Wasatch Front Seismic Risk Regional Seminar

Seminar 2: Earthquake Research and Mitigation

A Seminar for Earth Sciences and Engineering Professionals

November 30, 1994
Salt Lake Airport Hilton
Salt Lake City, Utah

EERI

EERI Regional Seminar Series
Funded by the Federal Emergency Management Agency

Cosponsored by
Utah Seismic Safety Commission
Utah Geological Survey
Seismograph Stations at the University of Utah
Departments of Civil Engineering, and Geology and Geophysics, University of Utah
Department of Civil and Environmental Engineering, Utah State University (Logan)
Civil and Environmental Engineering Department, Brigham Young University
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EERI REGIONAL SEMINAR SERIES

Regional seminars present the latest developments in earthquake engineering and research, and are tailored to meet the interests and needs of the region in which they are presented.

The Wasatch Front Seismic Risk Regional Seminars are designed to provide engineers, earth scientists, architects, and public officials in the western United States with state-of-the-art information on earthquake engineering, as well as provide the professional community with a better understanding of structural and non-structural earthquake damage mitigation techniques.

EERI

The Earthquake Engineering Research Institute is a national, non-profit, technical society of engineers, geoscientists, architects, planners, social scientists and public officials. The Institute is devoted to the advancement of the science and practice of earthquake engineering and the solution of national earthquake engineering problems to protect people and property from the effects of earthquakes. With FEMA support, EERI has developed this regional seminar to further the goals of NEHRP, and to provide the professional community in the western United States with innovative techniques to mitigate the risks of earthquakes.

FEMA

The Federal Emergency Management Agency (FEMA) is the Lead Agency of the National Earthquake Hazards Reduction Program (NEHRP). In fulfilling this role, FEMA supports conferences that enhance the effectiveness of earthquake hazard reduction science and technology, and that increase opportunities for participation by individuals who can then contribute to the advancement and progress of the Program.

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WASATCH FRONT SEISMIC RISK REGIONAL SEMINARS

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• EERI Regional Seminar Agenda

WASATCH FRONT SEISMIC RISK SEMINAR 2 • Earthquake Research and Mitigation

WEDNESDAY, NOVEMBER 30, 1994

8:00 - 8:30 am	Registration
8:30 - 8:40 am	Introduction/Welcome Ann Becker, *Chair* *Seismologist, Woodward-Clyde Federal Services*
8:40 - 9:00 am	FEMA EARTHQUAKE HAZARD MITIGATION PROGRAM Michael Mahoney, *Geophysicist, Mitigation Directorate* *Federal Emergency Management Agency*
9:00 - 9:40 am	WASATCH FRONT EARTHQUAKE GEOLOGY William Lund, *Deputy Director* *Utah Geological Survey*
9:40 - 10:20 am	WASATCH FRONT SEISMICITY AND EXPECTABLE STRONG GROUND MOTION Walter Arabasz, *Research Professor* *Geology/Geophysics, University of Utah*
10:20 - 10:50 am	Break
10:50 - 11:30 am	SOIL RESPONSE IN THE SALT LAKE BASIN Kyle Rollins, *Associate Professor, Civil Engineering Department, Brigham Young University*
11:30 am - 12:10 pm	PERFORMANCE-BASED SEISMIC DESIGN Lawrence Reaveley, *Professor* *Civil Engineering Department, University of Utah*
12:10 - 1:30 pm	Lunch
1:30 - 2:10 pm	ENERGY-DISSIPATING STRUCTURAL SYSTEMS Chris Pantelides, *Assistant Professor* *Civil Engineering Department, University of Utah*
2:10 - 2:40 pm	Break
2:40 - 3:20 pm	LIFELINE PERFORMANCE CONSIDERATIONS IN REGIONAL EARTHQUAKE PLANNING Peter McDonough, *Senior Engineer* *Mountain Fuel Supply Company*
3:20 - 4:00 pm	OVERVIEW OF UTAH SEISMIC SAFETY COMMISSION T. Leslie Youd, *Chair* *Utah Seismic Safety Commission, and* *Professor, Civil Engineering Department* *Brigham Young University*
4:00 - 4:40 pm	ROUNDTABLE QUESTION AND ANSWER

Ann Becker
Chair, Organizing Committee

Ann Becker is a Project Seismologist with the Seismic Hazards Branch of Woodward-Clyde Federal Services based in Oakland, California. She has over twelve years of consulting experience in the fields of engineering seismology and earthquake engineering. Her professional experience includes performing numerous seismic hazards assessments, both site-specific and regional, at sites throughout the United States, the Pacific Rim, the Caribbean, and Europe.

Dr. Becker has made presentations in seismology and seismic hazards to non-technical audiences as well as at professional society meetings and seminars. She holds a bachelor's degree in civil engineering and a master's degree in structural engineering. She received her Ph.D. in seismology from the University of California, Berkeley, specializing in near-source directivity effects on earthquake ground motions. She was named the EERI/FEMA NEHRP Graduate Fellow in Earthquake Hazard Reduction for 1992-1993.

FEMA Earthquake Hazard Mitigation Program

presented by

MICHAEL MAHONEY

Michael Mahoney is a Senior Geophysicist with the Federal Emergency Management Agency's (FEMA) Mitigation Directorate, Program Development Branch, and its predecessor, the Office of Earthquakes and Natural Hazards. Since 1991, Mr. Mahoney has been responsible for all of FEMA's National Earthquake Hazards Reduction Program (NEHRP) technical activities relating to new construction. He is presently serving as Project Officer for the soon to be completed 1994 update and the recently contracted 1997 update of the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*. In addition, Mr. Mahoney has been named as Project Officer for FEMA's Program for Reduction of Earthquake Hazards in Steel Moment Frame Buildings. He is also serving as Project Officer for the Problem Focused Studies Project and the Home Builder's guides to Seismic-Resistant and Wind-Resistant Construction.

During this time, Mr. Mahoney also served as FEMA's representative on a Federal interagency report on the Northridge earthquake, initiated FEMA's Estimating Losses from Future Earthquakes Project, and served as a technical advisor to FEMA's Hurricane Program, and was its representative on the Hurricane Andrew Building Performance Assessment Report and the Federal Interagency Hazard Mitigation Team Report.

From 1984 to 1991, Mr. Mahoney was employed as a Physical Scientist with FEMA's Office of Loss Reduction, which was part of the National Flood Insurance Program. Mr. Mahoney was responsible for coordinating work with the nation's model building code groups, managing FEMA's Post Flood Damage Assessment Project, and was the Project Officer for several technical publications. During this time, Mr. Mahoney was recognized for his role in the Federal response after Hurricane Hugo.

From 1978 to 1984, Mr. Mahoney was employed by Factory Mutual Engineering as a Loss Prevention Consultant. He received his Bachelor of Science degree in physics from Grove City College and his Master's degree in physics from Kent State University.

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3.	Soil Response in the Salt Lake Basin *Kyle Rollins, Brigham Young University, and Scott M. Adan, Karren and Associates*
4.	Performance Based Seismic Design *Lawrence D. Reaveley, University of Utah, and Christopher Rojahn, Applied Technology Council*
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1.

Review of Wasatch Front Earthquake Geology and Update on Recent Paleoseismic Studies

by

William R. Lund

William R. Lund

William R. Lund, Deputy Director of the Utah Geological Survey, Salt Lake City, serves as chief geologist overseeing the technical and editorial quality of all UGS products. He is editor of *Paleoseismology of Utah Special Studies Series* and principal investigator on several NEHRP funded paleoseismic investigations. He has been with the Utah Geological Survey since 1980 and worked as a consulting geologist from 1973 to 1980.

He has authored numerous publications and served as the president of the Utah Geological Association in 1991-92. He is a member of the Association of Engineering Geologists, the Geological Society of America, and the American Geophysical Union. He serves on the Utah Resources Development Coordinating Committee, the Utah Earthquake Advisory Board Earth Sciences Standing Committee, and the Utah Science Center Program Development Committee - Earth and Biosphere Program.

Mr. Lund received his B.S. in geology from the University of Idaho in 1973. He is a Registered Geologist in the states of Arizona and Oregon.

REVIEW OF WASATCH FRONT EARTHQUAKE GEOLOGY and UPDATE ON RECENT PALEOSEISMIC STUDIES

William R. Lund
Utah Geological Survey

Presented At
Earthquake Engineering Research Institute
Wasatch Front Seismic Risk
Regional Seminar
Salt Lake City, Utah
November 30, 1994

1 - Title Slide

- Twofold purpose for today's talk: (1) Set the geologic stage as it relates to earthquakes in the Wasatch Front region (WFR), and (2) provide an update of the most recent WFR paleoseismic studies.
- Information presented today represents the results of more than two decades of work by many organizations and individuals too numerous to mention individually.

2 - Regional Geologic Setting WFR

- Physiographic Provinces; Middle Rocky Mountains, Colorado Plateau, Basin & Range, Sierra Nevada
- Yellowstone caldera, Snake River Plain, plate tectonics implications
- Basin & Range extensional regime creates numerous north-south trending, normal-slip faults and typical Basin & Range topography.
- Wasatch fault zone is the most prominent of these faults, borders the B&R/MRM provinces and is the longest and most continuous normal fault in North America.

3 - Intermountain Seismic Belt

- Zone of intraplate earthquake activity extending from north-central Montana to northwestern Arizona.
- Three historical surface faulting earthquakes: 1934 Hansel Valley, Utah (M 6.6); 1959 Hebgen Lake, Montana (M 7.5); and 1983 Borah Peak, Idaho (M 7.3)
- Based on historical record, the threshold for surface fault rupture in the ISB is about M 6.5.

4 - Utah Earthquake Epicenters

- Earthquake epicenters outline the ISB in Utah.
- Larger historical earthquakes shown by stars. 1934 event is Hansel Valley earthquake (M 6.6),

Utah's only historical surface faulting earthquake.

- Only in rare instances can historical earthquake epicenters be assigned to a known fault.

5. - Utah Quaternary Fault Map

- Shows location of Quaternary (younger than 2 million years and therefore potentially active) faults in Utah.
- Faults are recognized by tectonic geomorphic features (chiefly scarps denoting surface displacement) .
- Faults are color coded according to timing of most recent movement.
- Note that the WFZ is the most prominent, but by no means the only active fault in Utah.
- Most of Utah's potentially active faults are located in the ISB.

6 - WFZ Location Map Showing Population Centers

- Of the WFR's potentially active faults, the WFZ has received the most attention, and rightfully so since it is the largest Quaternary fault in the region, shows abundant evidence of geologically recent surface faulting earthquakes, and trends through or near Utah's major population centers.
- Significance first recognized by G.K. Gilbert in 1880s
- Subject of intense study since the 1970s and particularly in the mid- to late 1980s when it was the focus of the National Earthquake Hazard Reduction Program.

7 - WFZ at the Mouth of Little Cottonwood Canyon (SE Salt Lake County)

- Note broad zone of deformation with numerous fault strands and graben formed by antithetic faults.

8 - WFZ Scarp Near South Fork Dry Creek (SE Salt Lake County)

- Multiple-event scarp approximately 6 meters high (modified by construction activity).

9 - WFZ (East Bench fault) trending through Salt Lake City

- Utah's critical concern in a nutshell - In Utah, active faults and citizens occupy the same space.

10 - Paleoseismology - Definition, Techniques, Information Acquired

- Two ways to learn about the seismicity (activity) of a fault: (1) the historical seismic record - short for this part of the world, and (2) paleoseismic studies which provide information on prehistoric, surface faulting earthquakes - allows us to extend a fault's earthquake history several thousand years into the past.

- Techniques used include: detailed geologic mapping, trenching, and geomorphic analysis of tectonic features.
- Information that <u>may</u> be acquired: number of events, amount of displacement, timing of events, recurrence intervals, slip rates, and fault geometry.
- WFZ was first normal-slip fault on which detailed paleoseismic studies were conducted, many of the principals that form the foundations of modern paleoseismology were developed on the WFZ.

11 - Geologic Map of Part of the WFZ

- 1980s NEHRP provided first reasonably detailed (1:50,000 scale) geologic maps of the WFZ.
- Fault zone is commonly expressed as a complex zone of fault strands and antithetic faults with associated graben.
- Relative age of geologic units cut by fault strands gives some idea of the age of faulting.

12 - Map Showing Location of Research Trench Sites on WFZ

- Sites where detailed paleoseismic trenching studies have been conducted.
- Trenches excavated across faults are carefully logged and analyzed to determine the number, size, and timing of surface faulting events.

13 - WFZ Segmentation Map

- Paleoseismic studies show that the WFZ consists of at least 10 independently seismogenic segments.
- The central six, Brigham City to Levan, experienced Holocene surface faulting, and the central five, Brigham City to Nephi, show evidence for multiple Holocene movements. 10,000
- End segments, three in north and one in south, show no evidence of movement in post-Bonneville time.

14 - WFZ Space/Time Diagram

- Distribution of surface faulting earthquakes through time on the fault segments.
- Each Holocene segment shows a discrete earthquake history and corresponding composite recurrence interval and slip rate.
- Composite recurrence intervals for fault segments commonly are in the one to two thousand year range, recurrence intervals between individual events may approach 5,000 years.
- Composite recurrence interval for the WFZ as a whole is about 400 years.

15 - Log East Ogden Trench EO-1

- Net slip 1 to 4+ meters per event, typically 1 to 3 meters.
- Paleomagnitudes calculated from net slip and/or surface rupture length typically M 6.8 - 7.3 (but up to M 7.5)
- Near surface fault geometry - high angle (80°+), pure dip-slip movement, down-to-the-west. Antithetic faults are high angle down-to-the-east.

16 - WFR Quaternary Fault Map Showing Faults With Paleoseismic Studies

- Faults other than the WFZ also present an earthquake threat to the WFR.
- Several faults have been the subject of paleoseismic investigations (color coded on map).
- Results are too numerous to report here, but in general, studies show the faults are capable of generating M 7+ earthquakes, but recurrence intervals are typically measured in tens of thousands of years.
- Many other potentially active faults require investigation to adequately characterize the WFR's seismic hazard.

17 - WFR Paleoseismic Study Update

18 - Benchmark Publications

- What has been reported above concerning the WFZ and other faults in the WFR is current through about 1992.
- Two benchmark publications present and/or summarize most of this information:

    1. **U.S. Geological Survey Professional Paper 1500**; *Assessment of Regional Earthquake Hazard and Risk Along the Wasatch Front, Utah* (1992), and
    2. **Utah Geological Survey Bulletin 127**; *Quaternary Tectonics of Utah with Emphasis on Earthquake Hazard Characterization* (1993).

- Since 1992, four paleoseismic studies have been or are being conducted which add to our knowledge of the earthquake hazard facing the WFR.

19 - WFZ Map Showing Locations of New Paleoseismic Studies

- Three new studies on the WFZ, one near Brigham City on the Brigham City segment, one at Kaysville on the Weber segment, and one at South Fork Dry Creek/Dry Gulch in Sandy on the Salt Lake City segment.

20 - New Information Brigham City Segment

- o Segment originally trenched by USGS/UGS (Personius, 1992). Identified two Holocene earthquakes, the most recent occurred 3.6 kya with the penultimate event at 4.7 kya.
- o New study by McCalpin and Forman (1994) excavated 14 trenches and identified 6 surface rupturing earthquakes since 13 kya.
- o A new (younger) most recent event identified at 2,377±100 yr B.P.
- o Individual recurrence intervals between events range from 1,148 yr to 4,943 yr.
- o Mean recurrence for past five events 1,424 yr.
- o Elapsed time since MRE 2,377 yr - 2.5 standard deviations larger than the mean recurrence.

21 - Aerial View Kaysville Trench Site

- o Reoccupation of the Kaysville site on the Weber segment - first site trenched on the WFZ (1978 by Woodward-Clyde Consultants). Study identified multiple paleoseismic events, but their timing was poorly constrained by a single radiocarbon age estimate.
- o New study employed both radiocarbon dating of low-carbon paleosol A-horizons and thermoluminescence dating of fine-grained, graben-fill sediments.

22 - New Information Weber Segment

- o New study revealed evidence for five to six paleoseismic surface faulting earthquakes in past 13 kyr.
- o Latest three events had net slips of 1.4 to 3.4 m, big spread in displacement values tends not to support "characteristic earthquake theory."
- o Two latest Kaysville events correlate well with two latest "well-identified" events on the same segment at East Ogden 25 km to the north.
- o Third East Ogden event (3.4-4.0 kyr) not recognized at Kaysville.
- o Post-Provo composite recurrence interval 2.2-2.6 kyr; MRE 0.6-0.8 kya.

23 - Map of the Salt Lake City Segment

- o Holocene earthquake chronology for SLC segment based on information from two trench sites (Little Cottonwood [1979] and South Fork Dry Creek [1985]), it was not possible to trench all of the scarps present at either site due to landowner constraints.
- o In 1992, a consulting firm conducting an investigation for a new subdivision excavated a long trench across the WFZ at Dry Gulch a few hundred meters south of the SFDC site.

24 - Dry Gulch Trench Log

- o The trench exposed a pair of stacked colluvial wedges on a scarp also present, but not trenched, at SFDC.
- o Radiocarbon age estimates from paleosols buried by the colluvial wedges provided evidence for a previously unrecognized surface faulting earthquake on the SLC segment.
- o Resampling at DG and re-excavation of a trench at SFDC to resample buried paleosols there, confirmed the previously unrecognized event.

25 - New Salt Lake City Segment Space/Time Diagram

- o The new event at about 2,400 yr B.P. adds a fourth event to the SLC segment Holocene earthquake record and reduces the segment's composite recurrence interval from 4,000±1,000 yr to 2,400±500 yr (2,150 yr for the past 6,000 yr).

26 - Geologic Map of the SFDC and DG Trench Sites

- o In 1994, the UGS, with partial funding from a USGS NEHRP grant, reoccupied the SFDC site and trenched all remaining scarps in an attempt to develop a complete earthquake chronology at a single location on the SLC segment. Hope to ensure that all Holocene surface faulting earthquakes on the segment have been identified.

27 - Location Map Oquirrh Fault Zone (Tooele County)

- o Study partially funded by a NEHRP grant conducted in 1992-1993 to investigate the earthquake hazard presented by Oquirrh fault zone.
- o OFZ selected for study because geologic mapping and scarp profiles indicated a probable latest Pleistocene/early Holocene age for most recent surface faulting. Fault is located close to Tooele City, Tooele Army Depot, and is only 20 km from the heavily populated Salt Lake Valley. Note location of two trench sites.

28 - Aerial View of Oquirrh Fault Zone

- o Well-developed scarp in unconsolidated basin-fill deposits trends along the west side of the Oquirrh Mountains for 10 km, extends an additional 12 km to the south marked as an abrupt, steep contact between unconsolidated deposits and the mountain front.
- o OFZ displaces (down-to-the-west) the Provo shoreline of Pleistocene Lake Bonneville, so MRE is younger than about 14 kyr.
- o Scarp profiles indicated a probable age of 9-13 kyr for time of last surface faulting.

29 - Pole Canyon Trench Log

- OFZ was trenched at two locations, Big Canyon and Pole Canyon.
- Trenching revealed evidence for three surface rupturing earthquakes during late Pleistocene and Holocene time. Evidence consisted of two stacked colluvial wedges and a degraded scarp free face for a third, older event.

30 - Oquirrh Fault Zone Paleoseismic Summary

- Three events, two late Pleistocene and pre-Lake Bonneville; one mid-Holocene and post-Lake Bonneville.
- MRE 4,300-6,900 yr B.P. - much younger than estimated by scarp profiles. Net slip 2.2-2.7 m.
- Penultimate event 20,300-26,400. Net slip 2.3 m.
- Antepenultimate event >33,950 yr B.P.
- Recurrence interval 13.4-22.1 kyr.

31 - Summary

- Paleoseismic studies performed on many northern Utah faults have greatly expanded our understanding of the Holocene history of large earthquakes in the WFR.
- Large surface-faulting earthquakes occur on average every 170 years on one or another of the active faults in the WFR.
- The Wasatch fault generates a surface-faulting earthquake somewhere along its six central segments about every 400 years.
- The Brigham City segment is the WFZ segment active during the Holocene with the longest elapsed time since a surface-faulting earthquake.

2.

Wasatch Front Seismicity and Expectable Strong Ground Motion

by

Walter J. Arabasz

Biosketch

— Walter J. Arabasz —

Dr. Walter J. Arabasz, a native of Massachusetts, received a bachelor of science degree in geology from Boston College in 1964, and his master of science and Ph.D. degrees in geology and geophysics from the California Institute of Technology in 1966 and 1971, respectively.

After completing his doctoral studies, which involved two years of geological and geophysical studies of the Atacama fault zone in northern Chile, he was awarded a post-doctoral fellowship from the New Zealand government for three years of earthquake research in New Zealand. Subsequently, he spent one year as a research scientist at the Lamont-Doherty Geological Observatory of Columbia University.

Dr. Arabasz joined the University of Utah as a research seismologist in 1974 and has been a member of the faculty since 1976. He is a research professor of geology and geophysics and has been director of the University of Utah Seismograph Stations since 1985. His current research focuses on earthquake hazard analysis, network seismology, and statistical patterns of earthquake occurrence in the Intermountain Seismic Belt.

He presently serves on the National Research Council's Committee on Seismology, its Panel on Seismic Hazard Evaluation, and on the Utah Seismic Safety Commission. He is also a member of the Board of Directors of the Seismological Society of America and is vice-chair of the Council of the National Seismic System.

Dr. Arabasz has provided professional consulting services on earthquake hazard evaluation for engineering firms, the International Atomic Energy Agency, the Department of Energy, the Soil Conservation Service, the Bureau of Reclamation, and the Electric Power Research Institute. He has had ongoing involvement in seismic hazard evaluations for the Jordanelle Dam in Utah and the Department of Energy's waste repository site at Yucca Mountain, Nevada.

Wasatch Front Seismicity and Expectable Strong Ground Motion

by

Walter J. Arabasz

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Salt Lake City, Utah 84112-1183

Summary

My purpose in this presentation is to give an overview, update, and review of relevant current research relating to the seismicity of the Wasatch Front region (Part I) and expectable strong ground motion (Part II).

Earthquake hazards and risk in the Wasatch Front region arise from two sizes of earthquakes: (1) infrequent large surface-rupturing earthquakes (magnitude 6.5 to 7.5) and (2) more frequent moderate-sized, but potentially damaging, non-surface rupturing earthquakes (below about magnitude 6.5). The first occur on identifiable active faults—notably the Wasatch fault—which have evidence of geologically recent movement. The second can occur on "hidden" faults and can cause great damage if they occur beneath an urbanized area. Most of the seven sizable mainshocks (magnitude 4.8 to 5.4) that have occurred in the Utah region since 1987 originated on concealed faults. There is a probability of 0.34 for a magnitude 6.5 or larger earthquake in the Wasatch Front region during the next 50 years, and a larger probability for smaller mainshocks.

As is the case elsewhere, strong ground motions in the Wasatch Front region at the same distance from earthquakes of the same magnitude may involve large variability, due variously to source effects associated with the fault rupture process, path effects related to wave propagation, and site effects due to soil conditions and topography. Probabilistically-assessed peak ground accelerations with a 10 percent probability of exceedance on soil sites in the Wasatch Front area reach a level of about 0.35*g* and 0.7*g* for exposure periods of 50 and 250 years, respectively (Youngs and others, 1987). Peak horizontal ground accelerations analytically estimated by Wong and Silva (1993) for "scenario" earthquakes of M_W 7.0 on the Salt Lake City segment of the Wasatch fault have median values ranging from 0.6 to 1.1*g* at three sites chosen to be representative of near-surface conditions in the Salt Lake Valley. Response spectra varied significantly among these three sites, due in part to the damping of high frequencies at the deep soft-soil sites. The peak accelerations predicted for the scenario earthquakes are comparable to the large peak accelerations caused by the highly destructive M_W 6.7 Northridge, California, earthquake in January 1994.

Part I. Wasatch Front Seismicity

Overview and Update

A comprehensive review of seismicity in the Wasatch Front area is given by Arabasz and others (1992), originally prepared as part of U.S. Geological Survey Open-File Report 87-585 in 1987. Smith and Arabasz (1991) review the seismicity of the Intermountain Seismic Belt. Here I will simply give a general overview based on these sources and references therein.

Utah is transected by the Intermountain Seismic Belt (**Figure 1**), a northerly-trending belt of earthquake activity within the interior of western North America that extends at least 1,500 kilometers from southern Nevada and northern Arizona to northwestern Montana. The Intermountain Seismic Belt is characterized by geologically active normal faults and shallow earthquakes less than 25 kilometers deep.

The earthquake threat in Utah has a dual aspect, relating to (1) infrequent large surface-rupturing earthquakes (magnitude 6.5 to 7.5) and (2) more frequent moderate-sized non-surface-rupturing earthquakes (below about magnitude 6.5). The first occur on identifiable active faults—like the Wasatch fault—which have evidence of geologically recent movement. The second are not constrained to occur on faults which can be seen at the surface and can occur anywhere throughout Utah's main seismic belt. The latter earthquakes can cause great damage if the source of energy release is beneath an urbanized area.

Utah's largest historical earthquake was a magnitude (M_W) 6.6 earthquake in 1934 in Hansel Valley, north of the Great Salt Lake—the only historical shock in the Utah region known to have produced surface faulting. Ground breaking with vertical displacements up to 50 cm occurred over a zone 12 km long. The largest historical shock in the Intermountain region was the 1959 Hebgen Lake, Montana, earthquake of magnitude (M_S) 7.5 (M_W 7.3)—an earthquake which approximates the upper size expectable in the Wasatch Front region. The Hebgen Lake earthquake caused 28 fatalities and produced dramatic geologic effects, including a catastrophic landslide into the Madison River and spectacular vertical displacements up to 5.5 m over a zone 26 km long. One other surface-rupturing earthquake has occurred historically in the Intermountain Seismic Belt. The magnitude (M_S) 7.3 (M_W 6.9) Borah Peak, Idaho, earthquake in 1983 produced 36 km of surface faulting with vertical displacements of up to 2.7 m.

Figure 2 gives a graphic overview of Utah's main seismic belt, depicted by more than 16,000 earthquakes instrumentally located since 1962—including several mainshocks of magnitude 4.8 or greater since 1987 (discussed in a later section). The data come from the University of Utah's regional seismic network, outlined in **Figure 3**. On average, several hundred earthquakes (including aftershocks) are located in the Utah region each year, of which roughly 10 to 20 are felt. Excluding aftershocks, about 9 independent mainshocks of magnitude (M_L) 3.0 or greater occur annually in the Utah region. (During 1993 the University of Utah detected and analyzed nearly 6,000 seismic events. Of these, 33 percent were local earthquakes, 41 percent were teleseisms and regional earthquakes, and 26 percent were blasts. A total of 1,980 earthquakes were located in the Intermountain Seismic Belt—including 1,355 within the Wasatch Front region and 1,619 within the broader Utah region, outlined in Figure 2.)

Two principal guides in judging where earthquakes are likely to occur in Utah are (1) the pattern of historical and instrumentally-located earthquakes and (2) the location of active faults—that is, faults that are considered likely to undergo renewed movements (and hence produce earthquakes) within a period of concern to humans. A third guide is the location of human activities such as the impoundment of reservoirs, the injection of fluids into deep wells, or mining that have the potential for triggering natural earthquakes in areas where tectonic strain energy has already accumulated.

Figure 4 shows the location of active faults and two representative samples of instrumentally located earthquake activity in the Wasatch Front area. As true for most of the Intermountain Seismic Belt, the small to moderate-sized earthquakes tend to be broadly scattered and are not simply associated with mapped active faults—emphasizing the danger of earthquake energy release on "hidden" faults, as occurred in the 1993 Northridge, California, earthquake. The two samples of seismicity compared in Figure 4 show a relatively stationary pattern of background activity, whose details are described by Arabasz and others (1992). Note that the intensely clustered seismicity in the lower right corner of each panel is predominantly induced by active underground coal mining. (One additional cluster in this area during the 1987-1993 time frame, however, relates to a magnitude 5.3 mainshock in 1988.)

Recent Earthquake Sequences

Since the time period of the data set analyzed by Arabasz and others (1992), seven mainshocks of magnitude 4.8 to 5.4 have occurred in the Utah region, and an eighth shock of magnitude 5.9 occurred to the north along the Idaho-Wyoming border. University of Utah seismologists carried out special portable-array studies following seven of these eight mainshocks, which occurred in the following order (see Figure 2): (1) Lakeside, Utah—September 1987, M_L 4.8 (Pechmann and others, 1993); (2) San Rafael Swell, Utah—August 1988, M_L 5.3 (Pechmann and others, 1991); (3) Bear Lake, Utah—November 1988, M_L 4.8 (Pechmann and others, 1991); (4) Southern Wasatch Plateau, Utah—January 1989, M_L 5.4 (Pechmann and others, 1991); (5) Blue Springs Hills, Utah—July 1989, M_L 4.8; (6) St. George, Utah—September 1992, M_L 5.8 (Pechmann and others, 1992, 1994); (7) Terrace Mountains, Utah—November 1992, M_L 4.8; and (8) Draney Peak, Idaho—February 1994, M_W 5.9 (Nava and others, 1994).

All of these moderate-sized mainshocks were below the threshold of surface rupture, and most reflected slip on concealed faults without apparent surface rupture. Because the station spacing of the University of Utah's regional seismic network allows reliable focal depths to be determined for only a small fraction of earthquakes in the Utah region (Arabasz and others, 1992), the correlation of seismicity with geological structure poses a formidable challenge. It is only by aggressively "chasing" aftershocks with portable-array field studies that such information can progressively be pieced together—all adding to our understanding of earthquake behavior in the region. I will illustrate with selected examples (**Figures 5–10**).

Relevant Current Research

Within the context of Wasatch Front seismicity, some other aspects of current earthquake research at the University of Utah—beyond direct studies of the spatial distribution and source mechanics of individual earthquake sequences—are timely to mention. The research relates to: (1) aftershock behavior and real-time hazard estimation after a mainshock, (2) real-time earthquake monitoring for rapid post-earthquake alert, (3) implications of lithospheric flexural rigidity for earthquake occurrence in the Intermountain Seismic Belt, (4) Global Positioning System (GPS) studies of crustal deformation and earthquake potential, and (5) just-beginning research into modeling Coulomb stress changes after normal-faulting earthquakes. Other research relevant to the estimation of strong ground motion is described in Part II.

Aftershock behavior and real-time hazard estimation.—We have been studying aftershock temporal behavior and earthquake clustering in the Utah region—aimed at (1) having the capability to determine probabilities for strong aftershocks or a larger mainshock when a sizable earthquake occurs (Reasenberg and Jones, 1989) and (2) understanding how space-time variations in seismicity in the Utah region may relate to tectonic processes and stress state (Arabasz and Hill, 1994). We've successfully developed a provisional "generic Utah" aftershock model for real-time hazard assessment that reveals significant differences in both the rate decay and rate of production of aftershocks in the Utah/Intermountain region compared to California (**Figure 11**). Compared to the "generic California" aftershock model of Reasenberg and Jones (1989), Utah aftershock sequences, on average, decay more slowly and are less "productive" by a factor of 4 to 5 in terms of normalized aftershock rates after 1 day and the cumulative number of aftershocks during the first 30 days. There is growing evidence that variable aftershock behavior reflects important physical aspects of earthquake sources. For example, research elsewhere suggests that aftershock duration may be directly proportional to mainshock recurrence time, and rate decay shows positive correlation with crustal temperature (and inferred stress relaxation time).

Real-time earthquake monitoring for rapid post-earthquake alert.—One of our key objectives at the University of Utah Seismograph Stations is to use the sensing and data-processing infrastructure of our existing regional seismic network to develop a real-time earthquake monitoring (RTEM) system—as described and advocated by the National Research Council (*Real-Time Earthquake Monitoring, Early Warning and Rapid Response*, National Academy Press, Washington, D.C., 1991). The costs for transforming any individual regional seismic network into an effective RTEM system ultimately ranges into the hundreds of thousands of dollars, but progressive steps can be made. We've made the following three-step transformation plan for our network: (1) Develop the basic capability to *automatically* process and transmit the location and size of a potentially disruptive earthquake, based on our existing 100-station network. (2) Develop effective communications links for automated broadcasting of rapid postearthquake alert to emergency-response managers. (3) Expand the distribution and quality of our network sensors—including the capability to record, process, and report within minutes the level and geographic extent of strong ground shaking. We've made significant progress on step 1 and are involved in experimental work with step 3.

Flexural rigidity, seismicity, and active faulting.—Theoretical studies of flexural rigidity are providing important new insights into the spatial location and underlying causes of seismicity and active faulting on a regional scale in Utah and the broader Intermountain Seismic Belt (Lowry, 1994; Lowry and Smith, 1994). Flexural rigidity is the resistance to bending, due to buoyancy forces, of the elastic layer of the outer part of the earth. It can be equivalently expressed in terms of an effective elastic thickness, which depends fundamentally on temperature, lithology, and stress state. Elastic thickness in the Intermountain region is shown in **Figure 12** and is about 6–12 km in the Basin and Range province, about an average of 20–25 km in the Colorado Plateau, and about 40+ km in the Middle Rocky Mountains (reaching a thickness of as much as 77 km) (Lowry, 1994). Most of the seismicity and major active faults in the Intermountain region appear to occur not where elastic thickness is lowest, but rather along a regional gradient from low to high elastic thickness. Lowry (1994) hypothesizes that the stresses responsible for the concentrated tectonic activity may originate locally due to lateral variations in crustal buoyancy properties (in particular, crustal thickness) rather than distant plate-tectonic interactions.

Global Positioning System (GPS) studies.—In 1992, R.B. Smith, C.M. Meertens, and student researchers at the University of Utah began a major project to measure crustal deformation along the Wasatch fault and other faults in the Wasatch Front area using high-precision Global Positioning System (GPS) satellite surveying (**Figure 13**). The goals of the project (Meertens and Smith, 1994) are: (1) to evaluate the relative importance of aseismic creep in local crustal deformation, including the possibility of precursory aseismic deformation that may precede a large earthquake; (2) to develop a baseline of GPS stations for measuring future coseismic deformation; (3) to measure rates of crustal deformation along individual segments of the Wasatch fault; and (4) to further an understanding of the mechanics of the Wasatch fault. A companion goal has been the training of qualified GPS observers from cooperating agencies in order to advance high-precision surveying for other engineering applications. The distribution of GPS stations established in 1992, 1993, and 1994, together with some preliminary results, are shown in Figure 13.

Coulomb stress changes.—Seismic slip on a fault occurs when a combination of shear and confining stresses exceeds what is called the Coulomb failure criterion, with the resultant stress conducive to slip called Coulomb stress (e.g., King and others, 1994). In some cases, stress changes can trigger or advance the timing of seismic slip on a pre-stressed fault already close to failure. Triggering of natural earthquakes by tidal stress changes of a few millibars appears to be rare, but natural earthquakes are well known to have been triggered by stress changes of the order of tenths of a bar to more than one bar caused by reservoir loading, fluid injection, and mining activity. Accumulating evidence from studies of California earthquakes suggests that stress transfer from one fault to another—due to a change in the static stress field resulting from a fault dislocation—is also an important factor in triggering or advancing the timing of seismic slip (e.g., Stein and others, 1992, 1994; Reasenberg and Simpson, 1992; Harris and Simpson, 1992; King and others, 1994). The example of the 1994 Northridge earthquake is illustrated in **Figure 14**. We are just beginning collaborative studies with California researchers to apply the modeling of Coulomb stress changes to normal-faulting earthquakes in Utah and the Intermountain region. The modeling is relevant to understanding changes in seismicity patterns, fault interactions, and the potential for a sizable earthquake to advance seismic slip on another fault by decades or even a century.

Some Remarks About Earthquake Recurrence and Probabilities

Before turning from Wasatch Front seismicity to expectable ground motion, it will be useful to consider briefly the rate of occurrence of moderate-sized and large earthquakes in the Wasatch Front area and probabilities of their occurrence. An understanding of these rates is essential for a perspective about expectable strong ground motions.

Based on instrumentally-recorded earthquakes since 1962, potentially damaging earthquakes of magnitude 5.5 and greater are projected to occur, on average, somewhere in the Utah region about once every 7 years and in the Wasatch Front region about once every 24 years (**Figure 15**). Eleven mainshocks of magnitude 5.5 or greater have occurred in the Utah region since 1900, the most recent being a magnitude 5.8 earthquake near St. George in 1992. The last in the Wasatch Front region was a magnitude 6.0 shock along the Idaho-Utah border in 1975.

Instrumental seismicity is basic for estimating the frequency of earthquakes up to about magnitude 6½ in Utah—the upper size of historical shocks to date. Instrumental monitoring is also essential for identifying and characterizing the behavior of "hidden" faults that aren't simply recognizable from the surface geology. To estimate how often large surface-rupturing earthquakes occur, we rely fundamentally on paleoseismologic data of the type described by William Lund (this volume). Although the data sets for historical seismicity and large prehistoric earthquakes are distinct, the respective rates of occurrence are in general agreement (see graph in Figure 15).

When we consider how often earthquakes occur, we speak of average rates of occurrence. However, earthquakes can generally be modeled by a random process in which the spacing between occurrences in time is quite variable, leading to clusters and gaps (**Figure 16**)—even though the process involves an *average* long-term rate of occurrence.

The most commonly used mathematical model for the occurrence of independent earthquake mainshocks is their representation by the well-known Poisson process, that is, a random memoryless arrival process in which events occur with a stationary average rate λ and with interevent times that have an exponential distribution.

The fundamental equation that describes the probability mass function for a Poisson distribution (e.g., Benjamin and Cornell, 1970) can be written in the form:

$$P[n \mid \lambda, t] = (\lambda t)^n \frac{e^{-\lambda t}}{n!}$$

where λ is the mean process rate and the random variable *n* represents the number of events occurring within a time interval *t*. The equation reads that the probability of *n* events, given λ and *t*, is computed from the right-hand side of the equation. It should be noted that λ may not be known with certainty. For a case in which a Poisson process is operative and *n* events are observed in *t* years, the most likely value of λ is *n/t*. However, the true value of λ can lie within a range whose confidence limits can be estimated using a maximum-likelihood approach (see Arabasz and others, 1989).

Given a Poisson process, one useful general case to consider is the probability of occurrence of one or more "events" during the next *t* years. (An event might be the occurrence of an earthquake or—for later reference—a happening in which a specified level of ground motion is exceeded at a site.) Simple probability theory leads to the formulation:

$$P[\text{1 or more events}] = 1 - P[\text{zero events}] = 1 - e^{-\lambda t}$$

When we ask the question, "What's the chance of an earthquake in the next *t* years?" the answer is, "It all depends." What size earthquake? Anywhere in the Wasatch Front area? In Utah? Anywhere on the Wasatch fault? On a specific segment of the Wasatch fault? Assuming a Poisson (random, memoryless) model? Assuming a time-dependent model?

During some period of time there clearly are differences, for example, between how often earthquakes can be expected to occur on a particular segment of the Wasatch fault, anywhere on the Wasatch fault, or anywhere in the Wasatch Front area, where there are many other active faults. When the behavior of many faults or fault segments is considered, the Poisson model of random behavior gives a good approximation of earthquake occurrence. However, for a specific fault segment that accumulates and releases strain energy, the time since the last big earthquake should influence the likely timing of future rupture. So we expect that a "time-dependent" model may be more realistic than a random model.

If we accept that large surface-rupturing earthquakes occur *on average* about once every 400 years somewhere on one of the Wasatch fault's central active segments (Machette and others, 1991, 1992), we can use the Poisson model to estimate the probability that one or more such earthquakes will occur during some period of time. During a 50-year period, the probability (conventionally specified between 0 and 1.0) is 0.12; in other words, there is "a 12 percent chance" of such a happening. For a 100-year period, the probability rises to 0.22. Nishenko and Schwartz (1990) made a preliminary attempt to estimate the probability of large surface-rupturing earthquakes on particular segments of the Wasatch faults using time-dependent models. Their results suggested that the 100-year probability of a large earthquake on any particular active segment of the Wasatch fault was less than 0.02, with the exception of the

Brigham City segment, which had a 100-year probability of 0.07. (At a meeting of the National Earthquake Prediction Evaluation Council in 1991, Nishenko presented estimated 100-year probabilities for the Brigham City segment of 0.07 to 0.20, depending on the assumptions made. These estimates are being revised by McCalpin and Nishenko (personal communication to W. Lund, Utah Geological Survey) using more up-to-date paleoseismologic data.

Finally, let's consider the probability of a sizable earthquake somewhere in the Wasatch Front region during the next 50 years, assuming the Poisson model. The instrumental seismicity data of Figure 15 indicate that the average return period for an earthquake of magnitude 6.5 or greater in the Wasatch Front area is 120 years. The probability of such an event during the next 50 years is 0.34. The geological data of Hecker, also shown in Figure 15, included her preferred estimate of 176 years for the average recurrence of surface-faulting earthquakes throughout the Wasatch Front region in the past 15,000 years. That rate would give a probability of 0.25 for such an earthquake somewhere in the Wasatch Front region during the next 50 years.

Part II. Expectable Strong Ground Motion

Introductory Remarks

I've deliberately used the modifier "expectable" to encompass both deterministic and probabilistic estimates of strong ground motion (explained below). Valuable information for the Wasatch Front region based on both approaches is now available. I'll attempt to convey this information within a broader context of the rapidly evolving and complex world of strong-motion seismology.

Strong motion implies ground motion of sufficient amplitude and duration to be potentially damaging to engineered structures. Large-amplitude seismic waves from large or nearby moderate-size earthquakes are the general cause. For an introductory overview of both seismological and engineering aspects of strong ground motion, see Reiter (1990). Valuable updates are contained in the proceedings of a recent seminar organized by the Applied Technology Council and funded by the U.S. Geological Survey (ATC/USGS, 1994).

The paucity in Utah of both strong-motion instrumentation and recorded strong-motion data is well known (e.g., Olig and Christenson, 1993). It might appear from Part I of this presentation that the University of Utah's regional seismic network records abundant earthquake data. Such networks, however, use velocity transducers designed to provide the necessary sensitivity for monitoring local and regional seismicity. **Figure 17** graphically compares the frequency response and dynamic range of regional-network stations with the frequency and amplitude range of strong motion. The recording of strong motion requires special low-gain instruments whose output is directly proportional to ground acceleration over a wide frequency range. The resulting recordings yield the ground-motion characteristics needed for earthquake engineering—peak ground acceleration, peak ground velocity, peak ground displacement, spectral content and shape, time history, and duration.

In order to place "expectable" strong ground motion in the Wasatch Front area in perspective, I'll proceed to outline (1) lessons learned from earthquakes elsewhere about strong motion, (2) causes of significant variability in strong ground motion, and (3) approaches to predicting strong ground motion in general. I'll then summarize available information about probabilistic and deterministic estimates of strong ground motion for the Wasatch area—pointing the reader to information on local site response presented by Kyle Rollins (this volume; see also Adan and Rollins, 1993).

Lessons Learned Elsewhere

Important lessons about strong motion learned from earthquakes worldwide have been succinctly summarized by a task group of the Earthquake Engineering Research (EERI, 1986) as follows:

- The characteristics of free-field ground motion are influenced by three major factors: source, travel path, and local conditions.
- Basic characteristics of strong motion are influenced by the source and travel path and modified by local conditions.
- For a given soil condition, the characteristics of strong ground motion (peak ground acceleration, peak ground velocity, peak ground displacement, duration, spectral content and time history) can vary significantly depending on the characteristics of the seismic source and the location of the site relative to the seismic source. **Depending on the situation, the variations in ground motion due to source effects can overshadow the effect of local soil conditions or the effects of local soil conditions can overshadow the source effects** [bold type mine].
- Seismic source directivity causes local amplification of ground motion (peak ground velocity, peak ground displacement, and spectral content) at sites toward which fracture propagation occurs.
- Near-source records contain a long-period pulse corresponding to the "fling" along the fault.
- The long-period pulse in the near field is usually unidirectional, as compared to the long-period motions associated with soft sites that may be related to several cycles of motion.
- Seismic strong motion is highly nonstationary.

Expanding on the above, one can add what's been learned from recent California earthquakes. For the Northridge earthquake (M_W 6.7, 17 January 1994), abundant strong motion data were collected throughout the Los Angeles area—including more data than ever before recorded within 25 km of a single earthquake source (USGS/SCEC, 1994). Peak horizontal ground accelerations (**Figure 18, left**) were greater than would be estimated from earlier data. Part of the explanation appears to be high seismic moment release over a relatively small fault area and a high stress drop of 150 bars (USGS/SCEC, 1994). The USGS/SCEC authors emphasize that, "The most important factor controlling the amount of strong shaking in one event is the distance of the site from the fault plane." Population centers along the Wasatch Front are vulnerable to near-source effects if a segment of the Wasatch fault or some other fault located immediately beneath one of these centers should rupture.

For the Loma Prieta earthquake (M_W 6.9, 18 October 1989), a major lesson was the occurrence of high peak accelerations at unusually large distances from the epicenter (**Figure 18, right**). Holzer (1994) emphasizes that, "Approximately 70% of the property losses caused by the earthquake occurred in localized areas 100 km away from the epicenter." The amplification of ground motions on soft soils and bay mud in the San Francisco Bay region (e.g., Borcherdt and Glassmoyer, 1992) was an important factor, but another important factor was the amplification of ground-motion amplitudes in the distance range of about 40 to 100 km due to critical reflections from the base of the crust (Somerville and Yoshimura, 1990). Comparing these distant effects with a map of the Wasatch Front, it becomes apparent that metropolitan areas like the Salt Lake Valley could experience damaging amplified ground motions from a large earthquake in distal parts of the Wasatch Front—say from a rupture of the Brigham City segment of the Wasatch fault.

Variability in Strong Ground Motion

It is well known that ground motions at the same distance from earthquakes of the same magnitude may involve large variability—due variously to *source effects* associated with the rupture process, *path effects* related to wave propagation between a source and site, and *site effects* due to soil conditions and topography (e.g., Heaton and Hartzell, 1994; Reiter, 1990). A complete review is beyond the scope of this presentation, but some notable factors—and uncertainties—relevant to potentially variable source and path effects in the Wasatch Front area should be mentioned. I leave discussion of site effects to Rollins (this volume). (See also Reiter, 1990, p. 147, for a tutorial explanation of "Why local site conditions affect ground motion.")

Seismic source effects include the nonuniformity, directivity, and geometry of the rupture process as well as the resulting radiation pattern of seismic energy and the stress parameter controlling dynamic ground-motion amplitudes. A directivity (Doppler-like) effect can cause higher-amplitude, shorter-duration ground motion at a site toward which rupture propagates, due to focusing of seismic energy. In the near-source region, directivity can produce marked effects at periods of about one second and longer due to a large velocity pulse near the beginning of strong motion (see Heaton and Hartzell, 1994). In design practice, peak vertical acceleration and vertical response spectra are commonly taken to be 2/3 of the corresponding horizontal values, but in the near-source region vertical motions are commonly observed to be comparable to horizontal motions. Singh (1985) describes engineering implications of near-source variations in ground motion.

Strong motion data associated with normal-faulting earthquakes are far fewer than for reverse and strike-slip faulting. Available observations suggest that normal-faulting earthquakes generate ground motions comparable to other types of faulting (e.g., Westaway and Smith, 1989). It is uncertain whether large Wasatch-fault earthquakes have high average stress drops, as one would infer from observations of Kanamori and Allen (1986) for major faults with long recurrence intervals. The stress drop of an earthquake controls the ratio of high- and low-frequency ground shaking (e.g., Heaton and Hartzell, 1994). The stress drop for the 1983 Borah Peak, Idaho, earthquake was of the order of 12 to 25 bars (see Wong and Silva, 1993), and an average stress drop of 36 bars has been reported by Stark and others (1992) for 28 normal-faulting earthquakes in the Basin and Range province.

Perhaps the most important path effects for earthquakes in the Wasatch Front region are what can be called "basin effects" **(Figure 19)**—(1) the focusing and resonance of upward propagating body waves above the interface between bedrock and basin fill and (2) the conversion at the edges of a basin of S-waves to surface waves which travel horizontally across the basin (e.g., Frankel, 1994; Spudich, 1994). A key point is that the basin effects can amplify and increase the duration of ground motion in ways not predicted by models using vertically propagating shear waves.

Surface waves produced by basin effects typically have periods of 0.5 to 5 sec or longer and can dominate ground motions at these periods for sites within the basin (Frankel, 1994). For ground motions at shorter periods, near-surface soil effects are generally thought to be more important, but not with certainty. To date, computer codes and processing capabilities have limited the analysis of basin-modified body waves only up to frequencies of a little more than 1 Hz. Wong and Silva (1993) summarize earlier studies on site amplification in the Salt Lake Valley, including analyses by various University of Utah researchers of two- and three-dimensional basin effects using finite-element or finite-difference methods. For the most recent of this work, see Olsen (1994) and Olsen and others (1993, 1994).

Towards Predicting Strong Ground Motion in General

Theoretical to empirical approaches.—The characteristics of strong ground motion that must be anticipated for engineering applications include both peak values in the time domain and spectral characteristics in the frequency domain. In practice, these various parameters are both "estimated" and "predicted" using combinations of empirical and theoretical methods (e.g., Boore and Joyner, 1994; Reiter, 1990). To convey what is actually done in practice, the following outline is particularly descriptive (Risk Engineering, Inc., 1994): "There are a range of possible methods of estimating strong ground motion at a site, from purely theoretical to purely empirical. These include the following:

(1) Purely deterministic methods start with a representation of fault rupture, propagate seismic waves through the earth's crust, and estimate theoretical time histories of motion at the site of interest.

(2) Analytical methods use a spectral representation of the seismic source and, through random vibration methods, estimate ground motion in the frequency domain, from which distributions of time-series maxima can be obtained. This method has proven to be accurate in California where abundant strong motion data are available for comparison.

(3) Modified-empirical methods start with models obtained from strong motion data in other regions (e.g., California) and modify the attenuation equations based on estimated differences in regional attenuation.

(4) Purely empirical methods use data from the region of study to derive equations estimating ground motion as a function of magnitude and distance."

Seismic hazard analyses (deterministic and probabilistic).—Despite its seemingly general meaning, the term seismic hazard analysis is generally used to imply the quantifying of the hazard of earthquake ground shaking at a site. The analysis can be either deterministic or probabilistic. A deterministic analysis involves a "scenario" earthquake. The location and magnitude of an earthquake is specified—perhaps the closest largest earthquake expectable—and then the analysis answers the question, "What's going to happen?" A probabilistic analysis asks the additional question, "How likely?" and it considers the aggregate effect of many earthquakes over a specified time period. Importantly, a probabilistic seismic hazard analysis (PSHA) incorporates uncertainty in input assumptions (due both to lack of knowledge and diversity of opinion), and it allows uncertainty in the final hazard results to be quantified at a confidence level.

The nitty-gritty details of a PSHA (e.g., Youngs and others, 1987) can be confusing. For simplicity, I'll describe the basic idea (**Figure 20**). There are well-established methods for quantifying the hazard of earthquake ground motion. The most important elements address the following questions about future earthquakes: (1) Where? How far away? (Involves the depiction of seismic source zones, either as discrete faults or as areas within which earthquake epicenters are expected to lie.) (2) How big? How often? (Involves describing the size distribution and rate of occurrence of earthquakes within each source zone.) (3) How severe the effects? (Given an earthquake of a particular size and at a particular location, what will be the characteristics of ground motion at some site of interest?)

A key point of Figure 20 is that we want to calculate the mean number of times—annualized—in which a certain level of ground shaking at a site will expectedly be exceeded, and the graphical result is called a seismic hazard curve (**Figure 21**). As one wag puts it, we want to calculate how often "bad" happens. The reciprocal of an annual frequency is called a return period, so a ground motion that occurs .002 times per year (annualized) simply occurs once every 500 years. Once we have a seismic hazard curve, we can use the Poisson model, described at the end of Part I, to answer the question, "What level of ground motion has a 10 percent probability of being exceeded in, say, 50 years?" This is equivalent to asking, "What level of ground motion has a 90 percent probability of not being exceeded during 50 years?" The answers for exposure periods of both 50 years and 250 years are shown in Figure 21. The example of Figure 21 happens to be the mean hazard curve for a soil site in Salt Lake Valley calculated by Youngs and others (1987), which brings us finally to expectable strong ground motion in the Wasatch Front area.

Towards Predicting Strong Ground Motion in Utah

Probabilistic seismic hazard analysis.—A rigorous probabilistic seismic hazard analysis for the Wasatch Front area was made by Youngs and others (1987; to be published in 1994 as part of USGS Prof. Paper 1500). The results will be familiar to many readers. The "bottom line" of the analysis in terms of peak ground acceleration on soil sites for exposure periods of 10, 50, and 250 years is shown in **Figure 22** (compare with Figure 21). According to Youngs and others (1987), peak accelerations for rock sites can be obtained by multiplying the soil site ten-percent-exceedance accelerations by a factor of 1.1 for the 10-year exposure period and 1.2 for the 50-year and 250-year periods. Abundant other details, including uncertainties, sensitivity analyses, and response spectra are given by Youngs and others (1987).

An exposure period of 50 years generally forms the basis for seismic building codes, intended to ensure minimum standards for earthquake resistant design and to prevent life-threatening collapse of buildings. Small to moderate earthquakes are the largest contributor to the probabilistic ground-shaking hazard for exposure periods of 50 years or less (Arabasz and others, 1992; Youngs and others, 1987). Ground shaking for a 250-year exposure period more closely approaches that expectable from a large surface-rupturing earthquake on the Wasatch fault.

Estimated ground motions from an M 7.0 earthquake on the Wasatch fault.—Potential strong ground motions in the Salt Lake Valley have been analytically estimated by Wong and Silva (1993) for scenario earthquakes of M_W 7.0 on the Salt Lake City segment of the Wasatch fault. The elements of their procedure (**Figure 23**) involve finite earthquake source modeling, use of a stochastic ground-motion model together with random vibration theory, and estimation of ground motion, on a site-specific basis, using an equivalent-linear soil-response approach. Peak horizontal accelerations for the scenario earthquakes have median values ranging from 0.6 to 1.1*g* at three sites chosen to be representative of near-surface conditions in the Salt Lake Valley. Response spectra varied significantly among these three sites, due in part to the damping of high frequencies at the deep soft-soil sites.

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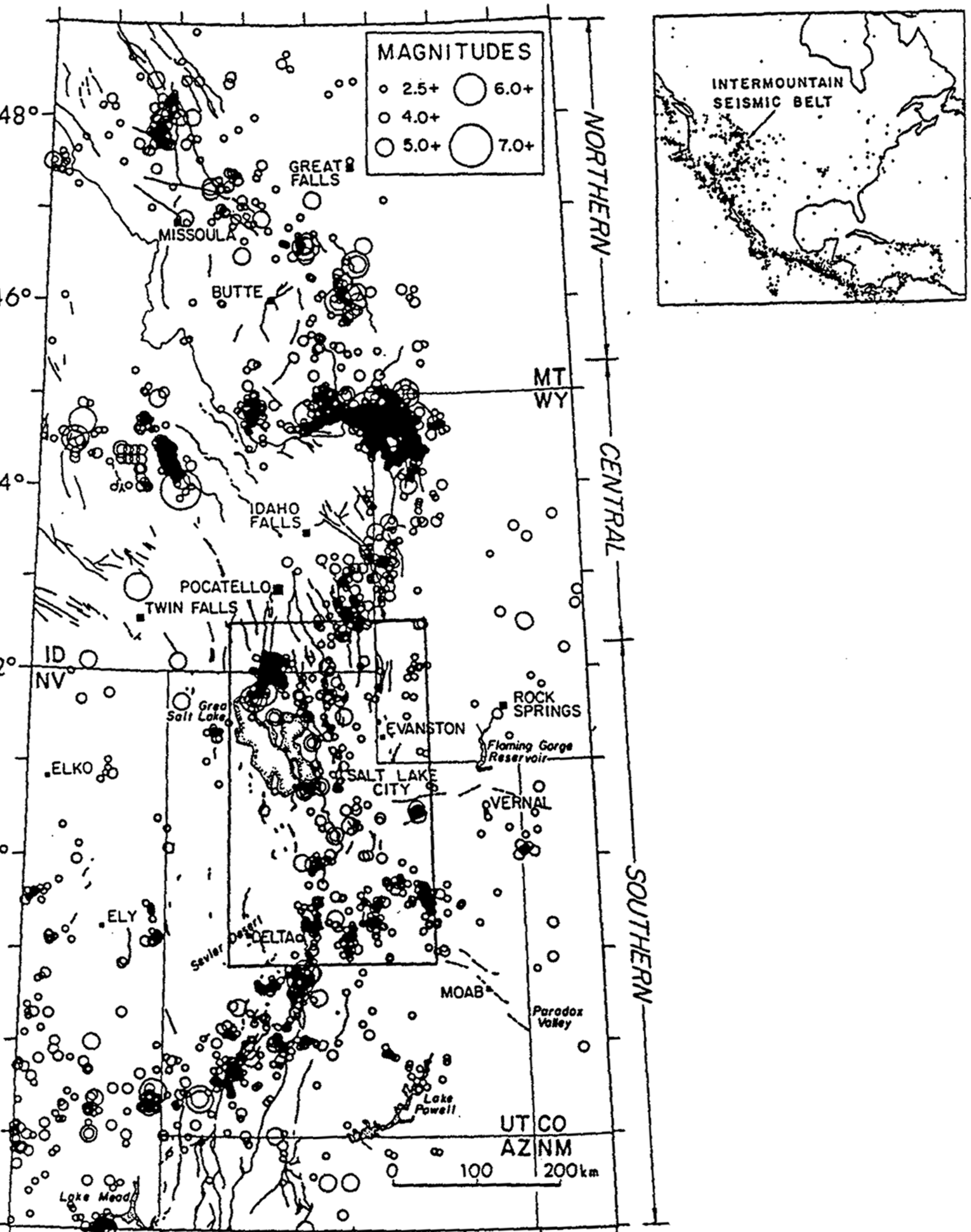


Figure 1. Map showing the setting of the Wasatch Front area of northern Utah (inset rectangle) with respect to the Intermountain Seismic Belt, 1900-1985 (from Smith and Arabasz, 1991).

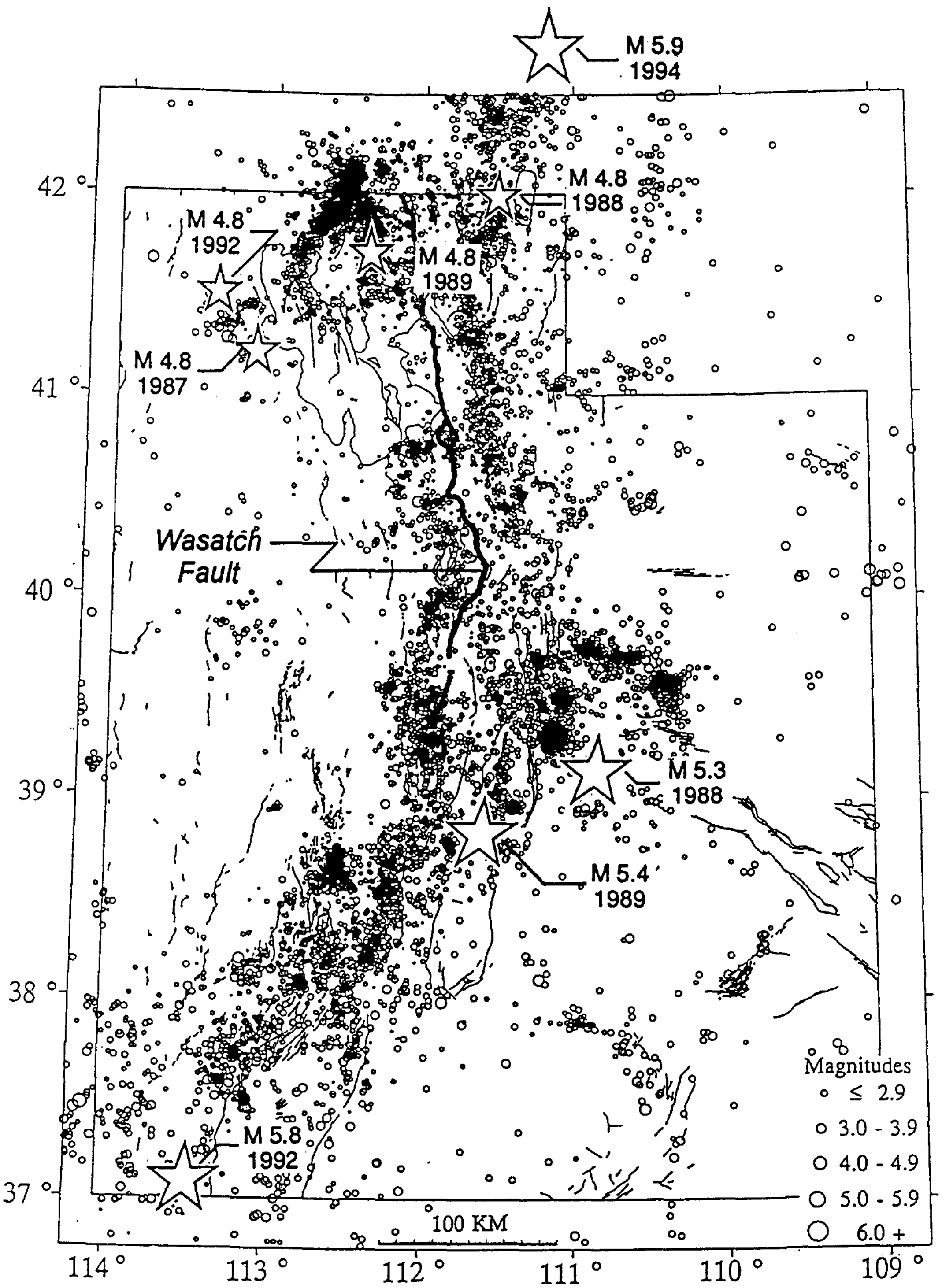


Figure 2. Map of Utah region showing the epicenters of more than 16,000 earthquakes located by the University of Utah Seismograph Stations from 1962 through 1993. Earthquakes of magnitude 4.8 and larger since 1987 shown as stars. Base map showing Quaternary faulting is from Hecker (1993).

University of Utah Regional Seismograph Network

October 1994

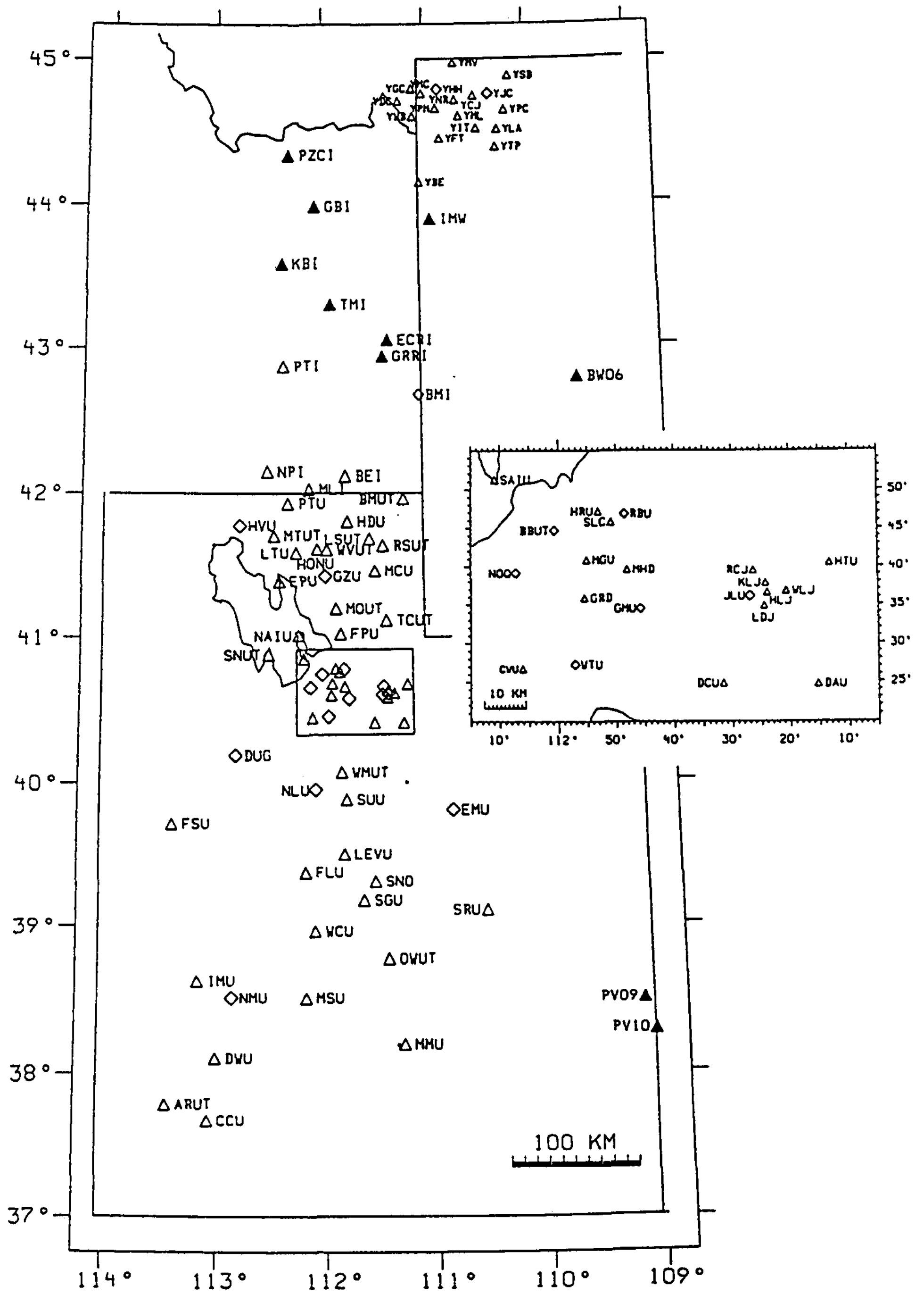
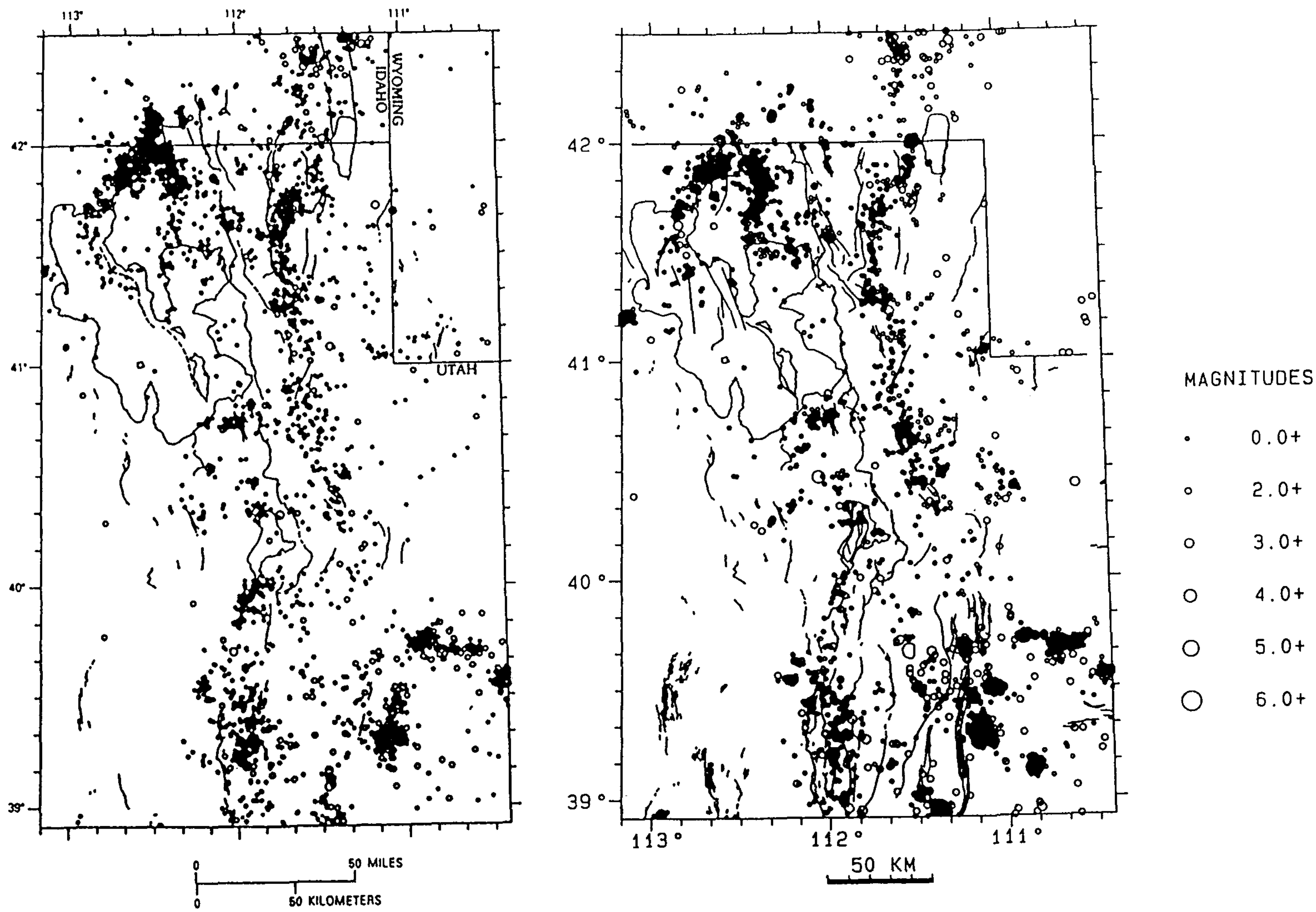


Figure 3. Map of remote seismograph stations making up the University of Utah's regional seismic network. Seismic data from each station are transmitted continuously by radio, telephone, and/or microwave and are centrally recorded on the University campus in Salt Lake City. The open triangles indicate stations maintained and operated by the University of Utah; the filled triangles, stations owned and operated by other agencies.



JUL 1978 to DEC 1986

JAN 1987 to DEC 1993

Figure 4. Seismicity of the Wasatch Front area. Comparison of 8.5-year period (left) from Arabasz and others (1992) with subsequent 7-year period. Base map of Quaternary faults on right is from Hecker (1993).

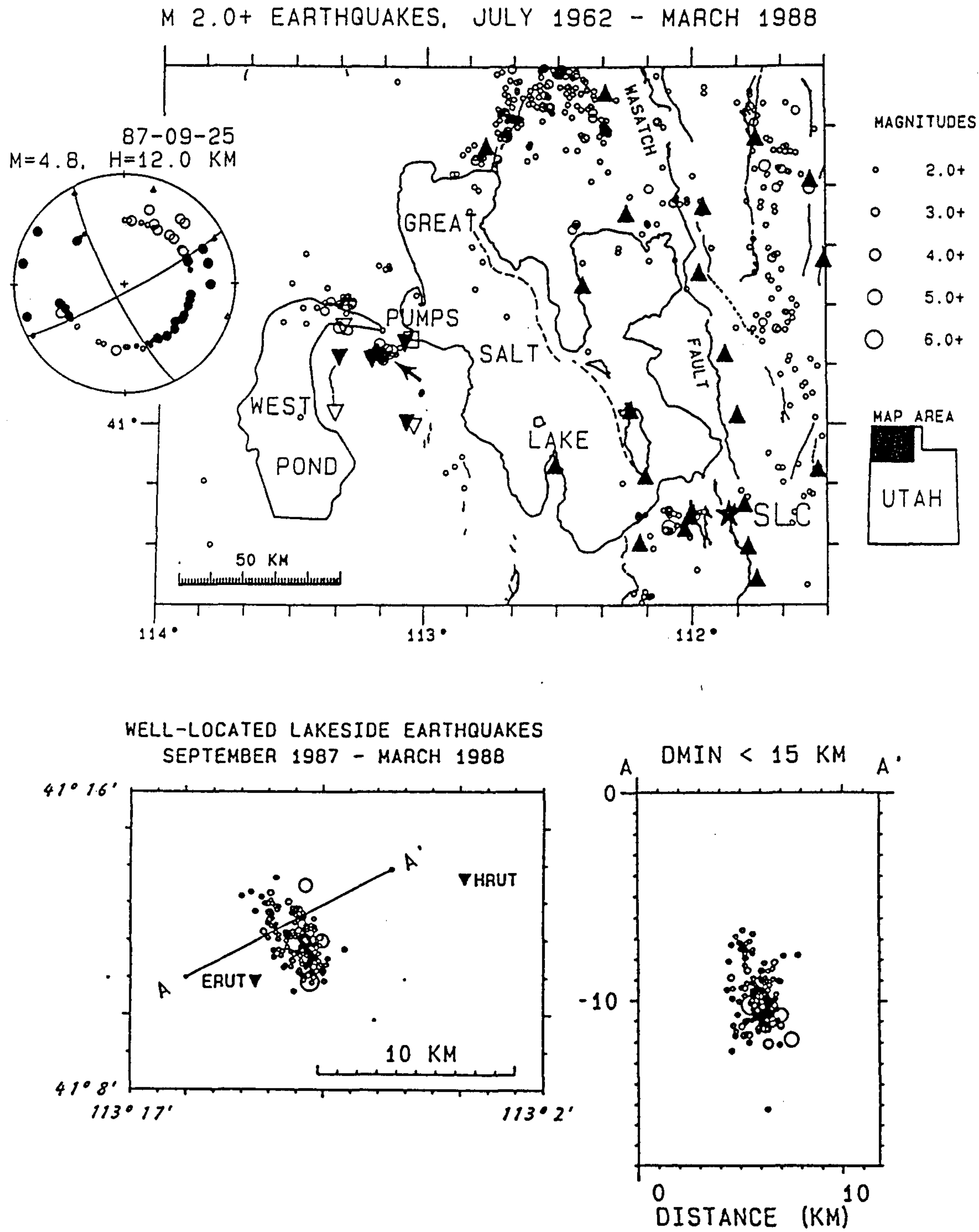


Figure 5. Details of the 1987-1988 Lakeside, Utah, earthquake sequence (from Pechmann and others, 1993). Map (above) shows the location (arrow) of the M_L 4.8 mainshock on 25 September 1987 close to the Great Salt Lake pumps. The mainshock focal mechanism (lower-hemisphere) is inset. Well-located aftershocks are shown in map view (lower left) and cross-section view (lower right). Right-lateral slip is inferred at a depth of 6-12 km on a steeply-dipping fault striking N27°W.

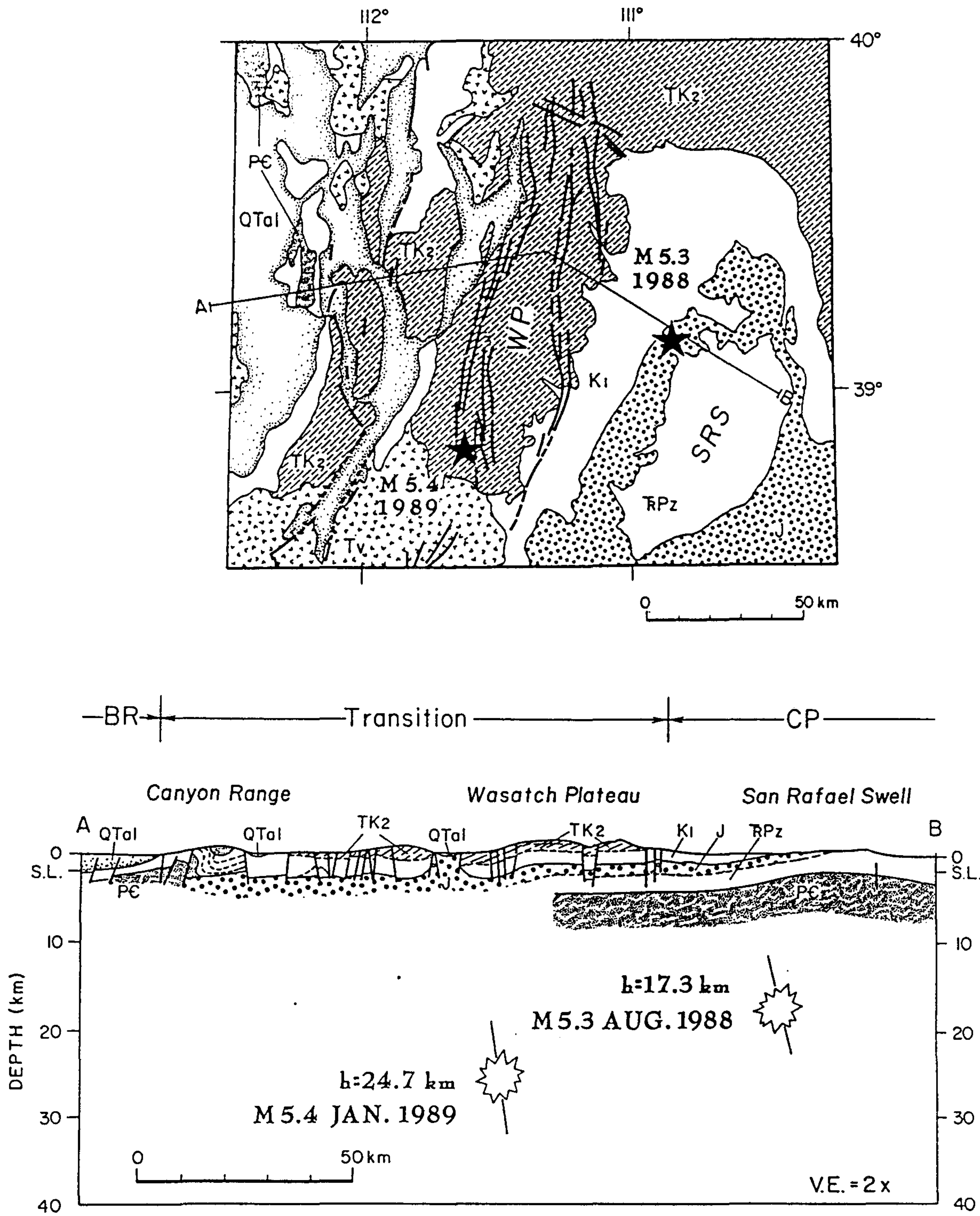


Figure 6. Map (above) and cross section (below) showing the setting of the 1988 M_L 5.3 San Rafael Swell earthquake and the 1989 M_L 5.4 Southern Wasatch Plateau earthquake in central Utah (from Pechmann and others, 1991). Details for these two earthquake sequences are shown in Figure 7.

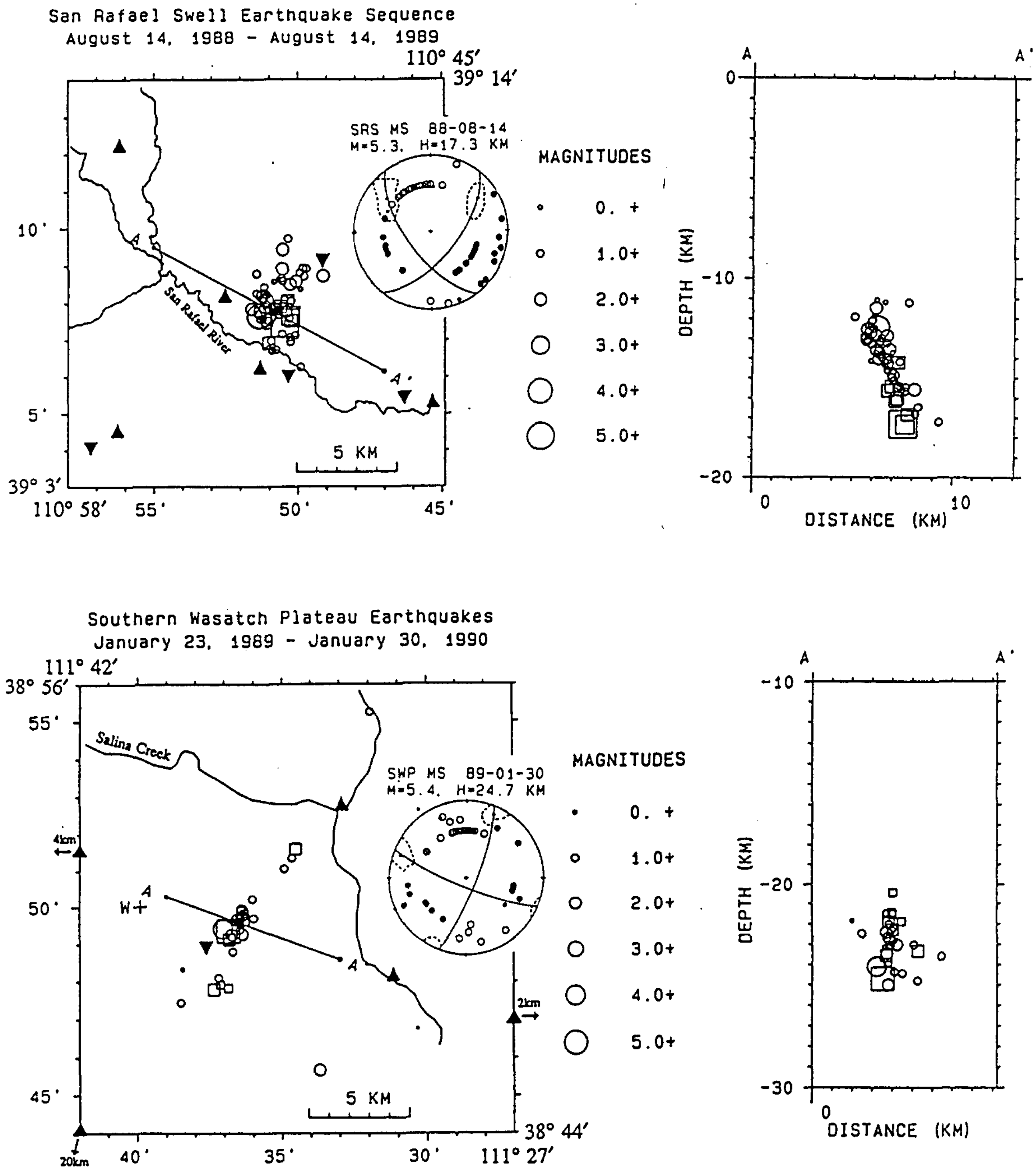
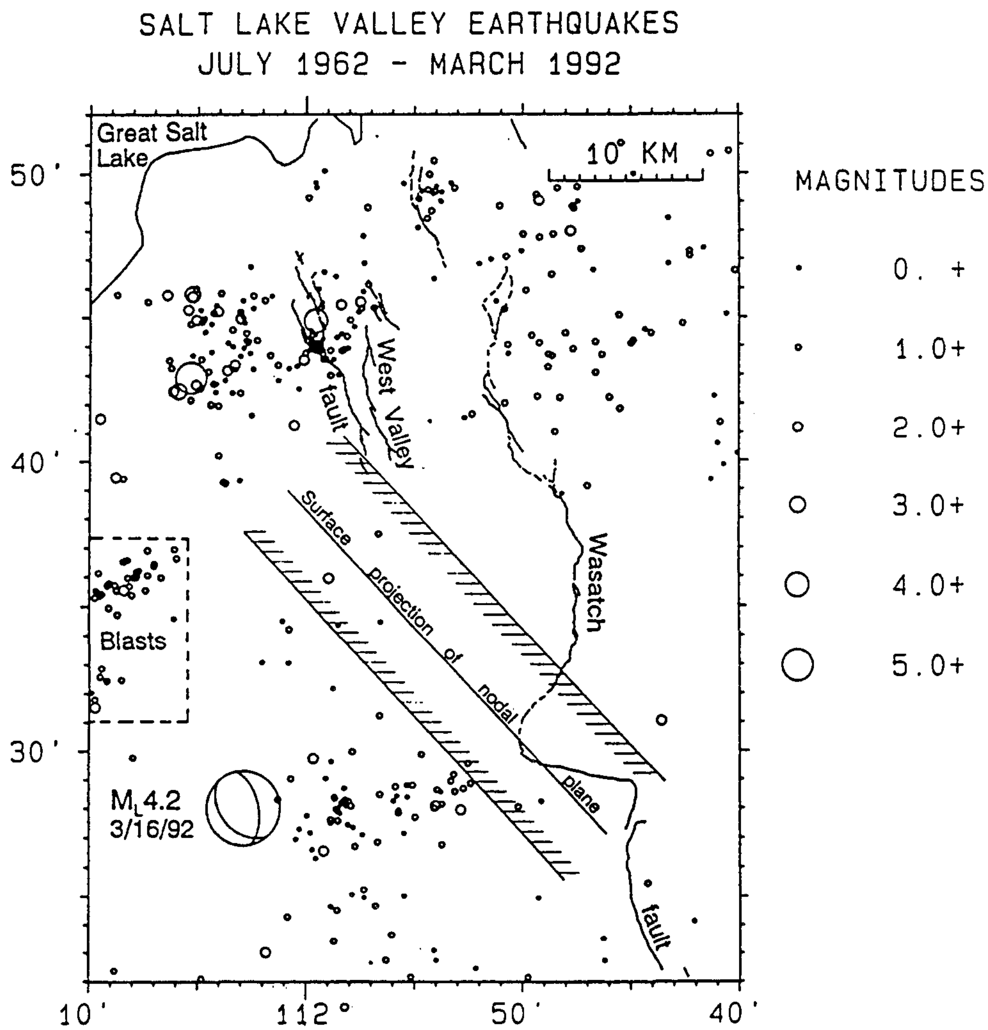


Figure 7. Details (from Pechmann and others, 1991) of the 1988 M_L 5.3 San Rafael Swell (SRS) earthquake sequence (above) and the 1989 M_L 5.4 Southern Wasatch Plateau (SWP) earthquakes (below) (see location map in Figure 6). For each, an epicenter map with an inset mainshock focal mechanism (lower hemisphere) is shown together with a cross section of well-located aftershocks. For the SRS sequence, oblique (left-lateral, normal) slip is inferred at 11 to 18 km depth on a buried fault striking NE and dipping 60° SE. For the SWP sequence, predominantly left-lateral slip is inferred at 21 to 25 km depth on a buried near-vertical fault striking NNE.



Seismicity map of the Salt Lake Valley showing epicenters of earthquakes located by the University of Utah from July 1962, when the University's regional seismic network began operating, through March 1992. The epicenter of the Western Traverse Mountains earthquake is indicated by the miniature version of the focal mechanism diagram from figure 2. The solid straight line marks the surface projection of the SW-dipping nodal plane of this focal mechanism, and the hachured lines parallel to it show the error bars on this projection (see text). The dashed box outlines epicenters of probable mining blasts which have not yet been removed from the earthquake catalog. The other solid and dashed lines are the surface traces of Holocene faults, taken from maps by Cluff and others (1970), Davis (1983a, b), Keaton and others (1987).

Figure 8. Map showing the location of the M_L 4.2 Western Traverse Mountains earthquake of 16 March 1992 in the SW part of the Salt Lake Valley together with background seismicity (from Pechmann, 1992). The mainshock occurred at a depth of 12.5 km and was followed by only one locatable aftershock. The 1992 mainshock focal mechanism indicates predominantly normal slip, and one of the two possible nodal planes can arguably be associated with a downdip projection of the west-dipping Wasatch fault.

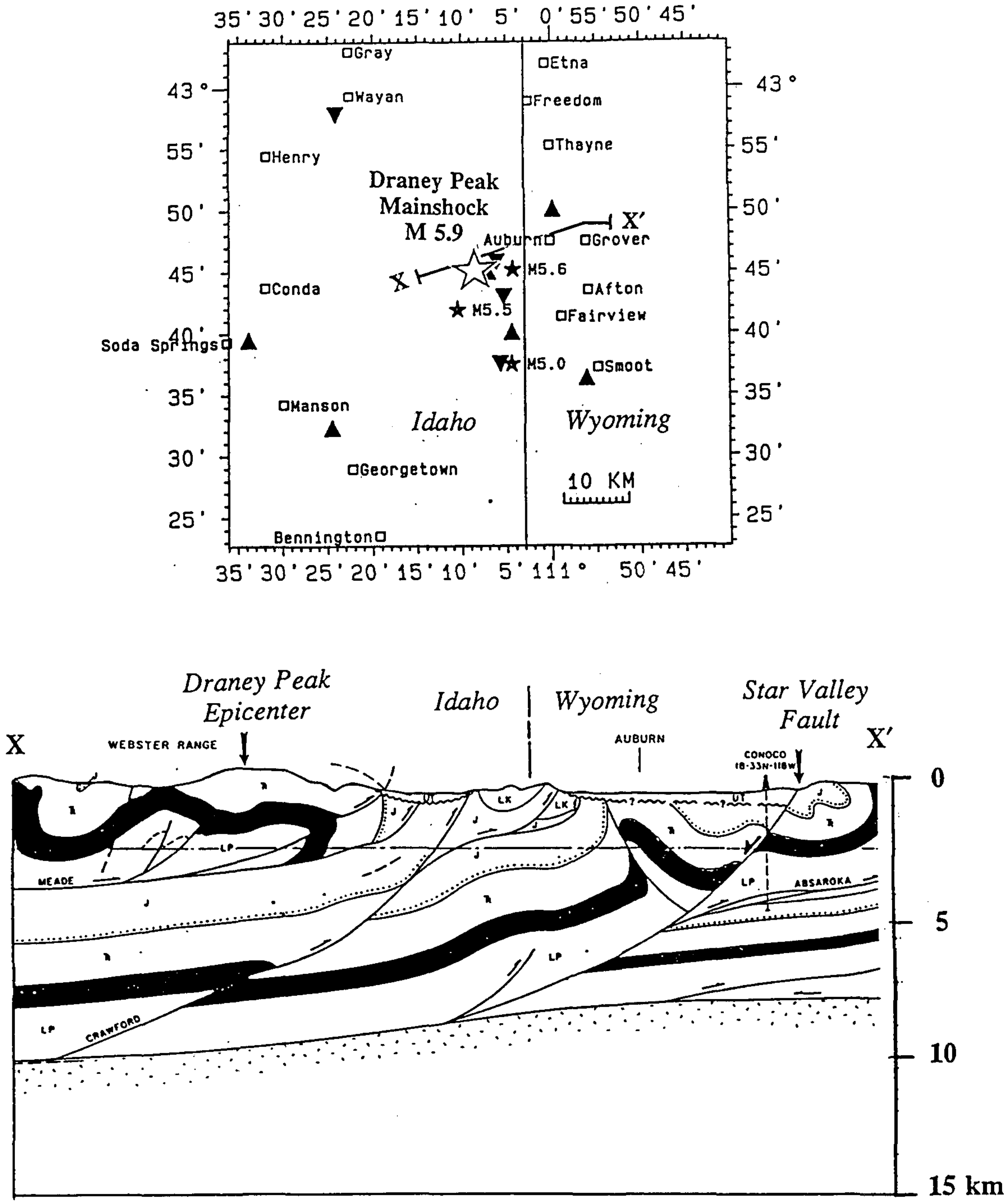


Figure 9. Setting of the M_W 5.9 Draney Peak, Idaho, earthquake of 3 February 1994 (after Nava and others, 1994). (Above) Map showing the mainshock epicenter (large star), the epicenters of the largest aftershocks (smaller stars), and the locations of temporary seismograph stations (triangles) installed by the University of Utah to study the aftershock sequence. (Below) Geologic cross section (after Royce, 1993), along the line of section shown in the map above, showing the position of the Draney Peak epicenter and the active Star Valley normal fault with respect to older fold-and-thrustbelt structure. The focal depth of the Draney Peak mainshock is poorly constrained within the 5 to 15 km depth range (see Figure 10 for more details).

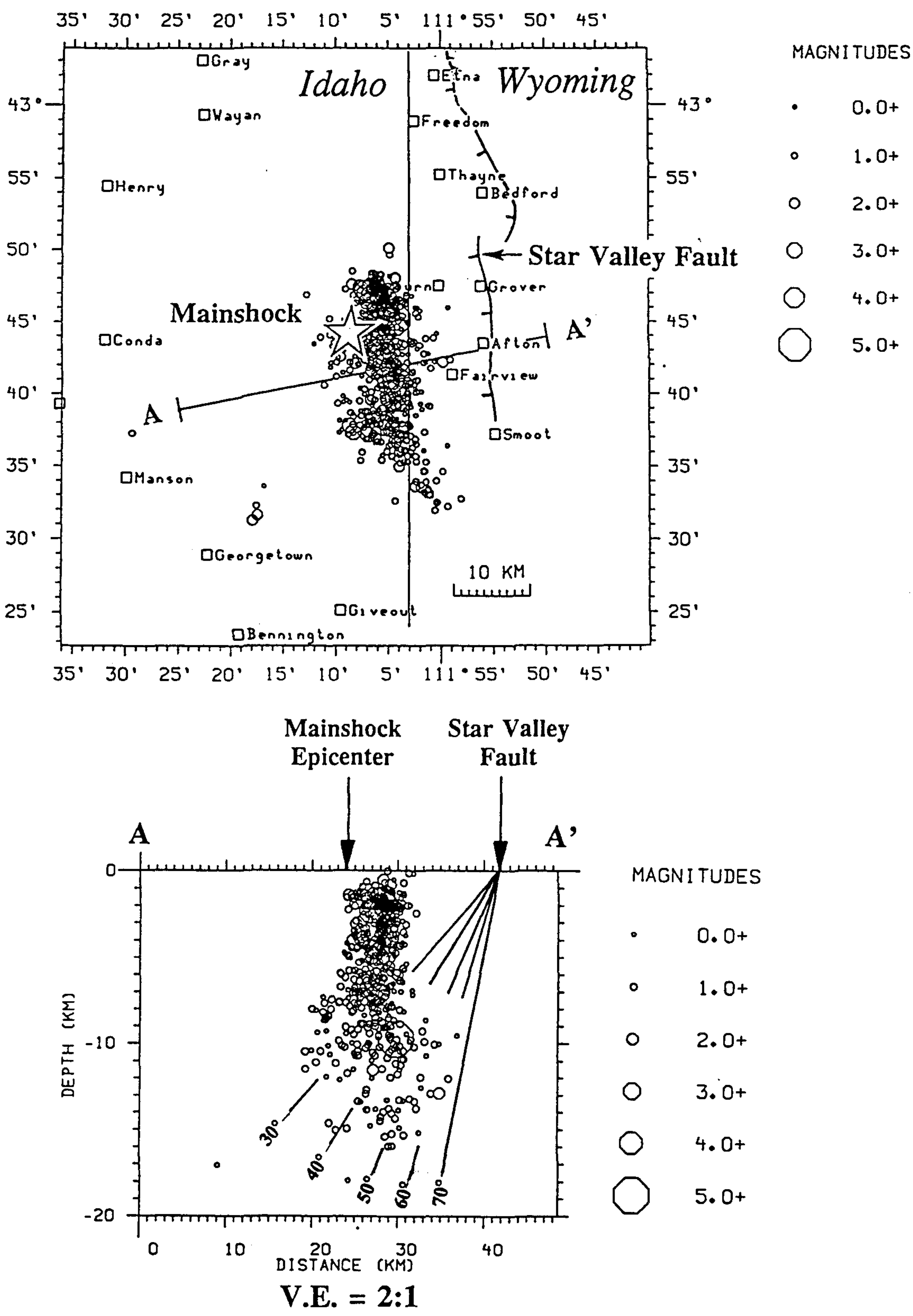


Figure 10.. Epicenter map (above) and cross section (below) showing the preliminary location of 934 better located aftershocks of the 1994 M_W 5.9 Draney Peak, Idaho, earthquake (after data from Nava and others, 1994). As noted in Figure 9, the mainshock focal depth is poorly constrained within the 5 to 15 km depth range. A range of down-dip projections of the Star Valley fault is shown for comparison.

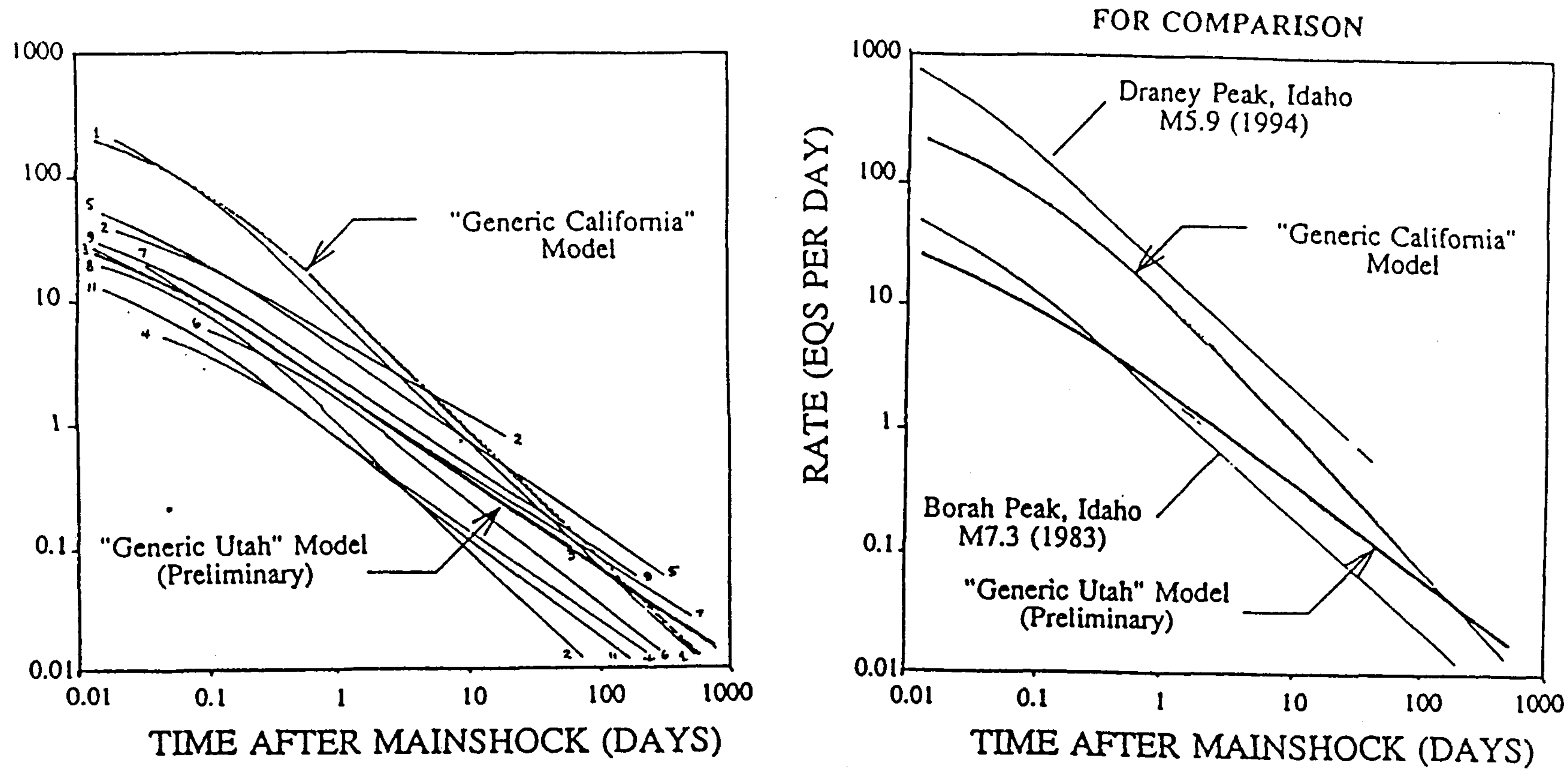


Figure 11. Rate decay and relative productivity of aftershock sequences in the Utah region, normalized to a cutoff magnitude 3.0 units below the mainshock (from Arabasz and Hill, 1994). Left side shows the model fits for a modified Omori rate-decay function for aftershock sequences associated with mainshocks of $4.5 \leq M_L \leq 6.0$ in the Utah region, 1975–1992. A median curve labeled "Generic Utah" model is compared to a similarly derived "Generic California" model of Reasenberg and Jones (1989). On average, the Utah aftershock sequences decay more slowly than the California model and are less "productive" by a factor of 4 to 5 in terms of aftershock rates after 1 day and the cumulative number of aftershocks during the first 30 days. The right side shows model fits for the 1983 Borah Peak and 1994 Draney Peak earthquake sequences compared to the California and Utah models.

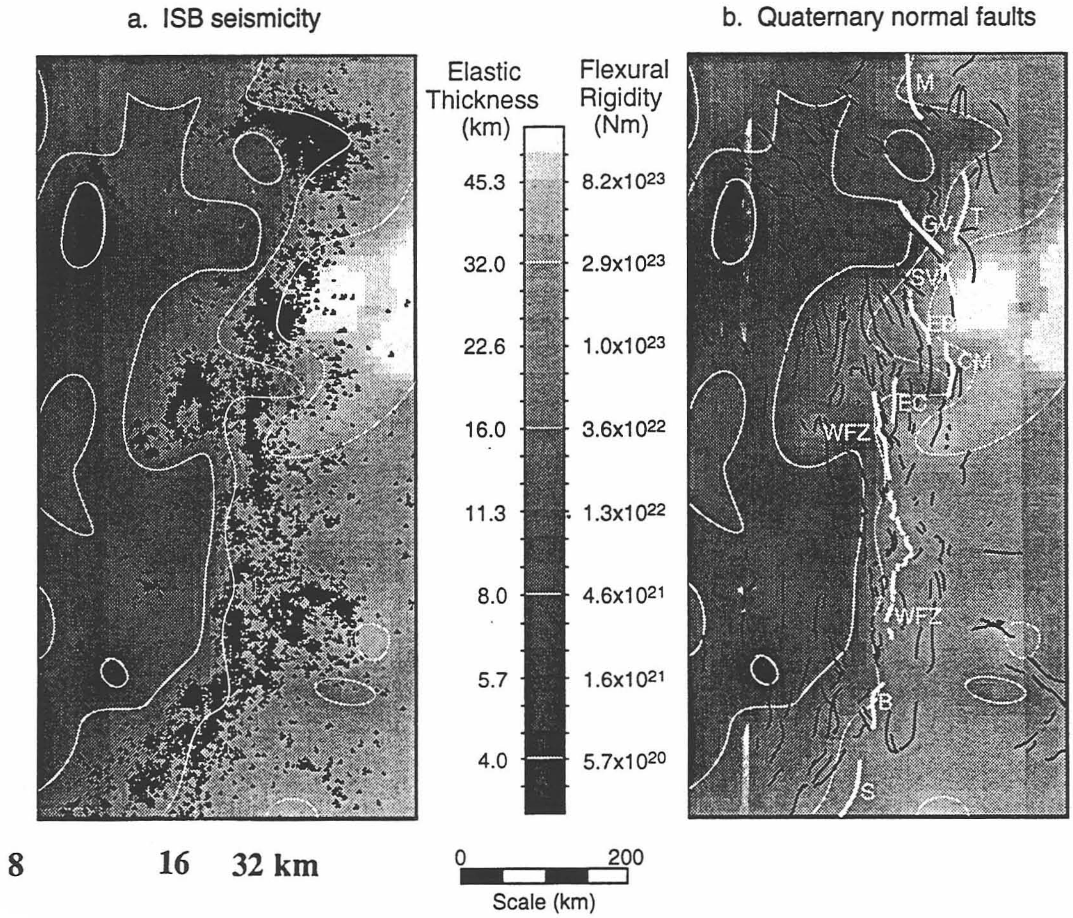


Figure 12.. Comparison of elastic thickness (a representation of flexural rigidity) of the lithosphere in the Intermountain region with seismicity (left) and the distribution of Cenozoic normal faults showing evidence of late Quaternary surface rupture (right), from Lowry (1994). (To aid interpretation of the gray scale, some of the elastic-thickness contours are labeled beneath the left-side panel.) The seismicity includes all located earthquakes of $M \geq 0$, 1962–1992, in the Intermountain Seismic Belt between southernmost Utah and Yellowstone Park (compare with Figure 1). Major faults with more than 1 km of offset are indicated in bold white in the right-side panel. The Wasatch fault zone is labeled WFZ. Most of the seismicity and major active faulting is observed to occur along a regional gradient from low to high elastic thickness. Lowry (1994) hypothesizes that the stresses responsible for the concentrated tectonic activity may originate locally due to lateral variations in crustal buoyancy.

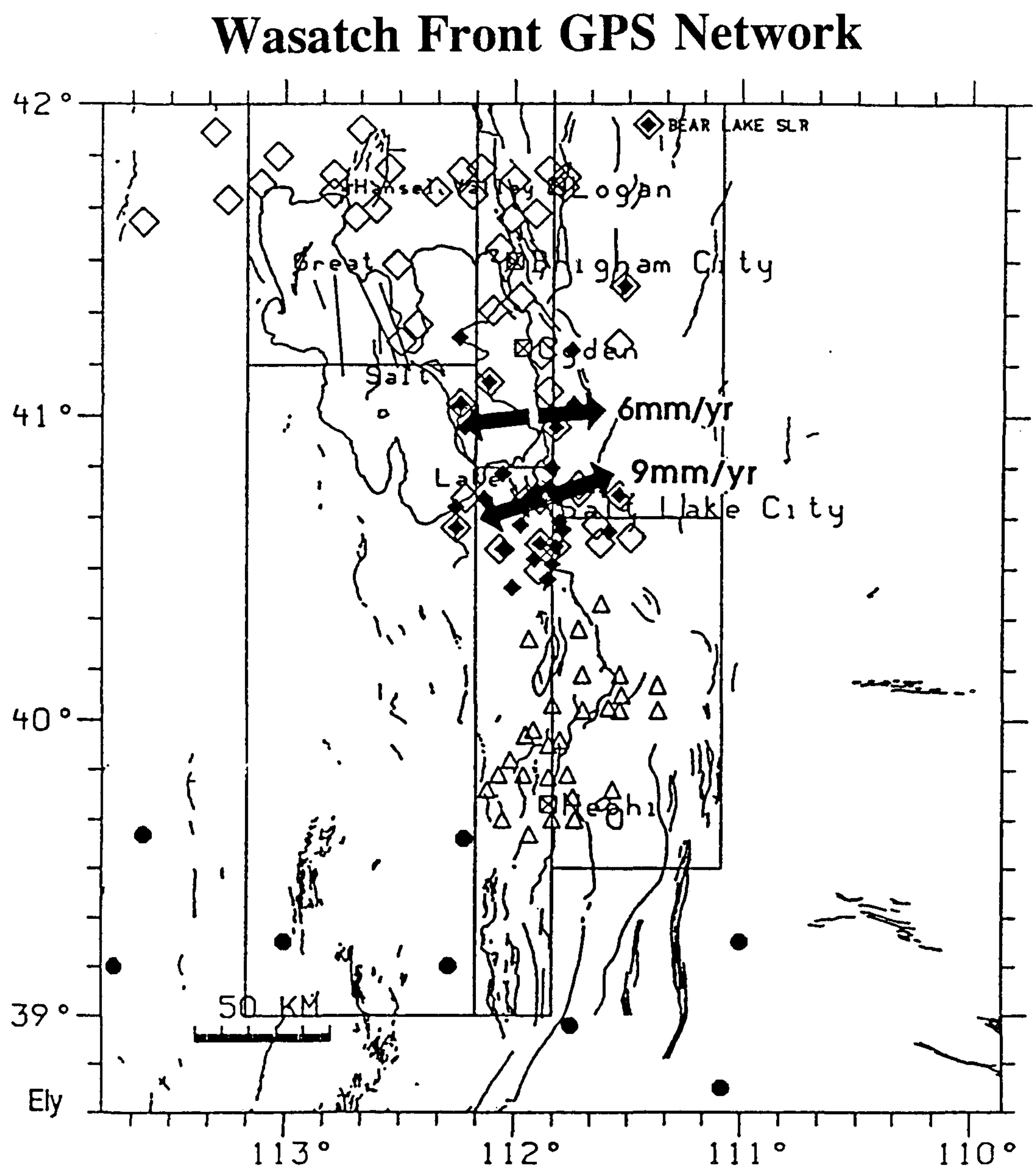


Figure 13. Map of the Wasatch Front region showing the locations of Global Positioning System (GPS) stations established by R.B. Smith, C.M. Meertens, and co-workers at the University of Utah as part of ongoing studies of crustal deformation and earthquake potential. Explanation: 1992 GPS survey = filled diamonds; 1993 GPS survey = open diamonds; 1994 GPS survey = open triangles; U.S. Geological Survey GPS Basin-Range profile = filled circles. Large arrows indicate measured deformation rates, across the entire 70-km-width of the Wasatch Front, of (1) 6 mm/yr determined from comparing 1992 GPS observations with older geodetic data and (2) 9 mm/yr determined from intercomparisons of GPS measurements made in 1992 and 1993 at the stations indicated by a filled diamond within an open diamond. These results should be considered preliminary.

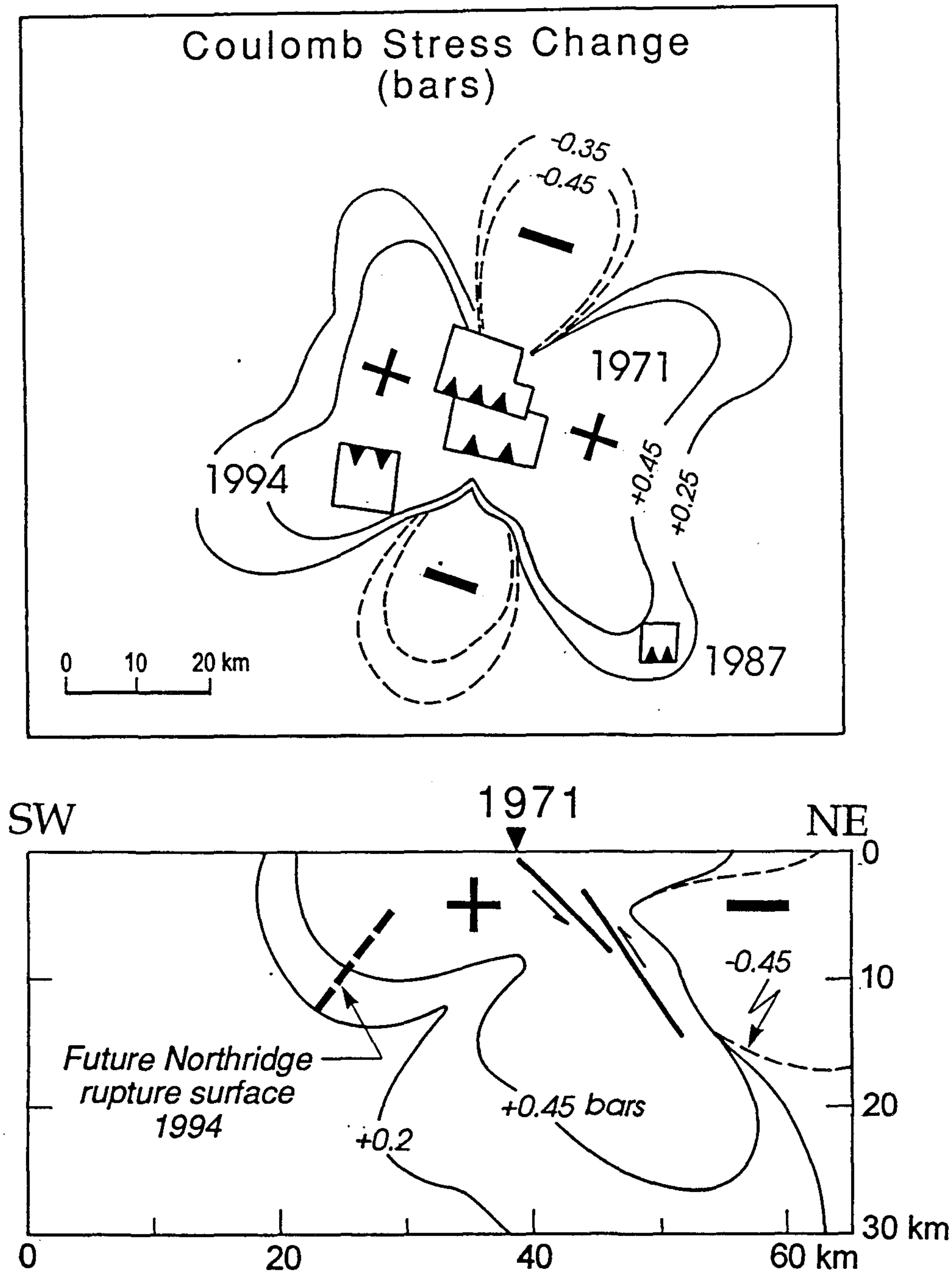
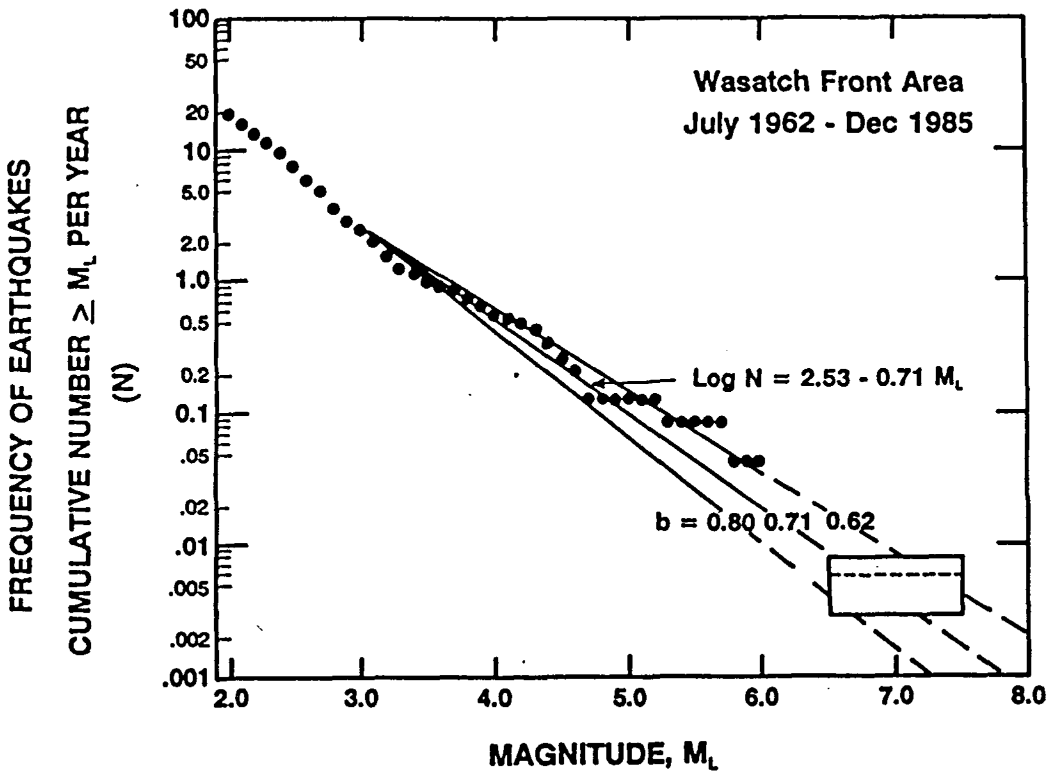


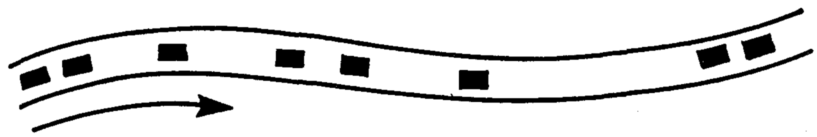
Figure 14. Illustration of calculated stress changes on optimally oriented faults caused by the 1971 M_W 6.7 San Fernando, California, earthquake—potentially advancing the occurrence of the 1987 M_W 6.0 Whittier Narrows earthquake and the 1994 M_W 6.7 Northridge earthquake (simplified from Stein and others, 1994). (Above) Generalized map of Coulomb stress change caused by the 1971 compound rupture (sawteeth on upper plates). Pluses and minuses indicate areas of stress increase and decrease, respectively, at a depth of 3 to 10 km. Stresses at the site of the 1994 Northridge earthquake were raised up to 2 bars; at the site of the 1987 Whittier Narrows earthquake, up to 0.5 bars. (Below) Generalized cross section of Coulomb stress change on optimally oriented faults, as above, along a section connecting the 1971 and 1994 earthquakes.



M_L	Average Return Period (years) Preferred Estimate	Range of Estimates
≥3.0	0.40	
≥3.5	0.90	0.81 - 1.0
≥4.0	2.0	1.7 - 2.5
≥4.5	4.6	3.4 - 6.3
≥5.0	10	7 - 16
≥5.5	24	14 - 40
≥6.0	54	29 - 100
≥6.5	120	60 - 250
≥7.0	280	120 - 630
3.0 - 3.5	0.71	.66 - .78
3.5 - 4.0	1.6	1.6 - 1.7
4.0 - 4.5	3.7	3.3 - 4.2
4.5 - 5.0	8.3	6.6 - 10
5.0 - 5.5	19	14 - 26
5.5 - 6.0	42	28 - 66
6.0 - 6.5	96	57 - 170
6.5 - 7.0	220	120 - 420
7.0 - 7.5	490	240 - 1000

Figure 15. Frequency of occurrence of mainshocks in the Wasatch Front area, in graphical form (left) and table form (right), based on instrumentally recorded seismicity, 1962-1985 (after Arabasz and others, 1992). For comparison, the small box in the graph is from Hecker (1993) and shows the estimated combined rate with which large surface-faulting earthquakes (magnitude 6.5–7.5) have occurred on active faults throughout the Wasatch Front area in the past 15,000 years.

EXAMPLES OF A RANDOM ARRIVAL PROCESS



CAR TRAFFIC ON A ONE-WAY ROAD

UTAH EARTHQUAKES
OF MAGNITUDE 5 OR GREATER

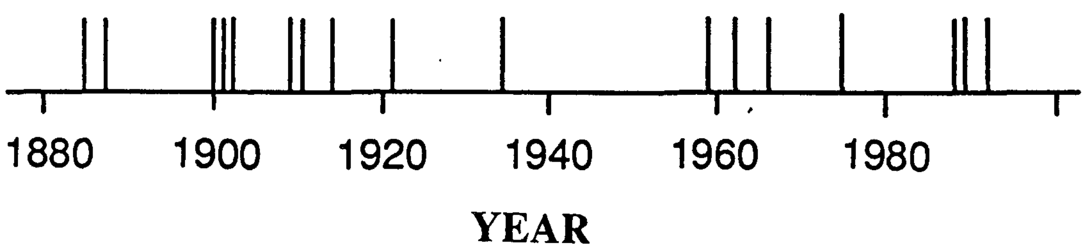


Figure 16. Examples of variable interevent spacing (clusters and gaps) due to a random arrival (Poisson) process. Such a process involves an *average* long-term rate of occurrence—say the arrival of an average number of cars per hour at some point, or the occurrence of an average number of earthquakes per decade. Because of randomness, the "events" are not uniformly spaced. By knowing the average long-term rate of occurrence, one can calculate the probability that zero, one, or some number of events will occur in a specified time interval.

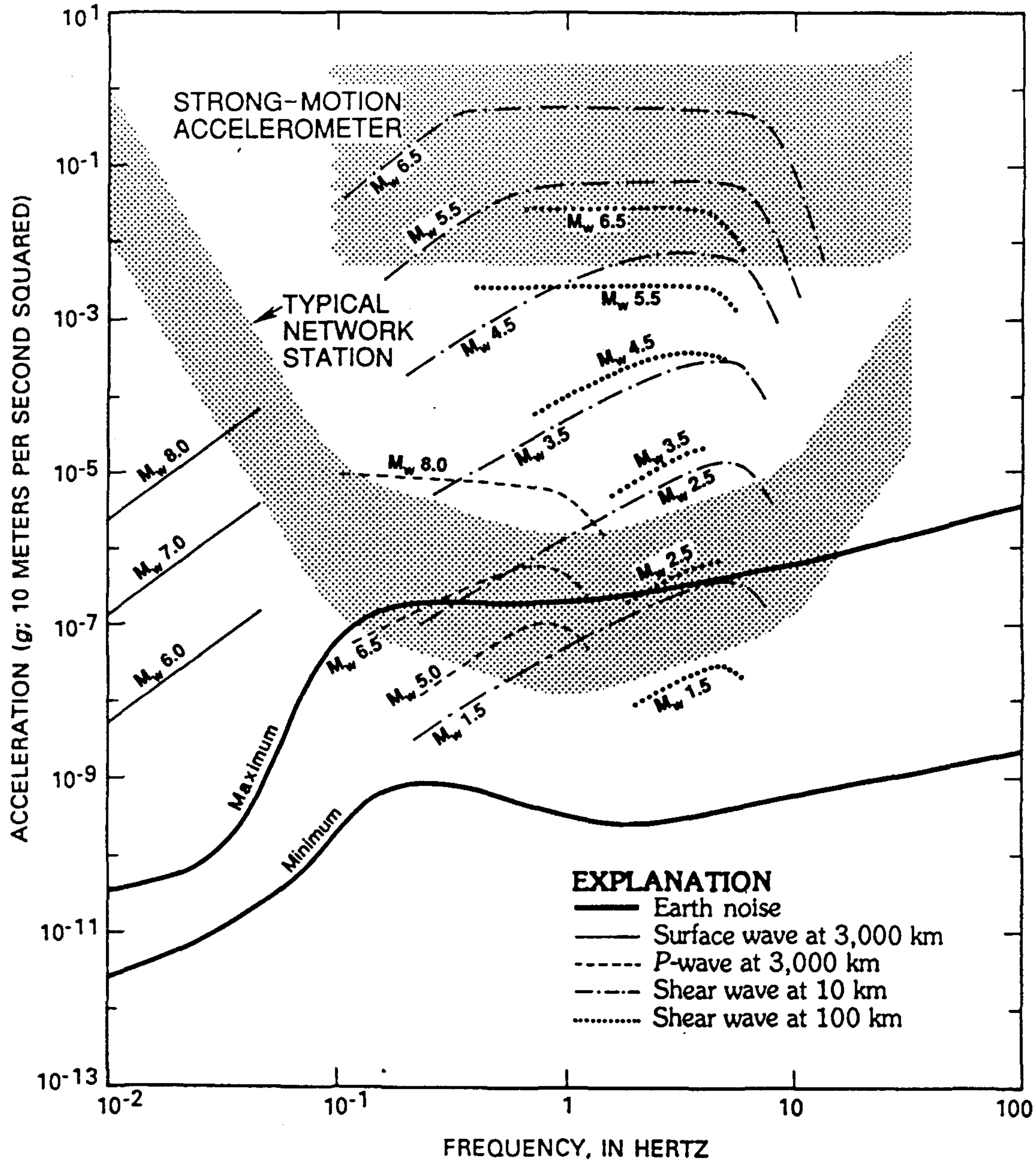


Figure 17. Plot of dynamic range versus frequency for (1) a typical regional-seismic-network station and (2) a typical strong-motion station (from Heaton and others, 1989). Also shown are the expected levels of ground motion for different seismic arrivals from earthquakes of different sizes and recorded at different distances.

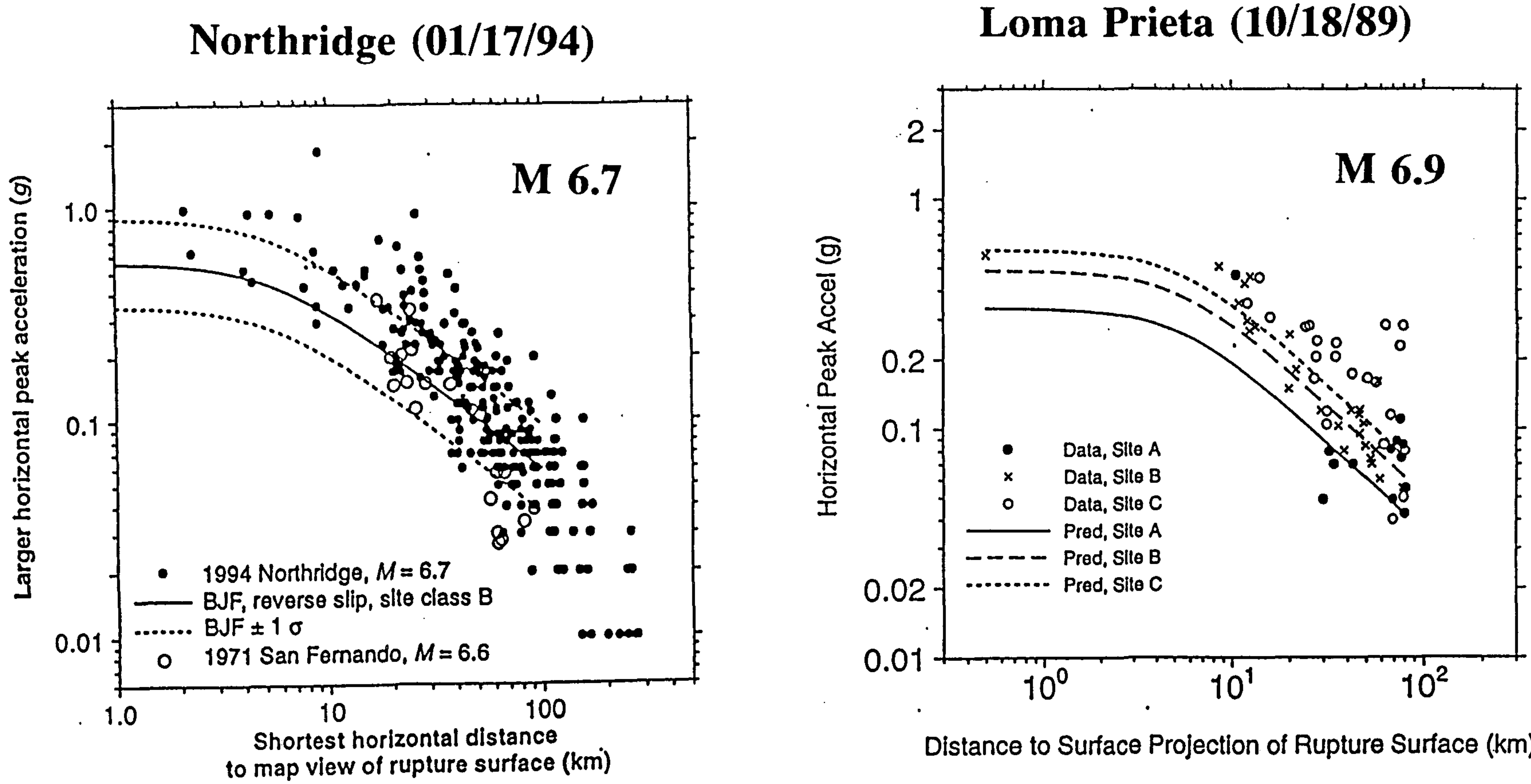


Figure 11. Peak horizontal acceleration as a function of distance for the 1989 Loma Prieta earthquake, compared with predictions from Boore *et al.* (1993).

Figure 18. Plots of peak horizontal acceleration as a function of distance for the Northridge, California, earthquake (left) (from USGS/SCEC, 1994) and for the Loma Prieta, California, earthquake (right) (from Boore and Joyner, 1994).

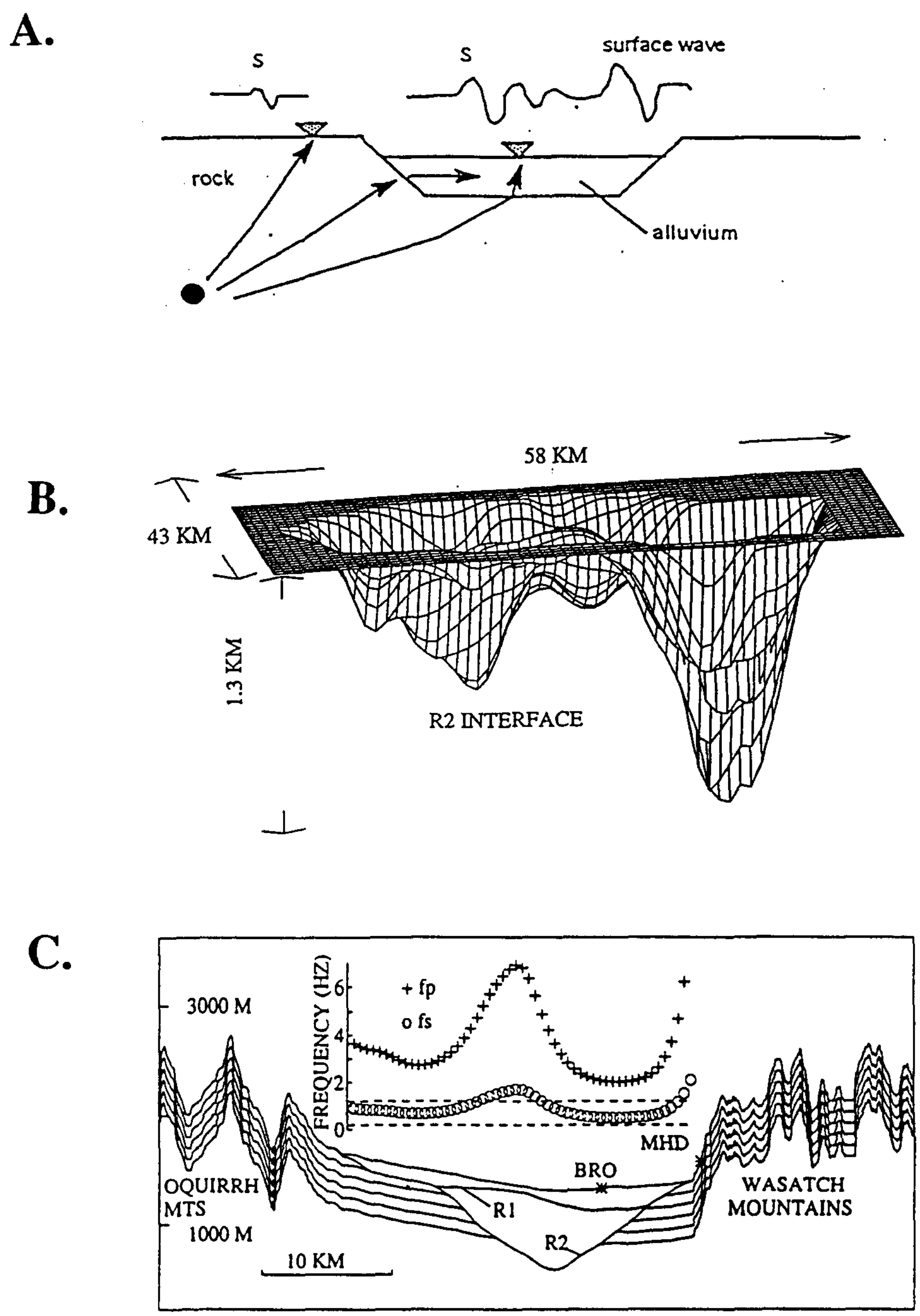
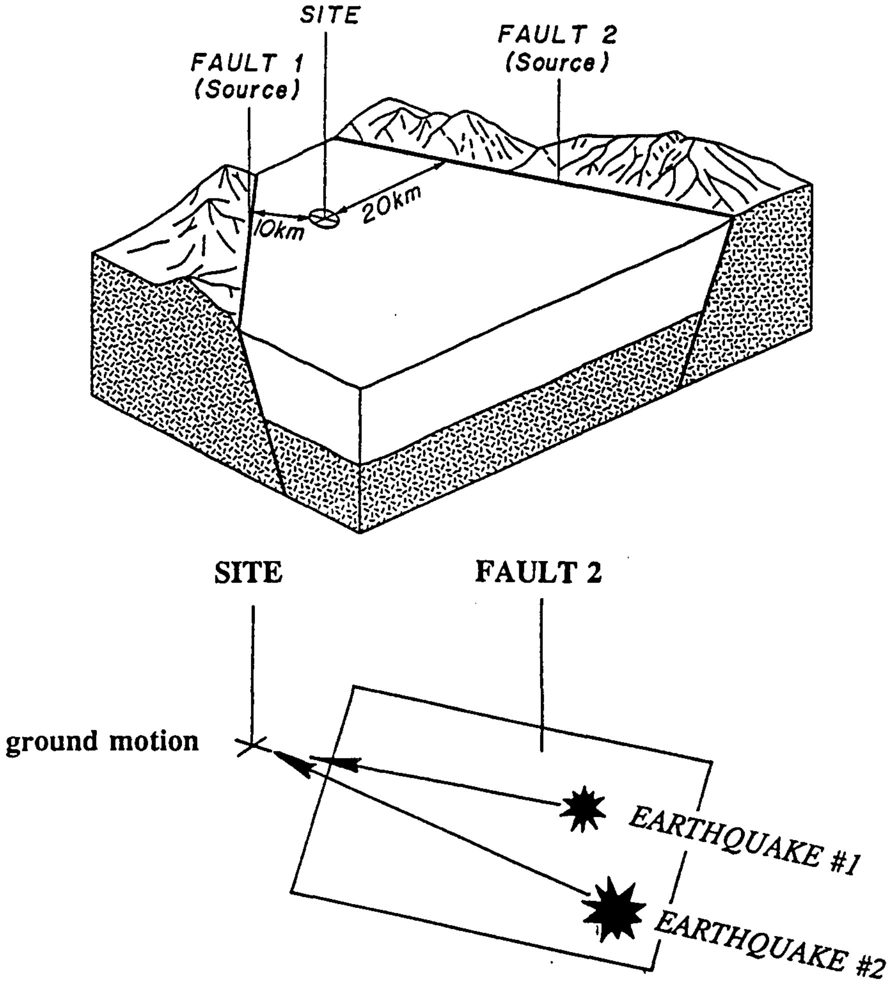


Figure 19. (A) Sketch (from Frankel, 1994) illustrating a vertical cross section of a sedimentary basin together with the hypocenter (filled circle) of an earthquake and simplified ray paths. Idealized seismograms for two sites (inverted triangles) are shown above. (B and C) Model representations of the Salt Lake Basin (from Olsen, 1994) showing (B) a 3-D perspective of the "R2" interface, an interface between semiconsolidated and consolidated basin fill and (C) a vertical cross section depicting a 2-D velocity model which includes topography, a near-surface velocity gradient in the bedrock, and a near-surface layer of low-velocity unconsolidated sediments (above "R1"). The inset in (C) shows the fundamental P-wave (+) and S-wave (o) vertical resonance frequencies for the unconsolidated sediment layer.



Probabilistic seismic hazard analysis (PSHA) basically involves a mathematical process to calculate the mean number of events per year in which the level of ground motion at a site exceeds some specified value.

Sum over all magnitudes and all locations within each source zone—and sum over all possible source zones—to get a Seismic Hazard Curve.

Figure 20. The basic idea of a probabilistic seismic hazard analysis for strong ground shaking (adapted, in part, from McGuire and Arabasz, 1990). Once the location and geometry of all sources of potential earthquakes are identified (as in the upper block diagram), the analysis proceeds as described in the lower part of the figure. Note that Earthquake #1 and Earthquake #2 are potential earthquakes.

SEISMIC HAZARD CURVE

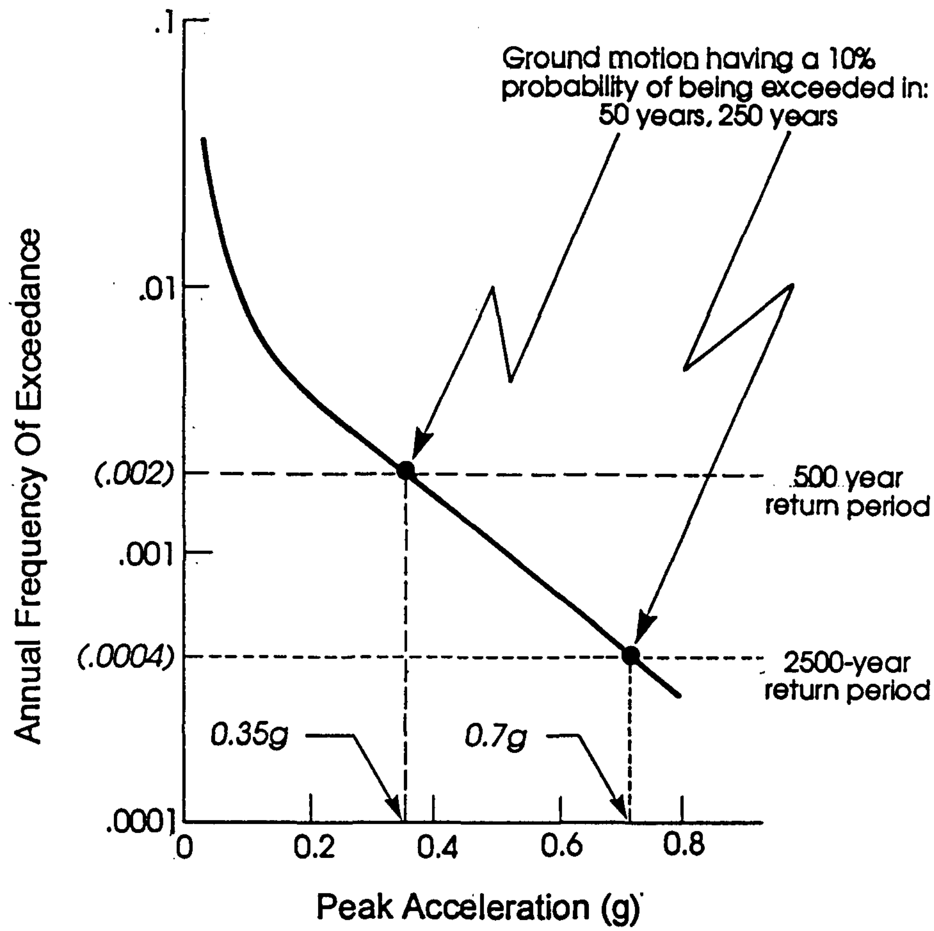
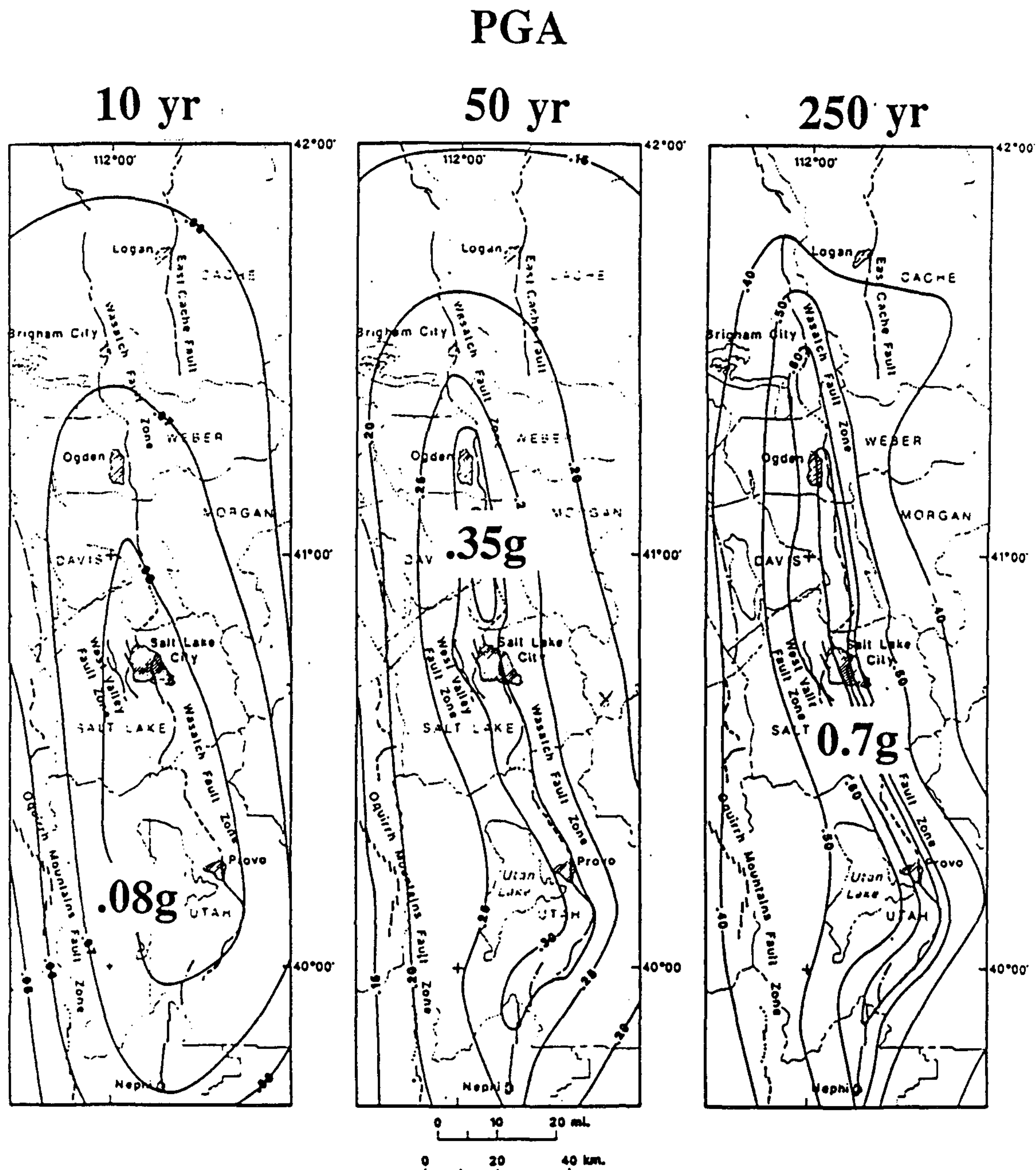


Figure 21. Graph of a simplified seismic hazard curve for a hypothetical site. From the procedure outlined in Figure 14, the curve gives the mean annual number of times (vertical axis) that a certain level of ground shaking (horizontal axis) is expected to be exceeded. The inverse (i.e., 1 divided by) an annual frequency is called a "return period." The ground motions having a 10 percent probability of being exceeded (or equivalently a 90 percent probability of not being exceeded) for some specified "exposure periods" are based on the Poisson model for random occurrence of events.



Contours of peak ground acceleration on soil sites with 10 percent probability of being exceeded in 10 years, 50 years and 250 years. Peak accelerations on rock sites are expected to be approximately 10 percent higher than the values shown on the map for 10 percent probability of exceedance in 10 years and approximately 20 percent higher than the values shown on the maps for 50 and 250 years.

Figure 22. Maps of probabilistic estimates of peak ground acceleration on soil sites in the Wasatch Front area for exposure periods of 10, 50, and 250 years (from Youngs and others, 1987).

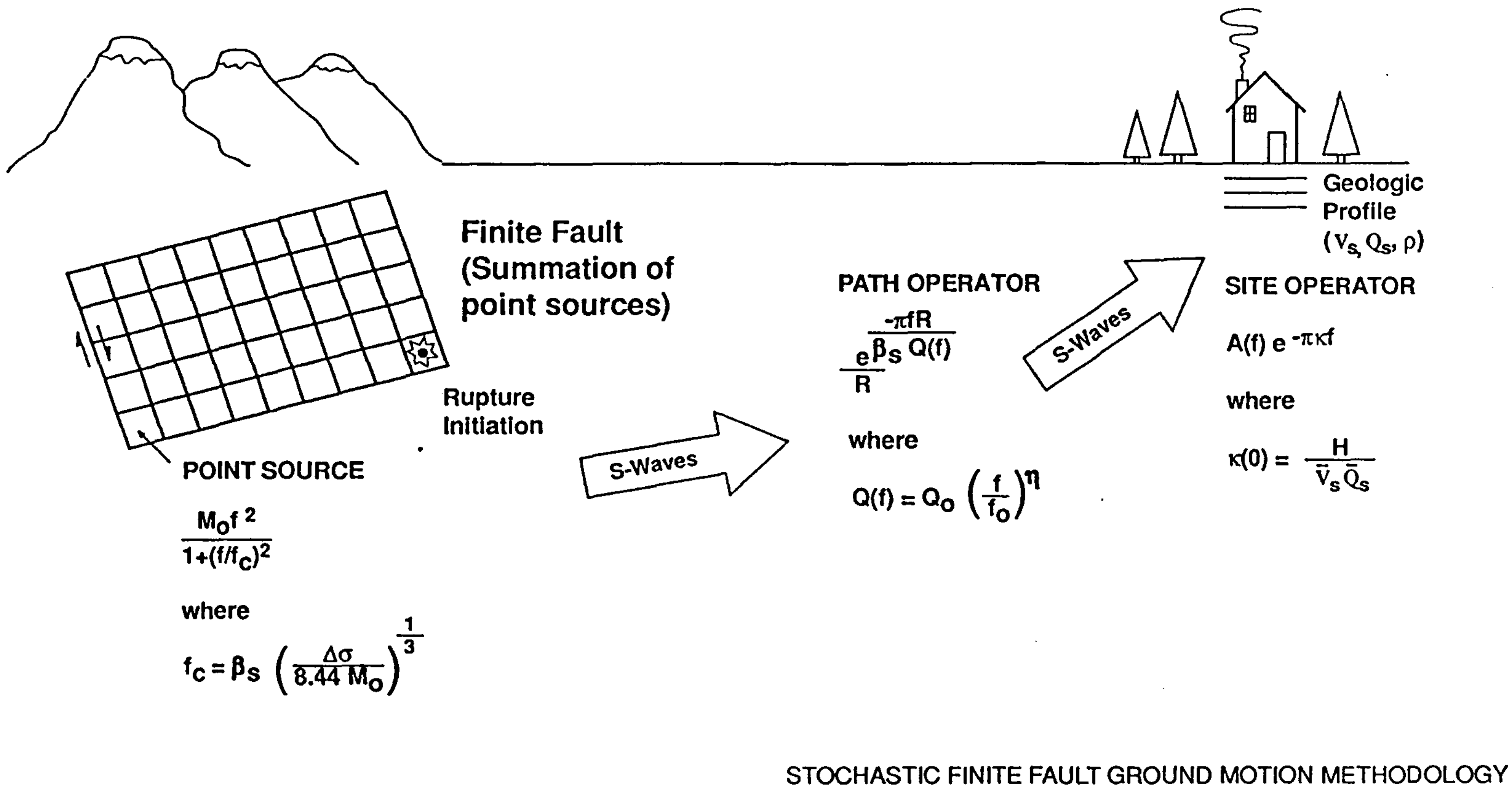


Figure 23. Diagram from Wong and Silva (1993) outlining the elements of a procedure used by them to characterize potential strong ground motion in the Salt Lake Valley, on a site-specific basis, from a local earthquake on the Wasatch fault of moment magnitude (M_W) 7.0.

3.

Soil Response in the Salt Lake Basin

by

Kyle Rollins

Kyle M. Rollins

Kyle Rollins graduated Summa Cum Laude with a B.S. in Civil Engineering from Brigham Young University in 1982. After working with RBG Engineering for about a year, he did his graduate work as a Berkeley Fellow under Professor H. B. Seed and received his Ph.D. in Geotechnical Engineering from the University of California at Berkeley in 1987. Upon graduation, he accepted a position on the Faculty at Brigham Young University and is currently an Associate Professor. Professor Rollins teaches graduate and undergraduate courses in geotechnical engineering and has been involved in research studies regarding earthquake engineering, soil improvement techniques and collapsible soils. He has published 27 technical papers and supervised 21 graduate students. Recently, Professor Rollins and his graduate students used computer models which they verified for soft soil profiles in the San Francisco Bay region to conduct ground response studies in the Salt Lake Valley. He is currently involved in an NSF-sponsored project to study the relationship between damage patterns observed in the Northridge earthquake and soil conditions.

SOIL RESPONSE IN THE SALT LAKE BASIN

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INTRODUCTION

The influence of local soil conditions on earthquake shaking and damage intensity has been recognized for many years. The 1985 Mexico City and the 1989 Loma Prieta earthquakes provided unmistakable evidence that local soil conditions can significantly alter ground motions in comparison with rock motions. During these earthquakes, substantial differences in recorded maximum accelerations were observed at soft soil sites in relation to rock sites. In the 1994 Northridge earthquake, strong motion data suggest that accelerations were amplified on stiff shallow sites at the edge of the basins. Conditions in some areas of the Salt Lake Valley have similarities with soft soils around San Francisco bay where soil amplification was observed during the 1989 Loma Prieta earthquake. In addition, soil profiles adjacent to the Wasatch mountains have similarities to sites where amplification was observed in the Northridge earthquake.

Because of variations in the depth and shear wave velocity of soil deposits, soil response can cause significant variations in ground motions across a valley and lead to major variations in damage patterns for a given earthquake. The influence of soil conditions on induced base shear force may reach 500% in some cases in comparison with forces on rock (Seed, 1987). Taller structures tend to experience greater damage when located on deep soil deposits while shorter structures experience more damage when located on shallow soil deposits. These effects are largely associated with resonance between the building and the soil deposit. This paper examines the possibility of soil amplification in selected areas of the Salt Lake Valley, estimates expected ground motion characteristics, and provides an evaluation of the potential for building damage for these areas.

SOIL CONDITIONS IN THE SALT LAKE VALLEY

Some generalizations about soil conditions throughout the Salt Lake valley can be made based on past geological studies and borehole logs. Typical soil profiles throughout the valley have been defined by over 25 deep borings drilled by the US Geological Survey (USGS) (Tinsley, 1991). These borings generally extended to either 61 m (200 ft) or bedrock and both soil and shear wave velocity profiles are available for each hole. Near the mountain fronts, soil deposits tend to consist of coarse alluvial deposits overlying bedrock at shallow depths. USGS borings in these areas encountered soil profiles dominated by layers of coarse sand and gravel. The average depth to the underlying rock or rock-like layer ($V_s > 760$ m/sec or 2500 ft/sec) was about 27 m (89 ft). Towards the center of the valley, soil profiles consist of deep, relatively soft lacustrine sediments deposited by Lake Bonneville, which covered the valley in Pleistocene time. Soils in this area consist predominantly of silts, sands, and clays and true bedrock is over 1500 m (5000 ft) deep as shown in Figure 1 (After Fox, 1983).

Based on amplification measurements of very low-strain motions produced by nuclear detonations in Nevada, the US Geological Survey (USGS) has proposed amplification zones throughout Salt Lake Valley as shown in Figure 2. The contours indicate the average ratio between spectral acceleration on soil to that on rock for very low-strain levels. While the amplification for earthquake motions are expected to be different do to soil nonlinearity, these contours do roughly correlate with soil type and depth and have been used to define four amplification zones for use in this paper. The four zones are bounded by the 1-3, 3-5, 5-8 and 8-11 contours shown in Figure 2.

Representative boring logs for each zone are presented in Figures 3-6. Based on data from 3 to 4 borings in each amplification zone, average properties such as soil consistency, soil depth, and shear wave velocity are summarized for each amplification zone in Table 1. There is a clear trend of decreasing shear wave velocity from the stiff, shallow sites near the mountain fronts to the soft, deep soil sites near the valley center. Average shear wave velocity profiles for the various zones are presented in Figure 7 and the shear wave velocity profile for the soft deep profiles is very similar to that for soft deep profiles on the margins of San Francisco bay.

Table 1 Characteristics of amplification zones in Salt Lake Valley, Utah.

USGS Contour Zone	1-3 Zone	3-5 Zone	5-8 Zone	8-11 Zone
Avg Depth to Rock (m)	28 (Shallow)	43 (Deep)	>60 (Deep)	>60 (Deep)
Shear Wave Velocity Range (m/sec)	180-915	215-790	150-490	137-425
Soil Consistency	Stiff	Medium Stiff	Medium Soft	Soft
Seismic Site Coefficient	S1 (1.0)	S2 (1.2)	S2-S3 (1.35)	S3 (1.5)
USGS Boring Sites within each Zone	UGS006 SUN042 LAI044	BON043 ROO046 TMP052	AVH015 MAG051 CCB056	EAP017 WAP019 RIV029 DUC053

GROUND RESPONSE ANALYSES FOR SALT LAKE VALLEY SITES

To evaluate ground response in each amplification zone due to strong ground shaking, analyses were performed using the 1-D computer program SHAKE (Schnabel et al, 1972). Shake computes the response of a layered system to vertical propagating shear (SH) waves using wave propagation theory. Analyses were performed at three to four sites in each zone where good information from USGS deep borings is available. These sites are tabulated in Table 1 and their locations are shown in Figure 2. Shear modulus degradation and damping curves were based on soil PI for cohesive soils and soil density for cohesionless soils using standard relationships shown in Figures 8 and 9 respectively.

Selection of Rock Input Motions

To evaluate ground response in the Salt Lake Valley, it is necessary to select rock motion records which would be typical of earthquakes along the Wasatch fault. Because of the absence of recorded ground motions, this selection requires some judgement. We selected nine rock motions for use in the ground response analysis which have response spectra shapes similar to mean shapes for rock proposed by Idriss (1985). These nine records represent a fairly wide range of ground motions which might be expected due to a rupture on the nearby Wasatch fault. The rock input motions were scaled to produce peak acceleration values of 0.35 g and 0.70 g. These acceleration levels generally correspond to peak ground accelerations on rock with a 90% probability of not being exceeded in 50 and 250 years respectively based on studies by Algermissen et al (1982) and Youngs et al (1988). These peak accelerations are primarily associated with M 7 earthquakes on the Wasatch Fault. The 250 year acceleration level is close to that expected from the maximum credible earthquake on the Wasatch fault for many locations in the valley based on typical attenuation relationships (Seed et al, 1986; Sun et al, 1988). SHAKE analyses were performed for all nine input records at two acceleration levels at each of the 13 sites.

Computed Peak Accelerations

For the rock input acceleration levels in this study (0.35 to 0.70 g), the computed peak ground accelerations are highest for the stiff sites and decrease as the soil profiles becomes softer. Computed peak ground accelerations on soil are plotted versus peak ground acceleration on rock in Figure 10 for the stiff shallow (1-3 Zone) and soft deep (8-11 Zone) soil sites. While soft soil profiles typically amplify peak acceleration at low acceleration levels, the softer profiles actually attenuate the peak acceleration at higher acceleration levels due to shear modulus degradation and increased damping at higher strain levels. This pattern is consistent with observations made by Idriss (1990) for soft soils.

The computed peak accelerations on stiff soils are somewhat higher than expected. These higher ground acceleration levels occur because the natural period of the soil is close to the predominant period of the incoming rock motions. Amplification of peak accelerations on stiff, shallow soil profiles is not without precedent. Seed (1987) reported that amplification occurred on stiff shallow profiles during the 1976 Friuli earthquake as shown in Figure 11. Recently, very high peak accelerations were recorded at several stiff soil sites on the fringes of the San Fernando and Los Angeles basins during the Northridge earthquake. In addition, building damage was concentrated along the basin fringes at several sites where stiff shallow soils are known to exist as shown in Figure 12.

Computed Response Spectrum Shapes

For each site, the mean acceleration response spectrum was calculated based on the computed response spectra for the nine input records. The mean spectra for all the sites in a zone were then plotted and a representative response spectrum was drawn to envelope the mean response spectra for the zone as shown in Figure 13. The representative spectra for each zone due to the 0.35 g and 0.70 g input motions are shown in Figure 14. In the short period range

(T < 0.8), spectral accelerations are much higher for stiff soils than for soft soils. At a period of 0.4 seconds, for example, spectral accelerations on stiff soils are twice that on soft soils for the 0.35 g input acceleration level. This results from the fact that the stiff, shallow profiles had site periods of near 0.40 seconds. At longer periods, (T > 1.0), the trend reverses and spectral accelerations are significantly greater on soft soils than on stiff soils. This results from the fact that the soft deep soil profiles investigated in this study have site periods of around 1.8 seconds. Overall, the stiff sites have the highest spectral accelerations but these high values occur over narrow period bands. The spectral acceleration peaks for the soft soils, while lower, occur over much wider period bands. Although the development of these representative spectra shapes does not eliminate the need for site-specific investigations for more important structures, it does provide some basis for comparison for future ground response studies in the valley.

EFFECT OF GROUND RESPONSE ON POTENTIAL BUILDING DAMAGE

Damage Potential Index Method

Although ground response studies show that spectral accelerations will be highest on soft deep soil profiles for long period buildings in Salt Lake valley, this does not guarantee that building damage will be higher in these zones. Consideration must also be given to the seismic code requirements used in designing buildings on these sites. For example, seismic codes have recommended higher design forces for long period structures on soft clay profiles since the mid 1970's. The damage potential index (DPI) approach developed by Seed and Sun (1989) provides a method for assessing the degree of protection provided by building code provisions in relation to the expected ground motions. The DPI is proportional to the ratio of the forces exerted by an earthquake to the lateral resisting force prescribed by the building code and is a function of building period. Because this index includes both building resistance and earthquake forces, the DPI can be important in evaluating the effects of soil amplification on structural damage.

The DPI is defined by the relation,

$$\text{DPI} = \frac{\text{Induced Force} \times \text{Duration of Force}}{\text{Design Resistance}} \tag{1}$$

After some simplification, the DPI can be expressed by the equation

$$\text{DPI} = \frac{S_v \cdot (\text{DWF})}{k \cdot R_f} \tag{2}$$

where S_v is the spectral velocity, k is the design lateral force coefficient used to design the structure, DWF is a duration weighting factor which depends on the duration of the earthquake, and R_f is the structural resistance factor which expresses the relative design resistance as affected by allowable stresses, load combinations, and construction quality.

Following the 1985 Mexico City earthquake (M 7.5), Seed and Sun (1989) computed the DPI for the heavy damage region of the Mexico City using both recorded and computed motions. A detailed survey was also made of the damage to different classes of structures in the heavy damage area. The damage intensity was defined as the ratio of the number of structures in any given category which suffered major damage divided by the total number of structures in that category existing in the heavy damage area. A strong linear correlation was observed between the DPI and the damage intensity. A DPI value of 61 m/sec (200 ft/sec) corresponded to an observed damage intensity of 30%. The DPI and damage intensity for the heavy damage area in Mexico City are plotted as a function of building period in Figure 16 and the highest DPI values are associated with buildings having periods near 2 seconds.

Lateral Force Coefficient Based on Seismic Code Provisions

The building code governing most construction in the Salt Lake Valley is the Uniform Building Code which generally patterns its seismic provisions after SEOAC recommendations. According to the present code (1994), the design lateral force, V, is determined by the following expression:

$$V = \frac{1.25 \cdot Z \cdot S \cdot I \cdot W}{R_w \cdot T^{0.667}} \qquad (3)$$

Where Z is the seismic zone factor, I is the importance factor, R_w is the structural system factor, T is the building period, W is the building weight and S is the soil site coefficient. The site coefficient ranges from 1.0 for rock/stiff (S_1) sites to 2.0 for very soft (S_4) sites. For this study, I was equal to 1, R_w was 10, and Z was 0.3 corresponding to seismic zone 3. Site coefficients for each amplification zone are listed in Table 1 along with the mean numerical value used in the lateral force calculations.

Computation of DPI Values for Salt Lake Valley

In calculating the DPI, a value of 1.0 was assigned for the duration weighting factor (DWF). This represents the expected duration of strong ground shaking for a M 7.0-7.5 earthquake on the Wasatch fault. The building resistance factor, R_f, was assigned a value of 1.2 based on expert opinion from local structural engineers. This factor relates the varying code requirements and construction standards of Salt Lake to San Francisco (R_f=1.3) and Mexico City (R_f=1.0). The computed DPI for all four amplification zones is plotted as a function of period for 50 and 250 year acceleration levels in Figure 15. Although the code requirements tend to equalize the DPI for various soil sites, the DPI is highest for stiff shallow sites at short periods and for soft deep sites at longer periods. In the short period range ($T < 0.7$ sec.), the DPI on stiff shallow sites is more than twice as high as for soft deep sites and predicted damage intensity which means damage would also be twice as high. At higher periods ($T > 1.5$ sec.), the DPI for soft deep soil sites is about 30% higher than for stiff shallow sites.

Comparison of DPI Values for Soft Soil in Salt Lake, San Francisco, and Mexico City

Comparisons of computed DPI values on soft soils in Mexico City, San Francisco, and Salt Lake are shown in Figure 16a. The DPI values for the 0.35 g rock acceleration input in Salt Lake are about 40% higher than those computed for soft soils in San Francisco for a similar rock input motion. While the comparisons represent response to roughly similar earthquake events, it should be recognized that the earthquake recurrence intervals for Utah are much longer than for San Francisco. DPI values for both Salt Lake and San Francisco are significantly lower than observed in Mexico City for the 0.35 g acceleration levels.

A comparison of DPI values for maximum credible earthquake events in San Francisco and Salt Lake City are also shown in Figure 16a. For San Francisco the maximum credible earthquake is a M 8+ earthquake on the San Andreas fault, while for Utah it would be a M 7.5 earthquake on the Wasatch fault. The damage intensity on soft Salt Lake Valley soils is approximately the same as that for San Francisco and both are similar to the damage observed in Mexico City. The similarity in DPI values is partially due to the fact that Salt Lake is in seismic zone 3 while San Francisco is in seismic zone 4.

Comparison of DPI Values for Stiff Soil in Salt Lake and San Francisco

A comparison of DPI values for stiff soil/rock sites in San Francisco and stiff, shallow soil sites in Salt Lake is shown in Figure 16b for the two acceleration levels studied. The DPI values are significantly higher for the Salt Lake profiles. The differences are particularly large at periods around 0.5 seconds where the DPI is nearly three times as high. For the maximum credible earthquake conditions, the computed DPI for the Salt Lake sites approaches that observed in the Mexico City heavy damage zone even at long periods. These results suggest that structures constructed on stiff, shallow soils in Salt Lake City will be subjected to increased risks of damage over a relatively large period range.

CONCLUSIONS

1. The Salt Lake Valley can be roughly divided into four zones based on geology, soil conditions, and probable ground response. These four areas have been designated as (1) stiff, shallow soils (2) medium stiff, deep soils, (3) medium soft, deep soils, and (4) soft deep soils.

2. Representative spectral shapes have been computed to characterize ground response in each of the four zones. At the 50 year acceleration level, response spectra shapes for stiff sites have the highest spectral acceleration, but these high values occur over narrow period bands. The spectral accelerations for the soft soils, while being lower, occur over much wider period bands.

3. The current, 1991, Uniform Building Code generally serves to equalize the DPI for various soil profiles, however DPI values are higher for stiff shallow sites at short periods and soft deep sites at long periods.

4. Based on DPI values, the predicted damage intensity on soft deep soils in Salt Lake Valley is similar to that in San Francisco for roughly similar earthquake events. For the 0.70 g input motions, the damage intensity equals that observed in Mexico City. The hazard posed by these observations is tempered by the fact that recurrence intervals are much longer for earthquakes in Salt Lake Valley than in San Francisco.

5. Based on DPI values, the predicted damage intensity on stiff shallow soils in Salt Lake Valley is 2 to 3 times higher than that for soft soils at short periods. This represents a significant hazard to low-rise structures around the edge of the Salt Lake Valley.

ACKNOWLEDGEMENT

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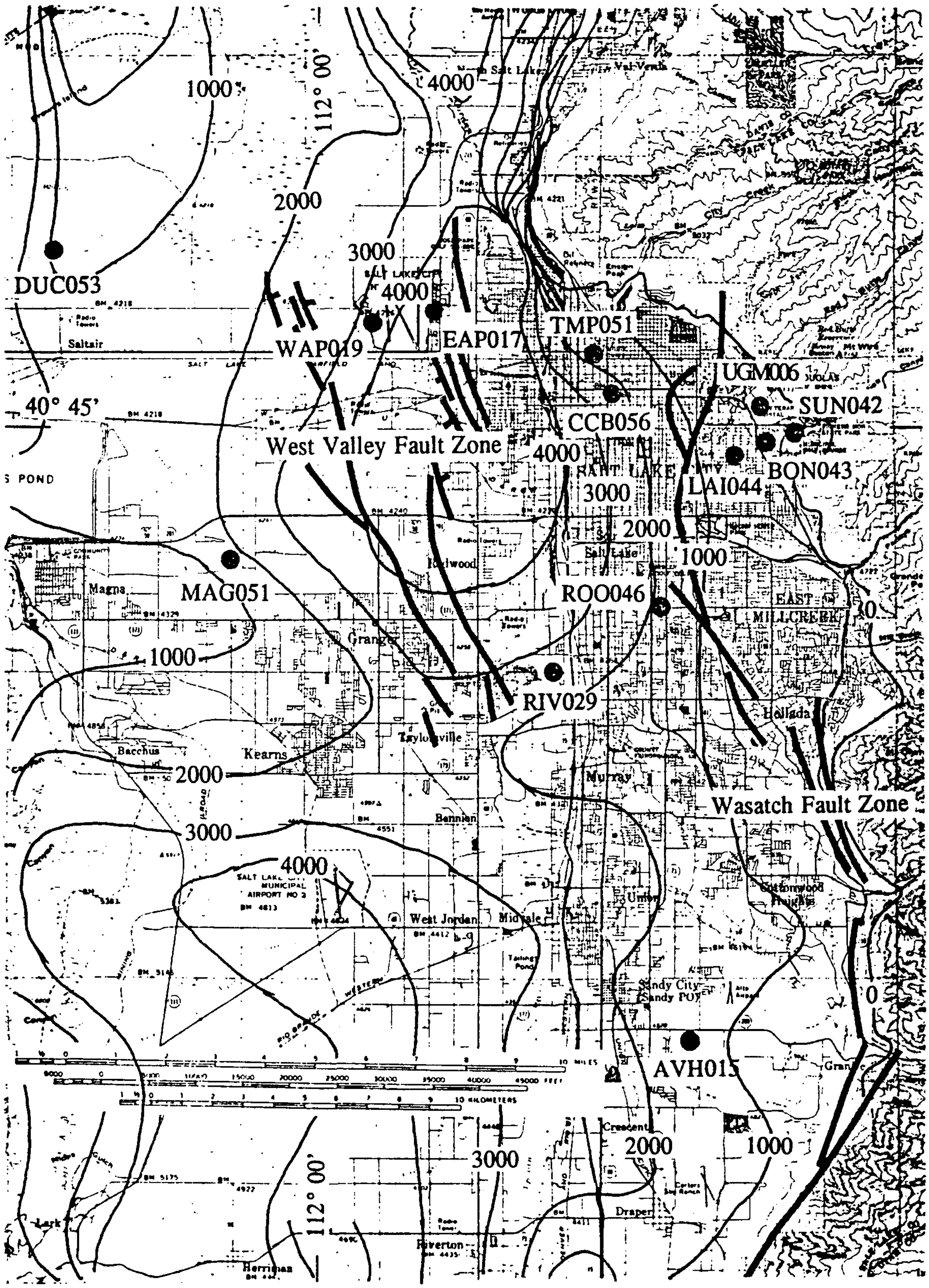


Figure 1 Contours of depth to bedrock in relation to deep borehole sites and fault zones.

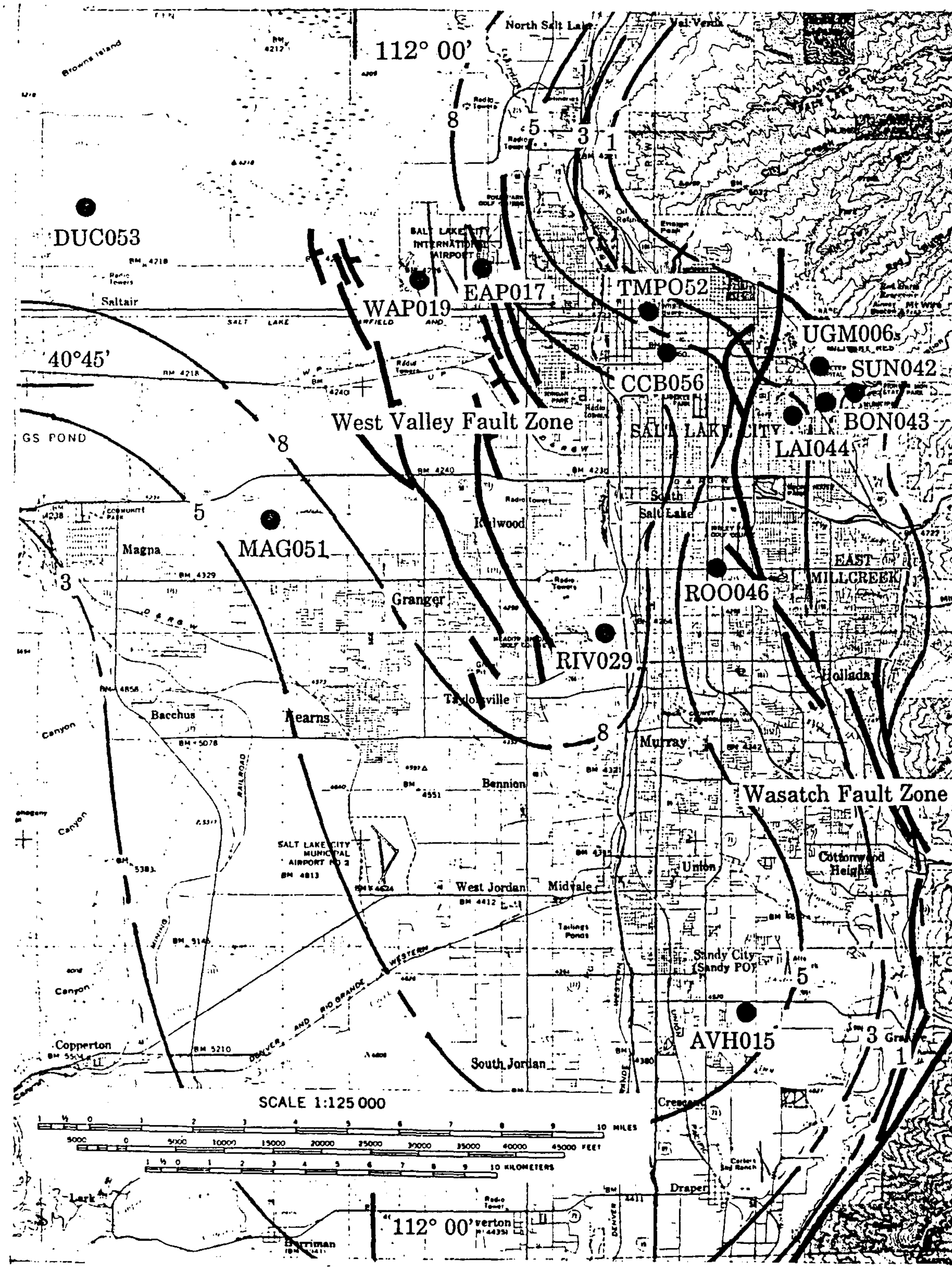


Figure 2 Test site locations, major fault zones, and contours of average spectral ratios on alluvium relative to bedrock in the Salt Lake Valley

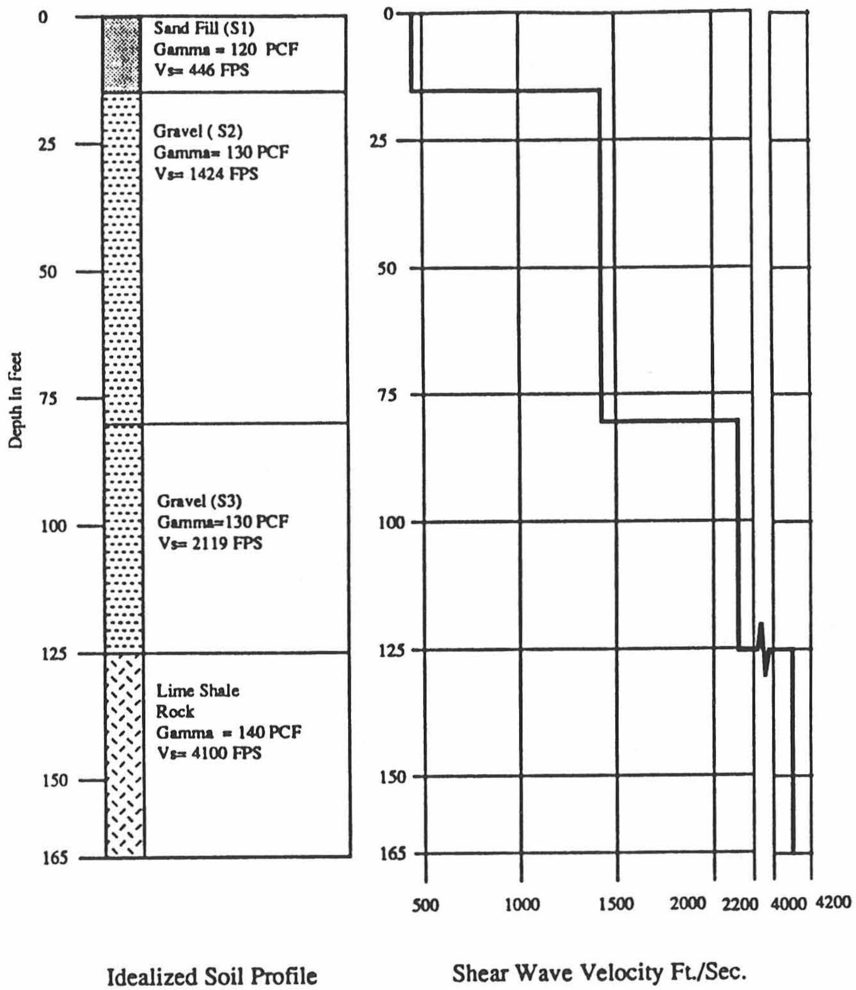


Figure 3 Soil and shear wave velocity profile at the Sunnyside Traing Center Site (SUN042), Stiff Shallow Site (after Tinsley et al, 1991).

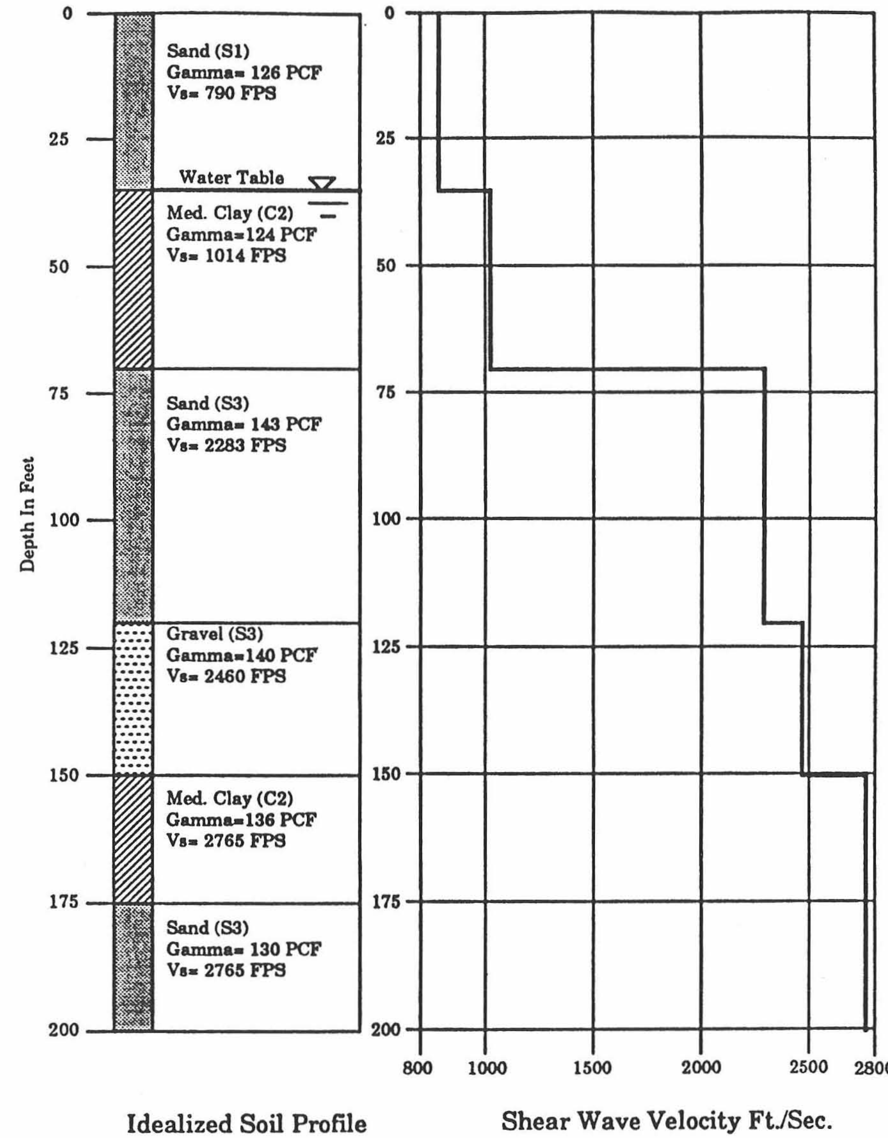


Figure 4 Soil and shear wave velocity profiles at the Bonneville Golf Course site (BON043), Med. Stiff Deep Site (After Tinsley et al, 1991).

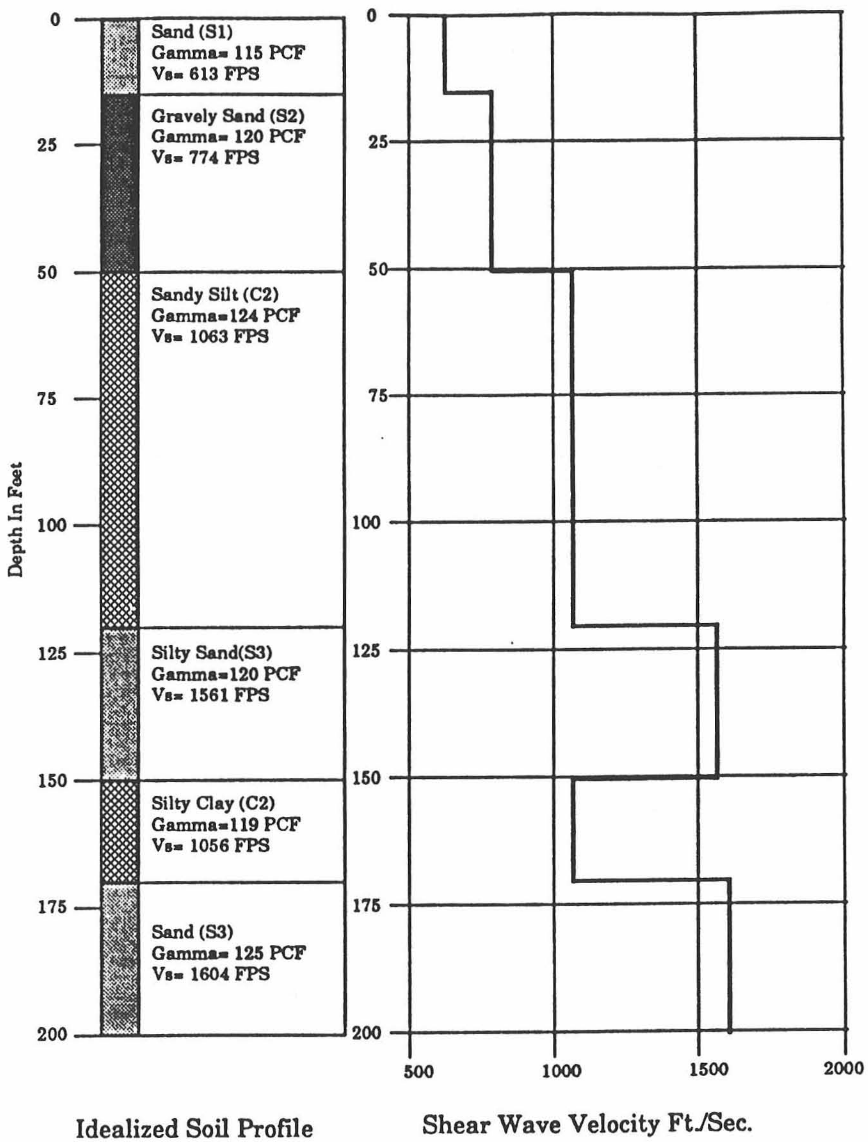


Figure 5 Soil and shear wave velocity profiles at the Alta View Hospital site (AVH015), Med. Stiff, Deep Site (After Tinsley et al, 1991).

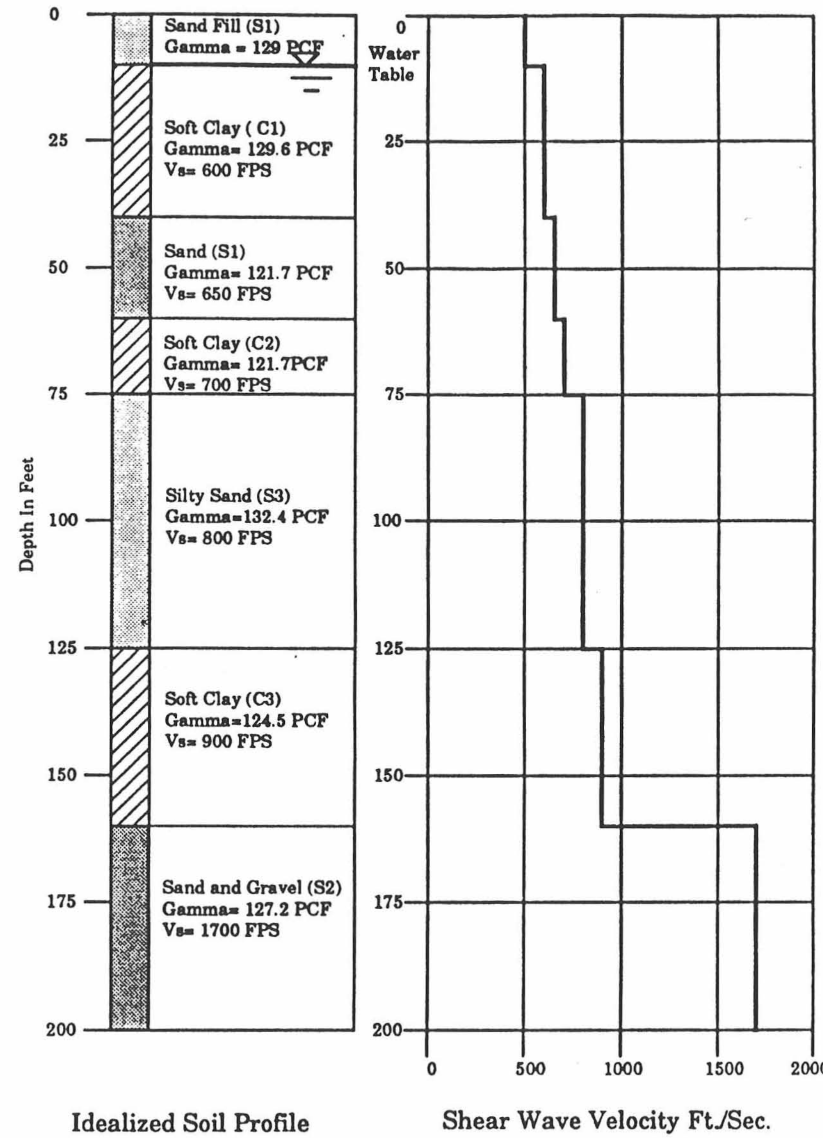


Figure 6 Soil and shear wave velocity prfoiles at the West Airport Site (WAP019), Soft Deep Site (After Tinsley et al, 1991).

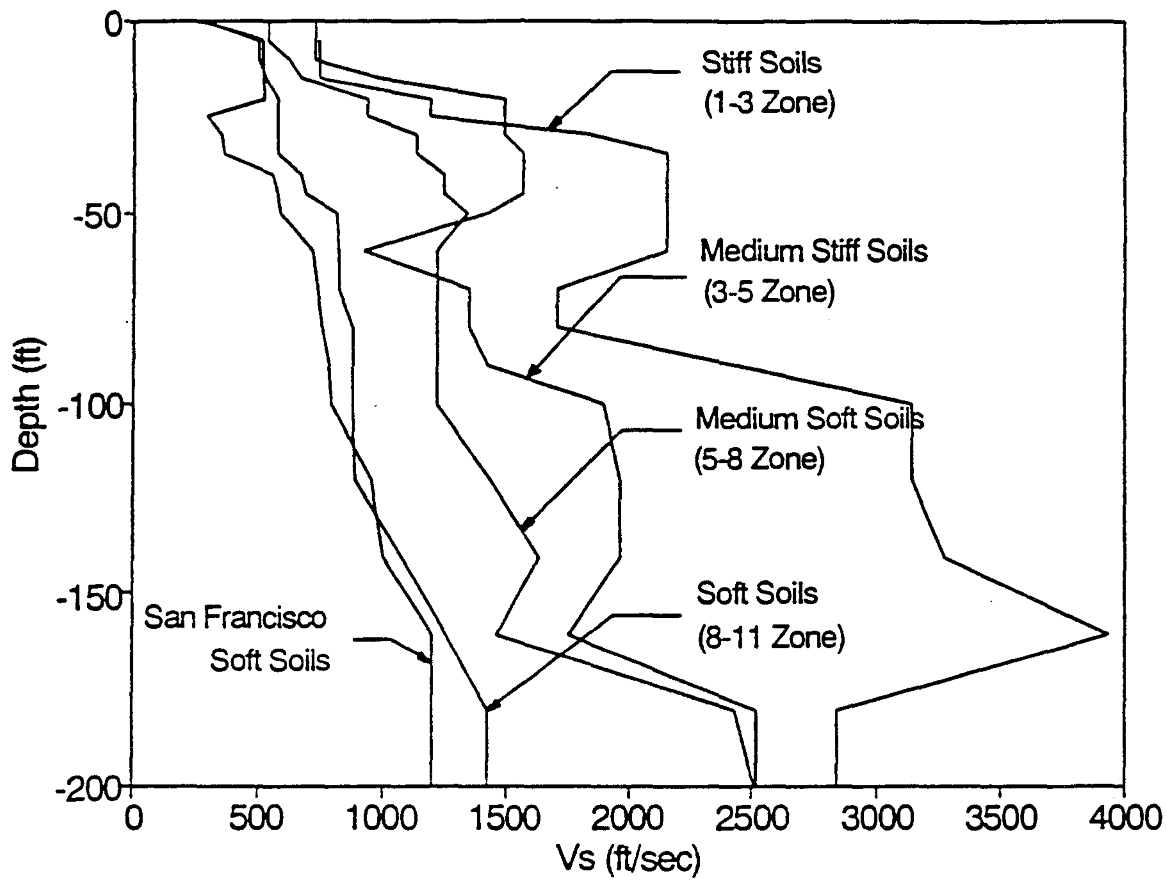


Figure 7 Average shear wave velocity profiles for Salt Lake Valley sites in comparison with average profile for soft soil sites on the margins of the San Francisco bay.

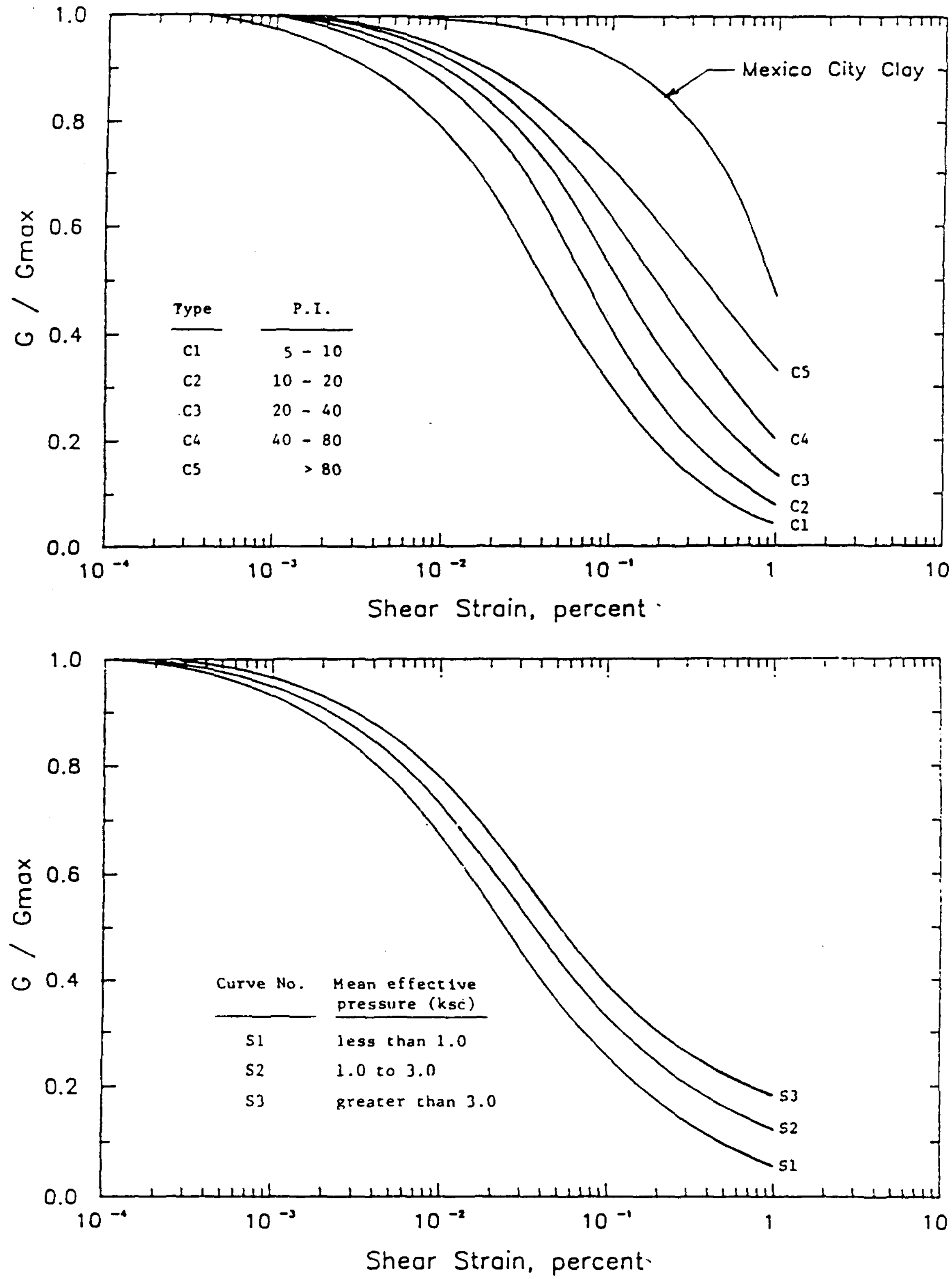


Figure 8 Normalized shear modulus degradation versus cyclic shear strain curves for clays as a function of plasticity index (PI) (Sun et al, 1988) and for sands with different mean effective stresses (Seed et al, 1986) used in the SHAKE analyses.

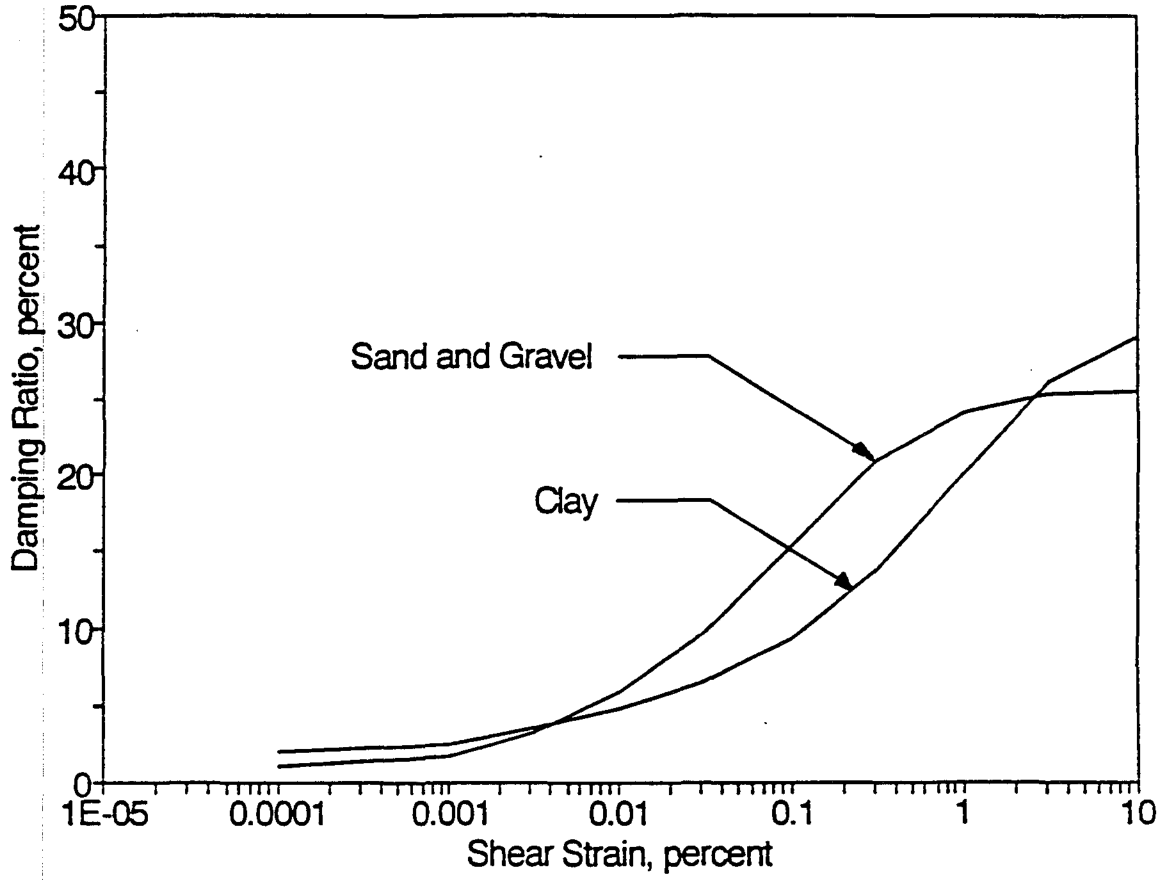


Figure 9 Damping ratio versus cyclic shear strain curves for clays and sands used in SHAKE analyses (Seed et al 1986; Sun et al, 1988)

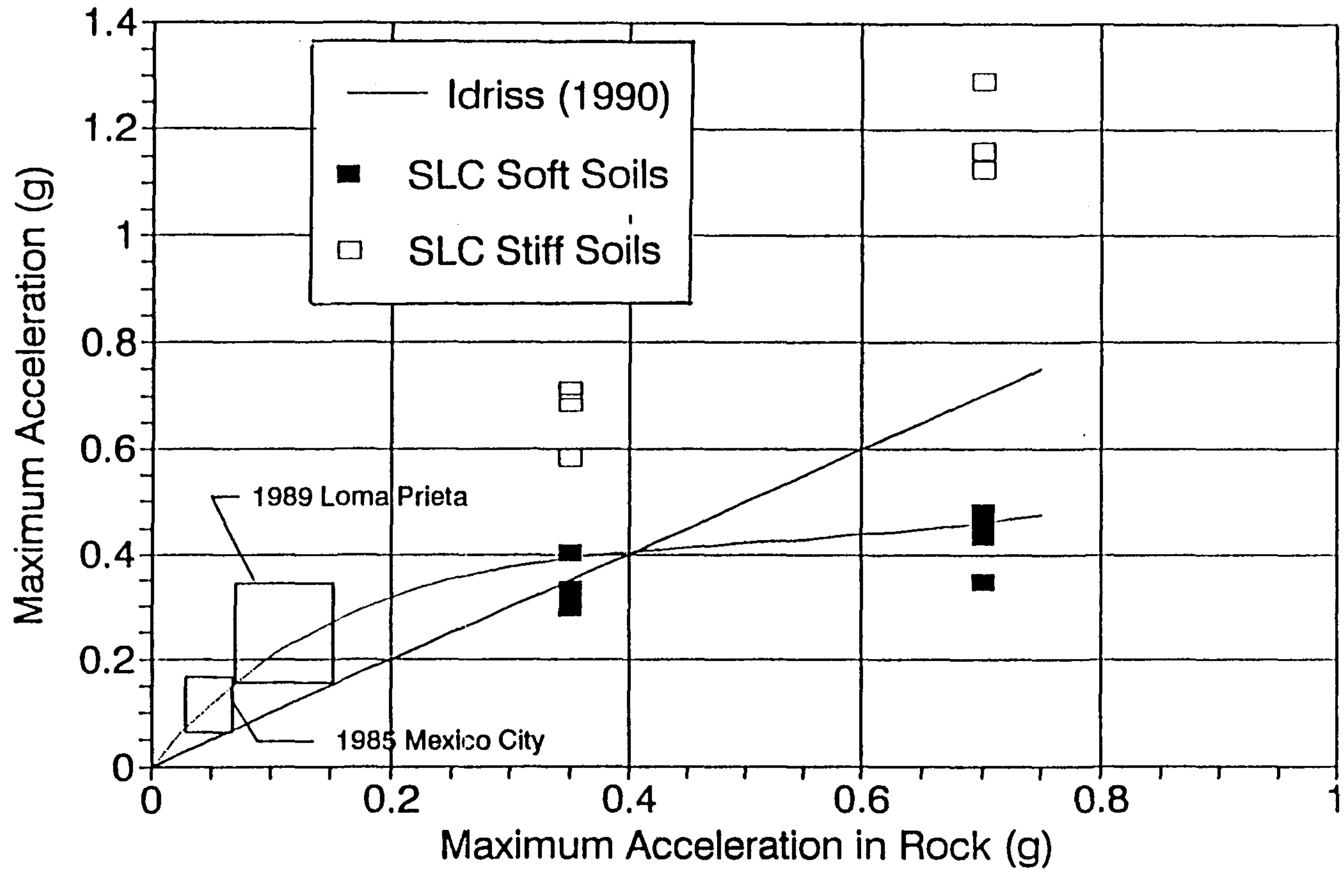


Figure 10 Peak ground acceleration on soil as a function of peak ground acceleration on rock at equivalent distances.

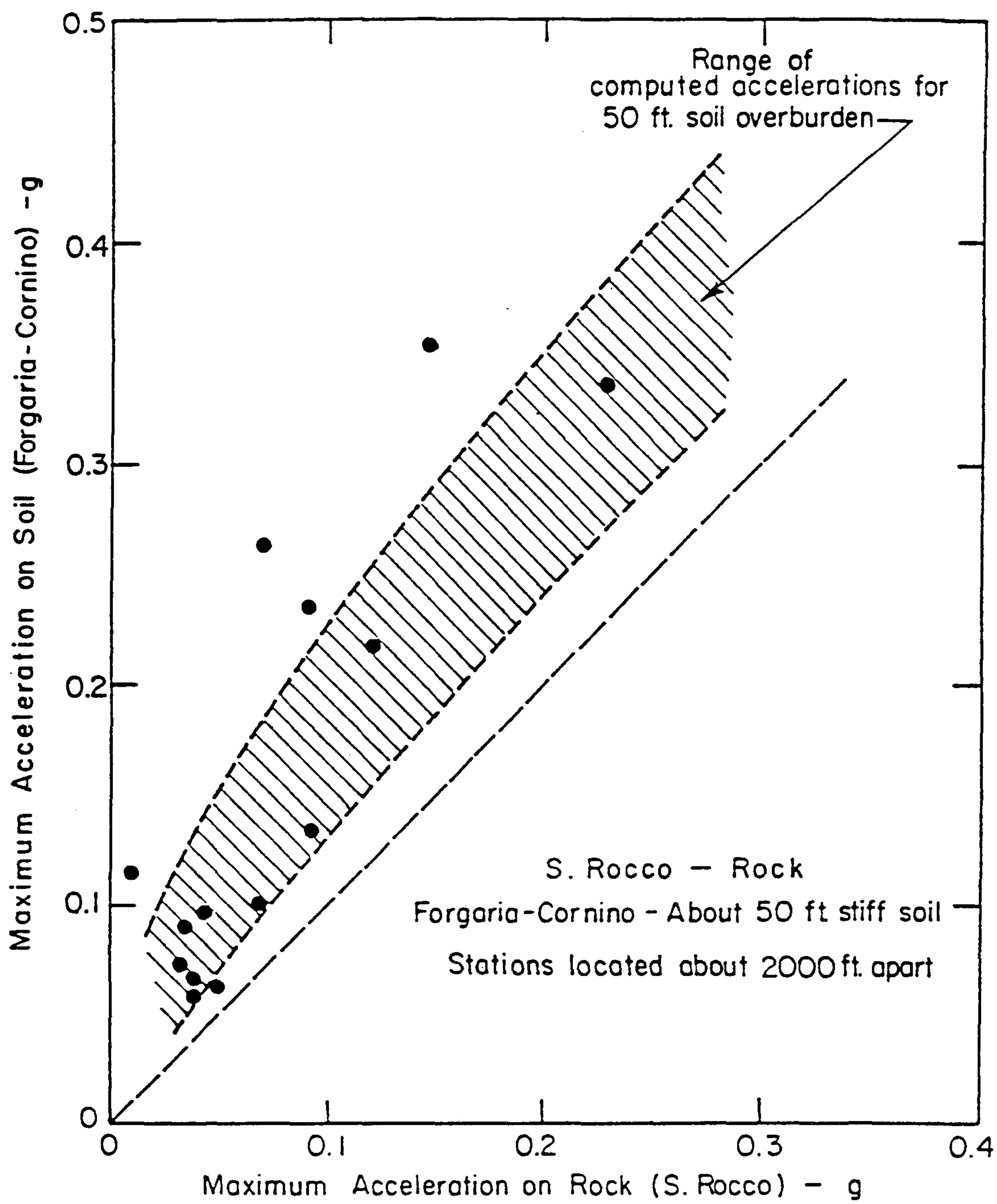


Figure 11 Amplificaiton of peak ground accelerations by stiff shallow soil layers (Seed, 1987; Muzzi and Vallini, 1977)

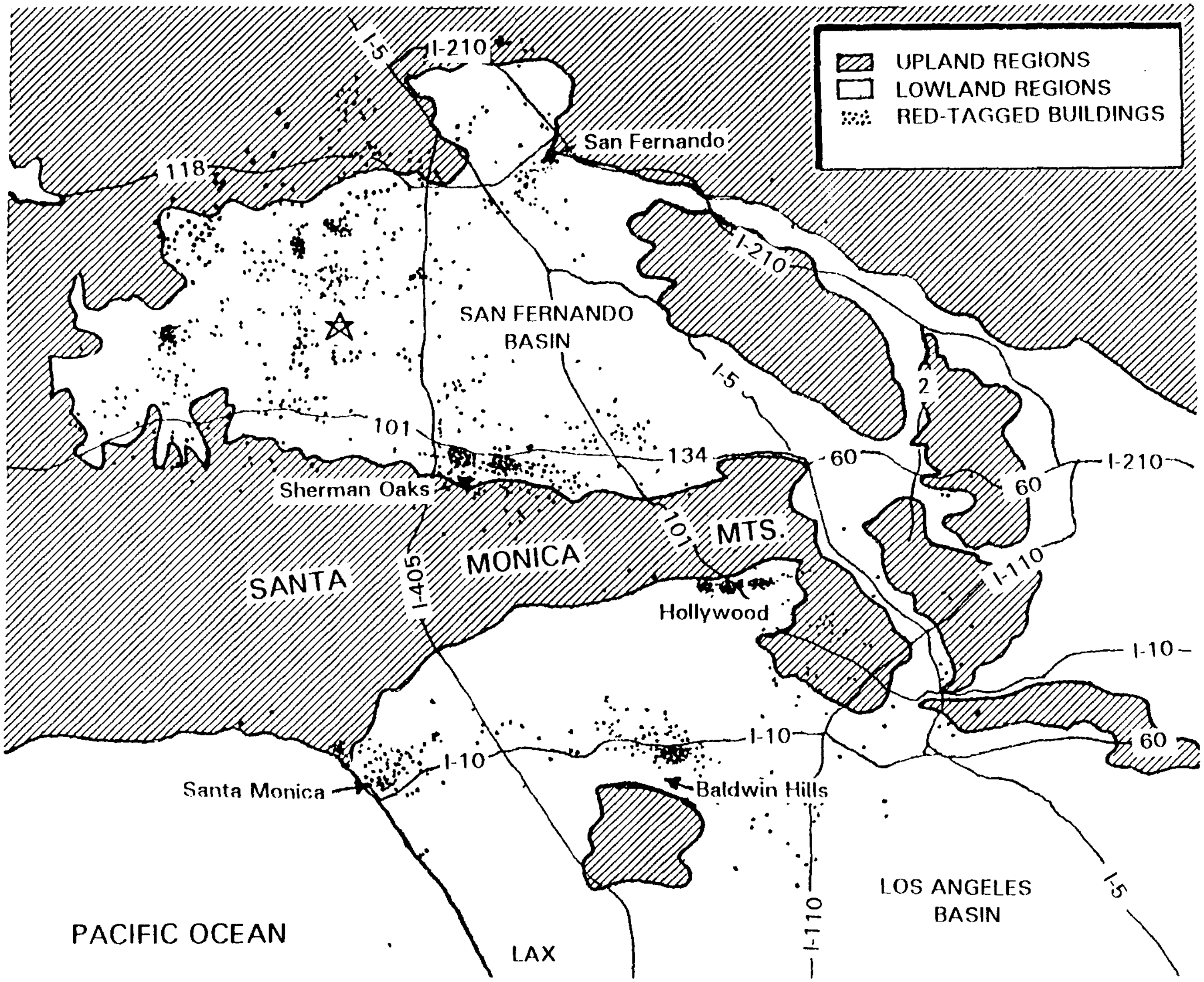


Figure 12 Concentration of red-taggeg buildings near basin fringes in the 1994 Northridge, California earthquake (Modified from EERI, 1994).

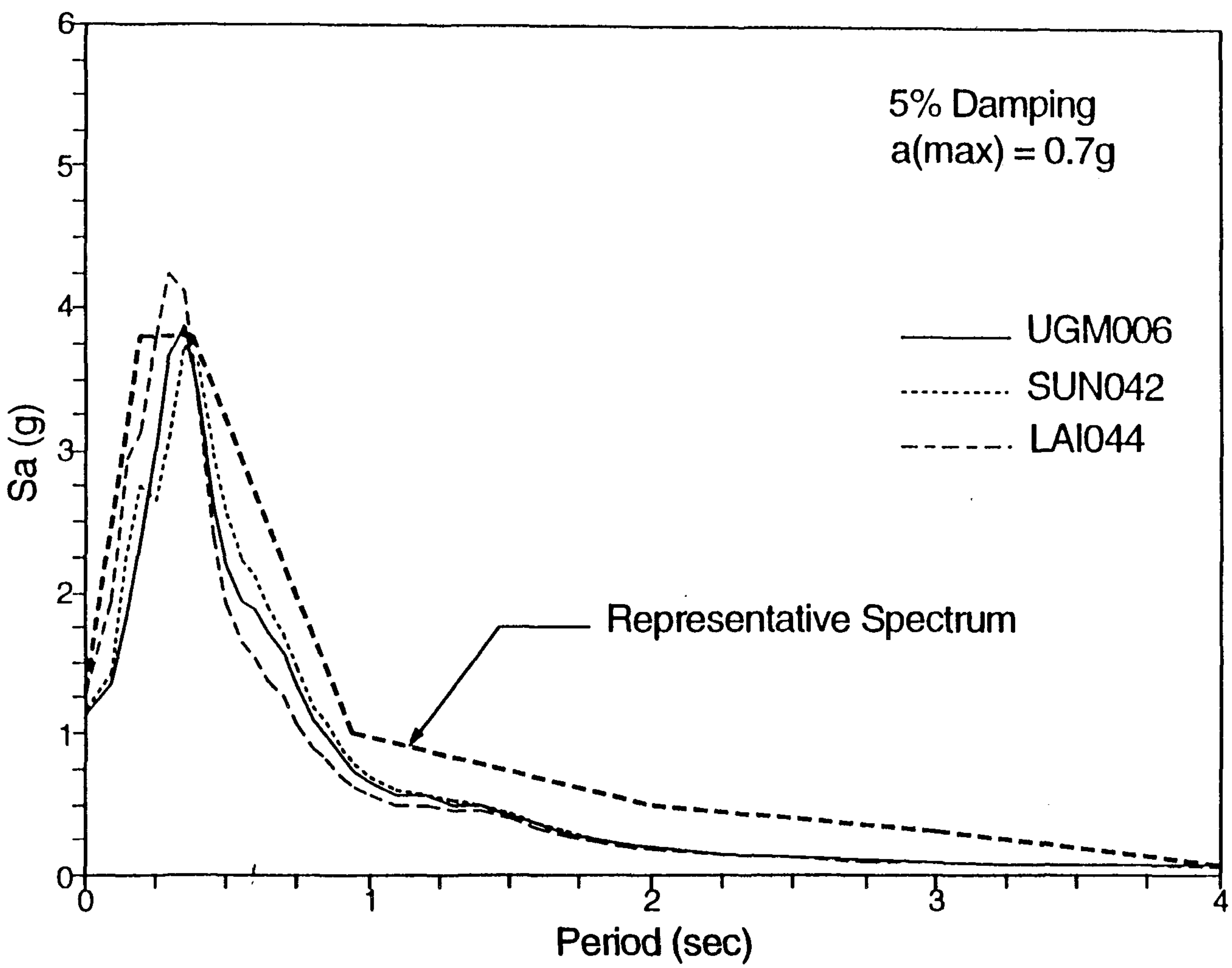


Figure 13 Mean acceleration response spectra and representative spectrum for stiff shallow soil profiles for a peak rock input acceleration of 0.35 g.

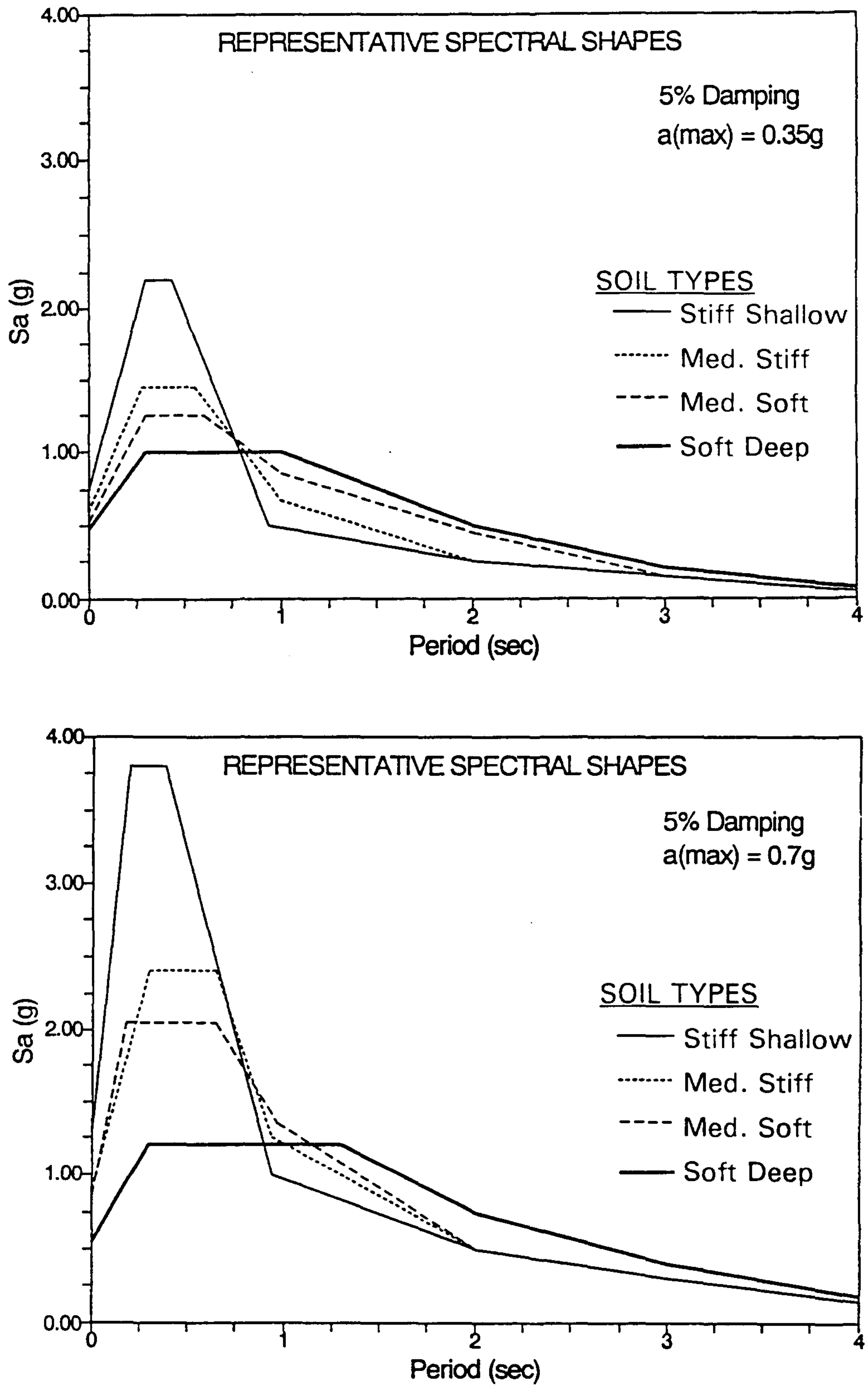
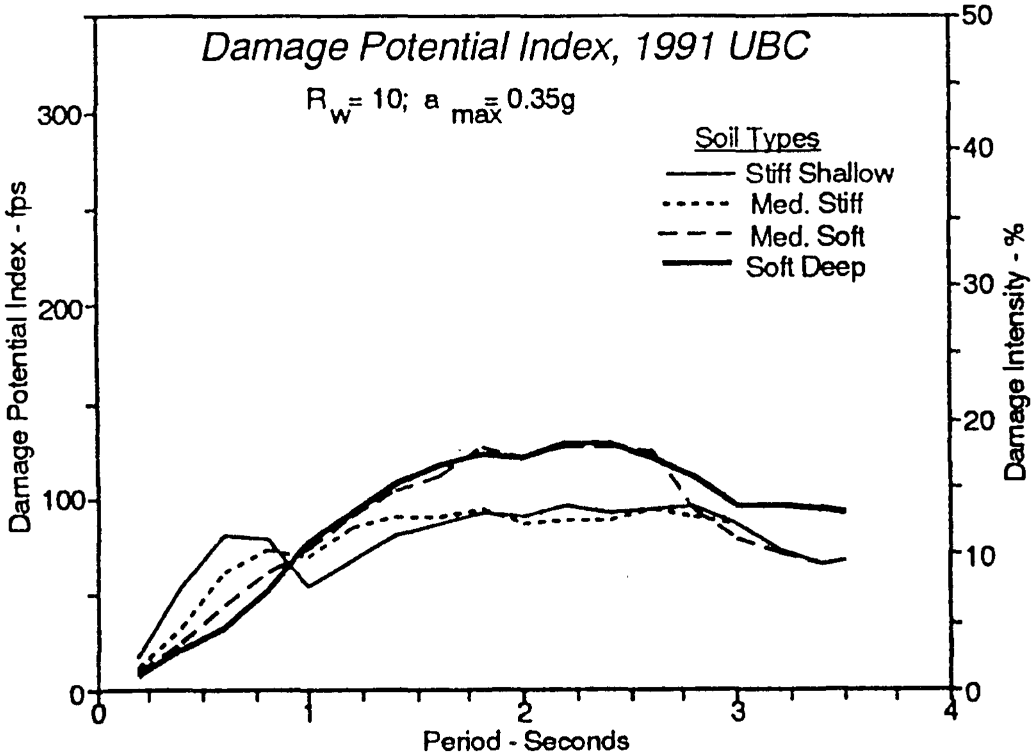


Figure 14 Representative spectra for four amplification zones in Salt Lake Valley for M 7 earthquakes with peak rock input accelerations of 0.35 g (top) and 0.70 g (bottom).



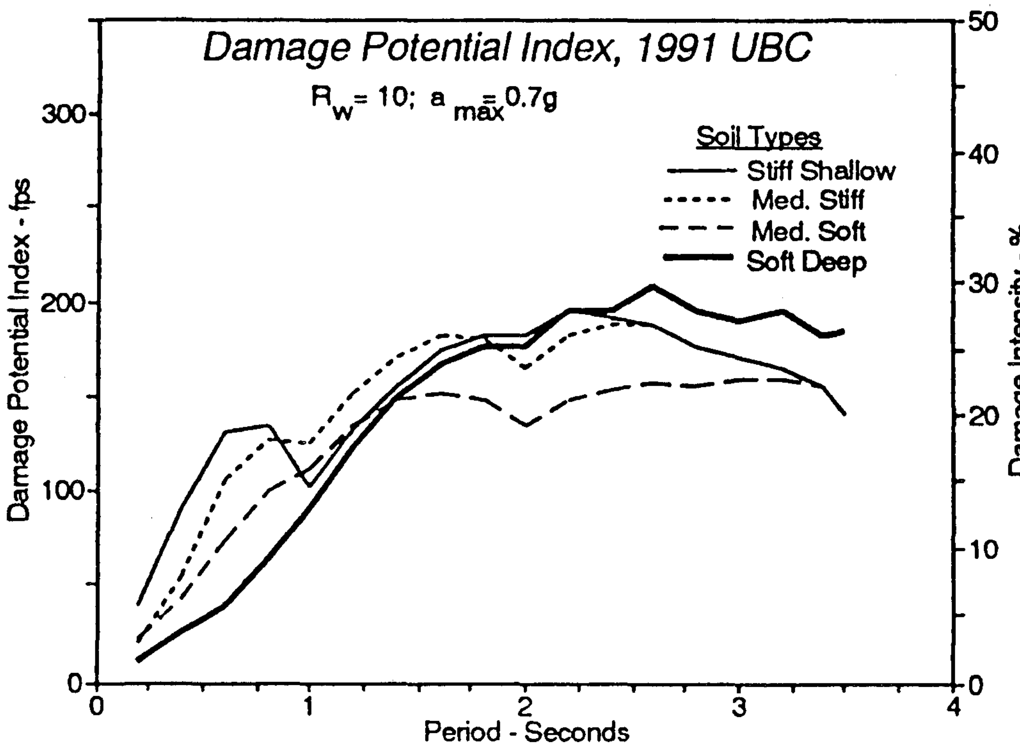


Fig. 15 Comparison of DPI values for Salt Lake Valley soils, for M 7 earthquake with (a) 0.35 g and (b) 0.70 g rock input accelerations.

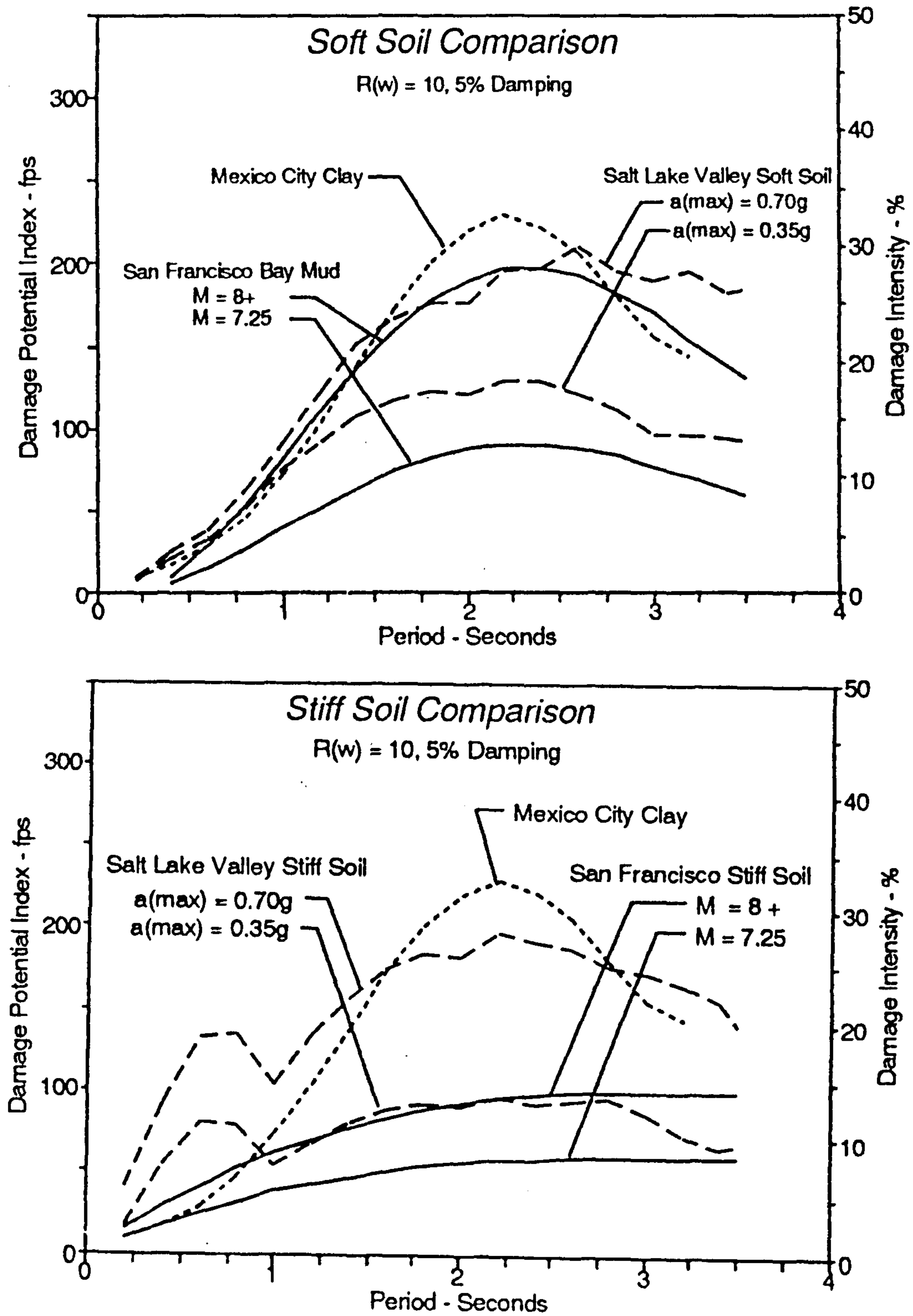


Fig. 16 Comparison of DPI in Salt Lake valley, San Francisco, and Mexico City for (a) soft soils and (b) stiff soils.

4.

Performance Based Seismic Design

by

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PERFORMANCE BASED SEISMIC DESIGN

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I. Introduction

Performance based seismic design refers to a design procedure whereby the design engineer is able to specify and predict, with some assurety, the performance (degree of damage) of a building for a given intensity of earthquake ground shaking. Model seismic building codes, such as the Uniform Building Code (UBC), which governs seismic design practice in most regions of the Western United States, have traditionally focused on providing for life safety and have not explicitly provided a means for designing a structure to meet specific performance criteria relating to the degree of expected damage. Over the last two decades the need for performance based design has begun to be recognized, and the codes have begun to change in that direction. Following the damaging 1971 San Fernando, California, earthquake, for example, model seismic codes began to move incrementally away from a purely life-safety intent to a position of greater emphasis on damage control through drift limitations. In addition, the concept of an "Importance Factor" was developed in recognition of the need to provide an additional margin of safety for critical structures, such as hospitals and emergency response facilities, that should remain functional after an earthquake. To this day, however, the codes have not evolved to the point that explicit means (methods and criteria) to design a structure to meet specific performance criteria have been provided.

As a result of the damaging 1989 Loma Prieta earthquake near San Francisco and 1994 Northridge earthquake near Los Angeles, which caused several billion dollars in damage, the need for performance based design criteria has become critical. It has become readily apparent that:

1. The general public has an expectation that newly designed buildings will perform at a level significantly better than life safety.

2. The economic impact of large scale earthquake damage to local and national economies is too devastating to accept the consequence of current practice.

3. Design professionals must do a better job of communicating to their clients and to the public the likely performance of buildings that they are designing.

4. Building codes must provide specified options for enhanced seismic performance.

In recognition of these concerns, professional and research organizations and government agencies, such as the California Seismic Safety Commission (CSSC), the National Science Foundation (NSF), U.S. Geological Survey (USGS), and Federal Emergency Management Agency (FEMA), have recently begun to develop and sponsor projects that will result in the development and implementation of performance based seismic design.

Such efforts include; a FEMA funded program by the Building Seismic Safety Council (BSSC), Applied Technology Council (ATC), and American Society of Civil Engineers (ASCE) to prepare *Guidelines for the Seismic Rehabilitation of Buildings*; a program by the California Seismic Safety Commission (CSSC) to develop methods for evaluating and rehabilitating the most vulnerable building types in California (Proposition 122 program); a project by ATC, funded by the National Center for Earthquake Engineering (NCEER) and National Science Foundation (NSF), to evaluate current code approaches and develop alternative strategies for a new generation of performance based seismic design methods (ATC-34 project); an effort by the Structural Engineers Association of California (SEAOC) to develop a framework for performance based design (Vision 2000 project); and a FEMA funded project by the Earthquake Engineering Research Center, University of California at Berkeley (EERC), to develop *An Action Plan for Future Studies on the Seismic Design of Buildings.*

The current leader and most influential effort in the development of performance based seismic design methods is the FEMA funded effort to prepare *Guidelines for the Seismic Rehabilitation of Buildings.* This five-year, $8-million effort, to be completed in September 1997, will provide nationally applicable Guidelines and Commentary that could be implemented on a voluntary basis and will enable engineers to design rehabilitation schemes to meet various performance goals. The Technical Guidelines are being prepared by the Applied Technology Council (ATC-33 project) and will undergo a consensus review by the Building Seismic Safety Council as well as reviews in two ASCE conducted Users Workshops. The remainder of this paper focuses on the various key technical issues pertaining to the performance based seismic design criteria and methods under consideration in the ATC-33 project. These include: performance goal definitions, ground motion specifications, rehabilitation objectives, damage states and related acceptance criteria, uncertainties in the construction process, analysis techniques, and soil bearing pressure issues.

II. Performance Goals

Preceding the current project to prepare *Seismic Rehabilitation Guidelines for Buildings*, FEMA funded a related Phase I effort to identify and resolve the issues that should be considered by the Guidelines writers (ATC-28 project). During their deliberations on the concept and need for varying performance goals, the ATC-28 project participants concluded that a variety of performance goals should be covered by the Guidelines. The rationale for including various performance goals included:

- The cost of providing seismic resistance in existing buildings is high, and often the cost increases rapidly with increasing performance criteria. Owners who expend funds to improve the seismic resistance of their buildings have a right to know what performance improvement will result.

- A set of requirements for Life Safety would always be specified and would form the core of the Guidelines; Life Safety may not be achievable however, or the owner may desire a higher level of performance (e.g., damage control).

- Strengthening may be mandated, creating political and legal pressures to clearly present the justification for the regulations and the underlying performance goals, which may vary from community to community, or from one kind of building to another.

- It may be advantageous to make mandated programs incremental, requiring a definition of acceptable "partial" strengthening and the expected performance.

- The strengthening may be voluntary, in which case the owner normally has already identified a specific performance goal (as opposed to "meeting the code") or may want to maximize benefit/cost relationships.

Ultimately, it was recommended that three performance goal levels and two performance goal ranges be included in the Guidelines. The performance goal levels, in order of decreasing expected damage, are: Collapse Prevention, Life Safety, and Immediate Occupancy. The performance goal ranges are: Limited Safety (a range less than Life Safety), and Damage Control (a range greater than Life Safety). These relationships are shown in Table 1. The current (preliminary) definitions and performance expectations for these levels and ranges are as follows:

Performance Levels:

Collapse Prevention. Collapse Prevention is that limiting post-earthquake damage state in which the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system; possible permanent lateral deformation of the structure; and to a more limited extent, degradation in vertical load carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity load demands. Although the building retains its overall stability, significant risk of injury due to falling hazards and similar damage may exist. The structure is probably not safe for re-occupancy as aftershock activity could induce collapse, and may not be economically feasible to repair.

Life Safety. Life Safety is that post-earthquake damage state in which significant damage to the structure may have occurred, but some margin against either total or partial structural collapse remains. The level of damage is lower than that for Collapse Prevention. Major structural and non-structural components should not have become dislodged and fallen, threatening Life Safety either within or outside the building. While injuries during the earthquake may occur, it is expected that overall, the risk of life threatening injury is very low. It should be possible to repair the structure; however, for economic reasons, this may not be practical.

Immediate Occupancy. Immediate Occupancy is that post-earthquake damage state in which only very limited damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake characteristics and capacities. Nonstructural damage is minimized such that basic access and Life Safety systems including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems remain operable, if power is available. There could be window breakage and slight damage to some light fixtures. It is expected that occupants could safely remain in the building; however, minor clean-up and inspection could be required. Repairs may be required, but need not be implemented prior to re-occupying the building. The risk of life threatening injury is negligible.

Performance Ranges

Damage Control. Damage Control is that continuous range of performance states which occur at lower damage levels than that defined for the Life Safety level. The expected performance may range from practically no damage to the level corresponding approximately to Life Safety. Damage Control may cover contents, structural and nonstructural elements, and may be driven by expected repair time, and operation interruption. Owners and designers often judge permissible damage levels in terms of repairability and cost of repair, especially in protecting investments. The extent of protection afforded to nonstructural components, including critical business equipment and contents, is often an element of performance levels within the Damage Control range.

Limited Safety. Limited Safety is that range of performance states which occur at more severe damage levels than that defined for the Life Safety level. This range may include varying amounts of protection against falling hazards. Anchoring of walls to diaphragms or bracing of parapets without consideration of overall structural stability levels represent design for performance within the Limited Safety range.

III. Ground Motion Specification

Performance goal definitions and expectations, as described above, provide only in part the needed criteria for performance based design. The second set of critical criteria are seismic hazard specifications defined in terms of ground shaking for a given return period. Seismic hazard specifications have been defined traditionally in existing codes as those having a 10% probability of exceedance in 50 years, which corresponds to a 475-year return period. This return period has been considered generally appropriate, for design purposes, in highly seismic zones such as California where the time frame for near maximum expected characteristic events is on the same order. Evidence gathered during the last decade or so on recurrence times for near maximum expected characteristic events for other regions of the country suggest that the 475-year return period may be inappropriate (for design purposes) for those regions, such as the Wasatch front, the New Madrid area, and much of the eastern U.S., where there is a large disparity between maximum expected characteristic event compared to maximum expected ground shaking for the largest event likely to occur within 475 years. For such regions, it has been proposed that return periods of 2500 years are more likely to yield maximum ground motions on the same order as those for the near maximum expected characteristic event. In any of these areas it is not known where the geologic clock is relative to the next major earthquake. It is therefore prudent to assume that it may occur in the near future.

IV. Design Objectives

The combination of a performance level with a seismic hazard specification provides a precise definition for what can be termed the "design objective." In the ATC-33 project, which is concerned with rehabilitation of existing buildings, "design objective" is referred to as "rehabilitation objective." For the ATC-33 project the following "rehabilitation objectives" have been defined:

- Standard Safety Objective. Life safety for ground shaking with a 10% probability of exceedance in 50 years and Collapse Prevention for ground shaking with a 10% probability of exceedance in 250 years, which corresponds to a 2500-year return period.

- Limited Objectives. Rehabilitation objectives less stringent than the Standard Safety Objective, such as performance levels less than Life Safety (Limited Safety) for ground shaking with a 10% probability of exceedance in 50 years, or Life Safety for a ground shaking with a higher than 10% (e.g., 50%) probability of exceedance in 50 years.

- **Enhanced Objectives**. Rehabilitation objectives that call for a higher level of performance than the Standard Safety Objective, such as Immediate Occupancy, or Damage Control, for ground shaking with a 10% probability of exceedance in 50 years, or Life Safety for ground shaking with less than 10% (e.g., 5%) probability of exceedance in 50 years and Collapse Prevention for ground shaking with less than 10% probability of exceedance in 250 years.

These rehabilitation objectives, with their corresponding earthquake return periods and performance levels, are shown in Figure 1.

V. Limit States

The performance levels as previously discussed were related to generalized descriptions of the condition of the building. These relationships are given in Table 1. Tables 2, 3, and 4 provide the necessary detail to relate the three performance levels to the damage that would be expected for various structural elements. As would be expected, the greater the expectation of good performance the less damage that is being accepted. It is important to note that immediate occupancy does not allow for significant damage to structural elements. This implies that the structure must respond in essentially an elastic manner with very limited yielding of any joints or major cracking of shear walls. This suggests that the system R-factors will be lower than the current code requirements for buildings when better seismic performance is desired. These tables are a combination of damage state limits and serviceability limit states for the structure as a whole. These tables are taken directly from the 50% draft of the guidelines document and should be expected to be changed during the balance of the project.

VI. Uncertainties in the Construction Process

When designing and constructing any new structure there exists a certain amount of uncertainty as to whether (1) the materials that make up the structure meet the specifications for the project, (2) the design equations adequately cover the specific phenomenon, and (3) the resistance of the member is as calculated by the nominal equation in combination with the various load effects. In the design of new construction, these uncertainties are expressed in the building codes thru the use of strength reduction factors, for concrete design and by using resistance factors, for steel design. Similar factors are being proposed for masonry and wood design. For concrete, the factors range from 0.90 (flexure) to as low as 0.60 (shear) in regions of high seismic risk. This wide range of values indicates that there is considerable uncertainty in the entire process for new construction.

When contemplating the seismic rehabilitation of an existing building, the uncertainties are greatly increased. This is due to the lack of knowledge that generally exists concerning the in-place materials, the details, and the construction techniques that were employed to originally create the structure. This lack of knowledge ranges from having essentially no information to having detailed drawings and construction records (including test information) of the project. Each piece of information that is missing adds to the uncertainties that exist under the best of conditions.

The only way to overcome these additional uncertainties is to employee an extensive field investigation program. Through field investigation the building size, configuration, material properties, etc. can be better understood and quantified. Many times the specifics cannot be adequately determined even if extensive investigations are done. Historical buildings are especially difficult due to their age and due to the restrictions on disturbing the historic fabric. As a consequence, the structural value of the existing systems must be discounted to compensate for these uncertainties.

The guide-lines project for the seismic Rehabilitation of Buildings (ATC-33) has approached this problem by introducing a new factor into the design process. This factor is termed a kappa factor (k). The k factor is to adjust for the degree of confidence in the information that is known. This factor could range from a low value of 0.40 up to a value of 1.0.

These uncertainties along with the possibilities of the deterioration of original systems must be quantified if performanced based seismic rehabilitation is to be effectively done. Economics dictates that the full resistance capacity of the existing structural system be taken into account. Because of the difficulties involved with and the costs associated with intensive field investigations many times the existing systems are ignored and complete new structural systems are added. This approach is very reliable but not necessarily economical.

Performance based design for seismic loads as applied to existing buildings will require an extensive amount of research before it can be applied to the full range of performance objectives. Reference 4 is an extensive treatment of this subject. The greater the expectation of building performance, the more the uncertainties must be reduced.

New building design and construction that is to be based on higher expectations of performance will generally require that the following three things be implemented if the desired results are to be achieved:

1. Better control on building configuration. The Building Codes define specific vertical and horizontal structural irregularities. Each of these structural configuration characteristics can adversely effect the structural performance. Irregularities such as soft story problems, in-plane discontinuities, and torsional irregularity, to name a few, are so significant that their presence will undoubtedly compromise the performance.

2. Better control on building design. The process of design review must be improved if high levels of performance are desired. Independent review by qualified individuals is imperative to achieve the desired results. Independent, objective critique will always improve a design.

3. Better control on building construction. The construction process must be better monitored. Construction observations and special inspections are mandatory to insure better performance. The best set of construction documents must still be interpreted correctly to be built according to the intent of the designer. The construction phase is the final point at which the check can be made.

These requirements are obviously important in the seismic rehabilitation process as well.

VII. Analysis Techniques

The concept of seismic design has evolved from a few simple rules of procedure to a complex set of somewhat prescriptive requirements, that if followed, will lead to acceptable performance of the structure as defined by the current codes That is, the structure will probably not collapse. When contemplating performance based design that could specify higher performance levels, it is necessary to evaluate the entire process of design.

The analysis portion of the design process has, for the most part, been based on static equivalent forces used in linear models of the structure. In reality the problem is one of dynamic forces acting on structures that are expected to have some level of non-linear response. The traditional analysis and modeling techniques are acceptable if we continue to have low expectations (performance) of our structural response. Economies may not be achieved. If higher performance goals (design objectives) are intended, better and more sophisticated analysis procedures are required. Possible analysis techniques are:

(A) Linear methods such as the equivalent lateral force procedure or dynamic analysis utilizing response spectrum or time history analysis.

(B) Non-linear methods such as static push-over approximations or non-linear time history approaches.

Linear methods are more applicable to new construction of regular configuration than in rehabilitation projects because the design codes for new buildings impose requirements on configuration, details, and systems that when taken in total should provide for redundant, ductile behavior of the new structure. The issue of redundancy in new construction is somewhat questionable in that there is no place where it is quantified.

When dealing with older buildings there is little assurance that the system will respond in an appropriate ductile manner. therefore, non-linear techniques are needed to account for all important linear and non-linear response characteristics.

Simple force/deformation curves can be utilized to great advantage in push-over type analysis techniques.

Figure 2 is an idealized example of such a curve. Each point where the curve changes slope indicates that some component in the structure has been stressed beyond its yield point and reached a condition where the entire section has plastified. Once this takes place the stiffness of the system is reduced. Each succeeding hinge that is formed reduces the system stiffness until the slope of the force deformation curve becomes negative. Analytical curves such as this allow for the assessment of a structures redundancy, ductility, and overall seismic resistance. Although contemplated for the rehabilitation process, curves of this nature would be extremely valuable when designing new structures.

Non-linear time history analysis is not practical at this time in an office setting, but will become routine as computer software that incorporates materials behavior is developed to handle the different structural systems.

VIII. Soil Bearing Pressures

Performance based seismic design concepts have created an environment in which all elements in the system have been scrutinized. During this process it has become apparent that the geotechnical issues are very important. Traditional soils reports have strived to provide guidance to the engineer that would lead to a very high level of performance during the life of the structure. The performance during an earthquake was not differentiated from any other load condition. Temporary soil bearing pressure allowable values have been taken as 1/3 to 1/2 greater than those values used under more normal conditions. These stress increases are easily justified because of the transitory nature of the loadings. In reality, the deformations under this procedure would not be much different for the various loading conditions. What this means is that the foundation procedures, as practiced, have been more restrictive than for the structure as a whole. Actual post earthquake observations have disclosed very few foundation failures that contributed to structural collapse. Lateral spreading effects due to liquefaction is another issue that must be considered, and certainly can contribute to structural damage and collapse.

To get a consistent design objective, the engineer must take another approach to the problem of allowable soil bearing pressures. If the design objective is to just prevent collapse of the structure, the footing could be allowed to settle many times the amount that is generally acceptable. Many simple buildings could allow as much as six inches of differential movement without compromising the structural stability. Others may only accept half of this amount. This recognition has lead to the formalization of a concept of a strength and stiffness envelope as shown in Figure 3. "The lower bound reflects the initial material properties during the first cycle of loading and the upper bound represents the effects of repeated loading" (Ref. 1). With each cycle of loading the stiffness of the soil will actually increase, and the overall deformation will be cumulative with each cycle.

The concept of designing the foundation of a structure to have relatively large deformation is foreign to most engineers. But the issue is dramatically brought into consideration when designing a building as part of a seismic rehabilitation process. The cost to add additional capacity to building foundations can be extremely expensive. If one chooses a design objective of preventing collapse, then it is possible using these concepts to minimize the foundation work. If a higher expectation is desired, the engineer may be forced to implement expensive foundation measures to minimize differential movement of the foundation system. This is an important breakthrough in thinking when considering allowable soil bearing pressures.

IX. Summary

Although the current practice of designing structures to not collapse during strong ground motion has served the nation for many years, it is clearly the time to develop procedures that can produce higher levels of performance with some assurety. The highly urbanized nature and complexity of our society demands improvement of our codes and design procedures.

Design objectives which are based on a specific performance goal coupled with a specific level of ground shaking have been defined. These definitions recognize the probabilistic nature of the seismic hazard. Three specific performance goals are, collapse prevention, life safety, and immediate occupancy. Performance levels below life safety are in the limited safety range. For life safety and above the performance levels are in the damage control range. Many owners and communities are desirous of having some of their facilities perform above the collapse prevention level.

The different performance levels are defined by variations in structural damage, general damage and egress capability. To be able to predict the variations in these factors according to different ground motions is a very formidable task. Formidable as it is, there is a driving societal need for the methodology and techniques to be developed and perfected. There are great uncertainties in the construction process. There are similarly great uncertainties in the analysis techniques that are used. Geotechnical factors such as appropriate soil bearing pressures, lateral spreading and liquefaction, and the general phenomenon of ground amplification of earthquake motion need to be better understood and quantified.

These issues which are being confronted in the Guidelines for the Seismic Rehabilitation of Buildings project must be addressed and resolved to be able to improve the performance of buildings subject to seismic loading in an economical way.

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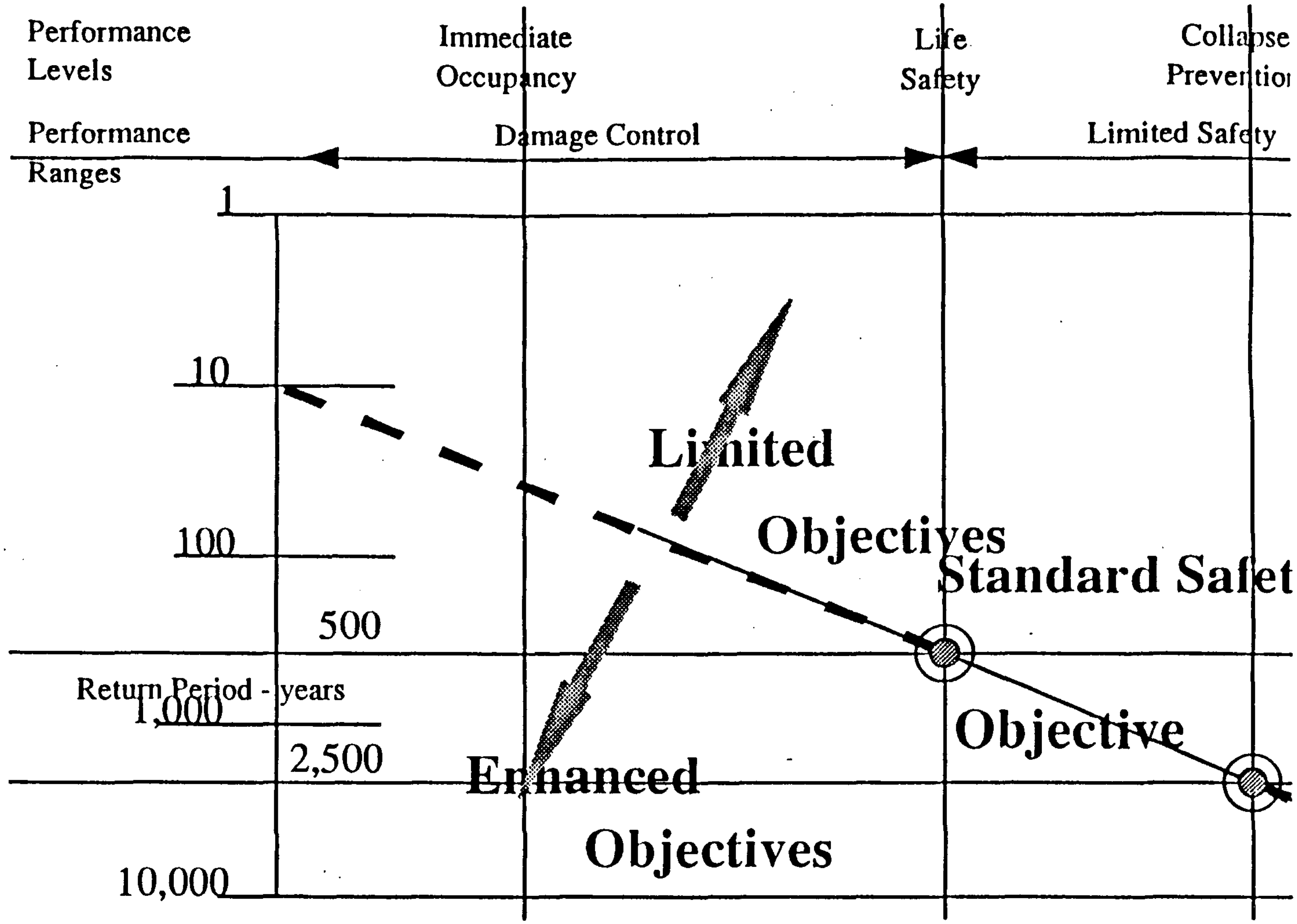


Figure 1 Rehabilitation Objectives

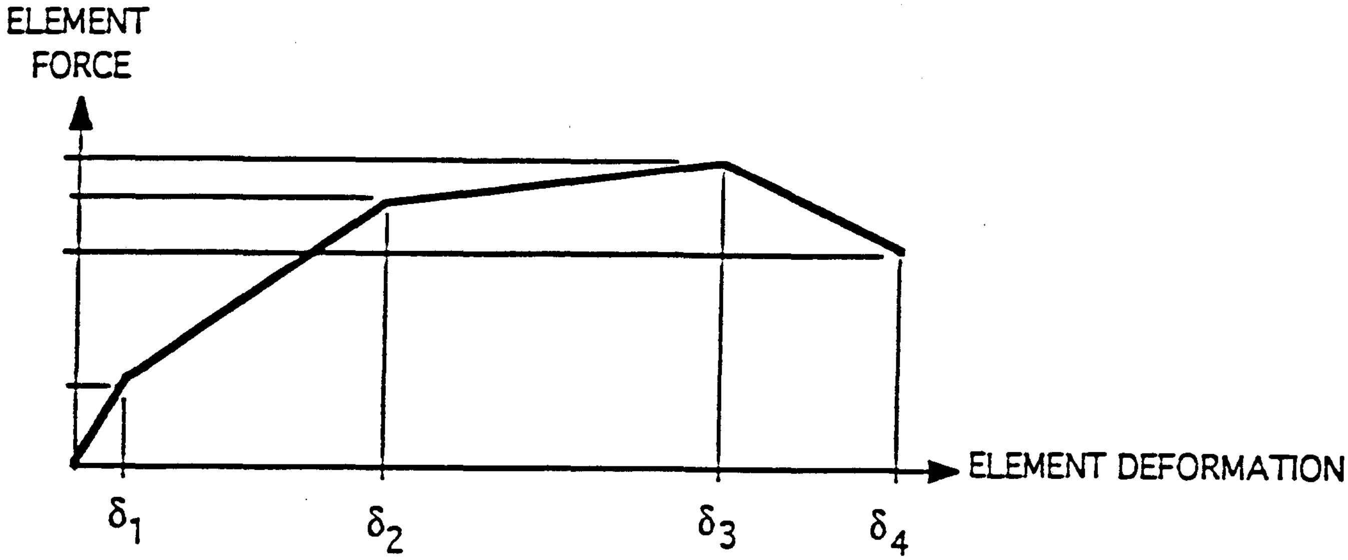
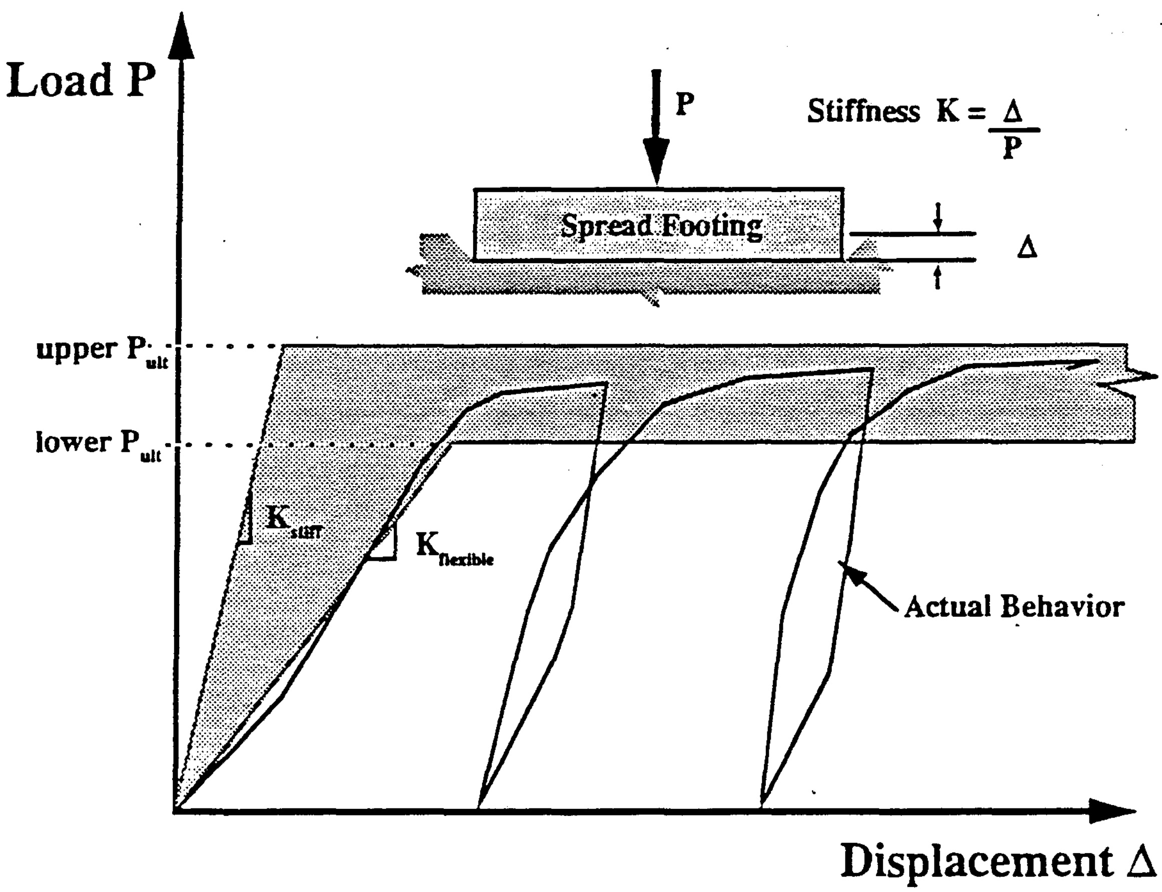


Figure 2



Load - Displacement Relationship for a Spread Footing

Figure 3 (Ref. 1)

Table 1 Damage Description and Performance Levels (ATC, 1994)

Performance Ranges	———Limited Safety——— Performance Range				———Damage Control——— Performance Range		
Performance Levels		Collapse Prevention Performance Level		Life Safety Performance Level		Immediate Occupancy Level	
Overall Structural Damage		Severe		Moderate		Light	
General		Small residual stiffness and strength but load bearing columns and walls function; no story collapse mechanisms but, large transient and permanent drifts; some exits blocked; infills and unbraced parapets failed or at incipient failure;		Some residual strength and stiffness left in all stories, gravity load bearing elements function; no out-of-plane failure of walls or tipping of parapets; some permanent drift; damage to partitions; building may be beyond economical repair		No permanent drift; structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, ceilings as well as structural elements. Elevators can be restarted. Fire protection operable	
Nonstructural Components		Not included		Falling hazards should not occur		Functional or property protection beyond prevention of falling hazards	
Comparisons with Performance Expected of NEHRP Buildings in 500 Year Event		Significantly more damage and greater risk		Somewhat more damage and slightly higher risk		Much less damage and lower risk	

Table 2 Performance Levels and Structural Damage - Vertical Elements

Elements	Type	Performance Level		
		Collapse Prevention	Life Safety	Immediate Occupancy
Concrete Frames	General		Maximum transient drift less than 2%	
	Primary	GD[1] elements: Extensive cracking and hinge formation; PD[2] elements: limited cracking and/or splice failure in some columns. Severe damage in short columns.	Extensive damage to beams; Spelling of cover and shear cracking for GD[1] columns. Minor spalling in PD columns. Joints cracked <1/8 inch thickness.	Minor structural cracking; limited yielding possible at a few locations, no crushing (strains below 0.003)
	Secondary	same as primary	GD[1] elements: Extensive cracking and hinge formation; PD[2] elements: limited cracking and/or splice failure in some columns. Short columns - severe damage	Minor spalling in a few places in GD columns and beams. Flexural cracking in beams, columns; shear cracking in joints < 1/16 inch width.
Steel Moment Frames	General	Large story drifts (transient and permanent); large p-delta amplification and residual lateral deflection	Max. transient drift about 2%, most recovered	
	Primary	Extensive distortion of beams and column panels	Hinges form; local buckling of some beam elements; severe joint distortion	Minor local yielding at a few places
	Secondary	Fracture of some flanges & welds	Local buckling of column panel zones	Minor buckling in a few places
Braced Steel Frames	General	Transient drift about 1.5%	Transient drift about 1%	Essentially linear response
	Primary	Extensive concentric brace yielding	Many braces yield or buckle but do not totally fail	Some minor yielding and buckling permissible
	Secondary	Beams on chevron pattern frames yield and permanently distort	Some braces and/or connections completely fail	Brace yields or buckles but does not fail
Concrete Walls	General	Shear Drifts reach 0.8%	Maximum shear drift about 0.5%	
	Primary	Major flexural and shear cracks; sliding at joints; extensive crushing; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated	Some boundary element distress including limited bar buckling; Some sliding at joints; damage around openings, some crushing and flexural cracking Coupling beams - extensive shear and flexural cracks; some crushing, but concrete generally remains in place	Minor wall cracking; cracks limited to about 1/16" width; Coupling beams - moderate cracking and minor spalling
	Secondary	Panels shattered virtually disintegrated	Major flexural and shear cracks; sliding at joints; extensive crushing; failure around openings; severe boundary element damage; Coupling beams shattered, virtually disintegrated	Some sliding at joints, damage around openings; Coupling beams - extensive shear and flexural cracks, some crushing

Notes:

1. GD refers to systems with good detaining, like that required by the NEHRP provisions for special concrete moment resisting frames
2. PD refers to systems with poor detailing, like that classified by the NEHRP provisions as ordinary concrete moment resisting frames

Table 3 Performance Levels and Structural Damage - Vertical Elements

Elements	Type	Performance Level		
		Collapse Prevention	Life Safety	Immediate Occupancy
Unreinforced Masonry Infill Walls[3]	General	Drifts reach 1.5%	0.8% maximum drift	
	Primary	Extensive cracking and crushing; portions of face course shed	Extensive cracking and some crushing but wall remains in place, no falling units	Minor cracking at opening and across panels
	Secondary	Extensive crushing and shattering; Some walls dislodge	same as primary	Extensive cracking and soe crushing but wall remains in place; no falling units
URM Bearing Walls	General	Large permanent drift	Permanent shear drift <1%	Negligible permanent displacement
	Primary	Extensive cracking and crushing of masonry; Some fallen units	Extensive cracking, some sliding of units in and our-of-plane, no units dislodged	Minor cracking; no spalling
	Secondary	Panels dislodge	same as primary	Some cracking and sliding of units
Reinforced Masonry Walls	General	Shear drift about 1.0%	Shear drift about 0.5%	
	Primary	Crushing; extensive cracking; damage around openings and at corners; some fallen units	Distributed cracks; minor crushing at corners but no failure	Minor cracks at few places; No spalling
	Secondary	Panels shattered virtually disintegrated	Crushing; extensive cracking; damage around openings and at corners; some fallen units	Distributed cracks; minor crushing at corners but no failure
Wood Stud Walls	General	Large permanent drift	Maximum transient drifts of about 2%; Some permanent drift	
	Primary	Connections loose, nails partially withdrawn, some splitting of members and panel; veneers shear off	Moderate loosening of connections and minor splitting of members	Minor cracking of plasters and veneers but no significant permanent drift or loosening of connections
	Secondary	Failure of plates, studs; sheathing detached	Connections loose, nails partially withdrawn, some splitting of members and panel	Moderate loosening of connections and minor splitting of members
Precast Concrete Connections	General	Components same as concrete walls and frames	Components same as concrete walls and frames	Components same as concrete walls and frames
	Primary	Some connection failures but no elements dislodged	Local crushing and spalling at connections, but no gross failure of connections	Minor working at connections; Cracks < 1/16 inch width at connections
	Secondary	same as primary	Some connection failures but no elements dislodged	Minor crushing and spalling at connections
Foundations	General	Major settlement and tilting	Total settlements < 2 inches and differential settlements < 1 inch	Minor settlements but no observable damage

[3] For limiting damage to frame elements of infilled frames, refer to the rows for concrete or steel frames

Table 4 Performance Levels and Structural Damage - Horizontal elements

System	Performance Levels		
	Collapse Prevention	Life Safety	Immediate Occupancy
Metal Deck Diaphragms	Large distortion with buckling of some units and tearing of many welds and seam attachments	Some localized failure of welded connections of deck to framing and between panels; minor local buckling of deck	Connections between deck units and from deck to framing intact minor distortions
Wood Diaphragms	Large permanent distortion with partial withdrawal of nails and splitting of elements	Some splitting at connections, minor withdrawal of nails	No observable damage
Concrete Diaphragms	Extensive flexural and shear cracking with crushing and local spalling	Some cracking and local crushing	No cracking larger than 1/16 inch
Precast Diaphragms	Connections between units fail, units shift relative to each other; crushing and spalling at joints	Extensive cracking and minor spalling at joints.	Some minor cracking along joints

5.

Earthquake Protective Systems

by

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EARTHQUAKE PROTECTIVE SYSTEMS

BY

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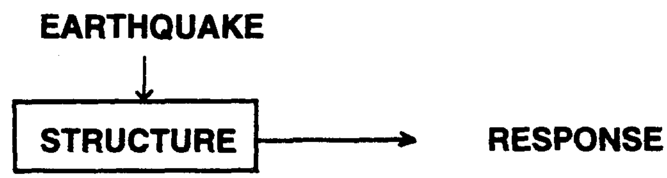
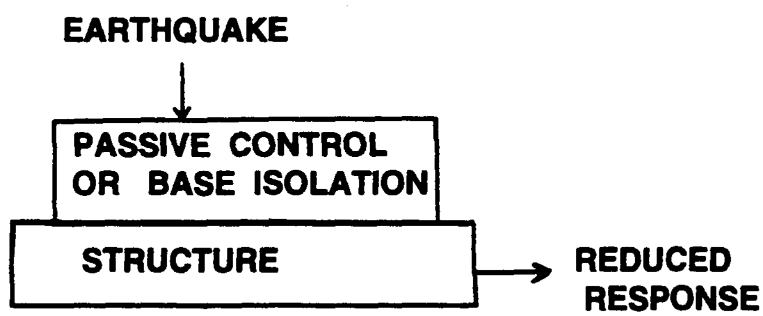
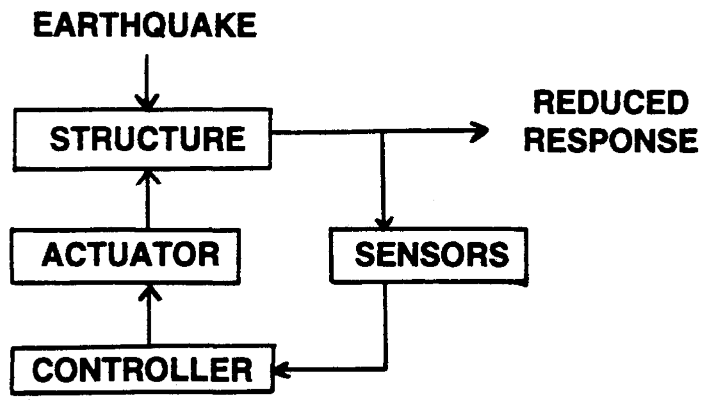


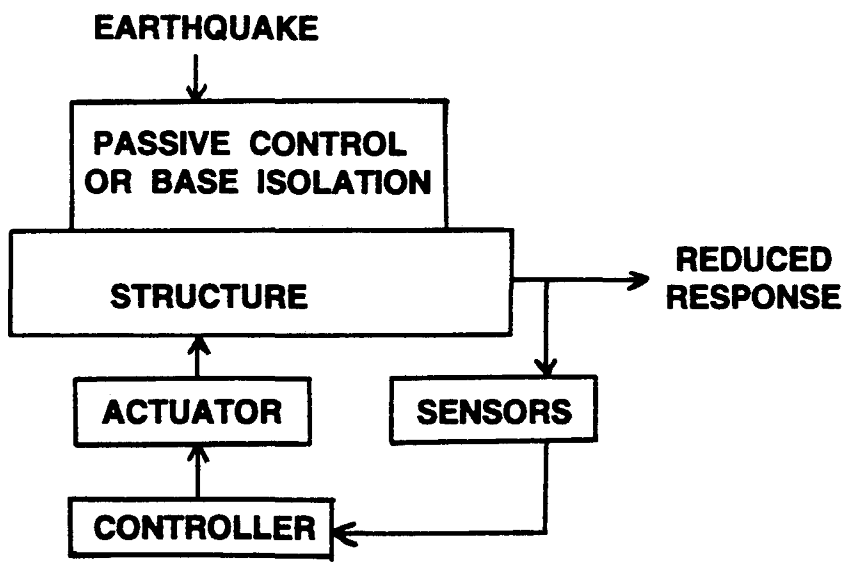
UNCONTROLLED STRUCTURE



PASSIVE CONTROL OR BASE ISOLATION



ACTIVE CONTROL SYSTEM



HYBRID CONTROL SYSTEM

EARTHQUAKE PROTECTIVE SYSTEMS

ISOLATION SYSTEMS	PASSIVE SYSTEMS	ACTIVE SYSTEMS	HYBRID SYSTEMS
ELASTOMERIC BEARINGS	METALLIC DAMPERS	ACTIVE VARIABLE STIFFNESS	SLIDING ACTIVE BEARING
LEAD-RUBBER BEARINGS	FRICTION DAMPERS	ACTIVE BRACING SYSTEM	ISOLATOR DAMPER
FRICTION PENDULUM	FLUID DAMPERS	PULSE SYSTEM	DAMPER & ACTIVE BRACE
HIGH DAMPING RUBBER BEARING	VISCOELASTIC DAMPERS	ACTIVE MASS DAMPER	
	TUNED MASS DAMPERS		
	TUNED LIQUID DAMPERS		

ISOLATION SYSTEMS :
* PLACED AT FOUNDATION OF STRUCTURE
* DETUNE STRUCTURAL SYSTEM FROM FOUNDATION
* REFLECT & ABSORB INPUT ENERGY
* REDUCED INPUT ENERGY TRANSFERRED TO SUPERSTRUCTURE

PASSIVE SYSTEMS :
* ABSORB INPUT ENERGY
* REDUCE DISSIPATION DEMAND ON BASIC STRUCTURAL SYSTEM

ACTIVE SYSTEMS :
* MODIFY MOTION OF BASIC STRUCTURAL SYSTEM THROUGH EXTERNAL POWER
* PROVIDE INSTANTANEOUS MONITORING
* INCREASE EFFECTIVE DAMPING & STIFFNESS OF BASIC STRUCTURE

HYBRID SYSTEMS :
* COMBINATION OF PASSIVE & ACTIVE SYSTEMS
* REDUCED EXTERNAL POWER DEMAND

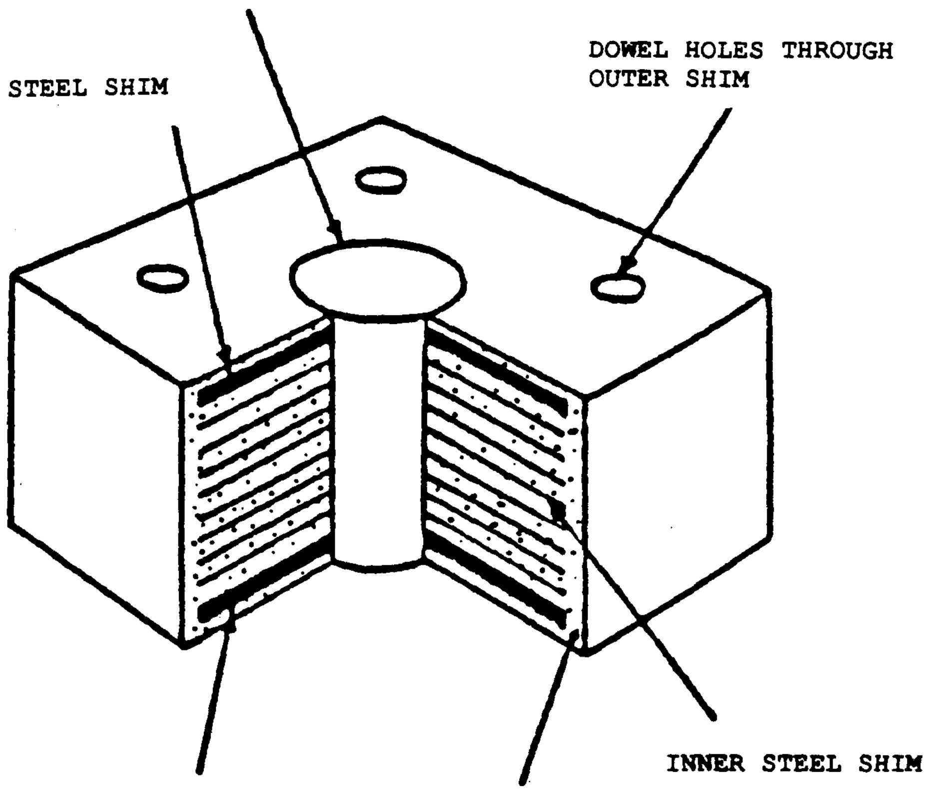
ISOLATION SYSTEMS

ELASTOMERIC BEARINGS

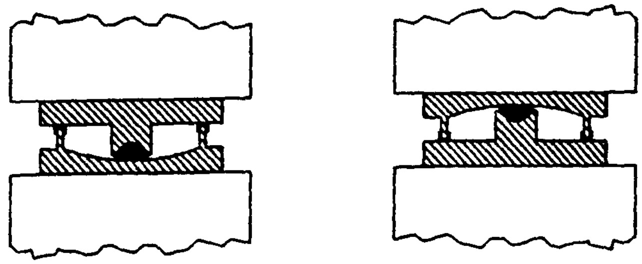
LEAD-RUBBER BEARINGS

FRICTION PENDULUM

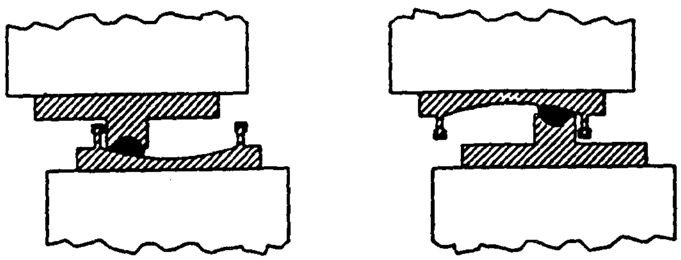
HIGH DAMPING RUBBER BEARING



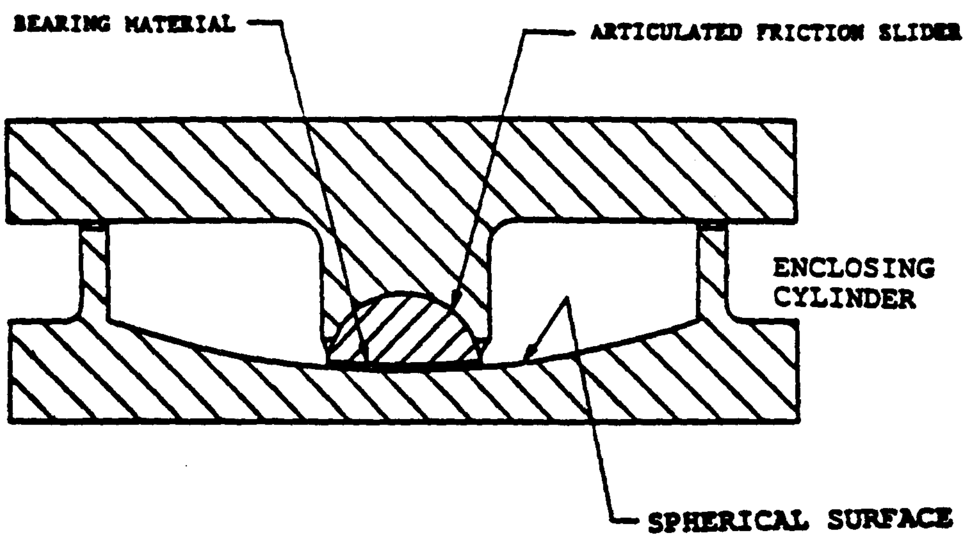
LEAD-RUBBER BEARING



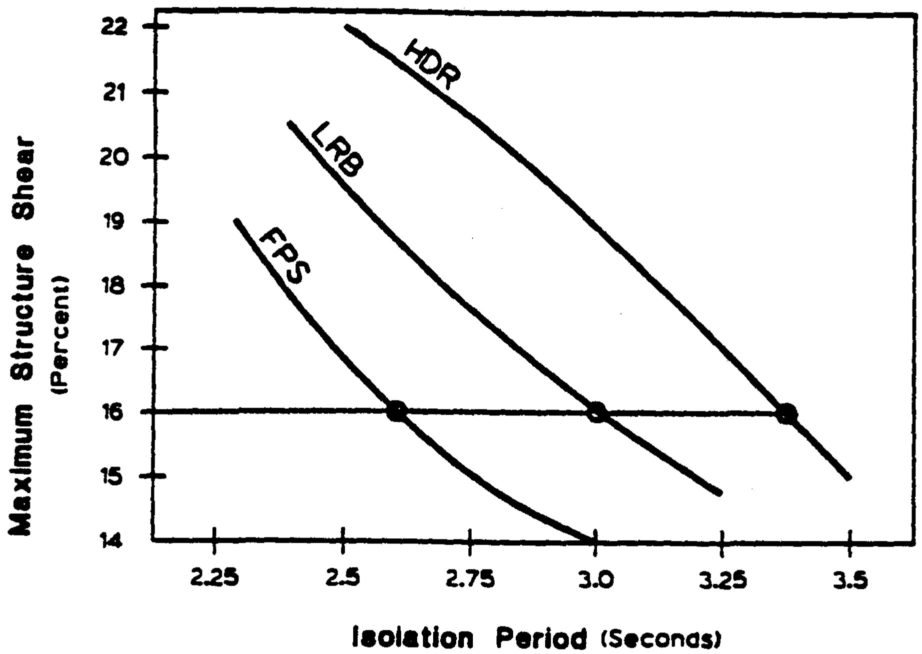
CENTER POSITION



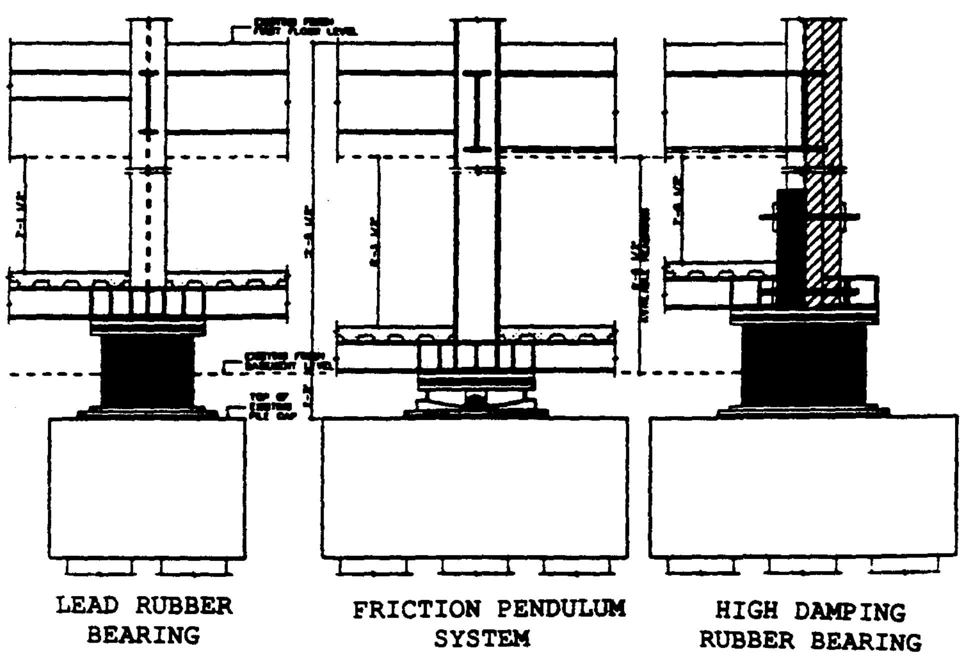
DISPLACED POSITION



CROSS-SECTION VIEW OF FPS BEARING



COMPARISON OF ISOLATION SYSTEMS



ARCHITECTURAL AND CONSTRUCTION ISSUES

EXAMPLES OF ISOLATION SYSTEMS

LEAD-RUBBER BEARINGS:

* SALT LAKE CITY AND COUNTY BUILDING
  170,000 SQ. FT., 1988

* C-I BUILDING, JAPAN
  407,000 SQ. FT., 1988
  SIZE 40-60 IN.
  ELASTOMER STRAIN @ 100% = 10 IN.
  LARGEST BASE ISOLATED IN JAPAN

FRICTION PENDULUM:

* 9TH CIRCUIT COURT OF APPEALS
  SAN FRANCISCO
  5-STORY, 330X265 FT, COMPLETED 1995
  LARGEST BASE ISOLATED IN U.S.

PASSIVE SYSTEMS

METALLIC DAMPERS

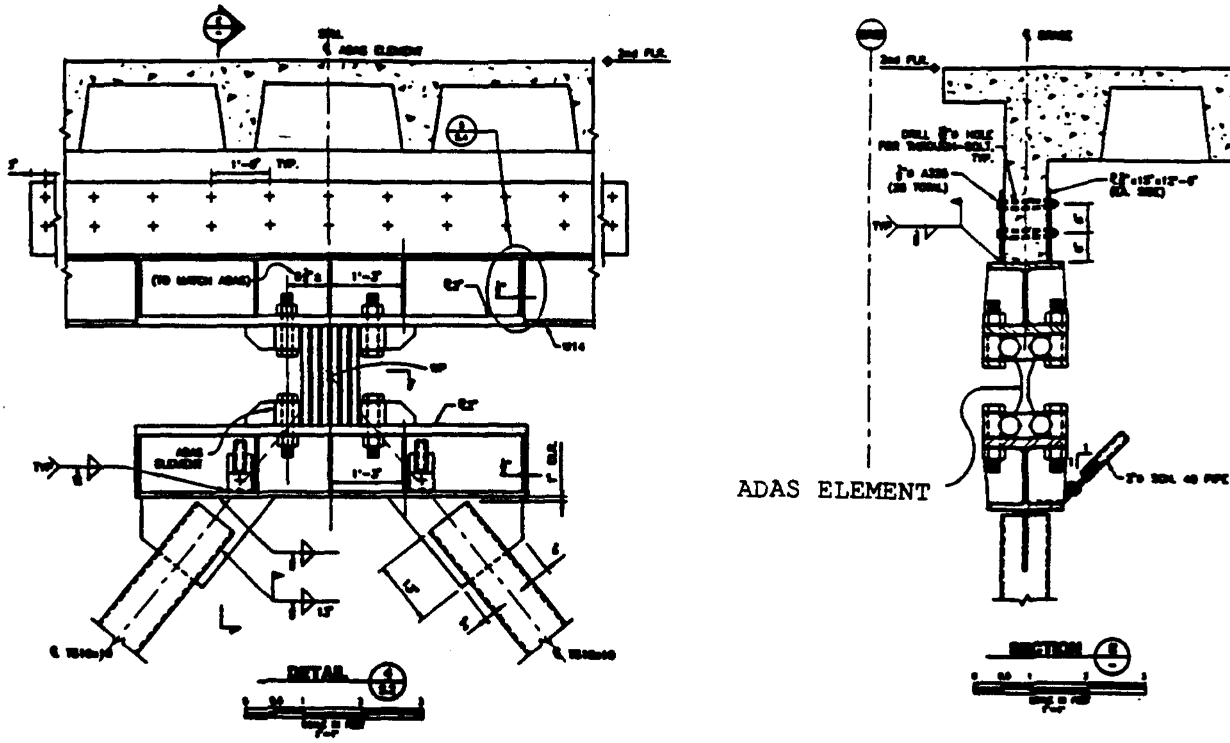
FRICTION DAMPERS

FLUID DAMPERS

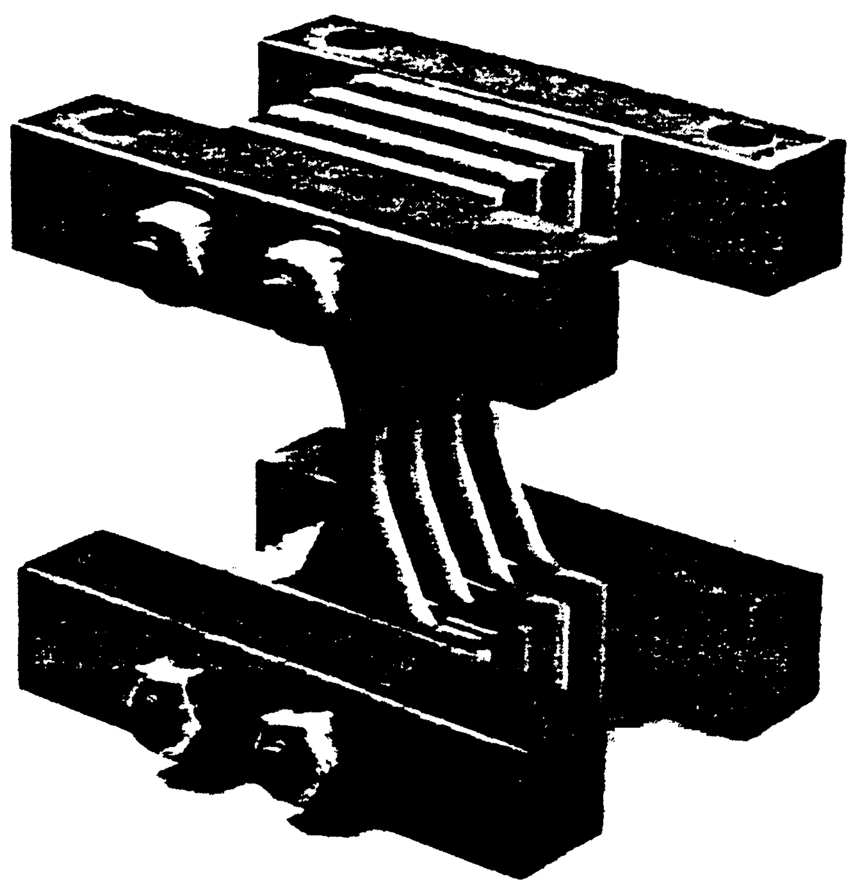
VISCOELASTIC DAMPERS

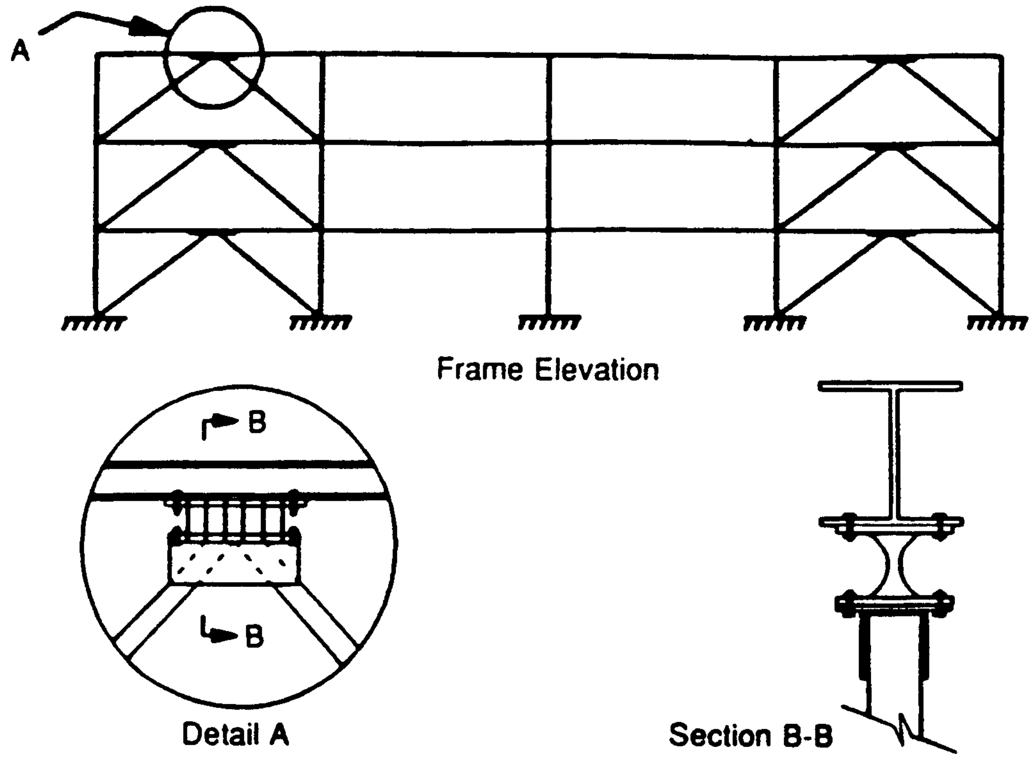
TUNED MASS DAMPERS

TUNED LIQUID DAMPERS

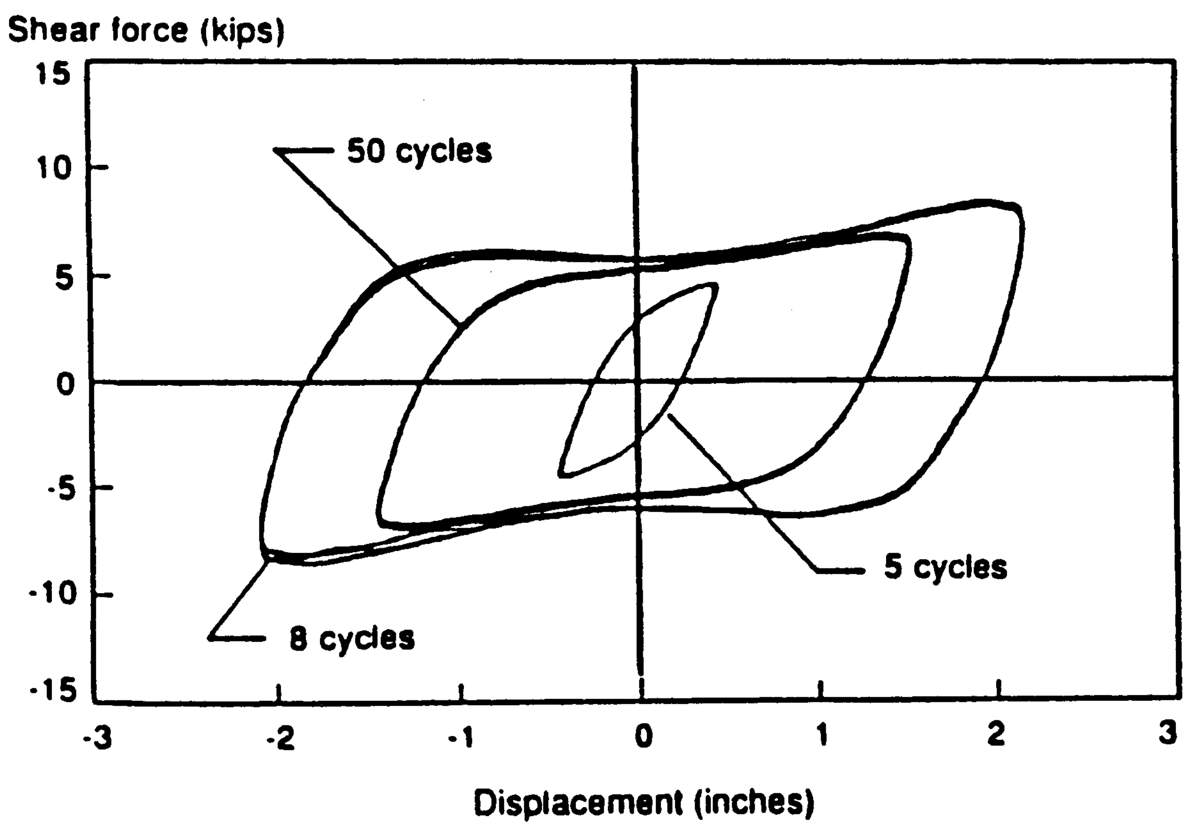


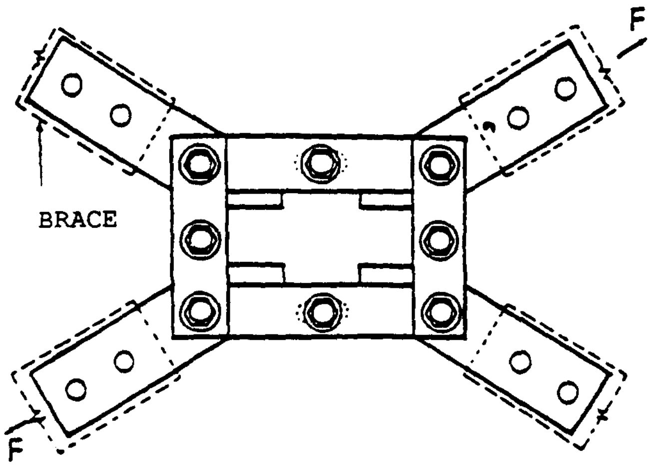
ADAS DETAILS

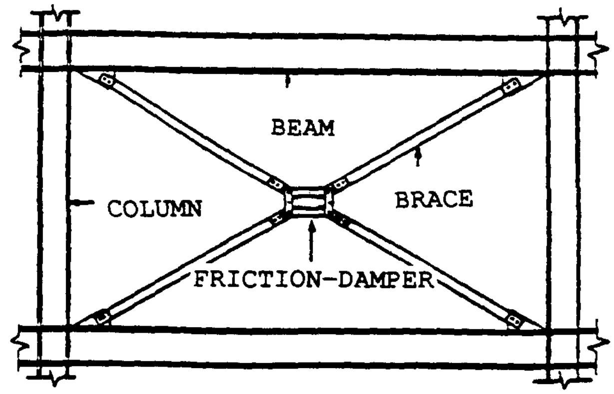




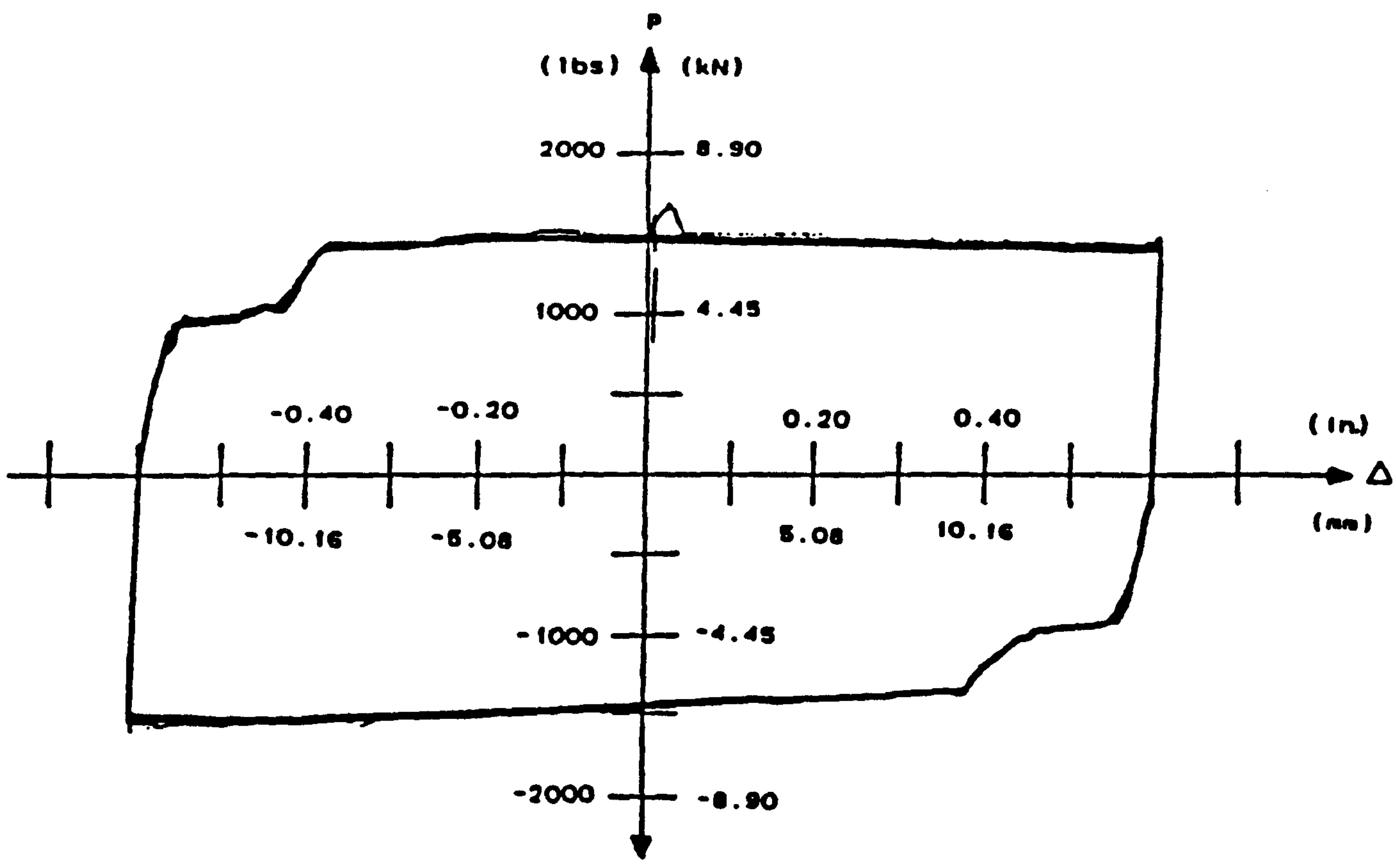
ADAS Element

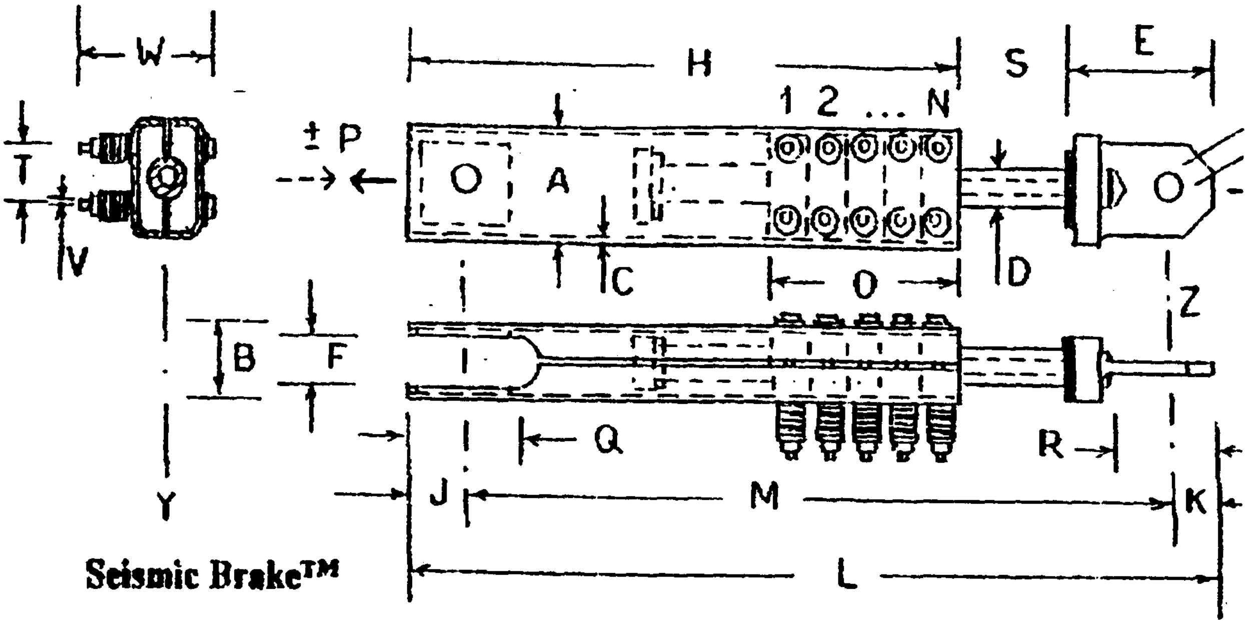




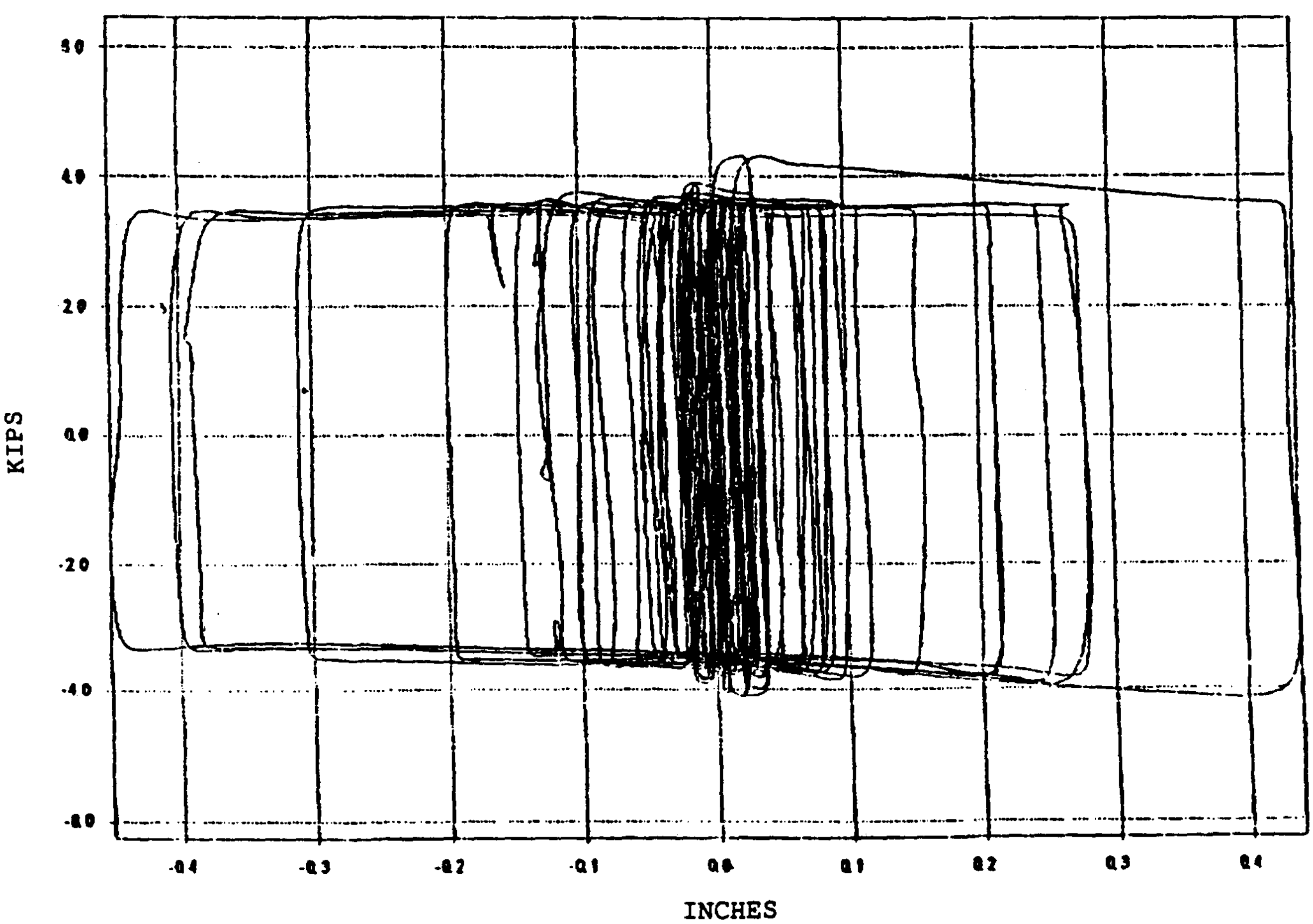


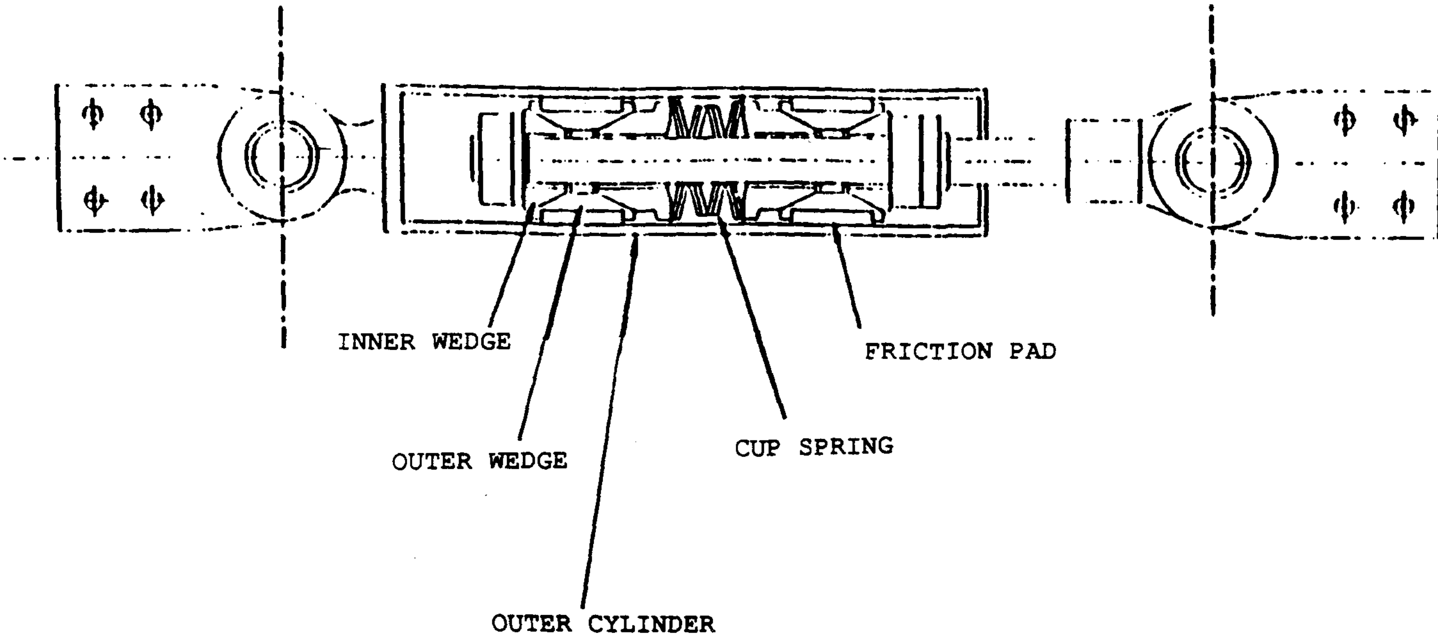
PALL FRICTION DEVICE



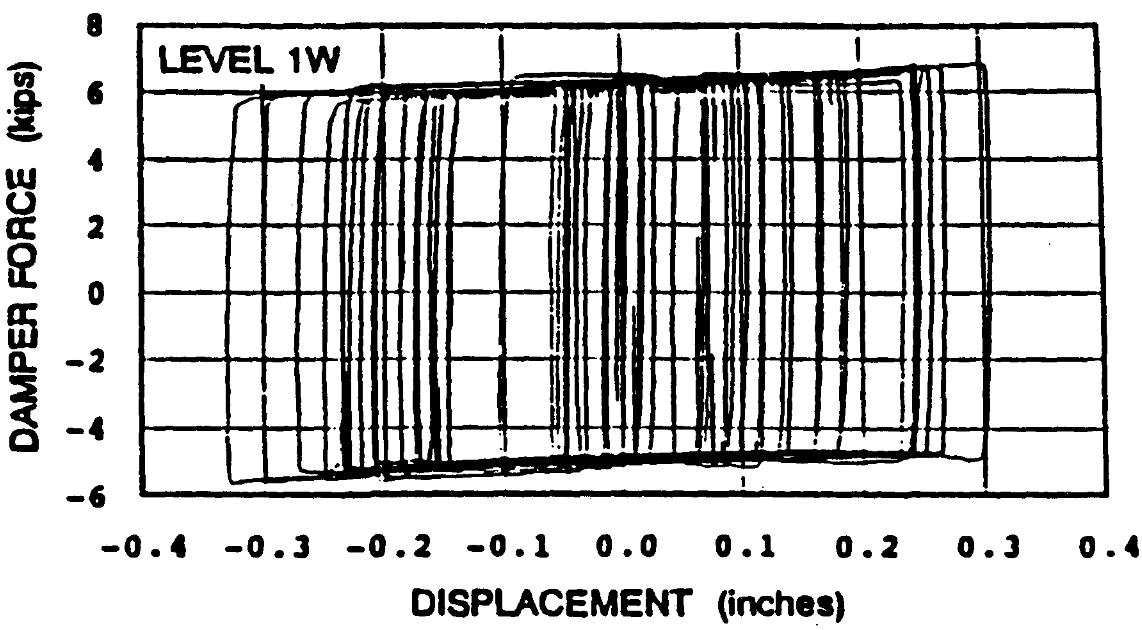


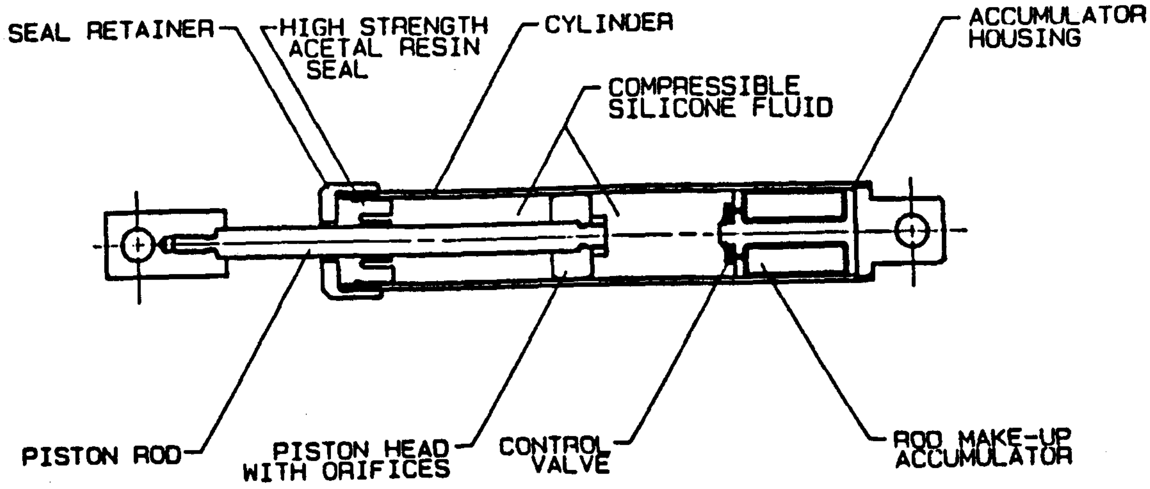
SEISMIC BRAKE



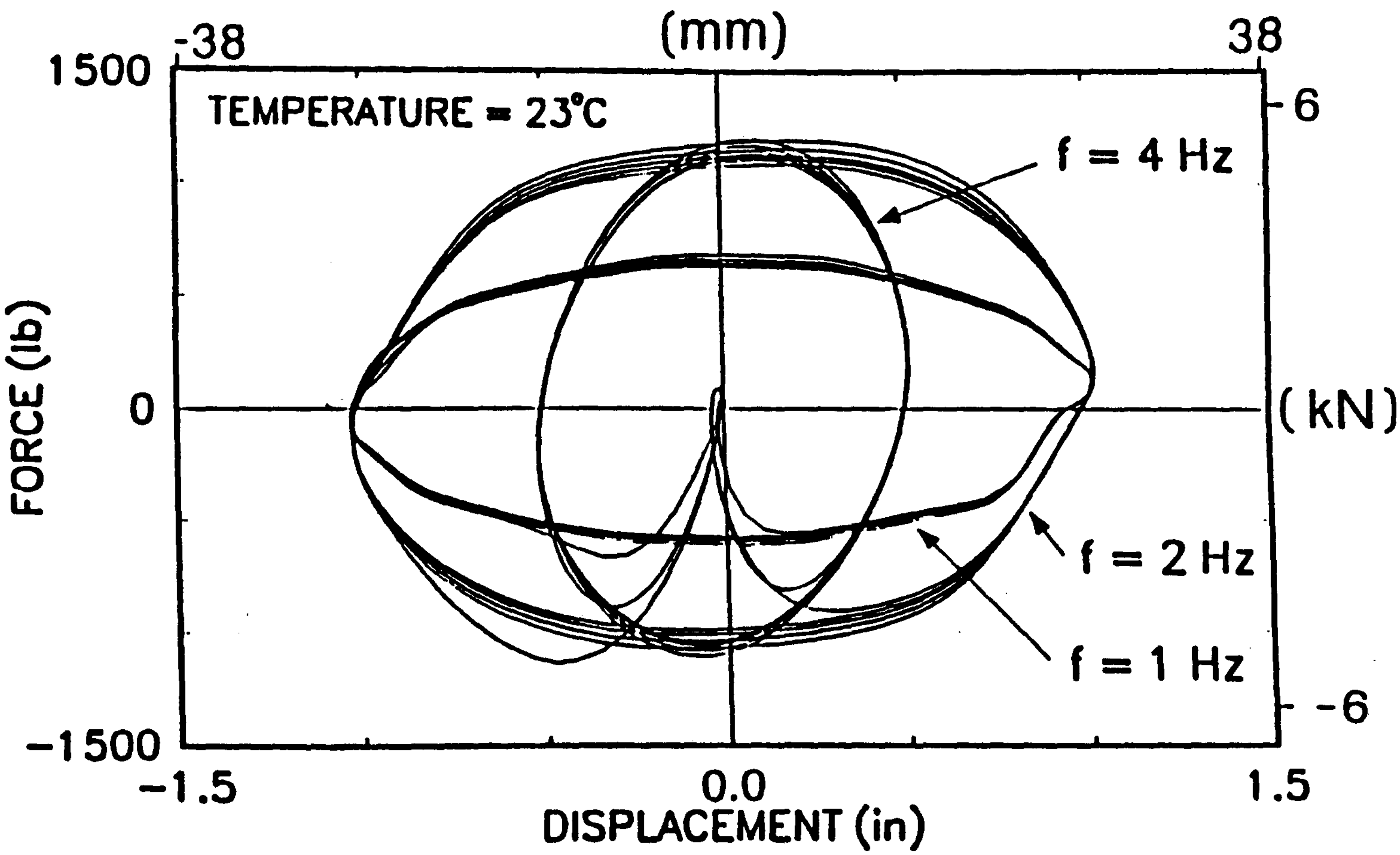


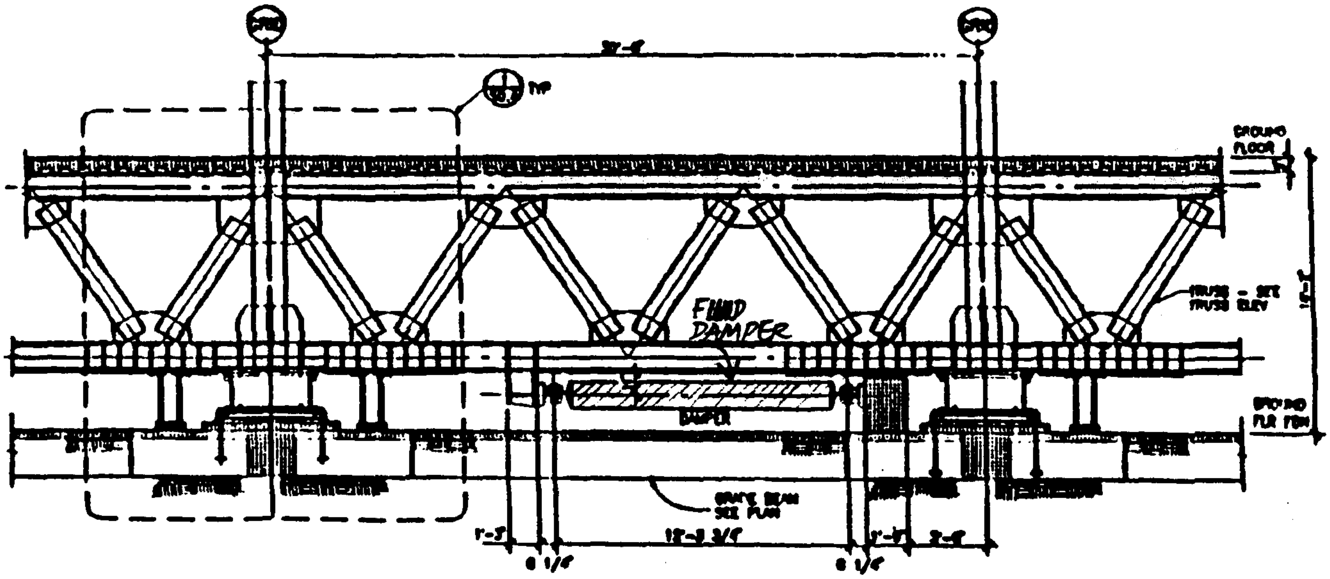
SUKIMOTO FRICTION DEVICE



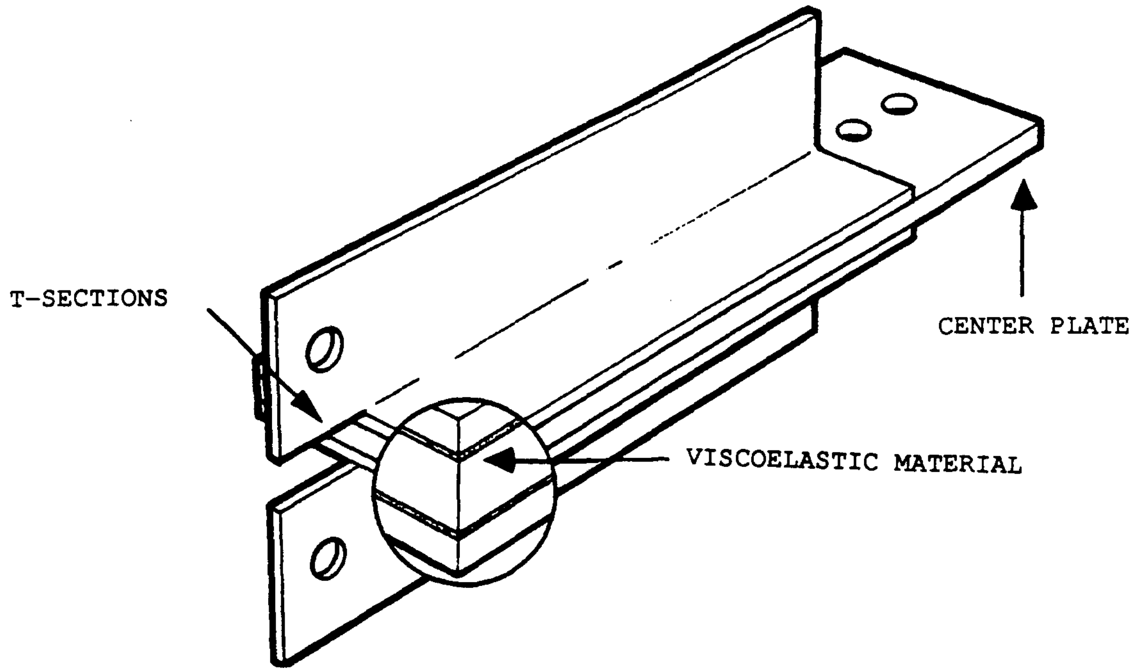


FLUID DAMPER

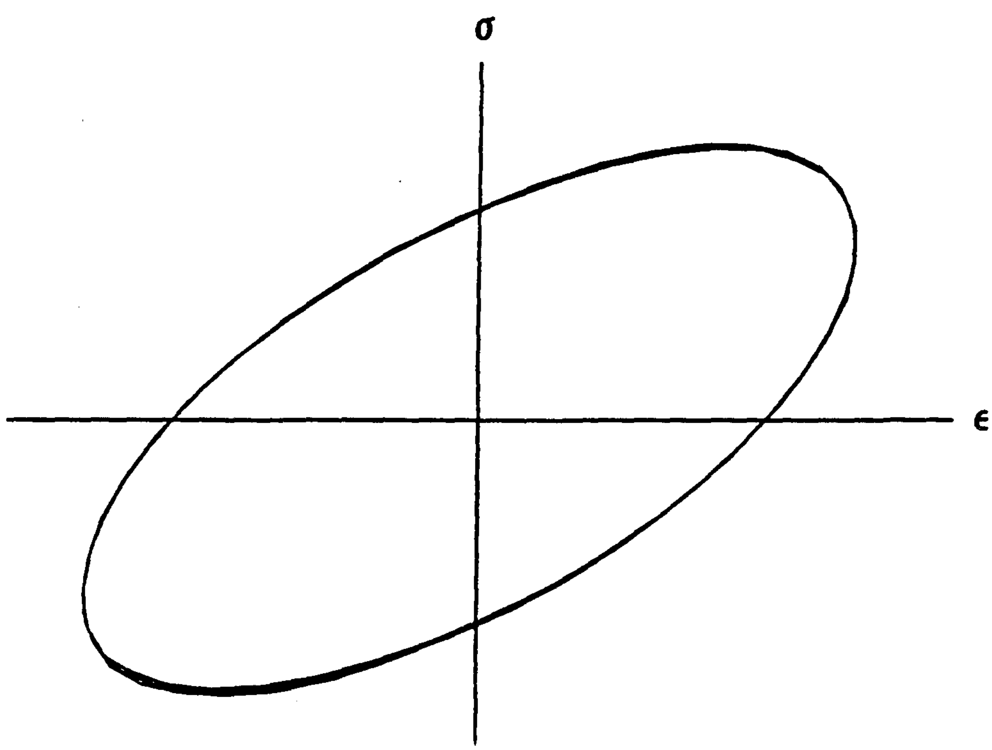


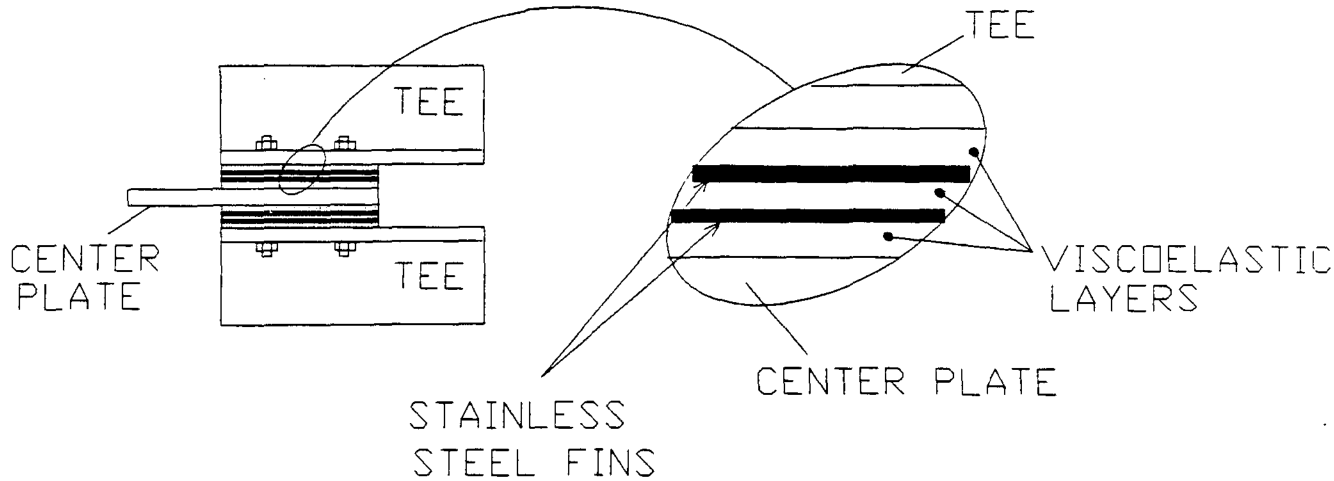


FLUID DAMPER

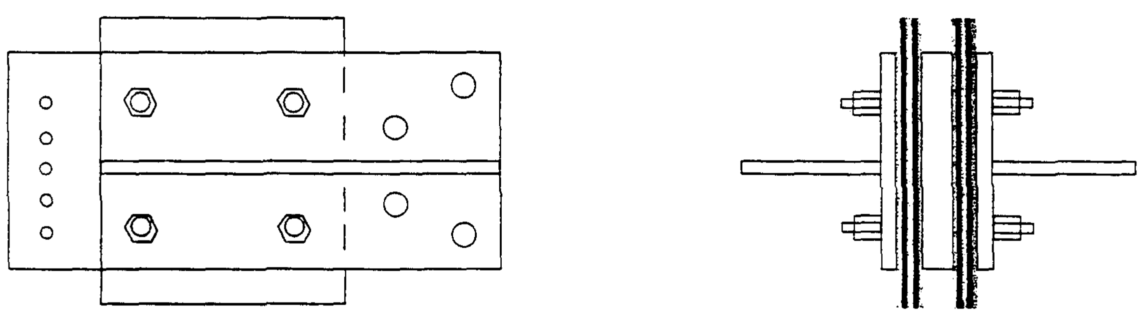


VISCOELASTIC DAMPER

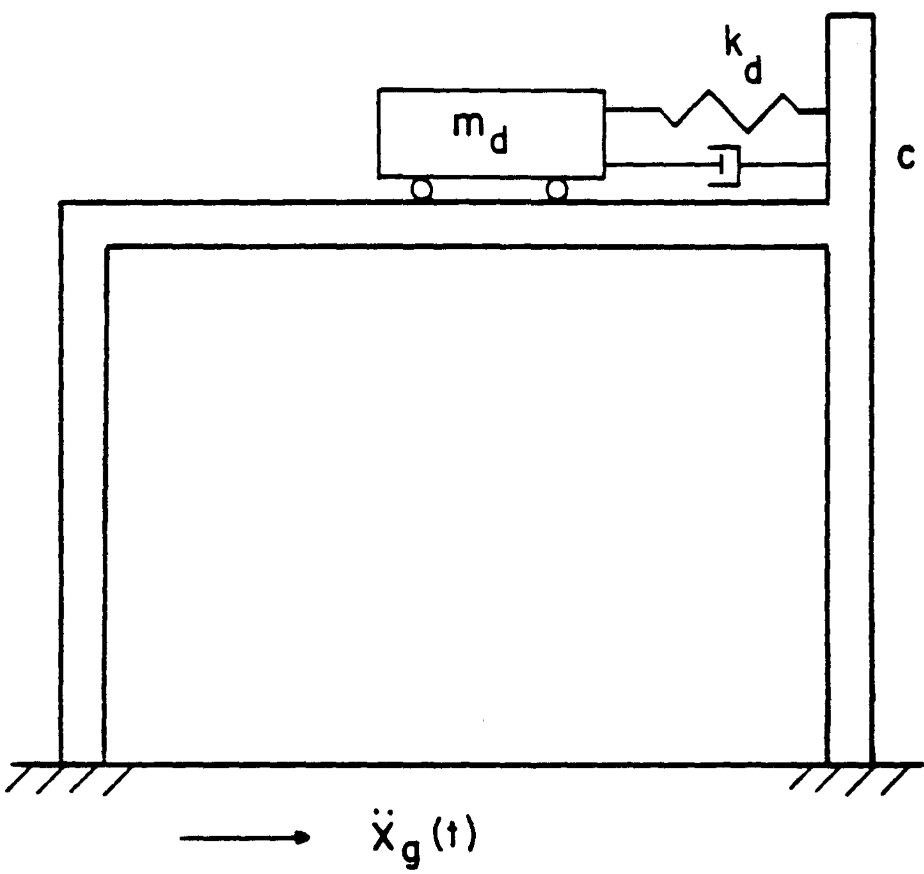




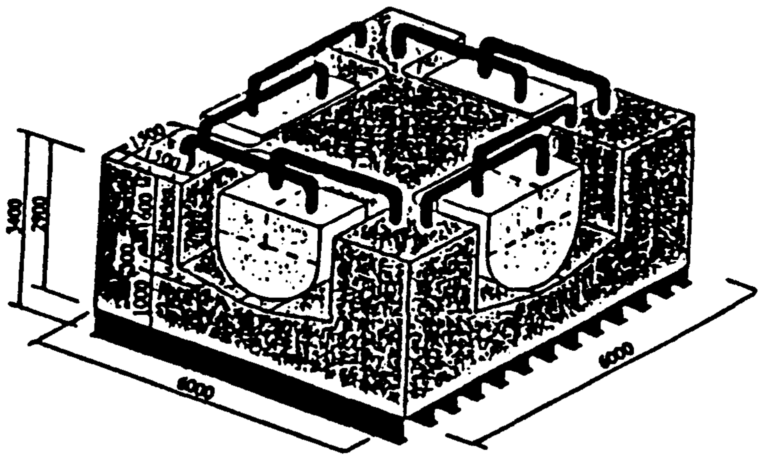
VE DAMPER - ELEVATION



VE DAMPER - PLAN

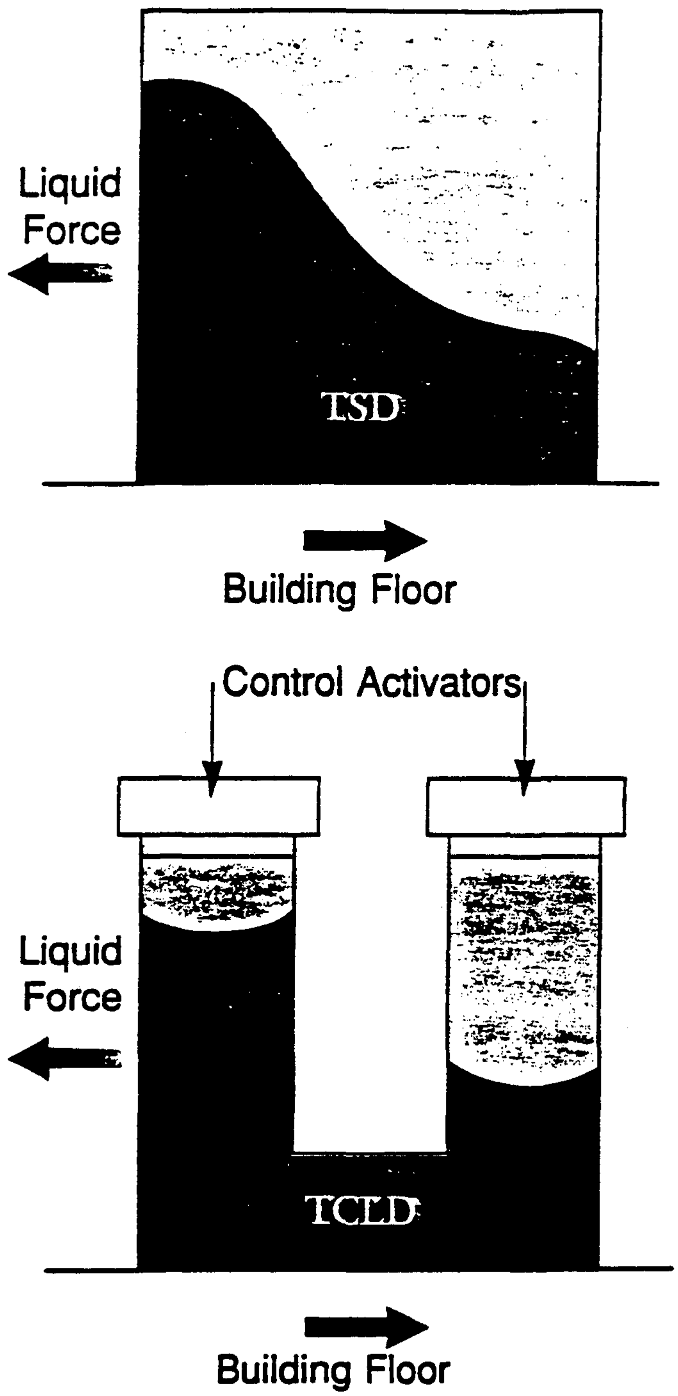


TUNED MASS DAMPER



Size	**6.0m x 6.0m x 2.9m**
Weight	**50.0 ton**
	(water ; 26.5 ton)
mass ratio	**0.8%**

TUNED LIQUID COLUMN DAMPER



TUNED LIQUID COLUMN DAMPER

EXAMPLES OF PASSIVE SYSTEMS

METALLIC DAMPERS : * WELLS FARGO BANK, SAN FRANCISCO
NONDUCTILE CONCRETE FRAME 2-STORY
14,000 SQ. FT.
7 ADAS DESIGN YIELD FORCE 150 KIP

FRICTION DAMPERS : * MCCONNEL BLDG. CONCORDIA UNIVERSITY
MONTREAL, CANADA
R/C FRAMES WITH FLAT SLAB
INTERCONNECTED 10- & 6-STORY BLDGS.
143 PALL FRICTION DAMPERS
SLIP LOADS 160 KIP

FLUID DAMPERS : * SAN BERNARDINO COUNTY MEDICAL
CENTER, COMPLETED IN 1997
6-STORY STRUCTURAL STEEL AND
CONCRETE-FILLED STEEL DECK
FLUID DAMPERS CAN MOVE 22 IN.

* JAPAN AIRLINE BLDG., TOKYO, JAPAN
26-STORY, 354 FT. HIGH, 102X357 FT
120 HI-DAM DEVICES

VISCOELASTIC DAMPERS: * SANTA CLARA CIVIC CENTER
13-STORY STEEL-FRAMED 167X167 FT
96 VE-DAMPERS MADE BY 3M
TARGET DAMPING = 17% CRITICAL
1/2 IN. THICK POLYMER

TUNED MASS DAMPERS: * CITICORP CENTER, NEW YOK CITY
PLAN 28,000 SQ. FT., 1979
HEIGHT: 920 FT
400 TON CONCRETE BLOCK

TUNED LIQUID DAMPERS: * HOTEL COSIMA, TOKYO, JAPAN
26-STORY, 348 FT, PLAN 60X42 FT
TLCD : WEIGHT = 50 TON
WATER = 36 TON, 20X20X9.5 FT

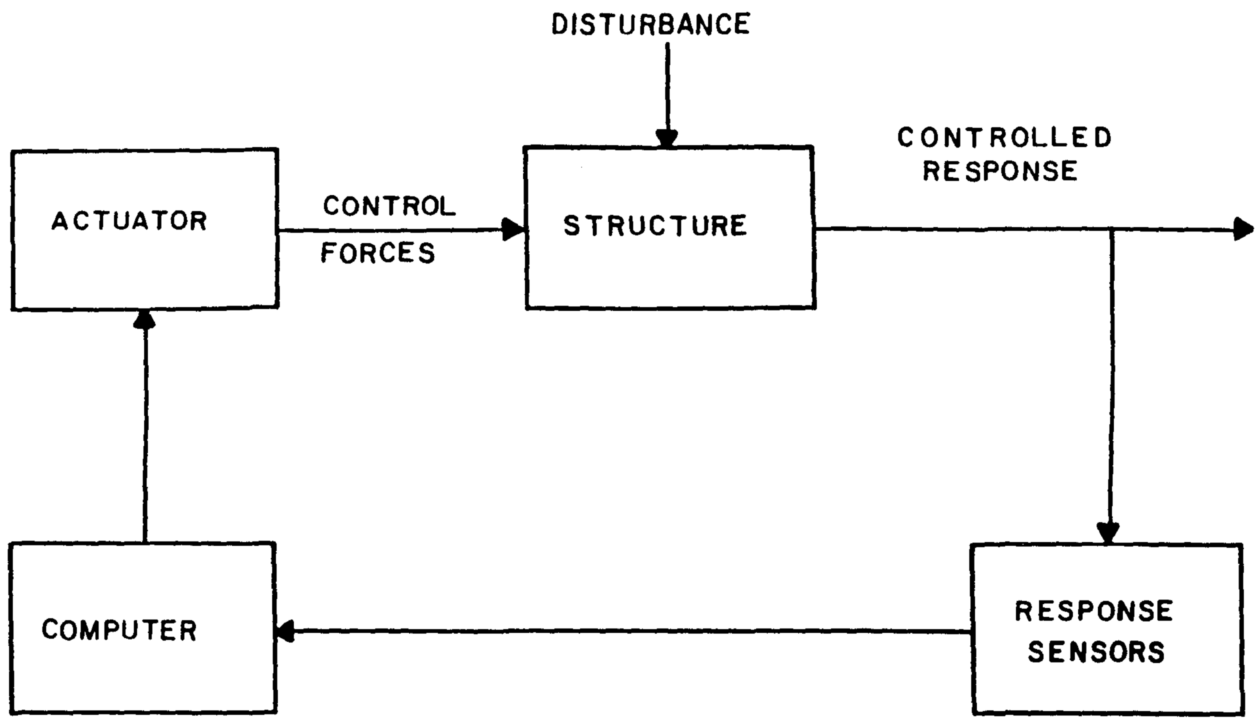
ACTIVE SYSTEMS

ACTIVE VARIABLE STIFFNESS

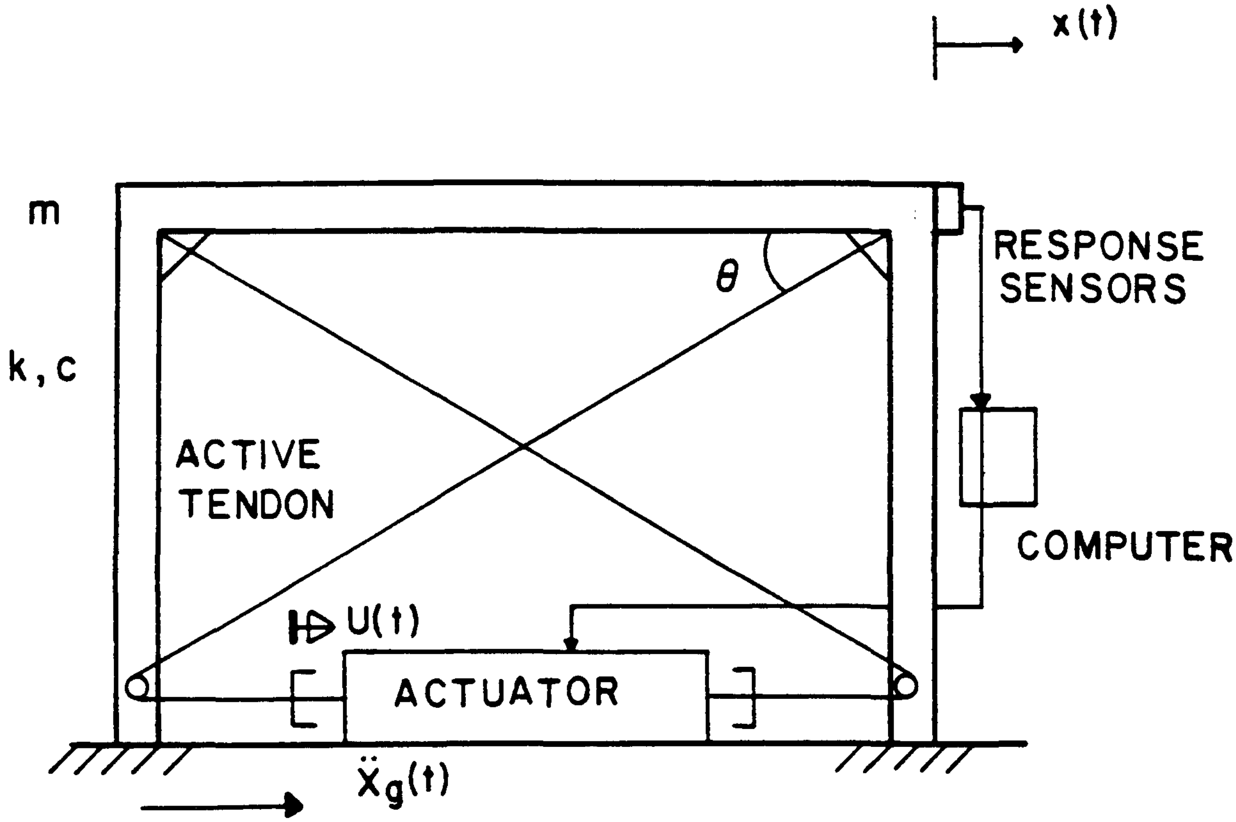
ACTIVE BRACING SYSTEM

PULSE SYSTEM

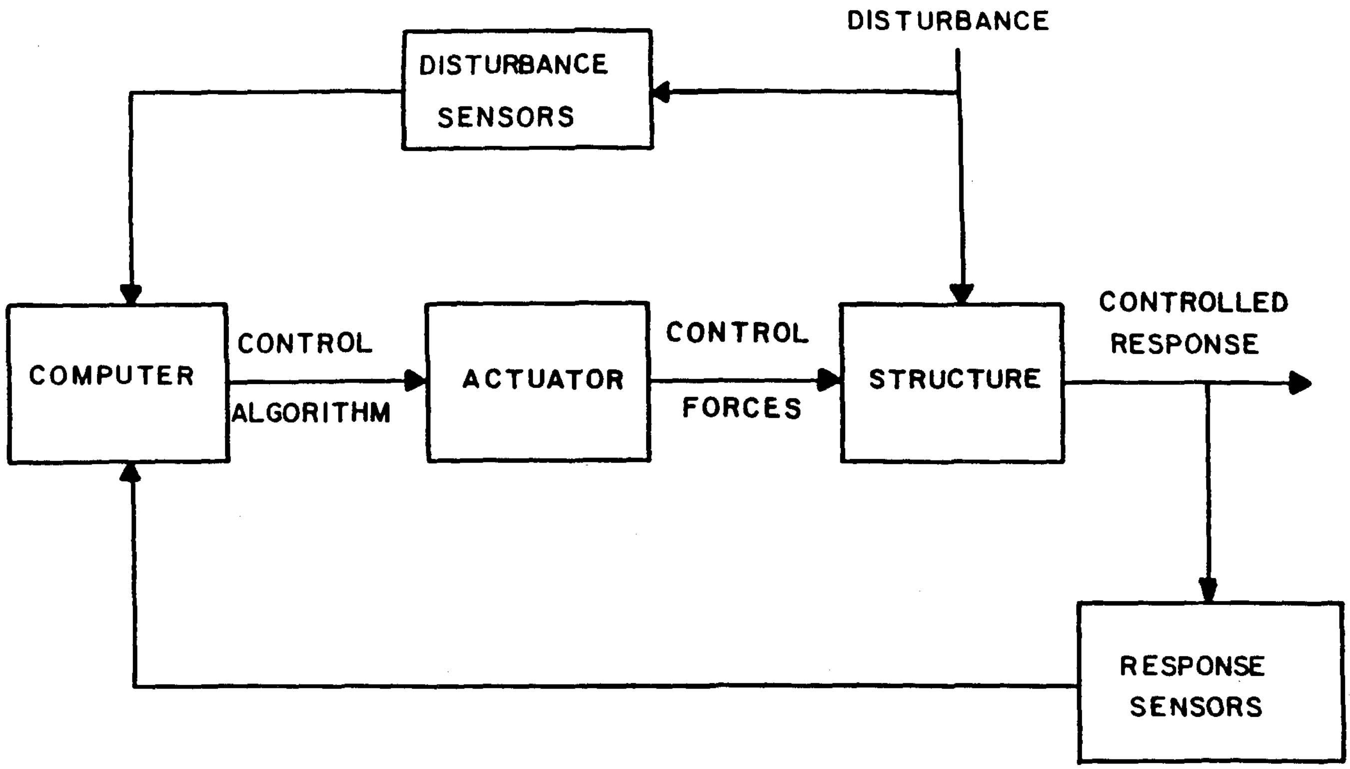
ACTIVE MASS DAMPER



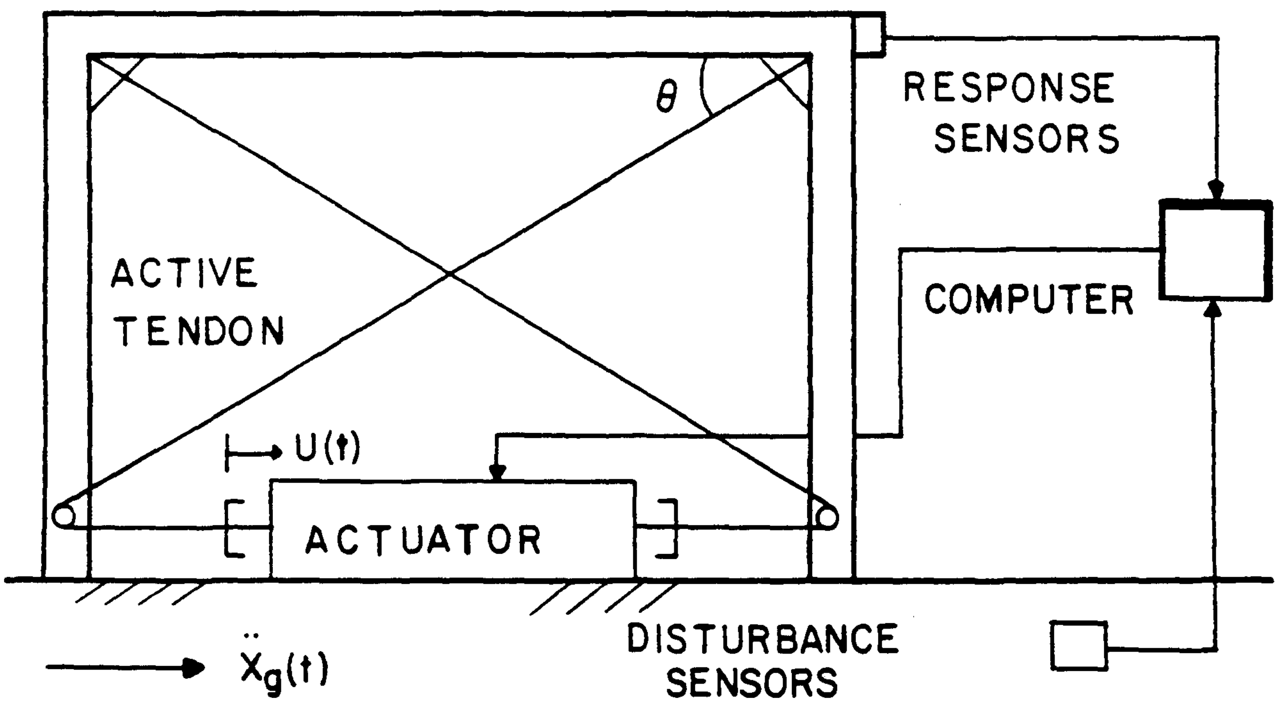
CLOSED-LOOP CONTROL



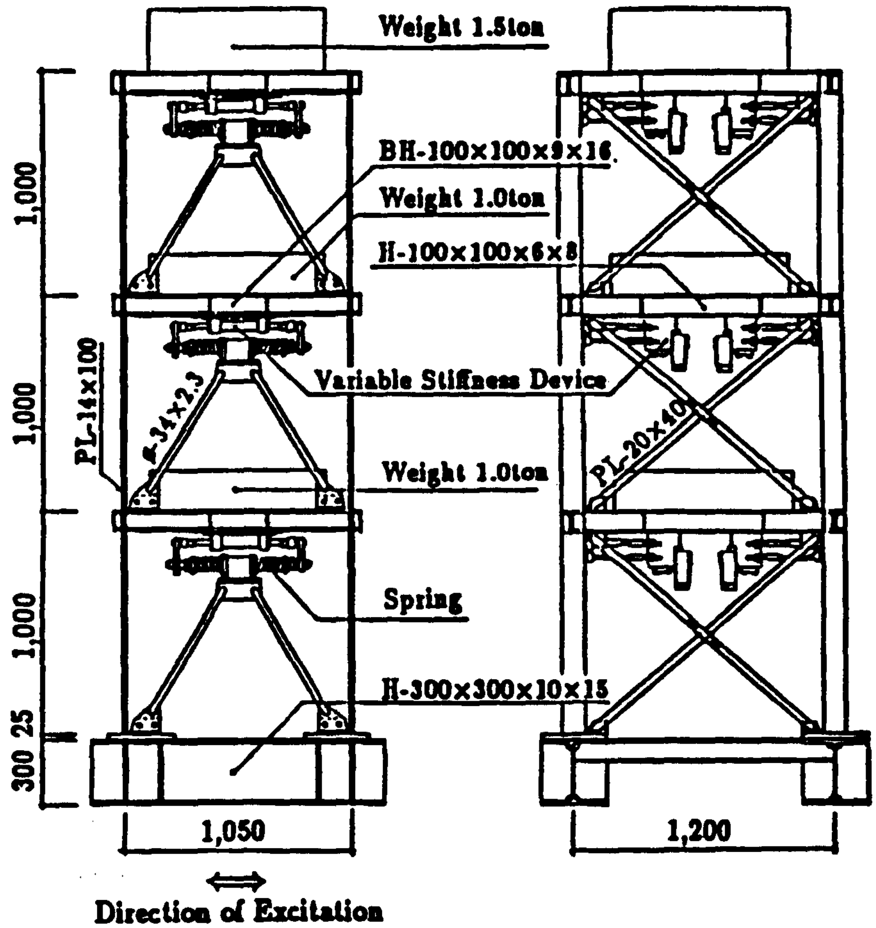
CLOSED-LOOP CONTROL



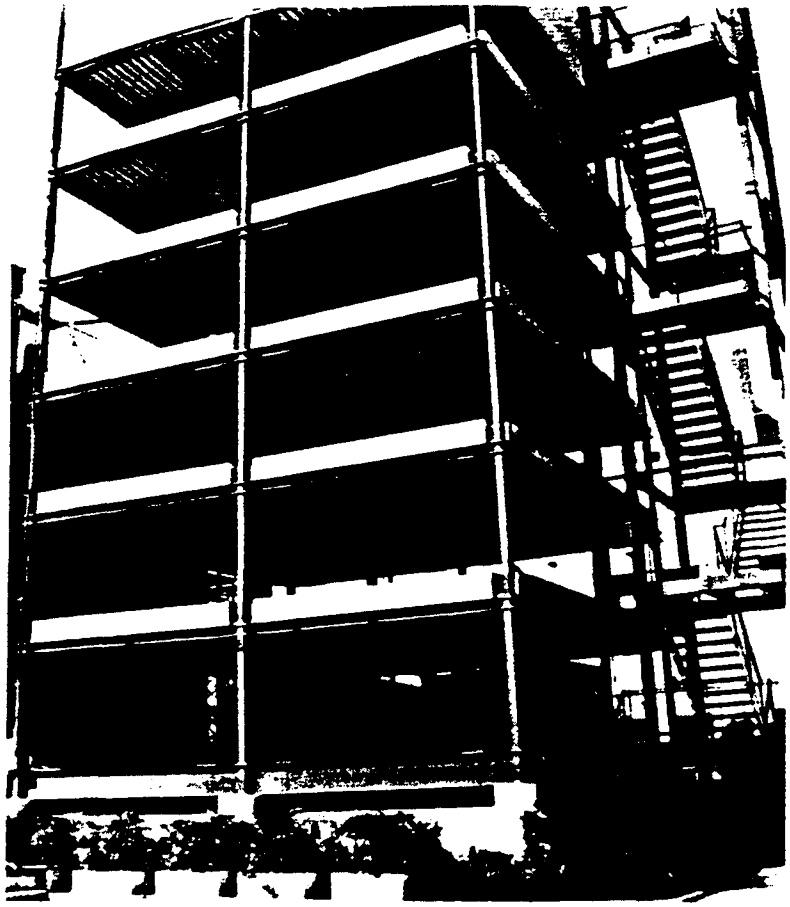
OPEN-CLOSED LOOP CONTROL



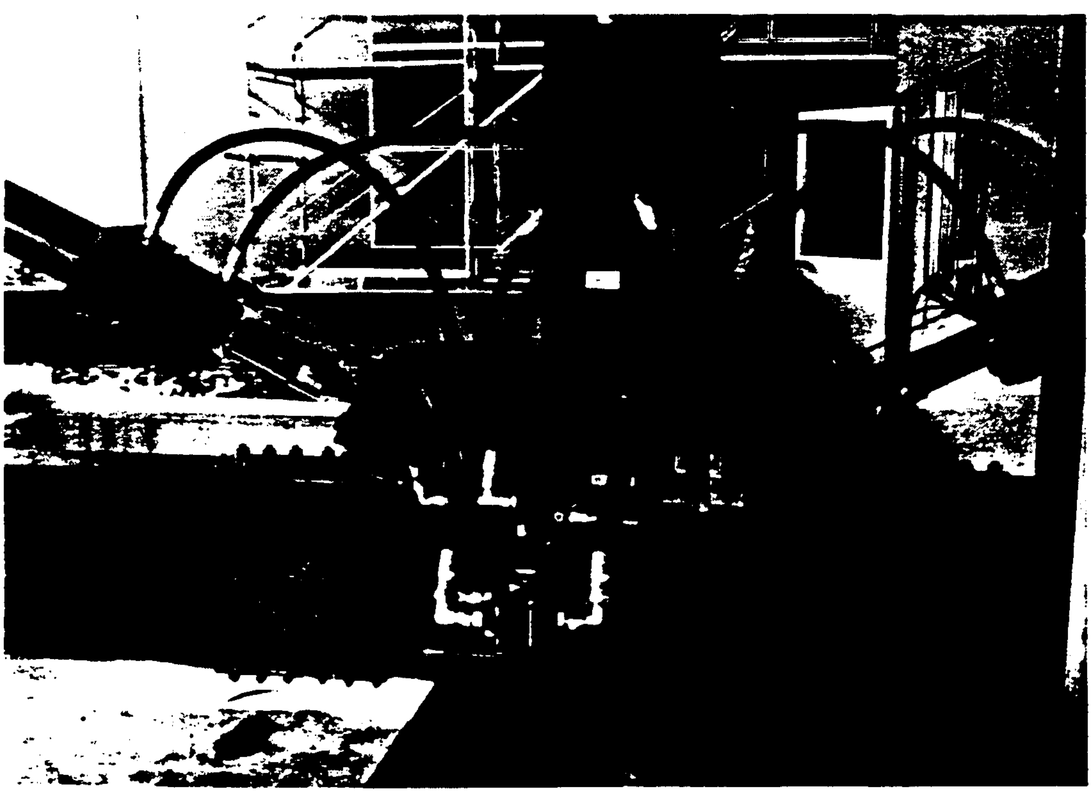
OPEN-CLOSED LOOP CONTROL

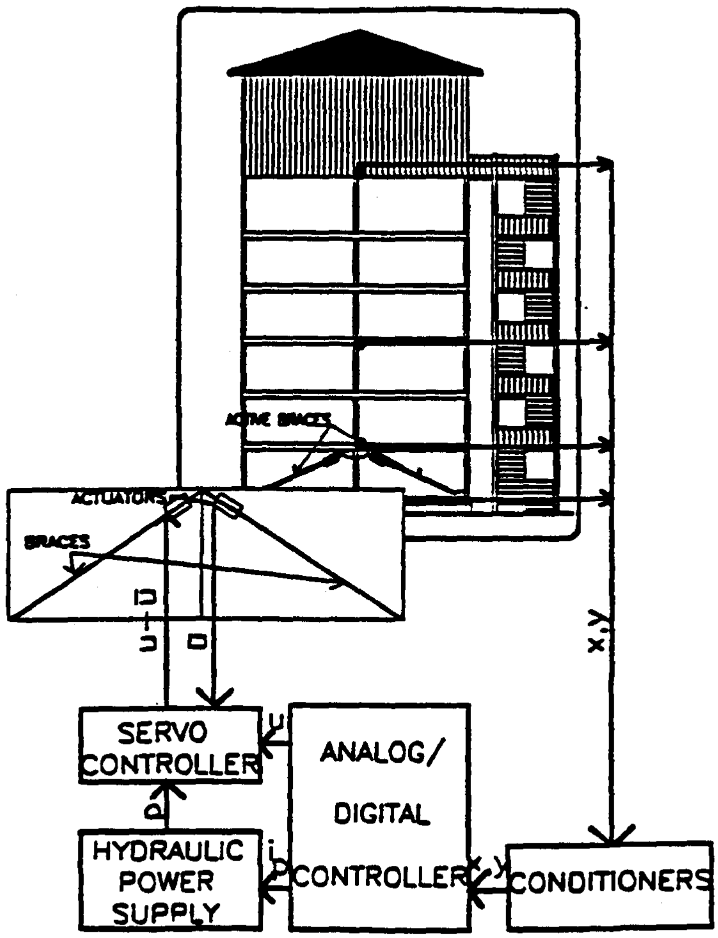


ACTIVE VARIABLE STIFFNESS

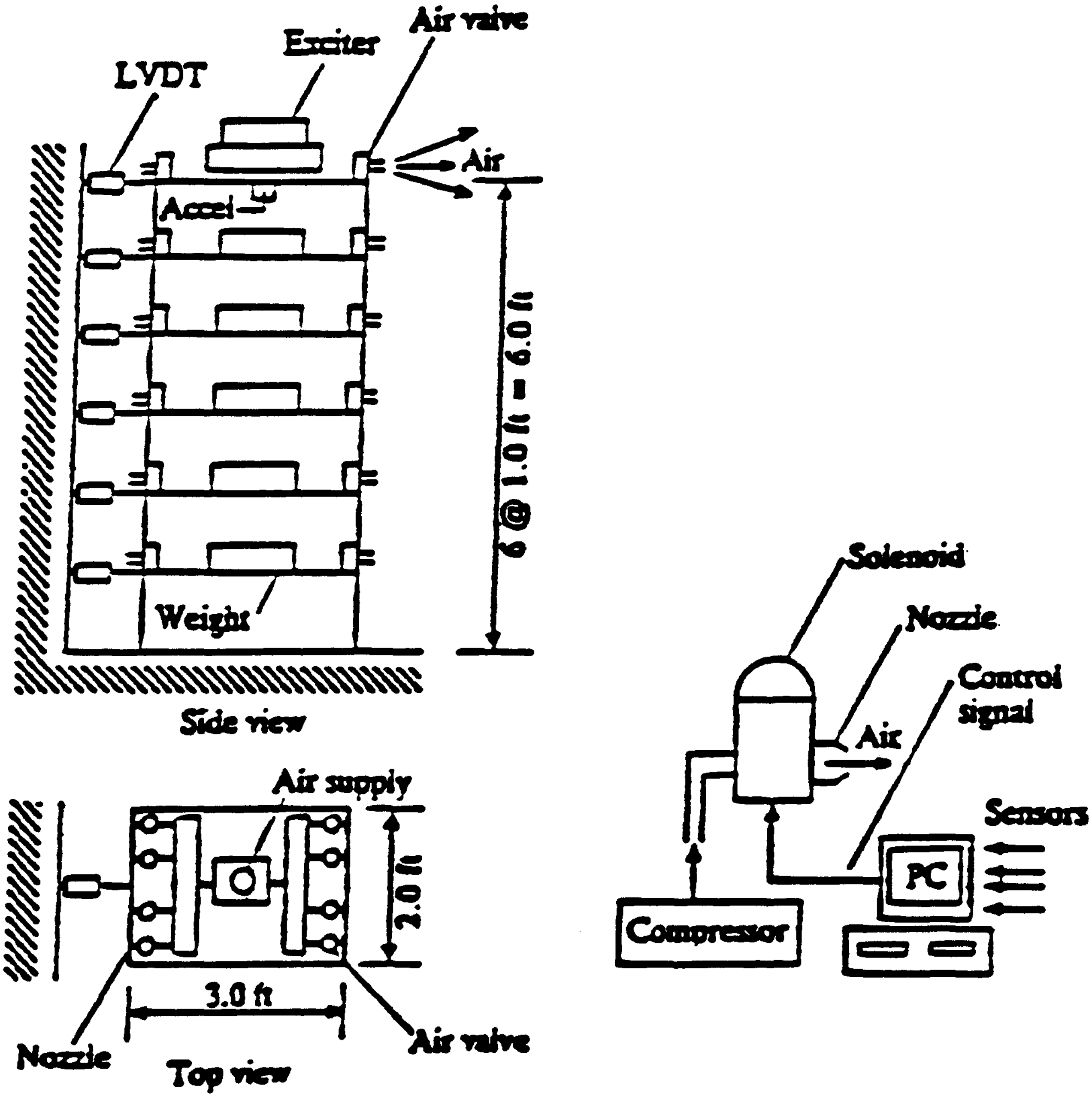


ACTIVE BRACING SYSTEM

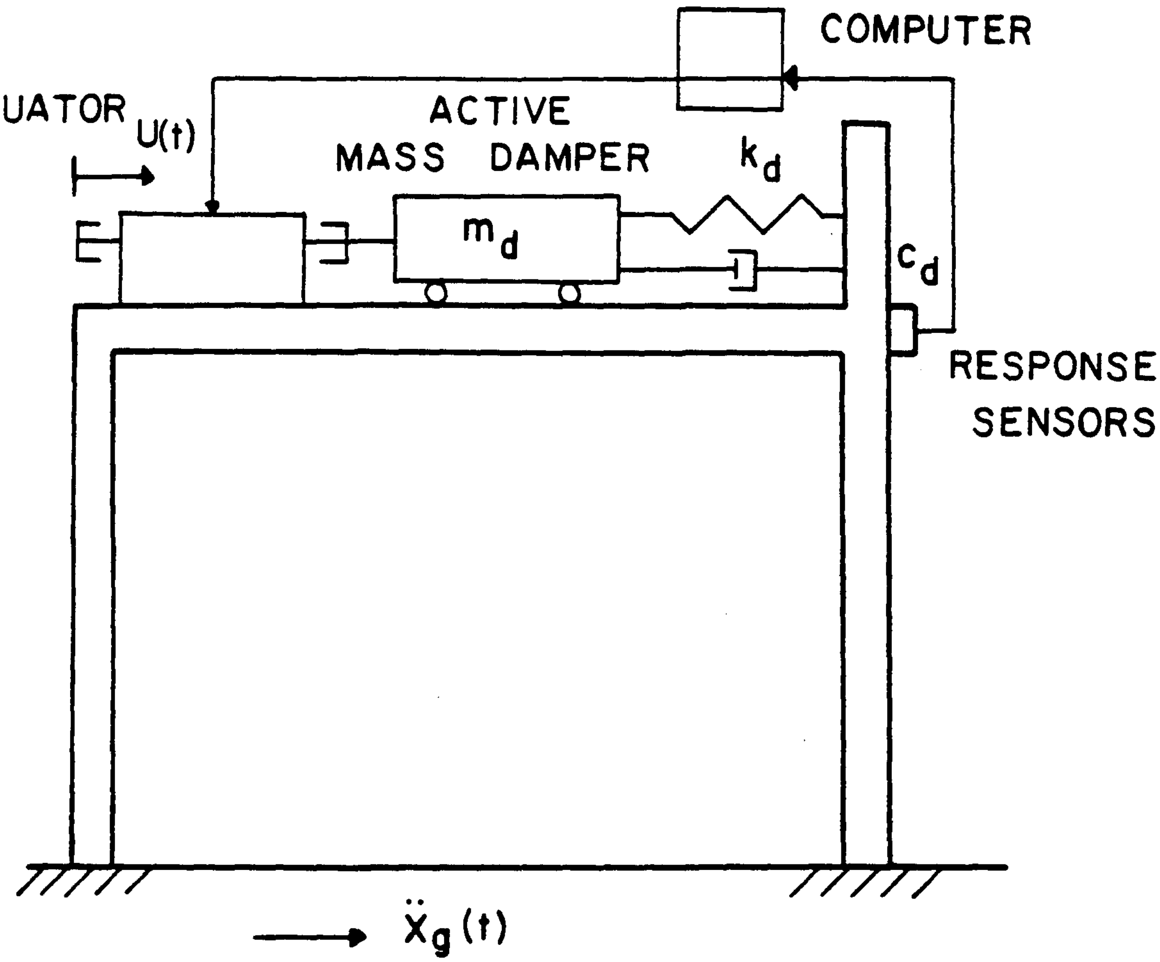




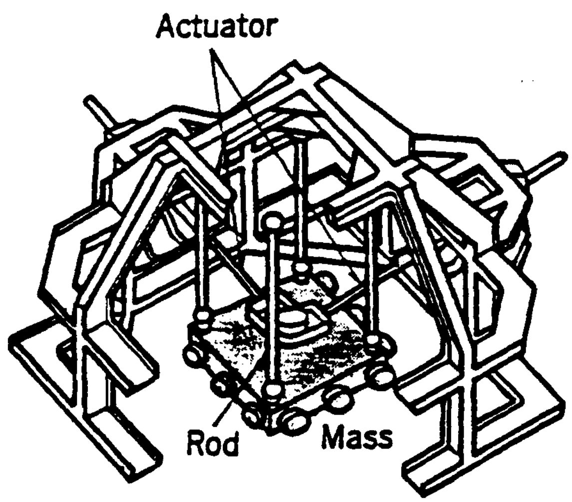
ACTIVE BRACING SYSTEM

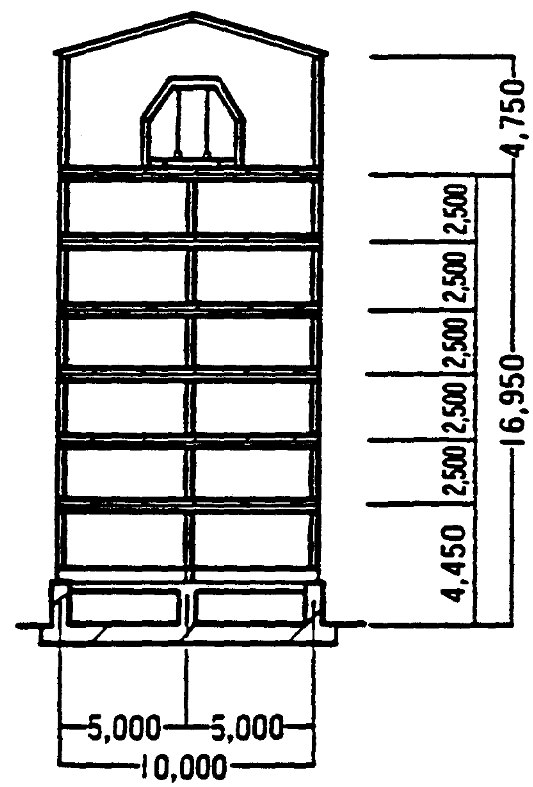


PULSE SYSTEM

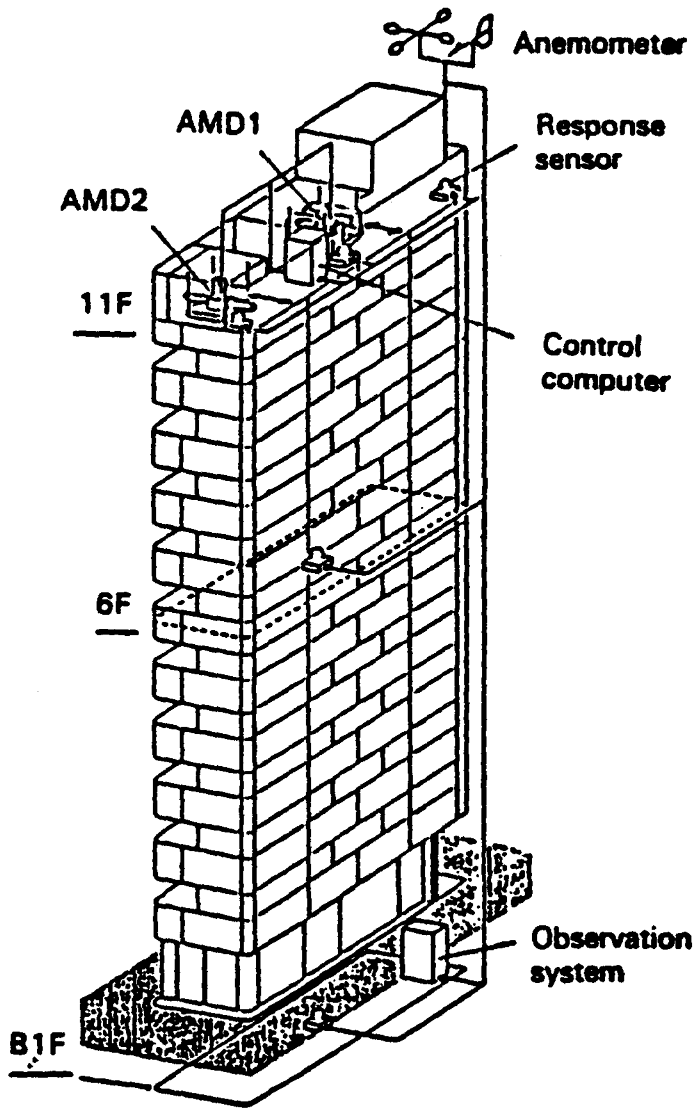


ACTIVE MASS DAMPER





ACTIVE MASS DAMPER



ACTIVE MASS DAMPER

EXAMPLES OF ACTIVE SYSTEMS

ACTICE VARIABLE STIFFNESS : * KAJIMA RESEARCH FACILITY
JAPAN , 3-STORY STEEL FRAME
RESEARCH FACILITY, 1990

ACTIVE BRACING SYSTEM : * TAKENAKA EXPERIMENTAL BLDG.
JAPAN, 6-STORY STEEL FRAME
RESEARCH FACILITY, 1990

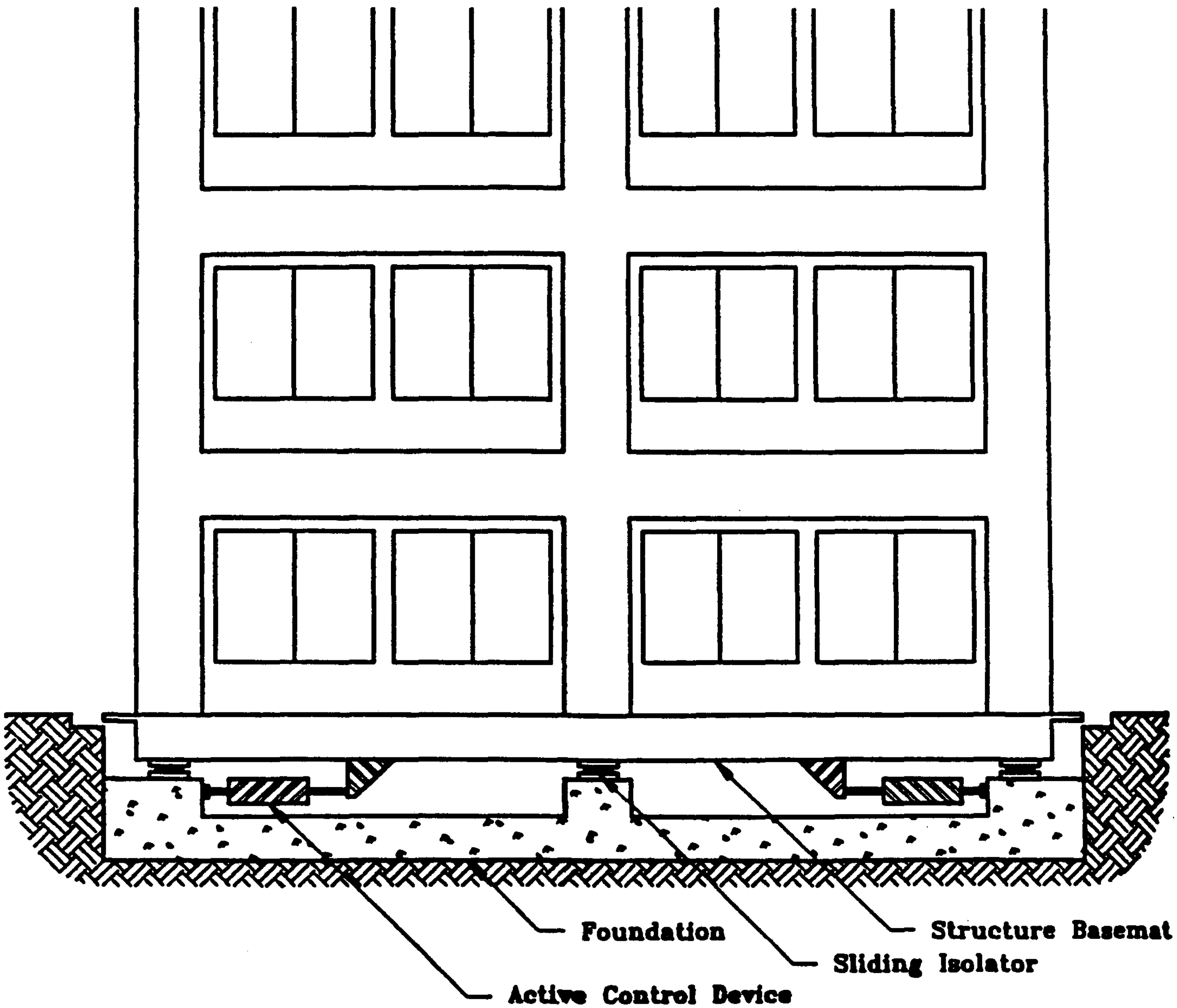
ACTIVE MASS DAMPER : * KYOBASHI SEIWA BLDG., JAPAN
11-STORY STEEL FRAME
OFFICE BLDG., BIAXIAL AMD
PENDULUM TYPE
WEIGHT = 6 TON (1% STRUCT.)
STROKE = ± 40 IN.
MAX. CONTROL FORCE = 10 TON

HYBRID SYSTEMS

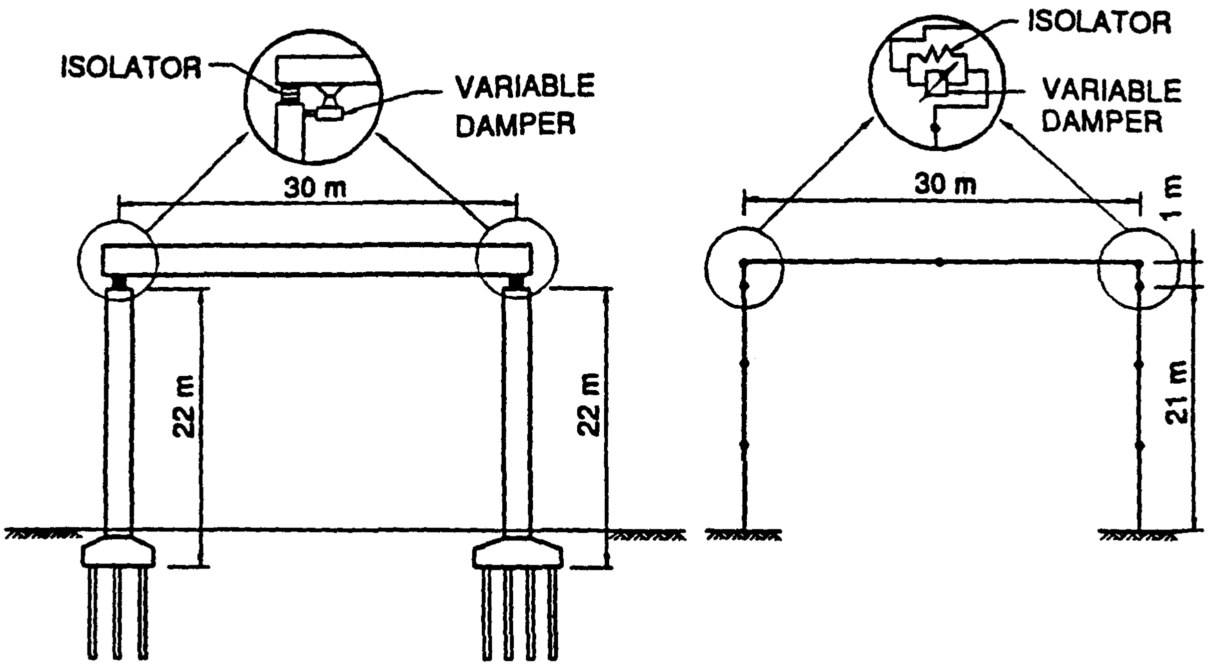
SLIDING ACTIVE BEARING

ISOLATOR DAMPER

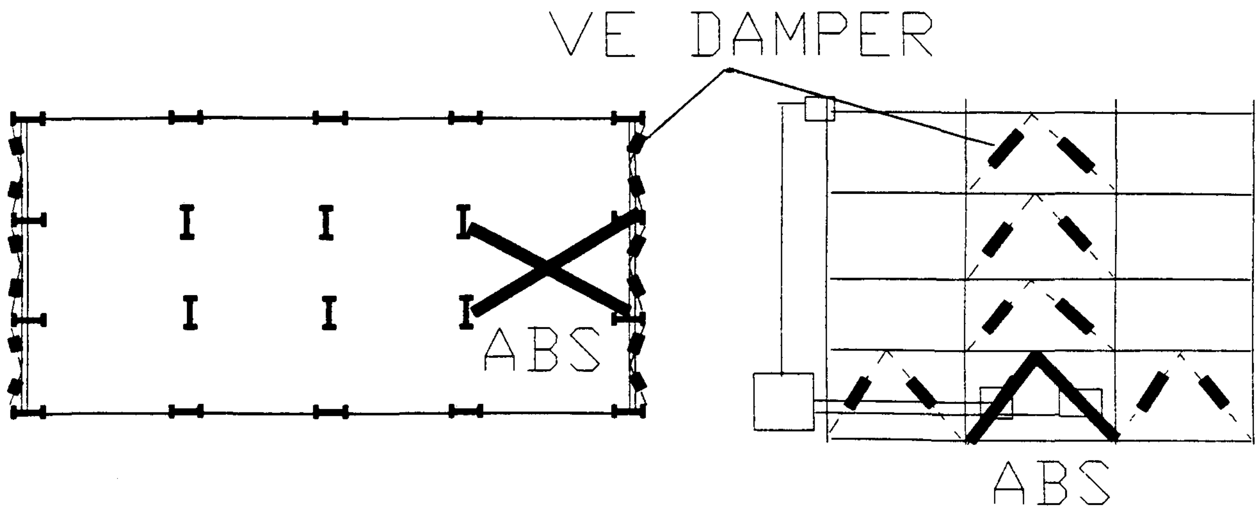
DAMPER & ACTIVE BRACE



SLIDING ACTIVE BEARING

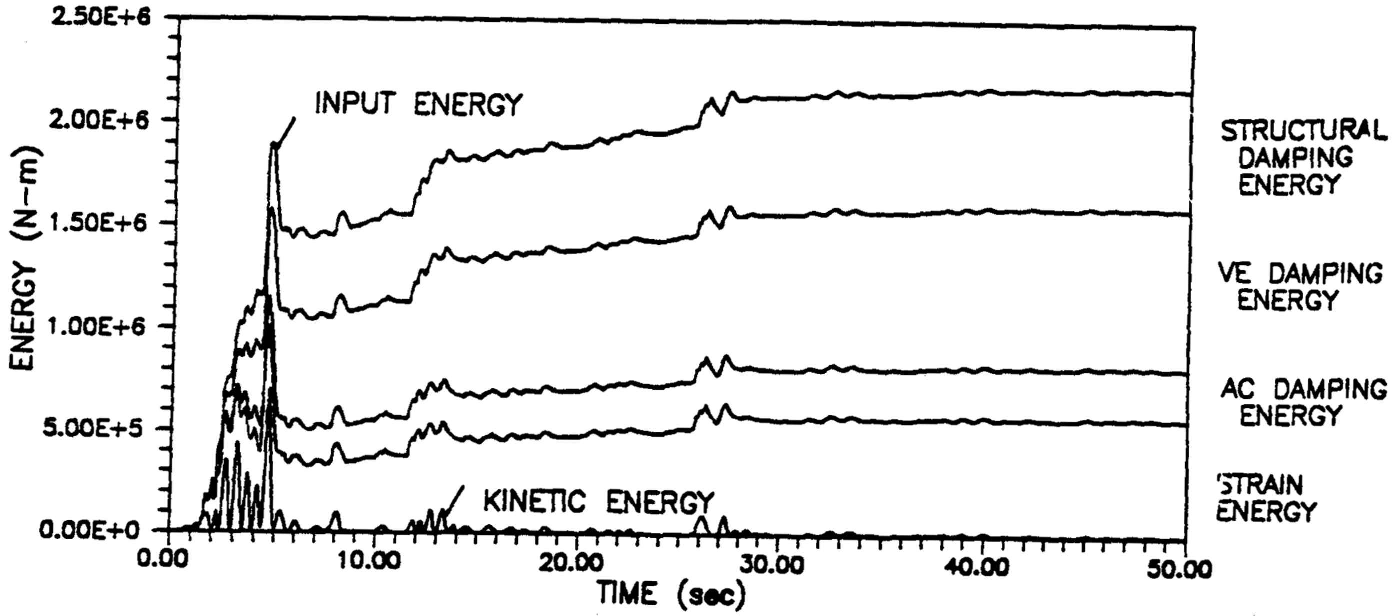


ISOLATOR DAMPER

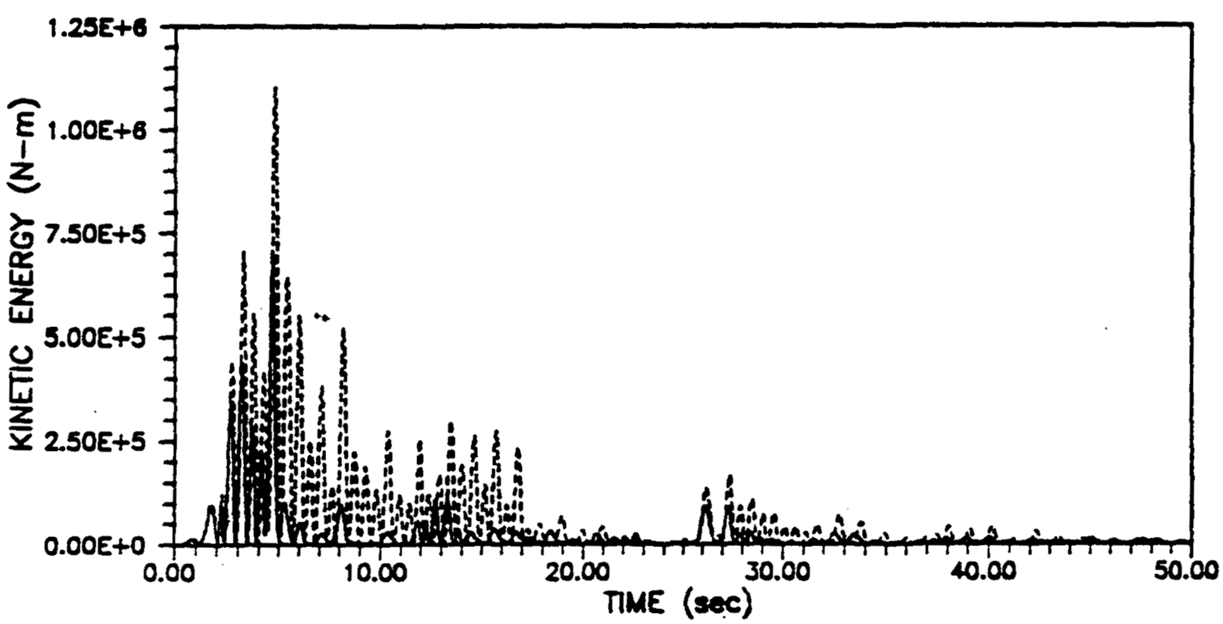


HYBRID CONTROL

DAMPER & ACTIVE BRACE



ENERGY TIME-HISTORY



KINETIC ENERGY TIME-HISTORY

6.

Lifeline Performance Considerations in Regional Earthquake Planning

by

Peter W. McDonough

Peter W. McDonough has been a civil engineer, involved in natural gas system design and construction, since 1971. Since 1978 he has been responsible for seismic design and mitigation for Mountain Fuel Supply Company's natural gas system in Utah, Wyoming and Idaho. Other current responsibilities include system wide environmental permitting, vacuum dehydration of new pipelines and design and project management of new transmission pipelines and pressure regulating stations constructed in northern Utah.

He is a member of E.E.R.I., a former chairman of the Greater Salt Lake Utility Coordination Committee's Disaster Preparedness Subcommittee and recently stepped down, after a four year term, as chairman of A.S.C.E.'s Technical Council on Lifeline Earthquake Engineering- Gas and Liquid Fuel Lifelines Committee. He has been a member of the Utah Earthquake Advisory Board's Engineering and Architecture Standing Committee, Utah Strong Motion Instrumentation Advisory Committee and Utah Advisory Council on Intergovermental Relations' Earthquake Task Force.

He is currently editing and co-authoring a book for A.S.C.E. on the seismic design of natural gas systems and is serving on the organizing committee for A.S.C.E.'s fourth U.S. Conference on Lifeline earthquake Engineering.

A licensed Professional Engineer, he earned a B.S. in civil and environmental engineering from Clarkson College of Technology, a M.S. in civil engineering from the Polytechnic Institute of New York and a certificate in management from the University of Utah.

LIFELINE PERFORMANCE CONSIDERATIONS
IN REGIONAL EARTHQUAKE PLANNING

BY
PETER W. McDONOUGH P.E.
MOUNTAIN FUEL SUPPLY COMPANY
SALT LAKE CITY, UTAH

Overview

(OVERHEAD #1)
Lifelines, in the engineering sense, are defined as the systems and facilities that provide essential services to our society. As a society becomes more industrialized and urbanized the need for services grow. Additionally, services that in the past may not have been considered critical become so, as requirements and demands of the society change.

Lifelines include utilities, both in the public and private sector. Among these can be listed electric, gas, water, sewer and telecommunications. They also include non-utility entities such as railroads, petroleum pipelines and tank farms, roads and bridges, air and sea ports and harbors.

Lifelines often traverse considerable distances, and thus may be subject to various earthquake related risks due to differences in terrain and soils. Examples of these might include interstate pipelines, electric transmission lines, railroads and highways.

A lifeline may also encompass an extensive urban area and may consist of many miles of system facilities located in city streets or alleys. Examples of this would include municipal or private utilities serving a community.

Finally, a lifeline may be situated on a single parcel of land, such as a port facility or a tank farm.

All three of these geographic situations lead to distinct challenges with regards to seismic hazard mitigation planning. The operator of a long distance pipeline, for example, needs to address the multiple earthquake scenarios that might be encountered anywhere along the route. However, this operator might only need consider the risk of one or two potential failures to the pipeline due to any given earthquake. The operator will typically have good records of the materials used and, prior to initial construction, may have had some discretion as to right-of-way location to minimize geologic concerns.

On the other hand, the local utility manager or owner may need to concentrate on one design earthquake which might cause multiple system failures. The requirements of the many individual customers who rely on the utility must be considered. And, because of the way most utility systems grow over many years, the operator may need to contend with different types of materials in the system, varying levels of record accuracy and the requirement that the utility extend to where the customers are, with little regard to geologic conditions.

The operator of an individual tank farm, sewage treatment plant or port facility perhaps can be considered in the middle of these two extremes. This type of facility occupies a certain piece of property and contains known equipment. Thus, the ability exists to extensively evaluate earthquake hazards based on a specific design earthquake with the knowledge of existing equipment and geologic site conditions.

Lifeline Vulnerabilities

(OVERHEAD #2)
Electric Power Systems

In many respects electric power systems are the most critical of lifelines, in that almost all other facilities as well as individuals are dependent on electric to power equipment and provide lighting. This will also hold true immediately after an earthquake when dependable power will be urgently required for any number of reasons.

Unfortunately, power systems oftentimes perform poorly during even moderate earthquakes, primarily due to individual component failures.

While overall seismic performance of electric generating plants has historically been good, several major components have shown susceptibility to damage. Turbines may be damaged due to unbalanced movement of the rotors or damage to bearings. Steam generators may be damaged by broken tubing or fuel piping. Miscellaneous equipment may be damaged due to inadequate anchorage.

Electric substations have repeatedly shown significant weakness during earthquakes, the majority of the damages being to porcelain insulators, where breakage is very common. Transformers, if not adequately braced or restrained, may move or topple. Failures of bus and bus supports often occur. Finally, failure of older station buildings and miscellaneous equipment or batteries may occur.

In general, transmission and distribution power lines perform well during earthquakes, with their primary vulnerabilities being possible transmission tower foundation failures and falling of pole mounted transformers, due to inadequate anchorage.

(OVERHEAD #3)
Gas and Liquid Fuel Pipelines

Fuel pipelines can generally be characterized as being either "transmission" or "distribution" facilities, with all liquid fuel facilities in the "transmission" category; the distribution of liquid fuel normally being done through tankage delivery.

Transmission pipelines are oftentimes long distance interstate facilities and are characterized by welded steel construction, segmented by valves and occasional compressor or pump stations. As mentioned above, the pipeline is likely to traverse many types of landforms and be subject to varying seismic hazard.

The primary vulnerability of a transmission pipeline is it's length and the higher probability of experiencing a seismic event somewhere along this length. The pipeline will typically respond very well to ground shaking, and moderately well to soil liquefaction. However, severe ground rupture or faulting may cause failure and resultant loss of product.

Natural gas distribution piping may consist of welded or mechanically coupled steel, fused or mechanically coupled polyethylene, or old bell and spigot cast iron. Depending on the material and coupling method, leaks or catastrophic failures may occur during moderate to large earthquakes. The often significant amounts of pipe within the service area or city may lead to many failures that must be addressed.

Gas fired equipment, such as furnaces, boilers and water heaters may be vulnerable to movement and subsequent failure and gas leakage if not anchored properly. The possibility of fire due to these failures makes this a critical issue.

(OVERHEAD #4)
Telecommunications

Within a telecommunications network, mechanical or electrical system components tend to be the most vulnerable parts. While telephone cables typically can be expected to survive a damaging earthquake with minimal damage the increasing use of fiber optic cable, with it's ability to carry more circuits than a typical copper trunk line, tends to increase the concentration of circuits into fewer trunk lines. Thus, the failure of one can impact many more customers.

The alignment of microwave towers may be compromised by earth movement, causing loss of service.

Switching facilities, often called "central offices" contain power panels, battery racks and computers which are critical to the facilities continued operation, but are very susceptible to movement if not seismically anchored. Smaller, "repeater stations" often face similar concerns.

A somewhat unique vulnerability of telecommunications systems is system overload, when the network simply cannot handle the number of calls attempting to be placed. Network management features can be used to redirect calls to minimize this problem. Continual growth of alternative telecommunications systems, such as cellular systems, also is reducing this concern.

(OVERHEAD #5)
Transportation Systems

Transportation systems include highways and railroads, with bridges and tunnels being common to both. Airports, river and sea ports and harbors are also considered within this category.

The most vulnerable component of a ground transportation system are the bridges. Damage to a bridge is never insignificant, with repairs typically being costly and time consuming. The most vulnerable components of the bridge itself tend to be the support bearings, abutments, piers and foundations. Failure of any of these can lead to bridge collapse. Failure of superstructure girders, anchor bolts and floor slabs may also occur.

Tunnels tend to respond well to ground shaking but may be damaged or rendered inoperable due to liquefaction, fault rupture or landslide effects.

Highway pavements may be impacted by landslides, liquefaction and ground rupture. Damage can take the form of soil slumping beneath the pavement, pavement cracking or heaving; any of which may cause at least temporary traffic diversion.

Railroads face the same vulnerabilities as highways, but to a greater extent in the sense that grade and alignment are much more critical. Overturned locomotives and cars will slow recovery efforts.

Airports face damage to runway pavements, control towers and buildings, as well as mechanical,electrical and communications systems. A major airport is a microcosm of a city, dependent on all utilities for it's continued operation.

(OVERHEAD #6)
Water and Sewage

Water and sewage systems may contain all or some of the following components: dams and associated structures; pipelines, tunnels and aqueducts; tanks- elevated, at grade or subgrade; wells; plant piping, equipment and structures.

Older, particularly earth fill, dams may be susceptible to liquefaction related failure or overtopping by seiche during near field or major regional earthquakes.

Open, concrete lined aqueducts often suffer cracks during moderate ground displacement. Cast iron water pipe and valves can be expected to experience joint separation and cracking, with subsequent loss of pressure. This is a critical concern with regard to fire fighting capabilities. Newer ductile iron and plastic pipes respond well to shaking but may still suffer joint separation if there is significant ground movement.

Concrete bell and spigot sewer pipe is also susceptible to joint separation. However, since sewers typically are not pressure pipes, but operate on gravity flow, failure locations may be harder to identify.

A blocked sanitary sewer can become a sewage digester, with the subsequent generation of methane gas and risk of explosion.

Earthquake induced grade changes may also impact gravity flow sewer pipes and effect the usefulness of downstream treatment plants. This, coupled with possible mechanical or structural plant damage may cause severe environmental concerns.

(OVERHEAD #7)
Liquid Storage Tanks

At grade, steel liquid storage tanks are used for water and petroleum products. Tanks may or may not be anchored to their foundations, and may have fixed or floating roofs. The most common earthquake induced damages to these non-pressurized tanks include wall buckling, due to sloshing of liquid contents; failure of adjacent pipe or valve connections; and weld failure between the base plate and wall.

Elevated water storage tanks act as inverted pendulums during earthquakes. If seismic considerations were not included in their design, they may be subject to catastrophic failure due to buckling of support members.

The principal concerns with horizontal of spherical pressure vessels are damage to supports and adjacent piping.

(OVERHEAD #8)
Secondary Losses

Failure of lifelines can have profound effects on all facets of life for the impacted population, including health, safety and economic. The secondary costs will most certainly outweigh the direct price of replacing or repairing the individual lifeline. For example, a recent Federal Emergency Management Agency (FEMA) funded study (ref.1) addressed possible economic losses to thirty six industry segments due to postulated lifeline failures during a major Wasatch Fault earthquake. Out of a total dollar loss of $5.4 billion, $3.9 billion (or 72%) was attributable to secondary losses, such as businesses having to shut down due to lack of utility services.

(OVERHEAD #9)
Collocation Issues

Yet another concern with regards to catastrophic lifeline failure is the effect of that failure on adjacent lifelines. Pipelines and electric transmission lines often are placed in utility corridors to minimize environmental impact. Utility lines are often attached to bridges. Within cities, it is likely that numerous utilities will occupy the same street.

Damage to one can cause damage to others, often resulting in increased repair time for all involved.

Failures of water mains in New York City have caused neighboring natural gas pipes to be washed out. Train derailments in California have caused damage to petroleum pipelines that were buried within the railroad right-of-way.

Another recent FEMA funded study (ref.2) addressed this issue regarding eleven sites in the Salt Lake Valley where significant collocation of lifelines occurs. One was an area contiguous to the Interstate 15-Interstate 215 merge in North Salt Lake. Besides the highways and bridge structures, the following lifelines are located in close proximity to each other at this site: two crude oil pipelines; two petroleum pipelines; two railroad tracks; two high pressure natural gas pipelines; buried fiber optic cables and electric power lines. The authors of the study estimate that earthquake damage restoration time will be increased 60% for the highway bridges, 12% for the pipelines, 145% for the railroads and 210% for the electric power lines, over the time it would take for repair if the lifelines were not as closely spaced.

(OVERHEAD #10)
Conclusion

The importance of lifelines after an earthquake cannot be overemphasized. Unfortunately, all lifelines are vulnerable to earthquake damage. This damage or loss of service can significantly impact overall restoration efforts for the entire community or region.

All lifeline operators, with the support of government regulators, should take practical and reasonable steps to minimize possible earthquake damages or mitigate the expected effects of such damages.

References

1. *Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous United States (ATC 25)*, Applied Technology Council, Redwood City, Ca., 1994.

2. Lowe, Phillip A., Po Lam and Don Ballantyne, *Lifeline Collocation Impacts Analysis-Application to Urban Areas (The Wasatch Front Fault in Salt Lake City, Utah, Area)*, INTECH Inc., Gaithersburg, Maryland, 1994.

OVERHEAD # 1

LIFELINES: THE SYSTEMS AND FACILITIES THAT PROVIDE ESSENTIAL SERVICES TO A SOCIETY.

THESE INCLUDE:

- **ELECTRIC**
- **GAS**
- **WATER**
- **SEWER**
- **TELECOMMUNICATIONS**
- **PETROLEUM PIPELINES**
- **TRANSPORTATION SYSTEMS**

OVERHEAD # 2

VULNERABILITIES

ELECTRIC POWER SYSTEMS:

GENERATING PLANTS:

- **TURBINES**
- **STEAM GENERATORS**
- **MISCELLANEOUS PLANT EQUIPMENT**

SUBSTATIONS:

- **PORCELAIN INSULATORS**
- **TRANSFORMERS**
- **BUS AND BUS SUPPORTS**
- **MISCELLANEOUS STATION EQUIPMENT**

TRANSMISSION AND DISTRIBUTION LINES:

- **TOWER FOUNDATIONS**
- **POLE MOUNTED TRANSFORMERS**

OVERHEAD # 3

VULNERABILITIES

GAS AND LIQUID FUEL PIPELINES:

TRANSMISSION LINES:

VARYING SOIL TYPES OR SEISMIC HAZARDS

DISTRIBUTION LINES:

MATERIAL:
- STEEL
- POLYETHYLENE
- CAST IRON

JOINING METHOD:
- WELDING
- HEAT FUSION
- MECHANICAL COUPLING
- BELL AND SPIGOT

EQUIPMENT DAMAGE:

CUSTOMER APPLIANCES
MISCELLANEOUS STATION EQUIPMENT

OVERHEAD # 4

VULNERABILITIES

TELECOMMUNICATIONS:

CONCENTRATION OF CIRCUITS IN FIBER OPTIC CABLES

"CENTRAL OFFICE" SWITCHING FACILITIES:

POWER PANELS
BATTERY RACKS
COMPUTERS

REPEATER STATIONS:

POWER PANELS
BATTERY RACKS
COMPUTERS

MICROWAVE TOWER ALIGNMENT

SYSTEM OVERLOAD

OVERHEAD # 5

VULNERABILITIES

TRANSPORTATION SYSTEMS:

BRIDGES:

- SUPPORT BEARINGS
- ABUTMENTS
- PIERS
- FOUNDATIONS
- GIRDERS
- ANCHOR BOLTS
- FLOOR SLABS

TUNNELS

HIGHWAY PAVEMENT:

- SOIL SLUMPING
- PAVEMENT CRACKING OR HEAVING

RAILROADS

- GRADE AND ALIGNMENT
- DEBRIS OR DAMAGED EQUIPMENT

AIRPORTS

- PAVEMENTS
- EQUIPMENT AND SYSTEMS
- MUNICIPAL UTILITIES

OVERHEAD # 6

VULNERABILITIES

WATER AND SEWAGE:

DAMS:

- LIQUEFACTION HAZARDS
- OVERTOPPING

AQUEDUCTS

PIPE:

- CAST IRON
- DUCTILE IRON
- PLASTIC
- CONCRETE SEWER PIPE

GRAVITY FLOW SEWER REQUIREMENTS

SEWAGE TREATMENT PLANTS:

- GRAVITY FLOW WITHIN PLANT
- MISCELLANEOUS PLANT EQUIPMENT

OVERHEAD # 7

VULNERABILITIES

LIQUID STORAGE TANKS:

AT GRADE TANKS:

WALL BUCKLING
ADJACENT PIPES AND VALVES
BASE PLATE/WALL WELD FAILURE

ELEVATED TANKS

PRESSURE VESSELS

OVERHEAD # 8

SECONDARY LOSSES

HEALTH:

- **ELECTRIC POWER SYSTEMS**
- **GAS AND LIQUID FUELS**
- **WATER AND SEWAGE**

SAFETY:

- **GAS AND LIQUID FUELS**
- **WATER AND SEWAGE**
- **TELECOMMUNICATIONS**
- **TRANSPORTATION SYSTEMS**

ECONOMIC:

- **LIFELINE LOSS IMPACTS ALL BUSINESS FACETS**

OVERHEAD # 9

COLLOCATION ISSUES

LIFELINES OFTEN COLLOCATED FOR:

- **ENVIRONMENTAL IMPACT CONCERNS**
- **COST OR CONVENIENCE**
- **UTILITY CONGESTION**

DAMAGE TO ONE CAN CAUSE DAMAGE TO ADJACENT LIFELINES.

INCREASED RESTORATION TIME MAY OCCUR FOR ALL IMPACTED LIFELINES.

OVERHEAD # 10

CONCLUSION:

THE IMPORTANCE OF LIFELINES AFTER AN EARTHQUAKE CANNOT BE OVERESTIMATED.

ALL LIFELINES ARE VULNERABLE TO EARTHQUAKE DAMAGE. THIS DAMAGE OR LOSS OF SERVICE CAN SIGNIFICANTLY IMPACT OVERALL REGIONAL RESTORATION EFFORTS.

LIFELINE OPERATORS, WITH THE SUPPORT OF REGULATORS, SHOULD TAKE PRACTICAL & REASONABLE STEPS TO MINIMIZE DAMAGE OR MITIGATE EXPECTED DAMAGE EFFECTS.

7.

Overview of Utah Seismic Safety Commission

by

T. Leslie Youd

T. Leslie Youd

T. Leslie Youd is chairman of the Utah Seismic Safety Commission. He is professor and chair of the Department of Civil Engineering at Brigham Young University, where he has been teaching since 1984. He teaches courses in civil engineering with an emphasis on geotechnical and earthquake engineering. His primary research areas are liquefaction, ground displacement and consequent damage, and hazard mapping.

From 1967 to 1984, Dr. Youd was a research civil engineer at the U.S. Geological Survey in Menlo Park, California. He conducted studies in earthquake engineering with an emphasis on liquefaction and ground failure caused by earthquakes. His studies included post-earthquake investigations following eleven major earthquakes on four continents; subsurface geotechnical investigations at several sites of past liquefaction; techniques for mapping liquefaction potential; and instrumentation of sites to monitor liquefaction and ground failure processes during future earthquakes.

Dr. Youd received his Ph.D. from Iowa State University in Civil Engineering in 1967. He is chairman of the Learning from Earthquakes Steering Committee of EERI and editor of the *Journal of Geotechnical Engineering*.

OVERVIEW OF UTAH SEISMIC SAFETY COMMISSION

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The Utah Seismic Safety Commission was created by an act of the 1994 Utah Legislature. This commission officially began work on July 1, 1994. The purposes of this paper are to (1) present a brief history of the commission and its predecessor organizations; (2) review the mission and responsibilities of the commission; (3) present the immediate and long term objectives; and (4) discuss influences that actions of the commission could have on improved earthquake safety in the State of Utah.

The following pages are taken from a document entitled A Strategic Plan for Earthquake Safety in Utah which has been produced by the Seismic Safety Commission and a predecessor organization called the Utah Earthquake Advisory Board. The commission has given high priority for immediate action to the seven strategies included in these pages. These seven were selected from about forty different strategies that are outlined in the above publication. These strategies and the above publication have been presented to the State and Local Affairs Committee of the Utah State Legislature for consideration as action items to take to the forthcoming 1995 legislative session.

INTRODUCTION

♦ *Mission*

The impacts of earthquakes are well known. This knowledge has come at great cost in lives and property. We must take advantage of this knowledge to adopt policies and take actions to save lives and prevent injuries, protect property, and reduce social and economic disruption from earthquakes in Utah. With the ultimate goal of making Utah a safer place to live, the mission of the Utah Seismic Safety Council (USSC), as for its predecessor, the Utah Earthquake Advisory Board (UEAB), is to function as a medium for state and local governments, the private sector, and the public to advance earthquake-related issues by developing, researching, and recommending seismic policies and approaches aimed at reducing Utah's earthquake hazards and managing Utah's earthquake risk. The USSC was given the charge to:

- ❑ Review earthquake-related hazards and risks in Utah.
- ❑ Prepare recommendations to identify and mitigate these hazards and risks.
- ❑ Prioritize recommendations for adoption as policy or loss-reduction strategies.
- ❑ Act as a source of information for earthquake safety and promote earthquake loss-reduction measures.
- ❑ Prepare a strategic seismic safety planning document before the 1995 General Legislative session.
- ❑ Update the strategic planning document and other supporting studies or reports.

To achieve part of its mission, the USSC has undertaken completion of this document prepared in draft form by the UEAB. The main points that the USSC and this document are attempting to make are the following:

1. There is a real and serious danger both of life-threatening and damaging earthquakes in Utah in our lifetimes.

2. We, collectively, can take significant actions to reduce the loss of life and property.

3. Implementing an earthquake-safety plan for Utah is a long-term process.

4. Strategies to safeguard lives and property from earthquakes must be sensitive to financial and regulatory burdens. Many actions can be taken now, without great expense, that will make Utah safer tomorrow.

♦ *Government Responsibility*

Government has a mandate to protect the health, safety, and welfare of its citizens. The government's role in improving earthquake safety is to foster, encourage, and, where necessary, require individual and collective action to deal responsibly with the earthquake threat. To reduce our vulnerability to earthquakes, we must take five types of actions to reduce losses: (1) improving our geotechnical understanding of earthquakes and earthquake hazards, (2) improving development and construction practices, (3) educating the public concerning earthquake hazards and how to respond during a hazardous event, (4) disaster-response planning, and (5) post-earthquake recovery planning. These actions necessarily involve an understanding of what will be effective in reducing risk and an appreciation of the willingness and ability of the people involved to take action.

Government, university, and private-sector scientists and engineers must work together to understand the earthquake threat to help determine which loss-reduction strategies are appropriate and cost-effective. Improvement of development and construction practices is primarily the responsibility of state, county, and municipal government agencies through adoption and enforcement of building codes, subdivision zoning, and retrofit ordinances. Public education is an ongoing process requiring coordination and cooperation among local school districts, state agencies, and universities to reach all citizens. Government agencies must develop disaster-response plans to identify: (1) the types of decisions that are likely to be needed when the expected earthquake event occurs, (2) who will make the decisions, and (3) how the decisions will be transmitted to the public and emergency-response personnel for implementation. Recovery plans are also needed to anticipate and meet the needs of communities as the post-earthquake recovery period unfolds over a period that may be as long as 5 to 10 years. These plans will help ensure a quick return to cultural and economic viability following an earthquake.

♦ *Governor's Objectives*

For effective strategic planning, within the realm of state government, plans should be developed in harmony with a statewide vision. The cornerstone of Governor Leavitt's planning agenda is a set of overall policy goals known as the "**Five Key Objectives.**" These objectives address issues critical to elevating Utah State Government to a new level of performance. They are:

1. Providing a world-class education.
2. Creating quality jobs and business climate.
3. Improving government.
4. Enhancing the quality of life for all Utahns.
5. Fostering self-reliance.

The strategies proposed in this document are consistent with Governor Leavitt's Key Objectives. They are also consistent with—and indeed many are already part of—the Utah State Legislature's strategic plan, *Utah Tomorrow.*

First, dealing with Utah's earthquake threat relies on earthquake science and engineering, much of it within Utah's system of higher education, involving the development and application of modern technologies in a world-class way.

Second, a healthy business climate in Utah depends on essential infrastructure—including the means to deal with a real and serious earthquake threat. As emphasized by a 1989 blue-ribbon panel (convened to review earthquake instrumentation in Utah), "Potential earnings will come...from increased willingness on the part of risk-conscious investors to fund large projects in Utah once the earthquake threat and the means to cope with it are better understood." A decision by state policy-makers to implement a strategic plan for dealing with Utah's earthquake dangers will favorably impress sophisticated risk managers, who increasingly will be involved, for example, in the siting of new industries or in decisions to fund private economic development.

Third, the strategies proposed in this plan are fundamental for ensuring quality of life in the form of safety for all Utahns in their homes, schools, workplaces, and neighborhoods.

Fourth, self-reliance involves education, which inherently involves information and public instruction, to deal with the complexities of modern life. One complexity is that earthquakes pose the greatest natural threat to life and property in Utah. These strategies are intended to help Utahns become progressively self-reliant in avoiding major loss of life and property in earthquakes.

♦ *History of Seismic Advisory Committees in Utah*

In 1976, the United States Geological Survey (USGS) published a study of the likelihood of and projected losses from major earthquakes in Utah. This study reported that a moderate to large earthquake was likely to strike the Salt Lake area within the next 100 years. The USGS considered "seismicity, geological history, population density, and distribution and physical status of structural and lifeline installations throughout the region." The USGS report appeared in the aftermath of a magnitude 6.0 earthquake in March 1975 in Pocatello Valley on the Idaho-Utah border. This earthquake was felt throughout the Salt Lake Valley and the northern part of Utah and damaged several buildings in Salt Lake County. The combined effect of the 1975 earthquake and the 1976 USGS report was to awaken political support for earthquake action among public officials representing Utah's urban areas.

Utah Seismic Safety Advisory Council, 1977-1981

State Representative Genevieve Atwood sponsored a bill in the 1977 Utah Legislature to create an earthquake advisory council to attend to seismic safety issues. The Utah Seismic Safety Advisory Council (USSAC) (see Appendix) was created and became the first successful effort to shape public policy for reducing earthquake risk in Utah. The USSAC mission was to "provide recommendations for a consistent policy framework for seismic safety in Utah, to recommend programs to reduce earthquake hazards, and suggest goals and priorities..." Their charge was to recommend a consistent and comprehensive public policy plan for earthquake risk reduction in Utah. Even though USSAC products were highly commended, no agency or group was given responsibility to follow through on the recommendations. Very few of the recommendations were implemented and none of the suggested legislation became law.

The USSAC, nonetheless, made a significant difference in earthquake risk reduction in Utah by:

- ❑ Linking several of the isolated scientists and earthquake-safety activists into a network.
- ❑ Focusing attention on earthquake hazards.
- ❑ Writing a series of reports that documented the status quo of earthquake preparedness and provided a framework for action.
- ❑ Bringing together local leaders with national experts.
- ❑ Providing visibility for all individuals and agencies who wanted to contribute to earthquake-hazard reduction.
- ❑ Providing an umbrella of political legitimacy to engineering, political, scientific, and other professional groups who lobbied their membership for increased acceptance of state-of-the-art techniques.
- ❑ Providing a supportive network that lasted beyond the lifetime of the organizations.

The USSAC conducted or commissioned numerous studies, sponsored meetings, issued reports, and in other similar ways dealt with the earthquake threat in Utah.

1981-1991

After the USSAC was dissolved in 1981 under the "sunset" provision of its enacting law, the role of coordinating a state earthquake program effectively passed to informal cooperative efforts among the Utah Geological Survey (UGS), the Utah Division of Comprehensive Emergency Management (CEM), and the University of Utah Seismograph Stations (UUSS). Federal attention to Utah's earthquake threat greatly increased from 1983 to 1988 as part of a special five-year focus on earthquake hazards and risk in the Wasatch Front region by the U.S. Geological Survey under the National Earthquake Hazards Reduction Program. As a result of the five-year program, earth scientists and engineers amassed a large body of technical information and reached fundamental agreement about the seriousness, extent, and nature of Utah's earthquake dangers. Despite a greatly-heightened public awareness of Utah's earthquake threat, numerous attempts to motivate state governmental action on earthquake issues were mostly unsuccessful. From 1989-1991, most of these efforts were coordinated through the Earthquake Task Force of the Utah Advisory Council on Intergovernmental Relations (UACIR). The UACIR's activities culminated in late 1990 when the Earthquake Task Force presented a list of critical needs for 1991 legislation to improve earthquake safety. As a result, six bills and one resolution which in some way dealt with earthquake safety, were introduced into the 1991 Legislature. All failed (through inaction rather than defeat), but the debate over the bills further increased awareness and gained support from many key legislators.

Utah Earthquake Advisory Board (UEAB), 1991-1994

In 1991, the Utah Earthquake Advisory Board (UEAB) was formed at the instigation of state officials and was funded through CEM by a supplemental grant from the Federal Emergency Management Agency (FEMA). Approval was gained to create the Board as an advisory group within the executive branch of state government, placing it under the Governor's Disaster Emergency Advisory Council. Under the terms of the Board's charter, Board members were chosen from leaders in their fields of expertise such as seismology, geology, structural engineering, geotechnical engineering, architecture, public policy, and emergency management. The makeup of the Board included members representing state agencies, local government, professional organizations, and the private sector (see Appendix).

The mission of the UEAB was to advance earthquake-related issues by developing, researching, and recommending seismic policies and providing a long-term strategic planning document to reduce Utah's earthquake hazards through managing the state's earthquake risk. With completion of the draft of this document, the UEAB achieved a major part of its mission and turned its responsibilities over to the Utah Seismic Safety Commission, effective July 1, 1994.

Utah Seismic Safety Commission, 1994 to present

State Representative Kim Burningham introduced legislation in the 1994 Utah Legislature to establish a commission to study and advance earthquake safety in Utah. HB 358 passed, establishing the Utah Seismic Safety Commission (USSC) and designating the Utah Division of Comprehensive Emergency Management and the Utah Geological Survey to provide staff support. The make-up of the USSC is similar to the UEAB but includes representatives from the Utah Senate and House of Representatives (see Appendix). The USSC was charged with preparing a strategic planning document for the 1995 Utah Legislature. With completion of this document, the duties of the USSC shift to facilitating implementation of the strategic plan and keeping it up-to-date.

♦ *Acknowlegements*

The USSC gratefully acknowledges the help of the UEAB and its standing committees, identified in the appendix, and UEAB staff support by: Bob Carey, Jim Tingey, Caryn Johnson, Nancy Barr, John Rokich, Jeanne Andersen, and Judy Watanabe of CEM; Gary Christenson and Janine Jarva of the UGS; and Sue Nava of the UUSS. These staff, along with UGS staff Barry Solomon, Noah Snyder, and Don Adams, helped prepare this report.

STRATEGY: Inform citizens about earthquake hazards and risks.

OUTPUT: Provide information and training targeted to meet individual or collective needs.

OUTCOME: All citizens are better able to prepare for and respond to an earthquake.

Background

Different elements of Utah society have different needs for information and training to deal with mitigating and responding to the earthquake threat. There exists significant demand for earthquake education materials and services which should be appropriate, readily available, and user-friendly.

Implementation

Programs would be targeted to each of the following population segments with the corresponding products:

1. General public – A free Earthquake Awareness Guide of earthquake services and materials widely distributed.
2. School teachers – Science and safety instructional materials.
3. Businesses – Guides and training for earthquake preparedness in the workplace for managers and employees, techniques to reduce losses and resume operations quickly after a disaster.
4. Architects, engineers, contractors – coordination of materials and training through professional associations and licensing agencies.
5. Local government – awareness program of materials, services, and information on laws, procedures, rules, and standards.

Responsible Agencies

Utah Division of Comprehensive Emergency Management
American Red Cross
University of Utah Seismograph Stations
Utah Geological Survey
Utah Office of Education
Utah Division of Occupational/Professional Licensing
Utah League of Cities and Towns
Utah Association of Counties
Uniform Building Code Commission
Structural Engineers Association of Utah
Utah Chapter, American Institute of Architects
Utah Chapter, American Society of Civil Engineers
Utah Chapter, Association of Engineering Geologists

Resources Needed

First year: Two person years- $80,000; materials- $75,000.
On-going: Training (1-3 FTEs) $40-120,000; materials $15,000.

STRATEGY: Establish community emergency response teams (CERTs) statewide.

OUTPUT: Trained volunteer community emergency response teams exist statewide.

OUTCOME: Reduce life, property, and environmental loss by providing more immediate response in a disaster.

Background

In the immediate aftermath (first 72 hours) of an earthquake, standard emergency services will not be available. Research has shown that most rescues and emergency services are provided by untrained volunteers spontaneously functioning in damaged neighborhoods. This initiative would provide very basic training for interested people in fire safety, light rescue, disaster medical operations, hazard inspection, and other services. Grouped together within each community, as a part of neighborhood groups, church groups, or professional organizations, these volunteers would be in place to act independently and spontaneously in the event of a disaster, known and trusted by the people they are helping. These volunteers will respond to their neighborhoods first, then go to staging areas to assist their local government's disaster efforts.

Implementation

Four steps are required: (1) orient elected officials, policy makers, police, and fire and emergency management personnel in the use of volunteers in disaster response; (2) identify citizen groups and volunteer organizations; (3) distribute information and hold workshops through local public safety organizations and community service groups; and (4) continue to provide technical assistance and recertification to CERTs wishing to provide community-based relief. The steps would be accomplished under the direction of local Emergency Program Managers, with assistance of fire and rescue agencies to train volunteer community emergency response teams and team leaders.

Responsible Agencies

- Utah Division of Comprehensive Emergency Management
- Local emergency program managers
- Fire and medical agencies
- Community groups of all types

Resources Needed

Funding needed to provide CERT training to local volunteer groups, to provide CERT safety equipment and basic supplies, and to manage and track statewide CERT teams and resources. Cost is about $40 per volunteer.

STRATEGY: Enhance communication capabilities for emergency responders.

OUTPUT: Develop a communication system that will allow for the use of new technologies and provide the capability of expansion during peak disaster use.

OUTCOME: Emergency response capability will be enhanced because the new communication system will allow for the interoperability of agencies to meet the requirements of multi-agency response.

Background

Public safety and local governmental agencies in Utah currently operate radio systems in the VHF 150 and UHF 450 frequency band. The availability of additional frequencies in these two bands for system expansion is very limited. With the advancement of technology comes the responsibility to develop a system that will allow for the use of this new technology. We must ensure that whatever the system is it allows for the interoperability of agencies to meet the requirements of multi-agency response. Most agree that radio coverage, combined with inadequate channel allocations, are the biggest problems in meeting the objectives of protection of life and property. During emergency situations, history continues to repeat itself with the inability of agencies to communicate with each other in an effective manner.

Implementation

A new communication network that will support both voice and data applications and accommodate current and future requirements needs to be developed. The system should support city, county, state, and federal agencies. All government agencies that are users or will have future communication needs will be requested to evaluate their present capabilities and their future communication requirements.

Responsible Agencies

Utah Division of Comprehensive Emergency Management
ARES/ RACES (amateur radio operators organizations)
Local governments
State agencies

Resources Needed

An 800 MHz system is currently being evaluated for future communication needs. Cost estimates are unavailable at this time.

STRATEGY: Improve safety of older public school buildings.

OUTPUT: Identify and reduce structural and non-structural seismic hazards in all, pre-1976 public school facilities.

OUTCOME: Safer facilities for students and teachers, as well as buildings useable in an emergency.

Background

A large number of public school buildings were designed prior to the 1976 Uniform Building Code seismic requirements. Additionally, some recent portable classrooms may not be adequately anchored to their foundation. Many schools have free standing bookshelves, file cabinets, and other heavy shelved items that are not secured and may cause harm. A major earthquake may cause significant property damage and injury to students and teachers. Additionally, these damaged structures will not be available for disaster relief efforts.

Implementation

Identify all schools and their associated hazard, structural, and non-structural problems. Initiate plan to mitigate, rebuild, or relocate the public school structures, and create a priority list to determine which buildings are the most hazardous. Study minimal cost methods of partially retrofitting schools, such as providing connections between wall and roof structures.

Responsible Agencies

State Office of Education
Individual school districts

Resources Needed

Funding for seismic studies provided in school district taxing policies. Studies by Salt Lake School District averaged $1,000 per building. A projected range would be from $500 to $5,000 per building, and would depend on the complexity of the structure and the degree of detail required in the study. Total cost for assessments of all school buildings would be on the order of $720,000.

Funding and technical expertise for seismic upgrades also funded by school district taxing. Costs for upgrades in Salt Lake averaged $833,333 per school, but costs will vary as indicated in the assessments. If the statewide average upgrade cost $500,000 per school, the total cost would be about $300 million.

STRATEGY: Improve the post-earthquake operational status of essential service buildings.

OUTPUT: All essential government services buildings need to be identified. Buildings constructed before 1976 are to be retrofitted or relocated as needed, to meet standards that will allow them to remain operational after earthquakes.

OUTCOME: The ability to provide uninhibited disaster relief services.

Background

Lessons learned in recent damaging earthquakes demonstrate the need to continue essential government services during and after an earthquake. Many facilities constructed during periods when codes were not as comprehensive as current codes, have sustained damages that restrict their use after an earthquake. Precautions must be taken to determine acceptable levels of facility performance to ensure post-earthquake availability of functions. Older essential services buildings that house emergency operations centers, law enforcement offices, and fire stations may not be able to remain functional after earthquakes. The potential loss of these functions poses an unacceptable risk because it would slow emergency response and result in unnecessary casualties and property damage.

Implementation

Using a uniform assessment, procedure the cataloging of location, hazard type, and structure vulnerability should be undertaken. Retrofit or relocation possibilities are then analyzed. Cost/benefit information is compiled and analyzed. Mitigation is then undertaken on a priority basis.

Responsible Agencies:
- Local building officials
- Division of Facilities Construction and Management

Resources Needed
- Funding to conduct an assessment of the facilities, generally less than $1,000 per building. Funding to rehabilitate the facilities on a priority basis depends on results of assessment.

STRATEGY: Develop incrementally a strong-motion program.

OUTPUT: Deploy at least 108 accelerographs in the seismic regions of the state to record strong ground shaking.

OUTCOME: The hazard of strong ground shaking from local earthquakes is better quantified so it can be correctly incorporated into safe, cost-effective design of buildings and other structures. Key information can also be rapidly available for crisis management.

Background

Measurements of actual ground-shaking are essential to ensure that buildings and structures in Utah are neither under-designed, posing a life-safety threat, nor over-designed, wasting precious resources. Engineers need, but lack, recordings of strong ground shaking from Utah earthquakes to design and construct earthquake-resistant structures (including buildings, highways, and dams) that are cost-effective. A 1989 blue-ribbon panel of national earthquake experts recommended that to obtain the necessary data, a minimum of 108 new strong-motion recording instruments (accelerographs) be installed in Utah. In 1992 the Utah Legislature appropriated $75,000 to the Utah Geological Survey (UGS) to begin a strong-motion instrumentation program, and an advisory committee of engineers and scientists was formed to guide the program. But funding was discontinued after one year. The need for strong-motion data for earthquake engineering persists--to be able to predict reliably what strong ground shaking must be anticipated and to know what forces damaged structures have experienced. Recent California earthquakes also emphasize that crisis managers quickly need reliable information on the severity and geographic extent of strong ground shaking for emergency response.

Implementation

The UGS and its strong-motion advisory committee believe that a viable strong-motion program can be established and maintained through an incremental approach. With creative planning, instruments can progressively be spread throughout the seismically dangerous areas of the state to optimize the chance of recording strong ground shaking wherever it occurs. To the extent feasible, innovative instruments will be purchased to allow at least some capability for rapidly assessing strong-motion information within minutes of a sizable earthquake along the Wasatch Front urban corridor in order to direct appropriate levels of response.

Responsible Agencies

Utah Geological Survey
University of Utah Seismograph Stations

Resources Needed

To purchase, deploy, and maintain 108 instruments over 20 years would require an annual appropriation of $150,000.

STRATEGY: Update estimates of direct losses expectable from earthquakes.

OUTPUT: Comprehensive studies to estimate the potential losses of life, number of injuries, and damages to structures and lifelines from earthquakes of various magnitudes and locations.

OUTCOME: Earthquakes are placed in a proper policy perspective based on credible projections of losses and societal impacts; emergency planning is improved; and long-term hazard-reduction activities are prioritized.

Background

Utah's last comprehensive forecast of earthquake losses was published in 1976 and is out of date. Subsequent studies have restrictively analyzed losses, say, to buildings only, or apply to restricted areas, such as Salt Lake County. In 1991, the Federal Emergency Management Agency (FEMA) funded the non-profit California-based Applied Technology Council (ATC) to develop methods to estimate losses, including casualties, associated with a magnitude 7.5 earthquake in Salt Lake City. However, it is uncertain if and when results will be made available. FEMA and the National Institute of Building Sciences (NIBS) have also developed a draft methodology (planned for release in 1996) to estimate earthquake losses various levels of detail, depending on available data bases and technical experience of those performing the analysis.

Implementation

In order to establish credible forecasts of earthquake losses in Utah, various methodologies, together with available information, must be carefully evaluated. This will require close coordination among technically diverse experts and the use of both scenario-based and probabilistic risk methods for damage and casualty estimates. Available methodologies include those developed by the ATC, FEMA/NIBS and the University of Utah Department of Geography. The Utah Seismic Safety Commission can provide a suitable forum for coordinating the interdisciplinary teams and studies required to produce well-founded estimates of direct losses expectable from earthquakes in Utah. These estimates must account for significant differences due to time of day and season. Also, loss estimates are needed for specific classes of buildings, such as schools, and for different levels of ground shaking accompanying moderate to large earthquakes, so that the cost-effectiveness of retrofit options and other loss-reduction measures can be realistically evaluated.

Responsible Agencies

- Utah Seismic Safety Commission
- Utah Division of Comprehensive Emergency Management/ Utah Geological Survey/ other data providers
- Utah Division of Risk Management/ other users of loss estimates
- Structural Engineers Association of Utah

Resources Needed

- Cost to review methods and determine needs: $30,000.
- Cost to apply University of Utah methods: Unknown.
- Cost to apply ATC-36 methods: Unknown.
- Cost to apply FEMA/NIBS methods to first earthquake scenario, $250,000.