POST EARTHQUAKE INVESTIGATION TEAM (PEQIT) MANUAL

AUGUST 2010

CSMIP INSTRUMENTED BRIDGES

	Station Name	Station No.	Bridge No.	Post Mile	Const.	No. of	Instr. Date
1	Klamath - Hwy 101/Klamath River Bridge	99710	01-28	01-DN-101-R4.04	1965	6	4/13/1995
2	Rio Dell - Hwy 101/Eel River Bridge	89973	04-16R	01-HUM-101-53.9	1941	18(15+3FF)	8/9/2001
3	Arcata - Hwy 101/Murray Road Bridge	89708	04-170	01-HUM-101-R92.99	1964	12(9+3FF)	4/6/1995
4	Eureka - Samoa Channel Bridge	89686	04-228	01-HUM-255-1.2	1971	33 (30+FF)	4/12/1996
5	Eureka - Middle Channel Bridge	89735	04-229	01-HUM-255-0.7	1971	17 (14+FF)	4/12/1996
6	Eureka - Eureka Channel Bridge	89736	04-230	01-HUM-255-0.2	1971	27 (24+FF)	4/9/1996
7	Rio Dell - Hwy 101/Painter Street Overpass	89324	04-236	01-HUM-101-R52.89	1976	20(17+3FF)	9/29/1977
8	Redding - I5/Shasta Lake Bridge	88194	06-21	02-SHA-5-R28.4	1940	18(15+3FF)	9/25/2002
9	Albion - Hwy 1/Salmon Creek Bridge	79683	10-134	01-MEN-1-43.00	1951	9(6+3FF)	3/17/1994
10	Fort Bragg - Hwy 1/Noyo River Bridge	79296	10-176	01-MEN-1-96.9	2006	24	4/4/2007
11	Leggett - Big Dann Creek Bridge	79761	10-30	01-MEN-271-4.9	1933	12(9+3FF)	2/27/1998
12	Leggett - Cedar Creek Bridge	79757	10-31	01-MEN-271-5.2	1933	12(9+3FF)	2/27/1998
13	Hopland - Hwy 101/Railroad Bridge	69760	10-81	01-MEN-101-R9.53	1966	16(13+3FF)	5/22/1997
14	Crescent City - Hwy 101/Smith Rvr Bridge	99190	1-20	01-DN-101-36.06		6	10/28/2003
15	Truckee - I80/Truckee River Bridge	76741	17-58L	03-NEV-80-20.23	1989	8(5+3FF)	10/24/1995
16	Jenner - Hwy 1/Russian River Bridge	69671	20-195	04-SON-1-19.72	1984	6	9/29/1993
17	Rohnert Park - Hwy 101 Bridge	68717	20-235	04-SON-101-13.88	1973	12(9+3FF)	5/3/1995
19	West Sacramento - I80/Yolo Causeway	67972	22-45	03-YOL-80-5.6	1962	12(9+3FF)	6/1/2000
20	Vallejo - Hwy 37/Napa River Bridge	68065	23-64	10-SOL-37-R7.39	1967	15(12+3FF)	6/14/2001
21	Novato - Hwy 37/Petaluma River Bridge	68778	27-13	04-MRN-37-14.47	1958	12(9+3FF)	3/5/1999
22	Oakland - Alameda Webster Tube	58139	33-106L	04-ALA-260-R1.20/R1.84		12	10/7/2003
23	Oakland - Alameda Posey Tube	58138	33-106R	04-ALA-260-R1.12/R1.87		12	10/7/2003
24	Hayward - Hwy 580/238 Interchange Bridge	58658	33-214L	04-ALA-580-30.80	1988	10(7+3FF)	6/11/1993
25	Oakland - Hwy 580/24 Interchange Bridge	58657	33-302H	04-ALA-580-45.23	1970	6	5/20/93
26	Oakland - Hwy 580/13 Interchange Bridge	58656	33-347S	04-ALA-580-R39.15	1965	6	5/26/93
27	Santa Clara- Hwy 237/Alviso Overpass	57748	34-470L/K	04-SCL-237-6.10	1994	21(18+3FF)	10/25/1995
28	Pacifica - Hwy 1/Devils Slide Bridge	58414	35-0331R		2008	27	9/15/2008
29	South San Francisco - Sierra Point Overpass	58536	35-130	04-SM-101-23.7	1957	16(13+3FF)	12/5/1985
30	Belmont - I280 Pedestrian Bridge	58678	35-285	04-SM-280-10.56	1973	6	11/19/1993
31	Half Moon Bay - Hwy 1/Tunitas Cr. Bridge	58754	35-31	04-SM-1-20.82	1962	9(6+3FF)	5/22/1997
32	Watsonville - Hwy 1/Struve Slough Bridge	47707	36-88R	04-SCR-1-R1.59	1990	9(6+3FF)	11/23/94
33	San Juan Bautista - Hwy 101/156 Overpass	47315	43-31E	05-SBT-156-0.00	1958	15(12+3FF)	5/77 & 6/02
34	Big Sur - Hwy 1/Pfeiffer Canyon Bridge	47729	44-60	05-MON-1-45.5	1968	18(15+3FF)	4/3/1996

PEQIT investigators should be aware of bridges with special features such as those with accelerometers. Your observations may be the only correlation available between recorded performance and the condition of the bridge. Take extra time to record everything of relevance to instrumented bridges.

CSMIP INSTRUMENTED BRIDGES CONTINUED

	Station Name	Station No.	Bridge No.	Post Mile	Const. Date	No. of Sensors	Instr. Date
35	Lake Crowley - Hwy 395 Bridge	54730	47-48	09-MNO-395-13.9	1969	9(6+3FF)	8/30/1995
36	Parkfield - Hwy 46/Cholame Creek Bridge	36668	49-36	05-SLO-46-54.77	1979	6	8/4/1993
37	San Simeon - Hwy 1/San Simeon Creek Bridge	37728	49-46	05-SLO-1-52.92	1984	12(9+3FF)	9/6/1995
38	Grapevine - I5/lebec Rd Bridge	24775	50-271	06-KER-5-4.1	1964	16(13+3FF)	6/30/1998
39	Ridgecrest - Hwy 395/Brown Road Bridge	33742	50-340	09-KER-395-R25.08	1966	9(6+3FF)	2/22/1996
40	Mojave - Hwy 14/Railroad Bridge	34715	50-402R	09-KER-14-15.32	1973	12	3/22/1995
41	Santa Barbara - San Roque Canyon Birgde	25749	51-104	05-SB-192-1.77	1984	9(6+3FF)	10/24/1996
42	Cuyama - Hwy 166/Cuyama River Bridge	25758	51-66	05-SB-166-R69.94	1980	12(9+3FF)	4/8/1997
43	Lompoc - Hwy 1/El Jaro Creek Bridge	26917	51-90	05-SB-01-R6.78	1968	11(8+3FF)	5/18/2000
44	Ventura - Hwy 101/Telephone Rd Bridge	25725	52-214L	07-VEN-101-R26	1961	12(9+3FF)	5/5/1995
45	Moorpark - Hwy 23/118 Bridge (Arroyo Simi)	24738	52-331L	07-VEN-023/118-21.0	1993	12(9+3FF)	5/8/1996
46	Los Angeles - I405/San Gabriel River Bridge	14690	53-1185	07-LA-405-0.02	1964	6	4/27/1994
47	Los Angeles - I10/405 Interchange Bridge	24670	53-1630G	07-LA-405-29.43	1963	7	9/13/1993
48	Palmdale - Hwy 14/Barrel Springs Bridge	24706	53-1794	07-LA-14-R57.37	1965	12(9+3FF)	12/8/1994
49	San Fernando - I210/Hwy 118 Bridge	24714	53-2102G	07-LA-118/210-6.0	1973	36(33+3FF)	4/17/1996
50	Pasadena - Hwy 134/210 Interchange Bridge	24689	53-2318G	07-LA-134-R13.25	1974	9(6+3FF)	4/21/1994
51	Los Angeles - I10/La Cienega Bridge	24704	53-2791	07-LA-10-8.8	1994	15	11/2/1994
52	Sylmar - I5/14 Interchange Bridge	24694	53-2795/97F	07-LA-5-24.5	1994	42(39+3FF)	12/20/1995
53	Devore - I15/215 Interchange Bridge	23650	54-783R	08-SBD-15-16.35	1969	6	12/18/92
54	San Bernandino - I10/215 Interchange	23631	54-823G	08-SBD-215-4.05	1966	37(34+3FF)	1/10/1992
55	Capistrano Beach - I5/Via California Bridge	13795	55-225	07-ORA-5-6.62	1960	12(9+3FF)	6/24/1999
56	Beaumont - I10/60 Interchange Bridge	12649	56-452F	08-RIV-10-6.67	1961	6	12/16/92
57	North Palm Springs - I10/62 Interchange Bridge	12666	56-474F	08-RIV-62-0.00	1962	7	6/30/93
58	Corona - I15/Hwy 91 Interchange Bridge	13705	56-586G	08-RIV-15-R41.57	1989	9	9/29/1994
59	San Diego - I5/Hwy 52 Interchange Bridge	03731	57-520L	11-SD-5-25.91	1966	24(21+3FF)	5/18/1995
60	El Centro - Hwy 8/Meloland Overpass	01336	58-215	11-IMP-8-43.6	1971	32(29+3FF)	4/78 & 12/91
61	Oakland - Caldecott Tunnel	58359				24	1984
62	Oxnard - Hwy 101/Santa Clara Rvr Bridge	25324				15	6/6/2007
63	Vallejo - Carquinez Bridge Wind Motion	68268			2003	12	2003

Instrumented Toll Bridges

	Name of Bridge	Station No.	Type and Length of Main Structure	Total Length (mile)	Year Built	No of Sensors.	Associated Geotechical Array	Instrum. Date
1	Antioch - San Joaquin River Bridge	67771	composite steel girder, 9437 ft	1.8	1978	15+3FF	1 array, 15 sensors	4/16/1998
2	Benicia - Martinez/I680 East Bridge	68322	concrete box girder, 2066 m	1.3	2008	95	2 arrays	1/17/2008
3	Benicia - Martinez/I680 West Bridge	68682	steel truss, 4884 ft	1.2	1962	78+3FF	none	10/11/2001
4	Hayward - San Mateo Bridge Trestle	58799	concrete girder, 30 ft	5.0	1967	27	1 array, 15 sensors	5/19/1999
5	San Mateo - San Mateo Bridge	58677	steel box girder, 750 ft	1.8	1967	64	see trestle	4/27/2000
6	Los Angeles - Vincent Thomas Bridge	14406	suspension, steel towers, 2512 ft	1.1	1964	26	3 arrays, 33 sensors	10/22/1981
7	Richmond - San Rafael Bridge	58258	2 steel truss cantilever spans, 2145 ft each	4.0	1956	82+3FF	1 array, 6 sensors	8/15/2005
7	San Diego - Coronado Bridge	03679	steel box girder, 1880 ft	1.6	1969	72	2 arrays, 27 sensors	9/25/2002
8	San Francisco - Oakland Bay East Bridge	58633	self-anchored suspension, steel tower, 622 m	2.2	under construction	9	1 array, planned	(199 in progress)
9	San Francisco - Oakland Bay West Bridge	58632	suspension, steel towers, 10051 ft	2.0	1936	79	1 array, 9 sensors	8/27/2003
10	San Francisco Bay - Dumbarton Bridge	58596	steel girder, 3150 ft	1.6	1982	26+6FF	none	6/10/1987
12	Vallejo - Carquinez/I80 East Bridge	68184	steel truss, 3350 ft	1.0	1958	79	1 array, 9 sensors	6/1/2002
13	Vallejo - Carquinez/I80 West Bridge	18185	suspension, steel box girder, concrete towers, 1056 m	1.0	2003	103	1 array, 9 sensors	10/3/2003

Downhole Arrays

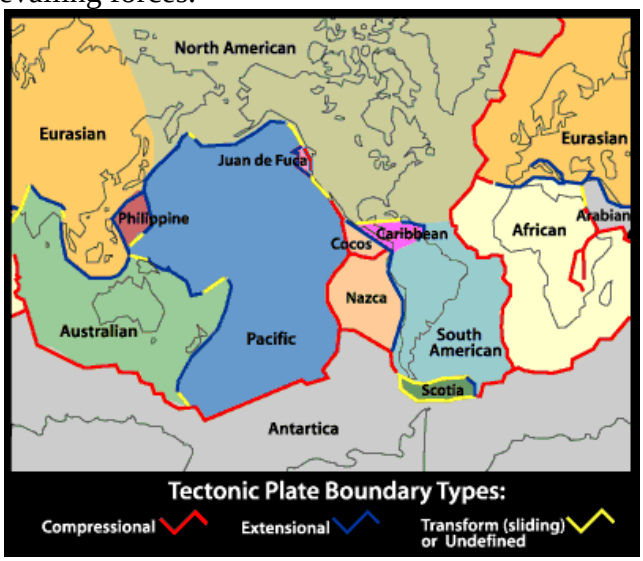
	Station Name	Station No.	No. of Sensors	No. of Depths	Sensor Depths (m)	Site Geology
1	Alameda - Posey & Webster Geotech Array	58137	12	4	Surface, 6, 13, 40	Deep alluvium
2	Antioch - San Joaquin River S. Geo. Array	67266	15	5	Surface, 11, 24, 51, 102	Deep alluvium
3	Benecia-Martinez Br., No Geotech Array	68321	9	2	Surface, 16, 39	Shallow fill over bay mud
4	Benecia-Martinez Br., So.Geotech Array	68323	9	2	Surface, 11, 35	Thin Alluvium over soft rock
5	Corona - I15/Hwy91 Geotech Array	13186	12	4	Surface, 7, 24, 46	Shallow alluvium over hard rock
6	Crockett – Carquinez Bridge Geotech Array #1	68206	9	3	Surface, 20, 46	Shallow clay over sed. rock
7	Crockett – Carquinez Bridge Geotech Array #2	68259	9	3	Surface, 61, 125	Shallow clay over soft rock
8	El Centro - Meloland Geotechnical Array	01794	12	4	Surface, 30, 100, 195	Deep alluvium
9	Eureka - Geotechnical Array	89734	15	5	Surface, 19, 33, 56, 136	Deep soft alluvium
10	Foster City – San Mateo Bridge Geotech Array	58968	12	4	Surface, 16, 22, 35	Shallow alluvium over rock
11	Half Moon Bay – Tunitas Geotech Array	58964	12	4	Surface, 5, 12, 45	Alluvium over soft rock
12	Hayward - San Mateo Br Geotech Array	58798	15	5	Surface, 10, 23, 46, 91	Deep alluvium
13	Los Angeles - La Cienega Geotech Array	24703	12	4	Surface, 18, 100, 252	Deep alluvium
14	Los Angeles - Vincent Thos Geo Array East	14785	12	4	Surface, 18, 46, 91	Deep alluvium
15	Los Angeles - Vincent Thos Geo Array W1	14783	12	4	Surface, 30, 91, 189	Deep alluvium
16	Los Angeles - Vincent Thos Geo Array W2	14784	9	3	Surface, 15, 30	Deep alluvium
17	Moorpark - Hwy118/Arroyo Simi Geotech Array	24185	9	3	Surface, 20, 47	Shallow alluvium over sed. rock
18	Novato - Petaluma River Geotech Array	68285	15	5	Surface, 10, 23, 54, 73	Alluvium

EARTHQUAKES

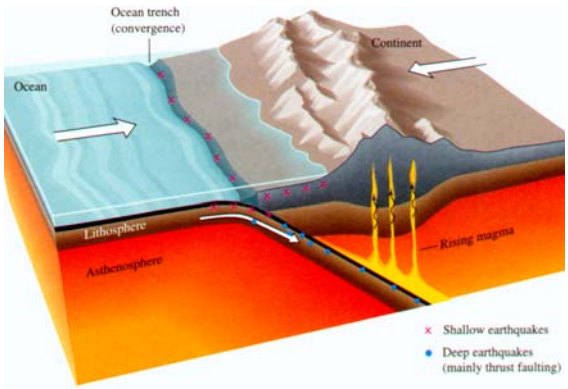
Sources of Earthquakes

Most earthquakes occur on faults at the boundaries of tectonic plates, especially in California. As the plates move, the faults become stressed until one suddenly ruptures, causing an earthquake. Plates move a few millimeters to centimeters in a year and it requires a few centimeters to meters of movement before an earthquake occurs.

There are different types of faults, reflecting the behavior of the prevailing tectonic forces. The figure below shows the earth's lithosphere divided into tectonic plates. The plate boundaries may be pushing together (compression or convergent boundaries), pulling apart (divergent or extensional boundaries), or sliding against each other (sliding or transform boundaries). Faults do not always follow the expected behavior from the global tectonic model due to significant local variation of prevailing forces.

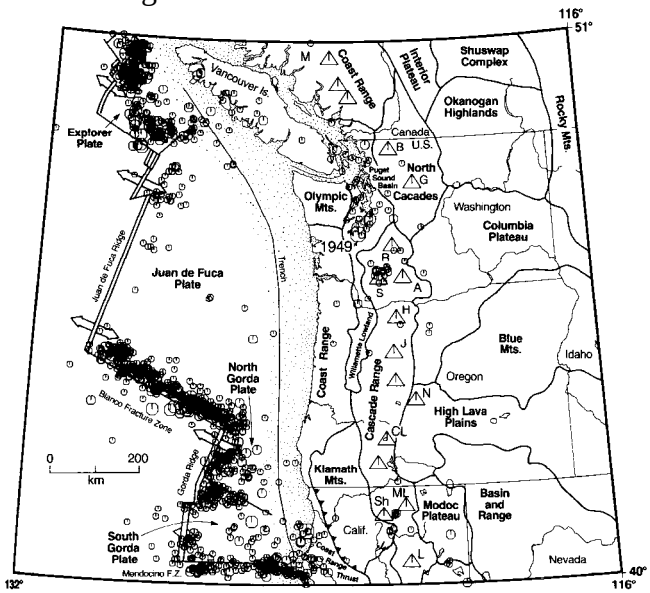


Compressional plate boundaries are where heavy oceanic plates collide with lighter continental plates; pushing the oceanic plate downward and causing thrust faulting. Compression against the continental plate forms mountains and the ensuing friction melts rock causing volcanoes. An example is along the coast of Northern California, Oregon, and Washington State where the Juan de Fuca and Gorda Plates are being pushed under the North American Plate.

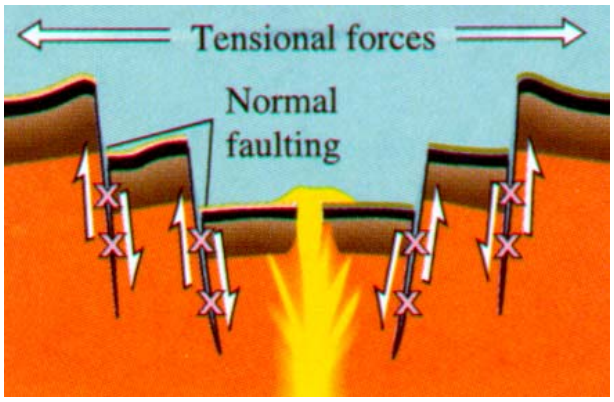


Compressional boundaries cause thrust faults.

This has created a string of volcanoes (shown with triangles in the figure below) along the Cascadia Mountain Range and it has also caused a series of moderate earthquakes (shown with circles). Globally, earthquakes in these 'subduction-zones' are responsible for about 90% of the seismic energy released by earthquakes and it is believed that the Cascadia Subduction Zone is capable of producing a very large earthquake which could devastate the Northwestern United States. However, this is an area of very low seismic activity for California, but the potential of a large earthquake with an MCE magnitude of 8.5 exists and is considered by Caltrans.



Extensional boundaries are where adjacent plates move apart. This occurs at spreading ridges between oceanic plates and at rift zones between continental plates. An example of an extensional boundary is on the west side of the Gorda and Juan de Fuca plates (shown above). Earthquakes occurring at extensional boundaries are shallow and smaller than at compressional boundaries.

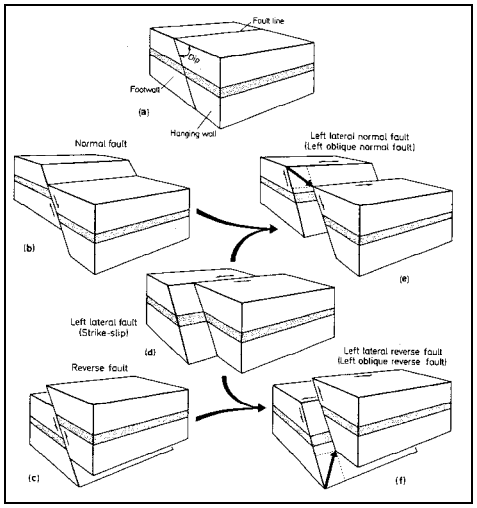


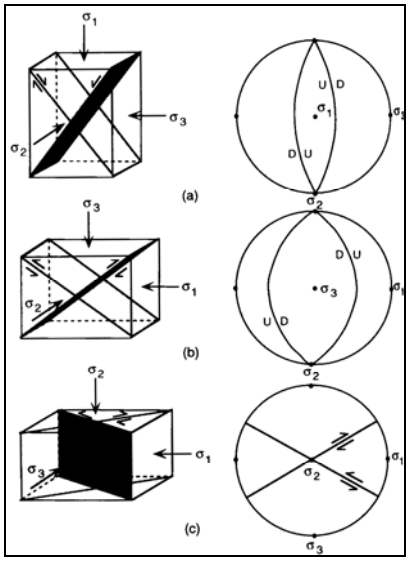
Extensional boundaries cause normal faults.

In California we are most familiar with the sliding action of the transform boundary between the Pacific and North American Plates. This has created many long strike-slip faults such as the San Andreas. We can also see transform fault zones between the extensional faults on the west side of the Juan de Fuca and Gorda fault ridges. During actual earthquakes, faulting can be considerably more complicated depending on the state of stress in the earth's crust. The figures below show how normal faulting (b) can be combined with strike-slip faulting (d) to create an oblique normal fault. Similarly, reverse (or thrust) faulting (c) can be combined with strike-slip faulting to cause an oblique reverse (or thrust) fault. Faults are left or right lateral based on the movement of the rock on the other side of the fault from where an observer is standing. Figure (a) shows some of the features of a fault, such as the fault line (or trace), the fault dip, the footwall extending under the dipping fault, and the hanging wall extending above the fault (a thrust fault is a reverse fault where the dip angle is less than 45° and the fault line often does not reach the ground surface).

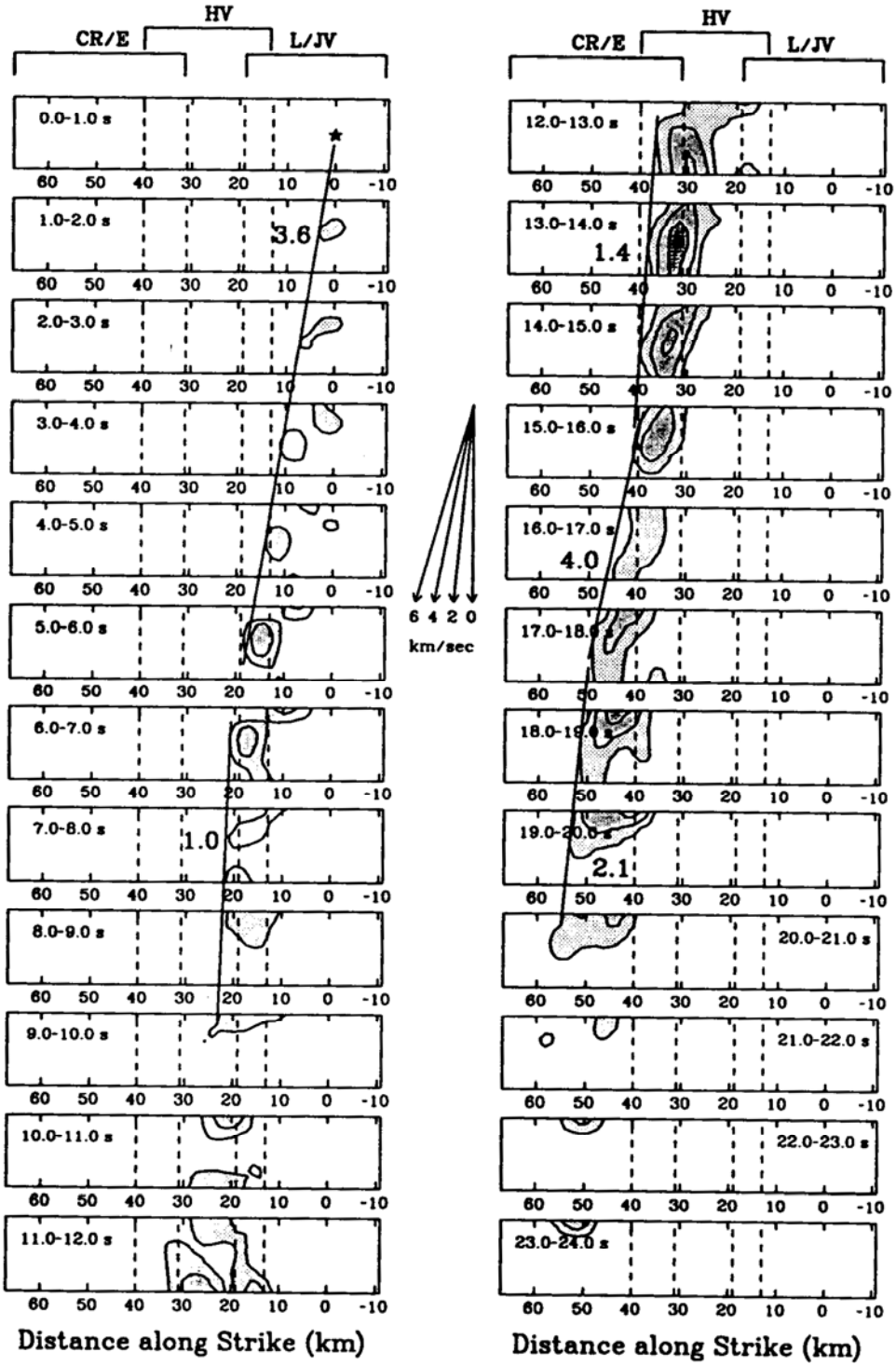
The drawing on the right shows stress blocks and stereographic projections of the maximum shear-planes in relation to principal stresses for a normal (a), reverse (b), and strike-slip (c) fault. The normal fault (with the hanging wall moving downward) occurs when the maximum principal compressive stress is vertical while a reverse fault (with the hanging wall moving upward) occurs when the minimum principal stress is vertical. However, pre-existing planes of weakness usually determine the location of the fault-plane rather than these models of isotropic material. However, the next section (on seismic waves) shows some additional uses for these 'beach-ball' diagrams.

Transform boundary and strike-slip fault.

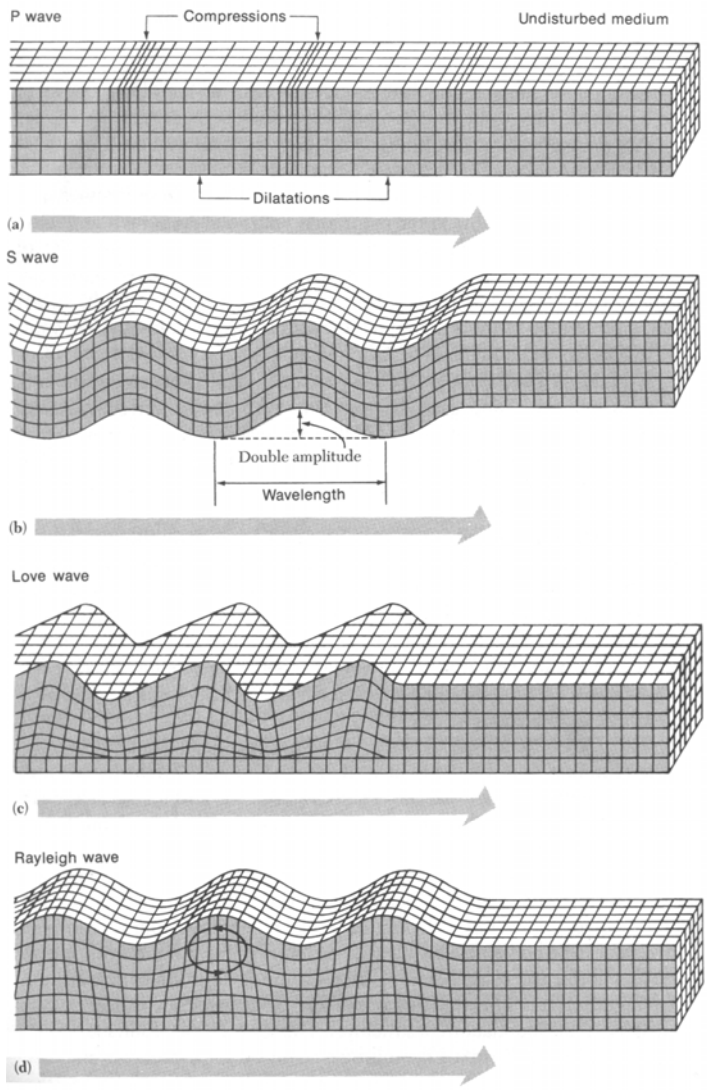




A rupture will suddenly relieve the accumulated stress at a fault and cause an earthquake. The direction and velocity of the rupture has gained considerable attention to engineers because these 'velocity pulses' can cause very large, long period motion that may knock over a bridge. The figure below shows the movement of a rupture 'front' along the Camp Rock/ Emerson Faults to the Homestead Valley Fault to the Landers/Johnson Valley Faults during the 1992 Landers, California earthquake.



Seismic Waves

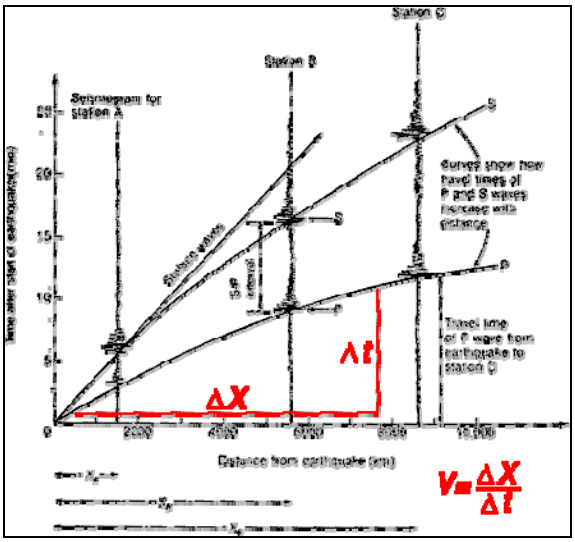


The fault rupture produces a series of waves moving out from the source. These waves cause the ground motion we feel during an earthquake. The compressive (or P) wave is the fastest wave at speeds between 1.5 and 8 kilometers per second through rock. It alternately pushes and pulls the ground in the direction of its motion. Next come the slower shear (or S) waves moving perpendicular to the direction of motion. P and S waves are called body waves because they propagate in a body of rock.

There are two slower waves called surface waves because they're confined to the ground surface. Love waves are similar to shear waves but without a vertical component of motion and the slowest Rayleigh waves move similar to ocean waves vertically and longitudinally in a retrograde motion along a vertical plane in the direction of travel.

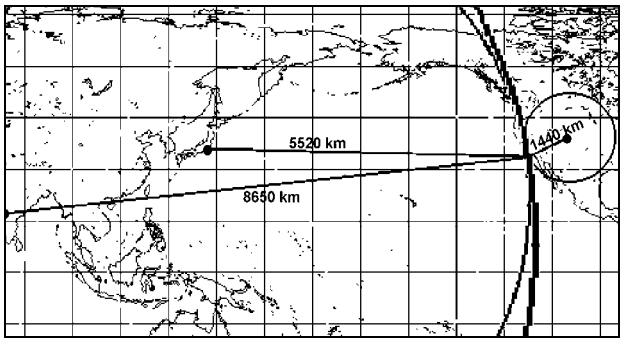
All of these waves, but particularly the shear waves, can damage structures. In fact, the worst damage occurs when geologic or topographic conditions amplify the waves at a bridge site. Moreover, the length and frequency of the waves plays a role in the characteristics of the ground motion. For instance, long bridges are damaged when adjacent bents are at the peak and trough of a wave resulting in out-of-phase motion. Certain soils are sensitive to long or short shear-wave velocities and can create resonance for structures with similar sensitivities.

Although wave speeds vary, the ratio between the average speed of a P wave and the following S wave is approximately constant. This fact enables seismologists to use the delay between the arrival of the P wave and the arrival of the S wave to get a quick estimate of the distance of the earthquake from the observation station. By multiplying the S-minus-P (S-P) time, in seconds by the factor 8 km/sec (5 miles/sec) will provide the approximate distance in kilometers from the earthquake source.

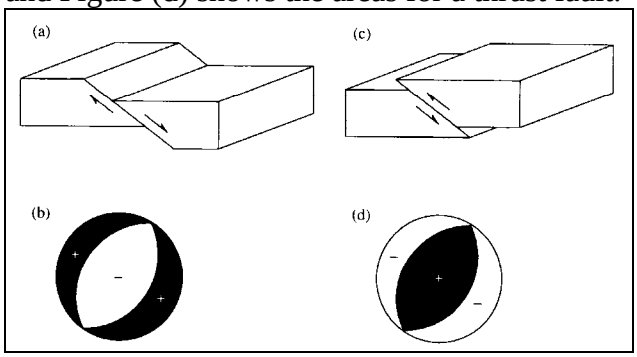


For instance, Station A has a separation of three minutes or 180 seconds between the P and S wave for a distance of 1440 km. Station B has 11.5 minutes or 690 seconds between the P and S waves for a distance of 5520 km form the source. Station C has 18 minutes or 1080 seconds between the arrival of the P and the S wave for a distance of 8650 km from the source. We can precisely locate the source of an earthquake from three recordings by drawing circles using the distances computed above as radii. Where the three circles intersect is the location of the earthquake.

For instance, if Station A is in Denver, Colorado, Station B is in Tokyo, Japan, and Station C is in New Delhi, India the intersection of circles drawn around these locations (and with radii derived above) would be Los Angeles, California. However, the error can be considerable, even for California's dense network.

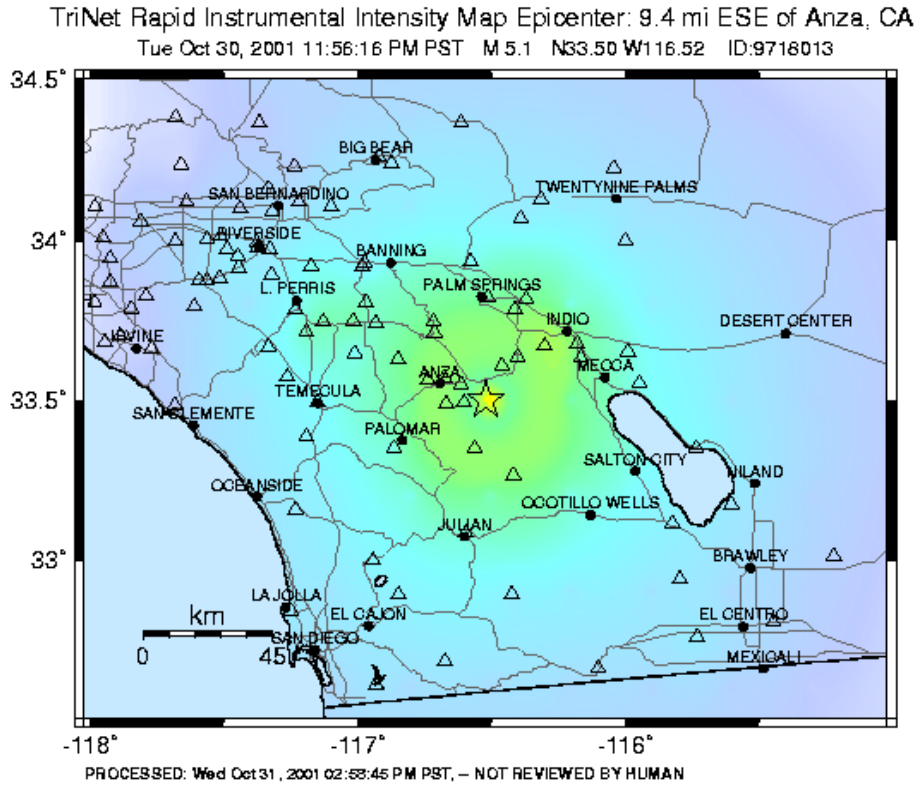


Seismograms have many other uses. For instance, a better way of deriving the stereographic projection of the fault plane is by mapping locations where the first P wave is dilatational (pulling) or compressional (pushing). Figure (b) shows dilation as white and compression as black for a normal fault and Figure (d) shows the areas for a thrust fault.



The Size of an Earthquake: Intensity and Magnitude

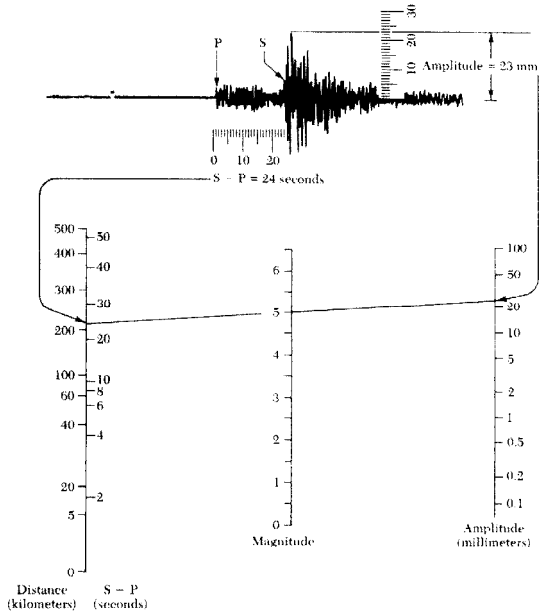
In the 19th century, the intensity of an earthquake was determined from the severity of damage at different sites. Isoseismal maps were created showing areas of equal intensity based on interviews with local residents and a survey of the damage. Intensity is measured today using the Modified Mercalli Scale that relates damage to the peak ground acceleration (as shown below). This is not a measure of the size but of the effects on buildings and on common structures.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

However, seismologists required a scale that could be used to compare the size of different earthquakes. Charles Richter developed the 'size' idea (in 1935) by using local magnitude (M_L) to measure earthquakes. Richter defined local magnitude as the base 10 logarithm of the maximum seismic wave amplitude (in thousandths of a millimeter) recorded on a standard seismograph at a distance of 100 kilometers from the earthquake epicenter. The standard seismograph had a mass suspended from a torsional spring that recorded the incoming waves as a function of time and of amplitude in millimeters. A logarithmic scale was used to make the magnitude a single digit number. Thus, when the amplitude of the waves increased by ten, the magnitude would increase one unit (from magnitude 4 to magnitude 5 - $10^5/10^4 = 10$). The following table shows how local magnitude is calculated from a seismograph of an earthquake but there are several modified or equivalent methods in use today.

Step 1: Measure the distance to the initial point of rupture from the time interval between S and P waves (S – P = 24 sec.).
Step 2: Measure the maximum wave amplitude on the seismograph (23 millimeters).
Step 3: Place a straight edge on the right using the previously derived distance (24 sec) and amplitude (23 mm) to obtain the local magnitude (M_L= 5.0).
Local magnitude was meant to measure earthquakes in Southern California. As acceptance of the magnitude scale grew, problems using the local magnitude were recognized by seismologists.



To obtain uniformity when measuring earthquakes, moment magnitude (M_W) was developed (shown below).

$$M_W = (2/3)[\log(M_o) - 16.05]$$

M_o is the equivalent seismic moment at the source as the two sides of the fault move in opposite directions during the earthquake

$$M_o = GAD \quad \text{(in dyne-cm)}$$

where 'G' is the shear modulus of the rock, 'A' is the area of the fault, and 'D' is the average slip or movement of the fault.
The maximum earthquake magnitude that can occur on a fault is related to the product of the fault length times its width (A, in cm^2) and the average slip (D, in cm). Caltrans uses the fault area and style of faulting in addition to other criteria to determine the moment magnitude of the maximum credible earthquake (MCE).

EARTHQUAKE INDUCED BRIDGE DAMAGE

Much of the bridge damage that occurs during earthquakes is a result of soil problems. Unstable soil can cause landslides and loose alluvium can settle or liquefy. Shaking of the bridge itself can cause failure of bridge members and connections or can cause excessive movement at expansion joints dropping bridge spans. There are also a variety of secondary effects such as surface ruptures occurring directly under a bridge, or tsunamis, avalanches, etc. During the 1958 Hebgen Lake earthquake in Montana, an enormous landslide changed the course of the Madison River causing floods that washed away several bridges. Bridges are sometimes damaged due to failure of nearby lifeline components. Broken water mains can wash away abutments, broken gas lines can incinerate bridges, and fallen buildings or other structures can damage and close bridges.
On the following pages are examples of typical types of earthquake induced damage that have occurred to bridges.

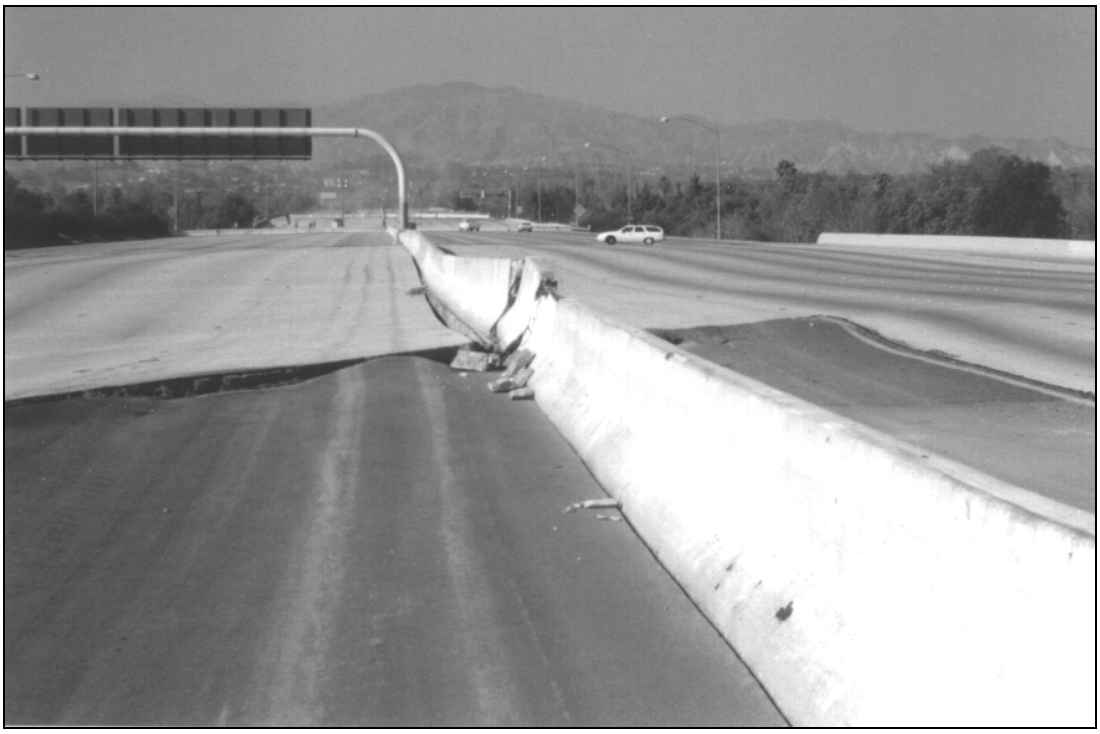
SOIL RELATED BRIDGE DAMAGE

Bridges Supported on Cohesionless Material.

Struve Slough Bridge after Loma Prieta.

Bridges Supported on Liquefiable Soils.

Rio Vascaya Bridge after 1991 Costa Rica Quake



Approach Settlement on I-118 after Northridge.

Bridges Supported on Sloping Ground

Landslide Damage in the Philippines.

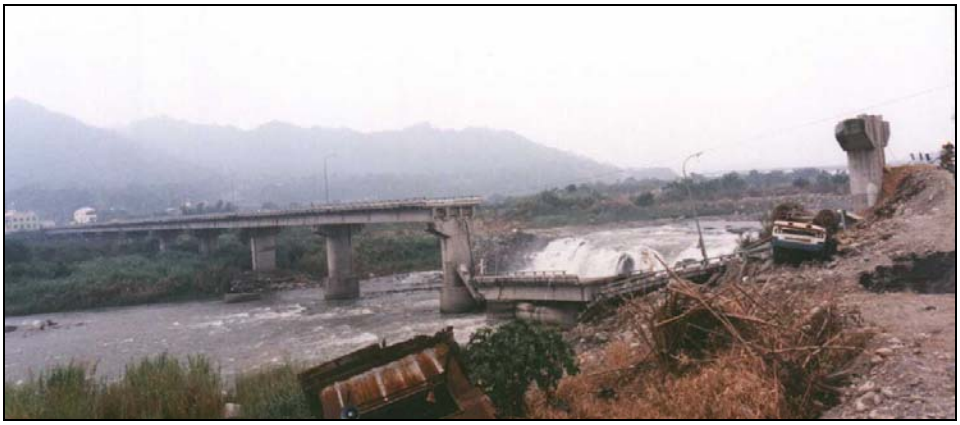
Foundation Damage

Foundation Movement at Kobe.

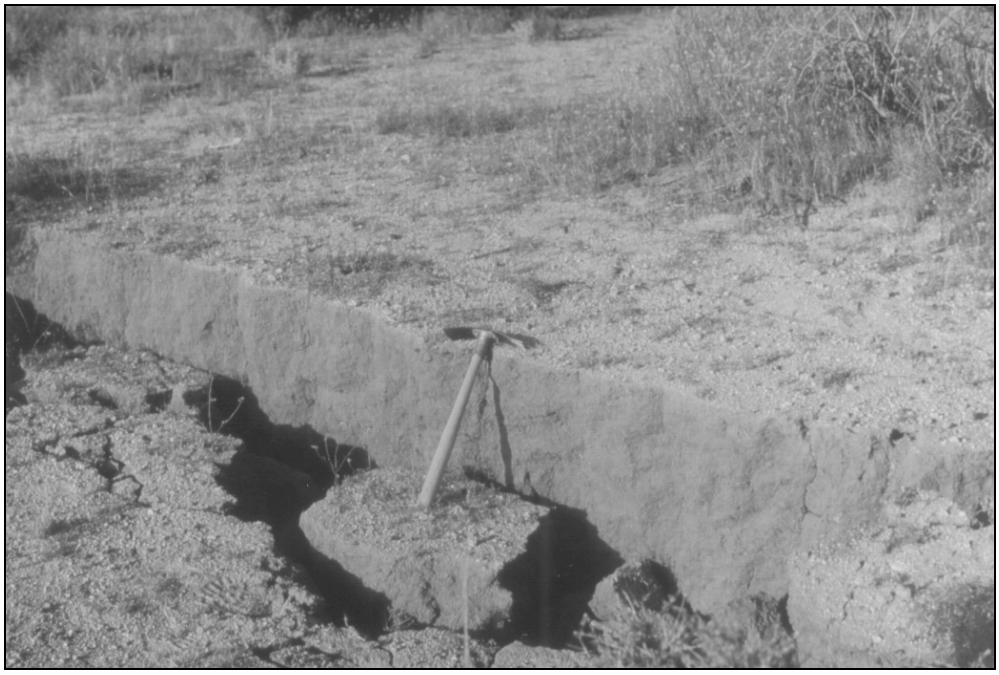
Abutment Damage

Tilting of Railroad Abutment in Kobe.

FAULT RUPTURE AND SURFACE DEFORMATION



Taiwan Fault Rupture.



Landers Fault Rupture.

Landers Surface Deformations

STRUCTURAL VIBRATION RELATED BRIDGE DAMAGE

Unseating

San Francisco Bay Bridge.

Minatogawa Interchange after Kobe.

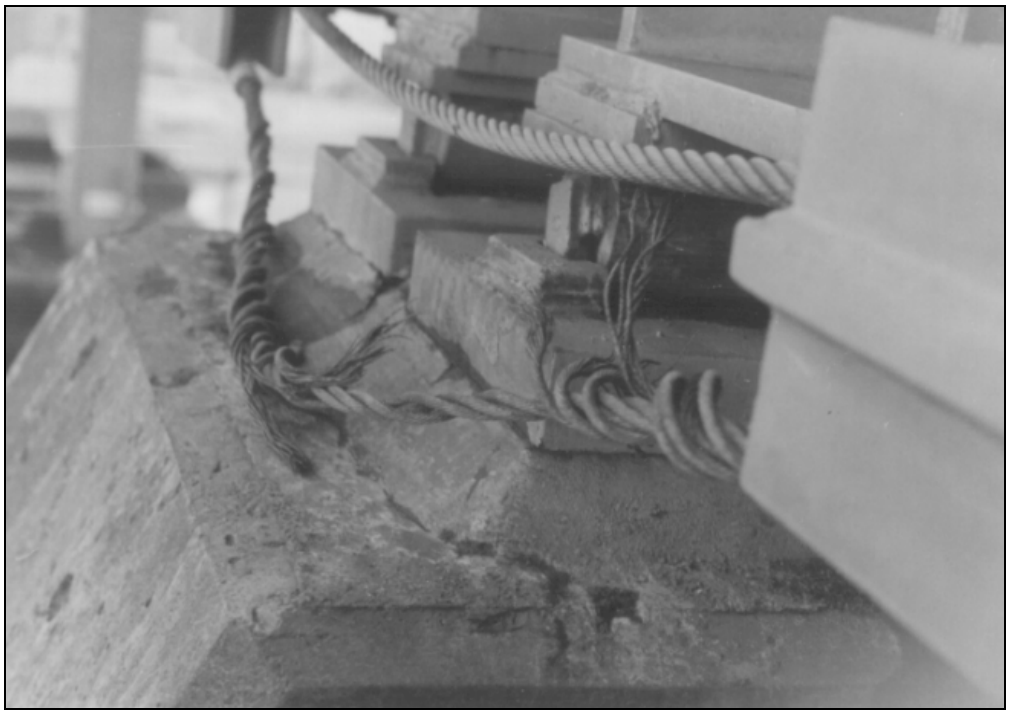
Mission Gothic UC after Northridge.

Gavin Canyon After Northridge.

Expansion Joint, Restrainer, and Shear Key Damage

Expansion Joint Damage.

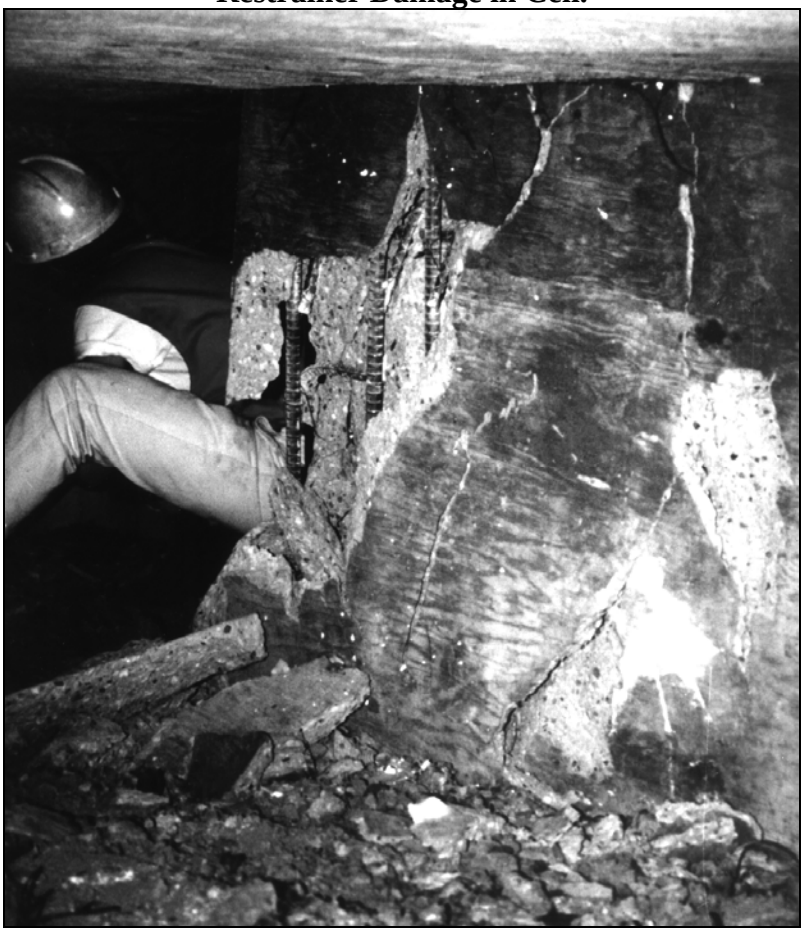
Restrainer Damage at Gavin Canyon.



Damaged Restrainer.

Broken Restrainers at Nishinomiya.

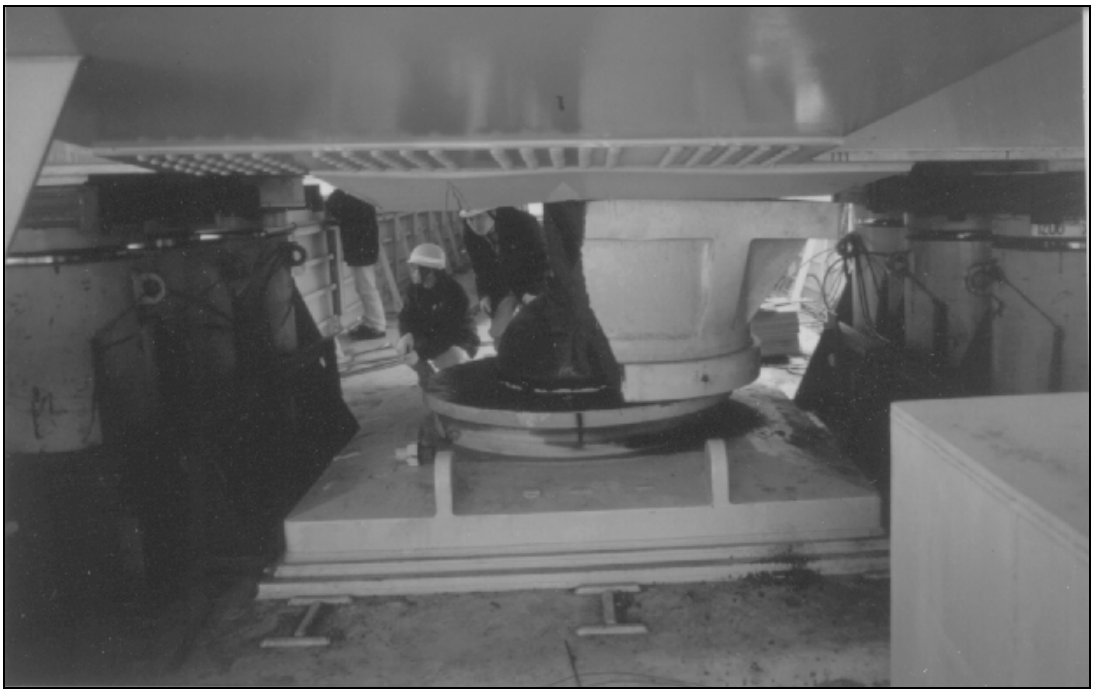
Restrainer Damage in Cell.



Shear Key Damage.

Bearing Damage

Missing Abutment Bearing After Northridge EQ.



Broken Bearing at Nishinomiya-ko Bridge.

Superstructure Damage

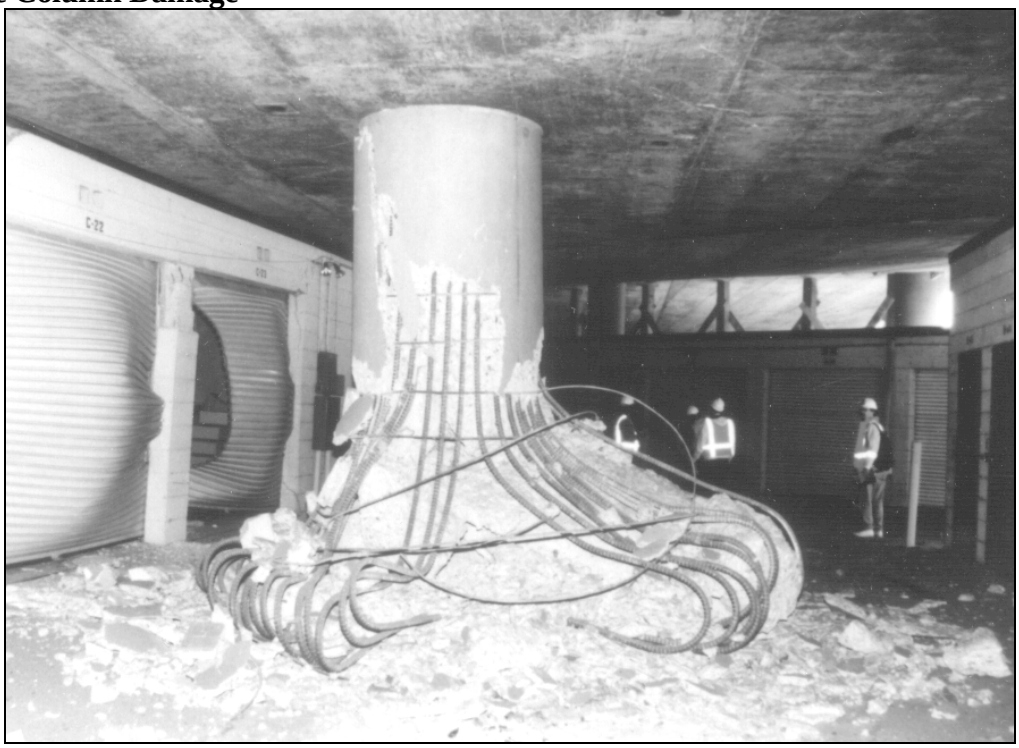
Superstructure Collapse on 14/5 Interchange during Northridge.

Damage at Takatori.

Napa River Bridge Precast Girder Damage.

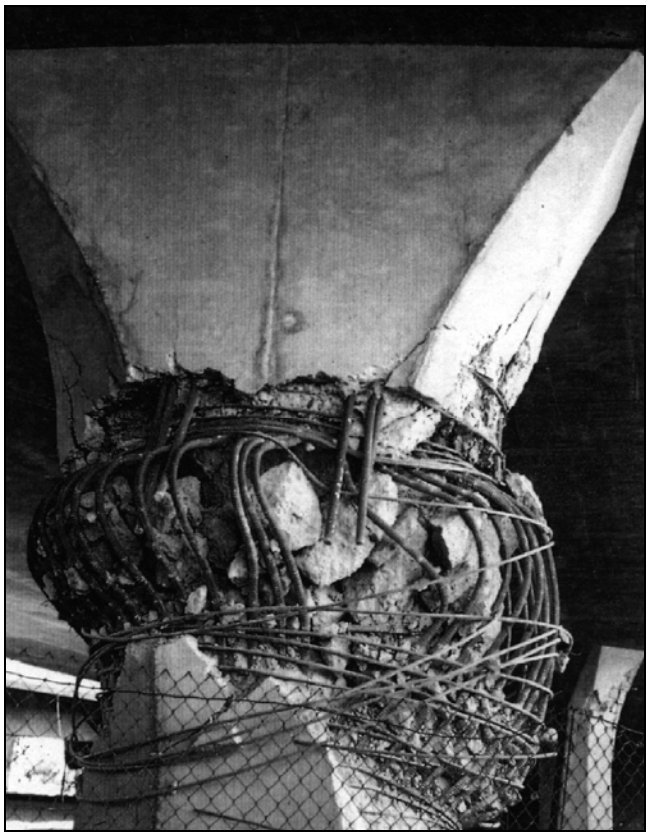
Damage to Rokko Island Bridge During the Kobe Earthquake.

Concrete Column Damage



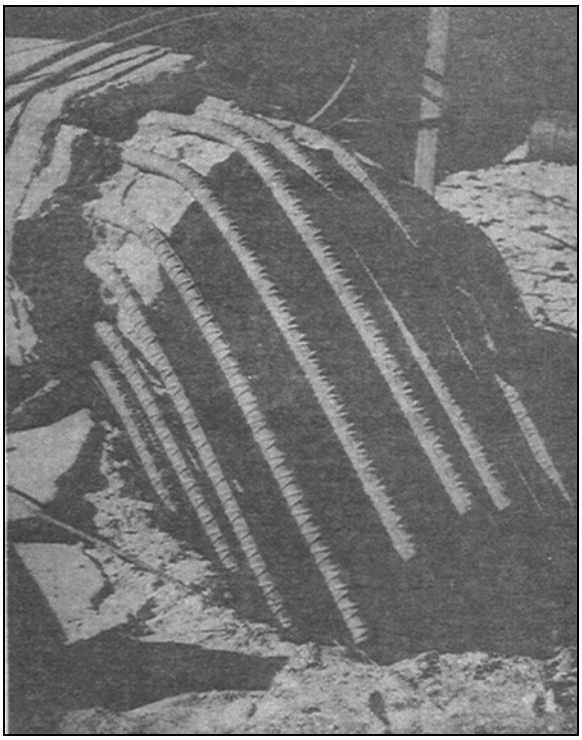
Damage at Santa Monica Fwy.

Minor Damage to Confined Bridge Column During Northridge EQ.



Column Flare Damage.

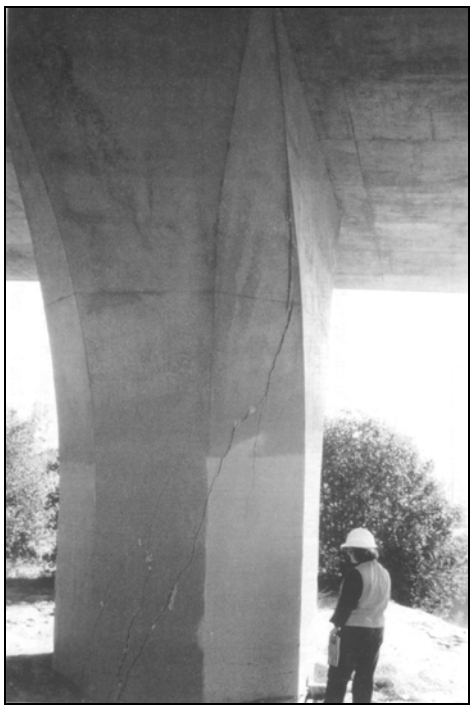
Midheight Flexural Damage.



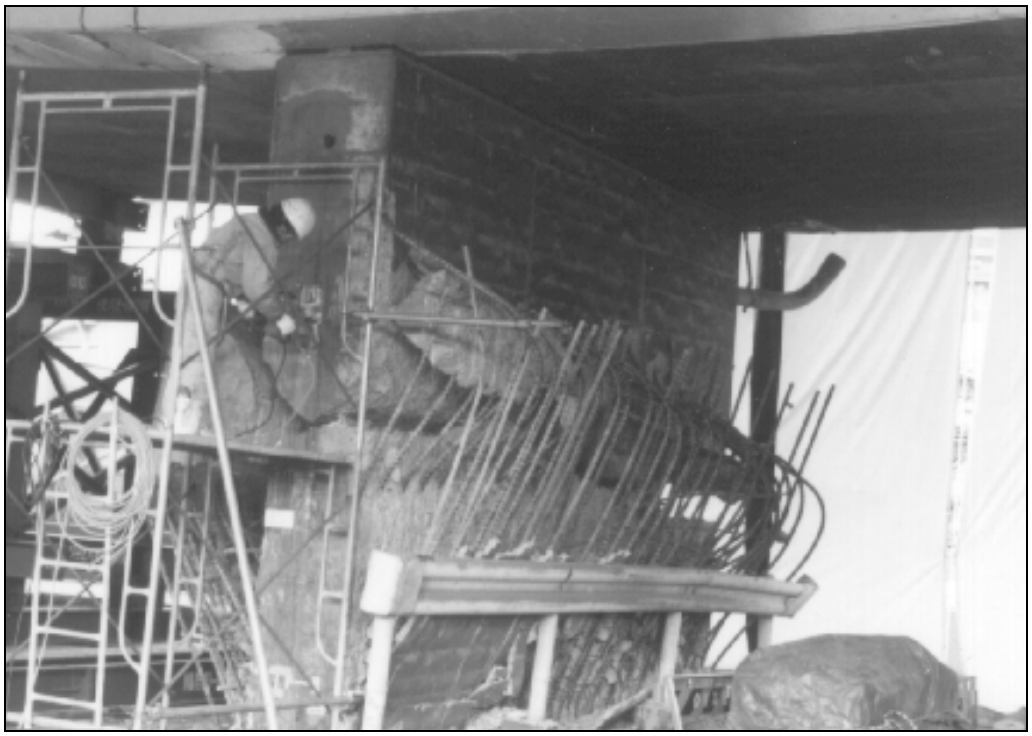
Longitudinal Bar Pullout.

Shear Column Damage During the Kobe Earthquake.

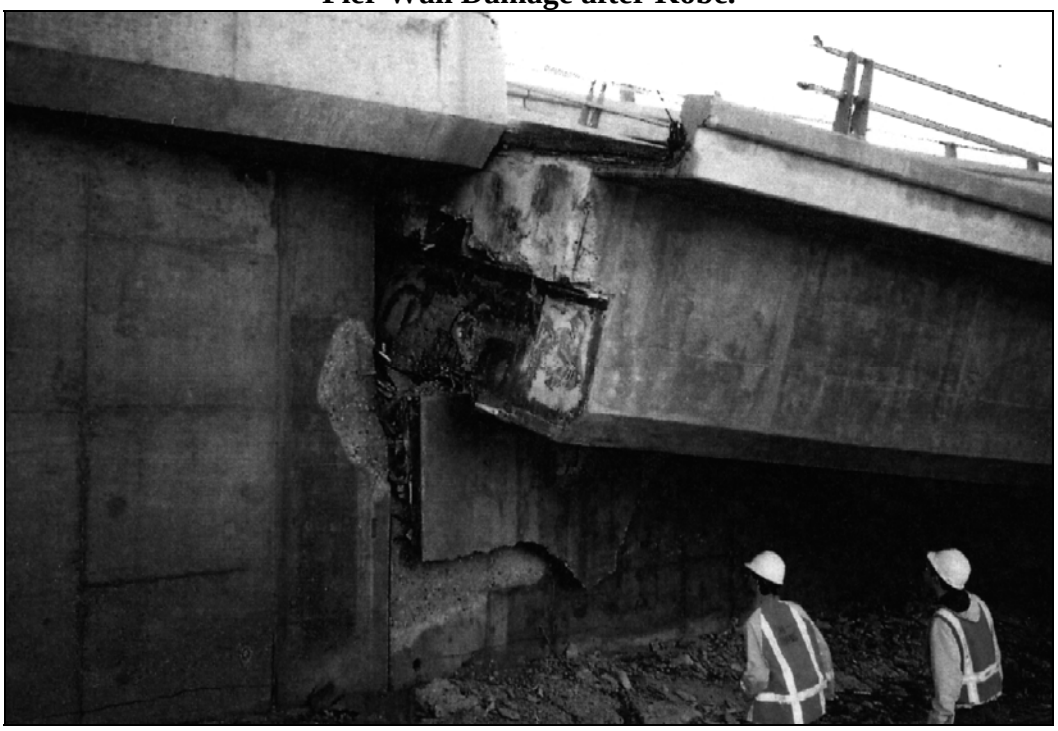
Poor Transverse Reinforcement.



Torsional Column Damage at 5/118 Interchange during the Northridge Earthquake.



Pier Wall Damage after Kobe.



The North Connector on the 14/5 Interchange Collapsed as a Result of Failure of Short, Stiff Pier #2 During the Northridge Earthquake.

Steel Column Damage

Steel Column Damage at Kobe.

Fifth Avenue Overhead after Loma Prieta.

Connection Damage

Joint Shear Damage.

Cypress Viaduct Damage.

Kobe Joint Damage

Pounding Damage

Southern Freeway Pounding Damage from Loma Prieta

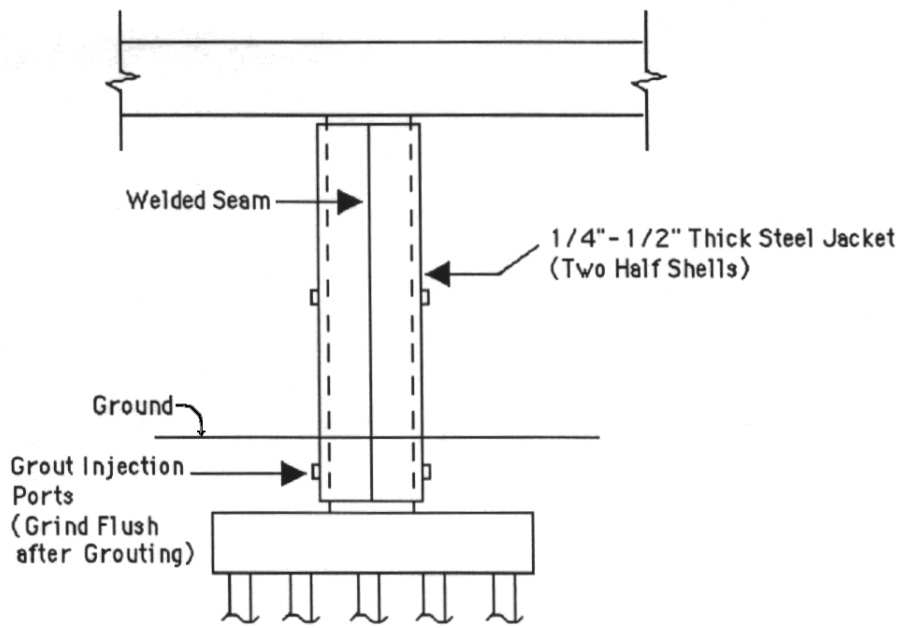
EVALUATING THE PERFORMANCE OF RETROFITTED BRIDGES

California has a large inventory of bridges retrofitted in the 1990's. There are columns wrapped in steel and fiber shells, hinges retrofitted with pipe-seat extenders, and abutments attached to large-diameter CIDH piles. One of the biggest responsibilities of the PEQIT is to evaluate the performance of retrofitted structures. Are there signs that the concrete columns have suffered damage inside the shells? Have the restrainers yielded (Structures maintenance is putting yield indicators on cable restrainers to alert engineers if the cables have yielded)? The PEQIT should carefully examine the retrofitted bridges to determine their performance. Typical (and not so typical) retrofits are shown below.

Caltrans Bridge Retrofit Program:

Not only are retrofits available for different bridge elements but they can be categorized by their ability to modify the strength, stiffness, ductility, or damping of bridge members. The following pages provide a sampling of some of these retrofits.

Bent Retrofits



Steel Column Casing Retrofit

The most common retrofit procedure for vulnerable columns is to wrap them in a steel shell. Steel shell retrofits are divided into two classes. A class F retrofit is when the space between the shell and the column is grouted to allow the columns to form plastic hinges during the earthquake. The class P retrofit is when the annulus is filled with compressible polystyrene that allows the columns to form pins. The class P retrofit protects the columns without the expense of having to design the entire bridge for the plastic column moment and shear. The class F retrofit greatly increase the ductility and shear capacity while adding only about 10% to the flexural strength of the columns. This has made them very popular and they are used in many situations. Sometimes, a short steel shell is just placed around a vulnerable area of the column. Steel shells are often used on rectangular columns that are so wide that the shells must be restrained with bolts or wide flange beams. Steel shells have even been used on nonprismatic columns and column flares.

Non-Prismatic Column with Steel Shell.

A variety of other materials have been developed to provide ductility to existing concrete bridge columns. The figure below shows a fiberglass wrapped column near Griffith park in Los Angeles after the Northridge earthquake. Composite material retrofits are now used in many of the same situations as steel shells. The one exception being that if a plastic hinge is required in a region with a lap splice, only steel shells provide enough confinement to prevent slip. However, composite fibers can safely be used when a class P retrofit is needed.

Griffith Park OC with Fiberglass Wrapped Concrete Columns.

There are many other materials and techniques used to provide confinement and increased ductility for concrete columns. These include carbon-fiber wrapped columns, wire wrapped columns, and concrete encased columns. However, because of unfamiliarity with some of these materials, a cautious approach should be adopted in their use.

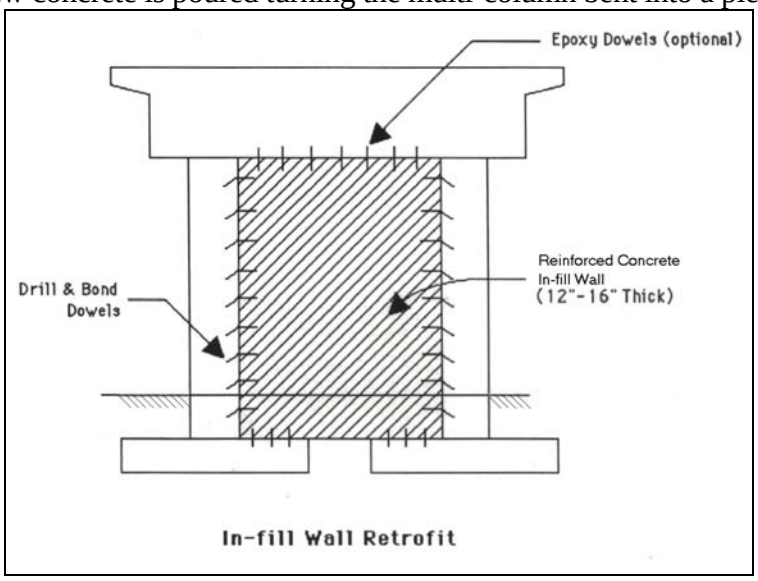
Bents are often retrofitted to increase their strength. A very popular retrofit for weak single column bents and pierwalls has been to built an outrigger bent cap and add pileshafts to the bent. Sometimes a single pileshaft is added to provide more strength but usually pileshafts are placed on each side of the bent. These retrofits may be used when the soil may liquefy and deep pileshafts can

continue to support the structure. Or they may be used when a steel shell cannot provide enough displacement capacity to make the bent survive the earthquake.

Outrigger Pileshaft Retrofit for Pier Wall.

The figure above shows a pier wall with an outrigger pileshaft retrofit. Note that the pier wall also has a steel shell that allows it to continue provide support during the earthquake. The advantage of the outrigger pileshaft retrofit is that it provides a great deal more ductility and flexural strength without appreciably increasing its stiffness.

Another popular retrofit provides increased strength in both directions and increased transverse stiffness. These in-fill wall retrofits are an inexpensive method of protecting multicolumn bents. Dowels are drilled and bonded to the existing concrete (as below), reinforcement is placed between the columns, and new concrete is poured turning the multi-column bent into a pier wall.



The ability to provide increased strength and stiffness in a particular direction is accomplished by attaching triangular shear walls to existing bents. These have been particularly popular for pedestrian overcrossings whose wandering geometry creates a center of mass far from the center of

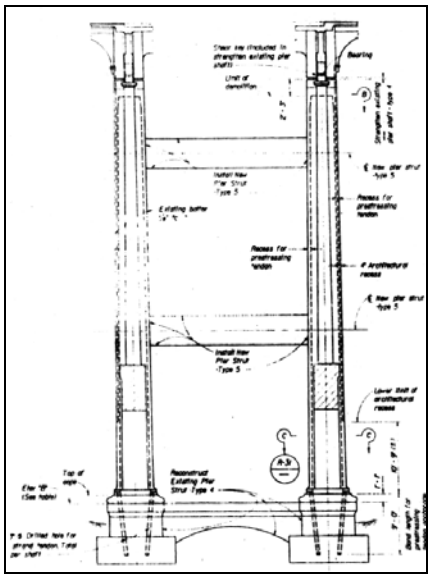
stiffness. Moreover, the single column bents for these delicate structures are usually incapable of handling earthquake forces. The shear wall is either placed to be normal to the curve or, as in this case, a wall is provided in two directions.

Triangular Shear Walls.

Isolation devices are often used to modify a bent's behavior during earthquakes. This is particularly effective for a stiff bent which is part of a flexible structure. We will look at this in more detail in the section on bridge system retrofit strategies. Another possible retrofit for bents is replacement. Sometimes a vulnerable portion of the bent will be replaced like an existing bent cap or connection. Often, a completely new bent is provided. Sometimes a single column will be used to support two structures. This can occur for double-deck viaducts or at interchanges. Since it would be exceedingly difficult for this column to survive a large earthquake, it is replaced with a separate column for each structure.

Link Beam Retrofit at the Santa Monica Freeway.

The use of link beams on multicolumn bents has been particularly effective. This method is used for very flexible columns that are too weak to handle large displacements. Link beams stiffen the bent and reduce the displacement. The figure above shows a link beam retrofit on the Santa Monica Expressway. Link beams can be designed to be weaker than the columns but to have great ductility. As the bent displaces transversely, the beams can absorb energy and protect the columns.

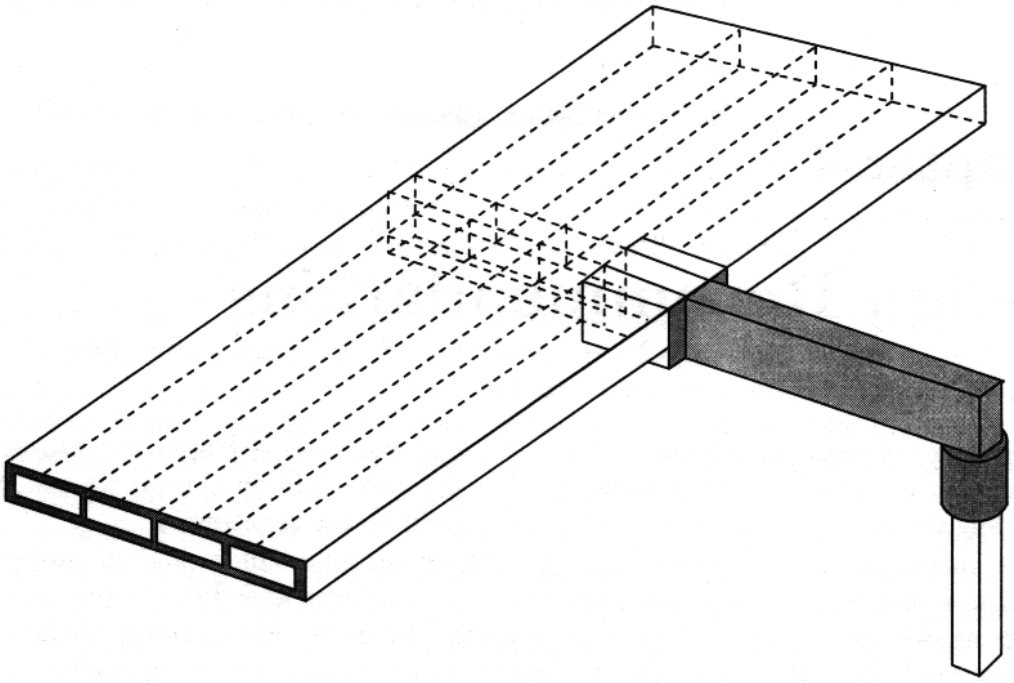


Link Beam and Post-Tensioned Column Retrofit.

Sometimes, the columns can be post-tensioned to provide them with added flexural and shear strength. This was done for the Colorado Street Bridge in Pasadena which had extremely weak columns with almost no reinforcement. A geology drilling unit was set up on the bridge deck and drilled straight down through the existing columns. Then prestressing strands were placed in the holes and post-tensioned. Also, link beams were added to stiffen the bents.

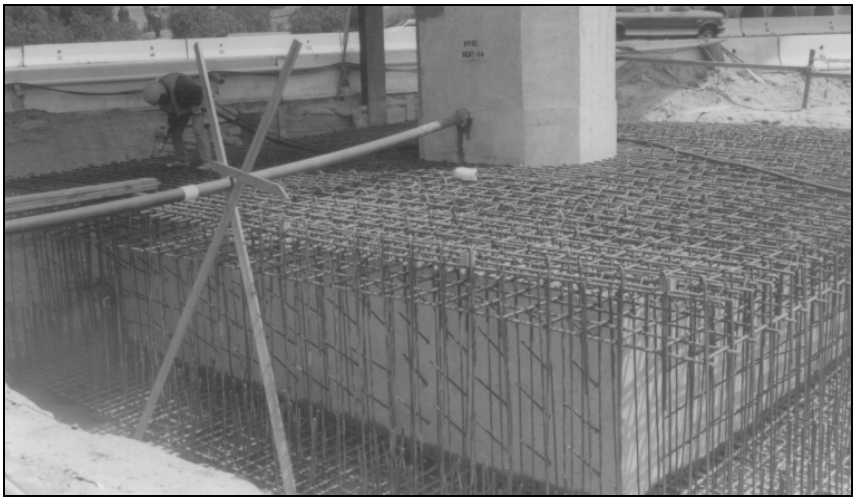
Steel bents have also been retrofitted. A common retrofit is to add steel plates to the flanges of steel bents. This increases their strength without adding much additional stiffness. Encasing steel bents in concrete is another alternative.

The preceding discussion is just a small sampling of the many bent retrofits that have been done in recent years. Most of these retrofits were the result of research and testing. Pier wall steel shell retrofits were investigated at UC Irvine. Most of the composite shell and link beam retrofits were tested at UC San Diego. UC Berkeley (Stojadinovic, 1995) has been looking at retrofitting Outrigger knee joints. Currently, older knee-joint connections have been removed and replaced with larger, more ductile joints. The testing program looked at the behavior of both long and short bent caps. A ductile retrofit and a strength retrofit strategy was studied using both a steel jacket and a post-tensioned reinforced concrete jacket. The final recommendation is for a strength retrofit using a steel plate jacket that forces plastic hinging in the column for both longitudinal and transverse motion. The figure below shows a retrofit which is composed of a concrete bolster that strengthens the connection to the superstructure, a steel plate jacket around the bent cap and the joint, and steel casing around the column.



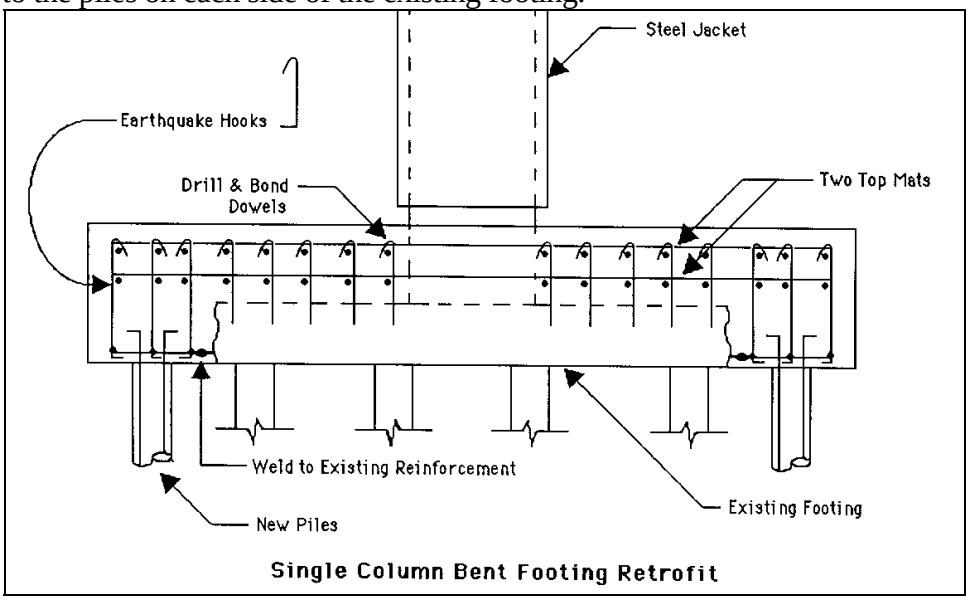
Final Recommended Knee-Joint Retrofit.

Foundation Retrofits



Reinforcement For Footing Retrofit.

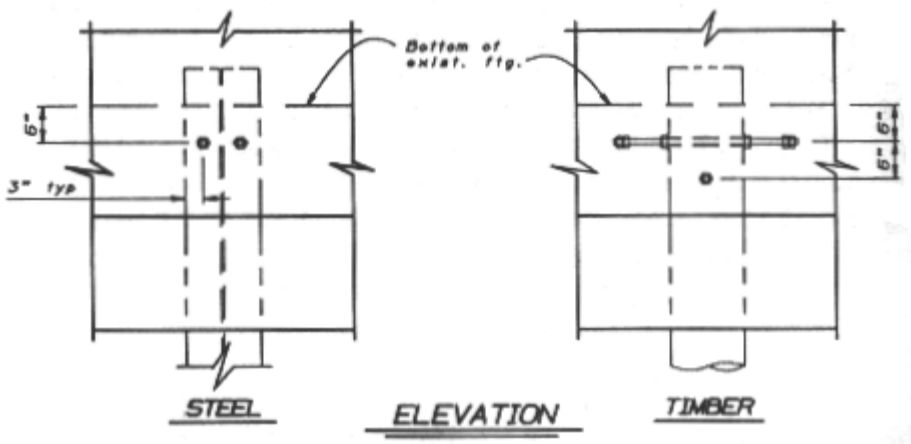
If the columns are strengthened or the footing is found to be inadequate, then a footing retrofit is required . This is often the case for single column bents that are fixed for moment. Sometimes the existing footing has no top mat of reinforcement, no ties to join the top and bottom mats, inadequate piles (particularly for tension), and too small a section for shear and flexure. The retrofit shown below may have additional piles driven around the footing perimeter and additional reinforcement and concrete placed around the existing footing. Drill and bond dowels connect the new footing to the old. These are designed to handle the shear friction at the interface. A beam is designed above the existing footing that must be strong enough to handle the moment and shear of the column being transferred to the piles on each side of the existing footing.



One of the problems that confronts engineers is the inability of many piles to act in tension as the foundation rocks back and forth. Both timber and steel piles (which are strong in tension) have very poor connections to the footing, older concrete piles have inadequate reinforcement, and many piles have inadequate friction to engage the surrounding soil. These problems have resulted in a number of innovative foundation retrofits. Sometimes holes are drilled through the existing footings and tie-down anchors are placed and grouted in the holes.

Tie-Down Rods being Connected to Existing Footing.

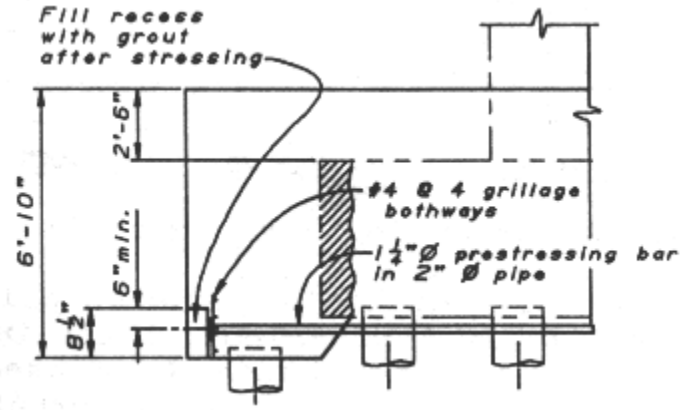
Sometimes the existing piles are reattached by excavating under the footing, attaching high strength bolts to the top of the piles, placing new reinforcement under the footing, and casting new concrete in the excavation.



Reattaching Existing Piles to Footing.

A testing program at Caltrans (Mason, 1995) identified a number of innovative new piles that are able to provide tension even in Bay mud. Some of these pile systems, like the Dutch Fundex System, are segmental and can be placed under bridge decks which make them well suited for retrofits.

In situations where there is not enough reinforcement in the bottom of footings to handle the moment, prestressing tendons can be used. This has the added advantage of increasing the shear capacity of the footing. The Coyote Wells Overhead on Route 8 in Imperial County had a footing retrofit. New piles were placed along the outside of the existing footing, and the footing was extended outward and below the existing footing. Then holes were drilled under the existing footing and through the new footing extension. Prestress ducts with prestress tendons were placed in the holes. When the tendons were stressed and anchored to the new footing, the required moment capacity was provided. The prestressing also helped to secure the footing extension and piles to the existing footing.

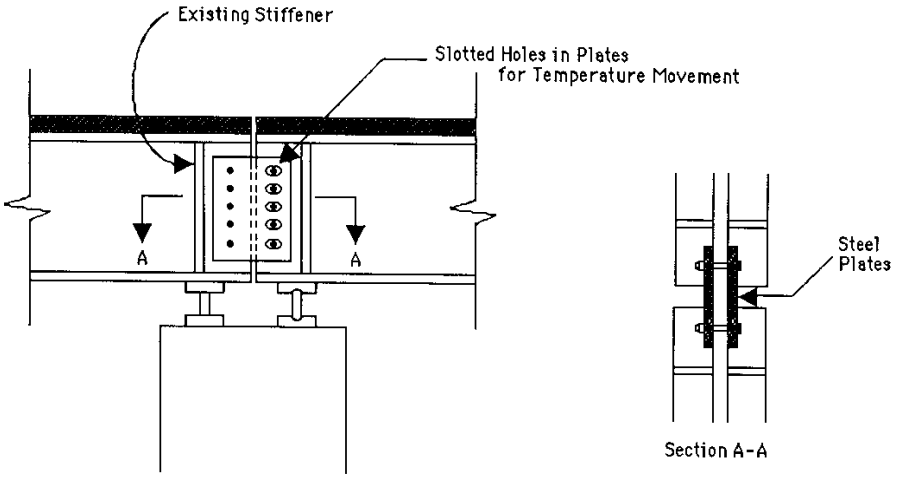


Prestressed Footing Retrofit at Coyote Wells.

Expansion Joint Retrofits

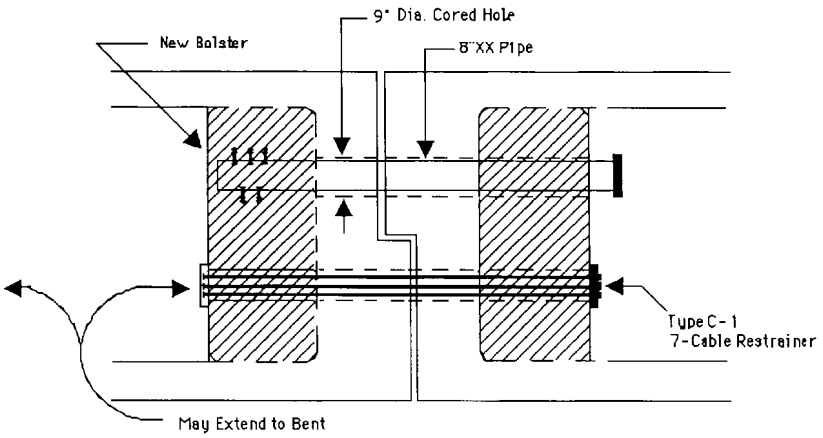
Retrofits to prevent the superstructure from falling at expansion joints have become more sophisticated over the years. Originally, cable restrainers were used to prevent superstructures from falling off narrow seats. However, because of the ambivalent performance of restrainers during past earthquakes, newer retrofits provide a more reliable method of preventing unseating, either by locking the joint or by extending the seat in various ways. For steel superstructures, plates are

sometimes used to connect the girders at expansion joints. This provides a strong, positive load path for the inertia force as well as providing a seat if one of the girders falls off of its support.



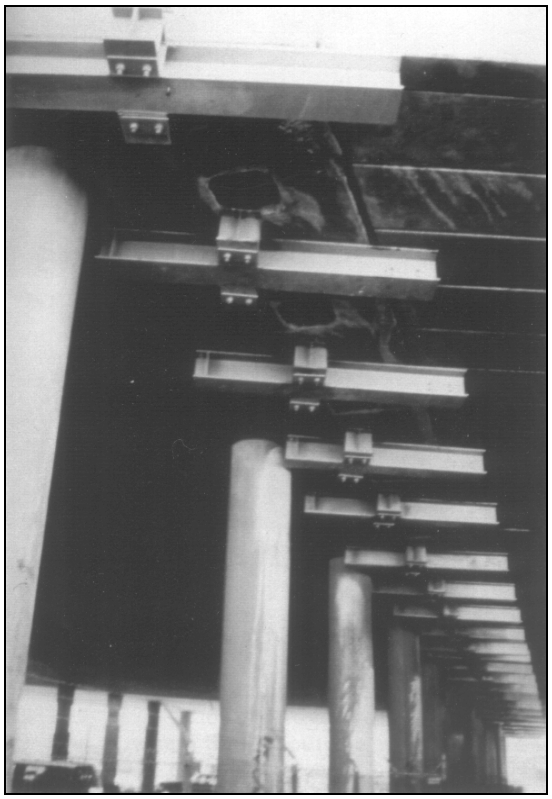
Steel Girder Hinge Plate Retrofit

Similarly, pipe seat extenders are placed at the hinges of reinforced concrete box girder bridges to prevent unseating. Holes are cored through the hinge, steel pipes are inserted in the holes, and concrete bolsters are cast at both ends. One end of the pipe is fixed to the bolster while the other end is free to translate back and forth. If the hinge opens too far, the double extra strong pipe can support the superstructure.



Retrofit Pipe Hinge Seat Extender

There are many variations to these devices that either lock or provide support at expansion joints. Steel girders were placed under the hinges at the Santa Monica Freeway in Los Angeles to provide a longer seat.



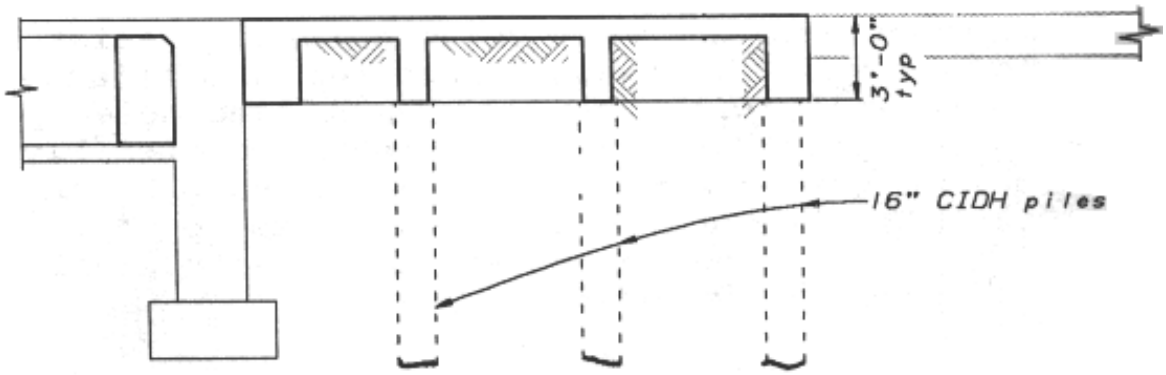
External Hinge Extenders on Santa Monica Freeway

Catcher blocks are often cast in front of the ends of steel and concrete girders to prevent them from falling. Transverse and longitudinal shear keys, vertical restrainers, keeper plates, etc. are all common retrofits. Sometimes steel rods and brackets are used to lock a short seat. One must provide abundant strength for these connections, as they are otherwise likely to break during large earthquakes.

Locking Device at Abutment Seat of Pedestrian Overcrossing.

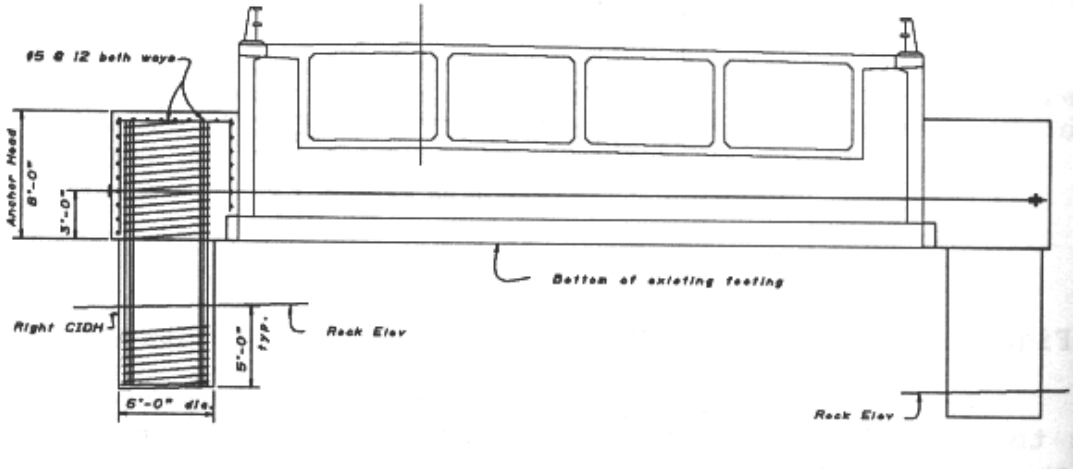
Abutment Retrofits

Stiffening and strengthening abutments to take more load and to immediately engage the soil has become a big part of new seismic retrofits. This is particularly helpful when the end bents are very stiff and would otherwise take much of the load and fail during a large earthquake. Timber is often placed between the abutment backwall and the superstructure end diaphragm to immediately engage the soil during an earthquake. Then special approach slabs, piles, and tieback systems help hold onto the bridge while absorbing energy. The figure below shows a waffle approach slab that is sometimes used to anchor the abutment to the surrounding soil.



Seismic Waffle Slab.

Similarly, concrete blocks and other devices extend behind the abutment to engage the soil. Concrete haunches (often supported on large diameter piles) are used to support the structure transversely. Often, the haunches on each side of the abutment are tied together with cables to make them work together as the abutment moves from side to side.



Abutment Haunch with CIDH Piles and Cable.

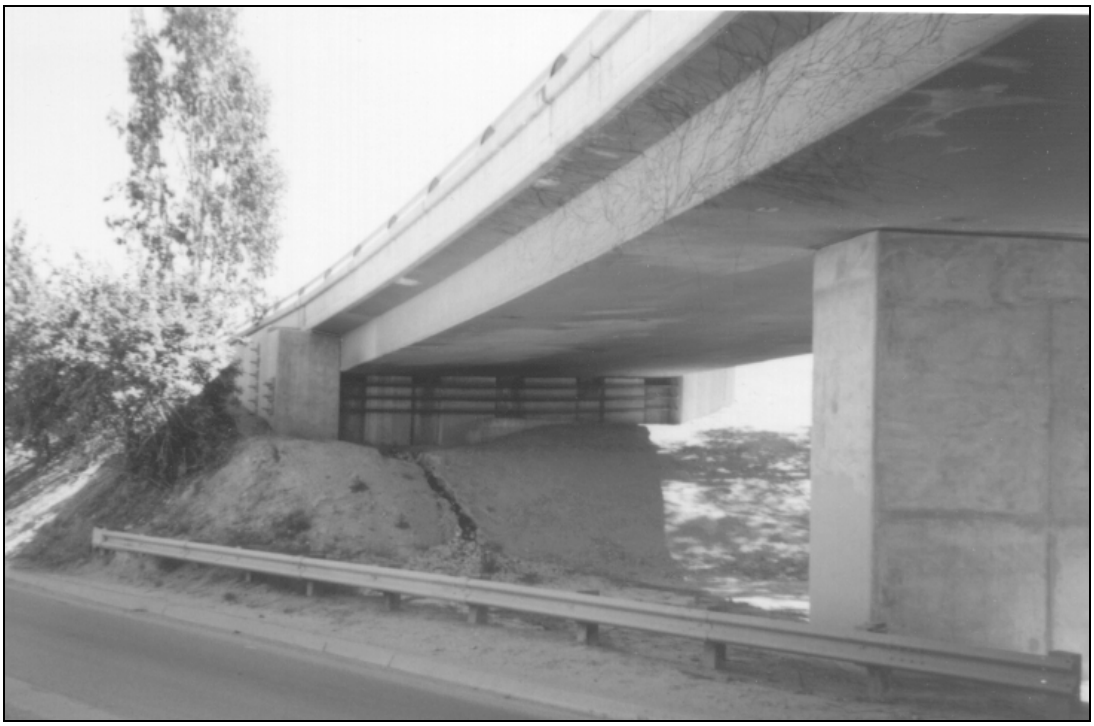
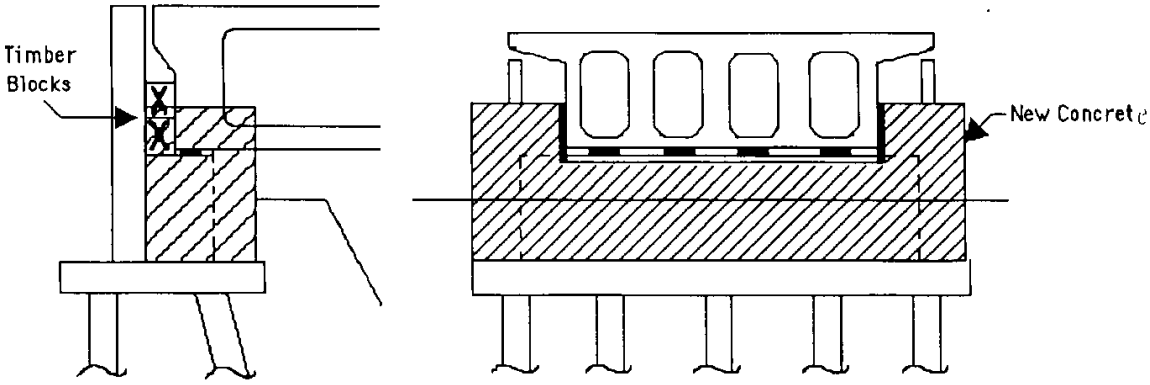


Photo of Abutment Retrofit with Haunches.



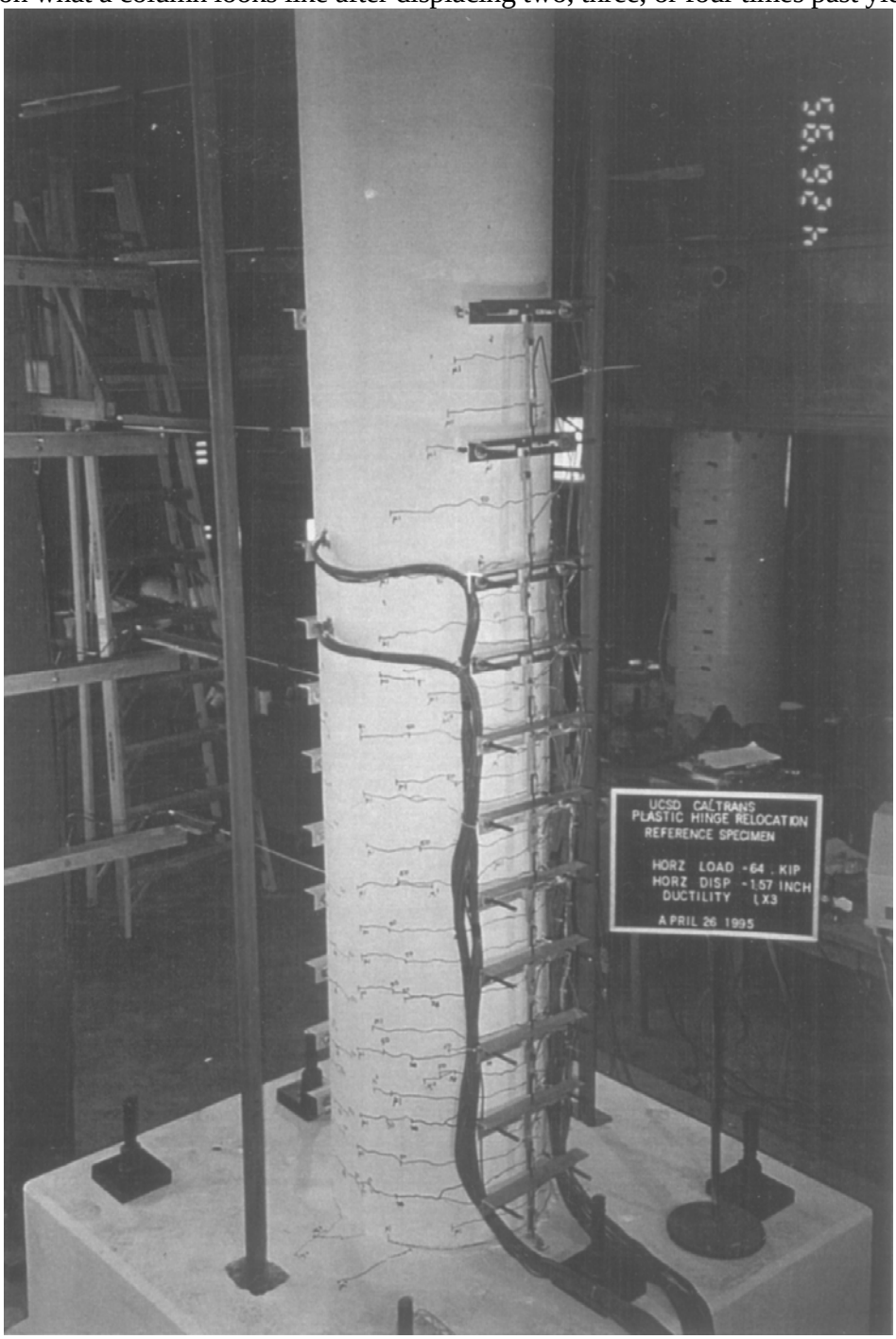
Exterior Shear Key Retrofit @ Abutment

Abutment Shear Key Retrofit

The figure above shows what used to be the most common bridge retrofit which was to provide exterior shear keys at abutments to prevent transverse movement of the superstructure. Note that timber blocking is provided in the gallery between the backwall and the superstructure so that the soil will be more quickly engaged during the earthquake, less banging will occur, and the brittle concrete backwall is protected.

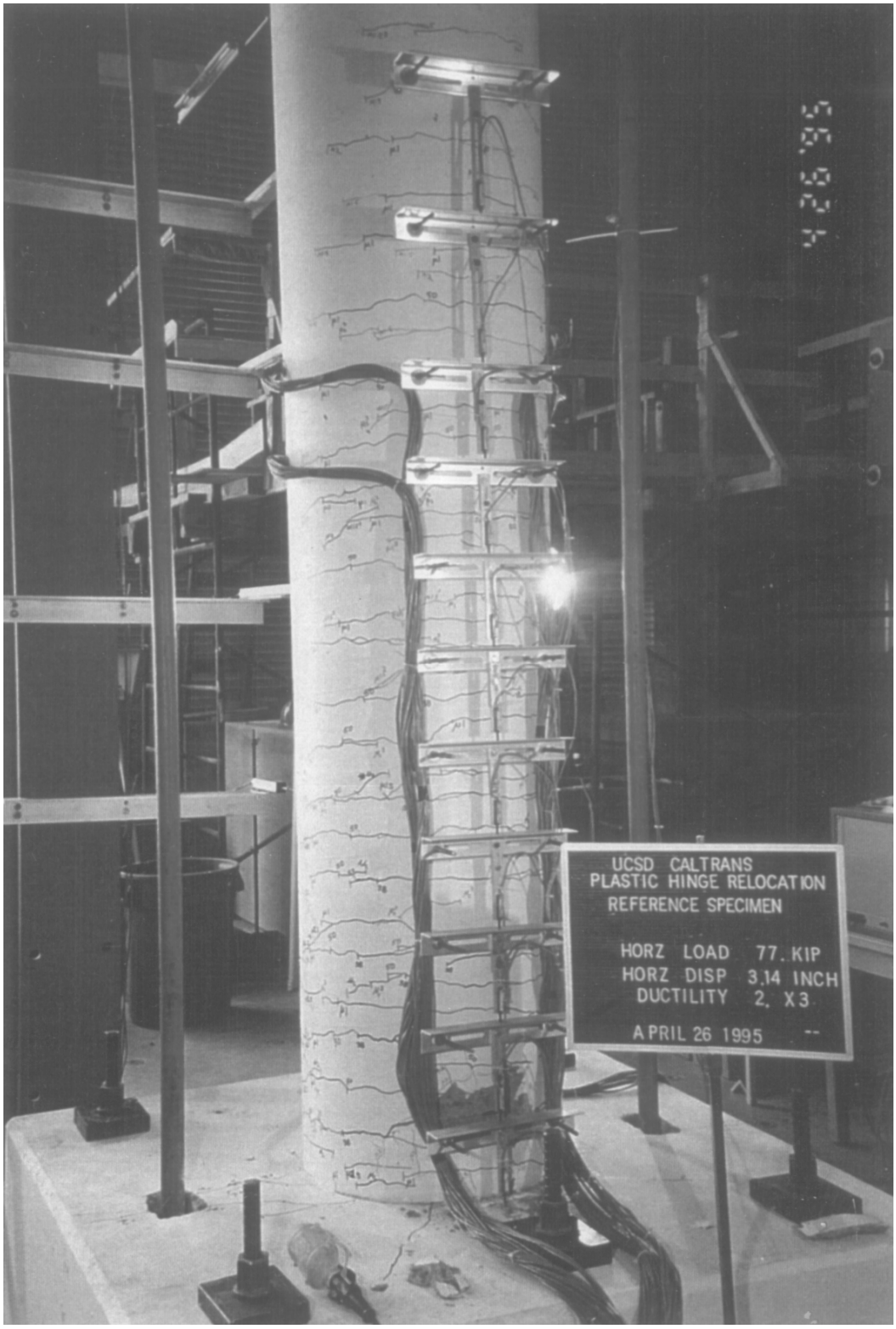
DAMAGE PHOTOS FROM THE TESTING OF BRIDGE COLUMNS

A good way to determine the extent of damage to bridges in the field is by familiarizing yourself with damage in the lab. By studying these and other photos, the investigator will obtain a better perspective on what a column looks like after displacing two, three, or four times past yield.

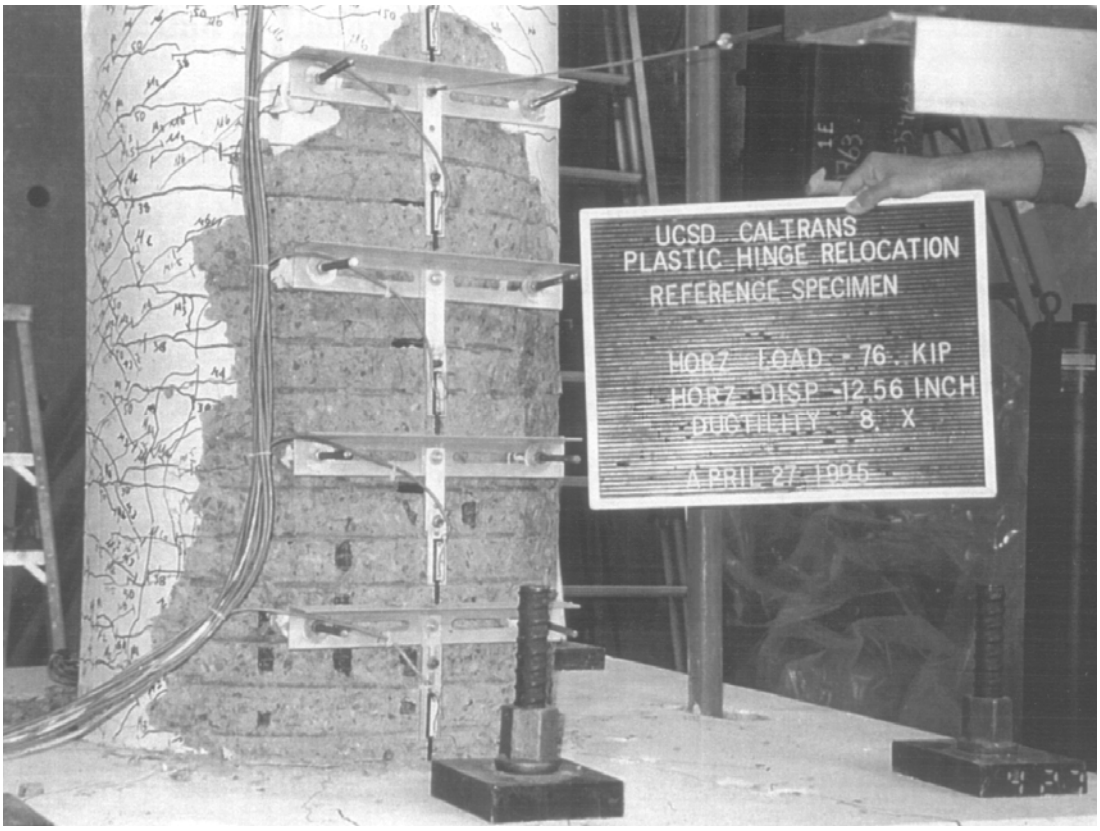


Third Cycle at Disp. = Ductility 1.0 at UCSD for Reference Column
(Relocation of Plastic Hinge Tests).

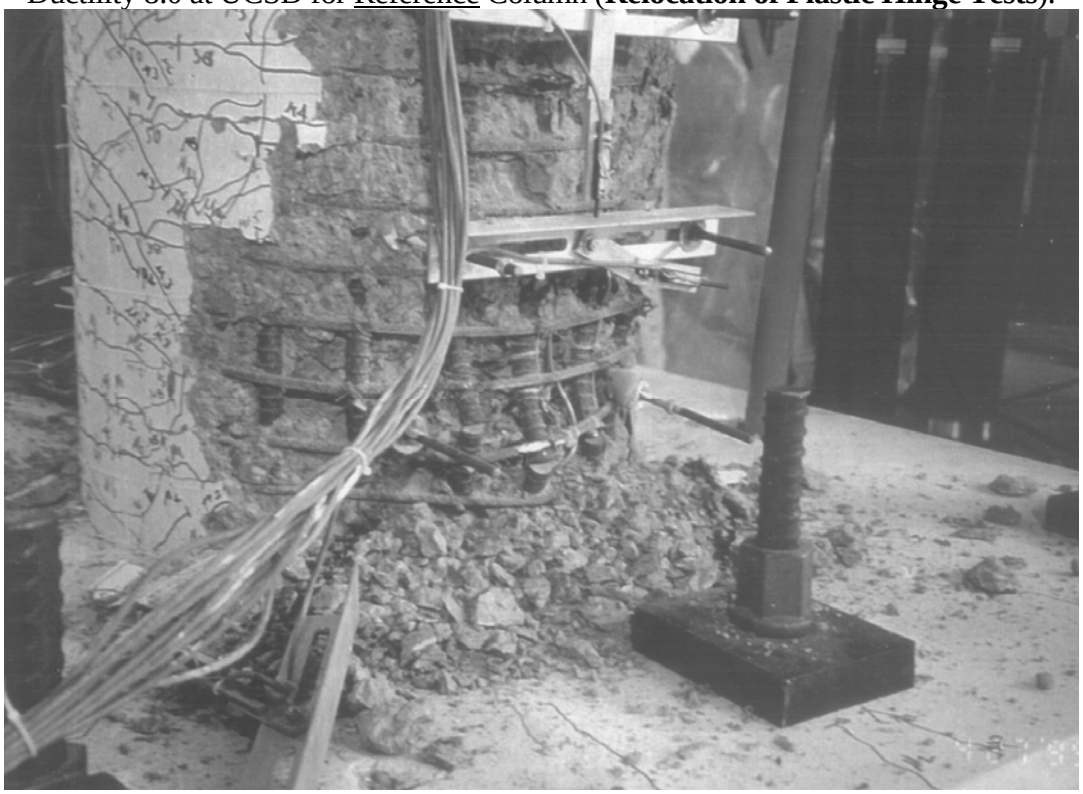
This specimen was designed conventionally to allow a column hinge to form at the footing. It is a 2 ft diameter, 12 ft tall concrete column with 20 #7 bars and a #3 spiral at a 3" pitch.



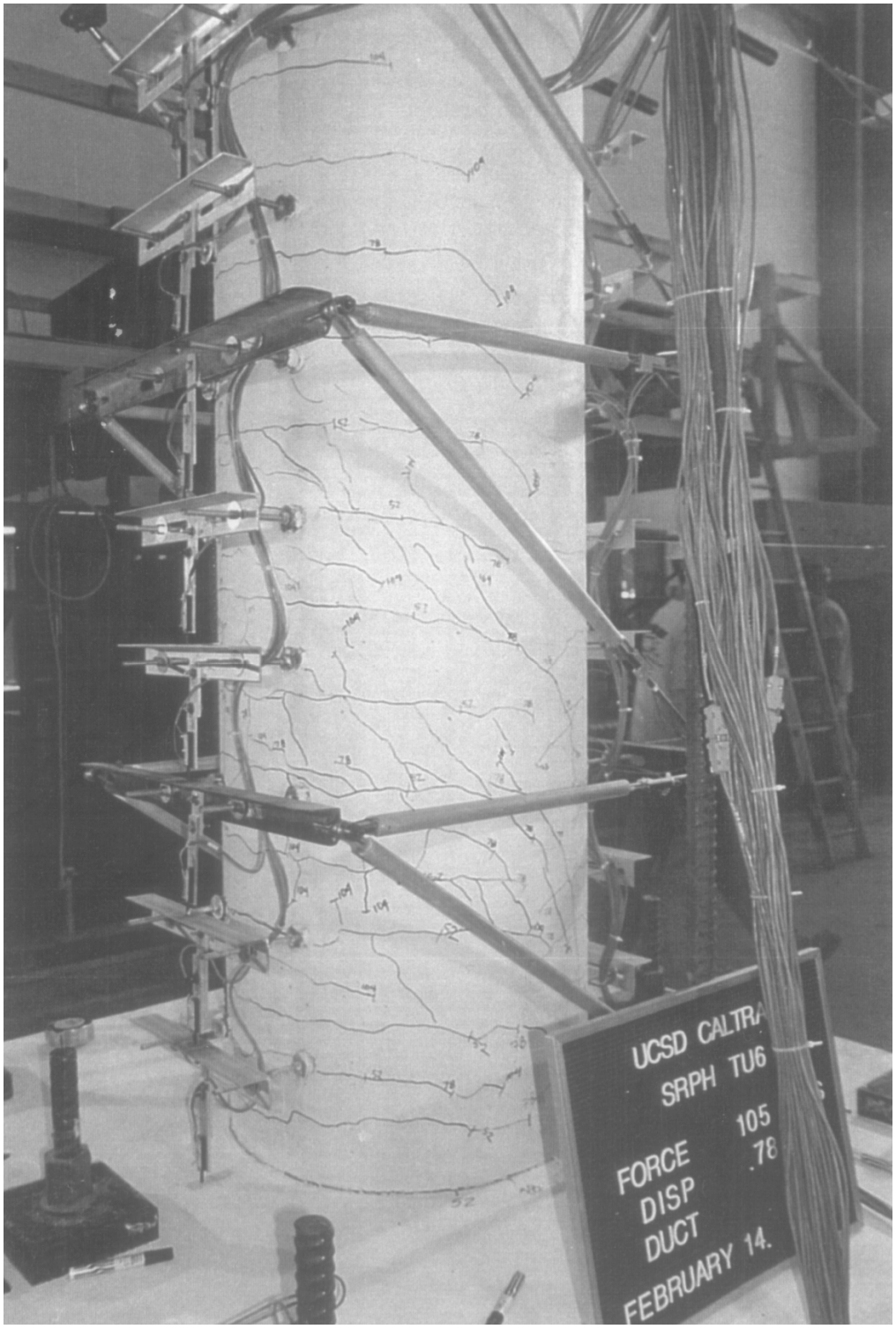
Third Cycle at Disp. = Ductility 2.0 at UCSD for Reference Column
(Relocation of Plastic Hinge Tests).



Ductility 8.0 at UCSD for Reference Column (**Relocation of Plastic Hinge Tests**).

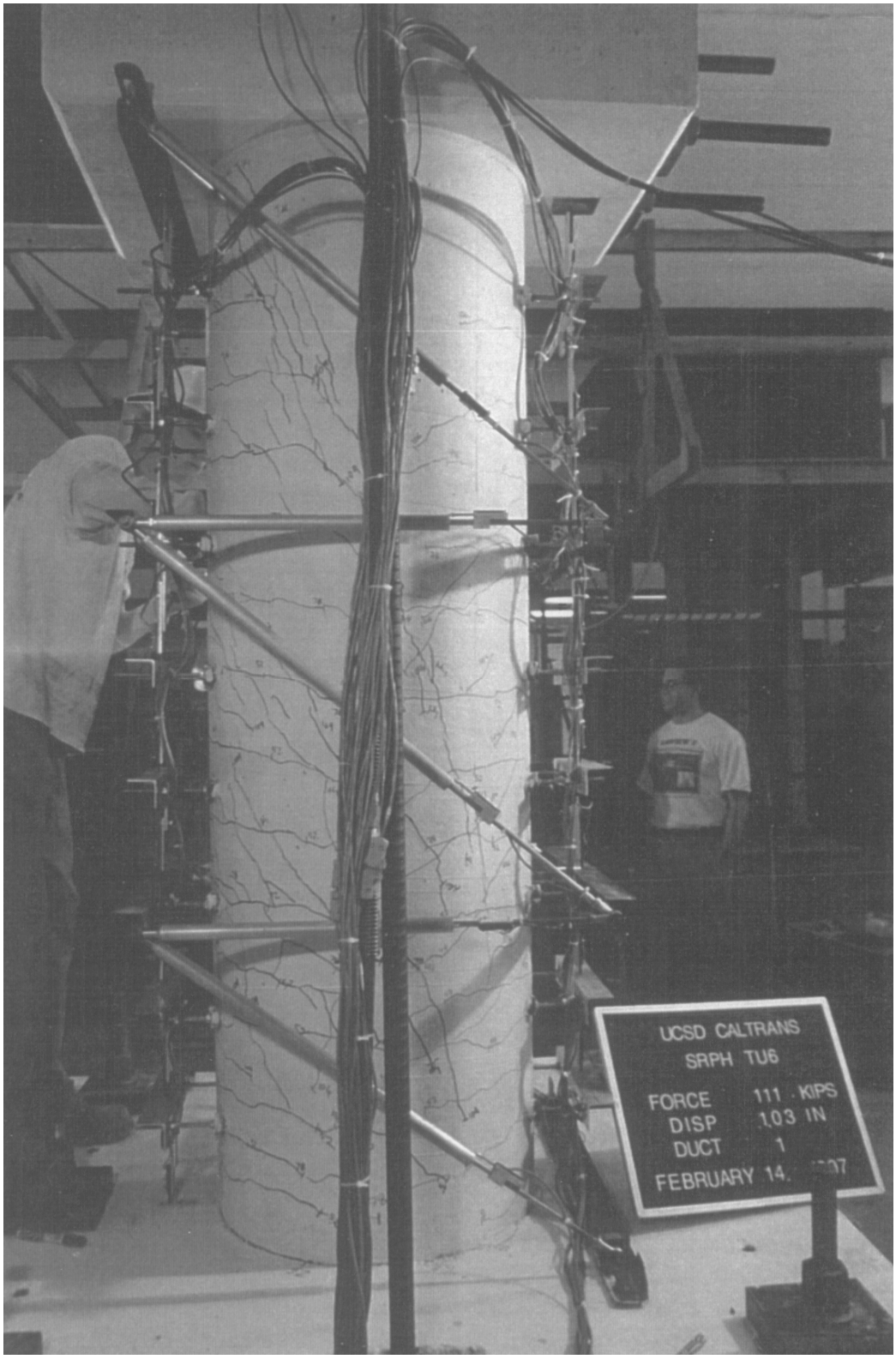


Failure at UCSD for Reference Column (**Relocation of Plastic Hinge Tests**).

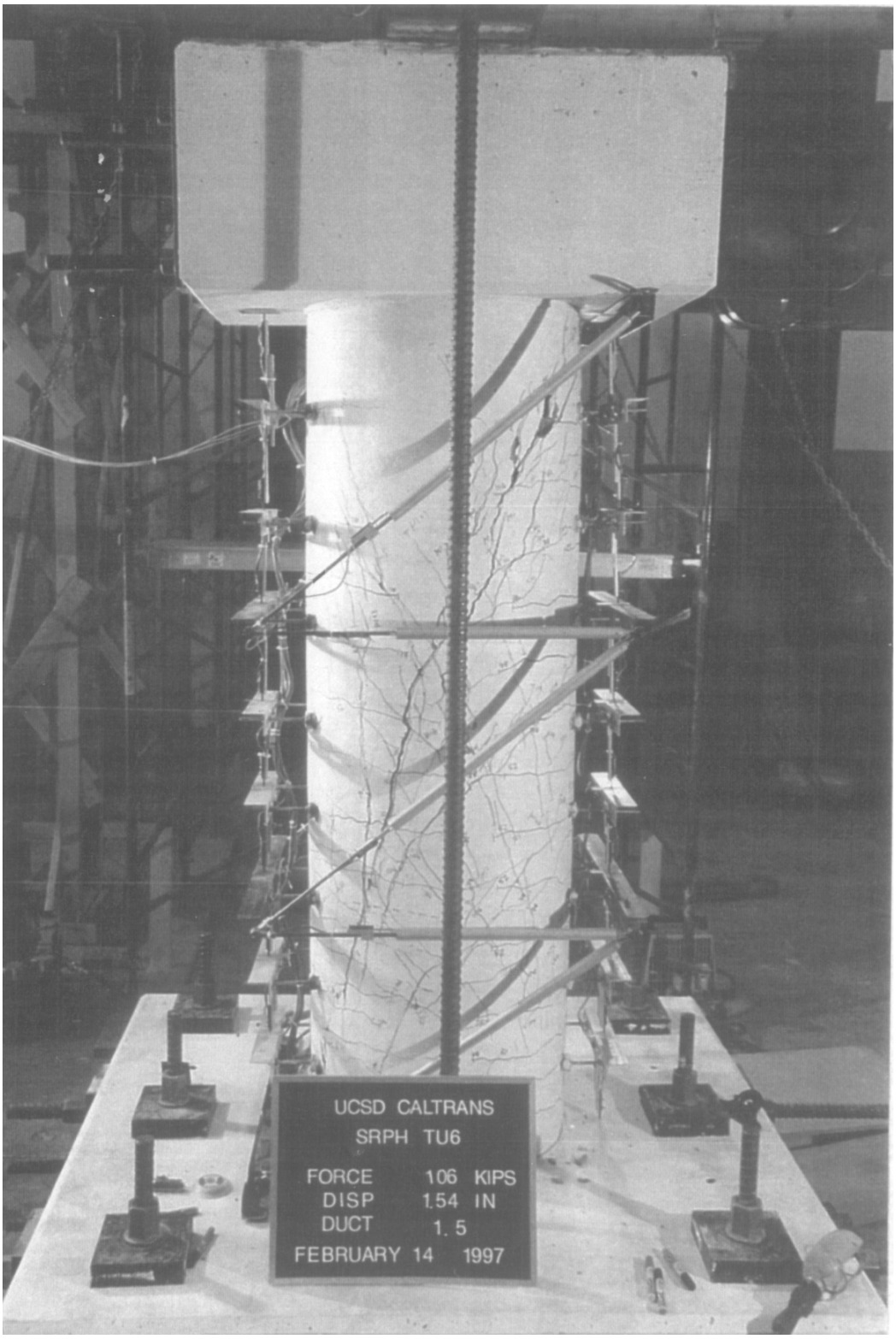


Yield for Shear Column at UCSD
(**Relocation of Plastic Hinge Tests**).

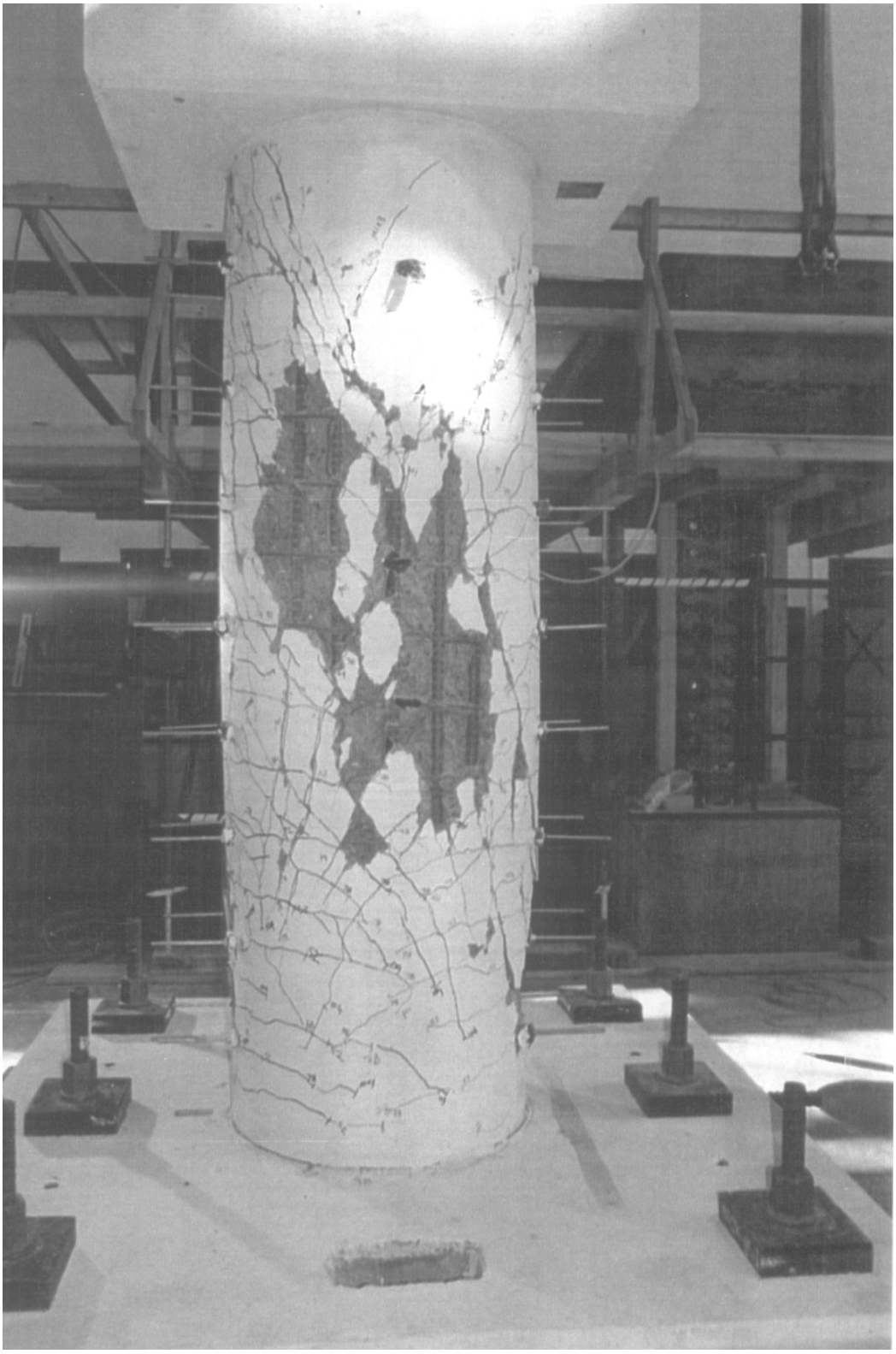
This specimen was designed with additional longitudinal reinforcement at the base of the column to force plastic hinging one column diameter above the footing. However, the transverse reinforcement was limited to cause a shear failure at a ductility of 2.0.



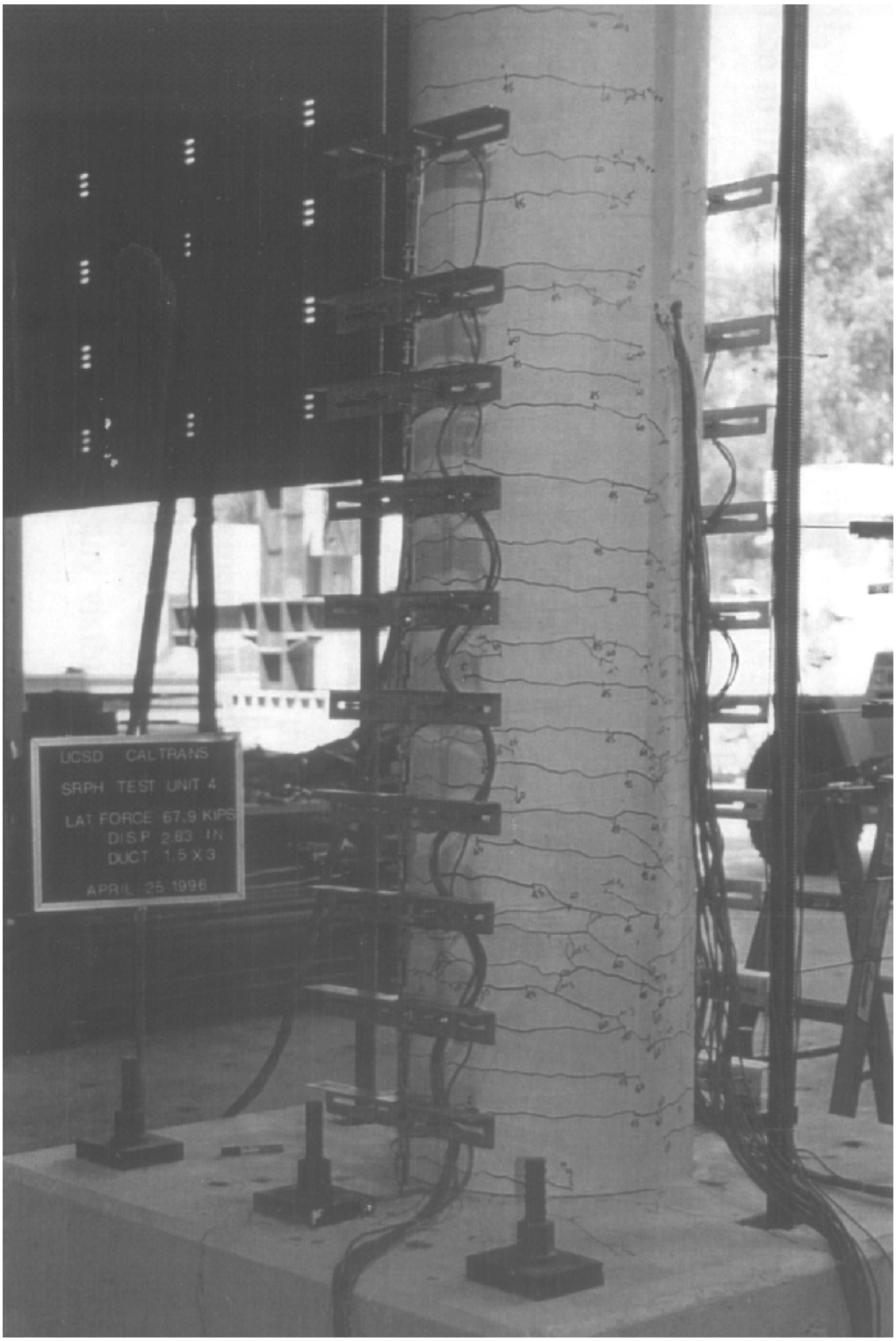
Ductility of 1.0 for Shear Column at UCSD
(**Relocation of Plastic Hinge Tests**).



Ductility 1.5 for Shear Column at UCSD
(**Relocation of Plastic Hinge Tests**).

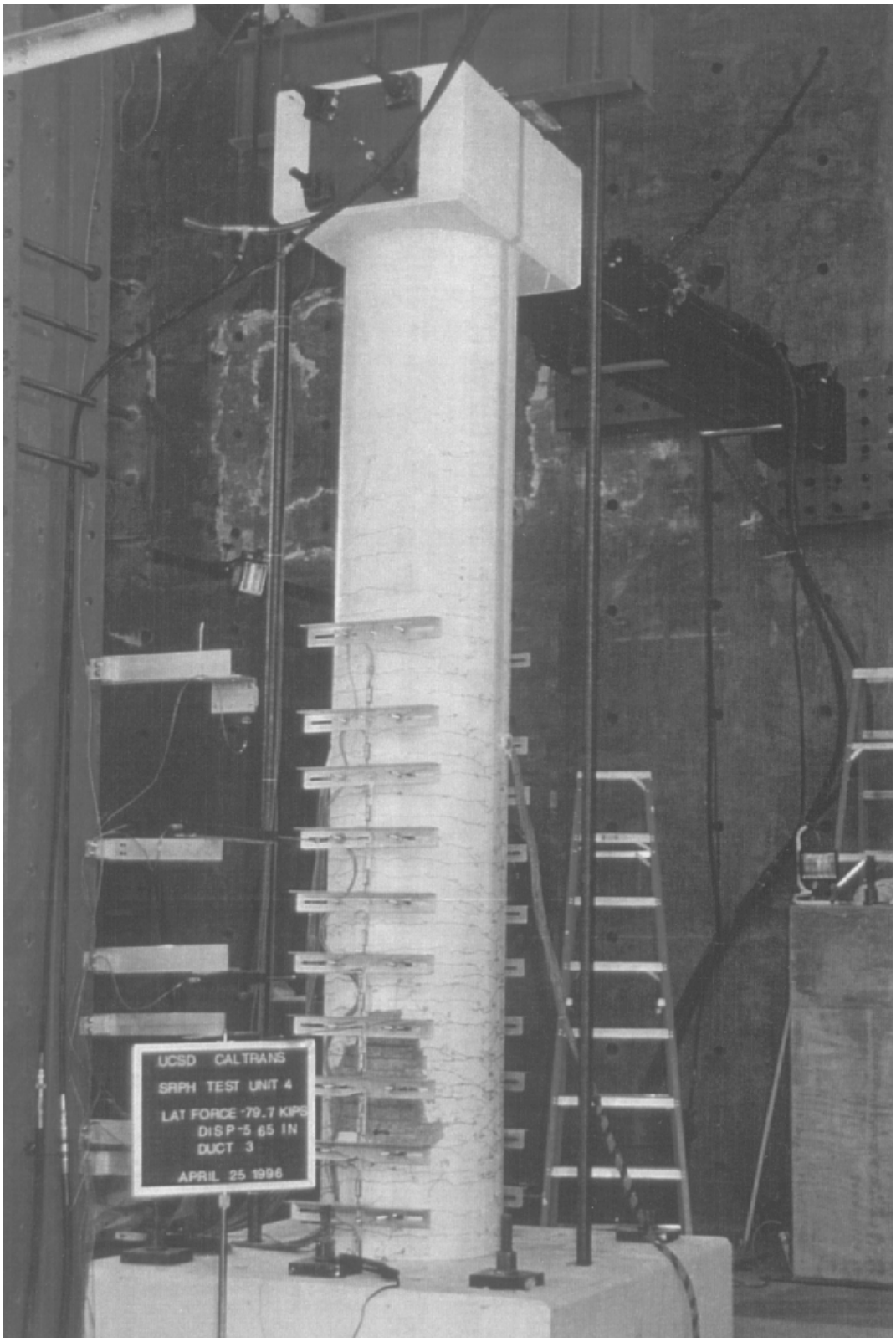


End of Test at UCSD for Shear Column (Ductility < 2.0)
(**Relocation of Plastic Hinge Tests**).

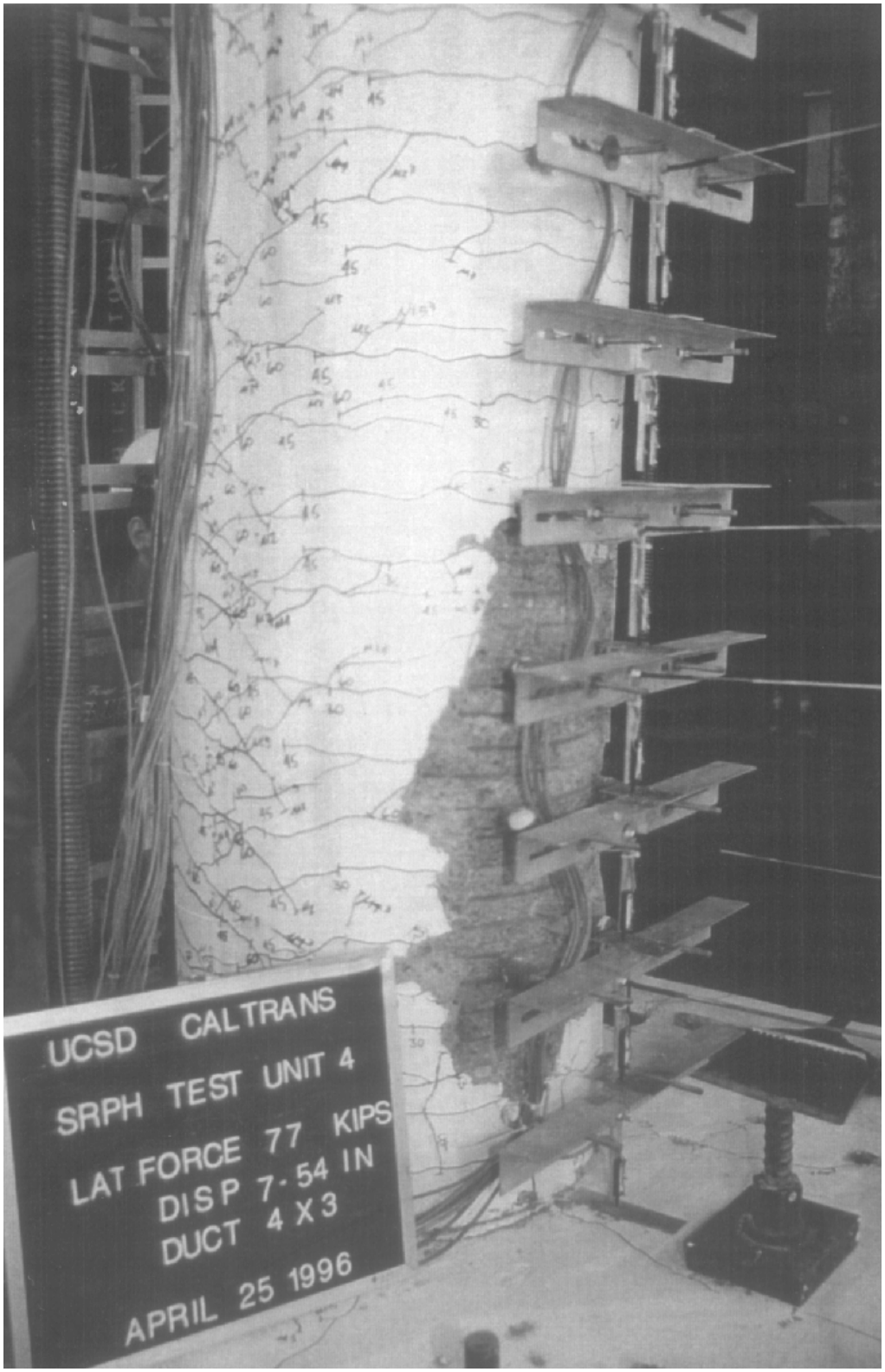


Ductility 1.5 at UCSD for Test Unit 4, Staggered Termination $_s$ = 2.3%.
(**Relocation of Plastic Hinge Tests**).

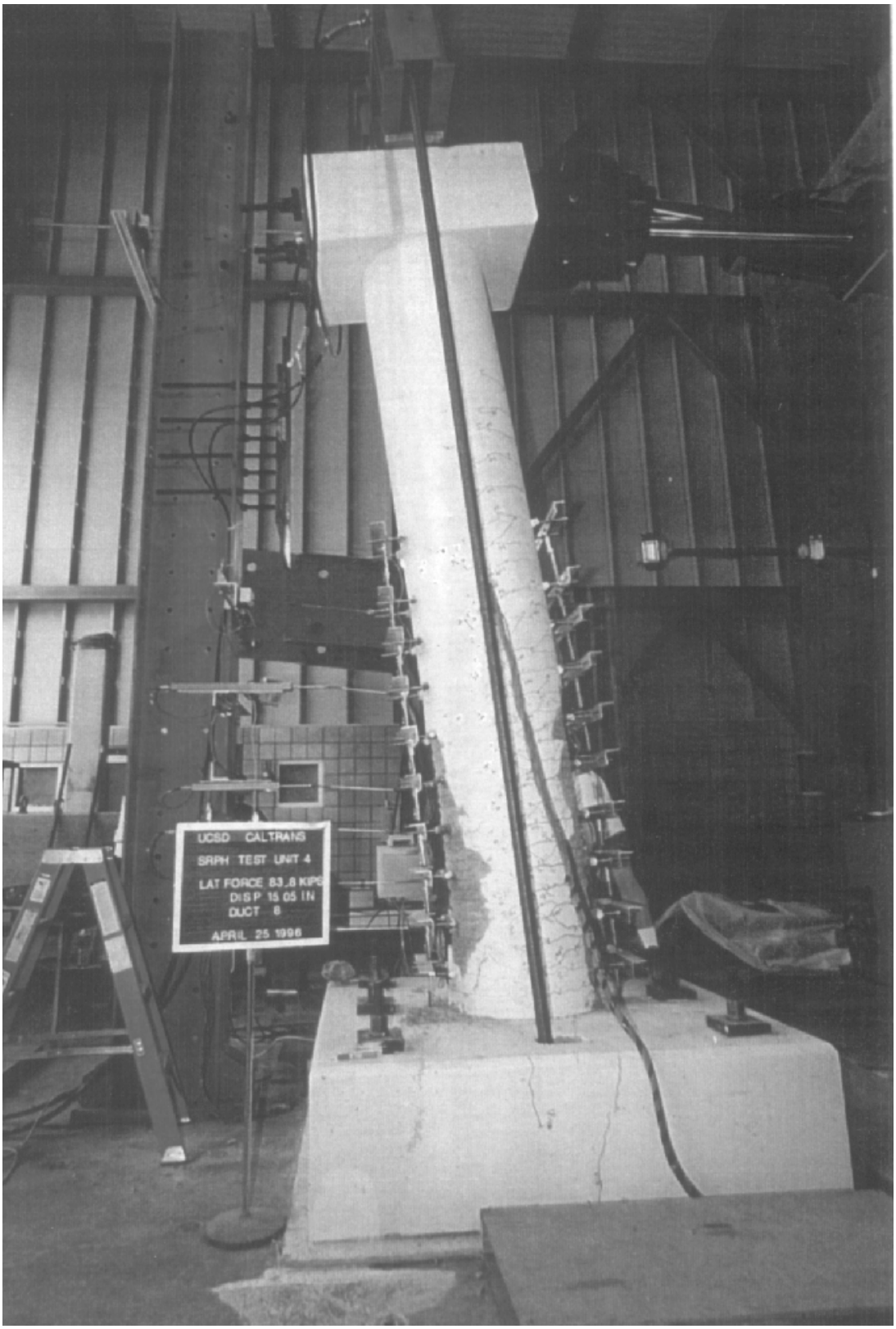
An extremely ductile column (#4 spiral w/ 1.5" pitch) with inner cage bars terminated to force the plastic hinge to occur 18" above the footing.



Ductility 3.0 at UCSD for Test Unit 4
(**Relocation of Plastic Hinge Tests**).



Ductility 4.0 at UCSD for Test Unit #4
(**Relocation of Plastic Hinge Location Tests**).



Ductility 8.0 at UCSD for Test Unit #4
(**Relocation of Plastic Hinge Location Tests**).

CALIFORNIA EARTHQUAKE HISTORY

Studying previous California earthquakes will give the investigator a better perspective on the earthquake they are investigating. This list includes California, Baja California, and Nevada events The **magnitude** listed here is the "summary magnitude". For most events prior to 1898 this is based on the adjusted intensity magnitude, and for events after 1898 it is the surface wave magnitude. The list includes known earthquakes with a magnitude of at least 6 and selected smaller events. The smaller events since 1898 all have at least one reported magnitude of at least 5.8, even if the summary magnitude is smaller. Some of these magnitudes may be different than what is reported in the Southern California or Northern California Earthquake Catalog. The **date and time** for each earthquake are given as "24 hour" time referenced to Greenwich Mean Time (now UTC). To convert a time to Pacific Standard Time (PST), subtract 8 hours. To convert a time to Pacific Daylight Time (PDT), where appropriate, subtract 7 hours. **Example:** The 1989 Loma Prieta earthquake occurred at 00:04 UTC on October 18, 1989 or 5:04pm PDT on October 17, 1989.

Ellsworth, W. L., "Earthquake History, 1769-1989" USGS Professional Paper 151 (modified to present)

DATE	TIME(GMT)	LATITUDE	LONGITUDE	MAG	LOCATION
1769 7 28	0 0	34 0.00	118 0.00	6.0	Los Angeles Basin
1800 11 22	2130	33 0.	117 18.00	6.5	San Diego region
1808 6 24	0 0	37 48.00	122 30.00	6.0	San Francisco region
1812 12 8	15 0	34 22.00	117 39.00	7.0	Wrightwood
1812 12 21	19 0	34 12.00	119 54.00	7.0	Santa Barbara Channel
1827 9 24	4 0	34 0.	119 0.	5.5	Los Angeles region
1836 6 10	1530	37 48.00	122 12.00	6.75	Hayward Valley
1838 6 0	0 0	37 36.00	122 24.00	7.0	San Francisco Peninsula
1852 11 29	20 0	32 30.00	115 0.	6.5	Volcano Lake, B.C.
1855 7 11	415	34 6.00	118 6.00	6.0	Los Angeles region
1856 2 15	1325	37 30.00	122 18.00	5.5	San Francisco Peninsula
1857 1 9	16 0	35 42.00	120 18.00	8.25	Great Fort Tejon quake
1857 9 3	3 5	39 18.00	120 0.	6.25	W. Nevada or E. Sierra
1858 11 26	835	37 30.00	121 54.00	6.25	San Jose region
1858 12 16	10 0	34 0.	117 30.00	6.0	San Bernardino region
1860 3 15	19 0	39 30.00	119 30.00	6.5	Carson City,Nevada
1861 7 4	011	37 48.00	122 0.	5.75	San Ramon Valley
1862 5 27	20 0	32 42.00	117 12.00	6.0	San Diego region
1864 2 26	1347	37 6.00	121 42.00	6.0	S. Santa Cruz Mountains
1864 3 5	1649	37 42.00	122 0.	5.75	E. of San Francisco Bay
1865 10 8	2046	37 0.00	122 00.00	6.5	S. Santa Cruz Mountains
1866 7 15	0630	37 30.00	121 18.00	6.0	W. San Joaquin Valley
1868 5 30	510	39 18.00	119 42.00	6.0	Virginia City, Nevada
1868 10 21	1553	37 42.00	122 6.00	7.0	Hayward fault
1869 12 27	155	39 24.00	119 42.00	6.25	Olinghouse fault, Nevada
1869 12 27	10 0	39 6.00	119 48.00	6.0	Carson City, Nevada
1870 2 17	2012	37 12.00	122 6.00	6.0	Los Gatos
1871 3 2	21 5	40 24.00	124 12.00	6.0	Cape Mendocino
1872 3 26	1030	36 42.00	118 6.00	7.6	Owens Valley
1872 3 26	14 6	36 54.00	118 12.00	6.75	Owens Valley
1872 4 3	1215	37 0.	118 12.00	6.25	Owens Valley
1872 4 11	19 0	37 30.00	118 30.00	6.75	Owens Valley
1872 5 3	1 0	33 0.	115 0.	5.75	Imperial Valley (?)
1872 11 12	0 0	39 0.	117 0.	6.0	Austin, Nevada region(?)
1873 11 23	5 0	42 0.	124 0.	6.75	Crescent City
1875 1 24	1200	40 42.	120 30.	6.0	Honey Lake
1875 11 15	2230	32 30.00	115 30.00	6.25	Imperial Valley to ColRv
1878 5 9	425	40 6.00	124 0.	6.0	Punta Gorda region
1881 2 2	011	36 0.	120 30.00	5.75	Parkfield
1881 4 10	10 0	37 24.00	121 24.00	6.0	W. San Joaquin Valley
1882 3 6	2145	36 54.	121 12.	5.75	Hollister
1883 9 5	1230	34 12.00	119 54.00	6.25	Santa Barbara Channel
1884 1 28	730	41 6.	123 36.	5.75	Klamath Mountains

1884	3	26	40	37	6.	122	12.	6.0	Santa Cruz Mountains
1885	1	31	545	40	24.	120	36.	5.75	Susanville
1885	4	12	4 5	36	24.00	121	0.	6.25	S. Diablo Range
1887	6	3	1048	39	12.00	119	48.00	6.5	Carson City, Neveda area
1888	4	29	448	39	42.00	120	42.00	6.0	Mohawk Valley
1889	5	19	1110	38	0.	121	54.00	6.25	Antioch
1889	6	20	6 0	40	30.00	120	42.00	6.0	Susanville
1889	9	30	520	37	12.	118	42.	5.75	Bishop region
1890	2	9	12 6	33	24.00	116	18.00	6.5	San Jacinto Fault region
1890	4	24	1136	36	54.00	121	36.00	6.25	Pajaro Gap
1890	7	26	940	40	30.00	124	12.00	6.25	Cape Mendocino
1891	7	30	1410	32	0.	115	0.	6.0	Colorado R. delta region
1892	2	24	720	32	33.00	115	38.00	7.0	Laguna Salada, B.C.
1892	4	19	1050	38	24.00	122	0.	6.5	Vacaville
1892	4	21	1743	38	30.00	121	54.00	6.25	Winters
1892	5	28	1115	33	12.00	116	12.00	6.5	San Jacinto fault region
1892	11	13	1245	36	48.00	121	30.00	5.75	Hollister
1893	5	19	035	34	6.00	119	24.00	5.75	Pico Canyon
1894	7	30	512	34	18.00	117	36.00	6.0	Lytle Creek region
1894	9	30	1736	40	18.	123	42.	6.0	Cape Mendocino region
1894	10	23	23 3	32	48.00	116	48.00	5.75	E. of San Diego
1896	8	17	1130	36	42.00	118	18.00	6.0	SE Sierra Nevada
1897	6	20	2014	37	0.	121	30.00	6.25	Gilroy
1898	3	31	743	38	12.00	122	24.00	6.5	Mare Island
1898	4	15	7 7	39	12.00	123	48.00	6.5	Mendocino
1899	4	16	1340	41	0.	126	0.	7.0	W. of Eureka
1899	7	6	2010	37	12.	121	30.	5.75	Morgan Hill
1899	7	22	2032	34	18.00	117	30.00	5.75	Lytle Creek region
1899	12	25	1225	33	48.00	117	0.	6.4	San Jacinto and Hemet
1901	3	3	745	36	0.	120	30.00	6.4	Parkfield
1903	1	24	527	31	30.00	115	0.00	6.6	Colorado R. delta region
1903	6	11	1312	37	24.00	121	54.00	5.5	San Jose
1903	8	3	649	37	18.00	121	48.00	5.5	San Jose
1906	4	18	1312	37	42.00	122	30.00	8.25	Great 1906 earthquake
1906	4	19	030	32	54.00	115	30.00	6.2	Imperial Valley
1906	4	23	910	41	0.	124	0.	6.4	Arcata
1907	9	20	154	34	12.00	117	6.00	5.3	San Bernardino region
1908	11	4	837	36	0.	117	0.	6.0	Death Valley region
1909	10	29	645	40	30.00	124	12.00	5.8	Cape Mendocino
1910	3	11	652	36	54.00	121	48.00	5.8	Watsonville
1910	3	19	011	40	0.	125	0.	6.0	W. of Cape Mendocino
1910	5	15	1547	33	42.00	117	24.00	5.5	Glen Ivy Hot Springs
1910	8	5	131	42	0.	127	0.	6.6	W. of Crescent City
1911	7	1	22 0	37	15.00	121	45.00	6.5	Calaveras fault
1914	2	18	1817	39	30.00	119	48.00	5.5	Truckee region
1914	4	24	834	39	30.00	119	48.00	6.0	Truckee region
1915	5	6	12 9	40	0.00	126	0.	6.2	W. of Cape Mendocino
1915	6	23	359	32	48.00	115	30.00	6.0	Imperial Valley
1915	6	23	456	32	48.00	115	30.00	5.9	Imperial Valley
1915	10	3	652	40	30.00	117	30.00	7.3	Pleasant Valley, Nevada
1915	11	21	013	32	0.	115	0.	7.1	Volcano Lake, B.C.
1915	12	31	1220	41	0.	126	0.	6.5	W. of Eureka
1916	2	3	5 3	41	0.	117	48.00	5.9	N. of Pleasant Valley,NV
1916	10	23	244	34	54.00	118	54.00	5.3	Tejon Pass region
1916	11	10	911	35	30.00	116	0.	6.1	S. of Death Valley
1918	4	21	2232	33	48.00	117	0.	6.9	San Jacinto
1918	7	15	023	41	0.	125	0.	6.5	W. of Eureka
1922	1	26	931	41	0.	126	0.	6.0	W. of Eureka
1922	1	31	1317	41	0.	125	30.00	7.3	W. of Eureka
1922	3	10	1121	36	0.	120	30.00	6.3	Parkfield
1923	1	22	9 4	40	30.00	124	30.00	7.2	Cape Mendocino
1923	7	23	730	34	0.	117	18.00	6.0	San Bernardino region
1925	6	4	12 2	41	30.00	125	0.	6.0	W. of Eureka
1925	6	29	1442	34	18.00	119	48.00	6.3	Santa Barbara
1926	10	22	1235	36	37.00	122	21.00	6.1	Monterey Bay
1926	10	22	1335	36	33.00	122	11.00	6.1	Monterey Bay

1926	12	10	838	40	45.00	126	0.	6.0	W. of Cape Mendocino
1927	9	18	2 7	37	30.00	118	45.00	6.0	Bishop region
1927	11	4	1350	34	42.00	120	48.00	7.3	SW of Lompoc
1932	6	6	844	40	45.00	124	30.00	6.4	Eureka
1932	12	21	610	38	45.00	118	0.	7.2	Cedar Mountain, Nevada
1933	1	5	651	38	46.00	117	44.00	5.9	Cedar Mountain, Nevada
1933	3	11	154	33	37.00	117	58.00	6.3	Long Beach
1933	6	25	2045	39	4.00	119	20.00	6.1	Yerington, Nevada
1934	1	30	2016	38	18.00	118	24.00	6.3	Excelsior Mountain,NV
1934	6	8	447	36	0.	120	30.00	6.0	Parkfield
1934	7	6	2248	41	15.00	125	45.00	6.5	W. of Eureka
1934	12	30	1352	32	15.00	115	30.00	6.5	Laguna Salada, B.C.
1934	12	31	1845	32	0.	114	45.00	7.0	Colorado R. delta
1935	2	24	145	31	59.00	115	12.00	5.3	Colorado R. delta
1936	6	3	915	40	0.	125	30.00	5.9	W. of Cape Mendocino
1937	3	25	1649	33	24.00	116	16.00	6.0	Buck Ridge
1940	2	8	8 5	39	45.00	121	15.00	6.0	Chico
1940	5	19	436	32	44.00	115	30.00	7.1	Imperial Valley
1940	12	7	2216	31	40.00	115	5.00	5.5	Colorado R. delta
1941	2	9	944	40	42.00	125	24.00	6.6	W. of Cape Mendocino
1941	4	9	1708	31	0.00	114	0.00	5.3	Gulf of California
1941	5	13	16 1	40	18.00	126	24.00	6.0	W. of Cape Mendocino
1941	7	1	750	34	22.00	119	35.00	5.9	Carpenteria
1941	9	14	1643	37	34.00	118	44.00	5.8	Tom's Place
1941	9	14	1839	37	34.00	118	44.00	6.0	Tom's Place
1941	10	3	1613	40	24.00	124	48.00	6.4	W. of Cape Mendocino
1942	10	21	1622	33	3.00	116	5.	6.5	Fish Creek Mountains
1942	12	3	944	39	42.00	119	18.00	5.9	N. of Wadsworth, Nevada
1945	5	19	15 7	40	24.00	126	54.00	6.2	W. of Cape Mendocino
1945	9	28	2224	41	54.00	126	42.00	6.0	W. of Crescent City
1946	3	15	1349	35	44.00	118	3.00	6.3	Walker Pass
1947	4	10	1558	34	59.00	116	33.00	6.4	Manix
1948	12	4	2343	33	56.00	116	23.00	6.5	Desert Hot Springs
1948	12	29	1253	39	33.00	120	5.00	6.0	Verdi, Nevada
1949	3	25	456	41	18.00	126	0.	6.2	W. of Eureka
1949	5	2	1125	34	1.	115	41.00	5.9	Pinto Mountain
1951	10	8	410	40	15.00	124	30.00	6.0	W. Of Cape Mendocino
1951	12	26	046	32	48.00	118	18.00	5.9	San Clemente Island
1952	7	21	1152	35	0.	119	1.00	7.7	Kern County earthquake
1952	7	21	12 5	35	0.	119	0.	6.4	Kern County
1952	7	23	038	35	22.00	118	35.00	6.1	Kern County
1952	7	29	7 3	35	23.00	118	51.00	6.1	Bakersfield
1952	11	22	746	35	44.00	121	12.00	6.0	Bryson
1954	1	12	2333	35	0.	119	1.00	5.9	W. of Wheeler Ridge
1954	3	19	954	33	17.00	116	11.00	6.2	Arroyo Salada
1954	7	6	1113	39	25.00	118	32.00	6.6	Rainbow Mountain, Nevada
1954	7	6	22 7	39	18.00	118	30.00	6.4	Rainbow Mountain, Nevada
1954	8	24	551	39	35.00	118	27.00	6.8	Stillwater, Nevada
1954	8	31	2220	39	30.00	118	30.00	6.3	Stillwater, Nevada
1954	10	24	944	31	30.00	116	0.	6.0	W. of Santo Tomas, B.C.
1954	11	12	1226	31	30.00	116	0.	6.3	W. of Santo Tomas, B.C.
1954	11	25	1116	40	16.00	125	38.00	6.5	W. of Cape Mendocino
1954	12	16	11 7	39	19.00	118	12.00	7.1	Fairview Peak, Nevada
1954	12	16	1111	39	30.00	118	0.	6.8	Dixie Valley, Nevada
1954	12	21	1956	40	56.00	123	47.00	6.6	E. of Arcata
1956	2	9	1432	31	45.00	115	55.00	6.8	San Miguel, B.C.
1956	2	9	1524	31	45.00	115	55.00	6.1	San Miguel, B.C.
1956	2	14	1833	31	30.00	115	30.00	6.3	San Miguel, B.C.
1956	2	15	120	31	30.00	115	30.00	6.4	San Miguel, B.C.
1956	10	11	1648	40	40.00	125	46.00	6.0	W. of Cape Mendocino
1956	12	13	1315	31	0.	115	0.	6.0	W. shore, Gulf of Calif.
1959	3	23	710	39	36.00	118	1.00	6.3	Dixie Valley, Nevada
1959	6	23	1435	39	5.00	118	49.00	6.1	Schurz, Nevada
1960	8	9	739	40	19.00	127	4.00	6.2	W. of Cape Mendocino
1966	6	28	426	36	0.	120	30.00	6.0	Parkfield
1966	8	7	1736	31	48.00	114	30.00	6.3	Gulf of California

1966	9	12	1641	39	25.00	120	9.00	6.0	Truckee
1968	4	9	228	33	11.00	116	8.00	6.5	Borrego Mountain
1968	6	26	142	40	14.00	124	16.00	5.4	Punta Gorda
1971	2	9	14 0	34	25.00	118	24.00	6.5	San Fernando
1973	2	21	1445	34	4.00	119	2.00	5.2	Point Mugu
1976	11	26	1119	41	18.00	125	42.00	6.3	W. of Orick
1979	8	6	17 5	37	7.00	121	31.00	5.7	Coyote Lake
1979	10	15	2316	32	36.00	115	18.00	6.5	Imperial Valley
1980	01	24	1900	37	50.00	121	47.00	5.8	Livermore
1980	5	25	1633	37	36.00	118	50.00	6.1	Mammoth Lakes
1980	5	25	1649	37	39.00	118	54.00	5.9	Mammoth Lakes
1980	5	25	1944	37	33.00	118	49.00	5.8	Mammoth Lakes
1980	5	27	1450	37	29.00	118	48.00	6.0	Mammoth Lakes
1980	6	9	328	32	12.00	115	5.00	6.4	Victoria, B.C.
1980	11	8	1027	41	7.00	124	40.00	7.2	W. of Eureka
1981	4	26	1209	33	8.00	115	39.00	6.0	Westmorland
1981	9	4	1550	33	40.00	119	7.00	5.9	N. of Santa Barbara Isl.
1981	9	30	1153	37	35.00	118	52.00	5.8	Mammoth Lakes
1983	5	2	2342	36	14.00	120	19.00	6.5	Coalinga
1983	7	22	239	36	14.00	120	25.00	5.7	Coalinga
1984	4	24	2115	37	19.00	121	39.00	6.1	Morgan Hill
1984	9	10	314	40	23.00	127	9.00	6.7	Mendocino Fracture Zone
1984	11	23	18 8	37	27.00	118	36.00	5.7	Round Valley
1985	8	4	12 1	36	8.00	120	10.00	5.9	North Kettleman Hills
1986	7	8	920	34	0.	116	36.00	6.0	North Palm Springs
1986	7	20	1429	37	34.00	118	26.00	5.6	Chalfant Valley
1986	7	21	1442	37	32.00	118	26.00	6.2	Chalfant Valley
1986	7	31	722	37	28.00	118	22.00	5.2	Chalfant Valley
1987	10	1	1442	34	3.00	118	5.00	5.8	Whittier Narrows
1987	11	24	153	33	4.00	115	47.00	6.2	Elmore Ranch fault
1987	11	24	1316	33	1.00	115	51.00	6.6	Superstition Hills
1989	10	18	0004	37	2.19	121	52.98	7.1	Loma Prieta
1991	8	16	2226	41	38.00	125	52.00	6.3	W. of Crescent City
1991	8	17	1929	40	17.00	124	14.00	6.2	Punta Gorda
1991	8	17	2217	41	41.00	126	3.00	7.1	W. of Crescent City
1992	4	23	450	33	58.00	116	19.00	6.1	Joshua Tree
1992	4	25	1806	40	20.00	124	14.00	7.2	Cape Mendocino
1992	4	26	741	40	26.00	124	36.00	6.5	Cape Mendocino
1992	4	26	1118	40	23.00	124	35.00	6.6	Cape Mendocino
1992	6	28	1157	34	12.00	116	26.00	7.3	Landers
1992	6	28	1505	34	12.00	116	50.00	6.2	Big Bear
1993	5	17	2320	37	9.00	117	50.00	6.1	Big Pine
1994	1	17	1230	34	13.00	118	32.00	6.7	Northridge
1994	9	01	1515	40	27.00	125	54.00	6.9	Mendocino Fracture Zone
1994	9	12	1223	38	49.00	119	37.00	6.0	Carter's Station, Nevada
1995	2	19	403	40	37.00	125	54.00	6.6	W. of Eureka
1995	9	20	2327	35	46.00	117	38.00	5.5	Ridgecrest
1996	7	24	2016	41	47.04	125	54.66	5.7	W. of Eureka
1997	1	22	717	40	16.32	124	23.64	5.7	Punta Gorda
1999	8	1	1606	37	23.40	117	4.80	5.7	Scotty's Junction,Nevada
1999	10	16	947	34	35.64	116	16.26	7.1	Hector Mine
2000	3	16	1520	40	23.16	125	16.74	5.9	Mendocino Fracture Zone
2003	12	22	1916	35	41.98	121	5.84	6.5	San Simeon
2004	9	28	1815	35	48.60	120	22.22	6.5	Parkfield

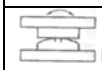
SAMPLE EARTHQUAKE EVALUATION FORM

POST EARTHQUAKE BRIDGE EVALUATION FORM Sheet Number_____

Inspector's Name _____ Affiliation_____ Date _____ Time _____

Earthquake Name_____ Country_____ Region_____

DESCRIPTION OF BRIDGE	DESCRIPTION OF DAMAGE
Bridge Name_____ Br. # _____ Route #___ Location_____ Bridge Bearing _____ Length _____ Number of Spans __Simple Spans Continuous Hinges yes ☐ no ☐ locations in spans_____ River Xing RR Xing Interchange Other ___	Overall Rating No Damage Minor Damage Moderate Damage Major Damage Collapse
Suspension Cable Stay Steel Truss Steel Arch Steel Box Girder Steel I Girder Concrete Box Girder Concrete T Girder Concrete Slab Precast Girder Concrete Arch Timber Arch Timber Truss Timber Girder Bascule Lift Swing Other _____	Dropped Spans yes ☐ no ☐ spans_____ Span Movement _____ Girder Damage _____ Other Superstructure Damage _____ _____ Indications of soil movement _____ _____

BEARING TYPES _____		Damage	Location	Description
Roller	Plane Sliding	Restrainer Hinge Joint	_____ _____ _____	_____ _____ _____ ___
 Rocker	Curved Sliding	Bearings Shear Key Abutment	_____ _____ _____	_____ _____ _____ ___
Knuckle	Disc	Bent Bent Cap	_____ _____	_____ _____ __
Leaf	Elastomeric	Column	_____ _____	_____ _____ __
Link	Isolation/Damping	Footing	_____ _____ _____	_____ _____ _____ ___

Abutments / Piers / Foundations	Roll #	Frame #	Photo Log
Abutments Seat Type Diaphragm Other _____ Piers Concrete Walls Single Col. Multicol. Steel Tower Single Col. Multicol. Other _____ Foundations spread pile pileshaft caisson other ____	_____ _____ _____ _____ _____	_____ _____ _____ _____ _____	_____ _____ _____ _____ _____ ___
Retrofit yes ☐ no ☐ Shear Keys ☐ Catcher Blocks ☐ Restrainers ☐ Abutments _____ Bents _____ Other___ _____	_____ _____ _____ ___	_____ ____ _____ _____ _	_____ _____ _____ _____

Additional Comments _____
_____
_____

SKETCH DAMAGE