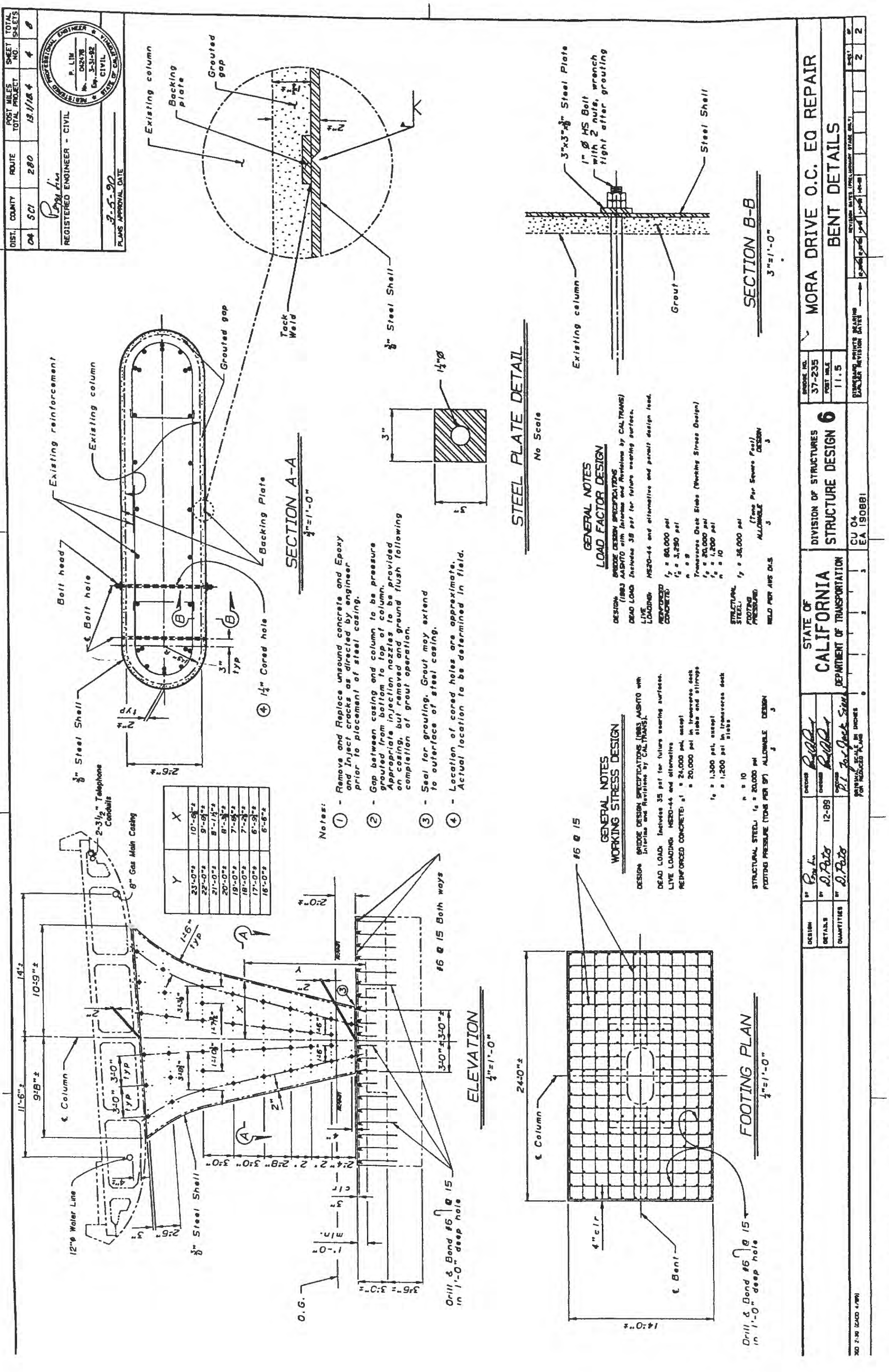


Figure 234.—Repair and retrofit details for Mora Drive Overcrossing.

BRIDGE MOTION

The fault rupture and ground motion during the earthquake has been well documented, and information can be found in many reports, some of which are referenced in the back of this section (Simpson 1994; Borcherdt, 1994). A simple description of the earthquake location, magnitude, and movement is provided in this section for the purpose of understanding bridge behavior. Estimates of the ground motion at each bridge site along with latitude, longitude, and distance to the epicenter were provided in the previous section which describes bridge damage. This section provides strong motion records for the six bridges that were instrumented at the time of the earthquake.

As has been previously stated, the earthquake occurred on October 17, 1989, at 5:04 p.m. Pacific Standard Time. It is thought that the main shock was the result of a 1.6-meter right-lateral strike slip and a 1.2-meter reverse slip along a left-stepping bend of the San Andreas fault in the Santa Cruz Mountains. At this location, the fault has a bearing of N. 50° W. and a dip of 70° SW. Fault rupture lasted about 6 to 8 seconds, with strong shaking lasting 10 to 15 seconds at most sites. The estimated earthquake location and magnitude according to the California Division of Mines and Geology is:

Epicenter: 37.037° N., 121.883° W. Depth 18 km
Magnitude: 7.0 M_L (BRK)7.1 M_S (NEIS)

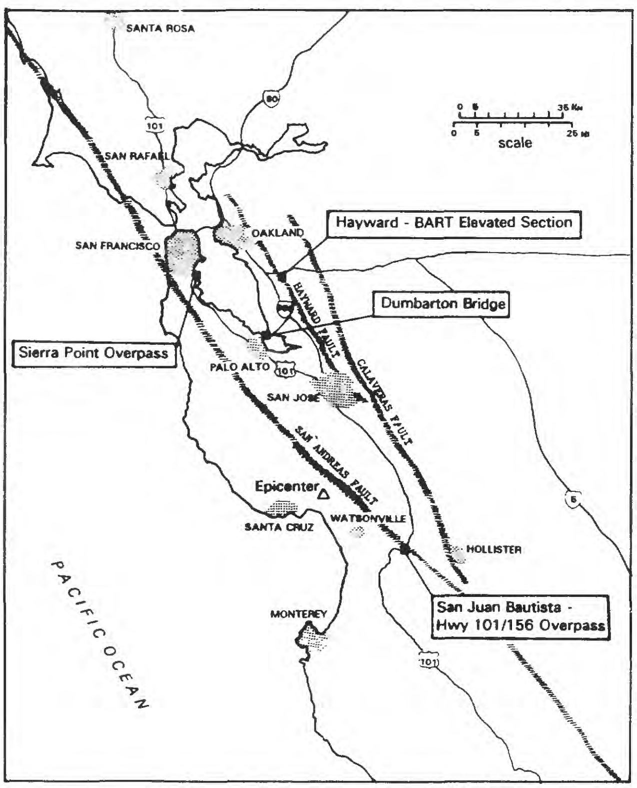


Figure 235.—Map of CDMG instrumented bridges that recorded earthquake motion (courtesy of California's Strong Motion Instrumentation Program).

A unique feature of this earthquake is the extent of bridge damage 100 km or more from the epicenter. It is felt that enhanced ground shaking due to areas of "soft soil" that amplified the motion caused most of this damage. There has also been speculation of other factors, such as the reflection of the seismic waves off the Moho discontinuity (the surface area between the crust and mantle) and from directivity effects. However, since a discussion of this subject is beyond the scope of this paper, interested readers should refer to the references listed at the back of this report.

Records of bridge motion during earthquakes are an engineer's best tool for checking assumptions about bridge behavior and for improving computer models. Unfortunately, only 10 bridges were instrumented in California at the time of the earthquake. After the earthquake the Governor's Board of Inquiry recommended instrumenting more bridges (Thiel, 1990), and Caltrans has worked out a partnership with the California Division of Mines and Geology that has resulted in 28 instrumented bridges as of August 1994.

There are strong-motion records for six bridges whose instruments were activated by the earthquake. These include four bridges with records processed by the California Division of Mines and Geology (CDMG) (fig. 235; table 21) (Shakal, 1989) and two partial instrumentations by the U.S. Geological Survey (Maley, 1989; Mork, 1994). The four bridges instrumented by CDMG are as follows:

1. Sierra Point Overhead is the first bridge in California to be retrofitted with lead/rubber isolation bearings.
2. The Dumbarton Bridge is a long, steel structure crossing San Francisco Bay.
3. The Hayward Bay Area Rapid Transit Bridge has three simple-span reinforced concrete box girder spans.
4. The San Juan Bautista Route 156/101 Overhead was only 20 miles from the epicenter. It is a six span steel bridge.

The two USGS-instrumented bridges are as follows:

1. The 101/280/680 Separation in San Jose had three instruments in a bay of the first span.
2. The toll building south of the Golden Gate Bridge had an instrument in the basement.

Table 21.—*Instrumented bridges and recorded earthquake motion*

Name of Bridge	Type of Structure	Damage	Year Built	No. of Sensors	Distance (in miles)	Maximum Horizontal Acceleration*		
						Free-field	Base	Bridge
Sierra Point OH	10 span steel girder	none	1957	13 + FF	53	*.11 g*	*.11 g*	*.39g*
Dumbarton Bridge	43 span steel and concrete box	very minor	1982	26 + FF	35	*.13 g*	*.13 g*	*.44g* *(.61 g)*
Hayward-Bart Viaduct	3 span concrete box	none	1967	13 + FF	46	*.16g*	*.15 g*	*.51 g* *(.59g)*
Rte 156/101 Sep.	3 span steel girder	none	1958	12	20	-	*.14g*	*.50g* *(.94g)*
101/280/680 Sep.	3 span concrete box	none	1974	3	21	-	-	*.18g*
Golden Gate Br.	suspension bridge	none	1936	1	60	*.24g*	-	-

***Value in parentheses indicates a spike in the record.**

SIERRA POINT OVERHEAD

DESCRIPTION OF BRIDGE

The Sierra Point Overhead carries Highway 101 over two sets of Southern Pacific Railroad tracks in South San Francisco (figs. 236 and 237). It was built in 1957. The horizontal alignment is a 7,500-foot-radius curve with an orientation of approximately N. 23° E. (figs. 238 and 239). The bridge has an unusual geometry. The north abutment has a 59° skew, and the south abutment has a 72° skew, while the column bents are normal to the centerline of the bridge. This means that the column bents near the abutments do not extend the full width of the bridge. The bridge has 10 simply supported spans varying from 26 to 100 feet in length. The superstructure is a composite concrete deck supported by 18 to 54-inch-deep steel girders. The girders are supported at the bents by 68-inch-deep transverse beams. One end of the girders is supported by rocker bearings, while the other end is bolted to the transverse beam. There is an expansion joint at each bent and at the two abutments. The transverse beams were originally supported by spherical pin-type steel bearings that sit on top of 3-foot-diameter, 25-foot-tall concrete columns on spread footings. The bridge is just south of a large hill that was cut slightly to make room for the bridge. It is also within 1,000 feet of the Bay.

Soil borings were done in 1950 and again in 1981 for the connector structure 100 feet south of Sierra Point Overhead (figs. 240 and 241). The 1950 borings were taken on the hill north of the bridge, while the 1981 borings were taken

Figure 236.—Sierra Point Overhead.

Bridge # 35-130 / Route 101 / Post Mile 23.7

Approximate Latitude & Longitude
N. Lat. 37.674° W. Long. 122.388°

Epicentral Distance
53 miles

Peak Ground Acceleration N/S U/D E/W
200' west of bridge 0.11 0.05 0.06

Length Width Skew Yr Blt YrRet
616' 117' @60° 1957 1985

Main Span Type
Concrete deck slab on steel girders.

Substructure Type
Four (3 foot diameter) reinforced concrete column bents (or less due to skew) on spread footings.

on the plain south of the hill. The 1950 log has one boring at the depth of the column footings. It indicates a stiff clay and fractured sandstone at that depth that was suitable to support the structure on spread footings. The 1981 borings show loose gravel and clay at the same elevation, indicating a different geology on the plain.

Sierra Point Overhead has undergone two modifications since it was built. In 1967, girders and a deck were added to the center of the bridge to increase the number of traffic lanes (fig. 242). In 1986 the first seismic retrofit in California to include seismic isolation was done on this bridge. The retrofit addressed the two seismic vulnerabilities of this structure, inadequate column capacity and the steel girders falling off the rocker bearings. The replacement of the steel bearings on the columns with lead/rubber isolation bearings and the addition of longitudinal rod restrainers addressed these weaknesses.

The fact that the bridge superstructure is a series of simple spans that are still free to rattle and that the end spans can bang against the abutments makes this bridge slightly less effective for isolation. Still, this retrofit should be more than adequate to protect the structure from major damage during a large earthquake. It is thought that during a large enough event, the back and side walls of the abutment would break, isolating the bridge from the ground motion. In the meantime, the abutment protects the vulnerable columns from the earthquake. Figure 243 shows the fixed girder ends, the spherical pin-type-steel bearing supporting a transverse beam, and the addition of longitudinal rod restrainers extending through the transverse beam on the bridge.

Figure 244 shows a lead/rubber isolation bearing installed on top of a column. All 27 columns for this bridge had the spherical bearings replaced with lead/rubber bearings. Also, as part of the retrofit, the bridge was instrumented with 13 sensors on the structure and three additional sensors nearby on the ground to obtain the free-field motion. Figure 245 shows the instruments mounted on the two westernmost columns of Bent 7 and on that portion of the transverse beam between them.

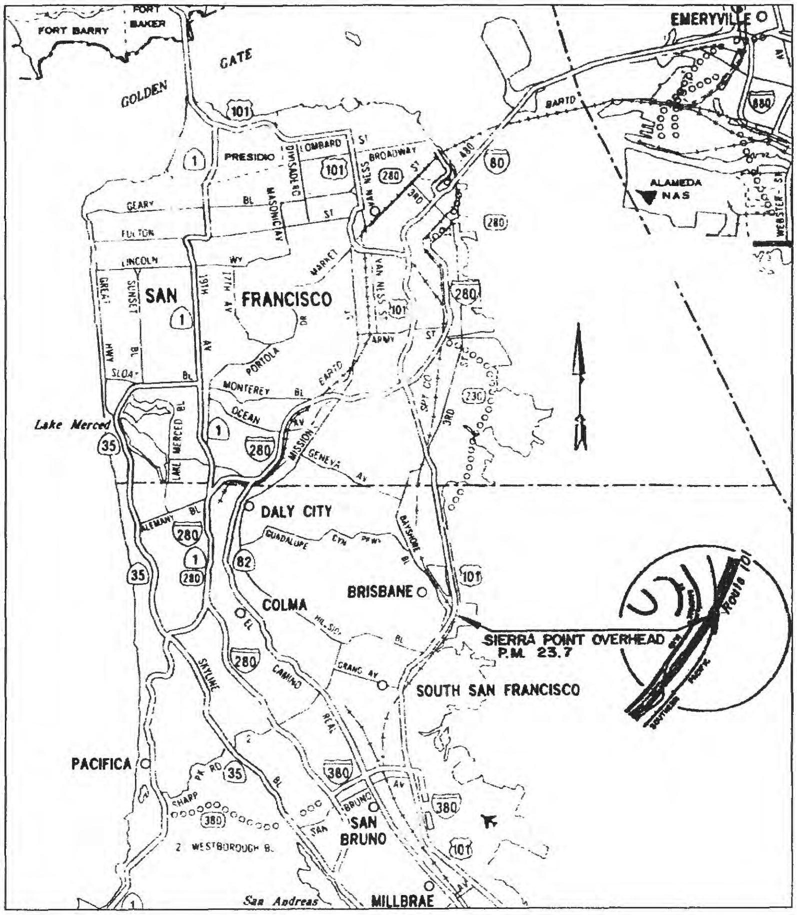


Figure 237.—Location of Sierra Point Overhead.

BRIDGE MOTION

During the earthquake the peak ground acceleration was 0.11 g parallel to the bridge and 0.06 g perpendicular to the bridge (fig. 246). The acceleration records on the bridge show some "chatter" resulting from the movement of the two simply supported spans attached to the transverse beam (fig. 247). Also, we see little reduction of acceleration above and below the isolators, reflecting the fact that the bridge was not isolated from the abutment motions.

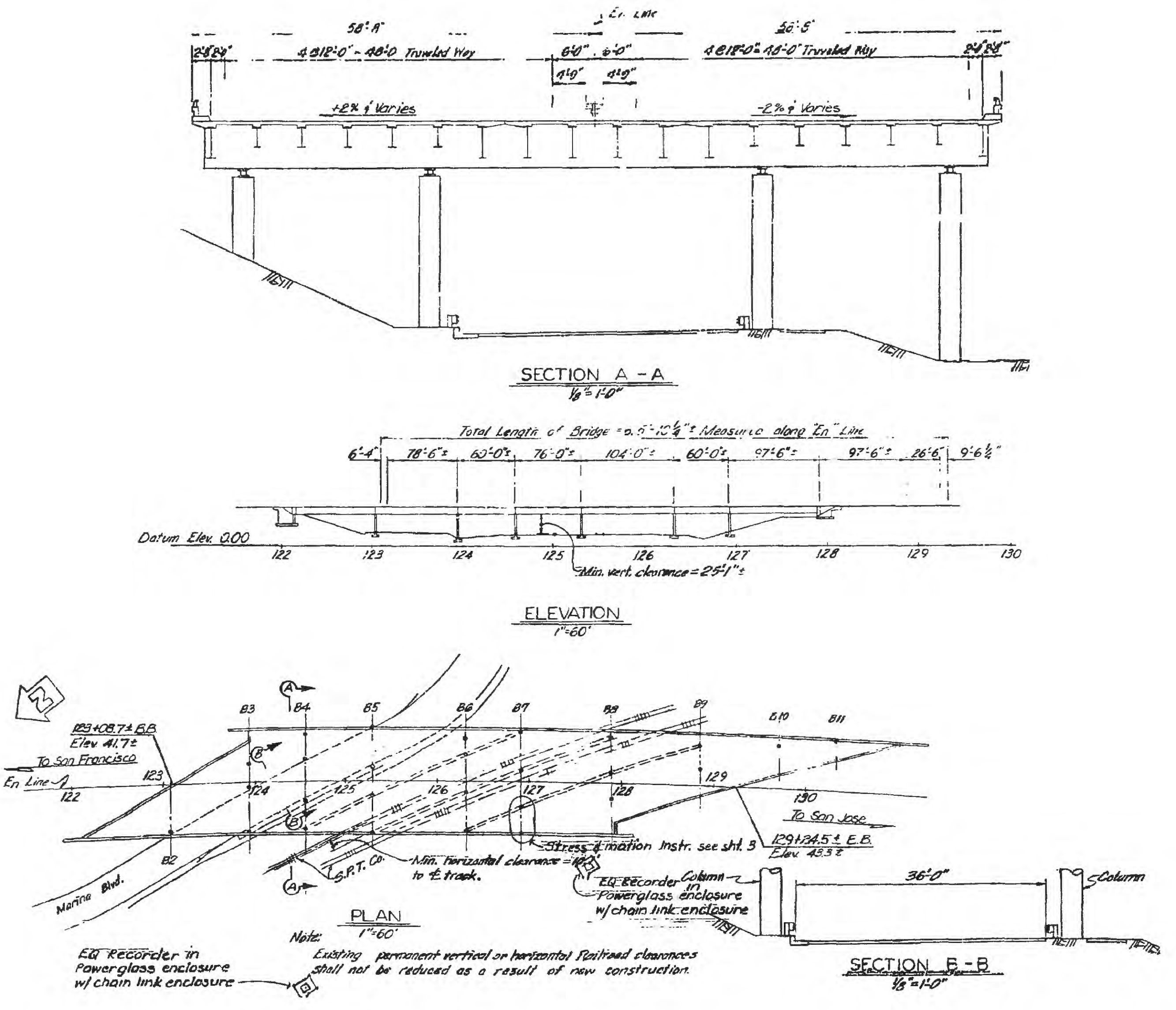


Figure 238.—General Plan drawing for seismic retrofit of Sierra Point Overhead.

Figure 239.—Aerial view of Sierra Point Overhead (Thiel, 1990).

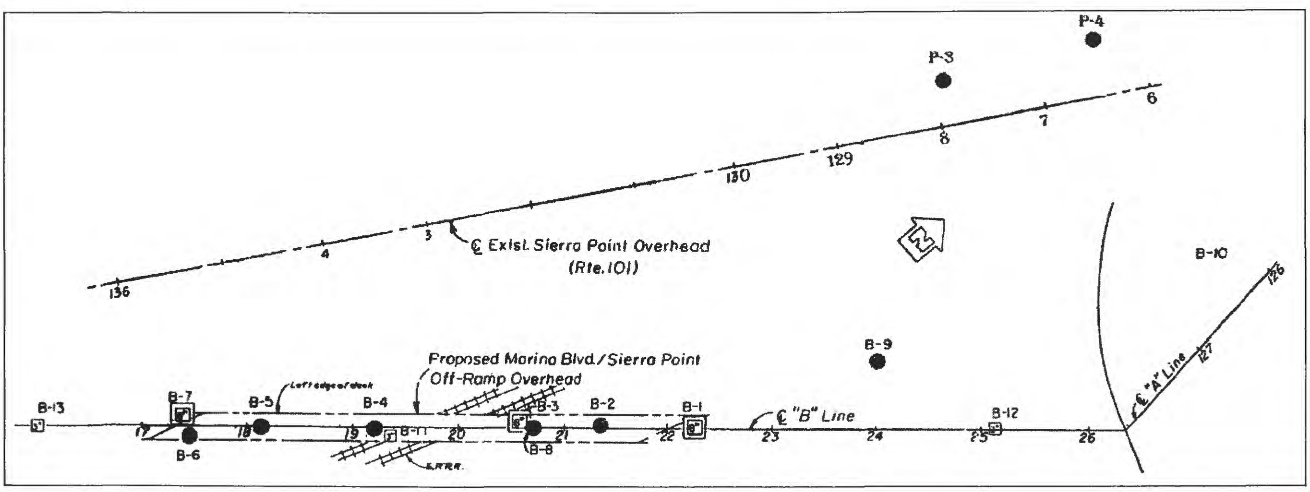


Figure 240.—Location of soil borings at Sierra Point Overhead.

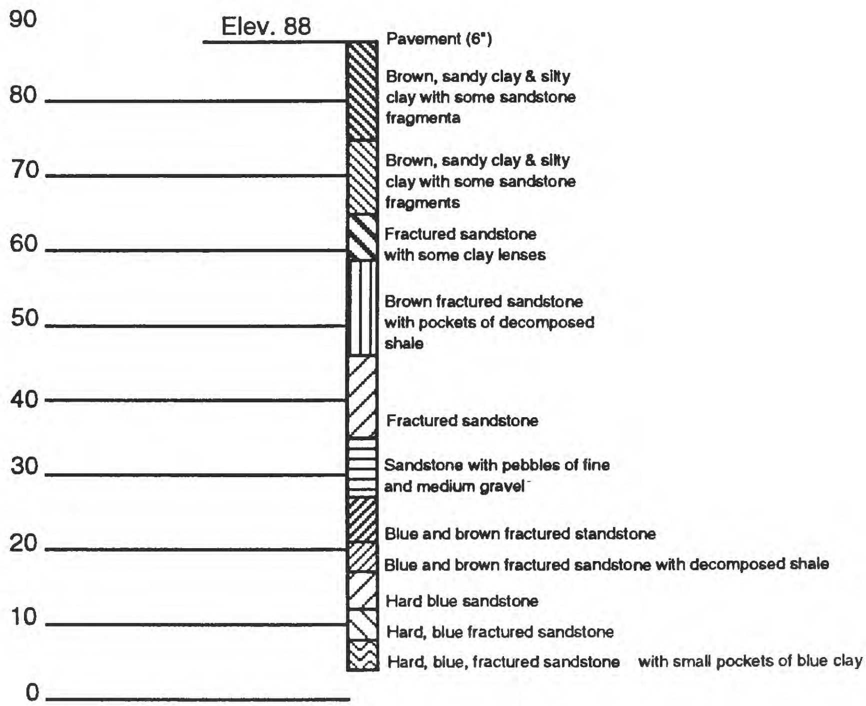


Figure 241.—Boring log for Sierra Point Overhead.

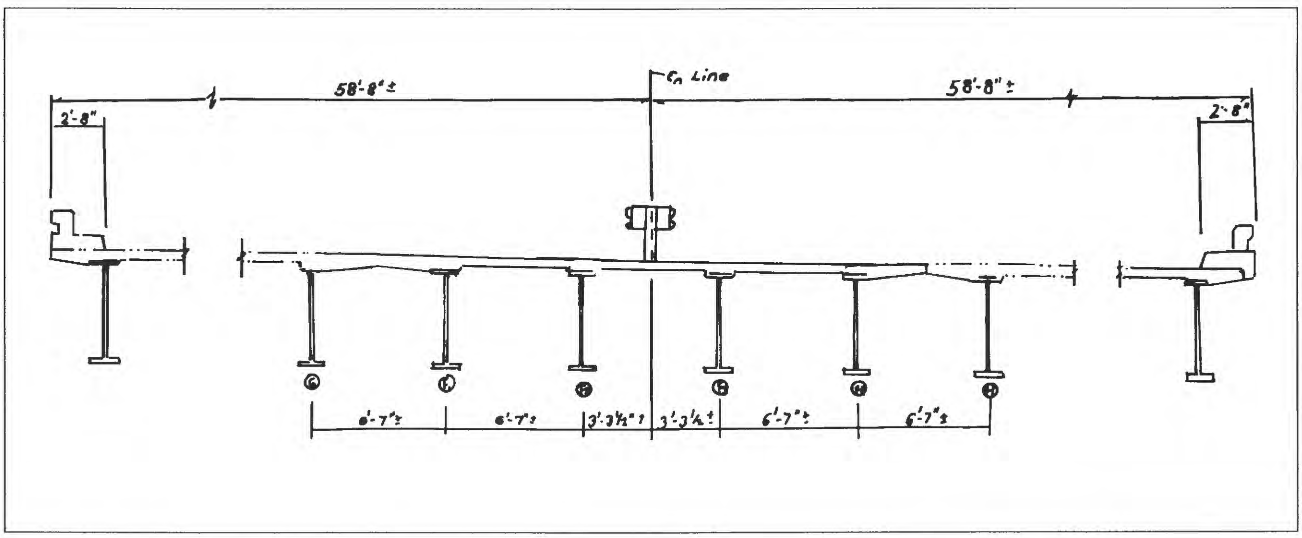


Figure 242.—Section drawing for widening of Sierra Point Overhead.

Figure 243.—Existing bearings (before removal) during retrofit of Sierra Point Overhead.

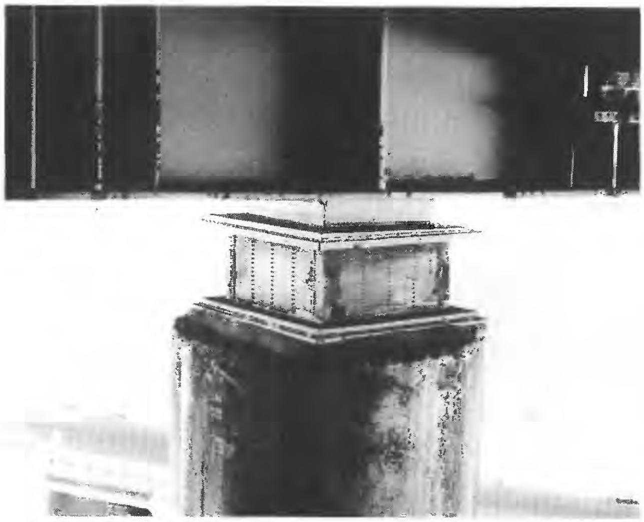


Figure 244.—Newly installed lead/rubber bearing during retrofit of Sierra Point Overhead.

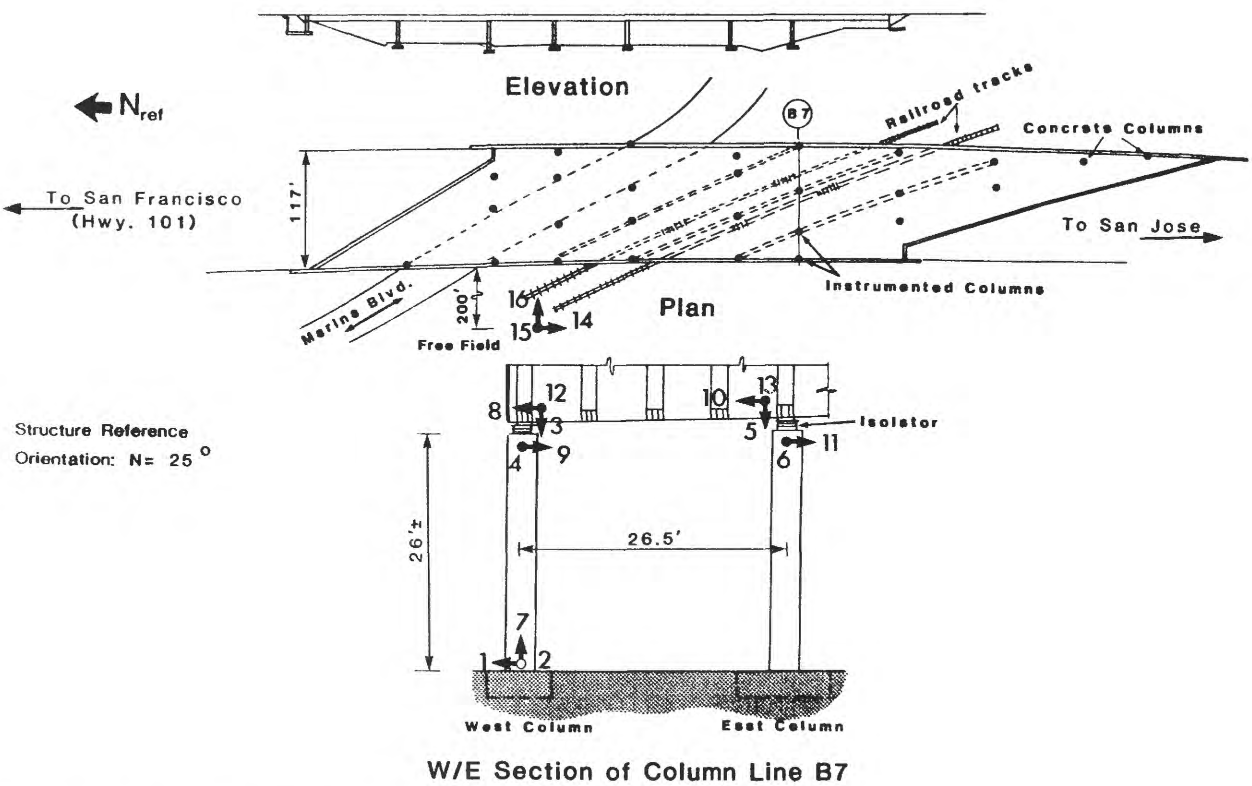


Figure 245.—Location of instruments on Sierra Point Overhead (courtesy of California's Strong Motion Instrumentation Program).

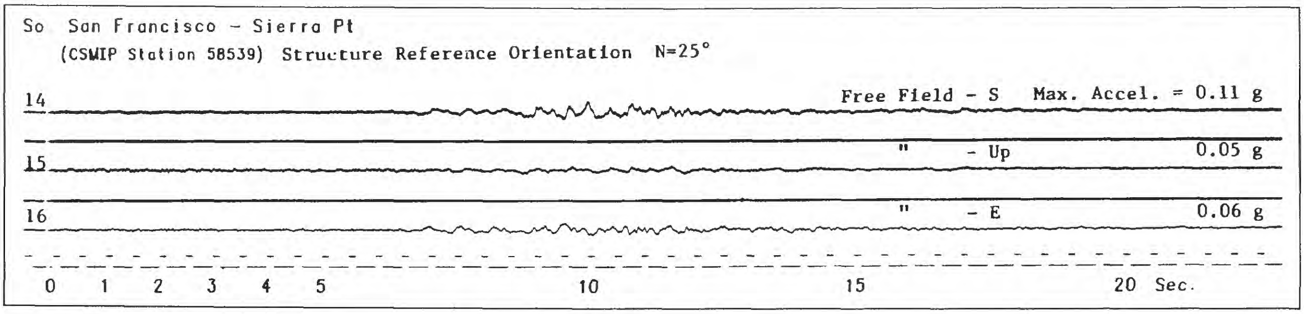


Figure 246.—Ground acceleration records at Sierra Point Overhead during earthquake (courtesy of California's Strong Motion Instrumentation Program).

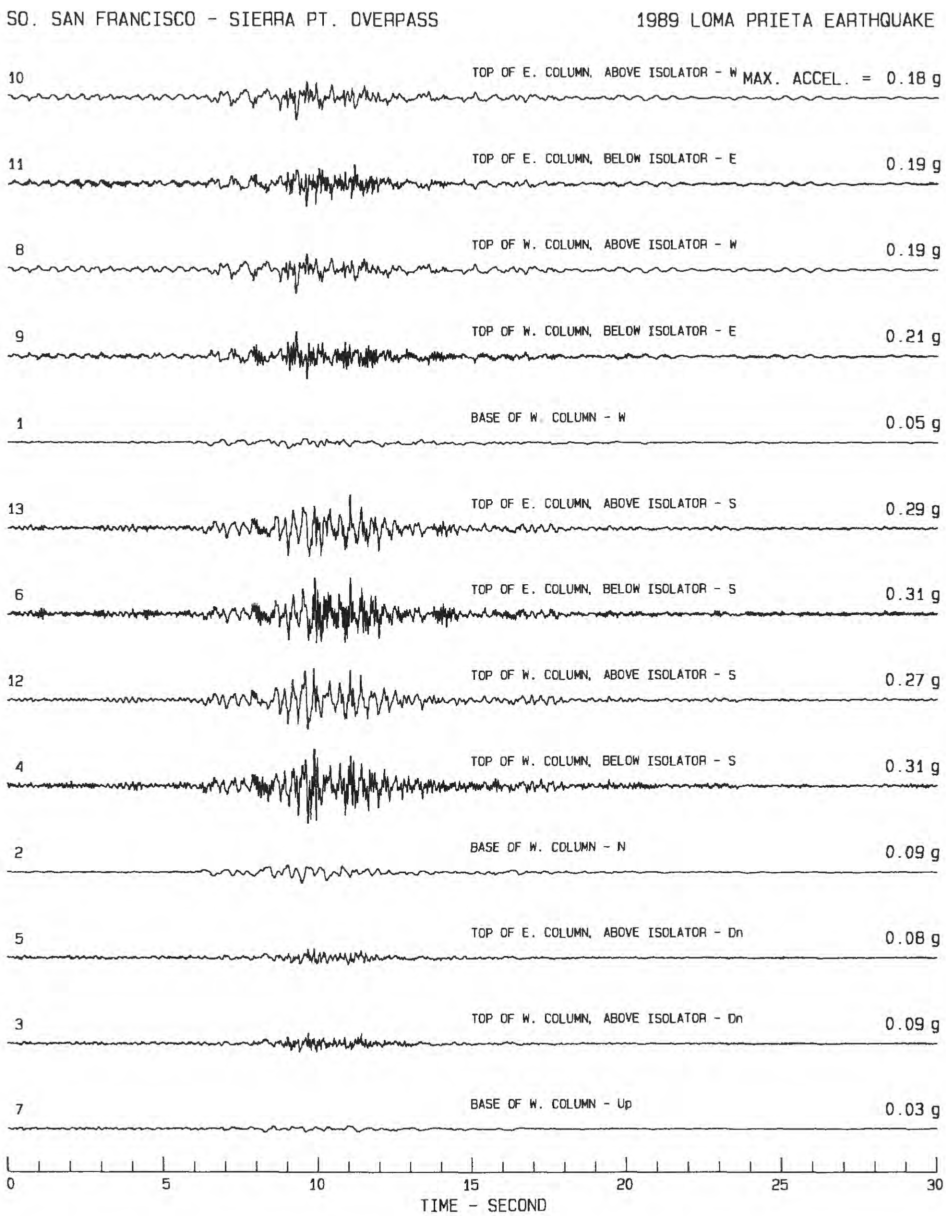


Figure 247.—Acceleration records for Sierra Point Overhead during earthquake (courtesy of California's Strong Motion Instrumentation Program).

Figure 248.—Dumbarton Bridge.

Bridge #35-038 /Route 84 /Post Mile R29.25

Approximate Latitude & Longitude
N. Lat. 37.674° W. Long. 122.388°

Epicentral Distance
35 miles

Peak Ground Acceleration	N/S	U/D	E/W
3/4 mi north of bridge	0.11	0.05	0.06

Length	Width	Skew	Yr Blt	YrRet
8,600'	72'	none	1981	none

Main Span Type
Continuous welded steel box girder

Main Substructure Type
Concrete with inclined hollow legs.

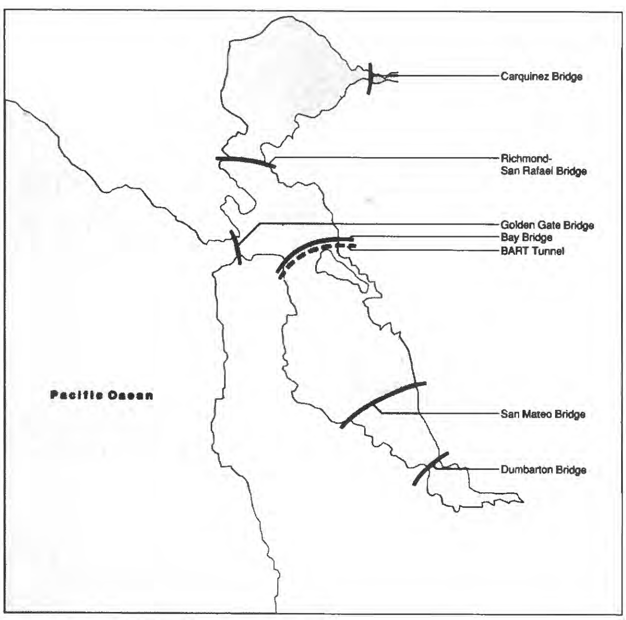


Figure 249.—Location of all Bay crossings.

DUMBARTON BRIDGE

DESCRIPTION OF BRIDGE

The Dumbarton Bridge (fig. 248) is the southernmost of the San Francisco Bay crossings (fig. 249). It was designed in the late 1970's, making it the newest of the crossings. Figure 250 shows the Dumbarton Bridge is composed of two 2,850.5-foot approach structures and a 3,150-foot main span. The approach spans are five prestressed concrete trapezoidal boxes supporting a lightweight concrete deck (fig. 251), and the main span is two steel box girders (fig. 252). The piers are two hollow, inclined concrete columns supporting a concrete bent cap. Since the site is covered in soft Bay mud, the structure is supported on 90-ton concrete piles to a depth of about 60 feet below sea level. Soil borings were done in 1995 that showed Franciscan bedrock at a depth of about 650 feet (fig. 253). Such a deep layer of soil tends to amplify long-period motion.

There are a variety of connections between the superstructure and the bent cap and between adjacent superstructure spans (fig. 254). This figure identifies 10 expansion joints located on the bridge. Two of these joints are deck hinge connections (fig. 255). There are also two types of connections between the superstructure and the bent cap. The pinned connection between the superstructure and bent cap is a slotted groove (fig. 256). The fixed connection between the superstructure and the bent cap are vertical bars that go through the superstructure diaphragm and into the bent cap (fig. 257). There was some disagreement on the ability of this fixed connection to

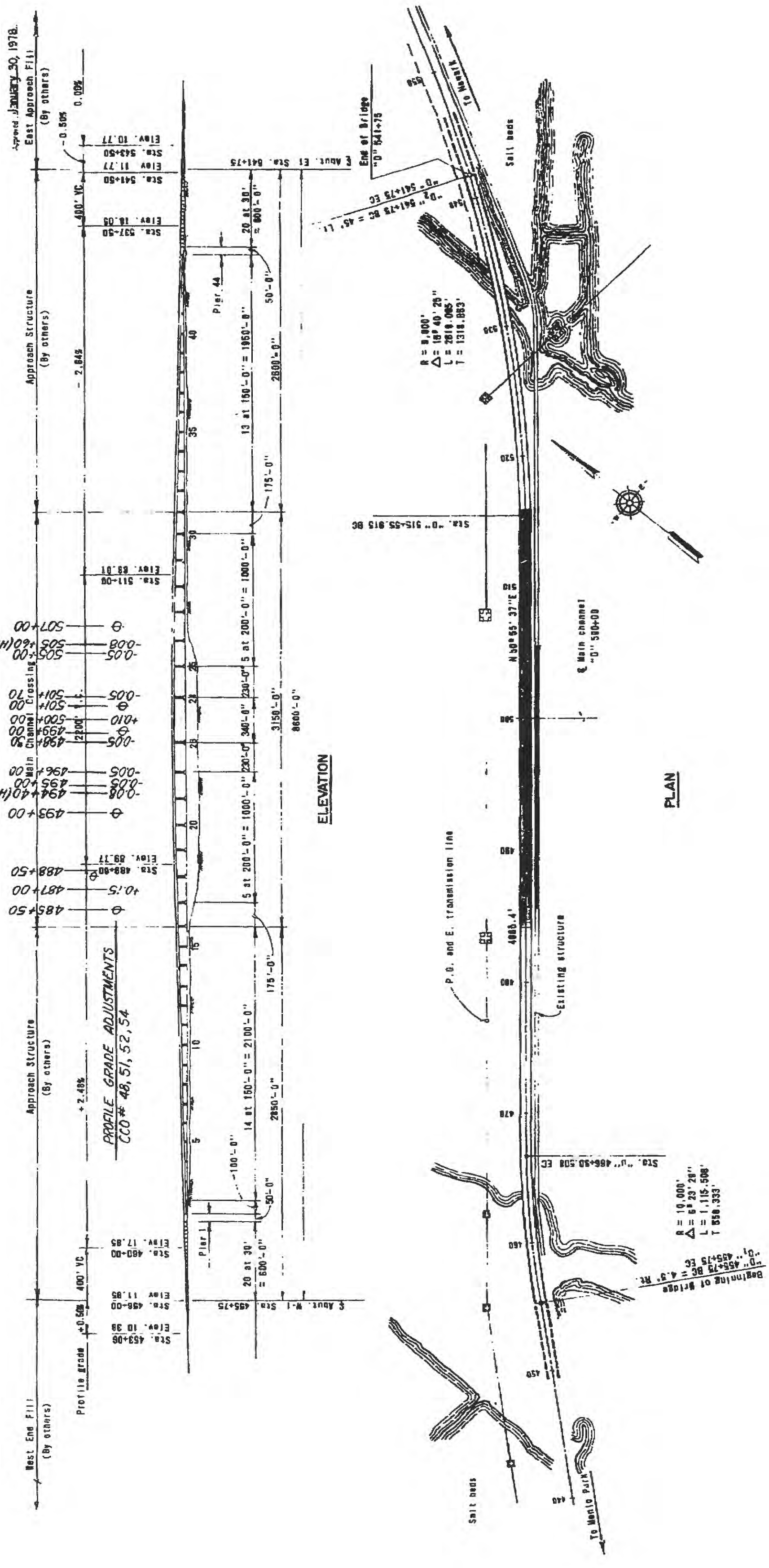


Figure 250.—General Plan drawing of Dumbarton Bridge.

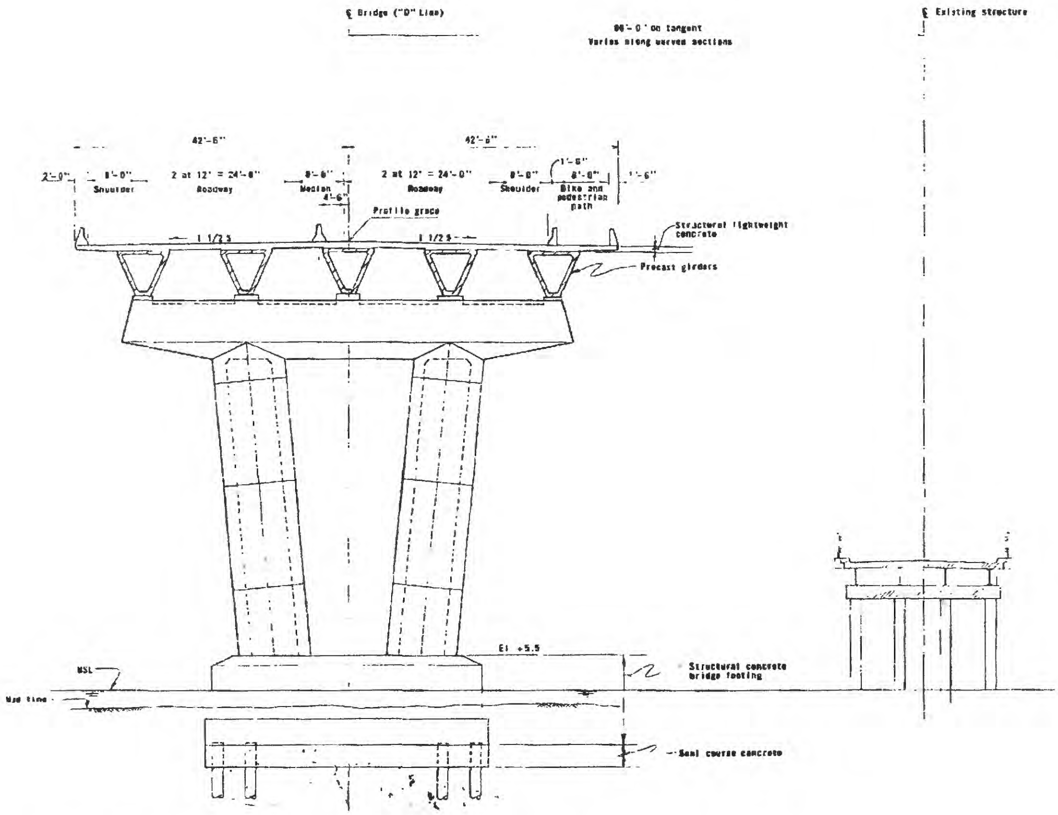


Figure 251.—Typical Section for approach spans of Dumbarton Bridge.

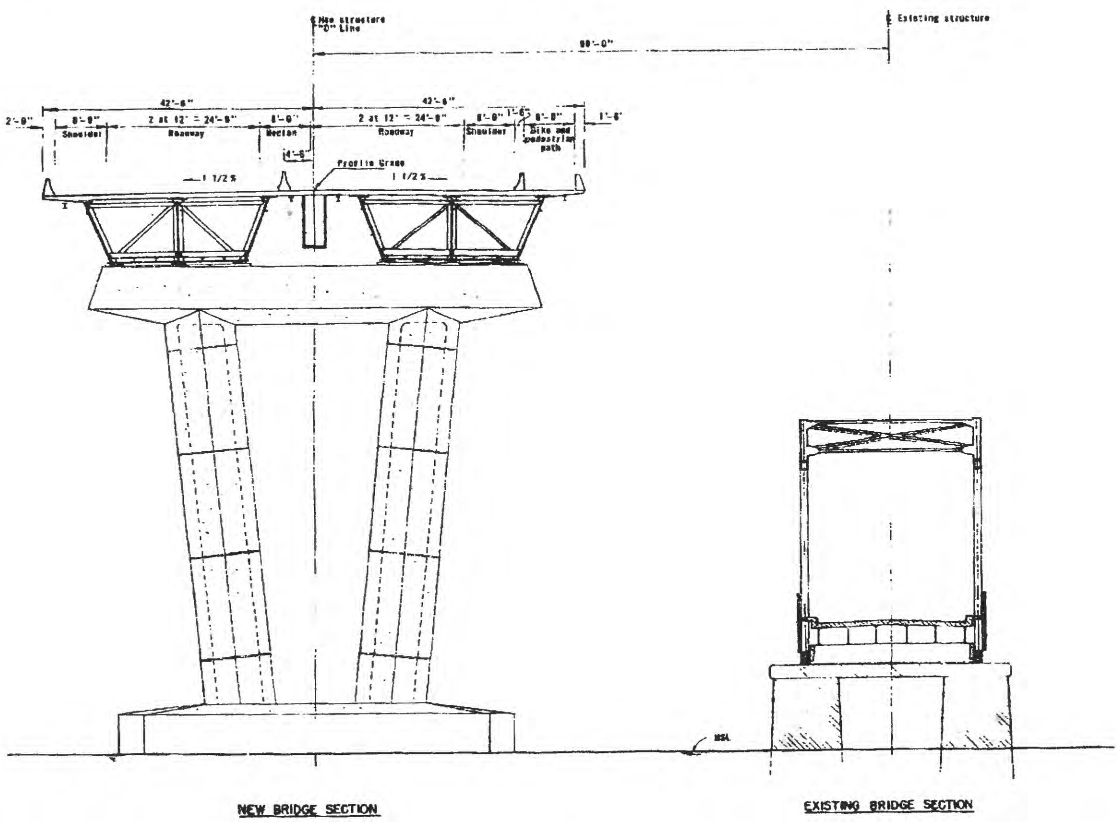


Figure 252.—Typical Section for main span of Dumbarton Bridge.

transfer moment at the joints. There have been three global analyses done so far on the Dumbarton Bridge. The first was a series of analyses done in the 1970's which were done to consider different bridge designs and the final design of the bridge (Baron and Hamati, 1975). The second analysis was to investigate the behavior of the bridge during the earthquake by making a linear, three-dimensional model of the bridge using SAP90 and inputting the free-field motion to see if the acceleration records from the earthquake could be duplicated (Fenves, 1992). An ambitious, nonlinear three-dimensional analysis using NIKE3D was to look at the bridge's behavior for the maximum credible earthquake (Heuze, 1995). However, this project was never completed.

BRIDGE MOTION

The Dumbarton Bridge was instrumented in 1985. The instruments were 26 synchronized Kinemetrics CRA-1 force-balance accelerometers with two recorders using a common trigger (fig. 258). An SMA-1 triaxial flexure accelerometer was installed near each of the approaches to record the free-field motion (fig. 259). All the instruments were installed by Kinemetrics in 1986. During the earthquake, all instruments except the east free-field accelerometer and the vertical sen-

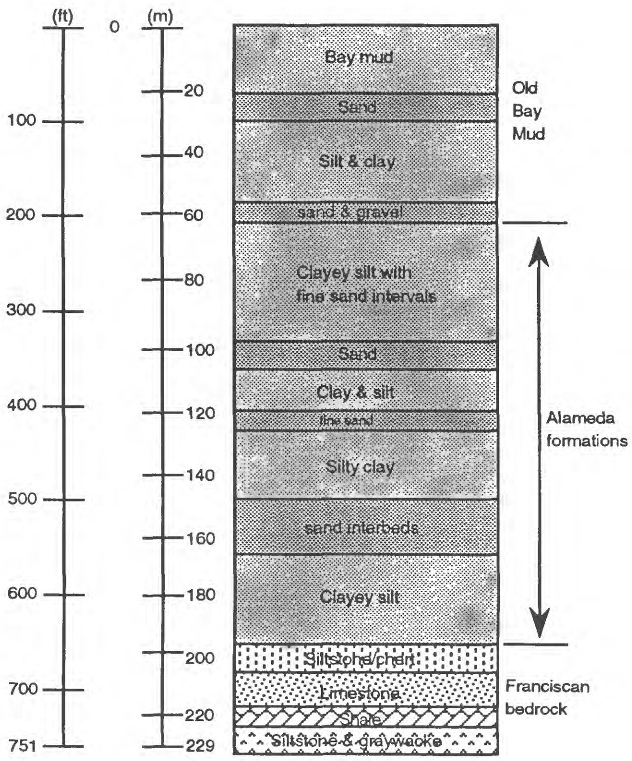


Figure 253.—Soil profile for west side of Dumbarton Bridge.

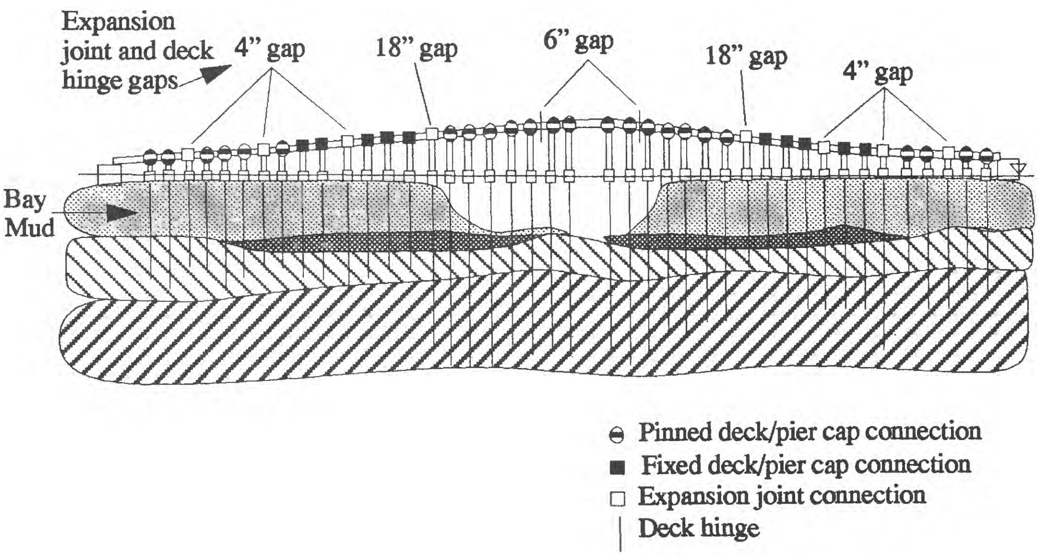


Figure 254.—Dumbarton Bridge with soil profile and superstructure connections (courtesy of Lawrence Livermore National Laboratory).

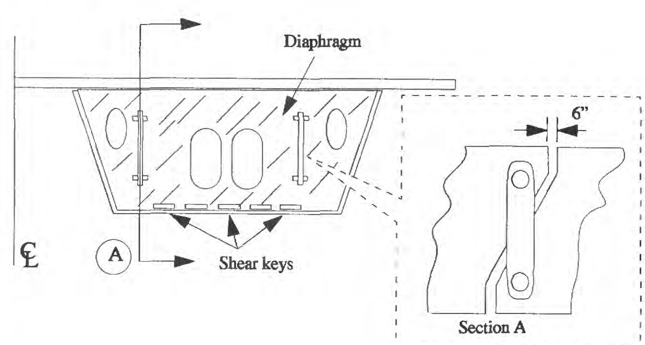


Figure 255.—Deck hinge connection at Dumbarton Bridge (courtesy of Lawrence Livermore National Laboratory).

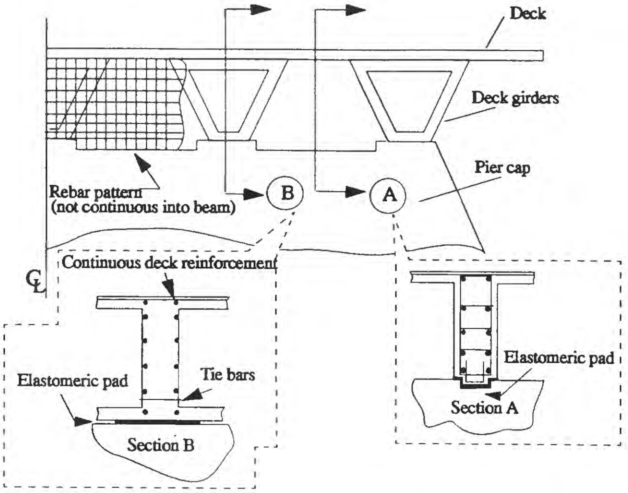


Figure 256.—Superstructure to pier pin connection (courtesy of Lawrence Livermore National Laboratory).

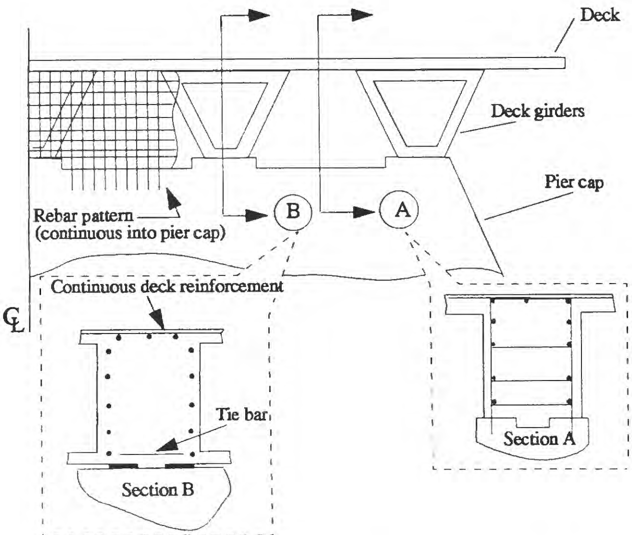


Figure 257.—Superstructure to pier fixed connection (courtesy of Lawrence Livermore National Laboratory).

sor at the foot of Pier 13 recorded the earthquake motion. After the earthquake the film was collected by Kinemetrics and given to CDMG for processing (Shakal, 1989). The two free-field recorders had separate triggers. Consequently, the west free-field record started 1.8 seconds after the recorders on the bridge. The west free-field accelerometer is located approximately three-fourths mile west of Pier 1 on the north side of the road (fig. 260). All other accelerometers are shown in figure 261. Note that the instruments are located on the foundations and bent caps of the west approach structure (Piers 13, 14, and 15) and the main channel crossing (Piers 17 and 21). The free-field channels have a true orientation, while the channels on the bridge are oriented to the bridge (fig. 261: table 22).

During the earthquake, the Dumbarton Bridge suffered very minor damage. Some concrete spalled from around the bolts that hold down the big steel plate expansion joints that span the 18-inch gaps at Piers 16 and 31. This was the only damage found on the bridge. The bridge was briefly closed after the earthquake until an inspection could be done. After the inspection, it was reopened and the spalls were repaired. The bridge inspection report is shown in figure 262.

Information about the free-field motion during the earthquake is available from California's Strong Motion Instrumentation Program (1991), which provides data on 44 ground-response stations. More detailed free-field and bridge motion are also available (California's Strong Motion Instrumentation Program, 1991b). This latter report provides time histories and response spectra for seven lifelines including the four bridges. There appears to be about 20 seconds of strong ground motion (fig. 263) for the Bay mud near the Dumbarton Bridge. The response spectra for this site shows a large increase in amplitude at 2 seconds. This long-period motion for soft clays is particularly dangerous for bridges which have long natural periods. The peak ground acceleration at this site was about 0.128 g in both lateral directions. This is much lower than would be expected for the maxi-

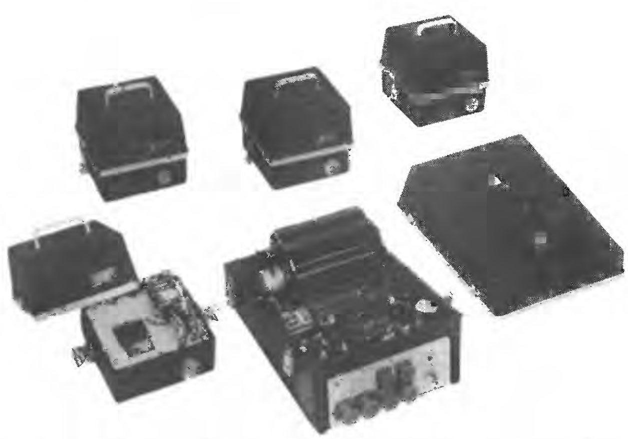


Figure 258.—Kinemetrics CRA-1 and recorder (courtesy of Kinemetrics Inc.).

Figure 259.—Kinemetrics SMA-1 recorder (courtesy of Kinemetrics Inc.).

Figure 260.—Aerial view of free-field accelerometers for the Dumbarton Bridge.

mum credible event at this site. A hazard assessment of the Dumbarton Bridge (Powers, 1993) indicates a peak ground acceleration of 0.5 g. The bridge shows a much longer duration of strong motion (about 60 seconds at the top of the piers) than at the ground. This suggests a relatively small damping ratio for the Dumbarton Bridge. The bridge also shows the highest accelerations longitudinally and very low accelerations vertically. Piers 17 and 21 show high-amplitude spikes, suggesting banging of expansion joints. Since the expansion joint at Pier 16 is 18 inches wide, this pounding is more likely to have occurred at the 6-inch hangers between Piers 21 and 22. The main structure appears to have a period of about 2.0 seconds longitudinally and 1.5 seconds transversely. The bridge motion is summarized in figures 264 to 266 and in table 23.

The capacity of the piers was well above the forces experienced during the earthquake (Fenves, 1992). Fenves suggests that the free-field instruments be placed closer to the bridge and that downhole instruments be placed at the site to give a better picture of the ground motion. Since the earthquake, borings were made down to 500 feet and instrumented to obtain motions all the way to bedrock for future events. An agreement has been reached with California's Strong Motion Instrumentation Program to maintain and process the records obtained from these instruments.

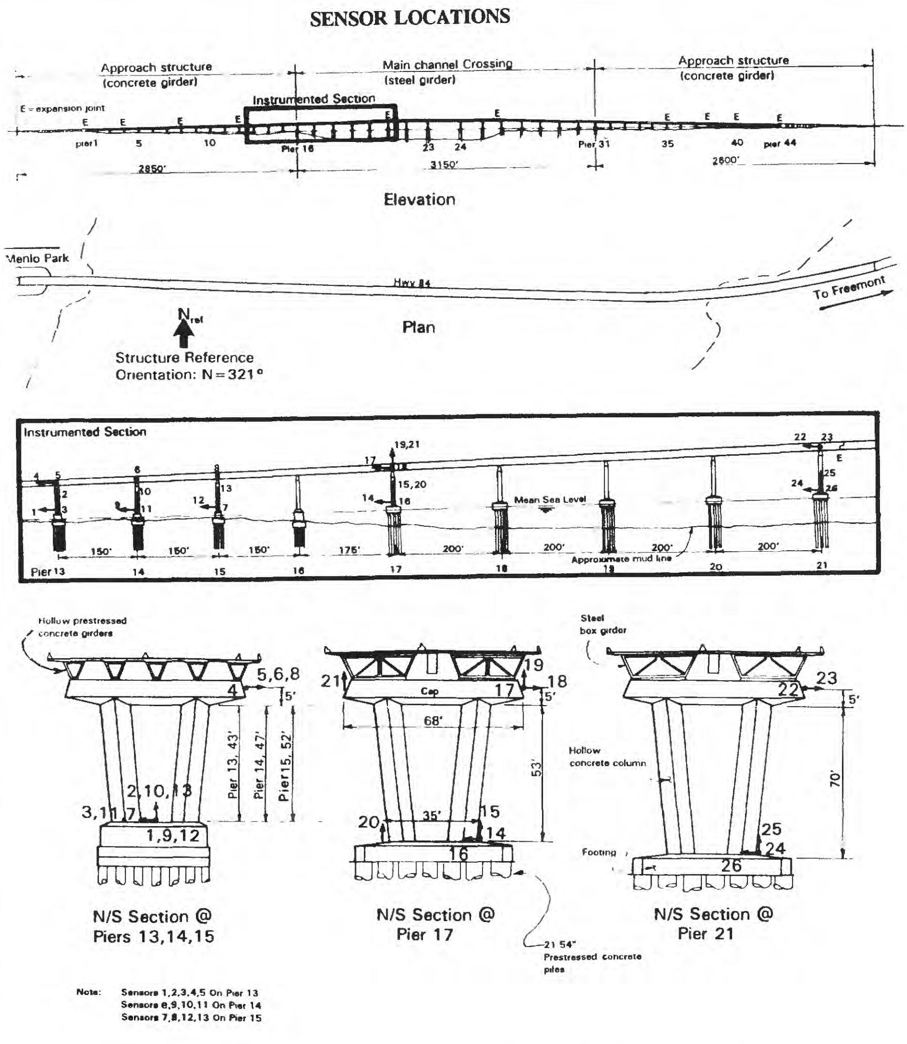


Figure 261.—Location of accelerometers at Dumbarton Bridge (courtesy of California Strong Motion Instrumentation Program).

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
SUPPLEMENTARY BRIDGE REPORT
DS-M19A (REV. 6 75)

Bridge No. 35-38

Location 04-SM-84-R
Dist - Co. - Rte - PM - City

10/17/89 EARTHQUAKE INSPECTION

Date of Investigation October 19, 1989

Name DUMBARTON BRIDGE

The bridge was checked on October 18, by driving across the structure and checking for misalignment, cracks and spalls at roadway level.

The bridge was inspected on October 19, by walking the steel span box girder (Spans 16 - 30). The only damage found was at the under side of the earthquake expansion joints at Pier 16 and Pier 31. These joints were designed to open and close up to 18 inches plus or minus under large earthquake loading with some damage. The damage is that the hold down bolt bolting the plate spanning the large gap have been pulled at top of the bolt sleeves resulting in the spalling about half of the slab thickness at each bolt.

The substructure, the concrete approach spans and the approach trestle (slab) spans were inspected from the adjacent fishing piers and from a State maintenance boat. No damage was apparent.

RECOMMENDATION

Repair the concrete deck spalls under the deck plate at expansion joints at Piers 16 and 31. 40039X89292X R-(H4221) $10,000

Richard W. White, P.E. RWW
C16762

by
Robert E. Keim

RWW/REK/art
cc: Dist. 04 (3)
coders

Figure 262.—Post-earthquake inspection report for the Dumbarton Bridge.

Table 22.—*Peak ground motion for Dumbarton Bridge*

Channel	Location	Orientation	Acceleration (g)	Velocity (in/s)	Displacement (in)
1	West Free Field	357°	.126	7.52	1.95
2	West Free Field	Up	.058	2.33	0.78
3	West Free Field	267°	.128	7.56	2.92

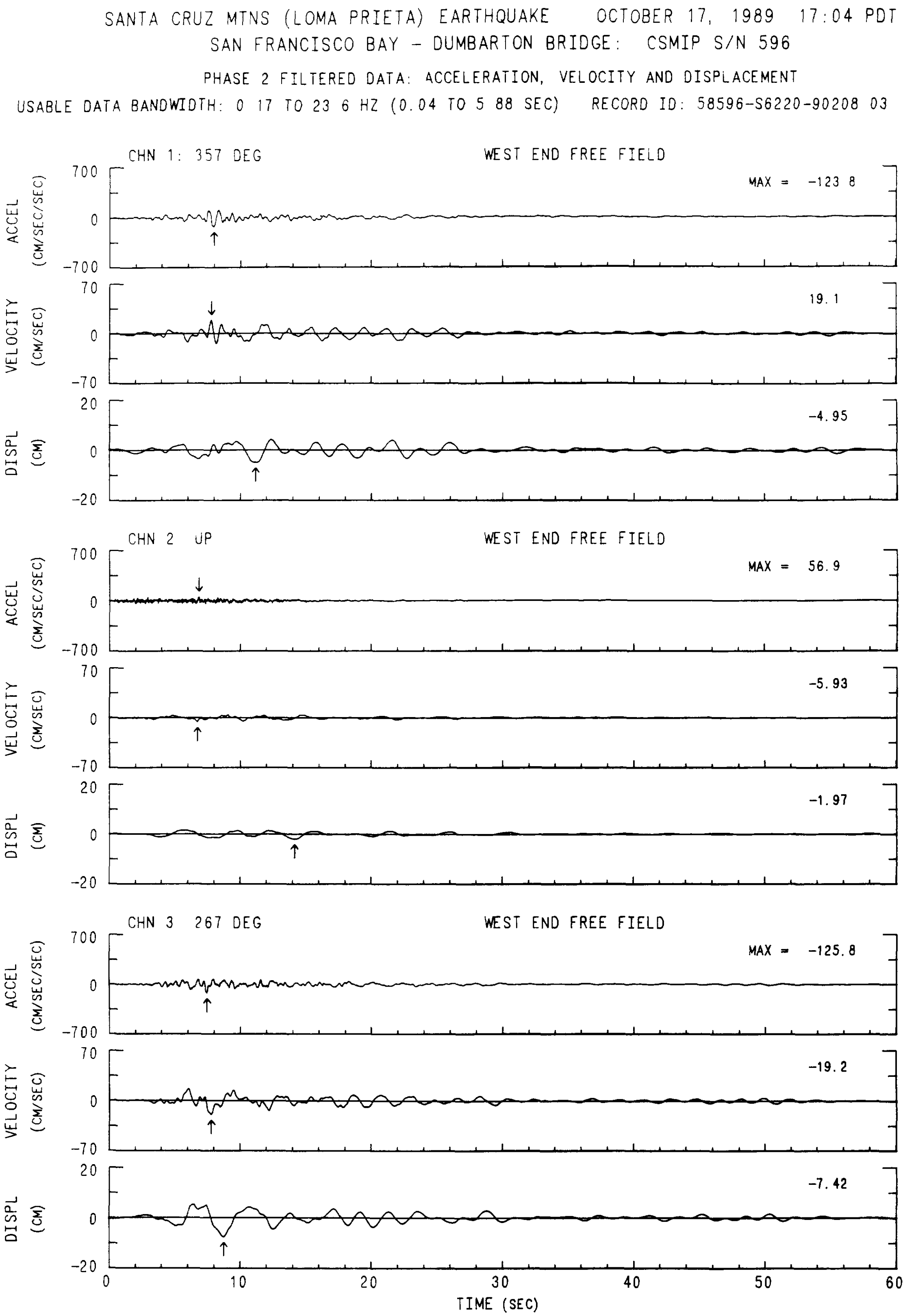


Figure 263.—Dumbarton Bridge free-field motions (courtesy of California Strong Motion Instrumentation Program).

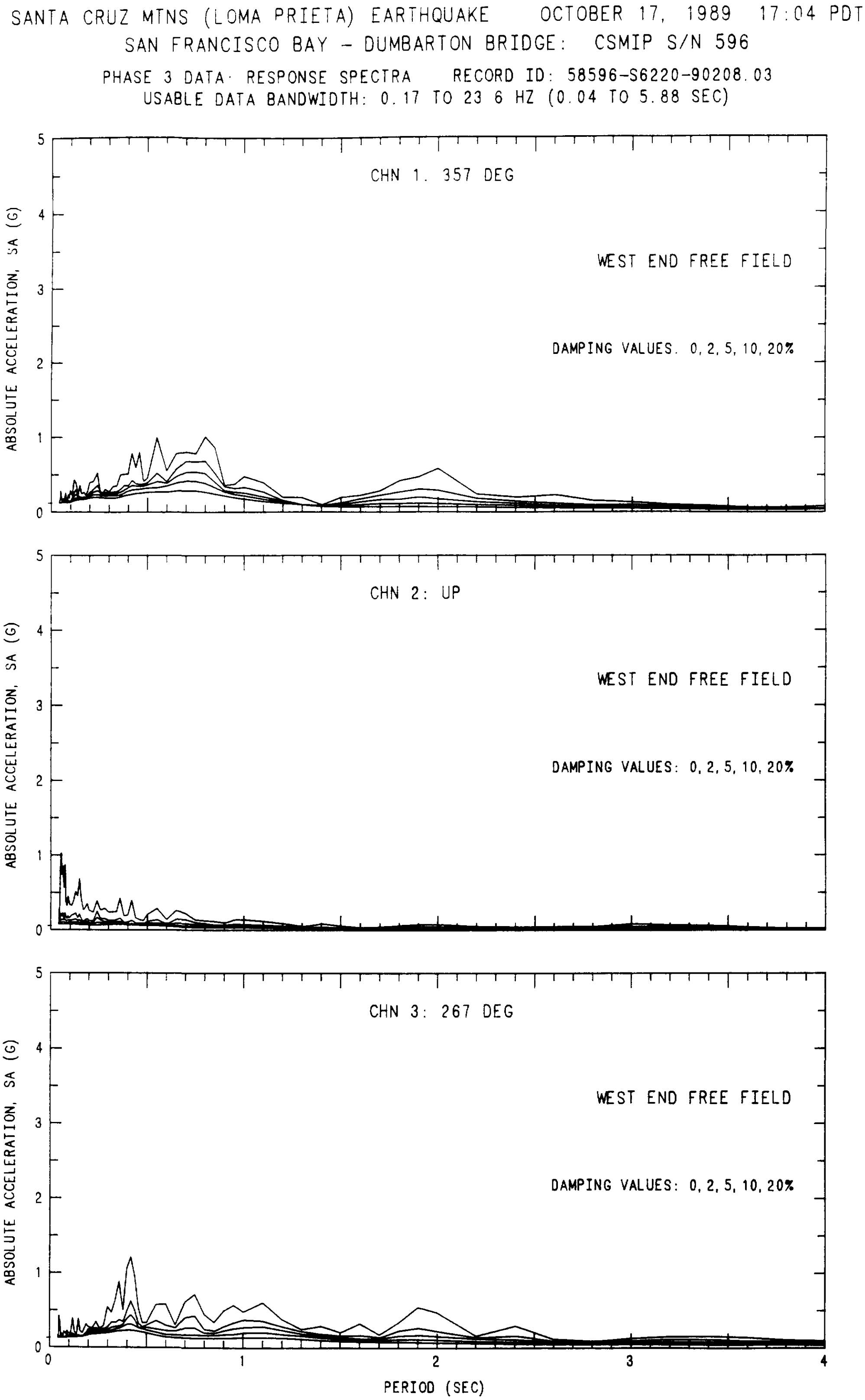


Figure 264.—Free-field response spectra for Dumbarton Bridge (courtesy of California Strong Motion Instrumentation Program).

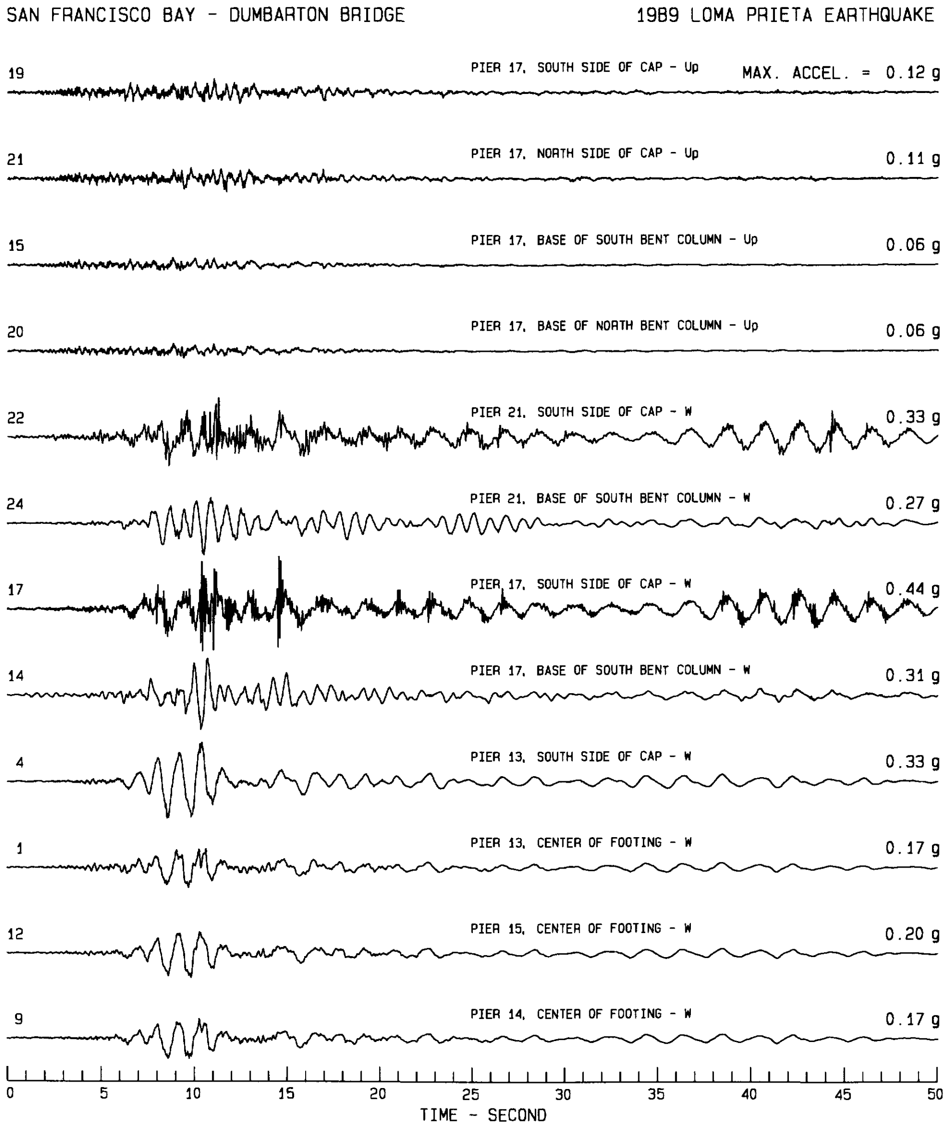


Figure 265.—Acceleration records for portion of Dumbarton Bridge (courtesy of California Strong Motion Instrumentation Program).

Table 23.—*Peak structural motion for Dumbarton Bridge (courtesy of California's Strong Motion Instrumentation Program)*

Channel	Location	Orientation	Acceleration (g)	Velocity (in.s)	Displacement (in)
1	Pier l3, Footing	West	0.167	12.6	4.13
2	Pier 13, Footing	Up	0.077	2.48	0.72
3	Pier 13, Footing	North	0.077	7.01	2.20
4	Pier 13, Cap	West	0.327	26.1	6.34
5	Pier 13, Cap	South	0.097	8.23	2.47
6	Pier 14, Cap	South	0.095	9.41	2.52
7	Pier 15, Footing	North	0.103	7.87	2.48
8	Pier 15, Cap	South	0.103	9.84	2.50
9	Pier 14, Footing	West	0.172	13.9	4.45
10	Pier 14, Footing	Up	0.070	2.11	0.69
11	Pier 14, Footing	North	0.100	7.56	2.43
12	Pier 15, Footing	West	0.205	16.7	4.61
13	Pier 15, Footing	Up		no record	

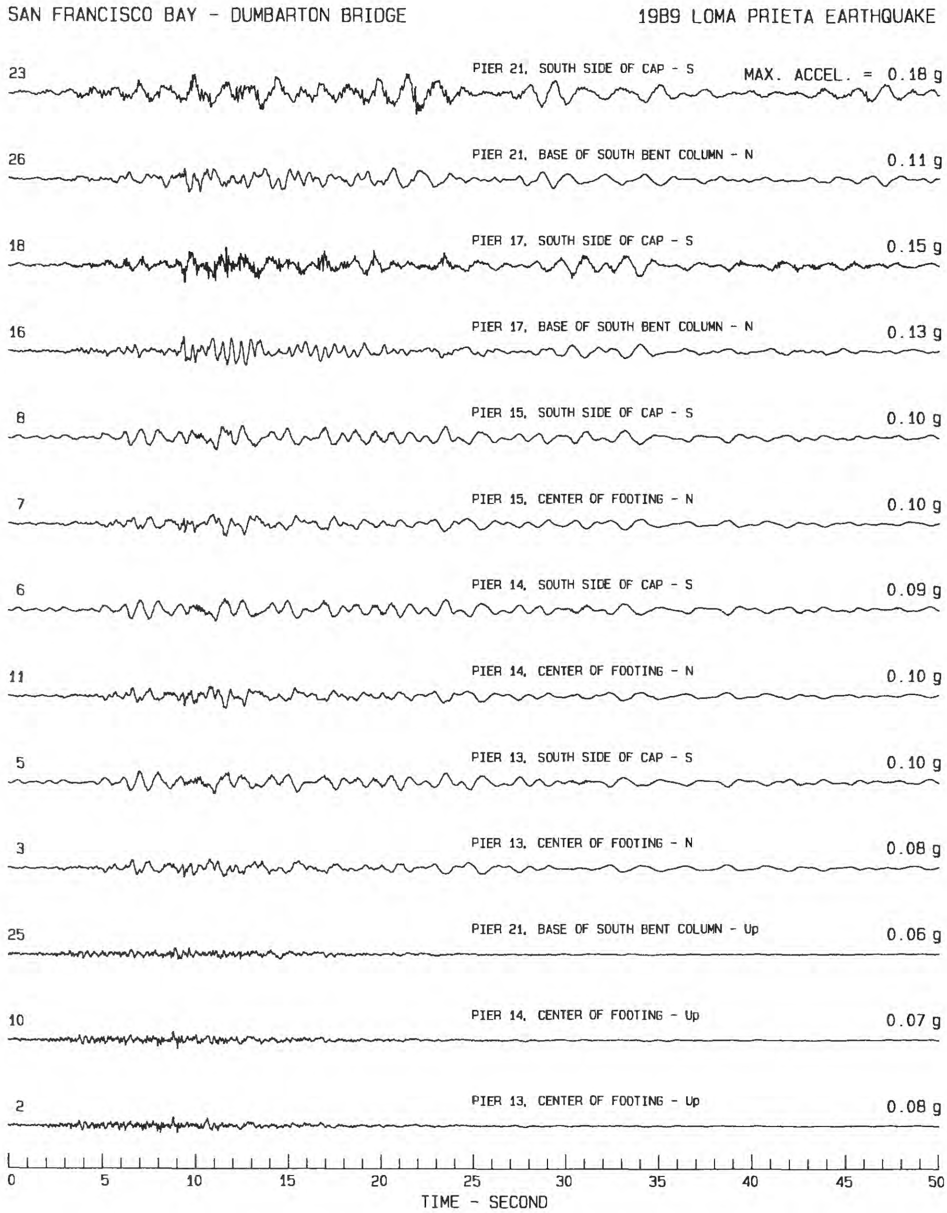


Figure 266.—Acceleration records for remainder of Dumbarton Bridge (courtesy of California Strong Motion Instrumentation Program).

Table 23.—*Peak structural motion for Dumbarton Bridge (courtesy of California's Strong Motion Instrumentation Program)*—Continued

Channel	Location	Orientation	Acceleration (g)	Velocity (in.s)	Displacement (in)
14	Pier 17, S. Column Base	West	0.307	15.9	3.77
15	Pier 17, S. Column Base	Up	0.064	2.65	0.84
16	Pier 17, S. Column Base	North	0.126	8.54	2.34
17	Pier 17, Cap	West	0.442	14.0	5.94
18	Pier 17, Cap	South	0.155	8.43	2.49
19	Pier 17, Cap	Up	0.115	3.89	0.93
20	Pier 17, No. Column Base	Up	0.061	2.46	0.79
21	Pier 17, No. Cap	Up	0.118	3.73	0.85
22	Pier 21, Cap	West	0.327	14.9	5.47
23	Pier 21, Cap	South	0.176	13.2	3.58
24	Pier 21, So. Column Base	West	0.266	13.3	3.61
25	Pier 21, So. Col. Base	Up	0.057	2.49	0.91
26	Pier 21, So. Column Base	North	0.107	8.98	2.47

HAYWARD BAY AREA RAPID TRANSIT ELEVATED STRUCTURE

DESCRIPTION OF BRIDGE

The instrumented portion of the Hayward Bay Area Rapid Transit Viaduct is a three-span portion of a long, elevated structure composed of single column bents, simply supported prestressed box girders, and continuous-steel rails (figs. 267 to 270). The columns are of hexagonal section with 5 feet between opposite faces, with two rings of #18 bars and an outer wrap of #5 spiral rebar at a 3-inch pitch (fig. 271). The columns sit on square footings with 16 to 18 1-foot-diameter reinforced concrete piles. The piles are 40 to 50 feet long, and all the outer piles are battered. Figure 272 shows the piles and the soil profile based on three soil borings taken at the site. Ground water was found at about 60 feet below the surface. The girders are attached on the north end by 5-inch-diameter concrete-filled pipes and sit on elastomeric bearings at the south end (fig. 273).

BRIDGE MOTION

This bridge suffered no damage from the earthquake. It was designed in 1980's using new seismic criteria. The bridge was instrumented with 16 sensors by California's Strong Motion Instrumentation Program (fig. 270). Three additional sensors were installed in a nearby parking lot to capture the free-field motion. The bridge motions are shown in figures 274 to 276. An analysis was done to study the bridge's performance for the earthquake and for the maximum event (Penzien, 1991).

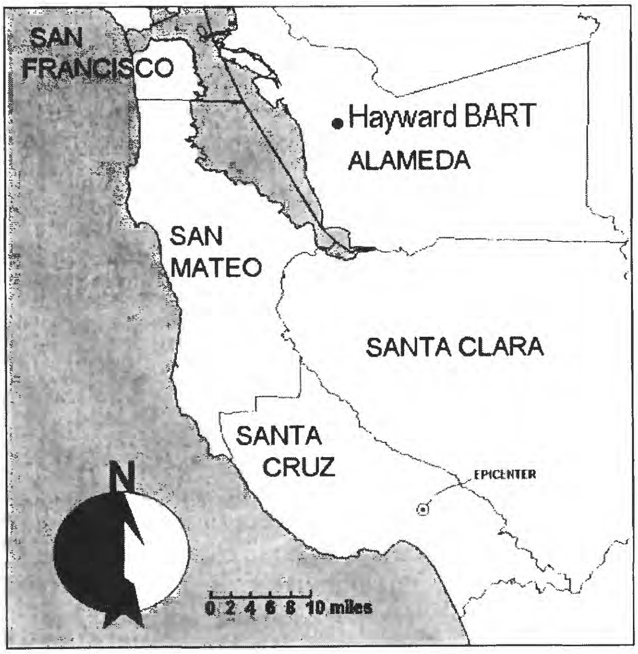


Figure 268.—Location of Hayward BART Viaduct.

Figure 267.—Instrumented portion of Hayward BART Viaduct (courtesy of California Strong Motion Instrumentation Program).

Bay Area Rapid Transit Structure

Approximate Latitude & Longitude
N. Lat. 37.671° W. Long. 122.087°

Epicentral Distance
46 miles

Peak Ground Acceleration	N/S	U/D	E/W
Parking Lot at bridge	0.15	0.05	0.15

Length	Width	Skew	Yr Blt	YrRet
3 span	26'	none	1967	none

Main Span Type
Prestressed concrete box girders

Main Substructure Type
Single column reinforced concrete bents on piles

Figure 269.—Aerial view of the Hayward Bart Viaduct and instrumented section.

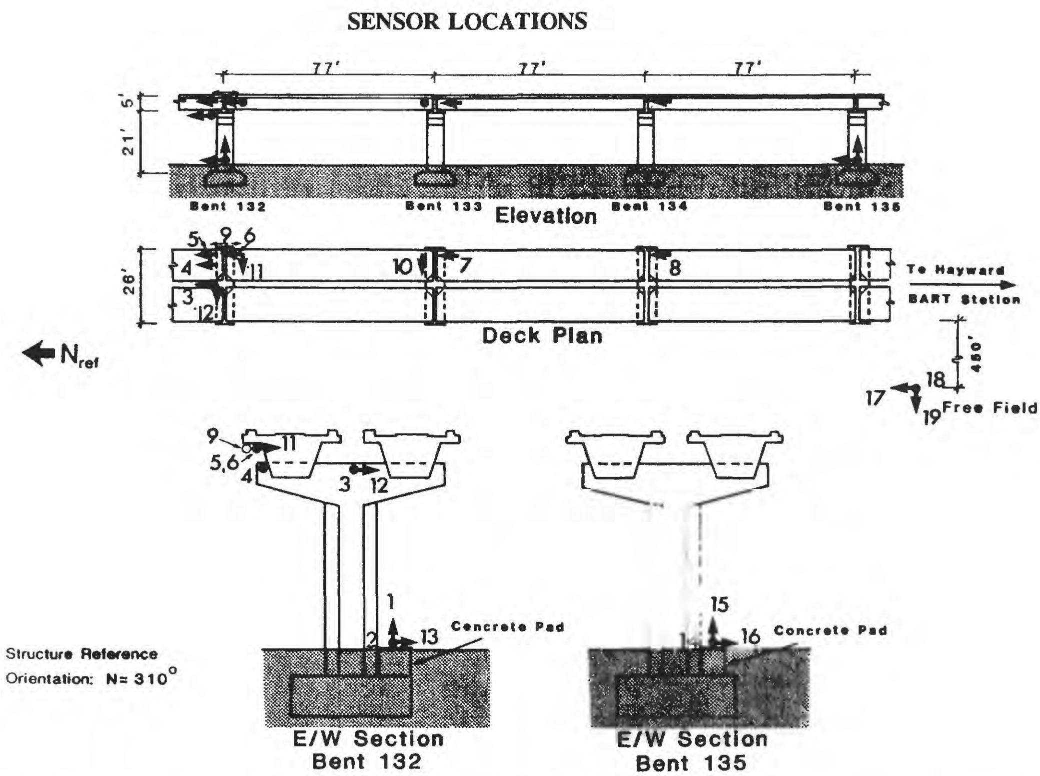


Figure 270.—Plan, elevation, typical section, and instrumentation for Hayward BART Viaduct (courtesy of California Strong Motion Instrumentation Program).

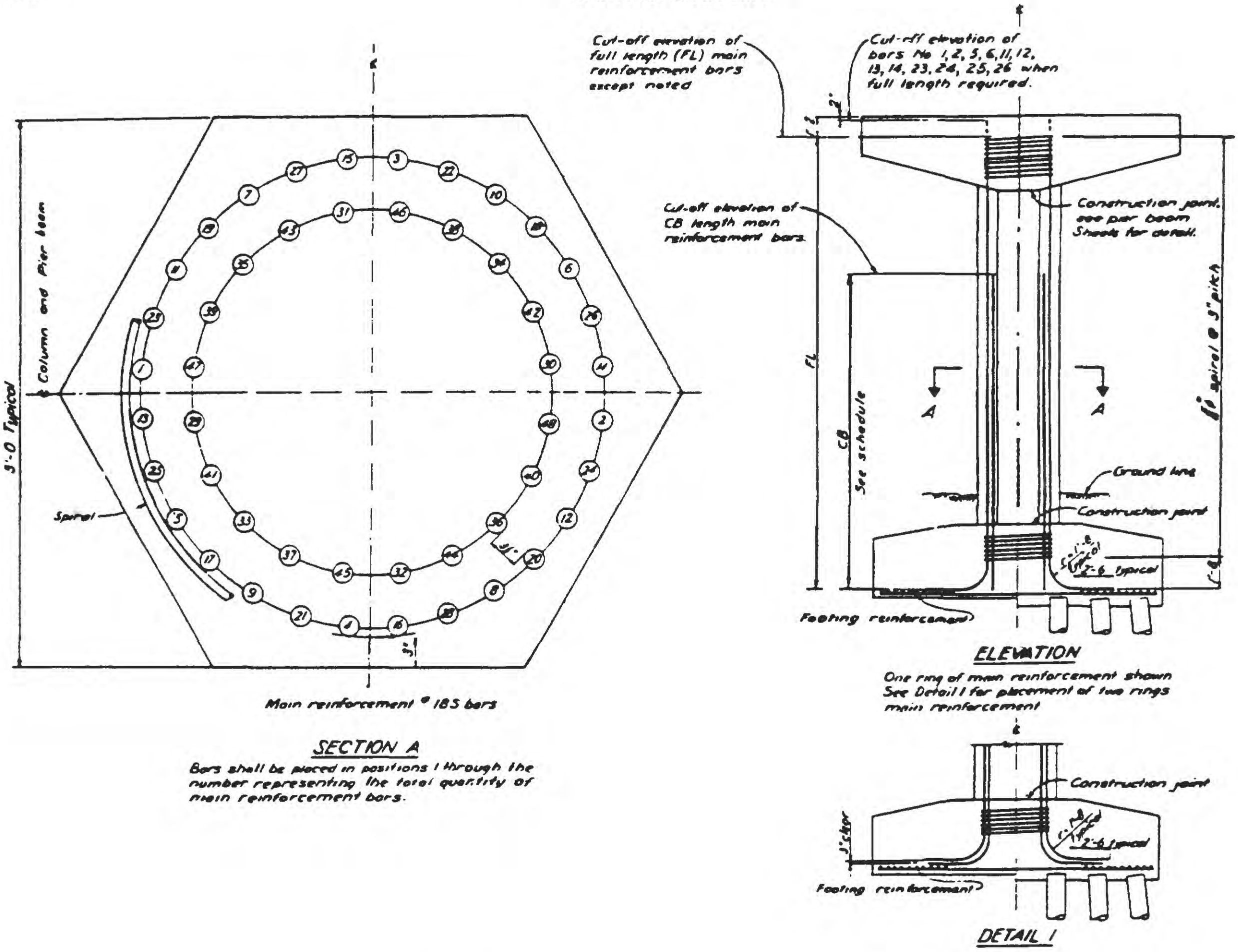


Figure 271.—Column details for Hayward BART Viaduct (courtesy of Joseph Penzien).

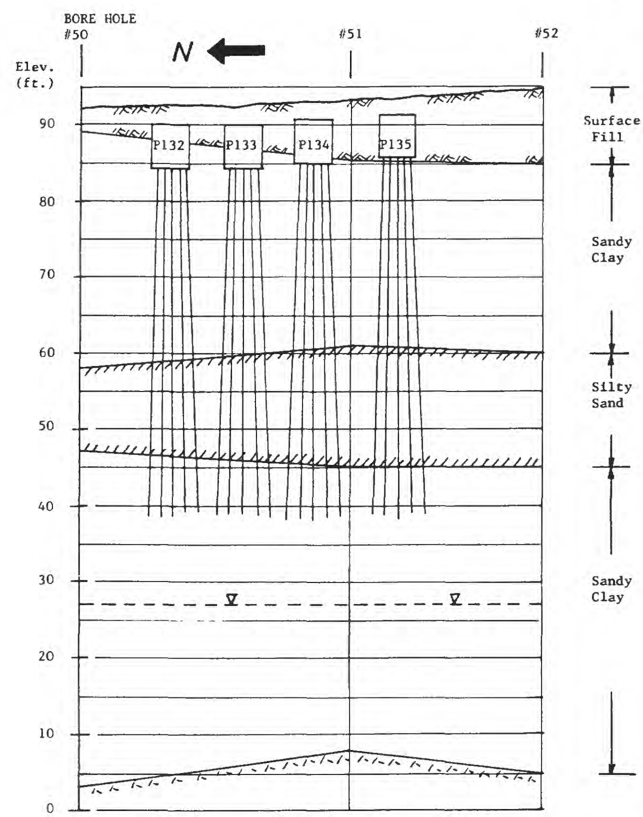


Figure 272.—Soil profile at Hayward BART Viaduct (courtesy of Joseph Penzien).

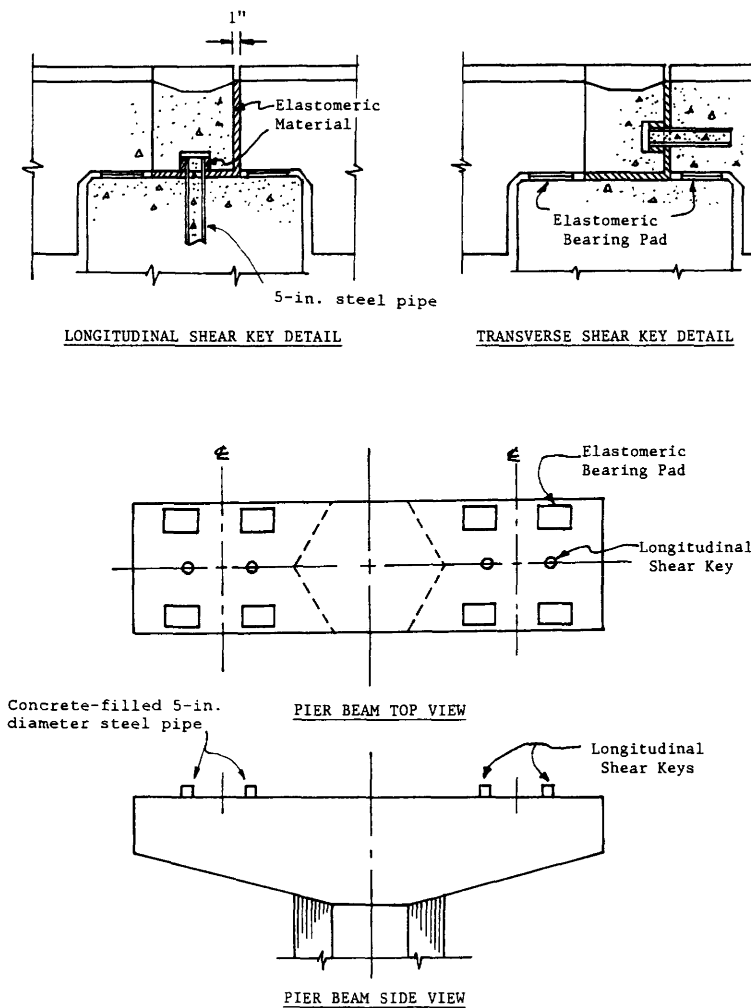


Figure 273.—Bearings on Hayward BART Viaduct (courtesy of Joseph Penzien).

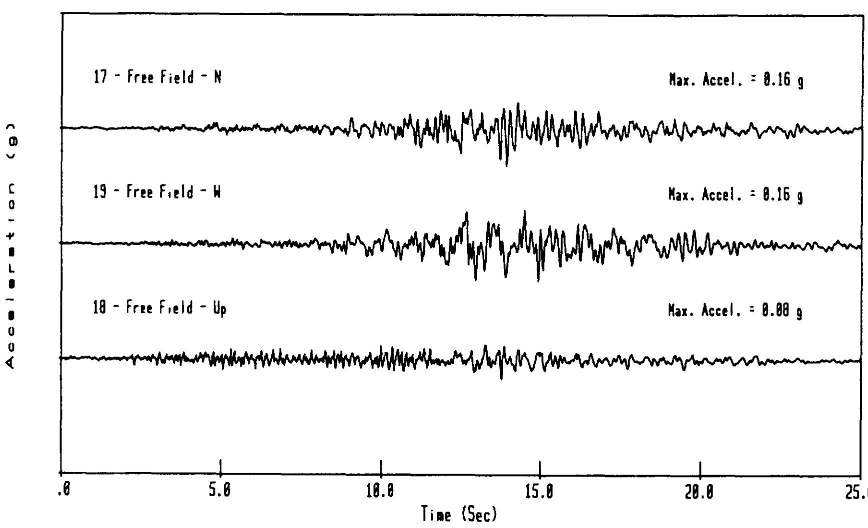


Figure 274.—Free-field ground motions for Hayward BART Viaduct (courtesy of California Strong Motion Instrumentation Program).

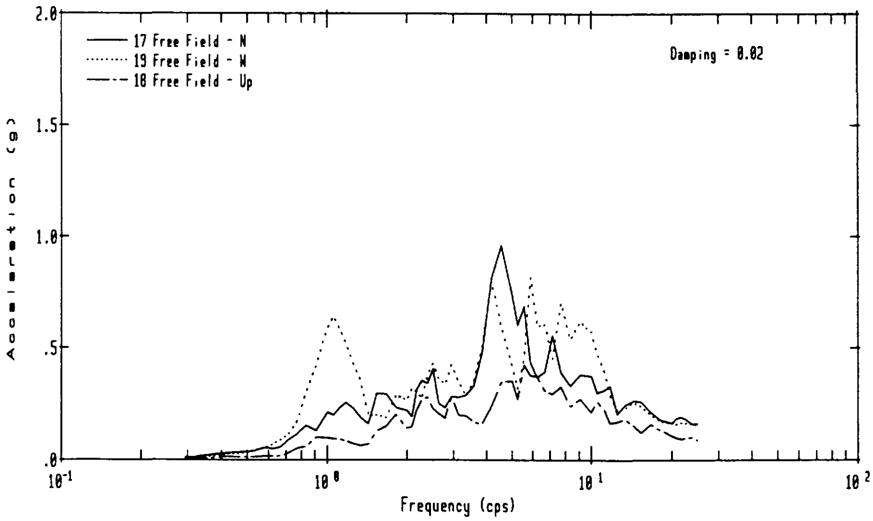


Figure 275.—Free-field response spectra for Hayward BART Viaduct (courtesy of California Strong Motion Instrumentation Program).

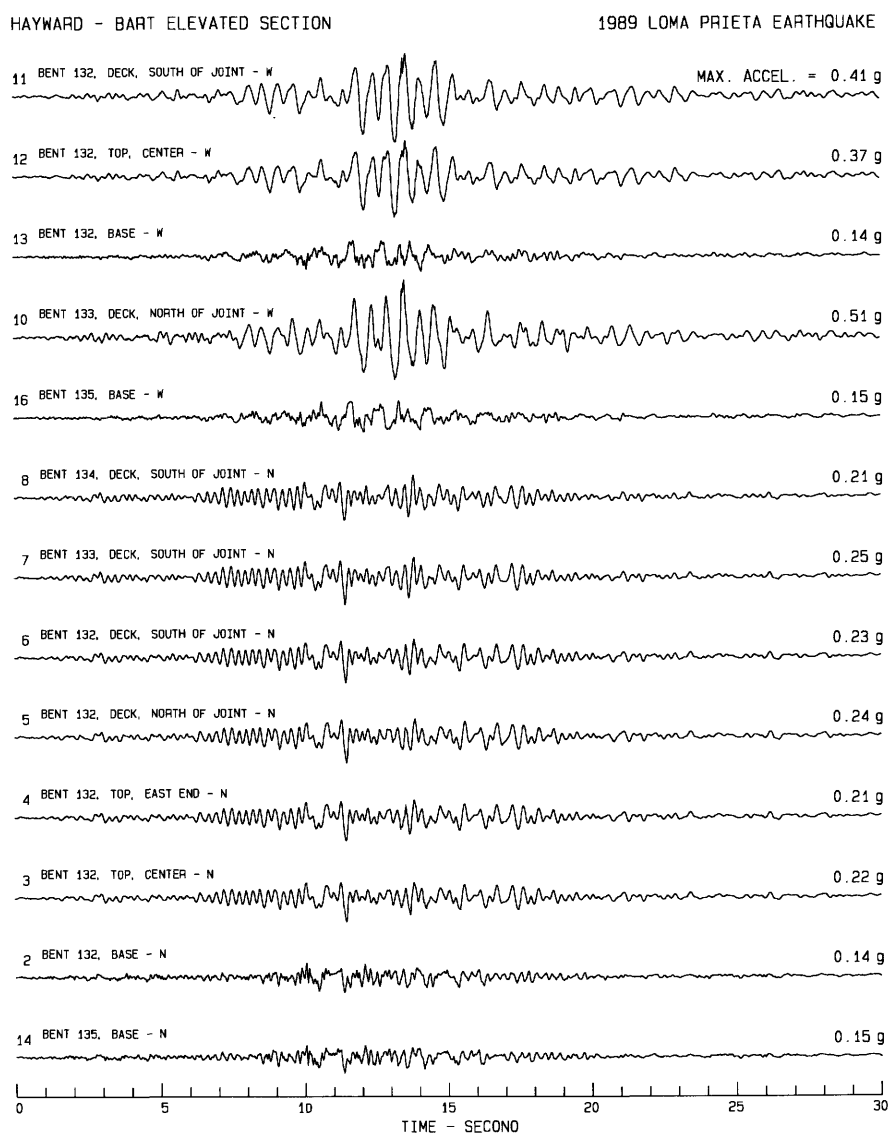


Figure 276.—Acceleration records for Hayward BART Viaduct (courtesy of California Strong Motion Instrumentation Program).

Figure 277.—Route 156/101 Separation (courtesy of California Strong Motion Instrumentation Program).

Bridge #44-107 / Rte 156 / Post Mile 95.44

Approximate Latitude & Longitude
N. Lat. 36.862° W. Long. 121.578°

Epicentral Distance	**Geology**
21 miles	Alluvium

Peak Ground Acceleration N/S U/D E/W
Hollister South & Pine 0.18 0.20
0.38

Length	**Width**	**Skew**	**Yr Blt**	**YrRet**
326'	35.5'	35°	1959	None

Main Span Type
Six span, steel girder bridge

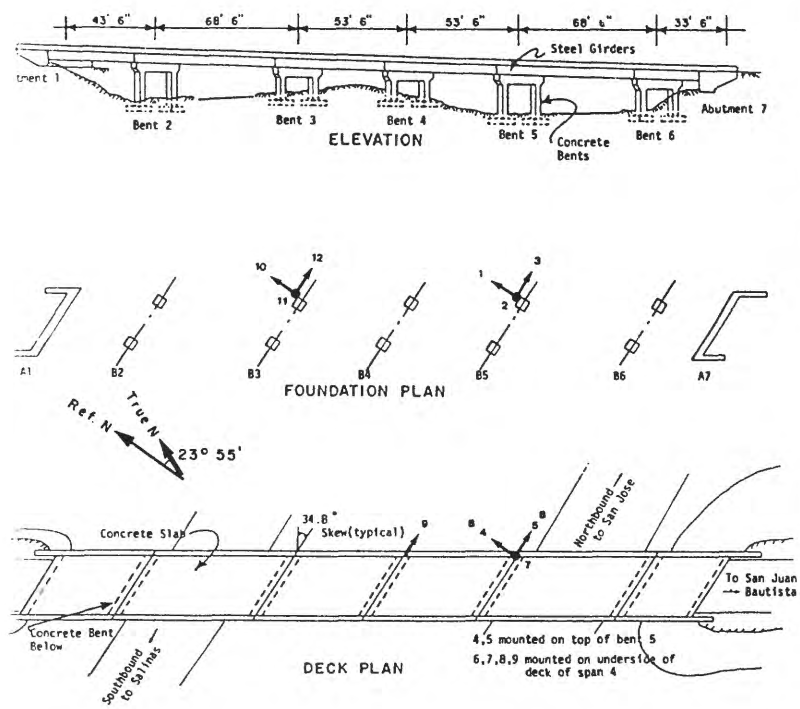
Main Substructure Type
Two column reinforced concrete bents on spread footings

SAN JUAN BAUTISTA-ROUTE 156/101 SEPARATION

DESCRIPTION OF BRIDGE

The Route 156/101 Separation is an older, steel composite bridge on two-column bents. It carries two lanes of traffic over Highway 101 near the town of San Juan Bautista (fig. 277).

SENSOR LOCATIONS



BRIDGE MOTION

This bridge was instrumented and recorded movement during the earthquake. Figure 278 shows instruments on Bents 3, 4, and 5. Figure 279 provides acceleration time histories at these locations. Station 8 (at the top of the right steel girder) recorded an acceleration of 0.50 g in the longitudinal direction. Unfortunately, there was not a free-field instrument nearby to study the input motion. However, two foundation-level instruments were used to study the bridge's behavior in a report written for California's Strong Motion Instrumentation Program (Kasai 1995). Moreover, during the 1979 Coyote Lake earthquake, a free-field recorder was located at a gas station across from the bridge. A doctoral thesis was written on this bridge's behavior during the Coyote Lake earthquake using strong motion records (Wilson, 1986). This bridge was the closest instrumented structure to the epicenter of the earthquake, but there was no damage.

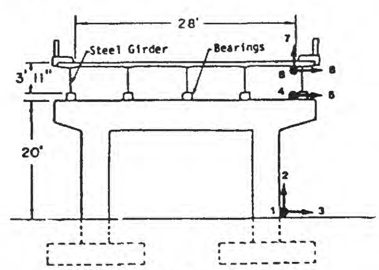


Figure 278.—Plans and instrument location for Route 156/101 Separation (courtesy of California Strong Motion Instrumentation Program).

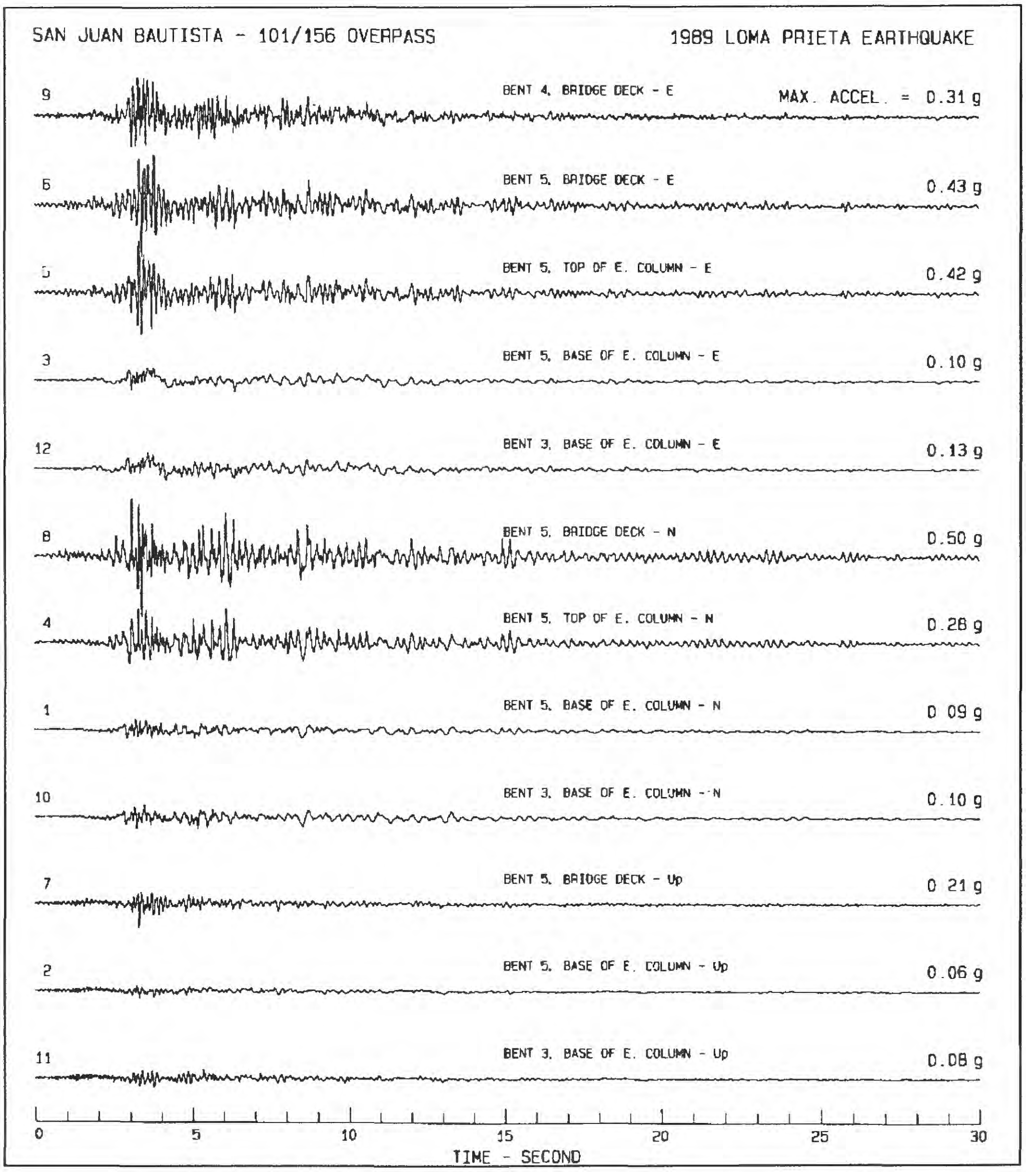
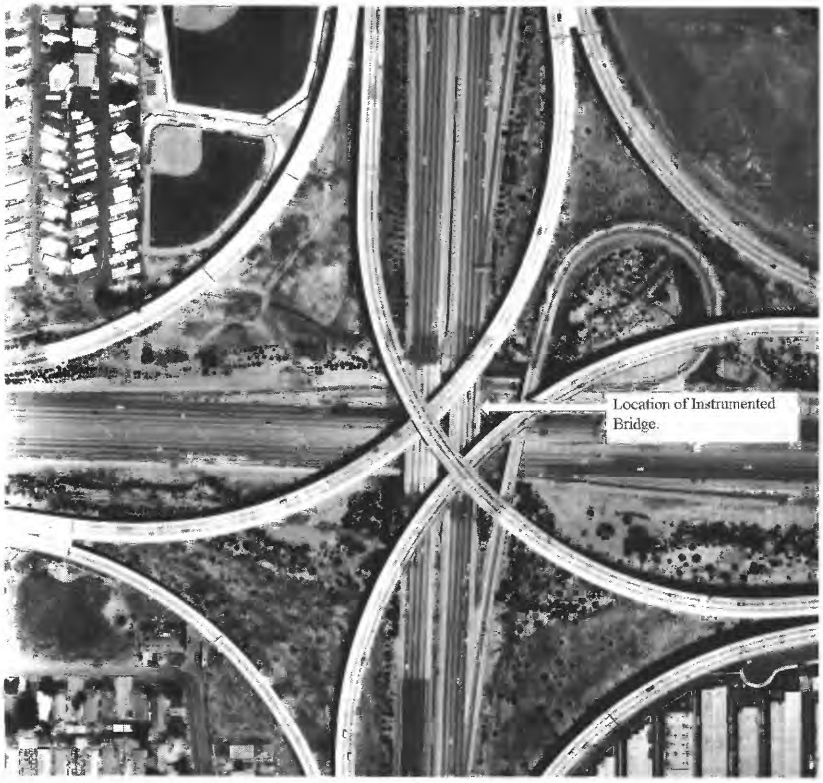


Figure 279.—Strong motion records for Route 156/101 Separation (courtesy of California Strong Motion Instrumentation Program).



Bridge #37-285F / Rte 280 / Post Mile 0.00

Approximate Latitude & Longitude
N. Lat. 37.340° W. Long. 121.851°

Epicentral Distance	Geology
21 miles	Alluvium

Peak Ground Acceleration	N/S	U/D	E/W
CSMIP Sta. 57356	0.07	0.08	0.09

Length	Width	Skew	Year Built	Yr Retrofit
276'	28'	None	1974	None

Main Span Type
Three span prestressed cast-in-place box

Main Substructure Type
Two column reinforced concrete bents on pile footings

Figure 280.—Aerial view of Route 280-680/101 Separation.

ROUTE 280-680/101 SEPARATION

DESCRIPTION OF BRIDGE

This is a fairly typical Caltrans' bridge (fig. 280). It is supported by end-diaphragm abutments that rest on footing seats with vertical pins. The columns have a large flare near the top and are cast monolithically with the deck but are pinned at the pile foundations (fig. 281). The bridge rests on deep alluvium, and the piles are driven into good material, as shown in the soil profile (fig. 283).

BRIDGE MOTION

Three instruments in the superstructure near Bent 2 recorded the bridge's motion during the earthquake (fig. 282). The free-field instruments for this structure, like the Route 156/101 Separation Structure just described, were not working during Loma Prieta. This limits the usefulness of these records. No analysis was done of this bridge after Loma Prieta. The ground records (from a nearby building) and the bridge records (fig. 284) are both surprisingly low.

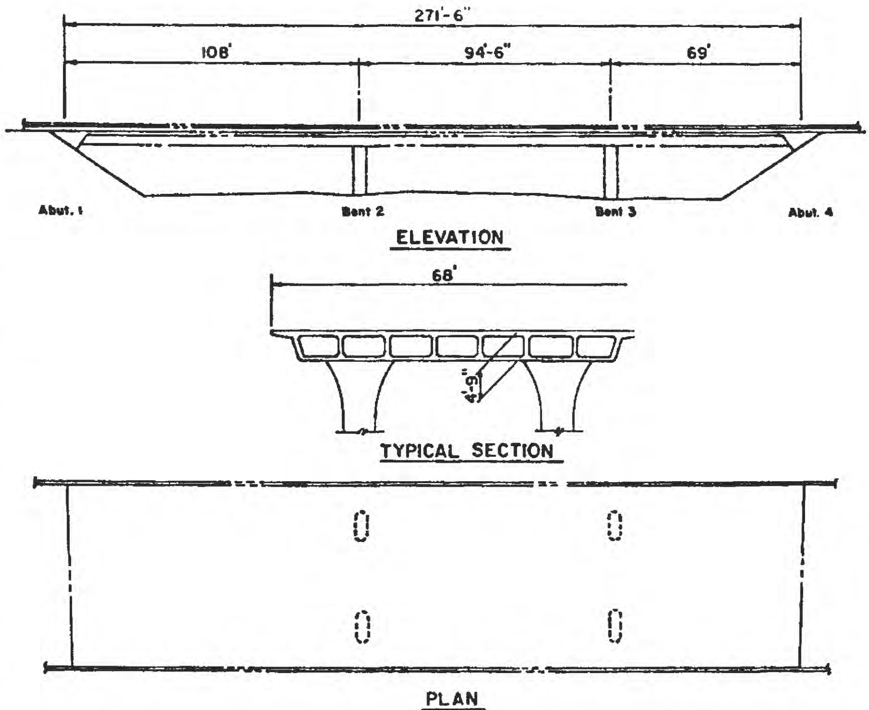


Figure 281.—Plan, elevation, and typical section of Route 289-680/101 Separation.

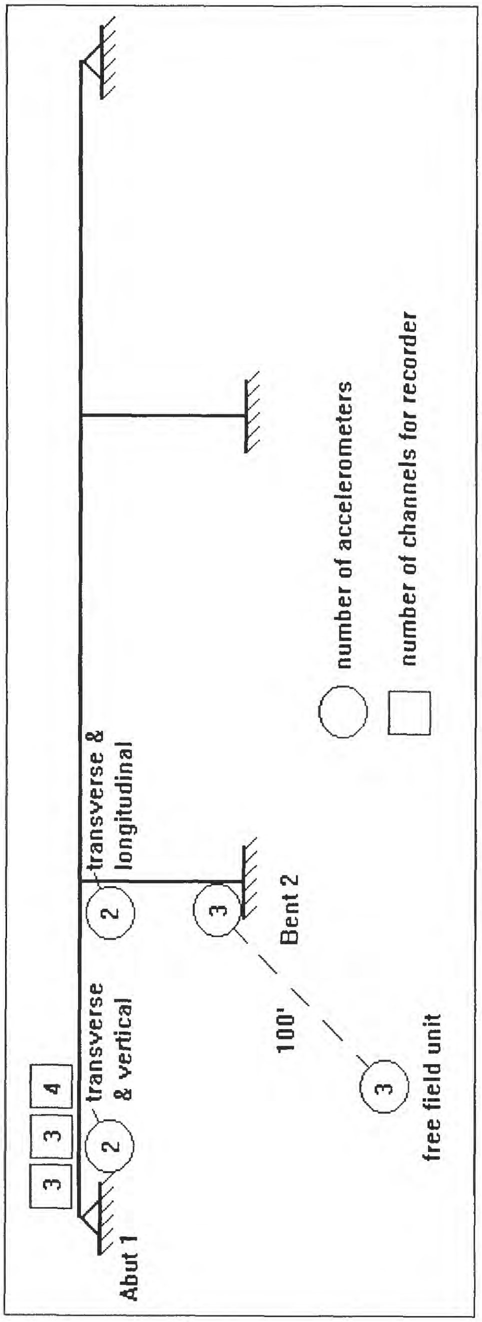


Figure 282. —Location of instruments for Route 280-680/101 Separation (courtesy of U.S. Geological Survey).

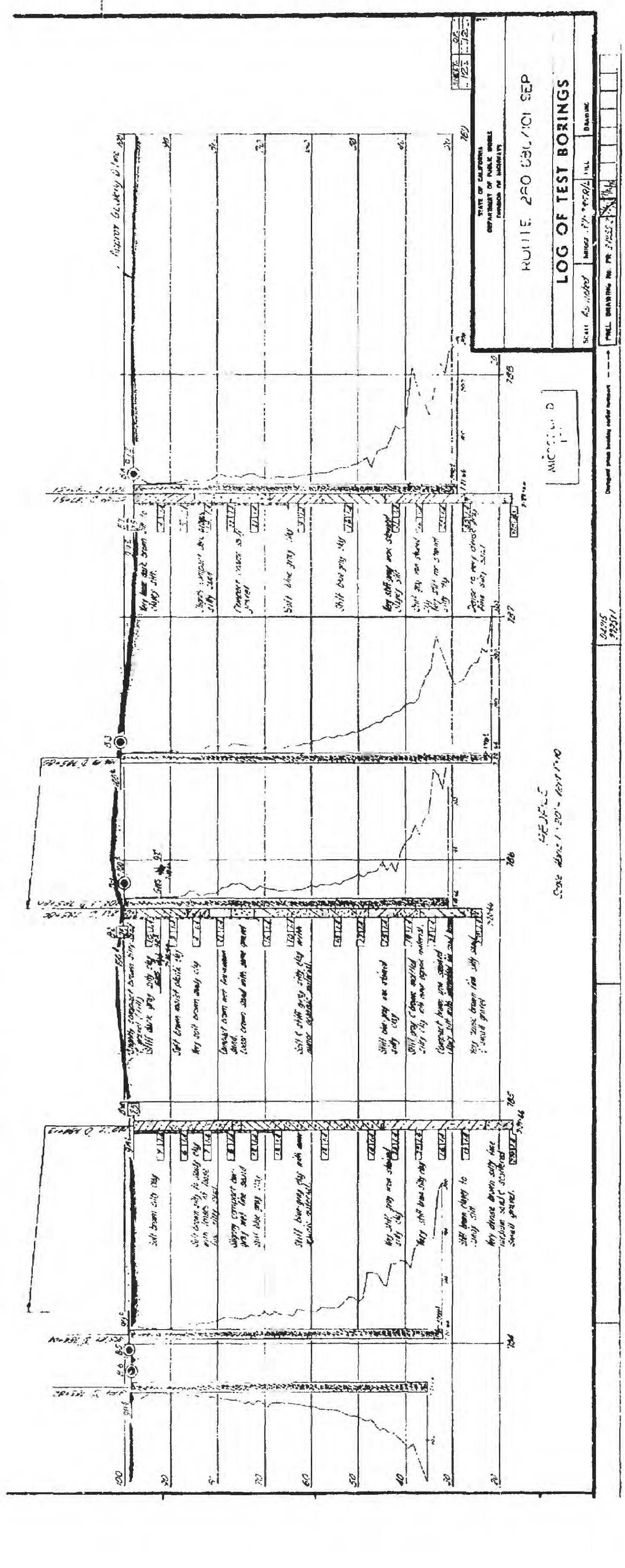


Figure 283.—Soil profile for Route 280-680/101 Separation.

U.S. STRONG-MOTION NETWORK	DIRECTION	CONSTANTS	MAX. ACCELERATION
Station No. 1571 37.340N, 121.851W San Jose 101/280/680 Fwy Interchange	322°	Sens. = 1.63 cm/g Freq. = 26.3 Hz Damp. = 0.6 crit	0.18g
SMA-1 No. 288 USGS/CDOT (Bridge) EARTHQUAKE OF	Up	Sens. = 1.84 cm/g Freq. = 26.3 Hz Damp. = 0.6 crit	0.08g
18 October 1989 - 0004 G.m.t.	232°	Sens. = 1.81 cm/g Freq. = 25.6 Hz Damp. = 0.6 crit	0.13g

Film speed = 1 cm/sec

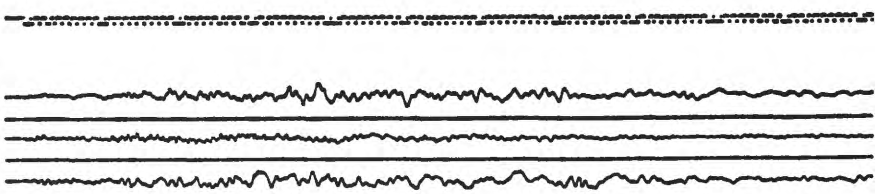


Figure 284.—Acceleration records for Route 280-680/101 Separation (courtesy of U.S. Geological Survey).

Figure 285.—Aerial view of Golden Gate Bridge.

GOLDEN GATE BRIDGE

This bridge (figs. 285 and 286) is owned and operated by the Golden Gate Bridge, Highway, and Transportation District. During the earthquake an instrument in the basement of the Administration Building of the Toll Plaza recorded the ground motion (fig. 287). There was a maximum horizontal acceleration of 0.24 g and no damage to any portion of the bridge. However, concern about a much larger earthquake has caused the District to begin a massive retrofit program for the bridge.

Route 101 between Marin and San Francisco Counties.

Approximate Latitude & Longitude
N. Lat. 37.806° W. Long. 122.472°

Epicentral Distance	Geology
62 miles	Alluvium

Peak Ground Acceleration	N/S	U/D	E/W
Administration Building	0.12	0.06	0.24

Length	Width	Skew	Year Built	Year Retrofit
8450'	90'	None	1937	None

Main Span Type
Suspension bridge

Main Substructure Type
Two 700' steel towers

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Figure 286.—Golden Gate Bridge (courtesy of the Golden Gate Bridge, Highway, and Transportation District).

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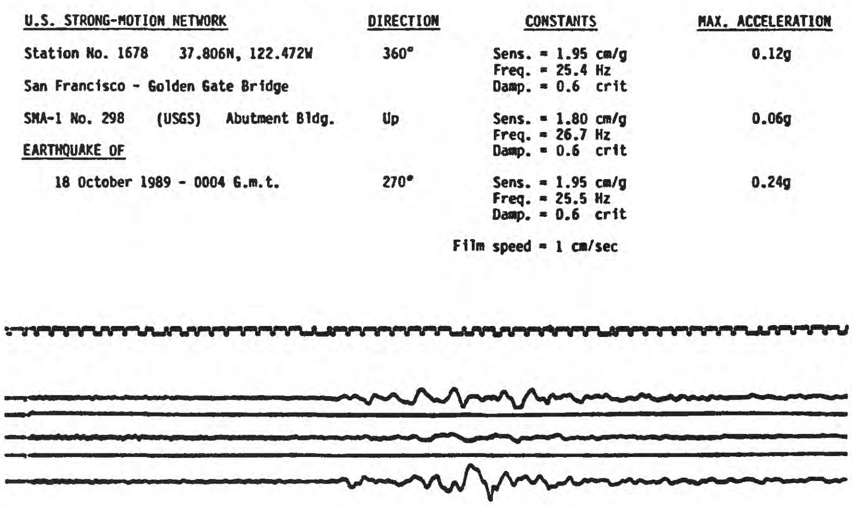

Figure 287.—Earthquake records of Golden Gate Bridge (courtesy of U.S. Geological Survey).

HIGHWAY DAMAGE

There was a great deal of damage to highways from landslides, liquefaction, and ground deformation that occurred as a result of the earthquake. Although there was no surface rupture, there were surface deformations that caused substantial damage to roadways. This section describes the performance of streets, roads, and highway structures other than bridges during the earthquake.

The area of road damage was quite extensive. Rock and land slides occurred in the Santa Cruz Mountains close to the fault and for 70 miles northward. Liquefaction-induced road damage was similarly spread out from Monterey County south of the fault to the City of San Francisco and Highway 80 about 60 miles north of the fault. The one constant was that wherever road damage occurred, poor soils were usually present. Although there were many damaged roads, the cost of bridge damage far exceeded road damage, owing to the heavy cost of repairing or replacing the double-deck bridges.

Roadways, along with culverts and retaining walls, sustained more damage from soil movement and ground deformation, than from shaking. In general, landslides were more damaging in areas of cuts, while settlement was more damaging in areas of fill. Caltrans currently does not design roads, culverts, and retaining walls for a seismic load, nor does doing so seem appropriate from what was seen after the earthquake.

There has been some speculation that earth-retaining structures, such as abutments and retaining walls, should be designed for a much larger earthquake force. However, Caltrans has not seen any damage from Loma Prieta or any other strong earthquake that would justify designing earth-retaining structures for an additional seismic load. There was no damage to any Caltrans retaining wall as a result of Loma Prieta. There were some signs of movement to locally owned retaining structures.

California has always had a problem with landslides on roads along its coast and mountain passes. Slide removal is routinely done to keep some roads open. Heavy rains frequently cause slides by increasing the overburden and reducing soil friction. This situation is exacerbated, not created, by earthquakes like Loma Prieta.

Highway damage from settlement, lateral spreading, and soil liquefaction are just beginning to be addressed (mostly in Japan) by soil modification before the facility is built. Roads built on well-engineered material suffered little damage during the earthquake.

It would appear that research and study into highway damage was not as intensive as research into bridge damage after the earthquake. However, geotechnical studies, particularly on the seismic behavior of weak clays and loose alluvium, have intensified as a result of the earthquake. Table 24 and figure 288 summarize state highway damage from the earthquake. The source for much of the information on highway damage comes from Heyes (1990). This is an excellent source of information on the earthquake.

There was also substantial damage to city and county streets and roads. Table 25 lists all local roads that were repaired with Federal emergency funds. Other information on local road damage comes from reports by city and county engineers, as well as researchers and geotechnical engineers that studied the damage. Not only city and county roads, but roads owned by state parks and beaches, roads owned by military bases, and private roads were impacted by the earthquake. Some of that damage is listed in table 26. These three tables, although not a complete listing, contain most of the major road damage that occurred.

Table 24.—*Damage to State highways and related facilities from the earthquake.*

Location	Damage
Route 17 in Santa Cruz and Santa Clara Counties	Many landslides and rockslides. Roadway damage from soil settlement. Ground surface rupture caused a vertical uplift of the road near the top of Santa Cruz Mountains.
Route 9 in Santa Cruz County	Moderate slide closed this road for several days.
Route 152 at Heckler Pass in Santa Cruz County	Rockslides, highway cracks, and settlement.
Route 236 in Santa Cruz County	Few small slides closed this road after earthquake.
Route 35 in Santa Clara County	Road was closed for 2 days while 6 small slides were removed.
Route 101 near Gilroy in Santa Clara County	Median barrier railing was badly cracked by Sargeant Overpass.
Route 280 in Santa Clara County	Soil heaved damaging roadway in fill area near Magdalena Avenue.
Route 1 and Mountain Lake in San Francisco	Liquefaction of fine-grained foundation soils severely damaged 300 feet of roadway. Roadway had to be repaired several times before settlement was controlled.
Route 1 in Marin County	Lone Tree landslide was accelerated by earthquake.

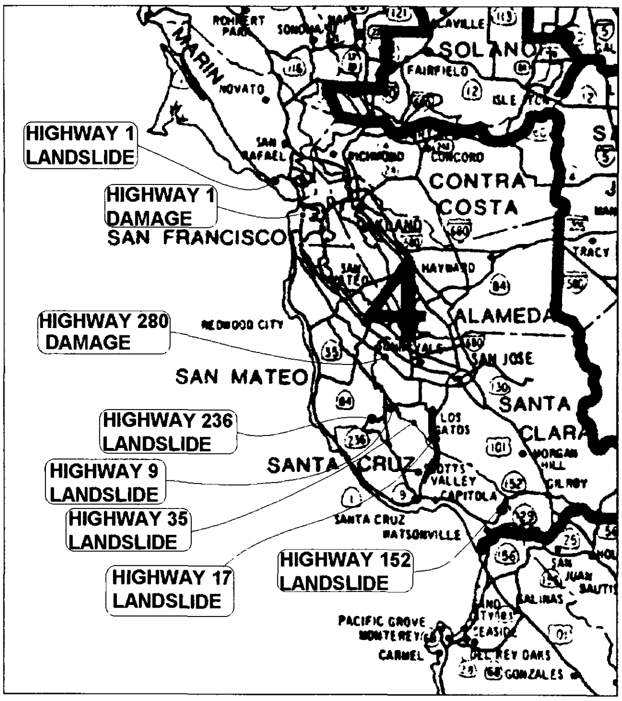

Figure 288.—Location of State highway damage.

Table 25.—Federal emergency relief funding of local roads

Road's Damaged	Location	Cost
Cervantes Street	San Francisco	194,374
Bay Street	San Francisco	67,792
Marina Blvd.	San Francisco	452,242
4th Street	San Francisco	67,233
Laguna Street	San Francisco	12,960
6th Street	San Francisco	435,792
5th Street	San Francisco	275,077
Mission Street	San Francisco	161,897
Folsom & Embarcadero	San Francisco	19,383
Folsom Street	San Francisco	321,532
South Van Ness	San Francisco	20,228
Valencia Street	San Francisco	377
14th Street	San Francisco	134,116
7th Street	San Francisco	546,768
Townsend Street	San Francisco	29,852
Hyde Street	San Francisco	3,115
Grove Street	San Francisco	89,871
17th Street	San Francisco	27.276
Sacramento Street	San Francisco	66.320
Washington Street	San Francisco	17,224
Pine Street	San Francisco	25,401
Stuart Street	San Francisco	19,400
Harrison Street	San Francisco	54,667
Howard Street	San Francisco	80,351
Market Street	San Francisco	39,300
Beach Park Blvd.	Foster City	10,920
Santa Theresa Blvd.	Santa Clara Co.	6,885
Soquel-San Jose	Santa Cruz Co.	2.358,371
Summit Road	Santa Cruz Co.	612,935
Capitola Drive	Santa Cruz Co.	74.998
Brommer Street	Santa Cruz Co.	12.497
East Cliff Dr	Santa Cruz Co.	63,473
Grahamhill Rd.	Santa Cruz Co.	39.093
Portola Road	Santa Cruz Co.	18.920
Valencia Road	Santa Cruz Co.	71,144
Freedom Blvd.	Watsonville	7,500
Pierce Road	Saratoga	44.027
Various Locations	Los Gatos	77,688
7th Street	Oakland	25,089
Cliff Drive	Capitola	59,579
Granite Creek Rd.	Scotts Valley	364.726
1-80 West Frontage	Emeryville	7,160
Industrial Blvd. & Laurel Street	San Carlos	21,125
California & Broadway	Burlingame	26,600
Bayshore & Rollin Rd.	Burlingame	26,600
Bear Valley Rd.	Marin Co.	4,555
Two Locations	Redwood City	15,064
West Frontage Rd.	Berkeley	213.100

Table 26.—*Damage to roads and tunnels not covered elsewhere*

Road	Damage
Jetty Road - Moss Landing State Beach in Monterey County	Liquefaction and lateral spreading of soil caused the roadway to cave in and damage to a corrugated metal culvert.
Treasure Island roadways	Road damage due to liquefaction at the Naval Base.
San Juan Watsonville Rd. south of Aromas Rd. in Monterey County	North side of roadway bank has slid 100 feet. Repair cost $4,000.
Cienega Rd. 0.3 miles south of Union Rd. in Santa Benito County	Pavement buckled. Repair cost, $0.
San Thomas Expwy. between Campbell and Hamilton in Santa Clara County	55 soundwall panels tilted. Repair cost, $47.016.
Industrial Blvd., Laurel St., Elm St., Cedar St., and Alameda Dr., City of San Carlos, San Mateo County	Some new culvert cracks noted. Repair costs, $23,660.
Broadway Street Tunnel between Mason and Hyde in San Francisco	Minor movement diaplaced some tiles. Cost, $0.

DAMAGE TO ROADS

Roads were damaged in a number of ways during this earthquake. Land and rock slides were prevalent throughout the mountains and along the coast. Liquefaction-induced settlement and lateral movement of roadways was common around San Francisco Bay where loose fills had been placed over the Bay mud. Close to the fault rupture, roads were damaged due to surface deformation of the ground. The location of road damage can be found on figure 288.

DAMAGE ON STATE HIGHWAY 17

The most significant road damage was on State Highway 17, which was closed for about 12 miles (fig. 289 to 291). This is at a location within a few miles of the epicenter of the earthquake. Some of the damage was rock and land slides that blocked the highway at several locations. In addition, densification of soil at embankments and behind retaining walls caused road settlements and cracked pavements. The concrete median barrier was damaged at several locations. A surface rupture with a vertical uplift of 1.5 feet damaged the roadway near the summit of the Santa Cruz Mountains. Repairs to the roadway left the highway closed to everyone except local residents and carpools for 32 days. The roadway was reopened on November 19. The repairs included excavation and flattening of slopes, grout injection, soil nailing, and rock fences.

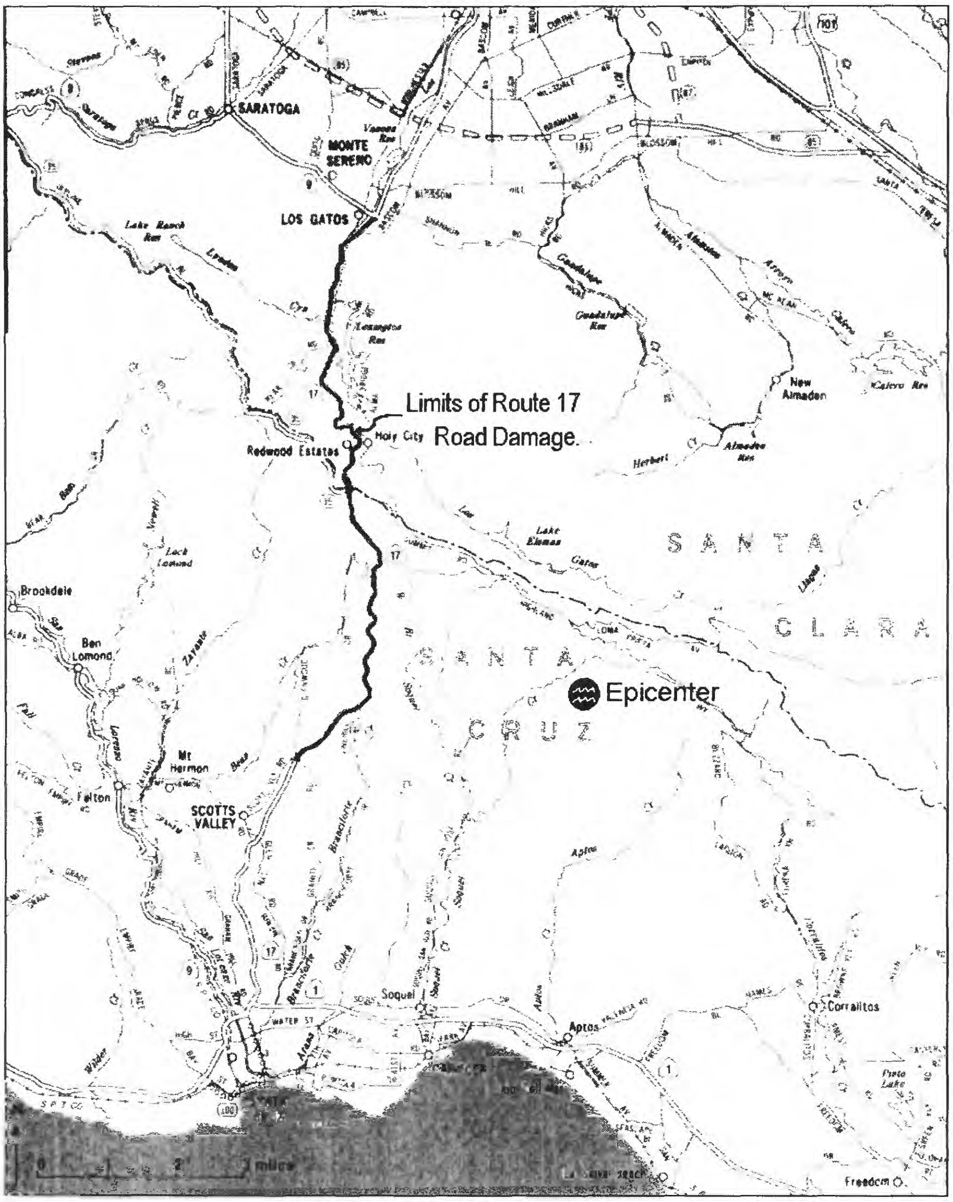

Figure 289.—Location of damage to Highway 17.

Figure 290.—Route 17 landslide.

Figure 291.—Route 17 landslide.

LONE TREE LANDSLIDE ON ROUTE 1.

This is an ancient landslide along the coastal bluffs of Marin County. It was remobilized but controlled by Caltrans following very heavy rains of January 1982. However, the Loma Prieta earthquake created a much faster rate of soil movement (figs. 292 to 295), eventually requiring a new road alignment behind the landslide. A detailed report describing this landslide is provided by Van Velsor and Walkenshaw (1992).

Landslides due to the earthquake also closed State Highways 9, 152, 236, and 35. More information is available on these landslides in the lifeline section of a report by Heyes (1990).

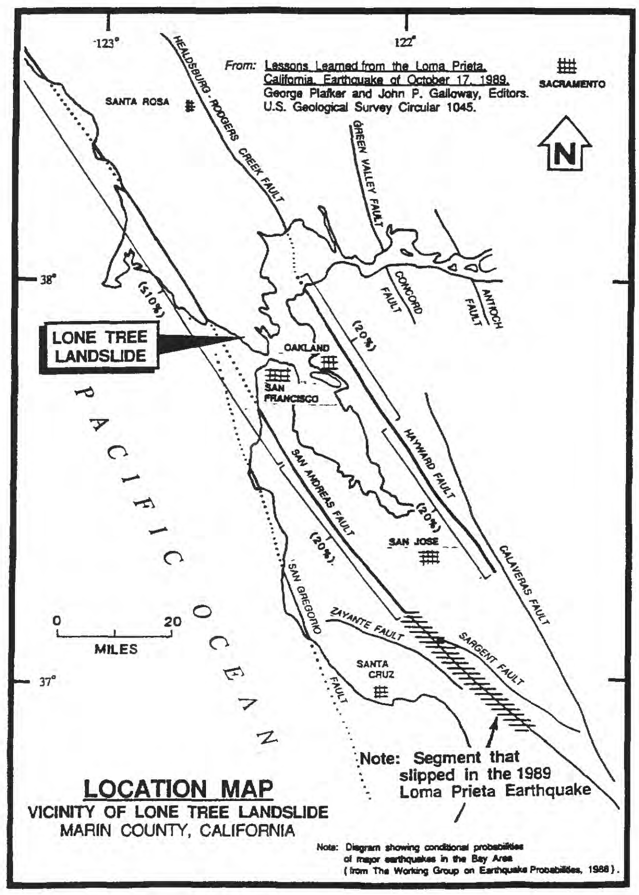

Figure 292.—Location of landslide in Marin County (courtesy of Joan Van Velser).

Figure 293.—Landslide along Route 1 in Marin County (courtesy of Joan Van Velser).

Figure 294.—Landslide damage along the coast (courtesy of Joan Van Velser).

Figure 295.—Sand boils and heaving at Interstate 80.

Figure 296.—Lateral spreading at Interstate 80.

EASTERN APPROACH TO THE SAN FRANCISCO OAKLAND BAY BRIDGE.

Interstate 80 runs south along the San Francisco Bay in Emeryville and then turns west and widens to become a toll plaza before crossing the Bay. Approximately 2 miles of this roadway from Powell Street to the Bay Bridge was damaged due to heaving, settlement, cracking, and lateral spreading following the earthquake (figs. 295 to 298). This damage did not affect traffic because a dropped span on the Bay Bridge closed this section of roadway. The road was regraded every day for a week before the movement subsided.

Figure 297.—Aerial photo showing location of Interstate 80 liquefaction damage.

Figure 298.—Repairs to Interstate 80.

SAN FRANCISCO CITY STREETS

Streets in San Francisco that were built on Bay mud or loose fills were damaged during the earthquake. This damage was often in the very same locations that had sustained damage during the 1906 San Francisco earthquake. Figure 299 was taken at the corner of Divisadero and Jefferson Streets in the Marina District. Differential settlement of the ground with accompanying buckling of asphalt pavement was common throughout the Marina (fig. 300).

Figure 299.—Road damage in San Francisco's Marina District (courtesy of Earthquake Engineering Research Institute).

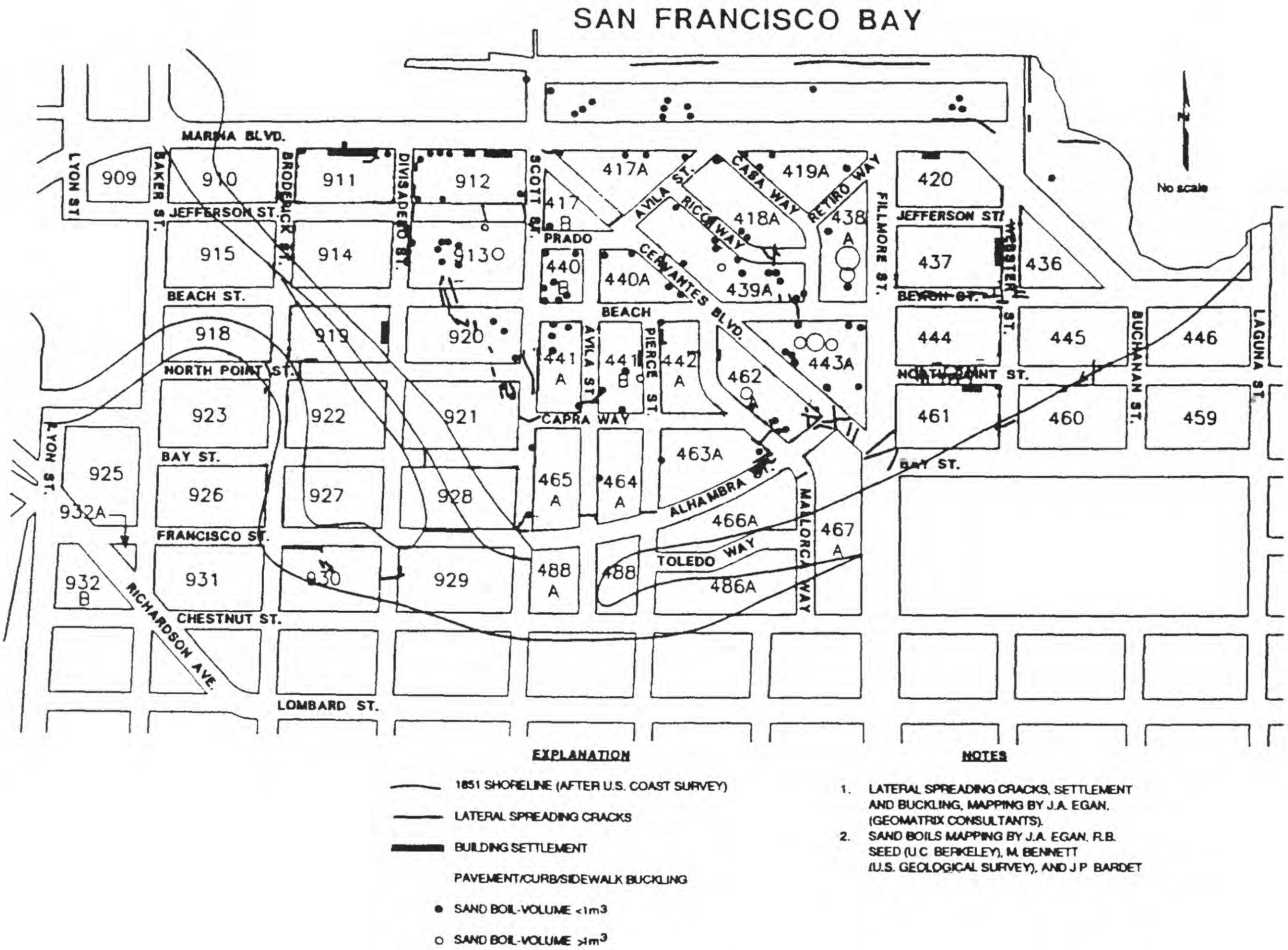

Figure 300.—Damage map of San Francisco's Marina District (Egan, 1990).

JETTY ROAD AT MOSS LANDING STATE BEACH.

South of the city of Santa Cruz, the approach road to Moss Landing State Beach was damaged due to liquefaction of the soil (figs. 301 and 302). About 300 feet of roadway subsided as much as 3 feet and spread 20 feet laterally.

Figure 301.—Damage to the access road at Moss Landing State Park (courtesy of Earthquake Engineering Research Institute) (view to north).

Figure 302.—Damage to the access road at Moss Landing State Park (courtesy of Earthquake Engineering Research Institute) (view to south).

Figure 303.—Aerial view of southern part of Treasure Island (at top).

Figure 304.—Lateral spreading damage to roadway at Treasure Island (courtesy of U.S. Geological Survey; photograph by Michael Bennett).

TREASURE ISLAND

Treasure Island is a Naval Station in the middle of San Francisco Bay. Land access to the island is limited to a causeway from Yerba Buena Island (fig. 303). During the earthquake this causeway (an embankment built over sand and Bay mud) experienced lateral spreading with cracks several inches wide. There was also widespread liquefaction damage to most of the roads on the island (fig. 304).

Several excellent reports analyzed the damage to the island after the earthquake. One is included in this Professional Paper series (Power, in press). Another report, to the United States Navy, also described the damage (Power, 1990).

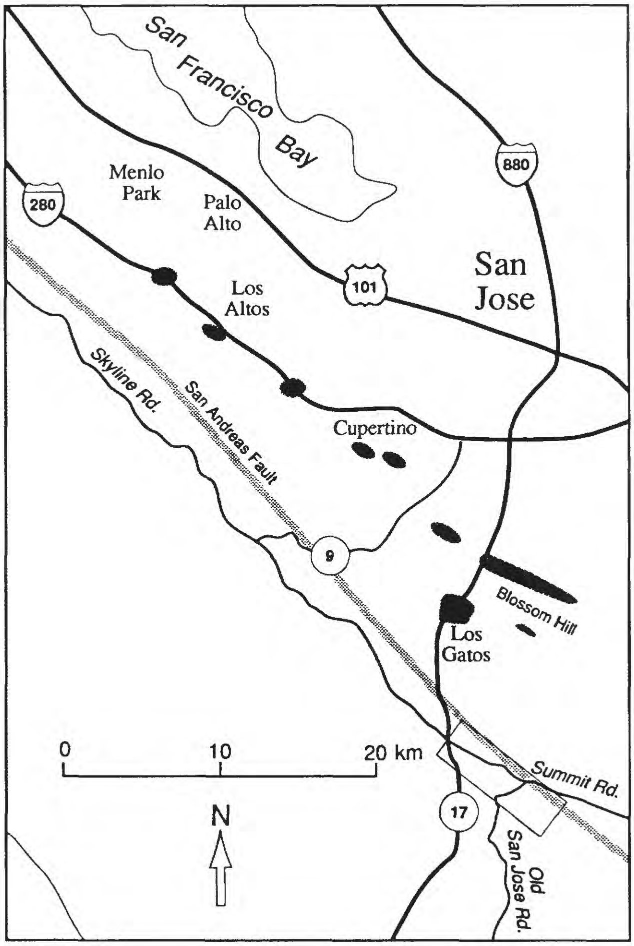

Figure 305.—Areas of compression damage along northeastern edge of Santa Cruz Mountains (rectangular area at lower right is area of fig. 308) (courtesy of Earthquake Engineering Research Institute).

Figure 306.—Buckled roadway (by traffic island) at Sand Hill Road.

Figure 307.—Close-up of damage at Sand Hill Road.

SAND HILL ROAD AT I-280 NEAR PALO ALTO

The fault rupture did not reach the ground surface; however, there were other types of surface deformation that occurred due to the earthquake. "Zones of compressional deformation run along the northeastern foot of the Santa Cruz Mountains between Blossom Hill and Palo Alto," according to Ponti (1990). "These are second-order tectonic features; they are probably not seismogenic but were triggered by tectonic rupture at depth," concludes the report.

These "second-order tectonic features" caused asphalt pavement to buckle and concrete curbs and sidewalk to break on city streets from Los Gatos north to Los Altos (fig. 305). At Sand Hill Road there was a movement of the ground that pushed a section of the road together until it failed in compression (figs. 306 and 307). Similar damage occurred to State Route 17 and Interstate 280.

Figure 309.—Damage to Morrill Road from 1989 (Loma Prieta) earthquake.

MORRILL ROAD AT THE SAN ANDREAS FAULT

There were many surface fractures in the Summit Road/ Skyline Ridge area near the San Andreas Mountains in Santa Clara County (fig. 308). This resulted in substantial road damage in the area (Ponti, 1990). Damage can occur to roads on good basement material when the pavement is within several miles of an active fault. Morrill Road suffered a transverse crack that closed the road after the earthquake (fig. 309). Surprisingly, a similar crack at the same location closed the road after the 1906 earthquake (fig. 310).

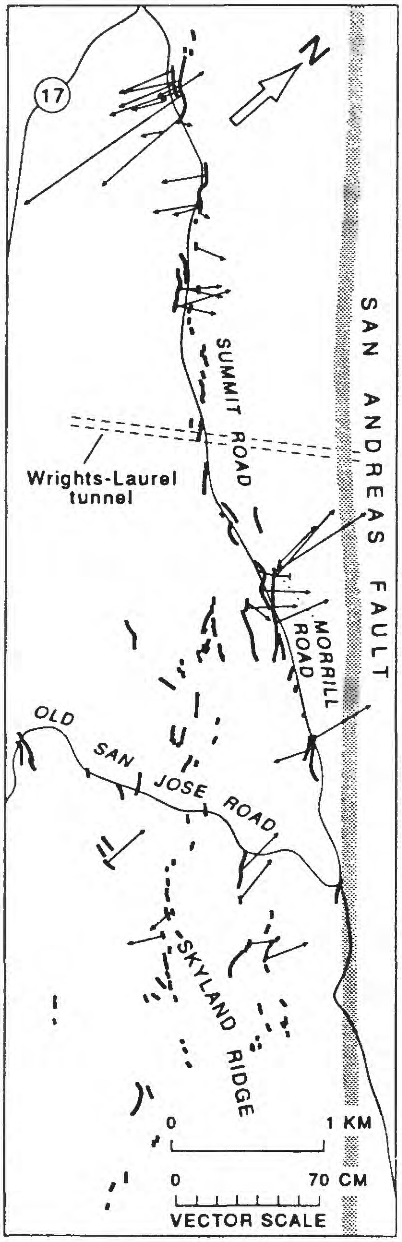

Figure 308.—Location of damage along Summit Road (fractures shown by heavy lines; arrows show direction of movement) (courtesy of Earthquake Engineering Research Institute).

Figure 310.—Similar damage to Morrill Road from 1906 earthquake.

DAMAGE TO TUNNELS

Tunnels without vulnerable structural elements like columns typically experience little earthquake damage. The rock and soil around the tunnel act to support it during ground shaking. This suggests that cut-and-cover tunnels must be more carefully designed for earthquakes than tunnels which are bored through the earth.

A potential problem for tunnels is due to changes of soil stiffness along their length. This could cause differential displacement, resulting in structural damage. Another problem occurs if a tunnel crosses a fault, which is also a problem for above-ground structures. Additionally, liquefaction is a problem for shallow tunnels, as it has been for culverts (see next section). Caltrans currently has no seismic design code for tunnels.

Several highway tunnels were in the area that suffered bridge damage from the earthquake. Caltrans underground structures unit investigated tunnel and culvert damage (Zelinski, 1993). There was no damage to any Bay Area Rapid Transit tunnel, including the long trans-Bay tube, a watertight steel shell extending under the San Francisco Bay, and the Berkeley Hills tunnel crossing the Hayward fault. No damage was reported for tunnels belonging to other light and heavy rail companies, cities, and counties. A railroad tunnel, the Wrights-Laurel tunnel (fig. 308), has been long abandoned but apparently has an offset from the 1906 earthquake; whether the 1989 movement of the San Andreas fault affected this tunnel is not known, as it is boarded up.

THE WEBSTER STREET AND POSEY STREET TUBE CROSSINGS

The only significant tunnel damage was to these 4,500-foot-long tubes which carry two lanes of traffic in each direction under the Oakland Estuary between the cities of Alameda and Oakland (figs. 311 and 312). The Posey Street Tube was built in the 1920's, and the Webster Street Tube was built in the 1960's. They are reinforced concrete tubes with a bituminous coating for water proofing , and they descend to about 70 feet below sea level. After Loma Prieta, some of the connections between the tube segments began leaking. It was thought that liquefaction of the soil around the tubes caused deformation at the joints. Earthquake repair drawings were made, and the tubes were epoxy injected to stop the leaking. In 1997 a more elaborate retrofit was planned that included (1) creating very stong, water-proof hinges at the joints, (2) compacting the soil, adding stone columns, and building a curtain wall to prevent liquefaction and pore pressure from pushing up the tubes, and (3) jet grouting columns to support the ends of the tube segments.

Figure 311.—Alameda entrance to Posey Street Tube (courtesy of Ben C. Gerwick Inc.)

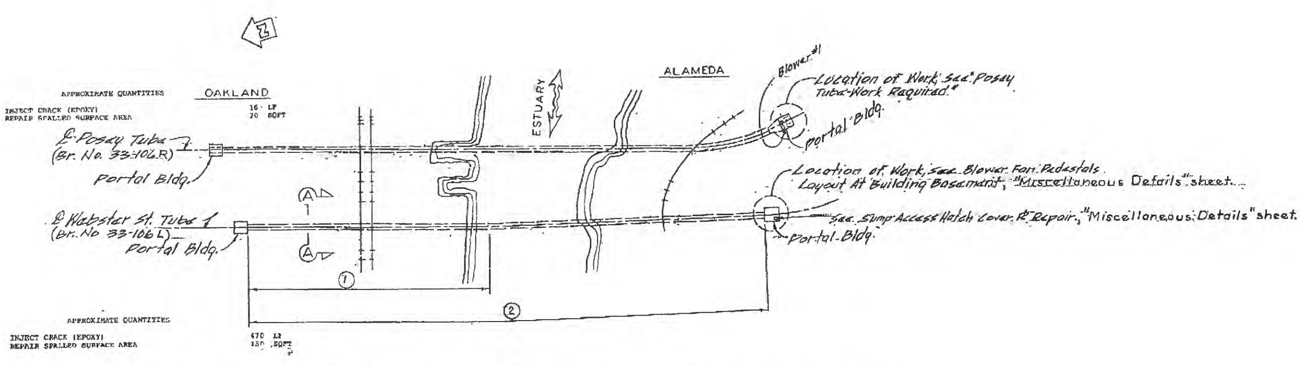

Figure 312.—Plan drawing of Posey and Webster Street Tubes showing location under Oakland Estuary.

FORT CRONKITE TUNNEL (Br. #27-39)

This in an unreinforced concrete arch built in 1954 that allows a single lane of traffic to go under Highway 101 in Marin County. It is 17 feet tall at its crown, 20 feet tall at its base, and 1,109 feet long (fig. 313). Although there were some indications of movement, and the tunnel was briefly closed by the Golden Gate Park Service, there was no earthquake damage to this tunnel.

PRESIDIO PARK TUNNEL (Br. #33-16)

This is a reinforced concrete arch tunnel built in 1955. It carries traffic on State Route 1 north to the Golden Gate Bridge. It is 1,300 feet long, 45 feet wide, 30 feet tall, and has a 17-foot minimum vertical clearance (fig. 314). It had no earthquake damage.

CALDECOTT TUNNEL (BR. #28-15)

These twin reinforced concrete arch tunnels take Route 24 traffic under the Oakland Hills between the Alameda and Contra Costa Counties (fig. 315). They were built in 1964 and carry two lanes of traffic each. They were inspected and found to be in excellent condition after the earthquake. This tunnel was instrumented and although the records are of low magnitude, they provided the first information on tunnel movement during an earthquake (fig. 316).

TUNNEL AT GEARY AND FIRST AVENUE IN SAN FRANCISCO

This is a four-lane two-directional reinforced concrete box tunnel. It had no earthquake damage.

STOCKTON STREET TUNNEL BETWEEN SACRAMENTO AND BUSH STREETS IN SAN FRANCISCO

This tunnel had no earthquake damage. There were some cracks in barrier rails at both ends of tunnel.

BROADWAY STREET TUNNEL BETWEEN MASON AND HYDE STREETS IN SAN FRANCISCO

This tunnel had very minor earthquake damage. Ground shaking removed about 15 tiles from the tunnel.

Figure 313.—Fort Cronkite Tunnel.

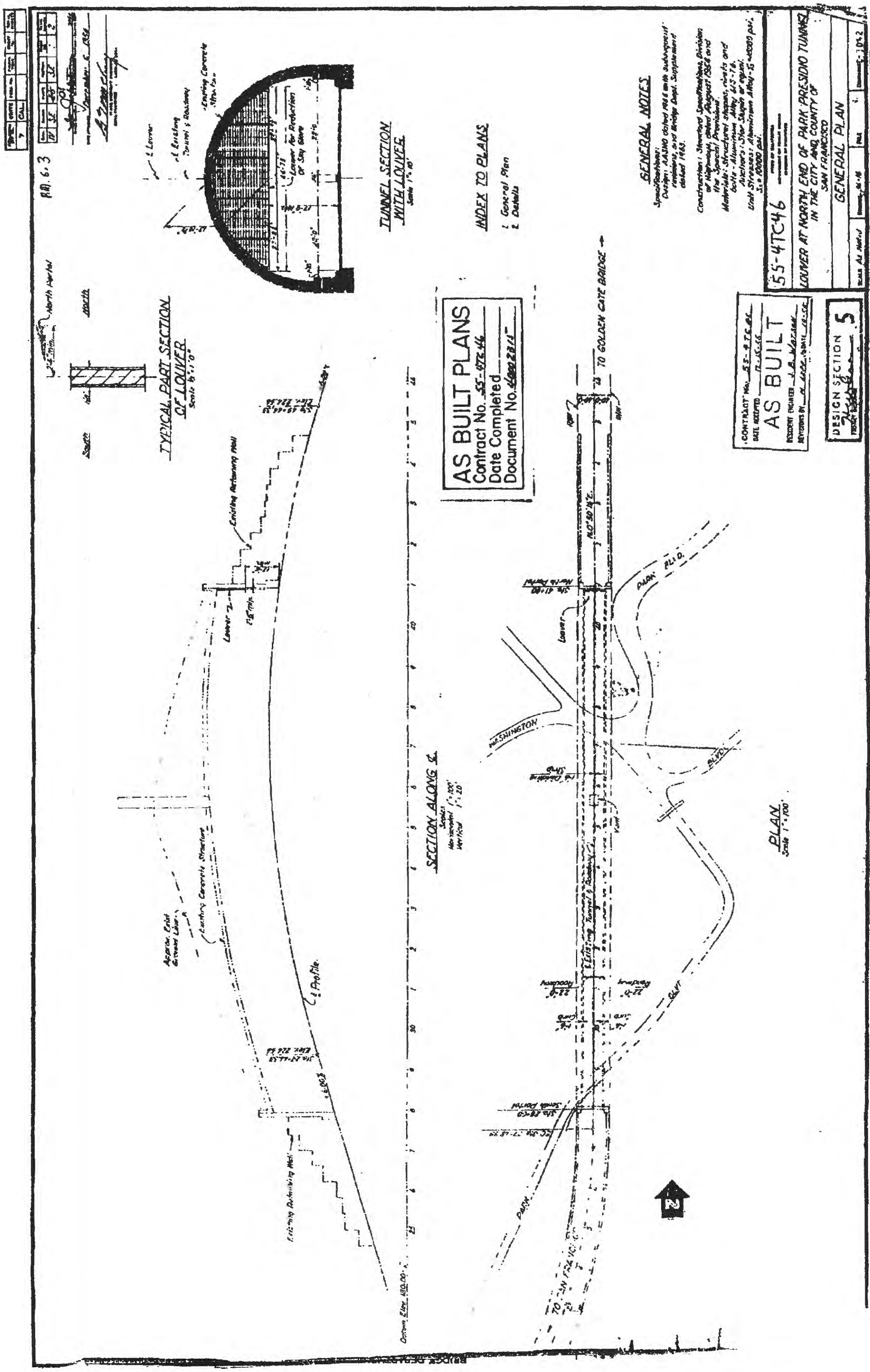

Figure 314.—General Plan drawing of Presidio Park Tunnel.

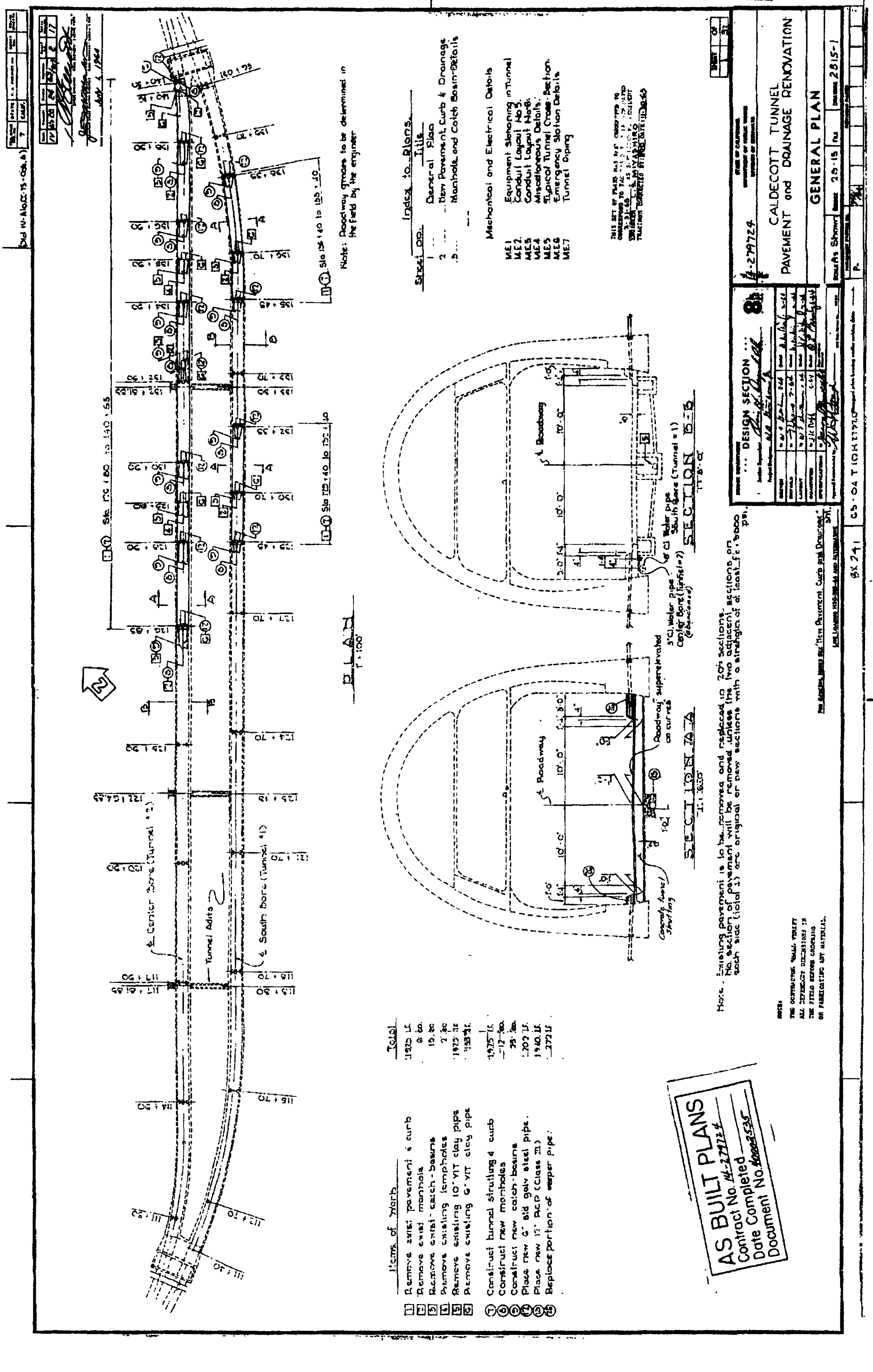

Figure 315.—General Plan drawing of Caldecott Tunnel.

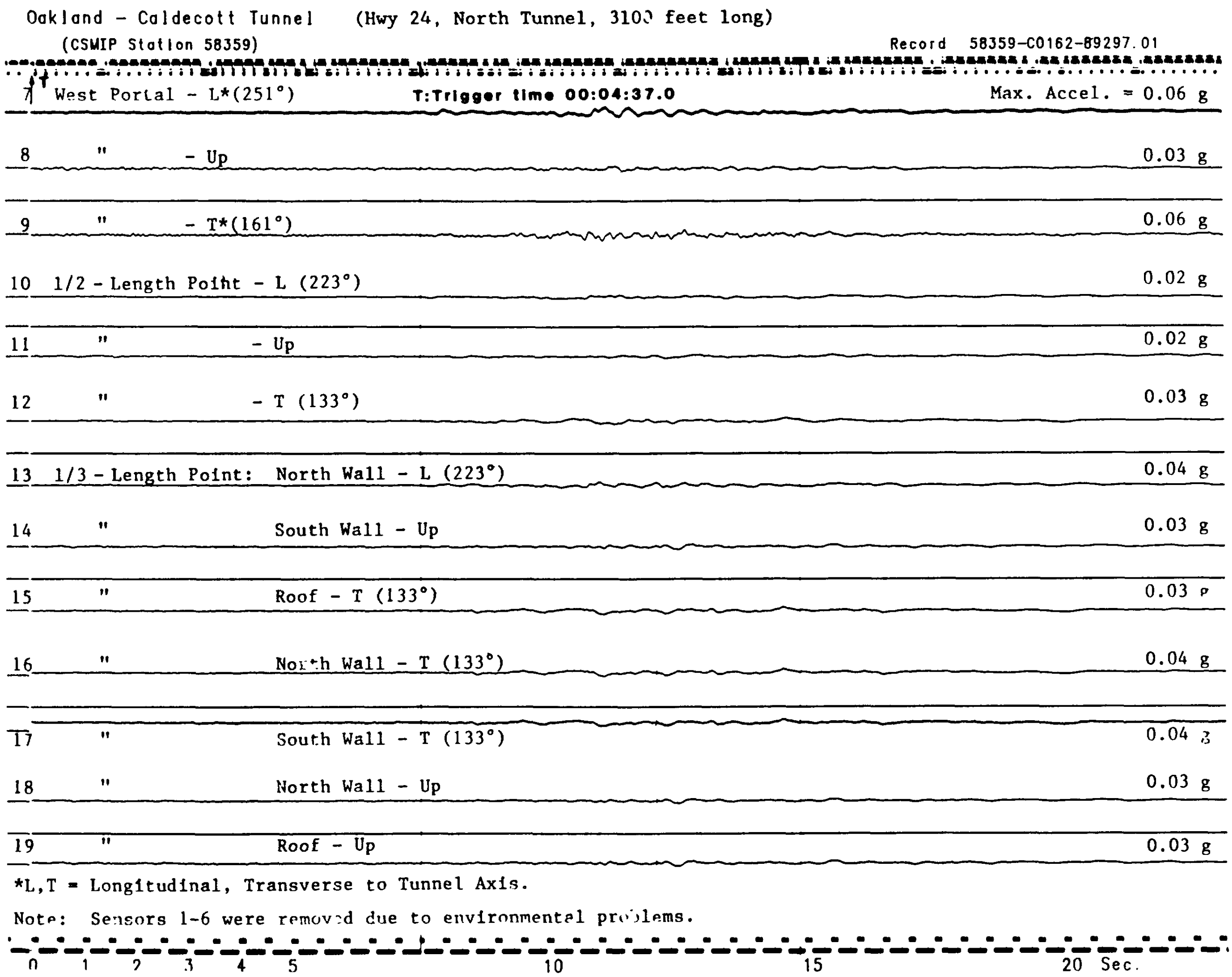

Figure 316.—Strong motion records for Caldecott Tunnel (courtesy of California Strong Motion Instrumentation Program).

DAMAGE TO CULVERTS

Culverts are commonly used to allow water to drain under (rather than across the surface of) roadways. There are several types of culverts, and each had its own type of earthquake damage. Corrugated metal pipe (CMP) culverts are most commonly crushed or pushed out of the ground during earthquakes as a result of liquefaction of the base material. Occasionally these culverts will be damaged when fill material slides downhill, as a result of being set on sloping ground or because of landslides. Reinforced concrete box culverts are occasionally damaged due to poor connection details between the walls and the roof or

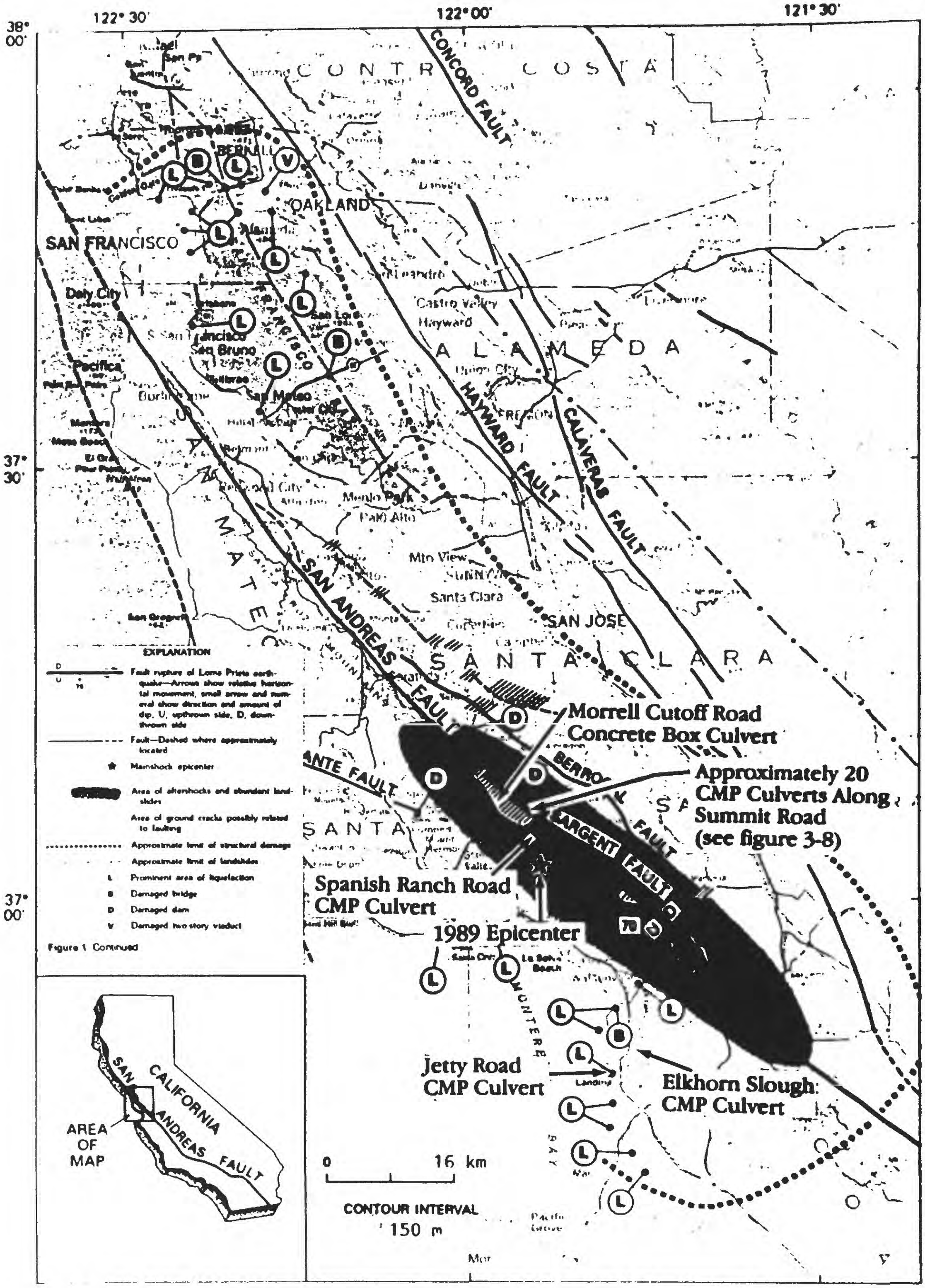

Figure 317.—Location of culvert damage (Youd, 1997).

floor of the culvert. If the roadway is at an angle to the culvert, one side may have less soil against the walls, which can cause problems during shaking.

A few culverts sustained damage during the earthquake (fig. 317). Much of the information on culvert damage comes from Youd (1997). With a few exceptions, the location of culvert damage matches the location of road damage. Usually, where ground deformation, liquefaction, or landslides damaged roads, they also damaged culverts, the exceptions being where structural damage occurred to concrete box culverts or where, surprisingly, the CMP survived roadway damage.

JETTY ROAD AT MOSS LANDING STATE BEACH

The liquefaction that damaged the access road to Moss Landing State Beach also damaged a 30-inch CMP used to allow tidal flow and drainage of Elkhorn Slough. Actually, the culvert had been partially plugged, creating a beautiful fresh water lagoon inland of the access road.

During the earthquake the soil around the culvert settled, allowing the roadway to fracture the culvert and forcing the two ends up and out of the water (figs. 318 and 319). The damaged culvert was replaced with six new culverts that converted a fresh water marsh to a barren tidal marsh (Youd, 1997).

Figure 318.—Culvert damage at Jetty Road (courtesy of Leslie Youd).

Figure 319.—Aerial view of culvert damage at Jetty Road (the culvert has been bent until it sticks out of the water) (courtesy of Leslie Youd).

ELKHORN ROAD CULVERTS

Similar to the damage at Moss Landing State Beach, 3 miles inland where Elkhorn Road crosses Elkhorn Slough are seven 36-inch-diameter CMP culverts. Again, liquefaction settled the supporting soil, crushing the culverts and pushing the ends as much as 2 feet out of the water (fig. 320). There has been little repair at this location. However, the culverts still seem to be functioning, with the ends submerged during high tide.

SPANISH RANCH ROAD CULVERT

This was the only example of culvert damage as a result of a landslide during the earthquake. This was a 36-inch-diameter CMP consisting of three 20-foot-long segments tied together with a 1- to 2-foot overwrap at each joint. During the earthquake, the outer third of the roadway slid down about 3 feet, slightly damaging the roadway (fig. 321). The overlaps at the joints were long enough to prevent the culvert from being pulled apart, although the culvert was slightly displaced and reoriented by the ground movement.

SUMMIT AND MORRILL ROAD CULVERTS

The third broad category of culvert damage is ground deformation as a direct result of or as a secondary effect of fault rupture. There were about 20 CMP culverts in the area of surface cracking, as shown in figure 318. No culvert damage was observed. This may have been good culvert performance for this kind of ground behavior or may have been simply a matter of luck. Youd (1997) observed that none of the culverts were intersected by the crack locations on the map. There was also a box culvert under Morrill Road in the same area. There was some damage to the culvert which, on closer inspection, is believed to have occurred before the earthquake.

TEMESCAL CREEK CULVERT

There was damage to a box culvert as a result of the earthquake. Temescal Creek flows under I-80 in Emeryville. The culvert is a double 10 by 10 foot reinforced concrete box, about 220 feet long, built under the freeway (fig. 322). There are also two 96-inch-diameter reinforced concrete pipe culverts immediately to the north. After the earthquake, the northernmost cell was inspected by rowboat. The adjacent cell was filled with mud and could not be accessed by boat (as were the pipe culverts). Cracks ran along the soffit and exterior wall of the north cell, and an expansion joint about 55 feet from the east end opened about 1 foot. This culvert is located where a great deal of liquefaction-related damage occurred to I-80. Apparently, this is another case where liquefaction damaged the roadway and the culvert. Figure 323 shows the general plan for the repair of earthquake damage to this culvert.

Figure 320.—Damaged culverts at Elkhorn Road.

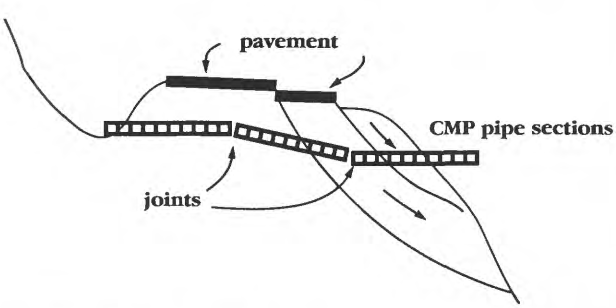

Figure 321.—Spanish Ranch Road culvert.

Figure 322.—Temescal Creek culverts.

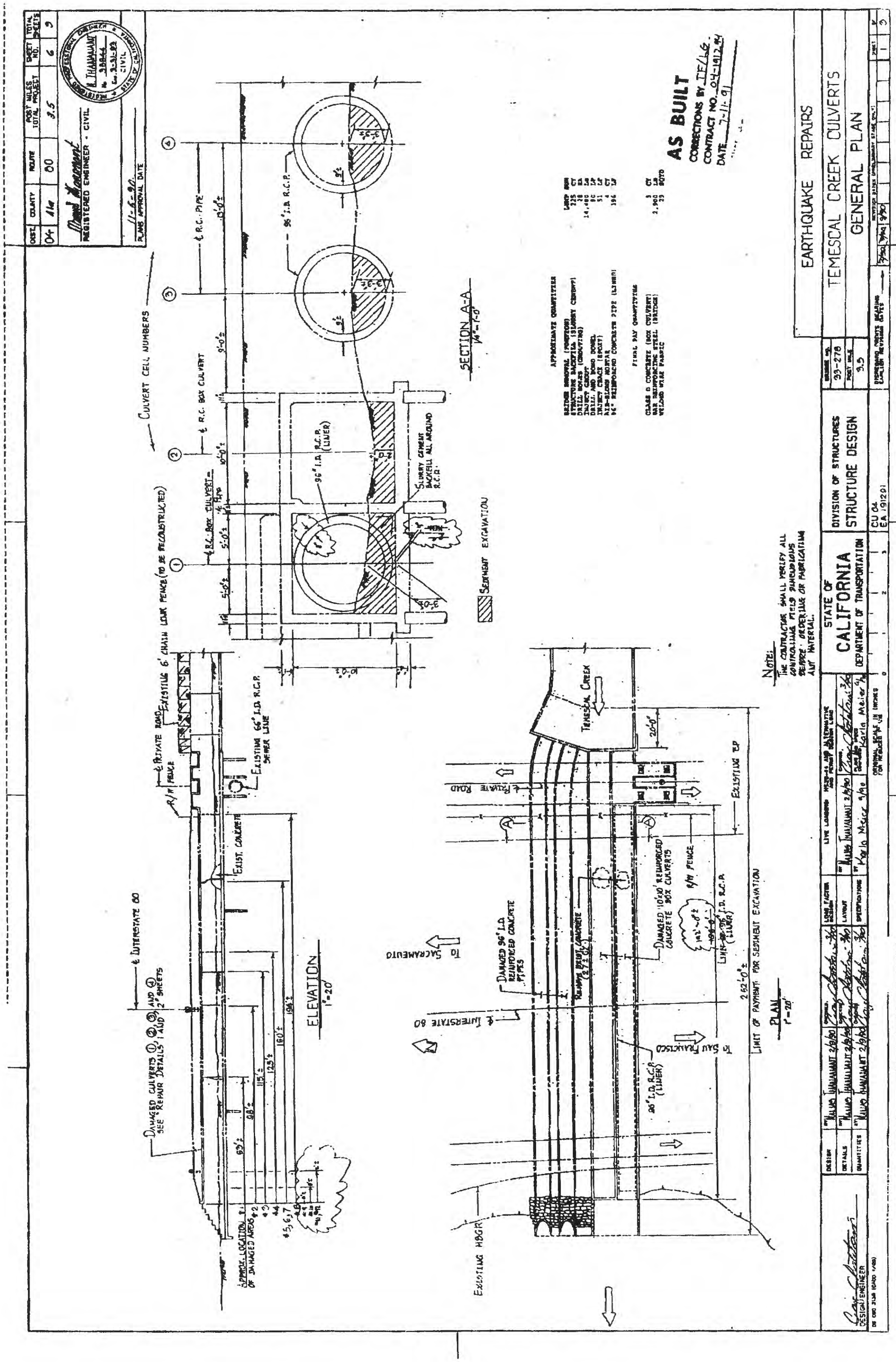

Figure 323.—General Plan drawing of repairs to Temescal Creek culverts.

DAMAGE TO EARTH RETAINING STRUCTURES

Where space for a roadway is limited, a variety of retaining walls have been devised to hold back the soil and provide enough room for a road. Caltrans considers its current retaining-wall design procedure so conservative that no special provisions for earthquake loads has been added. This policy has stood up well despite several large earthquakes; little damage has ever occurred to any Caltrans-designed retaining wall. These walls tend to receive some support from the adjacent soil during earthquakes.

There are many different types of retaining walls, including a variety of reinforced earth walls, soil-nail walls, tieback walls, gravity walls, etc., in the area that experienced bridge damage, and all performed well during the earthquake. The only walls known to have suffered damage were some crib walls in the Santa Cruz Mountains. The information on these walls was provided by Lew and Chieruzzi (1990).

CRIB WALL AT HIGHLAND ROAD IN SANTA CRUZ COUNTY

Crib walls are cells made out of precast concrete, metal, or timber elements that are filled with earth. Several crib walls in the Santa Cruz Mountains showed distortions due to settlement of the backfill (behind the wall) and movement of the wall face. The wall shown in figure 324 showed major distress halfway up the face and had backfill settlement of more than a foot. This wall may have been built at a previous landfill site.

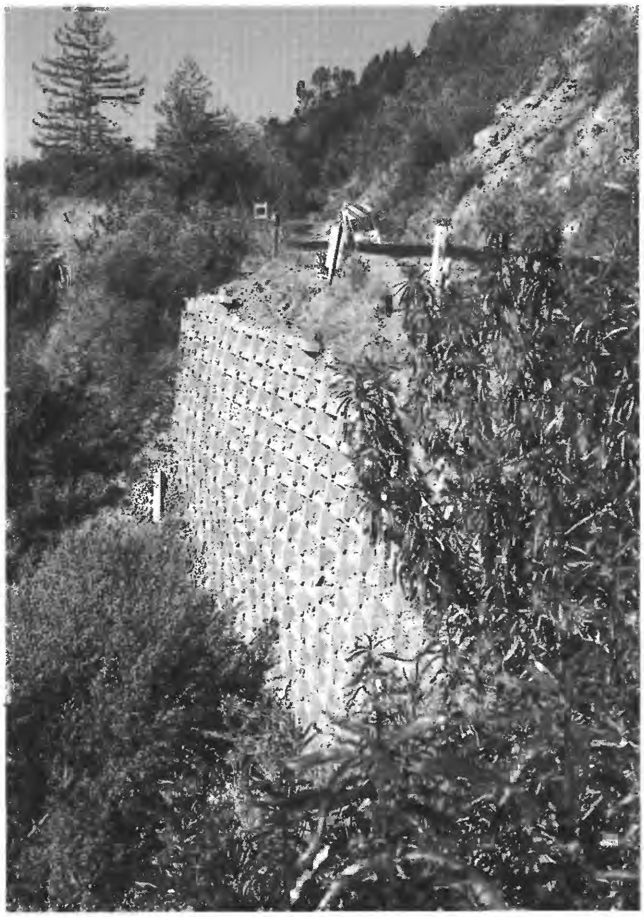

Figure 324.—Crib wall (courtesy of Earthquake Engineering Research Institute).

SOIL-NAIL WALL AT THE UNIVERSITY OF CALIFORNIA AT SANTA CRUZ

Soil-nail walls are used to temporarily hold back the soil during construction and sometimes as a permanent retaining wall. There were many soil-nail walls in the area of earthquake damage, including one (fig. 325) near the rupture zone. They all performed very well during the earthquake.

Figure 325.—Soil-nail wall (courtesy of Earthquake Engineering Research Institute).

DAMAGE TO LOCAL HIGHWAY SYSTEMS

City streets and county roads were also damaged by the earthquake. Some of the damage was identified through correspondence and is listed in table 27. Other damage was ascertained through requests for Federal disaster relief as shown in table 25. Still more local damage was discovered by earthquake engineers conducting investigations. Finally, some cities and counties have written reports and even books describing the earthquake's impact to their streets and roads. For instance, the City of San Francisco has written a comprehensive summary of damage to road systems (City and County of San Francisco, 1993), and Santa Clara County has written a volume on bridge damage and their retrofit program (Randall, 1994). The cost of highway damage for the city of San Francisco is summarized in table 28.

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Table 27.—*Damage to city transportation systems*

City or County	Impact	Commentary	Official
Livermore	No	None	John Hines, Public Works
Sonoma	No	None	Richard Rowland, Public Works
Monterey	No	None	Tom Reeves, City Engineer
Sausalito	No	None	O. Gary Plunkett, City Engineer
Half Moon Bay	No	None	William Smith, City Engineer
Sebastapol	No	None	Larry Koverman, Public Works
Seaside	No	Cracks on secondary roads	Richard Guillen, City Engineer
Novato	No	None	Bob Weil, Senior Civil Engineer
Hayward	No	None	Bud Simmons, Sr Civil Engineer
Clayton	No	None	Cathie Kelly, Secretary
Daly City	Yes	Repair of cracks and uplift took 1 yr	Mohinder Sharma, City Engineer
Palo Alto	Yes	Bike and Pedestrian Bridges	Michael Jackson, Public Works
Fremont	No	Began bridge retrofit program	T.M. Blalock, Public Works
Alameda	Yes	Liquefaction Damage	T.D. Edwards, City Engineer
Berkeley	Yes	Still working on repairs in 1993	Jeffrey Egeberg, Engineering
San Francisco	Yes	13.4 million dollars in damage	Department of Public Works

Table 28.—*Highway costs for the City of San Francisco*

Description	Number of Facilities	Project Cost
Streets	68	$12,951,981
Traffic Control	17	$152,430
Bridges	5	$20,000
Road Structures	21	$281,045

CHANGES TO ENGINEERING PRACTICE

Caltrans began designing for earthquake loads soon after the 1933 Long Beach earthquake. However, it was the 1971 San Fernando earthquake that resulted in most of the changes to the seismic specifications for bridges. After that earthquake, Caltrans began designing bridges for seismic loads using the maximum credible earthquake from Greensfelder (1974). Taking the maximum acceleration from the bridge location on a map along with the appropriate depth of alluvium at the site, the bridge engineer could choose the appropriate response spectra to perform a multimodal dynamic analysis of the structure. The response spectra were normalized from five significant California earthquakes on rock, amplified based on the depth of alluvium at the bridge site, and reduced for 5 percent damping. For a large bridge project, a site-specific response spectra was developed.

The bridge model included a bridge-abutment stiffness limited to a yielding force of 7.9 kips/ft2. Column foundations were originally modeled as pinned or fixed. Column moments taken from this elastic multimodal analysis were then reduced by a risk and ductility factor to approximate the true, nonlinear behavior of the bridge. This reduction in moments was based on Newmark (1971), who stated that for long-period structures (periods greater than about 0.7 seconds), the maximum displacement of elastic and inelastic structures (with the same period) is about equal. Therefore, although it was the moment that was being considered in the analysis, it was really the maximum displacement that was being controlled by the ductility factor (fig. 326). The values for ductility were based on tests done in New Zealand that showed that well-confined columns can undergo large displacements and curvature without failing (Park, 1971). An additional factor which considered the risk associated with different bridge elements was added to the ductility factor, resulting in the risk and ductility factor "Z" shown in figure 327 (Caltrans, 1983).

Using this reduced moment and the appropriate axial load, the column section and the amount of column steel were selected from an interaction diagram or the program YIELD from McBride (1992). This limited the forces in the bridge to the largest plastic moment that these columns could develop. Adjacent members, such as footings and bent caps were designed to be at least as strong as the plastic column moments. This forced yielding to occur in the columns, where damage could be controlled by well-confined spiral reinforcement. The abutments were then designed to add significant damping to the system. Sufficient seat widths and restrainers were provided at bridge joints to keep the superstructure from falling. Thus, during a very large earthquake, column reinforcement would yield, bridge joints might bang and spall, and the soil behind the abutment might degrade, but the bridge would be able to handle the earthquake due to the ductility of the bridge columns.

Assumptions of column ductility underwent intense scrutiny after the earthquake, (Priestley and Seible, 1991). They found that well-confined columns were able to achieve the ductility assumed in Caltran's design. Testing was also done to determine the ductility of old, unconfined columns, steel-shelled columns, and fiber-wrapped

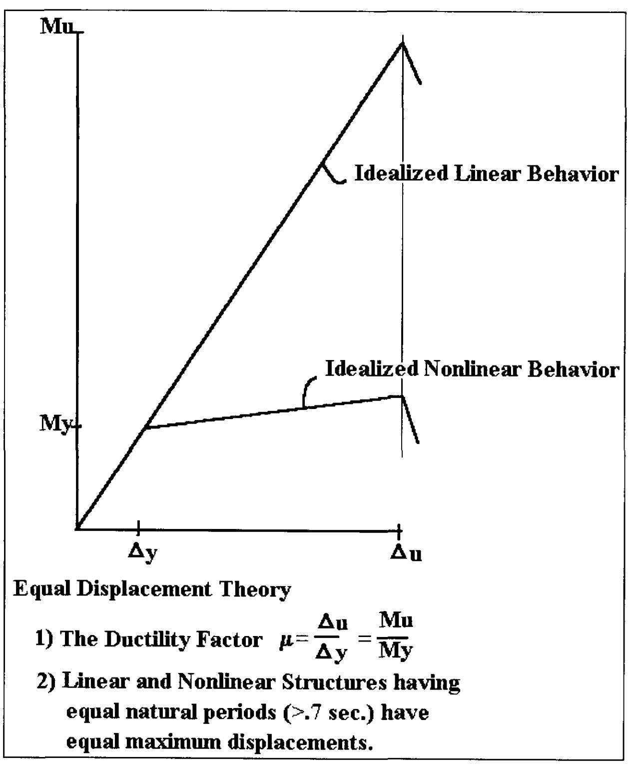

Figure 326.—Equal displacement theory for seismic bridge design.

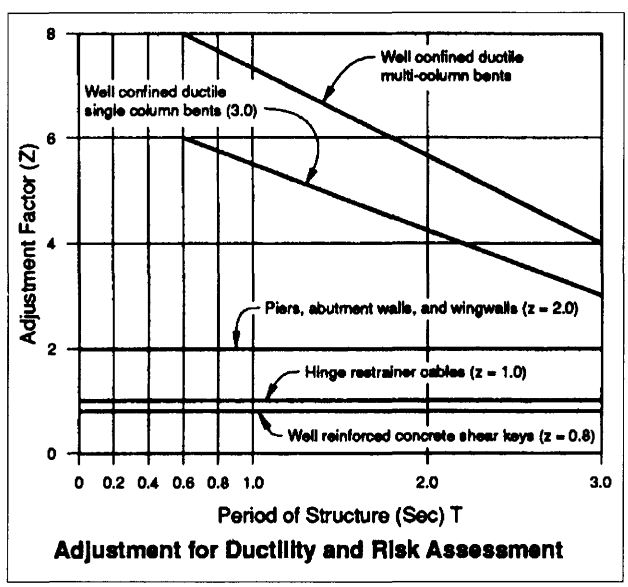

Figure 327.—Ductility and risk factor "Z" used in the seismic design of bridges.

columns. This was essential for the seismic retrofit effort that Caltrans undertook after the earthquake.

Looking at the bridge damage from the earthquake reveals a couple of significant contributing factors. Bridge columns designed after the 1971 San Fernando earthquake performed well. However, older columns with poor confinement and lap splices in areas of plastic hinging showed their vulnerability. Caltrans, therefore, accelerated the Phase 2 retrofit program, which provided column confinement with a steel shell for older single-column bents. These retrofits often required a column footing retrofit and sometimes an abutment retrofit as well. This was because the shelled column now could form a plastic moment which had to be resisted by the footing or because modifying the abutment was sometimes a cheaper way of protecting the columns, particularly for short bridges. Caltrans also began a program to retrofit bridges with multicolumn bents. This usually required checking the moment capacity of the bent cap. Along with the seismic retrofit program came a much more intense research program to test the assumptions of the seismic retrofit strategies. In many cases a seismic retrofit had to wait for test results to show the effectiveness of a design strategy.

After the earthquake, Caltrans began spending money on research equal to 1 percent of its construction budget, or about $5 million a year. The most important research done immediately after the earthquake was to test the ductility of older column designs and the ductility of the seismic retrofits of columns (Chai, 1991). Other significant research was the testing of shear walls (Haroun, 1993), studies of the shear capacity of column-superstructure and column-footing joints (Stojadinovic, 1995; Seible, 1994; Ingham, 1994), footing retrofit tests (Xiao, 1994), and studies on column shear and P-Delta effects (Priestley, 1993; MacRae, 1993; Ascheim, 1992; Mahin, 1991). Through this intense research effort, reliable values for the shear, moment, and displacement capacities of various bridge members were established, which were of great benefit to Caltrans' retrofit program.

The cost of designing a new bridge to perform well for a large earthquake is relatively small compared to the cost of retrofitting an existing bridge to perform as well. This is because it is difficult to change the dynamic behavior of a structure after it is built. The bridge research paid for itself many times over by identifying a larger seismic resistance in many bridges. Investigations into the damping values of bridges (Werner, 1993), research on the stiffness of soil-structure elements by Romstad (1996), Po Lam (1986, 1991), and others, and work to provide Caltrans with a nonlinear bridge-analysis program by Powell and Campbell (1994) and Prakash (1994), all contributed to an increased understanding of bridges and new analysis techniques that aided Caltrans' retrofit efforts.

Funds for seismic retrofits came from several sources. The State of California raised its sales tax one-quarter cent for a limited period to provide revenue for the seismic retrofit program. Federal funds that would have been used for the state transportation program were diverted to pay for retrofits. A total of $750 million was spent retrofitting 1,039 state-owned bridges. After the 1991 Northridge earthquake, phase 2 of this program was begun to retrofit an additional 1,155 state bridges at a cost of about $1.05 billion. A third retrofit program was begun to retrofit state-owned toll bridges. A $2 billion bond measure (Proposition 192) was passed by the people of California in November 1995 to pay for the phase 2 retrofit program. However, the cost of the toll bridge program will most likely exceed the remaining funds. A decision has yet (1997) to be made on how the additional funds will be obtained. The Golden Gate Bridge retrofit was paid for by raising tolls from $1 to $3. A program was also begun to retrofit locally owned bridges using Federal bridge replacement and rehabilitation funds. Additional retrofit programs may be required in the future for specific routes and locations. For instance, California and the Federal government are interested in a higher performance level for essential routes that must remain open after a disaster. This will require a future retrofit program. Bridges that are deemed vulnerable to large velocity pulses from nearby faults may one day be retrofitted. Likewise, most California bridges have not been analyzed for very long duration earthquakes. However, these retrofit programs may have to wait for large and very damaging earthquakes. In 1992, the 7.6 magnitude Landers earthquake occurred, but because it caused little damage, it was quickly forgotten. The driving force for money to improve the seismic vulnerability of highway systems has been disasters of a magnitude to evoke concern to the media, the government, and the public (not necessarily in that order).

Since there is a relatively small amount of money to make the 12,000 state bridges (and the 12,000 locally owned bridges) capable of surviving a maximum credible earthquake, a great deal of effort was expended in refining analysis tools and eliminating bridges from the program. The first step in this process was the development of a risk algorithm to list all bridges according to their vulnerability to the maximum credible earthquake. This algorithm looked at factors such as the year built, length, daily traffic, etc. The algorithm was run using a database that was created through a general plan review of all state bridges. Those bridges with good seismic details were eliminated from the database for this first general plan review. The result was a list of all state bridges from the most vulnerable, with a value of one, to the least vulnerable, with a value of zero. A total of 440 bridges, which included all vulnerable single-column bent bridges as well as all toll crossings, was removed from this list for immediate analysis and retrofit. Some single-column bridges with difficult details or some with single and multicolumn bents were left in the list for later review and retrofit. The rest of the bridges in the list went through a rank analysis, which was a refinement over the previous algorithm and considered such things as distance from a major fault. This algorithm (Sardo, 1993) was based on recommendations by the Caltrans Seismic Advisory Board. Then a second review of the most vulnerable bridges was begun by engineers to eliminate those from the retrofit program with good seismic details. The most vulnerable bridges from the database were put together into projects usually based on their location. This made it easier for a contractor to retrofit several bridges at once. Caltrans' Local Assistance Branch was in charge of turning the retrofit list into projects with separate expenditure authorizations. Because Caltrans' management felt their designers could not perform their regular work as well as all the retrofit work, the decision was made to hire consultants to do most of the retrofit design. A similar program to retrofit the 12,000 local agency bridges also was started. Local bridges in Los Angeles and Santa Clara Counties were managed by those counties, while the rest were managed by Caltrans. Federal bridge replacement funds became available to retrofit some of these bridges.

After a consultant or design section was given a retrofit project, they usually spent some time running calculations and coming up with a retrofit strategy. Then a strategy meeting was organized. This was usually attended by Caltrans engineers who had experience and expertise in seismic retrofits. The retrofit project engineer would describe the analysis procedure they followed and the retrofit strategy they proposed. Then Caltrans staff would try to find flaws in the strategy, things the engineer might have missed, possible mistakes, and tried to get the engineers to sharpen their pencils. This was because Caltrans was trying to eliminate all unnecessary expense. This resulted in many innovations in retrofit design. The dominant new retrofit strategy was to limit the displacement demand to less than the displacement capacity at the columns and by taking as much force as possible at the abutments. This strategy depended upon two excellent programs. The program COLX (which later became XSECTION) calculated the displacement capacity of single columns and the program FRAMEX (which later became wFRAME) calculated the displacement capacity of frames (Seyed-Mahan, 1995). The original method of comparing column moment demands from a multimodal analysis with column moment capacities (and checking the shear) would suggest a great deal of retrofitting was needed. By going to a displacement analysis, engineers could make retrofits that handled the maximum credible event with very little damage.

How much damage a structure was allowed to sustain during an earthquake became the subject of intense review after Loma Prieta. Eventually standards were developed for one bridge capacity at a functional demand level and a second capacity at a safety demand level for two types of bridges, important and regular structures. This criteria was developed with Caltrans' Seismic Advisory Board. However, for the vast majority of retrofits the criteria remained "no collapse." Therefore, much of a strategy meeting's effort was to eliminate retrofits whose purpose was to prevent damage rather than collapse.

Many different retrofit strategies and analysis procedures came out of Loma Prieta. The premier handbook for retrofit design after the event was by Priestley and Seible (1991). Caltrans revised its seismic retrofit specifications (Caltrans, 1995). These sources became standard practice by designers and consultants. The most important values for designers were column-displacement capacities and column-shear capacities since these values determined how much money was required for a retrofit.

Caltrans funded the Applied Technology Council (ATC) to look at Caltrans' seismic procedures and recommend improvements similar to what they did for the Federal Highway Administration (FHWA) after the 1971 San Fernando earthquake [ATC-6, 1981]. This work eventually became ATC-32 (Nutt, 1996). Among the innovations in this document are new response spectra, new ductility factors, and improved procedures for member shear and joint shear design.

Not only did Caltrans engineering practice change as a result of the Loma Prieta earthquake, but concerns following this earthquake led to new policies for most bridges in the United States. Information on the national impact of the earthquake was provided by Buckle (1995). Immediately after the earthquake, the Federal Highway Administration (FHWA) required that all federally funded bridges be designed using ATC-6 seismic provisions, which were more rigorous than the current specifications used by most states. The American Association of State Highway and Transportation Officials (AASHTO) contracted with the National Cooperative Highway Research Program (NCHRP) to develop a new bridge specification using load and resistance factor design and modifications based on knowledge gained from the Loma Prieta earthquake. In 1990, the ATC-6 specifications became Division 1-A of the Standard Specifications. Moreover, AASHTO asked NCHRP to review these provisions in light of the Loma Prieta earthquake. The National Center for Earthquake Engineering Research (NCEER) began this work as project NCHRP 20-7/45 and published its final report in 1991 (American Association of State Highway and Transportation Officials, 1991).

In 1991 the United States Congress authorized the Intermodal Surface Transportation Efficiency Act (ISTEA). Among its provisions were a 6 year research program into the vulnerability of existing highway systems. Some of the information provided in this report, like the performance of culverts and roadways, was obtained through this research project. Another part of this project was the revision of FHWA directives (Buckle, 1995). All this work was begun out of concerns with older highway performance shown during the Loma Prieta earthquake.

In 1992 the FHWA awarded a research contract to NCEER to study performance criteria, spatial variations, and seismic details for moderate seismic zones.

In 1993, many states began to examine the seismic vulnerability of their large bridges out of concern with the damage sustained by the San Francisco-Oakland Bay Bridge during the Loma Prieta earthquake.

The Loma Prieta earthquake also influenced the bridge design codes in Canada and Japan. Researchers from all over the world came to study highway damage and what the implications were for their own countries. The earthquake showed all too clearly what could happen when a highway department has a large inventory of older, seismically deficient bridges. It also showed how the earthquake could damage bridges many miles from the fault rupture. Most importantly, it provided an increased level of funding to evaluate and respond to these problems.

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CONCLUSION

This report summarizes some of the ways that the Loma Prieta earthquake impacted highway systems. Some areas are discussed more thoroughly than others. For instance, the impact on state and Federal highways is addressed more than city streets and county roads. More information is provided on bridges than on other highway structures. The concerns of engineers are more represented than those of planners or administrators. However, information on these other topics is available. Specific references are listed for each bridge and at the end of each main section. Readers looking for a more complete listing of references related to the Loma Prieta earthquake can refer to the Earthquake Engineering Research Center (EERC) National Information Service for Earthquake Engineering (NISEE) Library. They have recently completed two large projects to provide references for the Loma Prieta earthquake. The first project was funded by the National Science Foundation (NSF) Loma Prieta Clearinghouse Project of Ongoing Research Reports with an annotated bibliography of 2,224 references. The second project is a USGS-funded archive for a permanent record of raw research data sets. Both of the projects will be available on CD-ROM and on the world wide web. Their internet address is "http://www.eerc.berkeley.edu". Caltrans also has a homepage that provides information about California's highway system. However, this homepage is not as comprehensive. One day, all the information in the world will be easily accessible from a personal computer. Until then, bridge plans can be obtained by writing to Caltrans Office of Structural Maintenance and Investigations, P.O. Box 942874, Sacramento, CA 94274.

Large earthquakes can cause human casualties, costly damage, and closures on highway systems. We have seen how the Loma Prieta earthquake resulted in death and injury on older structures such as the Cypress Viaduct (an elevated freeway) and the San Francisco-Oakland Bay Bridge (a long truss bridge). Recent earthquakes demonstrate that Caltrans has learned the life safety lessons of the Loma Prieta earthquake well. They have prioritized their entire bridge inventory, performed a structural analysis on the most vulnerable structures, and developed retrofits that are meant to prevent serious injury during future earthquakes. They are supporting a research program to make certain that all seismic procedures are state-of-the-art, and they have developed performance criteria to assure that life safety is maintained for the maximum credible earthquake. However, more work needs to be done on the lifelines and economics issues. Road closures caused by the Loma Prieta earthquake resulted in significant impacts to the area's economies. The collapse of a section of the East Bay Bridge required commuters to take ferries or drive far south to the San Mateo Bridge or north to the Richmond Bridge to cross the Bay. The collapse of the Struve Slough Bridges on the Pacific Highway isolated the communities in that area and required a long detour. Likewise, landslides in the mountains and along the coast closed several highways and created economic impacts to those regions. Caltrans addresses economic issues with a second performance level that sets serviceability requirements for a more frequent event. However, the cost of developing a probability-based earthquake for the bridge site and the expense of designing bridges for a much higher performance level has resulted in its use only on important bridges. Unfortunately, politics or the cost of a retrofit often determines a bridge's importance rather than a detailed risk assessment.

Therefore, Caltrans may be dealing more effectively with technical rather than the social and economic issues following this earthquake. These technical issues include the following:

1. The behavior of connections and particularly how shear is transferred at reinforced concrete joints has become a major issue in research and in many new retrofits. Immediately after the earthquake the solution for column to superstructure joints was to provide so much reinforcement in the joint that they became very expensive and difficult to build. A testing program has resulted in an empirical method for joint design that should give good performance during large earthquakes. Outrigger knee joints are still an area of concern. For new and retrofit designs, Caltrans has been using pinned connections wherever possible or providing edge beams to reduce the torsion in outrigger bent caps.

2. A more sophisticated determination of site-specific ground motions using the shear-wave velocity of the soil, the peak ground acceleration at the site, and the magnitude of the event is now used in bridge design.

3. Geotechnical considerations have assumed much greater importance following Loma Prieta. More realistic soil stiffnesses at foundations and abutments and site-specific ground motions at locations with problem soils are now standard. Concerns with liquefaction have prompted soil remediation at several bridge sites.

4. More sophisticated analysis procedures including the use of nonlinear analysis for important bridge projects are now routinely done at Caltrans.

5. More sophisticated retrofit strategies that consider the stiffness, strength, and ductility of bridge elements and systems to keep capacities above demands are routinely done.

Highways are an important part of American life. They are used to get to work, to transport goods, to carry emergency vehicles, and to bring equipment to damaged lifelines. And like other lifelines, highway performance can be severely diminished as a result of large earthquakes. The Loma Prieta earthquake was a moderate event and yet it had a large impact on highways throughout the Bay Area. Those responsible for highways in areas of high seismicity may do well to consider the lessons of the Loma Prieta earthquake. The main lesson is that by reducing the vulnerability of highway facilities from damage, human lives, other lifelines, and the economies of communities can be protected.

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