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Seismic Awareness: Transportation Facilities

A Primer for Transportation Managers on Earthquake Hazards and Measures for Reducing Vulnerability Plus Appendices

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The purpose of this report is to alert managers and operators of transportation enterprises, in both the public and private sector, to the dangers of earthquakes t buildings and possible precautions to be taken. The nature of seismic hazards in t U.S. is described. These include the probability of occurrence of an earthquake in given area, as well as its likely intensity. The probable effects of earthquakes o particular structures (vulnerability) are examined. Vulnerability is dependent on nature of the structure and local geology as well as seismic severity. The history and current practice of structural design to counteract seismic phenomena are given and retrofit practice is discussed. Economic considerations are evaluated, with emphasis on the cost effectiveness of building new reinforced structures versus retrofitting existing structures. Three appendices provide technical explanations the material presented in the text proper.

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PREFACE

Awareness of the need for counteracting the effects of earthquakes was greatly heightened by the disastrous Loma Prieta Earthquake of 1989. A Presidential Executive Order (1990) directed all federal agencies to produce and implement a plan to assure that seismic considerations are incorporated into new buildings constructed with federal monies or leased for federal functions.

In addition to carrying out the this Order, the Department of Transportation has begun investigating ways of minimizing the vulnerability of the nation's transportation system to seismic activity, including finding cost-effective improvements for both the design of new buildings and structures as well as for the retrofitting of existing buildings and structures.

The purpose of this report is to raise the level of understanding of seismic phenomena among transportation executives and operating managers: where and when to be concerned about it and how to take advantage of the latest engineering practices in this area. There are three appendices, added for those who wish to pursue the subject in greater technical detail. They include discussions of probabilities of occurrence and intensity of earthquakes, the likely damage to a particular structure, a summary of past and present seismic design, and economic evaluations of possible precautions.

The report was prepared by Parsons Brinkerhoff Quade & Douglas Inc. on contract to the U.S. Department of Transportation, Research and Special Programs Administration, and sponsored by the Office of the Secretary for Transportation Policy.

Mr. P. Witkiewicz of the Transportation Systems Center was the technical monitor for the work reported herein. His cooperation and suggestions are gratefully acknowledged.

METRIC/ENGLISH CONVERSION FACTORS

ENGLISH TO METRIC

LENGTH (APPROXIMATE)

1 inch (in) = 2.5 centimeters (cm)
1 foot (ft) = 30 centimeters (cm)
1 yard (yd) = 0.9 meter (m)
1 mile (mi) = 1.6 kilometers (km)

AREA (APPROXIMATE)

1 square inch (sq in, in^2 = 6.5 square centimeters (cm^2)
1 square foot (sq ft, ft^2 = 0.09 square meter (m$_2$)
1 square yard (sq yd, yd^2) = 0.8 square meter (m^2)
1 square mile (sq mi, mi^2) = 2.6 square kilometers (km^2)
1 acre = 0.4 hectares (he) = 4,000 square meters (m^2)

MASS - WEIGHT (APPROXIMATE)

1 ounce (oz) = 28 grams (gr)
1 pound (lb) = .45 kilogram (kg)
1 short ton = 2,000 pounds (lb) = 0.9 tonne (t)

VOLUME (APPROXIMATE)

1 teaspoon (tsp) = 5 milliliters (ml)
1 tablespoon (tbsp) = 15 milliliters (ml)
1 fluid ounce (fl oz) = 30 milliliters (ml)
1 cup (c) = 0.24 liter (1)
1 pint (pt) = 0.47 liter (1)
1 quart (qt) = 0.96 liter (1)
1 gallon (gal) = 3.8 liters (1)
1 cubic foot (cu ft, ft^3) = 0.03 cubic meter (m^3)
1 cubic yard (cu yd, yd^3) = 0.76 cubic meter (m^3)

TEMPERATURE (EXACT)

[(x-32)(5/9)] $^{\circ}$F = y $^{\circ}$C

METRIC TO ENGLISH

LENGTH (APPROXIMATE)

1 millimeter (mm) = 0.04 inch (in)
1 centimeter (cm) = 0.4 inch (in)
1 meter (m) = 3.3 feet (ft)
1 meter (m) = 1.1 yards (yd)
1 kilometer (km) = 0.6 mile (mi)

AREA (APPROXIMATE)

1 square centimeter (cm^2) = 0.16 square inch (sq in, in^2)
1 square meter (m^2) = 1.2 square yeards (sq yd, yd^2)
1 square kilometer (km^2) = 0.4 square mile (sq mi, mi^2)
1 hectare (he) = 10,000 square meters (m^2) = 2.5 acres

MASS - WEIGHT (APPROXIMATE)

1 gram (gr) = 0.036 ounce (oz)
1 kilogram (kg) = 2.2 pounds (lb)
1 tonne (t) = 1,000 kilograms (kg) = 1.1 short tons

VOLUME (APPROXIMATE)

1 milliliters (ml) = 0.03 fluid ounce (fl oz)
1 liter (1) = 2.1 pints (pt)
1 liter (1) = 1.06 quarts (qt)
1 liter (1) = 0.26 gallon (gal)
1 cubic meter (m^3) = 36 cubic feet (cu ft, ft^3)
1 cubic meter (m^3) = 1.3 cubic yards (cu yd, yd^3)

TEMPERATURE (EXACT)

[(9/5) y + 32] $^{\circ}$C = x $^{\circ}$F

QUICK INCH-CENTIMETER LENGTH CONVERSION

INCHES	0	1	2	3	4	5	6	7	8	9	10

CENTIMETERS 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 25.40

QUICK FAHRENHEIT-CELSIUS TEMPERATURE CONVERSION

$^{\circ}$F	-40°	-22°	-4°	14°	32°	50°	68°	86°	104°	122°	140°	158°	176°	194°	212°
$^{\circ}$C	-40°	-30°	-20°	-10°	0°	10°	20°	30°	40°	50°	60°	70°	80°	90°	100°

For more exact and or other conversion factors, see NBS Miscellaneous Publication 286, Units of Weights and Measures. Price $2.50. SD Catalog No. C13 10286.

EXECUTIVE SUMMARY

Appendix A is one of three appendices that provide a technical basis for "Seismic Awareness: Transportation Facilities", a report written for transportation facility managers to educate them to the potential for seismic hazards directly effecting their facilities (new and existing), to present a suggested approach to evaluate facility vulnerability and to address seismic design aspects of new and existing facilities.

Appendix A describes the nature of seismic *hazards* in the United States. These hazards include the probability of occurrence of an earthquake in a certain area, as well as its likely intensity. Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes. It is important to understand the hazard associated with a site in order to know the earthquake dangers to which facilities in the area might be exposed.

Appendix B, included in a separate volume, discusses the *vulnerability* of transportation facilities. Appendix C, also included in a separate volume, summarizes current seismic design and retrofit practices in the United States.

This appendix describes the earthquake hazards that exist for transportation facilities. The origin of earthquakes is introduced to provide some insight into the causes of these hazards. The variation by region is described and a simple method of determining earthquake hazards for a given facility is explained. More refined methods are also described.

Hazards vary in different regions of the country. California is notorious for severe earthquakes, while there are few in the Midwest. Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes. On a large scale, the probability of an earthquake and the severity that can be expected can be predicted depending on the region. This regionality has been determined through a review of historical records and is currently presented on maps providing various ground motion parameters for the various regions of the country which categorize areas by their seismic activity and level of risk.

The parameter most commonly used to measure seismic hazard is the *peak horizontal ground acceleration coefficient*. This is the acceleration of the ground, expressed as a percentage of gravity, resulting from ground shaking during an earthquake. Typically, this acceleration coefficient indicates the estimated peak ground acceleration that statistically has a 90 percent probability of not being exceeded within 50 years. In areas of moderate seismicity such as New York City or Boston, this acceleration coefficient is approximately 0.15, while in areas of high seismicity in California, it may be higher than 0.40, which translates into significantly higher seismic design forces for structures in this region.

These acceleration coefficients are plotted on maps showing, with contours, their variation across the country. Figures A3-2 and A3-3 indicate ground acceleration coefficients from the 1991 NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings. These same maps are included in the Building Officials and Code Administrators International (BOCA) National Building Code. Similar maps are included in other codes.

It should be noted that in some cases the existing regionalization maps are insufficient or inappropriate for design purposes, making it necessary to perform a project specific or site specific seismic hazard analysis.

SEISMIC AWARENESS:
TRANSPORTATION FACILITIES

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EXECUTIVE SUMMARY

There are many transportation facilities across the country that are vulnerable to serious earthquake damage. This has been pointed out all too clearly by the damage sustained by facilities in recent earthquakes. The costs to society of this vulnerability are large. Inadequate structures cost lives, disruption to transportation systems costs the productivity of the nations work force and results in increased transportation costs, and repairing earthquake damage costs money. The gravity of the threat is often not appreciated by transportation facility managers and operators.

The Department of Transportation (Seismic Safety Committee) has underway a number of responses to the seismic design issue. The Presidential Executive Order of 1990 (No. 12699) directed all federal agencies to produce an implementation plan to assure seismic design considerations are incorporated into new buildings constructed with Federal moneys, or leased for Federal functions.

One of the areas of concern to the committee is that very few of the individuals responsible for the operation of various transportation enterprises, in either the public or the private sector, in all the modes have an adequate appreciation of the seismic vulnerabilities they face. Further, they are unaware of the improvements that are possible, both in pre-construction design, and in the retrofitting of existing structures, that can reduce these vulnerabilities. The purpose of this report is to increase the level of understanding of transportation executives and operating managers about seismic phenomena, where to be concerned about it, and how to take advantage of the latest engineering practices to obtain the protection that is possible.

The aspects of earthquakes that should be of more concern to the transportation executive can be broken down into three categories. This report explores these categories as follows:

1. *Seismic Hazards in the U.S.* The nature of seismic *hazards* in the United States is described. These hazards include the probability of occurrence of an earthquake in a certain area, as well as its likely intensity. Seismic hazard is dependent on location, geology and the existence of subsurface features that can cause earthquakes.

2. *Seismic Vulnerability of Transportation Facilities.* Vulnerability here refers to the likely consequences of the expected seismic event on a particular structure. Unlike seismic hazard, vulnerability applies to specific structures. It is dependent on the expected seismic hazard, as well as the structural characteristics of the facility and the *local* geology of the site.

3. *Seismic Design and Retrofit Practice.* Current seismic design and retrofit practices in the United States are summarized. Also, the history of seismic design is described, illustrating the evolution of seismic design technology. Finally, the economic considerations involved are evaluated, emphasizing the cost-effectiveness of incorporating seismic design elements into new structures during pre-construction design as opposed to retrofitting existing structures.

The facility managers play a key role in reducing the vulnerability exposure of the transportation network. This report takes the first step in raising their consciousness to the threat that earthquakes pose to their facilities. It shows that there are steps that can be taken to reduce vulnerabilities to acceptable levels - through proper seismic design prior to construction, and through seismic retrofit of existing facilities. More detail is provided in the three Appendices for technical staff, or readers who want to learn more about this topic.

Appendix A: Seismic Hazards in the U.S.
Appendix B: Seismic Vulnerability of Transportation Facilities.
Appendix C: Seismic Design and Retrofit Practice.

1.0 INTRODUCTION

Seismic design and retrofit practice in the U.S. today ranges over extremely wide levels in technical, managerial and policy areas. The level of awareness of federal agencies regarding this topic also varies widely. However, recent experience, i.e., the Loma Prieta Earthquake of 1989 which resulted in sixty-two deaths and six billion dollars in damage has had a profound effect on both the engineering profession and the ultimate users of seismic design information. The Office of the Secretary for Transportation (OST) has underway a number of responses to the seismic design issue. The Presidential Executive Order of 1990 (No. 12699) directed all federal agencies to produce an implementation plan to assure seismic design considerations are incorporated into new buildings constructed with Federal moneys, or leased for Federal functions.

The Department's Seismic Safety Committee has undertaken additional activities dedicated to minimizing the vulnerability of the nation's transportation system to seismic activity. This work is a long term effort involving every transportation mode which, among other goals, seeks cost effective improvements for both the design of new buildings and structures as well as the retrofitting of existing buildings and structures.

One of the areas of concern to the committee is that very few of the individuals responsible for the operation of various transportation enterprises, in either the public or the private sector, in all the modes have an adequate appreciation of the seismic vulnerabilities they face. Further, they are unaware of the improvements that are possible, both in pre-construction design, and in the retrofitting of existing structures, that can reduce these vulnerabilities. The purpose of this report is to increase the level of understanding of transportation executives and operating managers about seismic phenomena, where to be concerned about it, and how to take advantage of the latest engineering practices to obtain the protection that is possible.

This report serves as a short primer for transportation executives. It is written to convey the fundamental concepts, without becoming too technical. For more detailed reference, however, it is supported by three technical appendices.

This report is organized to logically describe the level of seismic hazards that exists throughout the United States and to explain the nature of the vulnerability of various transportation facilities to damage from the existing hazards. The report discusses both seismic design and seismic retrofit practice and how these engineering approaches seek to mitigate the hazards and vulnerability.

Chapter 2 discusses some of the adverse effects of recent U.S. earthquakes to include loss of life, damage to facilities, and interruptions to various services. Chapter 3 describes earthquake hazards and how the hazard can be determined for a given geographic area. Chapter 4 discusses the vulnerability of facilities to earthquake damage, and provides a method of determining the vulnerability of a specific facility. Chapter 5 summarizes the current state of seismic design and retrofit practice and provides insight into economic considerations, because ultimately any actions; whether of a research, analysis, design, or construction aspect, require expenditures that are competed for from other areas. Chapter 6 concludes with a summary of the findings and presents a preliminary approach to developing a set of seismic design guidelines for DOT-sponsored facilities.

Three technical appendices have been provided to supplement this report with more detailed information on the above issues:

Appendix A: Seismic Hazards in the U.S.	Appendix A describes the nature of seismic *hazards* in the United States. These hazards include the probability of occurrence of an earthquake in a certain area, as well as its likely intensity. Seismic hazard is dependent on location, geology and the existence of subsurface features that can cause earthquakes.

Appendix B: Seismic Vulnerability of Transportation Facilities.

Appendix B discusses the *vulnerability* of transportation facilities. Vulnerability here refers to the likely consequences of the expected seismic event on a particular structure. Unlike seismic hazard, vulnerability applies to specific structures. It is dependent on the expected seismic hazard, as well as the structural characteristics of the facility and the *local* geology of the site. Vulnerability is also distinguished from hazard in that hazard is a naturally occurring phenomena that man is unable to affect, while vulnerability is dependent on human factors that we have control over and can change - like the construction of a building or the steepness of an earth slope.

Appendix C: Seismic Design and Retrofit Practice.

Appendix C summarizes current seismic design and retrofit practices in the United States. It gives some history on seismic design, illustrating the evolution of seismic design technology. It also explains in some detail the methods currently used for the design of different types of transportation facilities and components. Finally, it reviews the economic considerations involved and emphasizes the cost-effectiveness of incorporating seismic design elements into new structures during pre-construction design as opposed to retrofitting existing structures.

2.0 DAMAGE FROM RECENT U.S. EARTHQUAKES

2.1 General

Earthquakes are real, they happen daily and they cause injury, death and property damage. Every year in the U.S. hundreds of earthquakes occur, most of them too weak to be felt, or to cause appreciable damage. Also, most of them occur in the extreme western portions of the U.S. largely in California and Alaska. Thus, most of the U.S. has a relatively low level of awareness regarding seismic engineering issues.

Earthquakes result from the sudden and violent release of elastic energy within the earth as the result of movements along geologic structures. This energy is mainly the result of stresses built up during tectonic processes consisting of the interaction between the earth's crust and the interior of the earth.

Global tectonics is a concept based on an earth model characterized by a small number (10 - 25) of large thick plates composed of both continental and oceanic crust. Each plate "floats" on a viscous underlayer and moves independently of the others, grinding against them at the common boundaries. Figure 2-1 shows the boundaries of the major plates, while Figure 2-2 shows the distribution of earthquakes around the world. The plate edges coincide well with the epicenters of most frequent earthquake activity as can be seen by comparing these figures. As a result of the immense pressure and temperature within the inner layers of the earth, the relatively thin outer crust is continually subject to movement. Most movements are gradual and can only be detected by careful measurements. Some, however, are the result of sudden releases of elastic energy as the large plates making up the earth's crust move relative to each other. It is these violent releases that typically cause the phenomenon we call earthquakes.

The location of an earthquake is usually referred to as its epicenter, which is the point on the earth's surface directly above the crustal disturbance. The effect of an earthquake can be very far-reaching as the ground vibrates under the propagation of the generated waves.

The destructive phase of an earthquake may vary in duration from a few seconds to about one minute. It is estimated that throughout the world there are over a million earthquakes every year. The majority are quite weak and many occur in remote unpopulated areas and therefore are noticed only by scientists. It is estimated that a large earthquake (greater then magnitude 6) occurs about once every week.

The size of an earthquake can be described in terms of the impact it has on the developed environment of the area (people and structures), a semi-subjective measure, intensity, or it can be classified according to the quantitative measure of the energy released, magnitude. Both methods have their importance.

The intensity scale used in the United States is known as the Modified Mercalli Scale. This scale uses Roman numeric classification from I to XII to describe the intensity based on the impact to the surroundings. For example, Intensity I refers to an event detectable only by instruments, while Intensity XII implies almost complete destruction. Significant building damage begins at intensity VII to VIII. Table 2-1 lists the damage associated with the various Modified Mercalli Intensities. A given earthquake is characterized by its peak intensity, since the intensity for a given earthquake varies with location. Plotting the intensity observed at various locations for a single earthquake indicates the attenuation of the earthquake effects with distance, and also indicates the importance of local soil conditions.

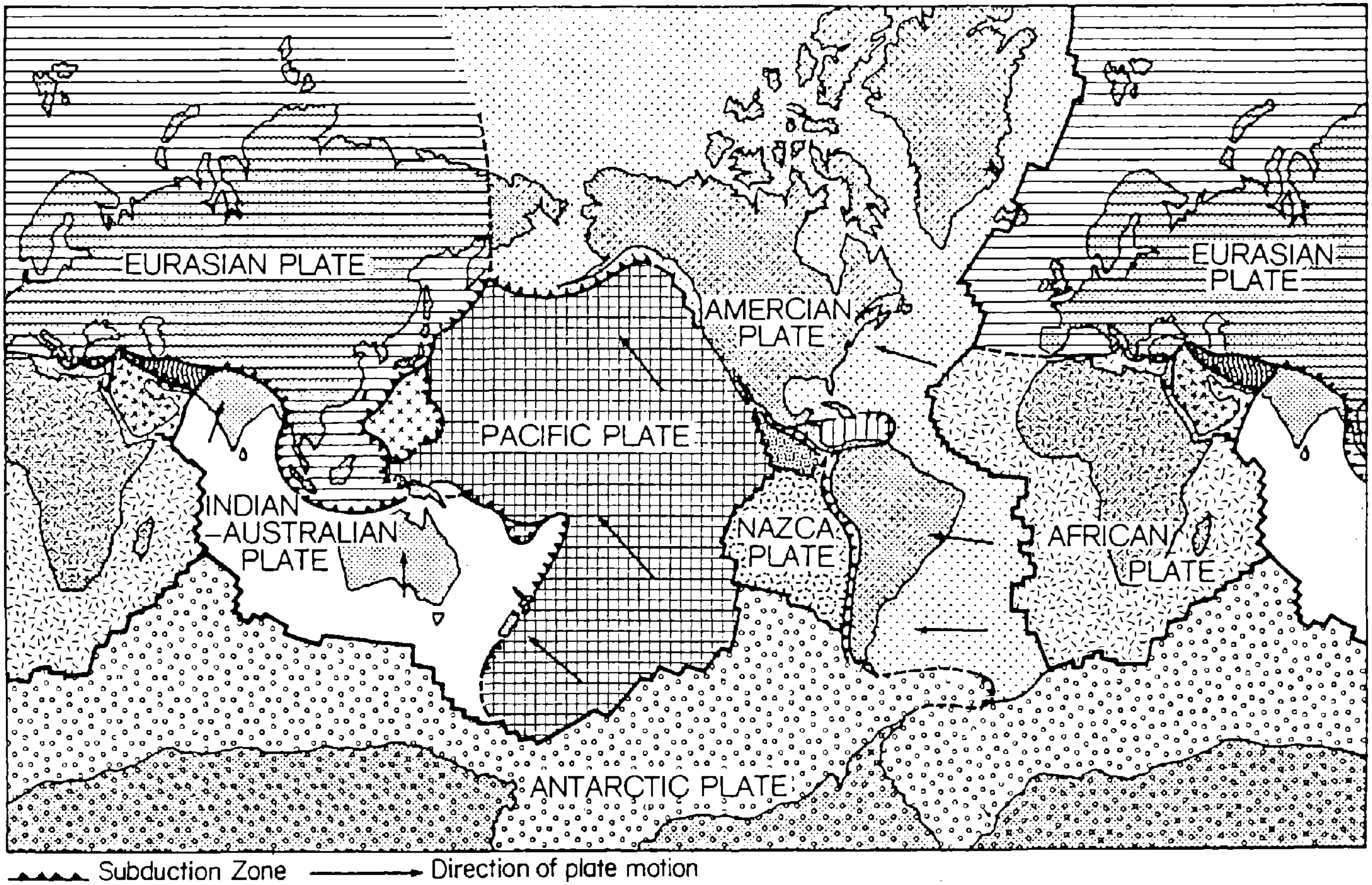


Figure 2-1 - Tectonic Plate Map of the World

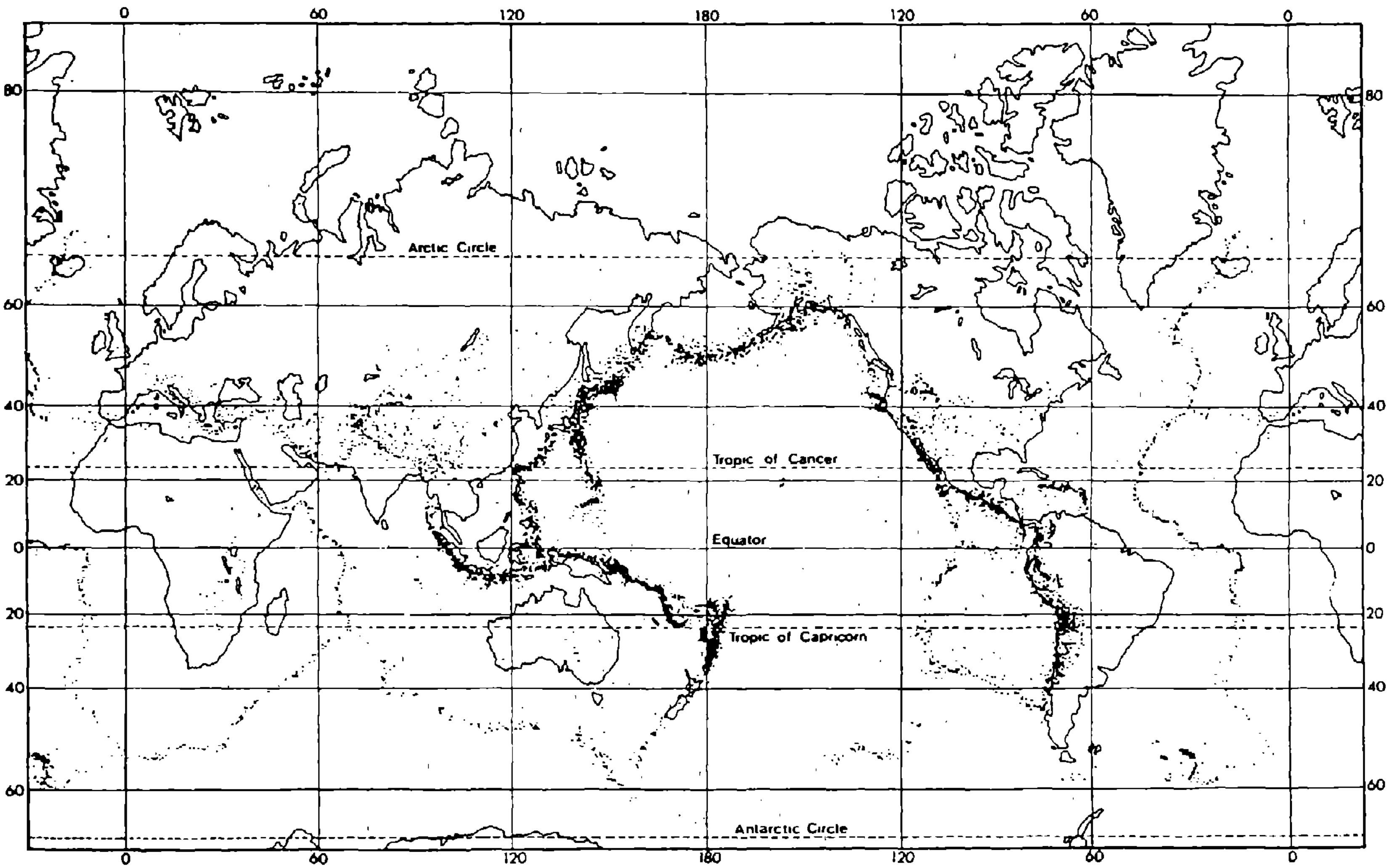


Figure 2-2 - Worldwide Earthquake Distribution

TABLE 2-1

Modified Mercalli Intensity Scale

I	Not felt by people, except under especially favorable circumstances.
II	Felt only by persons at rest on the upper floors of buildings. Some suspended objects may swing.
III	Felt by some people who are indoors, but it may not be recognized as an earthquake. The vibration is similar to that caused by the passing of light trucks. Hanging objects swing.
IV	Felt by some people who are indoors, by a few outdoors. At night some people are awakened. Dishes, windows and doors are disturbed; walls make creaking sounds; stationary cars rock noticeably. The sensation is like a heavy object striking a building; the vibration is similar to that caused by the passing of heavy trucks.
V	Felt indoors by practically everyone, outdoors by most people. The direction and duration of the shock can be estimated by people outdoors. At night, sleepers are awakened and some run out of buildings. Liquids are disturbed and sometimes spilled. Small, unstable objects and some furnishings are shifted or upset. Doors close or open.
VI	Felt by everyone, and many people are frightened and run outdoors. Walking is difficult. Small church and school bells ring. Windows, dishes and glassware are broken; liquids spill; books and other standing objects fall; pictures are knocked from the walls; furniture is moved or overturned. Poorly built buildings may be damaged, and weak plaster will crack.
VII	Causes general alarm. Standing upright is very difficult. Persons driving cars also notice the shaking. Damage is negligible in buildings of very good design and construction, slight to moderate in well-built ordinary structures, considerable in poorly built or designed structures. Some chimneys are broken; interiors and furnishings experience considerable damage; architectural ornaments fall. Small slides occur along sand or gravel banks of water channels; concrete irrigation ditches are damaged. Waves form in the water and it becomes muddied.
VIII	General fright and near panic. The steering of cars is difficult. Damage is slight in specially designed earthquake-resistant structures, considerable in all well-built ordinary buildings. Poorly built or designed buildings are damaged; interiors experience heavy damage. Frame houses that are not properly bolted down may move on their foundations. Decayed pilings are broken off. Trees are damaged. Cracks appear in wet ground and on steep slopes. Changes in the flow or temperature of springs and wells are noted.
IX	Panic is general. Interior damage is considerable in specially designed earthquake-resistant structures. Well-built ordinary buildings suffer severe damage with partial collapse; frame structures are thrown out of plumb or shifted off their foundations. Unreinforced masonry buildings collapse. The ground cracks conspicuously and some underground pipes are broken. Reservoirs are damaged.

X Most masonry and many frame structures are destroyed. Specially designed earthquake-resistant structures may suffer severe damage. Some well-built bridges are destroyed and dams, dikes and embankments are seriously damaged. Large landslides are triggered by the shock. Water is thrown onto the banks of canals, rivers and lakes. Sand and mud are shifted horizontally on beaches and flat lands. Rails are bent slightly. Many buried pipes and conduits are broken.

XI Few, if any, masonry structure remain standing. Other structures are severely damaged. Broad fissures, slumps and slides develop in soft or wet soils. Underground pipelines and conduits are put completely out of service. Rails are severely bent

XII Damage is total, with practically all works of construction severely damaged or destroyed. Waves are observed on ground surfaces, and all soft or wet soils are greatly disturbed. Heavy objects are thrown into the air, and large rock masses are displaced.

In contrast to the Mercalli Scale, the Richter Scale is a quantitative instrumental measurement of the magnitude, or strength of an earthquake on a logarithmic scale. The largest magnitude ever measured on the Richter Scale is 8.9, and the lowest magnitude that can be felt by people is about 2. Lower magnitudes, even negative ones, can be measured by instruments. (Negative earthquake magnitudes are possible because of the logarithm of a number less than one is negative.) An increase of magnitude of one, say, from 6 to 7, corresponds to an increase in total energy release by a factor of about 30. The logarithmic nature of the Richter Scale is often overlooked by the general public and news reporters. An approximate correlation between Richter Magnitude and Practical intensity is provided in Table 2-2.

TABLE 2-2

Correlation Between Richter Magnitude and Practical Intensity

Richter Magnitude	Practical Intensity
1	Detectable only by instruments
2	Barely perceptible even near epicenter
4.5	Detectable within 20 miles of epicenter
6	Moderately destructive
7	A major earthquake
8	A great earthquake

The following paragraphs summarize the effects of five significant events that occurred in the U.S. within the past 20 years, highlighted by the Loma Prieta event of 1989 in which 62 people died and $6 billion damage was inflicted in the area. These paragraphs review the performance of various kinds of facilities, with property damage, and other adverse effects noted. Also, consideration is given to possible mitigative efforts that, if in place at the time of the event, could have reduced the magnitude of the adverse consequences.

2.2 Anchorage (1964)

At 5:36 p.m. on Good Friday, March 27, 1964, Anchorage and all southern Alaska within a radius of about 400 miles of Prince William Sound was struck by perhaps the strongest earthquake to have hit North America within historic time. The magnitude of this great quake has been computed by the U.S. Coast and Geodetic Survey at 8.5 on the revised Richter scale. Its epicenter was about 80 miles east-southeast of Anchorage near the head of Prince William Sound. Reportedly, the quake was felt throughout most of Alaska, including such remote points as Cape Lisburne, Point Hope, Barrow, and Umiat, 600 to 800 miles north of the epicenter on the Arctic Slope of Alaska, and at Fort Randell, 800 miles south-west at the tip of the Alaska peninsula.

The duration of the earthquake at Anchorage can only be surmised owing to the lack of strong-motion seismograph records. Although seismographs have since been installed, none were present in Southern Alaska at the time of the quake. Intense seismic motions seem to have lasted 3 to 4 minutes, possibly longer. Where localized ground displacements occurred, as in or near landslides, strong motions may have lasted appreciably longer, after strong seismic shaking had ceased. The durations at Anchorage, timed by several eye witnesses on wrist or pocket watches, ranged from 4 minutes 25 seconds to 7 minutes. Even longer durations were reported outside the Anchorage area. In some areas people reportedly were thrown to the ground by the force of the acceleration and were unable to regain their footing.

Total earthquake damage to property in the Anchorage area could not be fully evaluated and perhaps will never be fully known. Nine lives are reported to have been lost - five in the downtown area, three at Turnagain Heights, and one at the International Airport. In less than 5 minutes, more than 2,000 people, including apartment dwellers, were rendered homeless, according to press estimates. The loss of life was less in Anchorage than in some of the small coastal towns, where many people were killed by sea waves. But Anchorage, because of its much greater size, bore the brunt of the property damage and property losses reportedly were greater there than in all rest of Alaska combined.

Early estimates of total damage, tended to be larger than later ones. According to the Anchorage Daily Times of April 9, 1964, 215 homes were destroyed in Anchorage and 157 commercial buildings were destroyed or damaged beyond repair. At Turnagain Heights alone, 75 or more dwellings were destroyed. The final total damage estimate for Alaska, exclusive of personal property and loss of income, was about $311 million. Scores of buildings throughout Anchorage sustained damage requiring repairs costing many thousands of dollars.

Roads and railroad facilities were badly damaged. In the downtown area, many streets were blocked by debris, and in landslide areas, streets and roads were completely disrupted. Differential settlement caused marginal cracking along scores of highway fills throughout the Anchorage Lowland. In the Alaska Railroad yards where landslide debris spread across track and damaged or destroyed maintenance sheds, an estimated $2,370,700 damage was sustained. Cars and equipment were overturned, and car shops were damaged by vibration. Along the main line of the railroad, bridges failed, fills settled, and tracks were bent or buckled. At Potter, near the south margin of the Anchorage Lowland, several hundred feet of track was carried away in an area that has had a long history of repeated sliding.

At the Anchorage International Airport, the control tower failed under seismic vibration and collapsed to the ground, killing one occupant and injuring another. The airport terminal building, although tied structurally to the tower, was only slightly damaged.

Damage was caused by direct seismic vibration, by ground cracks, and by landslides. Direct seismic vibration affected chiefly multistory buildings and buildings having large floor areas, probably because of the long period and large amplitude of the seismic waves reaching Anchorage. Most small buildings were spared. Ground cracks caused capricious damage throughout the Anchorage Lowland. Cracking was most prevalent near the heads or within landslides but was also widespread elsewhere. Landslides themselves caused the most devastating damage.

Triggering of landslides by the earthquake was related to the physical engineering properties of the Bootlegger Cove Clay, a glacial estuarine-marine deposit that underlies much of the Anchorage area. Most of the destructive landslides in the Anchorage area moved primarily by translation rather than by rotation. Thus, all the highly damaging slides were of a single structural dynamic family despite wide variations in size, appearance, and complexity. They slid on nearly horizontal slip surfaces after loss of strength in the Bootlegger Cover Clay. Some failures are attributed to spontaneous liquefaction of sand layers.

In most translation slides, damage was greatest in graben areas at the head and in pressure-ridge areas at the toe. Many buildings inside the perimeters of slide blocks sustained little damage despite horizontal translations of several feet. The large Turnagain Heights slide, however, was characterized by a complete disintegration and drastic lowering of the prequake land surface. Extensive damage back from the slide, moreover, was caused by countless tension cracks. Geologic evidence indicates that landslides similar to those triggered by the March 27 earthquake have occurred in the Anchorage area at various times in the past.

The very large magnitude of this earthquake, coupled with the soft, loose, and deep soil deposits, combined to produce severe damage that would have been difficult to prevent with reasonable engineering and construction procedures. The extensive mass soil movements reflected the vulnerability of the foundation materials. The town of Valdez, only 45 miles from the epicenter was essentially destroyed and, after some consideration, was rebuilt on a different, more stable area. Still, much was learned from this event, since many small structures, that were part of mass slides, or other large lateral movements, were not structurally damaged, though because of excessive movement and damage around them (tension cracks in the ground and loss of utilities) their useful function was lost.

Some examples of the damage sustained in this earthquake are shown in Figures 2-3, 4 and 5.

Figure 2-3 - Damage to Air Traffic Control Tower, Anchorage (1964)

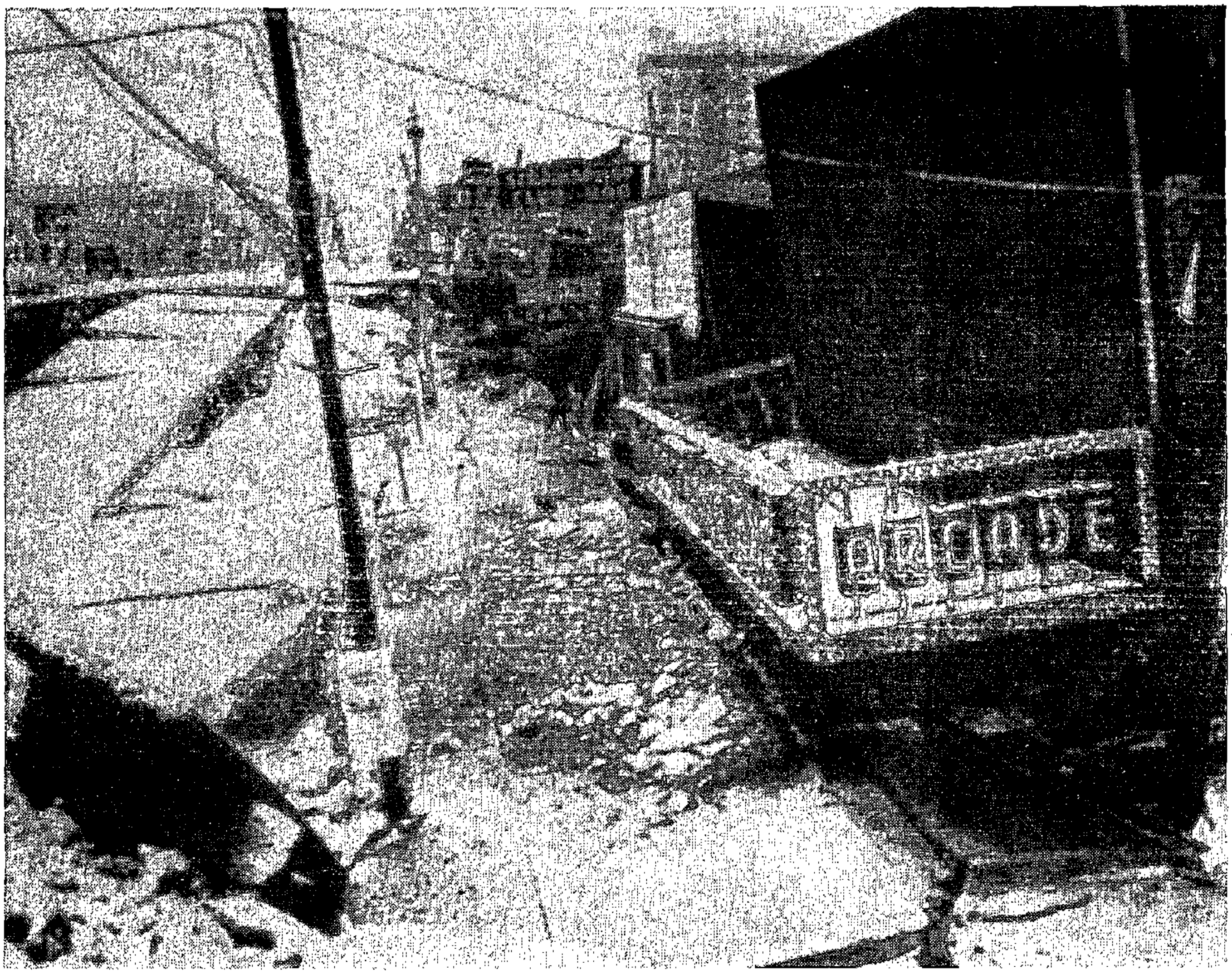


Figure 2-4 -Ten Foot Subsidence of Street at Head of Landslide, Anchorage (1964)

Figure 2-5 -Damage to Department Store Building, Anchorage (1964)

2.3 San Fernando (1971)

The San Fernando earthquake, with a Richter magnitude of 6.6, occurred at 6:01 A.M. on February 9, 1971. The earthquake's epicenter was in the San Gabriel Mountains located north of Los Angeles.

The earthquake caused 58 deaths, (47 were due to the collapse of the non-earthquake resistant Veterans Hospital), and over 2,500 hospital-treated injuries in the San Fernando Valley, which had a population of over 1,200,000 at the time of the quake.

Strong ground motion lasted 12 seconds, and peak ground accelerations as high as 1.25g (1.25 times the acceleration of gravity) were recorded in the vicinity of the Pacoima Dam. These motions were greater than any previously recorded. The damage to nearby wood frame dwellings and to hospitals indicated that the building codes needed revision.

Direct damage to buildings and other structures exceeded ½ billion dollars. This amount was divided about equally between public and private property. Most of the severe damage and major losses were along the southern foothills of the San Gabriel Mountains and along a narrow band of surface faulting that runs east-west on the valley floor.

The epicenter was close to four metropolitan freeway routes with numerous bridges. These bridges sustained heavy damage. A total of 62 bridges were damaged, mostly in a zone 5 miles long, located 6 to 10 miles from the epicenter. The observed damage identified many code deficiencies, and the earthquake resulted in profound changes to seismic code provisions.

The collapse of the Foothill Freeway overpass (Figure 2-6) was caused by inadequate support width at the girder supports. The earthquake movement caused the girders to slide off the piers. It was noted that adjacent bridges with wider supports experienced movement, but not collapse (see Figure 2-6).

Other deficiencies were noted in the reinforcing steel of pier columns. Inadequate *spiral* reinforcing, tying the vertical bars together allowed the concrete within to crumble, and the vertical bars to buckle (see Figure 2-7). Also, inadequate embedment of vertical reinforcing bars in concrete footings allowed the bars to pull out of the footing under earthquake loading (see Figure 2-8).

Serious damage also was sustained by buildings considered earthquake-resistant at the time, by dams located up-stream from densely populated areas, and by public utilities and roadways, that are the lifelines of cities (see Figure 2-9).

Buildings that survived the earthquake without collapse met the intent of the building code; however, from an economic viewpoint, many may be considered failures. Facilities that were undamaged or only slightly damaged were able to quickly reopen. Facilities such as Olive View Hospital (Figure 2-10), Pacoima Memorial Lutheran Hospital, and Holy Cross Hospital suffered major damage and required years to fully recover. The resulting loss of market share and revenue far exceeded the property losses.

The San Fernando earthquake, although moderate in energy release and in amount of surface rupture, led to post-earthquake studies that provided significant new data and information concerning the effects of an earthquake on bridges, building structures, on the operations and services of public utilities, and transportation facilities. Human reactions and response to an earthquake, emergency in a metropolitan area, engineering problems related to soils and foundations, and man's knowledge and adjustments to the seismic, geologic and geodetic features of the physical environment were also studied. Following the San Fernando earthquake, CALTRANS initiated a seismic upgrading of bridges and other vulnerable structures. As a result of this quake, bridge and building codes were revised to provide more effective seismic-resistant design, and the seismic safety of dams in California was reexamined.

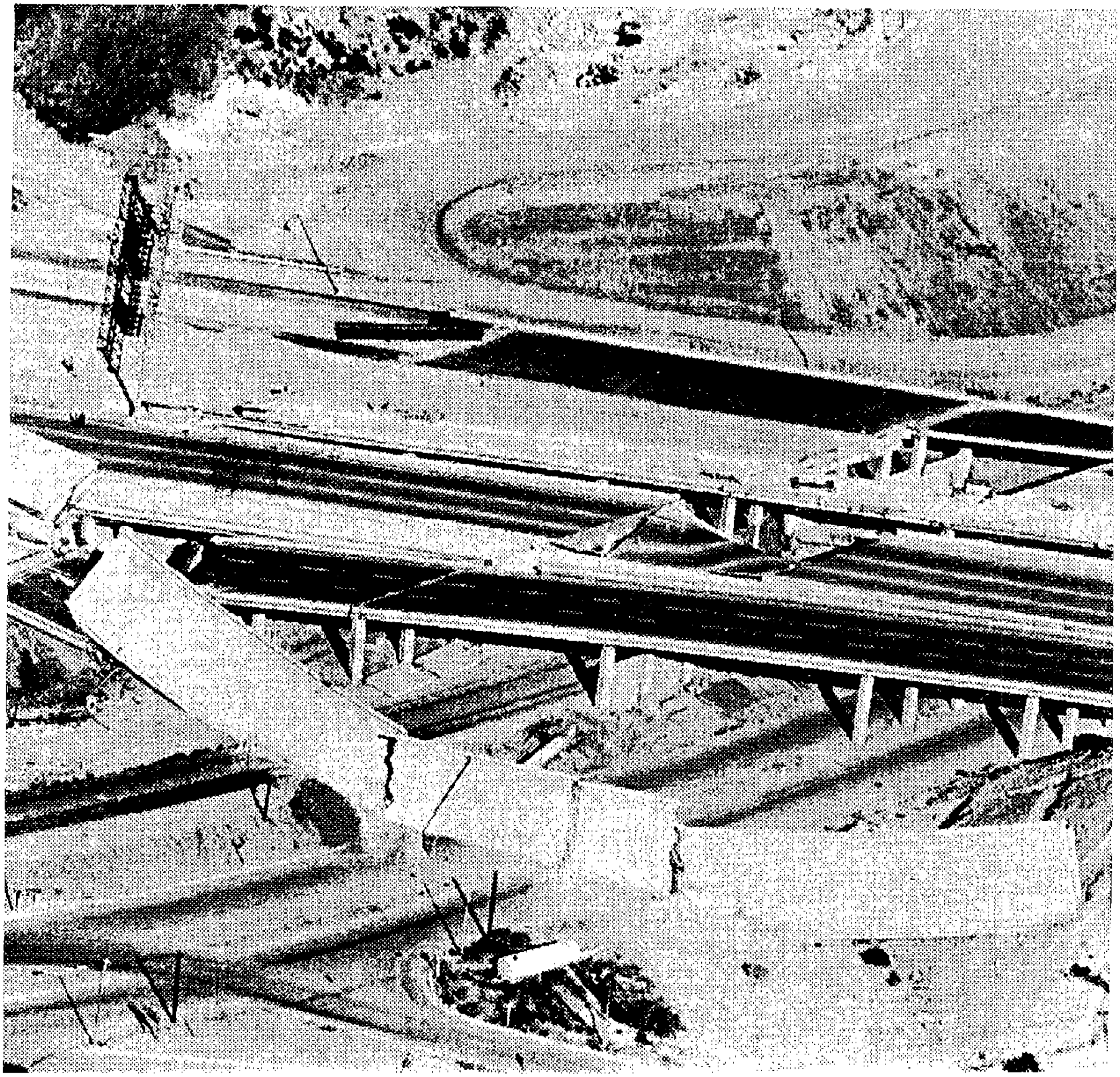


Figure 2-6 - Structural Damage, Foothill Freeway Overpass, San Fernando (1971)

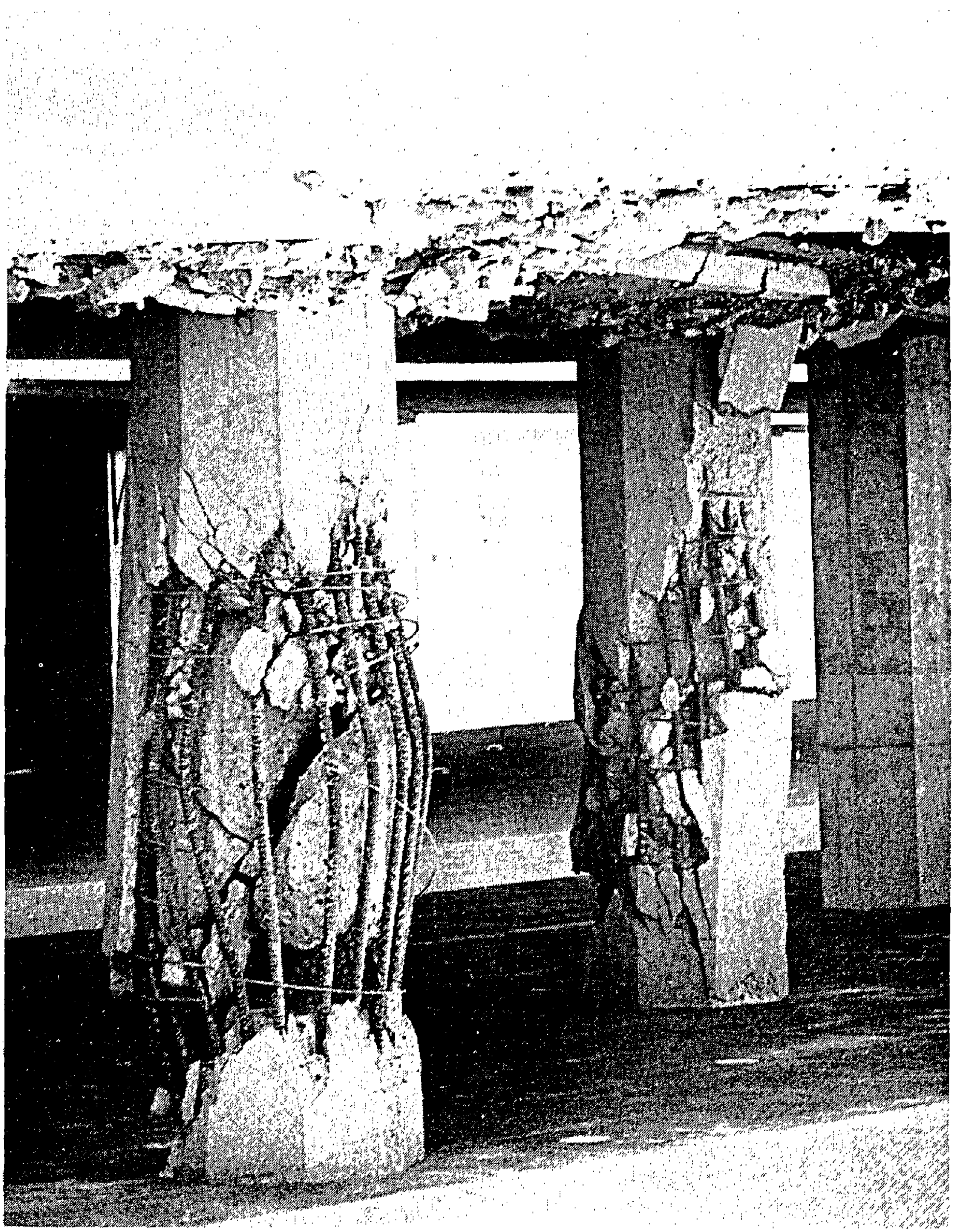


Figure 2-7 - Structural Damage, San Diego Freeway, San Fernando (1971)

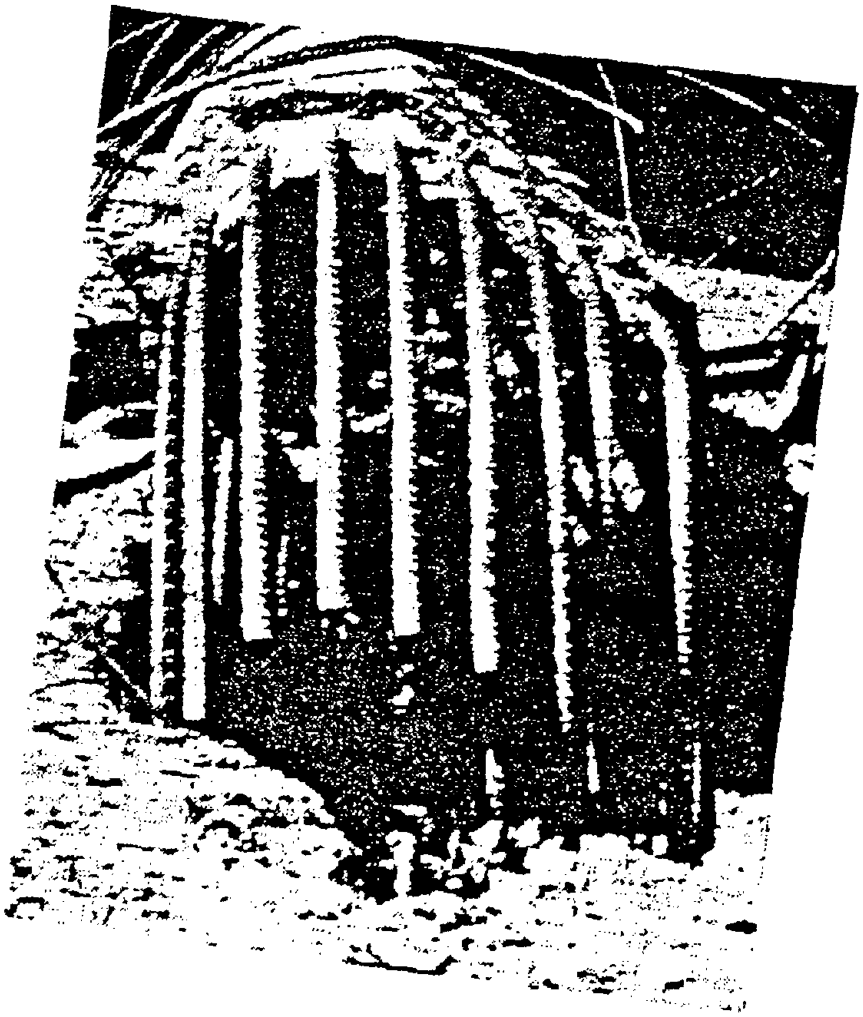


Figure 2-8 - Structural Damage, Route 210/5 Interchange, San Fernando (1971)

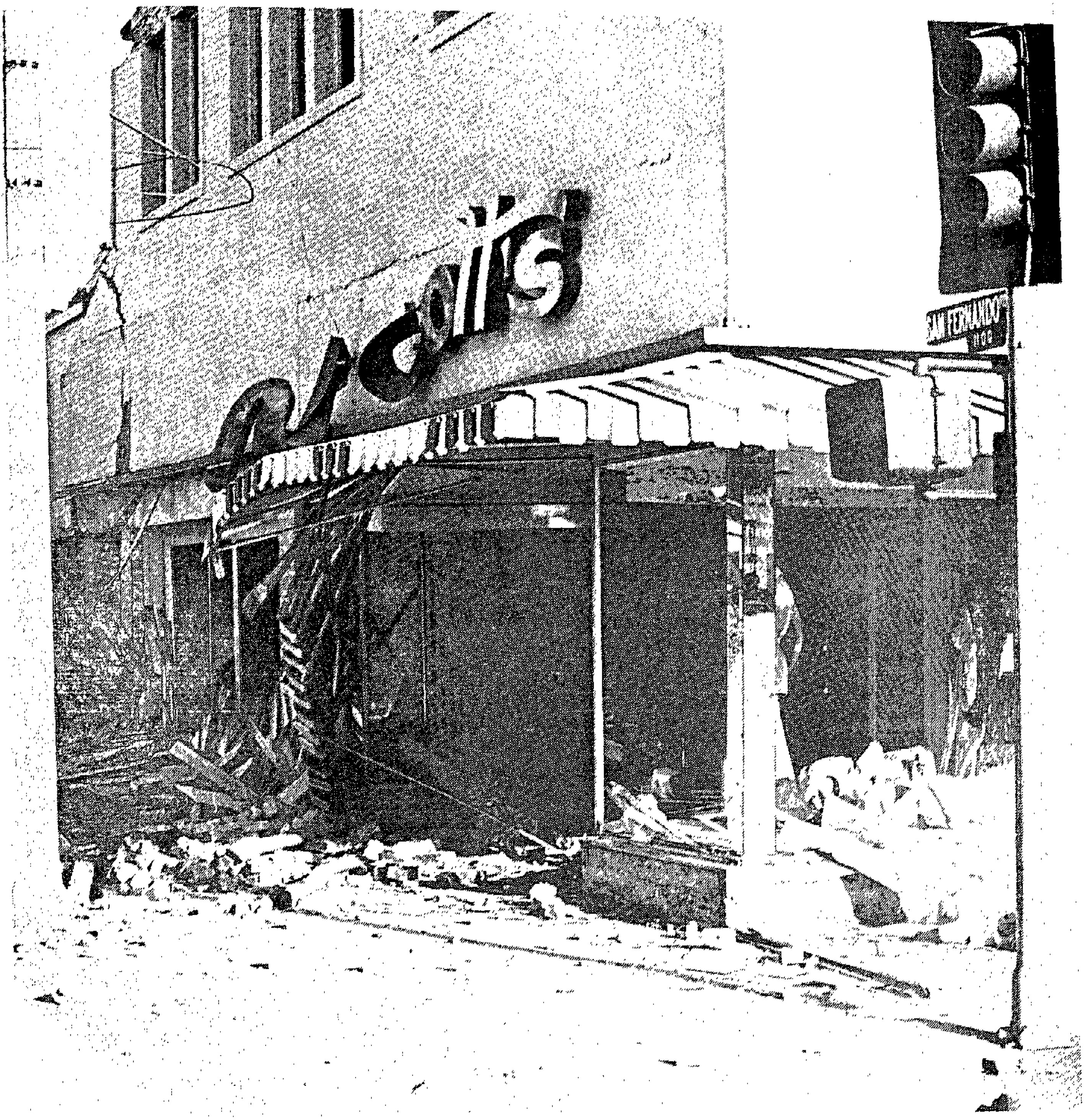


Figure 2-9 - Building Damage, City of San Fernando, San Fernando (1971)

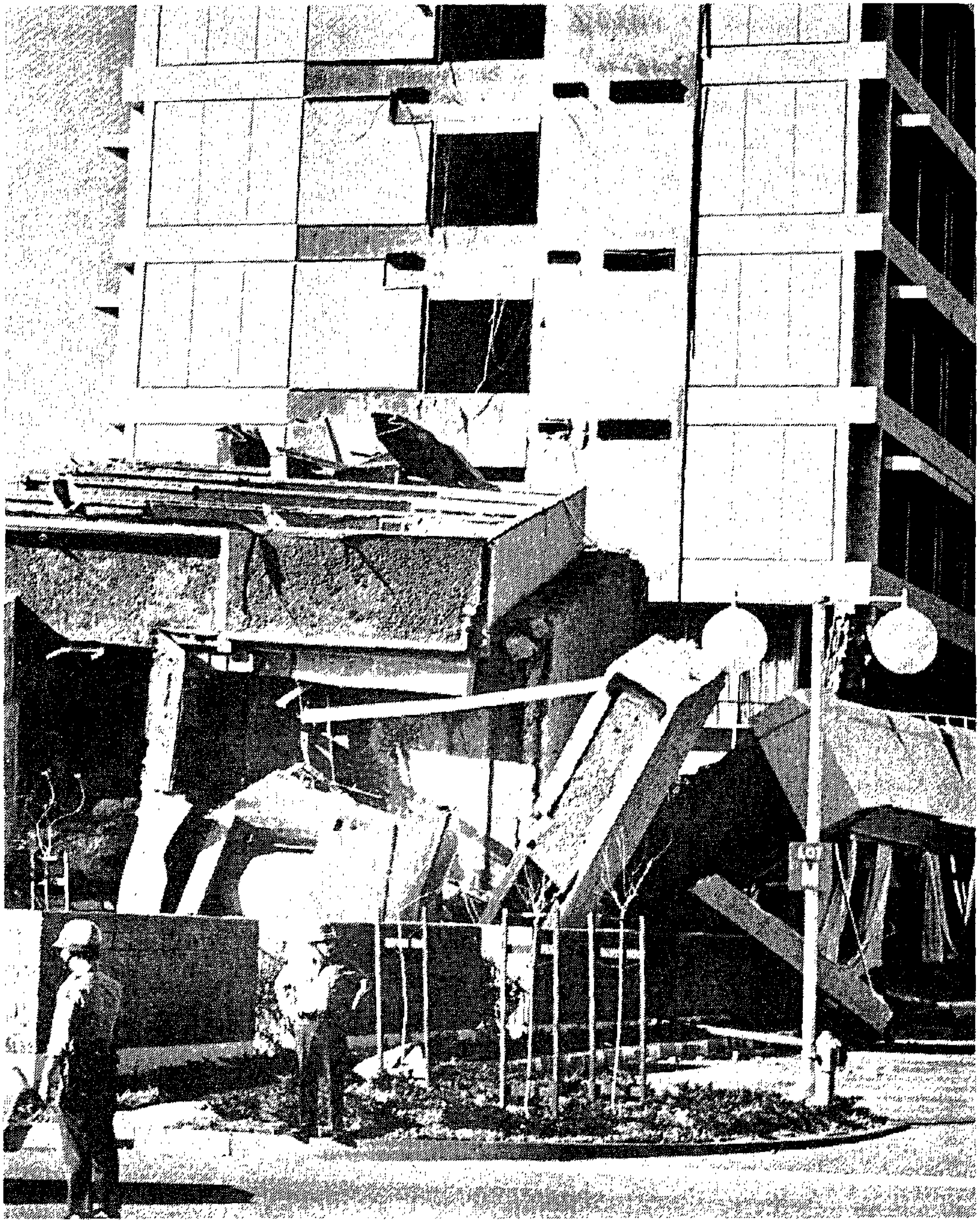


Figure 2-10 - Damage at Olive View Hospital, San Fernando (1971)

2.4 Northern Kentucky (1980)

Shortly before 3 P.M. on July 27, 1980, an earthquake struck near Sharpsburg, Kentucky, approximately 31 miles northeast of Lexington. The earthquake was followed by about 30 aftershocks centered in Sharpsburg, with several in the surrounding hills. Although this was only a moderate seismic event, several of its features were unusual. It was the largest earthquake in at least 200 years in that area, and the worst damage occurred to structures located in the town of Maysville, some 30 miles from the epicenter.

The area in which the earthquake occurred in a region, in which the frequency and relative size of earthquakes are generally considerably less than in other areas of the southern U.S.

The magnitude of the July 27 event was 5.3 on the Richter scale. According to available information on earthquake history, no event of comparable size has been recorded in this area during the last 200 years. The earthquake was felt from Toronto, Canada, to the Gulf Coast.

Although there was a significant amount of damage to structures in Maysville, estimated at approximately $1,000,000, and lesser amounts in towns near the epicenter of the earthquake, there were no structural failures. 86 businesses and residences sustained major damage (loss exceeding $5,000), and 220 structures suffered minor damage.

Many of the buildings damaged were built either in the late eighteen hundreds or early nineteen hundreds. Damage consisted primarily of cracked chimneys, cracked masonry walls and plastered ceilings, separated walls at the roof line of buildings, cracks and bulges in concrete slabs on grade, and broken windows. There did not appear to be damage to non-structural building systems. Except in Maysville, where some chimneys were broken off near the roof line, the majority of damage occurred at the top of the older chimneys from dislodged bricks. Cracks in masonry walls appeared to be typical stress concentration cracks, normally found at corners of wall openings. Modern construction survived the earthquake quite well.

The importance of this earthquake lies in the fact that it occurred in an area that, historically, had not experienced much earthquake activity. The area was considered one of low seismic risk. There are many areas that, today are considered to have low seismic risk. The Sharpsburg earthquake taught us, however, that there are no areas that are completely immune and that seismic design is important to all areas of the country, no matter how low the perceived risk. The extent and type of damage caused by the earthquake upon older buildings led to arguments favoring greater earthquake resistance in buildings throughout the nation.

Kentucky, like many other regions of the country, historically has disregarded consideration of earthquake forces in the design and construction of their structures. This is explainable because these regions and the people personally have not before felt an earthquake. The construction in northern Kentucky seems to be representative of construction in many other parts of the eastern United States. An extensive amount of structures are old, maintenance varies from neglect to good, and earthquake forces have not been considered very much. Attention has to be focused in these areas to the vulnerability of these buildings to the occasional damaging earthquakes, and more responsible maintenance and construction practices should be introduced.

2.5 Whittier (1987)

At 7:42 A.M. on Thursday, October 1, 1987, a magnitude 5.9 earthquake occurred east of Los Angeles near the city of Whittier, California. There were numerous aftershocks, the largest being a magnitude 5.5 in the early hours on October 4, which caused further damage to structures weakened by the main shock.

The earthquake occurred along a previously unrecognized fault at the northwestern end of the Puente Hills. No surface rupture due to fault movement was evident. Peak ground accelerations as high as 0.45g were recorded. Strong ground motion was recorded over a wide area. For example peak accelerations of 0.40g were observed in downtown Los Angeles 12 miles from the epicenter. The total duration of significant ground motion was about 15 seconds, but strong ground motion lasted only about 5 seconds.

Four persons were killed and numerous heart attack deaths were also attributed to the quake. Hospitals across the Los Angeles basin treated a total of 1,349 earthquake-related injuries on Oct. 1 and after the largest aftershock on Oct. 4. Estimates of property damage to public and private structures approached $350 million.

The earthquake effects in different communities varied, depending on local subsurface conditions. Areas underlain by more recent, loose, fine-grain soils suffered heavier damage, while areas founded on rock were less strongly affected. In general, modern, engineered buildings performed well in this earthquake, with few well-designed structures experiencing significant damage. The most vulnerable were unreinforced brick buildings, older wood frame, pre-cast concrete tilt-up, and older non-ductile (brittle) reinforced concrete structures. This illustrates once again that these types of buildings have the least strength to resist even moderate, short duration shaking.

2.6 Loma Prieta (1989)

On October 17, 1989, at 5:04 P.M., a magnitude 7.1 earthquake struck the San Francisco Bay area. The epicenter of the earthquake was 60 miles south of San Francisco in the Santa Cruz mountains. The devastating ground shaking produced by the earthquake lasted for approximately 10 seconds and was felt as far away as San Diego and western Nevada. This earthquake was the largest in Northern California since the 1906 great San Francisco earthquake, (magnitude 8.3), and can be ranked as one of the more costly natural disasters in California history, if not the United States.

Seismic shaking, which affected a region of more than 400,000 square miles from Los Angeles northward to the Oregon border, was triggered by rupture of the crust along 25 miles of the southern Santa Cruz Mountain segment of the San Andreas fault.

The Loma Prieta earthquake and its aftershocks resulted in widespread damage to a variety of structures over an area of approximately 3,000 square miles. The California Governor's Office of Emergency Services estimated the damage as follows :

- 62 deaths (42 were caused by the collapse of the multiple-deck Cypress viaduct in Oakland)
- 3,757 injuries.
- the San Francisco Bay Bridge was unusable for 1 month
- over $6 billion property damage
- number of homes damaged: 18,306
- approximately 12,000 people were at least temporarily displaced from their homes.
- 376 businesses were destroyed, 2,575 were damaged

Most buildings of seismically resistant construction, built according to recent building codes survived the earthquake with little damage, typically limited to cosmetic damage to cladding and partitions, and disarray of contents. Older structures were hardest hit, with failure of many unreinforced masonry and some reinforced concrete buildings throughout the effected area (Figure 2-11). The expensive real estate development in San Francisco's Marina District was heavily damaged, caused by locally amplified shaking and by permanent deformation of the ground due to liquefaction of the sands and debris used to fill the former lagoon. Figure 2-12 shows a badly damaged apartment building in the Marina District where three people died.

Damage to the area's transportation infrastructure was extremely heavy. The earthquake caused the collapse of a 50 ft section on the upper deck of the Bay bridge, the collapse of a 3,970 ft section of the Cypress structure, major damage to several bridges, and minor damage to over 100 other bridges (ISSULF 1990). Figure 2-13 illustrates the collapsed section of the I-880 Cypress Structure.

Underground structures performed well. Of particular note, the BART underground transit system, survived the earthquake with almost no damage.

2.7 Conclusion

The preceding paragraphs indicate two things. They indicate the type of damage that can occur in an earthquake, but more importantly, they indicate that earthquakes can occur anywhere.

Table 2-3 lists the effects of some recent larger earthquakes occurred in North America. These estimates give an idea of the scale of damage incurred in past earthquakes, and what can be expected from future earthquakes.

It is acknowledged that earthquakes will occur in the future. It is understood what type of damage can be expected from the various types of construction. It is well documented how to minimize earthquake damage through proper design and retrofit practices. What cannot be predicted is where future earthquakes will occur. It could be anywhere. Facility managers would certainly be well advised to take aggressive action to ensure that their facilities are designed and constructed appropriately to survive a seismic event.

Figure 2-11 - Damage to Masonry Buildings in Oakland, Loma Prieta (1989)

Figure 2-12 - Damage to Apartment Building in San Francisco, Loma Prieta (1989)

Figure 2-13 - Collapsed Upper Deck of the I-880 Cypress Structure, Loma Prieta (1989)

Event	Date of Occurance	Magnitude on Richter Scale	Distance from Major City (Miles)	Duration (Seconds)	Maximum Ground Acceleration (Ft./Sec.^{2})	Deaths	Injuries	Temporary Displace-ments	Property Damage (Millions)
Loma Prieta, CA	10/17/89	7.1	60-San Francisco	15	1.00g	62	3,757	12,000	$350
Whittier, CA	10/1/87	5.9	250-Mexico City	15	0.45g	4	1,349	10,359	
Southeastern Illinois	6/10/87	5.6	125-St. Louis						
Northeastern Ohio	1/31/86	5.0	25-Cleveland		0.18g				
Mexico City	9/19/85	8.1	250-Mexico City	180	0.20g	8,000		40,000	
Coalinga, CA	5/2/83	6.7			0.59g				$500
Eureka, CA	11/8/80	7.1							
Northern Kentucky	7/27/80	5.3	31-Lexington	30	0.05g				$1.5
Imperial County, CA	10/15/79	6.6	16-Caluco	11.8	1.74g	0	0	0	$30
San Fernando, CA	2/9/71	6.6	80-Los Angeles	12	1.25g	58	2,500		
Prince William Sound, AL	3/27/64	8.4	75-Anchorage	Several Minutes	0.25g	125			

Table 2-3 - Damage Summary - Recent U.S. and Mexico City Earthquakes

3.0 EARTHQUAKE HAZARDS

3.1 General

The term *earthquake hazards* refers to the probability of occurrence of an earthquake in a certain area, as well as its likely intensity. Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes. It is important to understand the hazard associated with a site in order to know the earthquake dangers to which facilities in the area might be exposed.

This section gives some background on seismic hazards, and provides a method for determination of hazard for a given location. This subject is covered in more detail in Appendix A.

3.2 Potential Impacts From Earthquakes

The impacts from earthquakes of most concern in this study concern property damage and loss of life. Historically, the most dangerous aspects of seismic activity have been the effects earthquakes have had on man-made structures. Most deaths in recent times have not been the direct result of the earth's actions, such as ground shaking, earth rupture, volcanic eruption, or tidal waves, but the result of the failure of the man-made structures within which people live and work. Failure of infrastructure facilities such as buildings, highways, and transit systems, exposes the population to direct risks of injury and death. In addition, there are longer term social problems associated with disruption of communications, vital services, and the damage to utilities.

Earthquakes can cause damage in a number of ways. Damage to facilities occur through *primary, secondary,* and *tertiary* hazards. Primary hazards are those which can be directly related to the earthquake. They include such phenomenon as ground vibration and fault rupture. Secondary hazards are those potentially dangerous situations triggered by the primary hazards. These include foundation settlement, landslides, soil liquefaction, and tsunamis. Tertiary hazards result from structural damage caused by the primary and secondary hazards and are often the most serious. These include such events as flooding due to dam failure or fire following an earthquake. In fact, most of the property damage in the 1906 San Francisco earthquake was due to the great fire, not the ground shaking itself. Primary and secondary hazards are the subject of this Section. Tertiary hazards are minimized through the accomplishment of sound seismic design, the subject of Section 5 and Appendix C.

The most common and most damaging hazards from earthquakes are as follows:

Ground Shaking: Ground shaking refers to the vibration of the ground produced by seismic waves arriving at a site. Vibrations originate in the bedrock which undergoes explosive movement, releasing the stresses built up from restrained movement at the edges of tectonic plates. The vibrations propagate up through soils overlying the bedrock, causing ground shaking at the surface. The soil undergoes an oscillating acceleration as it moves back and forth. Man-made structures are supported on foundations built in or on soil or rock mass. Movement of the soil or rock, and therefore of the foundations, results in movement of the structure. The associated accelerations induce forces within the structure. Structural damage occurs when the forces exceed the capacity of the structural members. If the damage is extensive enough and the member is critical to the integrity of the structure, total failure or collapse results.

Ground Displacement: Ground displacement causes structural displacement which must be accommodated with allowance for shake spaces between buildings, etc. For underground structures such as tunnels, however, ground displacement is of primary concern. As the soil mass vibrates back and forth it "racks" or tilts. The structure must be capable of assuming this shape.

Surface Faulting: This is the offset or tearing of the ground surface by differential movement across a fault during an earthquake. For long linear systems, typical of many transportation facilities, surface faulting is an important potential hazard that must be considered.

Landslides: Steep earth slopes that are stable under static loads can become unstable during earthquakes. Seismic lateral loads can be sufficient to overcome the internal frictional forces within the soil mass that keep it in place under static loads causing a sliding failure. Facilities located within the effected zone either above, or below such a slope are threatened by this potential hazard.

Liquefaction: Loose sandy soils with high ground water levels can *liquefy* during dynamic earthquake loading. This phenomenon occurs when water trapped within the voids between soil particles prevents the compaction and settlement that the soil would otherwise undergo as it is shaken in an earthquake. The soil particles lose contact with each other and the soil mass assumes a liquid-like state. This causes the soil to lose its capacity to support loads, resulting in foundation failure. Structures can float up, sink down, or tilt over depending on the magnitude and distribution of loads.

Settlement: Fill, or loose soils can densify during ground shaking, causing dramatic settlement. Structures located in these areas are susceptible to damage.

Failure from surface faulting, liquefaction, landslides and settlement can often be avoided in new structures with proper siting of the facility since the site's predisposition for such extreme behavior can be determined in advance. It is more difficult to remedy an existing structure already located in a deficient site with the above characteristics. Possibilities include dynamic compaction, vibroflotation, excavation and replacement of substandard soil, grouting and strengthening with long piles.

Ground shaking and displacement are accommodated through proper consideration of seismic forces and movements in the structural design of the facility. This subject is covered in Section 5.0, and Appendix C.

3.3 Hazard Maps

Hazards vary in different regions of the country. California is notorious for severe earthquakes, while there are few in the Midwest. Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes. On a large scale, the probability of an earthquake and the severity that can be expected can be predicted depending on the region. This regionality has been determined through a review of historical records and is currently presented on maps providing various ground motion parameters for the various regions of the country which categorize areas by their seismic activity and level of risk.

The parameter most commonly used to measure seismic hazard is the *peak horizontal ground acceleration coefficient*. This is the acceleration of the ground, expressed as a percentage of gravity, resulting from ground shaking during an earthquake. Typically, this acceleration coefficient indicates the estimated peak ground acceleration that statistically has a 90 percent probability of not being exceeded within 50 years. In areas of moderate seismicity such as New York City or Boston, this acceleration coefficient is approximately 0.15, while in areas of high seismicity in California, it may be higher than 0.40, which translates into significantly higher seismic design forces for structures in this region.

Although earthquakes are more probable in some areas than others, it should be recognized that all states have at least some potential for seismic activity. It is a fallacy to think that some areas are immune. Earthquakes can happen anywhere. The risk can be greatest, in fact, in those areas that historically have experienced few earthquakes where structures have been built without seismic resistant details. The consequences can be catastrophic.

There are two types of ground acceleration that are commonly used. Two types are needed to accurately characterize the intensity of design ground shaking for structures with differing characteristics:

EPA (or A_a): Effective peak acceleration coefficient. This indicates the acceleration resulting from a *near field* earthquake, with the epicenter located close to the structure site. It will generally control the design of more rigid structures (those less than 5 stories in height).

EPV (or A_v): Effective peak velocity-related acceleration coefficient. This indicates the acceleration resulting from a *far field* earthquake, with the epicenter located far from the structure site. It will generally control the design of more flexible structures (those greater than 5 stories in height).

These acceleration coefficients are plotted on maps showing, with contours, their variation across the country. These maps can be found in various forms in most building codes. Figures 3-1 and 3-2 indicate EPA and EPV, respectively from the 1991 NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings. These same maps are included in the Building Officials and Code Administrators International (BOCA) National Building Code. Similar maps are included in other codes.

It should be noted that although peak ground acceleration is the most commonly used hazards analysis parameter for above ground structures such as buildings and bridges, the peak ground velocity and peak ground displacement have been found to be useful in the evaluation of seismic hazard for underground structures. This is primarily because the seismic response of underground structures are more sensitive to earthquake - induced ground deformations than ground accelerations.

Recent geological and seismological evidence has also suggested that peak ground velocity and displacement in the Central and North-Eastern United States may sometimes be a better indicator of seismic hazard than peak ground acceleration. This is due to the unusual frequency content unique to this area.

In any event, based on the current state of the art, the hazards maps described above are a good indicator of seismic hazard for most facilities.

Regionalization maps indicating ground motion parameters have been steadily evolving since earthquake study began, and they will continue to evolve as more data becomes available.

3.4 Hazard Analysis Procedure

Seismic hazard is usually defined as the "expected occurrence of a future adverse seismic event". Often, it is mistakenly thought to be synonymous with seismic risk, which is defined as the "expected consequences of a future seismic event". Consequences may be loss of life, economic loss and the socioeconomic impact of the event on the affected region. From these definitions, it is apparent that the various regionalization maps are based on hazard analysis rather than risk analysis, although sometimes these maps are referred to as seismic risk maps.

Method for Typical Facility:

For the usual case, the regionalization maps can be used to determine earthquake hazard. Regionalization maps have been developed using a probabilistic approach to indicate the earthquake hazard for a given region. That is, they indicate an estimate of the maximum earthquake intensity that statistically can be expected with a specified probability within a specified time interval. The most widely accepted parameters indicate a 90% probability within a design life of 50 years. All of the maps described above have been developed on this basis. For example, the NEHRP EPA map (Figure A3-2), indicates that there is a 90% probability that an earthquake with an effective peak acceleration coefficient of no higher than 0.40 will occur within 50 years, in San Francisco. The acceleration coefficient is, thus, a simple representation of earthquake hazard, or the expected occurrence of a future seismic event.

Examination of the maps indicates the variation by region. *It should be noted that the latest maps indicate that there are no regions within the United States that are immune from earthquakes.* All areas have some hazard that should be considered by the transportation facility manager.

The facility manager must determine whether the level of hazard and design life inherent in the maps is appropriate for the facility(ies) in question (e.g. 90% certainty in 50 years). The maps are intended for typical buildings, bridges or other facilities. If closing a facility due to earthquake damage will disable a key transportation system, or if the facility is critical to emergency preparedness or post-earthquake recovery a lower hazard level may be appropriate. A few examples are major water crossings like the San Francisco-Oakland Bay Bridge, key interchanges, and airports. Where the typically accepted hazard levels are too low, the following method should be used.

Method for Non-Typical Facility:

In some cases the existing regionalization maps are insufficient or inappropriate for design purposes, making it necessary to perform a project specific or site specific seismic hazard analysis. These situations, among others, may be:

- design of a critical facility for which the existing hazard results are judged to be insufficient;
- revelation of new seismological and geological evidence to nullify the existing hazard results;
- design of a facility in an area adjacent to a fault.

Appendix A gives a detailed procedure for a site specific probabilistic hazard analysis.

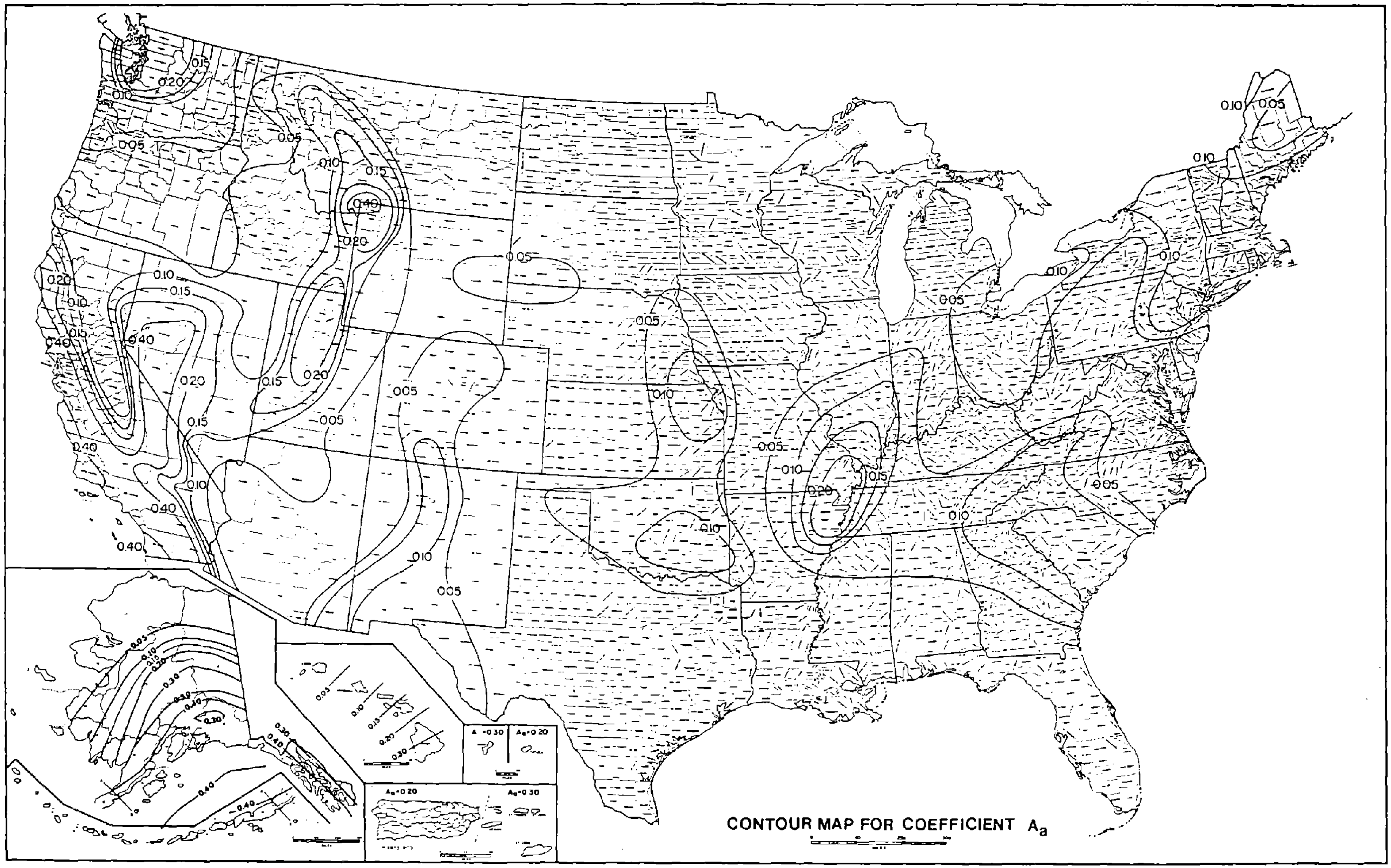


Figure 3-1 - Effective Peak Acceleration (EPA) Coefficient Contour Map

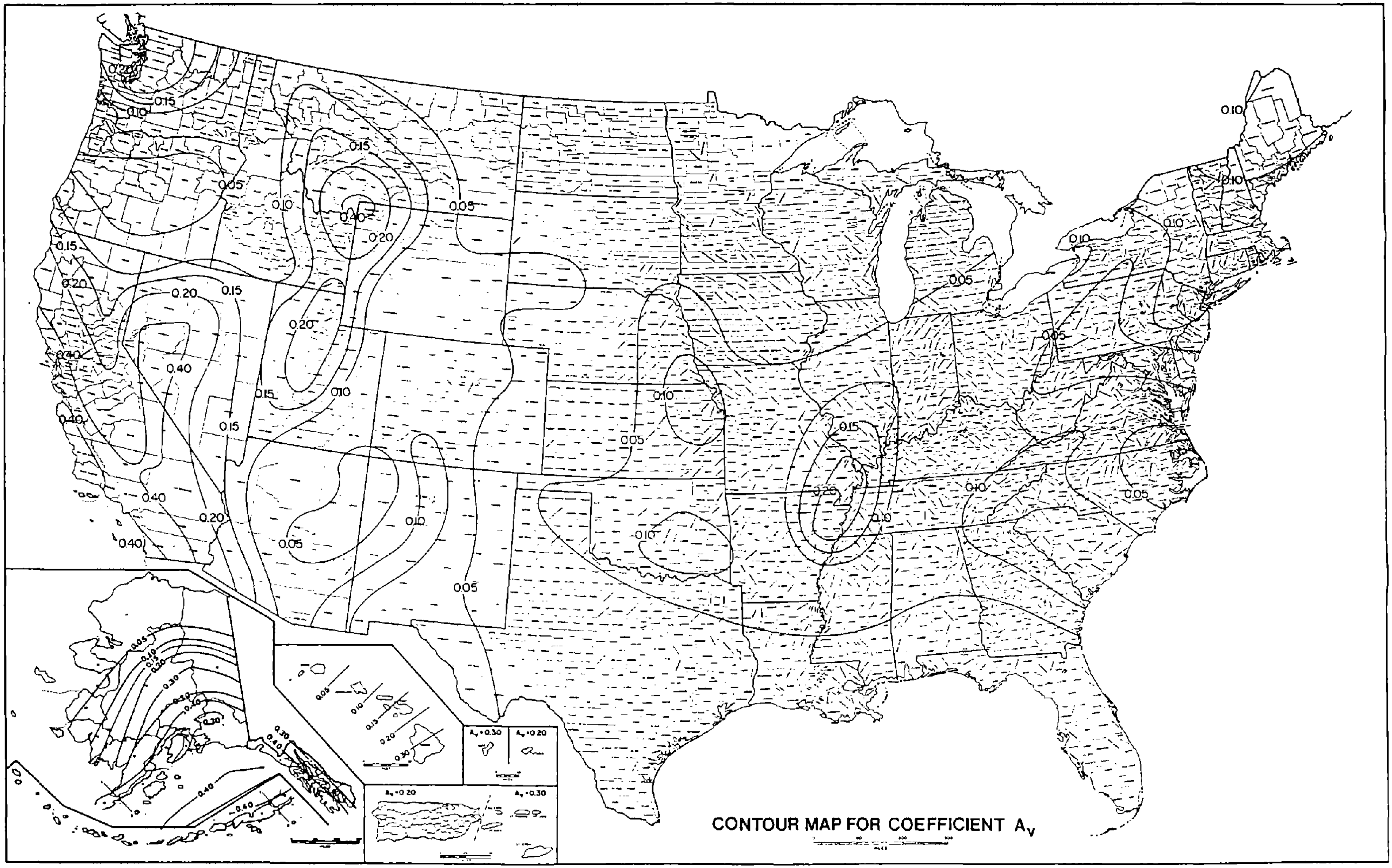


Figure 3-2 - Effective Peak Velocity (EPV) Coefficient Contour Map

4.0 SEISMIC VULNERABILITY OF TRANSPORTATION FACILITIES

4.1 General

Vulnerability here refers to the likely consequences of the expected seismic event on a particular structure. Unlike seismic hazard, vulnerability applies to specific structures. It is dependent on the expected seismic hazard, as well as the structural characteristics of the facility and the *local* geology of the site. Vulnerability is also distinguished from hazard in that hazard is a naturally occurring phenomena that man is unable to affect, while vulnerability is dependent on human factors that we have control over and can change - like the construction of a building or the slope of an earth embankment.

The task of evaluating the seismic vulnerability of transportation facilities will be considered in two steps:

1. Preliminary screening

2. Detailed evaluation

It is the goal of this section to describe broad, general guidelines to approach the first step, the preliminary screening of those facilities. However, it is our aim that those guidelines will also provide the general basis for the more detailed evaluation.

This subject is covered in more detail in Appendix B. When this procedure is used, the potential damage and loss may be estimated as a per cent of replacement value.

4.2 Transportation Facilities and Their Major Functional Components

Transportation facilities are made up of different types of structures, substructures and equipment. Several broad categories of facility types representing the major functional components of each transportation system are selected for this study. This classification, originally developed for the Applied technology Council (ATC-13 and ATC-25), is based on functional characteristics rather than structural engineering characteristics. Classifying in this way does not break down facilities directly according to vulnerability characteristics; however, it helps assess the impact from social and economic standpoints, and it helps the transportation facility manager identify the category to which his facility belongs. It serves its purpose for initial screening of vulnerability. A more refined breakdown is necessary for the detailed study.

Transportation facilities can be broken down into the following major functional components:

1. Highway Transportation System

   - Major Highway Bridges
   - Conventional Highway Bridges
   - Highway Tunnels
   - Freeways/Conventional Highways
   - Local Roads

2. Railway Transportation System

   - Railway Bridges

- Railway Tunnels
- Railway Track and Roadbeds
- Railway Terminal Stations

3. Transit System
   - Heavy Rail (Similar to railway system)
   - Light Rail (Bridges similar to highway bridges)
   - Terminals
   - Garage
   - Platforms

4. Air Transportation System
   - Airport Terminals (including Control Towers)
   - Airport Runways and Taxiways

5. Sea/Water Transportation System
   - Ports and Harbors
   - Cargo Handling Equipment

4.3 Seismic Vulnerability Characteristics

Factors that may affect a structure's or its elements' vulnerability potential to earthquake hazards include the following:

- Construction material
- Structural geometry and configuration
- Load-resisting system (framing system)
- Age
- Construction quality
- Design standard, or building code to which the structure was built
- Soil foundation material (ground condition)

Past experience indicates that design, construction quality and structural detailing play a major role in seismic performance of structures. The factors presented above should all be considered in a vulnerability assessment. Some considerations in identifying vulnerability characteristics for major transportation components are outlined in Appendix B.

4.4 Seismic Vulnerability Assessment Procedure

4.4.1 Preliminary Screening for Seismic Vulnerability

In order to provide a quantitative assessment it is necessary to establish a procedure to estimate the damage and the consequent losses for a given facility or structure exposed to a certain seismic environment.

Unfortunately, while there are a great deal of earthquake performance data, actual quantified earthquake damage and loss data are limited. One way to develop this required data is to draw on the experience and judgment of earthquake engineers. This approach has been used in a study funded by FEMA to produce an earthquake damage evaluation data base for California (ATC-13). This valuable data base, and therefore the approach, has since been used by others to conduct seismic vulnerability and loss study at a nationwide level (ATC-25) and regional level (Massachusetts Civil Defense Agency, 1990).

It is recommended that this methodology be followed by transportation facilities managers to perform an initial screening of their facilities to determine their vulnerability to seismic events. This methodology is described in detail in Appendix B, and is summarized as follows:

1. **Quantify Seismic Hazard (MMI):**

   The purpose of this task is to identify the earthquake shaking characterization that is most appropriate for estimating earthquake damage and losses. The Modified Mercalli Intensity (MMI) scale has been selected to express the damage - ground motion relationship for facility damage evaluation. The great preponderance of available damage-motion data in the form of MMI prompted this selection.

   The scale consists of 12 categories (Table 2-1) of ground motion intensity from I (not felt) to XII (total damage). Several relationships between the MMI and the peak ground acceleration have been proposed in the literature as presented in Figure 4-1. Utilizing the Trifunac and Brady relationship will allow the assessment of damage/loss through the use of peak ground acceleration, available from seismic hazard maps as described in Section 2.

2. **Identify Facility Functional Component Classification:**

   Identify the functional component classification from the list presented in Section 4.2.

3. **Identify Non-Standard or Special Construction**

   Identify any deviations from the norm for the structure under consideration. The damage probabilities developed in the ATC studies apply to facilities having standard construction in California.

4. **Identify Regional Classification**

   To apply the damage probability data to regions outside of California, one must account for the variation in seismic design practice in different regions. ATC-25 suggests an approach in which the United States is divided into five regions based on the history of seismic design practice. The division is based on a NEHRP seismic map as presented in Figure 4.2 of Appendix B.

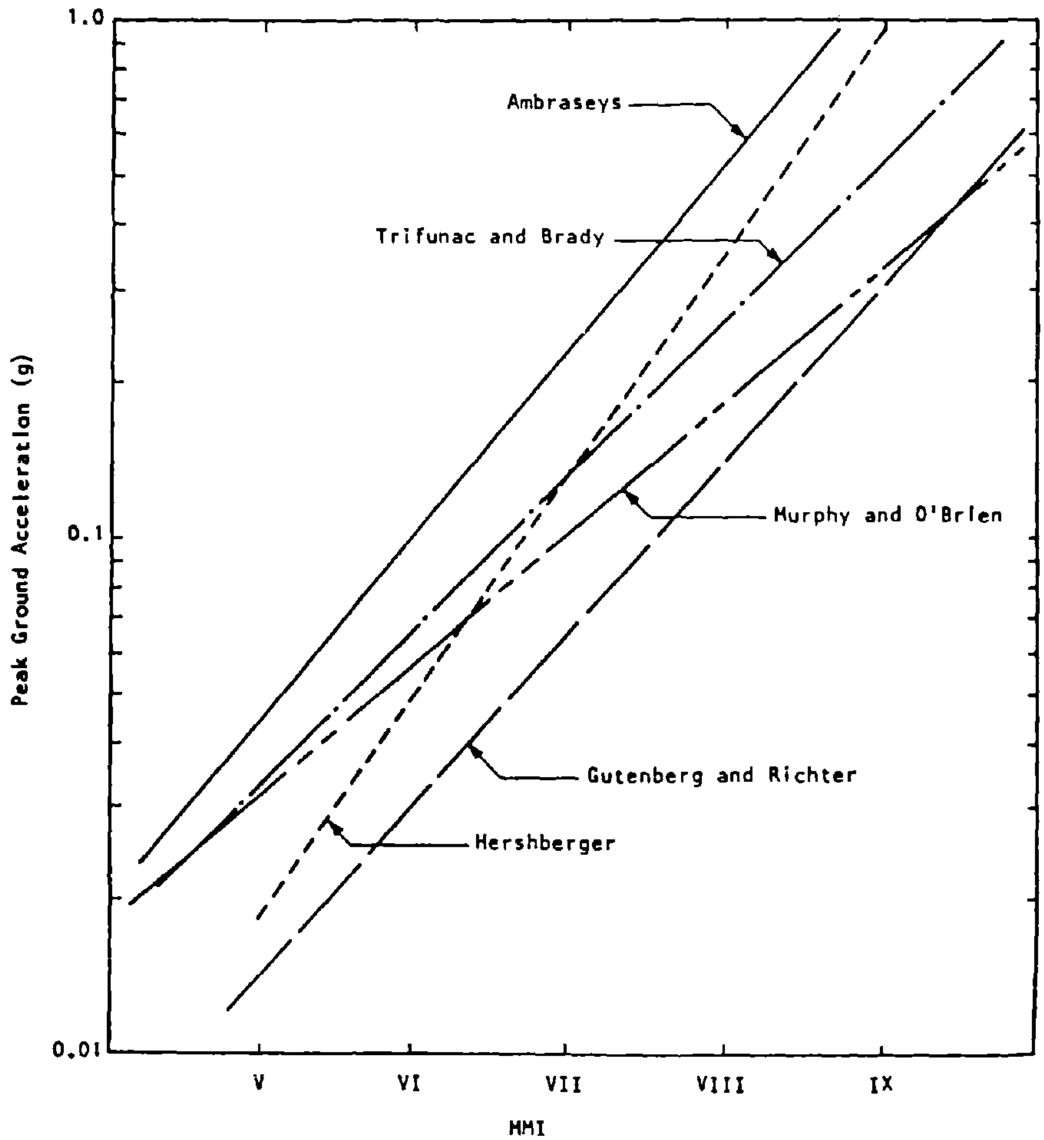


Figure 4-1 - Acceleration Versus Modified Mercalli Intensity Relationships

5. Estimate Vulnerability (Potential Damage and Loss):

Determine potential damage and loss as a percent of replacement value, using curves developed in the ATC-13 and ATC-25 studies (See Appendix B, Figures 4-3 through 4-14). This percentage is read from the curves, given facility type, MMI intensity level and the standards for the region under which the facility was constructed as shown in Figure B4-2.

After the vulnerability has been determined as above, other factors should be considered by the transportation facility manager in this preliminary screening process. Importance of the facility to life safety, emergency preparedness and post-earthquake recovery, and socio-economic impacts should be considered when making decisions on programming of further study or construction work related to seismic vulnerability.

The method presented in this section is a valid approach for preliminary screening for seismic vulnerability. It can be useful for large groups of structures/facilities with similar characteristics to give an overall view of the vulnerability of each group as a whole, or to set up priorities of individual structures within the group. However, because of the great complexities and variations of real structures it should not be relied upon to give accurate results for a specific structure. This is the subject of the next section, where recommendations are given for the detailed analysis.

4.4.2 Detailed Analysis for Seismic Vulnerability

New Structures

For any new structure, the given state-of-the-art in seismic design presents the opportunity to create a safe facility that will result in acceptable facility performance during and after a seismic event. These design techniques are explained in detail in Appendix C. The vulnerability, assuming proper design methods are used, should be minimal. Specifically, it should behave in accordance with code objectives; it should sustain minor damage, but should not collapse or threaten the life safety of its inhabitants.

Existing Structures

For existing structures/facilities, the problem of assessing the level of seismic vulnerability and potential damage for a specific facility in detail is much more complex and is beyond the scope of this report. In general, this type of study will require the expertise of structural and geotechnical engineering professionals experienced in seismic analysis and design methods. The general issues that must be addressed, however, are discussed below.

The detailed vulnerability assessment starts with a detailed seismic hazard assessment. The procedure for performing this assessment is covered in Appendix A. It involves consideration of the geographic location of the facility, potential earthquake sources, recurrence rates and geology in the area in order to arrive at seismic ground motion parameters, usually effective peak ground acceleration.

Once the seismic hazard has been determined, the vulnerability of the facility is determined. Many factors must be considered. The assessment must incorporate facility type - whether it is at grade, above ground or below ground. The design of the structural framing system and structural details must be evaluated to determine whether they have any impact on the vulnerability of the facility. Of particular importance is the building code to which the structure was originally constructed, and the adequacy of its seismic code provisions. The local geology and foundation details must be evaluated for vulnerability effects. General attributes of the facility must be taken into consideration including its age, occupancy, use, importance for emergency preparedness and port-earthquake recovery, its replacement costs and potential costs associated with loss of revenue. These issues are discussed in detail in Appendix C.

Large Scale Facility Evaluation

Assessment of seismic vulnerability on a large scale poses its own special problems. It is necessary to evaluate large numbers of facilities from technical, economic, and political viewpoints, with the objective of arriving at conclusions for the need for seismic retrofit, reconstruction, change of use, or other means of achieving a satisfactory level of facility vulnerability.

A three phase approach is recommended for seismic vulnerability evaluation. Phase I would be a data collection and preliminary screening process using the methods described above. Phase II would be a detailed evaluation of critical facilities identified in Phase I. Phase III would consist of the establishment of remedial rehabilitation to facilities with unsatisfactory vulnerability. This would consist of facility closure, retrofitting, change of use, or new construction.

5.0 CURRENT SEISMIC DESIGN AND RETROFIT PRACTICE

5.1 General

Vulnerability to earthquake damage can be reduced by good engineering design. Research is constantly revealing new and more effective methods of designing for seismic resistance. Seismic design and retrofit practice in the U.S. has evolved quickly within the last century. Until the 1950's, there were virtually no seismic provisions in force, except for some limited requirements in California. Today, engineers have the capability of designing structures to withstand earthquakes with a high level of reliability.

This section provides a review of basic seismic design principles, and describes the various codes and criteria that have been developed to date. The latest state of the art in seismic design theory is summarized. Engineering and design methods currently being practiced in the United States for the various structure types are described. Finally, the additional construction cost of incorporating seismic resistance is discussed.

This subject is covered in more detail in Appendix C.

5.2 Fundamentals of Earthquake Engineering

Designing buildings and structures to be resistant to earthquakes is a complicated problem. The forces a structure will experience from an earthquake depend on many seemingly unrelated factors including: site soil conditions, the structure's geometry, type of framing, connection detailing (how members are connected together to resist the applied structure loads - see section 5.4), and the magnitude, frequency and duration of the earthquake ground-shaking excitation. Often, an iterative approach is necessary, consisting of: estimation of seismic forces using assumed structural member properties, design of members, calculation of revised force estimates using the designed structural member properties, and repetition of the process until a satisfactory design results.

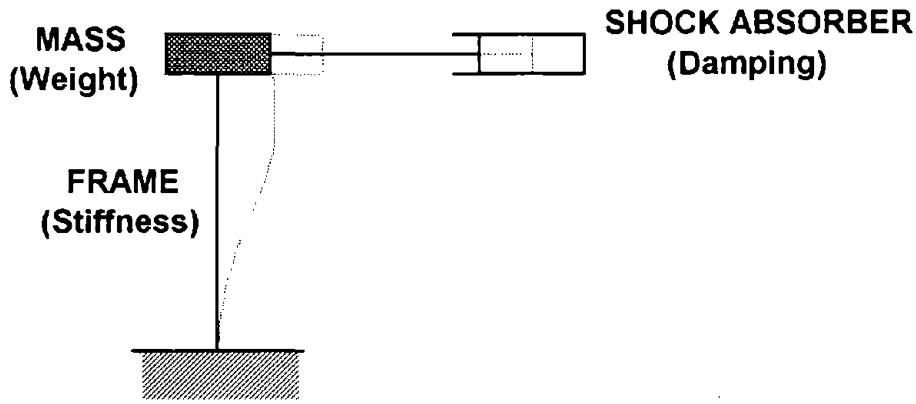
There is more uncertainty in seismic design theory than in other areas of structural design. The accuracy with which earthquake forces can be predicted is approximate at best. Even with extensive subsurface investigation, soil properties can only be estimated, as can the nature and severity of the anticipated earthquake. The seismic design problem is largely one of simplification based on reasonable assumptions.

Earthquakes cause ground shaking that in turn shakes the structure. What this shaking does to a structure depends on a number of factors: the *ground acceleration* caused by the earthquake, the structure's *fundamental period of vibration*, the structure's *ductility*, and the structure's *damping*. Also, the frequency content of the ground motion, and its *resonance* with the structure are important elements that will determine the magnitude of the earthquake load. These concepts are described below.

Insight can be gained into these ideas by considering the simplified model of a structure shown in Figure 5-1. As indicated, the structure has mass, or weight, stiffness, and damping characteristics.

The factors affecting the seismic performance of a structure are as follows:

Ground Acceleration: The earthquake induced vibration in the earth crust causes an acceleration in the soil or rock surrounding a structure. Values of ground acceleration can be obtained from maps developed statistically from historical data on past earthquakes. The most commonly used ground acceleration is that which has a 90% chance of not being exceeded in fifty years. These are the same types of ground acceleration maps described in Section 3, and shown in Figure 3-1 and 3-2.



- *Mass represents weight of structure.*
- *Frame represents stiffness of structure's framing system.*
- *Shock absorber represents structure's capacity to dampen movement*

Figure 5-1 Simplified Structural Model.

Fundamental Period of Vibration: A period of vibration is the length of time a freely vibrating structure takes to vibrate through one complete cycle (see Figure 5-2). In the case of a tuning fork there is one fundamental period that describes its movement. In reality a tuning fork, building, or any structure, vibrates in a complex fashion with many different periods. There is one period, however, that predominates. This is the fundamental period. The structure's mass and stiffness determine its fundamental period of vibration.

Ductility: A ductile structure is capable of sustaining large earthquake induced movements without fracture. It is distinguished from flexibility in that the ductile structure imparts a high resisting force early in its movement which remains nearly constant throughout the movement (see Figure 5-3). A flexible structure, on the other hand, imparts a low resisting force which steadily increases as the movement increases, until fracture occurs. Throughout ductile movement, energy is absorbed by the member, dissipating the energy of vibration. This is characteristic of structural steel structures and heavily reinforced concrete structures.

Damping: This is the physical phenomenon that causes a freely vibrating structure to taper off over time and eventually come to rest. As a tuning fork vibrates, the air dampens the movement through friction, causing it to taper off gradually. A tuning fork immersed in water experiences much more damping, and tapers off almost immediately. Damping in a structure is caused primarily by friction loss within structural components, and by ductility.

Resonance: When a structure's fundamental period of vibration is close to the period of induced vibration (ground shaking), the structure experiences *resonance*. Under these conditions, the structure's vibration increases in magnitude without bound until the structure fails. Damping puts a limit on the magnitude of the vibration that can occur. For example, in the thick soft clays of Mexico City, the 1985 earthquake resulted in ground-surface motions with a dominant period around 2 seconds, which unfortunately was also the natural period of a large number of 8 to 10 story buildings which were badly damaged. See Appendix A 4.3.

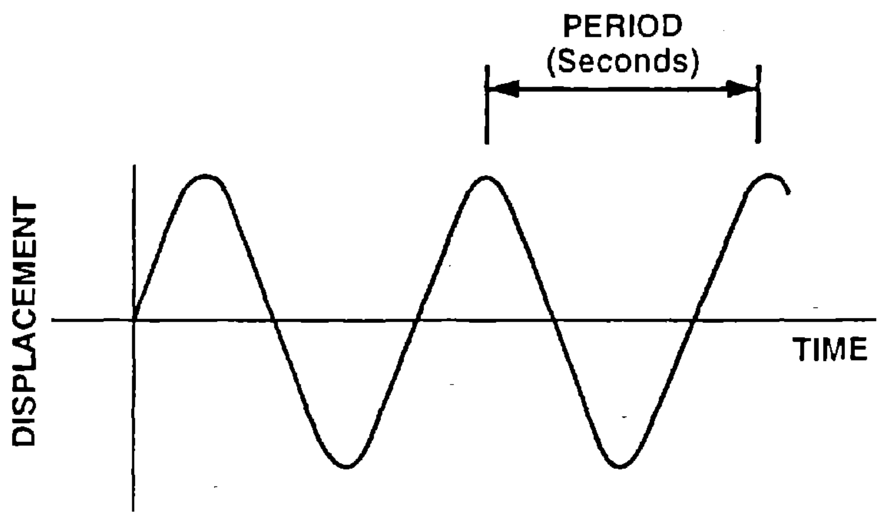


Figure 5-2 Period of Vibration.

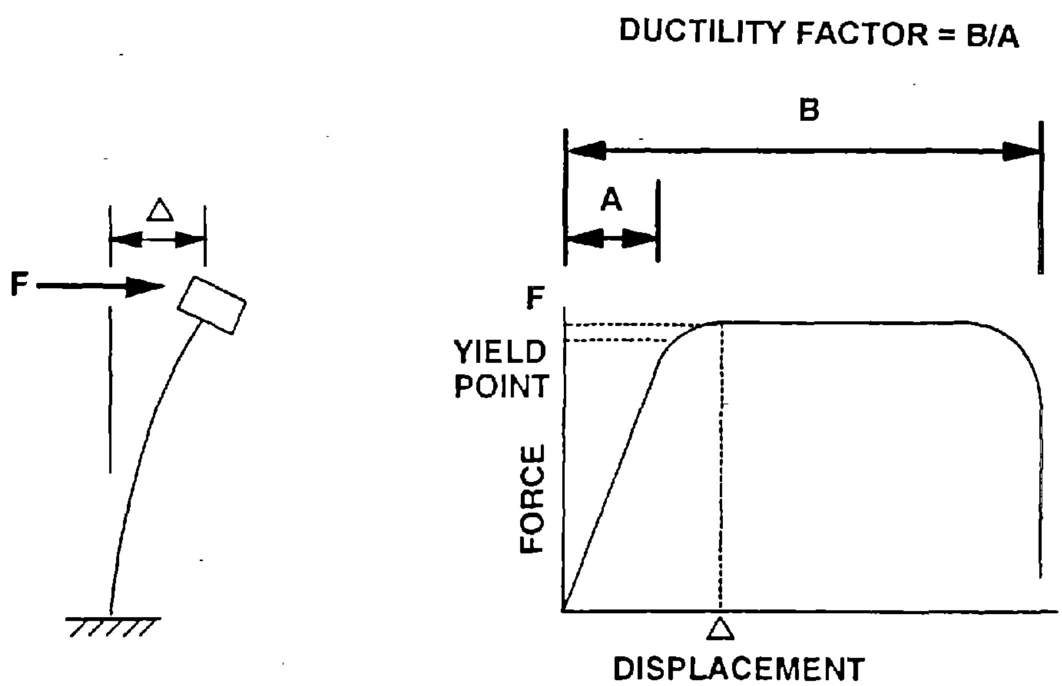


Figure 5-3 Ductility.

5.3 Applicable Codes

Seismic design code requirements are presented in a number of building codes, governing the design of different types of structures. Most code provisions in the United States addressing the design of structures for earthquakes have the same basic philosophy. They are concerned primarily with life safety. The Applied Technology Council's (ATC) *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC 3-06) published in 1978, for example, states that the objectives of the provisions are to provide buildings with the capacity to:

- resist minor earthquakes without damage;
- resist moderate earthquakes without structural damage, but with some non-structural damage;
- resist major earthquakes without collapse, but with some structural as well as non-structural damage.

Structures would be prohibitively expensive if they had to be designed to withstand earthquake forces with no damage. Since earthquakes are generally a rare occurrence, codes attempt to prevent collapse, but not damage.

There are many different types of transportation facilities, as described in Section 4. Design of these types of facilities can be conveniently broken down into five categories, as follows:

1. *Buildings:* Railway terminal buildings, rail transit terminal buildings, airport terminal buildings and port facility buildings.
2. *Bridges:* Major and conventional highway, railway and rail transit bridges.
3. *Marine Structures:* Piers and wharves.
4. *Subsurface Facilities:* Highway, railway and rail transit tunnels, retaining walls and bulkheads.
5. *At Grade Facilities:* Freeways, highways, local roads, tracks, roadbeds, runways and taxiways.

Currently, there are extensive code provisions for categories 1 and 2. There is limited guidance for the design of marine structures or subsurface facilities (categories 3 and 4), however the design of these features certainly must incorporate seismic considerations. Usually project specific criteria are developed. At grade facilities (category 5) have minimal impact from earthquakes, and typically seismic design is not considered for this category, except for potential damage from landslides, liquefaction, and surface faulting displacement. This report, therefore, only covers design and retrofit practice for the first three categories.

Seismic design requirements for buildings are prescribed in building codes. Local jurisdictions usually have their own building codes. Typically, towns or cities adopt the state building code, modified with special provisions for their locality. State codes are usually based on one of three model codes:

The Uniform Building Code (UBC) — Published by the International Conference of Building Officials, Whittier, CA.

The BOCA National Building Code (BOCA) — Published by the Building Officials and Code Administrators International, Chicago, IL.

The Standard Building Code — Published by the Southern Building Code Congress International, Birmingham, AL.

The codes require that all buildings, except for small residential buildings in low risk areas and agricultural storage facilities, be designed for earthquake effects. Design requirements vary according to building type and ground acceleration.

Highway bridges are covered by the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for the Design of Highway Bridges*. The seismic provisions of the AASHTO code are similar in concept to the provisions in the building codes, but there are some requirements unique to bridges.

The American Railway Engineering Association (AREA) *Manual for Railroad Engineering* is used for the design of railroad bridges. This code gives no specific provisions for seismic design. In practice, the AASHTO code is usually used for seismic design of railroad bridges.

5.4 General Seismic Design Procedure

Seismic design follows the general procedure summarized below. There are some differences with some structure types, but in general the procedure is the same.

1. **Determine Design Earthquake Parameters:** Ground motion parameters are obtained or derived as described in Section 3 and Appendix A.

2. **Perform Geotechnical Evaluation:** A subsurface investigation program is carried out to identify potential hazards related to the subsurface conditions, including the following:

   - **Liquefaction:** Some soils can liquefy during dynamic earthquake loading if it is loose, and has a high groundwater table. This causes the soil to lose its bearing capacity, resulting in a foundation failure. There is not much that can be done under these conditions. For a new structure an alternate location may be prudent. It is necessary, therefore, to identify whether there is potential for liquefaction at the proposed facility site.

   - **Slope Instability:** Steep slopes adjacent to the structure, or supporting the structure could become unstable during an earthquake. These areas are checked for stability under seismic loads.

   - **Settlement:** Fill, or loose soils could densify during ground shaking, causing dramatic settlement. Soil characteristics are checked for susceptibility to this phenomenon.

3. **Develop Preliminary Structural Design:** The structural framing layout is developed along with preliminary member sizes.

4. **Perform Seismic Analysis:** Usually a computer model will be generated by structural engineers to determine the seismic forces in structural members, along with vibratory displacements.

5. **Design Structural Members:** The structural components are designed for the internal member forces calculated as above. The preliminary sizes are modified as required to withstand the calculated internal member forces. If member sizes change significantly, it may be necessary to run another structural analysis with the new sizes.

Special detailing is designed for all structures to ensure that the assumed ductility can be accomplished. Detailing of structural components refers to the design and configuration of connections, reinforcing steel, and other *parts* of structural members. It involves structural design on a *micro* scale, whereas structural framing design (the sizing and layout of beams and columns) is structural design on a *macro* scale.

Proper detailing is critical to the successful performance of a structure during an earthquake and has received recent scrutiny in building codes. Well designed detailing will ensure that the structure has the ability to withstand the movement necessary to dissipate earthquake energy.

The structural members to be addressed include the following:

- Framing members - beams, columns, and bracing.
- Diaphragms - floor slabs and roofs serving to distribute horizontal loads through diaphragm action.
- Foundations.
- Connections - beam/column connections, bracing connections, diaphragm connections, column base plates, etc.

6. **Detail for Displacement:** Seismic joints, or shake spaces, are provided between adjacent buildings or portions of buildings to prevent impact damage during shaking. Also, structural components are detailed to accommodate anticipated displacements.

7. **Check Drift:** Horizontal deflection is checked for conformance with code limitations, accounting for increased movement resulting from the ductility of the framing system. The framing may need to be redesigned to comply with the prescribed limits.

8. **Design Foundation Components:** Footings, pile caps and walls are designed for the applied loads. They must be capable of sustaining the loads, and also of transferring the loads to the surrounding soil or bedrock.

9. **Design Non-structural Components:** Parapets, cladding, and some architectural, electrical and mechanical fixtures are designed for seismic loads. Though they may not be part of the structure, they can jeopardize life safety if they collapse.

10. **Construction Inspection:** Some codes make special requirements for inspection during construction to ensure that components critical to seismic resistance are constructed properly.

This procedure is used for most structures. Complex or unusual structures may require a more complex approach. This is covered in more detail in Appendix C.

5.5 General Retrofit Procedure

The term "seismic retrofit" is used to describe the construction of improvements to improve the performance of existing structures during an earthquake such that their earthquake vulnerability can be reduced to an acceptable level.

Many structures currently in use as part of transportation facilities were designed and constructed prior to the development of modern seismic design techniques. There are, therefore many facilities that are vulnerable to serious earthquake damage. Seismic retrofits are accomplished in order to restore them to an acceptable level of safety based on newly developed technology.

The problem is far-reaching. Most structures currently in use were constructed under old building codes with insufficient seismic provisions, and they are vulnerable to serious earthquake damage. No region of the country is immune.

Probably the biggest problem facing the transportation facilities manager, and the first step in a retrofit program, is the prioritization of required retrofit work, according to hazard, vulnerability, risks and cost/benefits analyses. The criteria for prioritization for seismic retrofitting for any transportation facility should include the following:

- *Vulnerability.* The methods discussed in Section 4 and Appendix B should be utilized to determine seismic vulnerability of the facilities for comparison and ranking.
- *Importance of structure to the transportation network.* If closing a facility due to earthquake damage will disable a key transportation system, then it qualifies as an important structure. Also if the facility is critical to emergency preparedness or post-earthquake recovery, it is an important structure. A few examples are major water crossings like the San Francisco-Oakland Bay Bridge, key interchanges, and airports.

- *Cost for repair/replacement versus cost for seismic retrofit.* The cost of replacing or repairing a structure after an earthquake is usually much more than the cost of constructing a seismic retrofit modification prior to an earthquake. Structures with a high ratio of repair or replacement cost compared to the seismic retrofit cost generally should receive a high priority for retrofit.

- *Adjacency of earthquake faults.* Facilities located near major fault systems are likely to see higher accelerations or larger displacements than those located at greater distances. These facilities are likely highly vulnerable to major damage and should receive special attention.

- *Site soil conditions.* Structures founded on unconsolidated fills, or deep soft soils will experience very different excitation than those founded on rock. If a structure is founded on soils susceptible to major liquefaction, major damage or failure is often likely; alternative sites or routes should be evaluated since retrofitting the structure will have little effect on a liquefaction failure.

- *Redundancy of structural system.* Redundancy in the structural system should be considered. For example multiple-column piers can absorb more damage than single-column piers, and continuous bridges are not as vulnerable to displacements as simply supported bridges. Structures with little redundancy are more vulnerable and should receive higher priority.

Once a structure has been selected for retrofitting, a *retrofit strategy* must be developed. A retrofit strategy is a plan to provide adequate ductility, strength, and stiffness to a structure. The strategy must consider the complete structure in addition to each of its elements. Some strategies may require only increased ductility, while other strategies may require more strength and stiffness. In some cases a strategy may use lower strength or stiffness to force another structural component to absorb the majority of the earthquake energy, thereby protecting the other members from damage.

One concept important for the understanding of retrofitting is the d*emand-to-capacity ratio.* This is the ratio of dynamic demands on a member to that member's capacity, or the ratio of the seismic force that will be imposed on a member to the maximum force that the member can safely carry. Since the need to retrofit a structural component depends on the demand-to-capacity (D/C) ratio, a deficient structural component requires a seismic retrofit method that reduces the demand-to-capacity ratio.

Either capacity must be increased or demand reduced. Capacity can be increased by strengthening the structural component. Alternatively, demand can be reduced by changing the structure's vibratory characteristics so as to subject the structure to lower earthquake forces (but necessarily accompanied by larger displacements).

The general procedure for development of a Retrofit Design Program is summarized below:

1. Select structure for retrofitting. ATC-6-2 gives a method and criteria for identification of those structures where retrofitting is advisable.

2. Obtain information on structure design and construction, as well as foundation and site soil conditions.

3. Perform field reconnaissance of structure and site vicinity.

4. Determine liquefaction potential and dynamic settlements of site soils.

5. Determine capacity of existing structure.

6. Determine dynamic demands on structure from expected earthquake forces.

7. Compare dynamic demand with structure capacity.

8. Determine areas of inadequate capacity.

9. Establish a retrofit strategy.

10. Design the retrofit.

11. Construct the retrofit.

Table 5-1 summarizes some seismic deficiencies, along with alternatives for remedial retrofits and comparative costs.

5.6 Economic Implications

An important consideration for the facility manager is the cost of incorporating good seismic design practice into new construction. This is difficult to determine, and there does not appear to be one universal solution which applies to all situations. However, some general observations can be made.

The design philosophy used for a facility clearly has economic implications. A facility could be designed such that it would suffer only minor damage in a major earthquake. However, the cost increase would be considerable. Conversely, it could be designed at a lesser cost, to protect life safety during an earthquake, but with some damage. Thus, in the development of a design philosophy, clearly stated objectives are important to ensure that available resources are spent prudently.

5.6.1 New Structures

In general, the cost of a structure is related to the amount of material used to construct it. For conventional loads, the dimensions of the structure are proportioned relative to that load. Thus, the amount of material and the cost of the structure are generally related to the magnitude of the loads. It is difficult, however, to clearly specify the individual costs contributed by each load because design is based on the controlling combination of all loads.

In the case of earthquake provisions, it is even more difficult to generalize regarding the specific contribution to the cost of a structure. The earthquake loading may or may not govern the design. In California, it may be expected that the specific choice of structural details is controlled by seismic considerations. In other areas this may not be the case. Further, as has been discussed in the preceding sections, an earthquake does not simply contribute a load to the structure (like live or wind load), requiring additional strength, but it makes a demand for displacement requiring flexibility and ductility. Indeed, increasing the size and thus the rigidity of structural members can increase the amount of the earthquake load they attract and make them less able to meet the displacement demands. This fundamental difference is important in defining costs associated with seismic provisions. Also, just as there are differences in the state-of-the-art treatment of aboveground and underground structures, it is expected that the seismic ramifications of cost will differ for each.

Buildings

There is a slight increase in building construction cost resulting from incorporating seismic resistance. Studies performed by the Building Seismic Safety Council (BSSC) in 1983-84 indicate that incorporation of seismic resistance into buildings would increase the construction cost as follows:

- **Cities Without Seismic Provisions:** 29 trial designs were completed in 5 cities that, at the time, had no seismic provisions in their local building codes (Chicago, Fort Worth, Memphis, New York and St. Louis). The average projected increase in total building construction costs, attributable to incorporation of seismic provisions was 2.1%.

- **Cities with Seismic Provisions:** 23 trial designs were completed in 4 cities that did not have seismic provisions in their local building codes (Charleston, Los Angeles, Phoenix, and Seattle). The average projected increase in total building construction costs, attributable to incorporation of the new seismic provisions was 0.9%.

Bridges and Other Elevated Structures

Similar studies were performed by AASHTO, and the results were included in their Commentary to the Standard Specifications for Seismic Design of Highway Bridges, dated 1983, including Interim Specifications dated 1985, 1987-88 and 1991. The average cost increase was approximately 6%. The percent increase varied with structure type and, of course, acceleration coefficient. One continuous span concrete bridge increased by 45%. All but three were below 10%, however.

Subsurface Facilities

No studies were found for increased costs for underground structures. It is anticipated that increased costs would be low, less than 5%, based on the relatively small demands put on subsurface structures during past earthquakes.

Retrofitting

The economic implications of implementing a seismic design/retrofit policy are truly enormous. The United States Department of Transportation Seismic Committee is currently pursuing a seismic retrofit program. Thousands of facilities are involved, and the seismic resistance of many older structures is largely unknown. Just completing an inventory to begin assessing seismic vulnerability for existing structures will be a major undertaking.

In general, the cost of constructing a seismic retrofit to a substandard structure is orders of magnitude higher than the cost of constructing a seismically resistant structure in the first place. Designing and constructing a structure for seismic loads generally adds about 1% to 6% to the cost of the facility, while designing and constructing retrofits can exceed 100% of the replacement cost, in which case abandonment or total reconstruction would be warranted.

The cost of retrofits can vary widely, however. They can range from the replacement of airport control tower glass windows with Plexiglas, to the total reconstruction of bridge abutments. Similarly, the cost/benefit ratio can vary widely. Costs, benefits, risks and vulnerability must all be considered in the development of a seismic retrofit program.

The following table summarizes some seismic deficiencies, along with alternatives for remedial retrofits and comparative costs.

TABLE 5-1

Seismic Retrofit Costs

Potential Vulnerability	Retrofit Alternatives	Relative Cost[1]
Liquefaction	• Dynamic Compaction	• Low
	• Vibroflotation	• Moderate
	• Excavate and Replacement	• Low to High
	• Grouting	• High
	• Strengthen With Long Piles	• High
Landslides (including embankments and dikes at port facilities)	• Stabilizing Berms	• Low to Moderate
	• Flattening Slope	• Low
	• Horizontal Drains	• Low
	• Reinforcing Dowels	• Moderate to High
Bridge Superstructure Failure	• Cable Restrainers	• Low
	• Increase in Beam Seat Support Width	• Low to Moderate
	• Base Isolation Bearings	• Low
	• Keeper Blocks at Bearings	• Low
	• Replacement of Rocker Bearings with Elastomeric Bearings	• Low
Bridge Column Failure	• Steel Jackets	• Moderate
	• Column Replacement	• High
	• Supplemental Columns	• High
Bridge Substructure/ Foundation Failure	• Base Isolation Bearings	• Low
	• Increase in Footing Size	• Low to Moderate
	• Installation of Piles or Caissons	• Moderate
	• Replacement of Substructure	• High

Potential Vulnerability	Retrofit Alternatives	Relative Cost[1]
Fuel and Gas Piping (Airports and Harbors)	• Automatic Shut-off Valves • Independent Regulators	• Low • Low
Moment Resistant Framed Building Failure	• Base Isolation Bearings • Framing Modifications	• Low • Low to High
Unreinforced Masonry Shear Wall Failure	• Abandonment • Total Reconstruction • Supplementary Framing, Foundations • Grouted in Supplemental Reinforcing	• High • High • High • High
Failure of Airport Control Tower Windows	• Replacement of Glass with Plexiglas	• Low
Non-Structural Components: (Mechanical, Electrical, HVAC, etc.)	• Bracing and Anchorage	• Low

Note 1. Relative costs are defined approximately as follows:

- Low: Less than 10% of facility cost.
- Moderate: Between 10% and 50% of facility cost.
- High: Greater than 50% of facility cost.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

There are many transportation facilities across the country that are vulnerable to serious earthquake damage. This has been clearly shown by the damage sustained by facilities in recent earthquakes. The costs to society of this vulnerability are large. Inadequate structures cost lives, disruption to transportation systems costs the productivity of the nation's work force and results in increased transportation costs, and repairing earthquake damage costs money. The gravity of the threat is often not appreciated by transportation facility managers and operators.

The country finds itself in this vulnerable condition for a number of reasons. A lack of understanding of seismic hazards and design principles, by engineers and designers as well as facility owners, managers and operators has led many to ignore the dangers. Also, many facilities were designed and constructed prior to the development of modern seismic analysis and design techniques. Seismic design is a relatively new field. Until the 1950's, there were virtually no seismic provisions in force, except for some limited requirements in California. The technology has evolved rapidly within the last 40 years, and today engineers have the capability of designing structures to withstand earthquakes with a high level of reliability. Those structures designed and built before this evolution are liable to be deficient in their ability to resist earthquakes. It is important to note that there are still many uncertainties in the field, and technology will continue to develop as new lessons are learned from future earthquakes, and from continued research.

There is an urgent need for reducing the seismic vulnerability of the transportation network in the U.S. The task is daunting at first glance, but it can be accomplished if a logical, methodical approach is developed.

The facility managers play a key role. This report takes the first step in raising their consciousness to the threat that earthquakes pose to transportation facilities. It serves to provide insights to where there should be concern, and where there need not be concern. It shows that there are steps that can be taken to reduce vulnerabilities to acceptable levels - through proper seismic design prior to construction, and through seismic retrofit of existing facilities. More detail is provided in the Appendices for technical staff, or readers who want to learn more about this topic.

Some of the measures described in this study can be accomplished by the facility manager, and some can not. It is up to the individual manager to determine when engineering assistance should be sought from others.

The USDOT is committed to this endeavor. All future grants will be conditioned on the seismic recognition in the construction they sponsor.

6.2 Recommendations for Seismic Mitigation Strategy

Figure 6-1 summarizes all of the steps the transportation executive should take to evaluate whether the vulnerability of the facilities under his/her jurisdiction represent a reasonable level of risk.

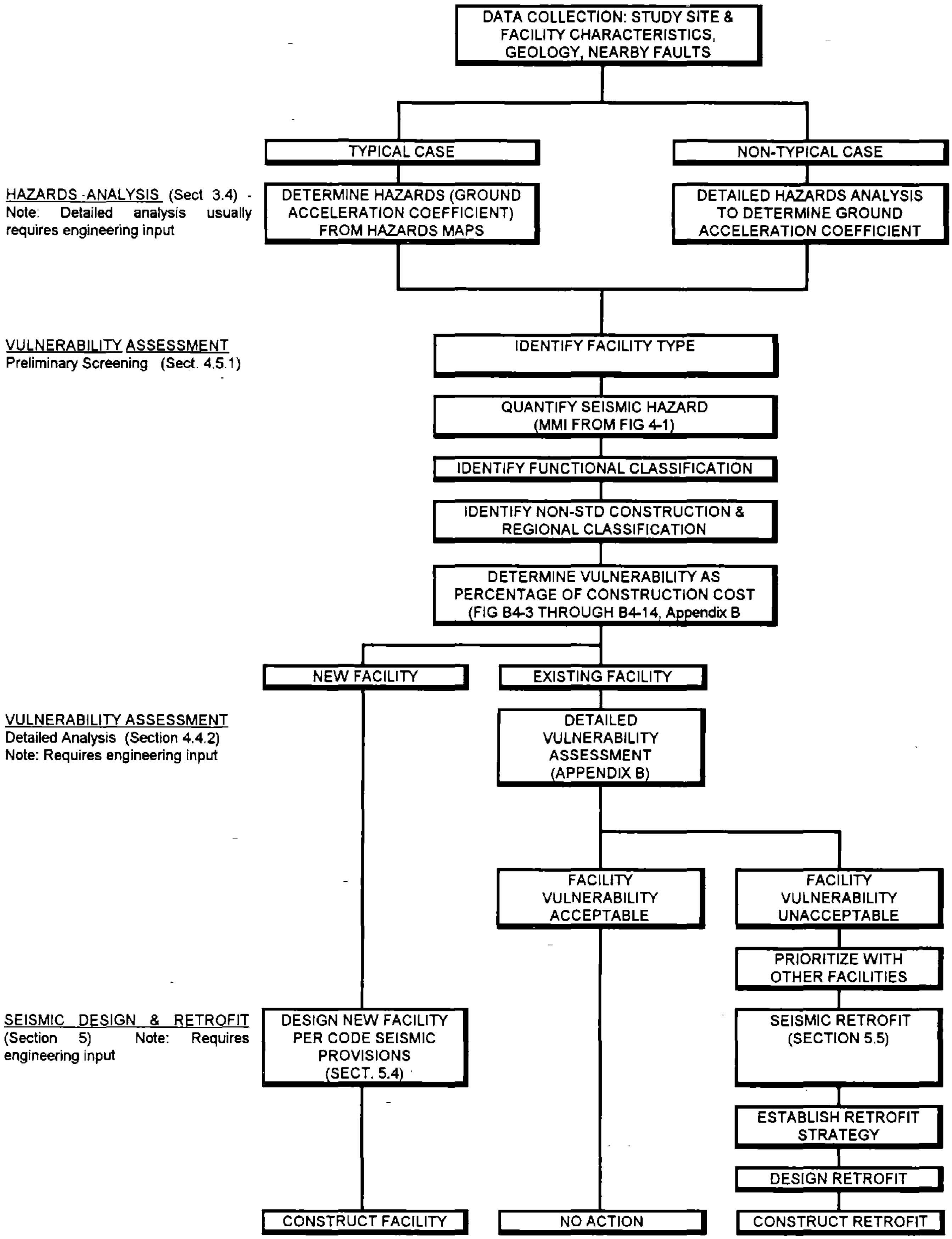


Figure 6-1- Seismic Mitigation Strategy Flowchart

7.0 REFERENCES

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Map 1.	ATC	County Map - Coefficient Aa
Map 2.	ATC	County Map - Coefficient Av
Map 3.	ATC	Contour Map for Coefficient Aa
Map 4.	ATC	Contour Map for Coefficient Av
Map 5.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 6.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 7.	USGS	0.1 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 8	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 9.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
Map 10.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 11.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
Map 12.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years (California)

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APPENDIX A

SEISMIC HAZARDS IN THE UNITED STATES

A

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SEISMIC HAZARDS IN THE UNITED STATES

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A 1.0 INTRODUCTION

Appendix A is one of three appendices that provide a technical basis for "Seismic Awareness: Transportation Facilities", a report written for transportation facility managers to educate them to the potential for seismic hazards directly effecting their facilities (new and existing), to present a suggested approach to evaluate facility vulnerability and to address seismic design aspects of new and existing facilities.

Appendix A describes the nature of seismic *hazards* in the United States. These hazards include the probability of occurrence of an earthquake in a certain area, as well as its likely intensity. Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes.

Appendix B, included in a separate volume, discusses the *vulnerability* of transportation facilities. Vulnerability here refers to the likely consequences of the expected seismic event on a particular structure. Unlike seismic hazard, vulnerability applies to specific structures. It is dependent on the expected seismic hazard, as well as the structural characteristics of the facility and the *local* geology of the site. Vulnerability is also distinguished from hazard in that hazard is a naturally occurring phenomena that man is unable to affect, while vulnerability is dependent on human factors that we have control over and can change - like the construction of a building or the steepness of an earth slope.

Appendix C, also included in a separate volume, summarizes current seismic design and retrofit practices in the United States. It gives some history on seismic design, illustrating the evolution of seismic design technology. It also explains in some detail the methods currently used for the design of different types of transportation facilities and components. Finally, it reviews the economic considerations involved and emphasizes the cost-effectiveness of incorporating seismic design elements into new structures during pre-construction design as opposed to retrofitting existing structures.

A 1.1 Purpose of This Appendix

Appendix A describes the earthquake hazards that exist for transportation facilities. The origin of earthquakes is introduced to provide some insight into the causes of these hazards. The variation by region is described and a simple method of determining earthquake hazards for a given facility is explained. More refined methods are also described.

A 1.2 National Earthquake Hazards Reduction Program

The majority of available information on this subject was obtained from the National Earthquake Hazards Reduction Program (NEHRP). This program resulted from the *Earthquake Hazards Reduction Act of 1977 (Public Law 95-124),* which required that the Director of the Federal Emergency Management Agency (FEMA):

> *"Prepare, in conjunction with the other Program agencies, a written plan for the program, which shall include specific tasks and milestones for each Program agency, and which shall be submitted to the Congress and updated at such times as may be required by significant Program events, but in no event less frequently than every 3 years."*

The primary NEHRP agencies are: the National Science Foundation (NSF), the United States Geological Survey (USGS), National Institute of Standards and Technology (NIST), and FEMA. FEMA has the primary responsibility for coordinating and planning NEHRP.

The most recent Program plan, *National Earthquake Hazards Reduction Program Five-Year Plan: 1992-1996* (1991) has been adjusted by reviewing the results of on-going program activities, and by reviewing the results of actual seismic events, e.g., the 1989 Loma Prieta earthquake. The Loma Prieta earthquake caused intense legislative scrutiny of the nation's potential earthquake hazards and how to best address them. That scrutiny culminated in the passage in 1990 of *Public Law 101-614*, an amendment to the Act and the most comprehensive legislation to address earthquake hazards since 1977. Some of the provisions of the *NEHRP Re-authorization Act of 1990 (P.L. 101-614)* include:

- update of the original goals and objectives of the Earthquake Act;
- definition of specific Program responsibilities for each of the Program agencies;
- a requirement for FEMA to establish an Advisory Committee for the Program;
- an entire section, with accompanying requirements, devoted to seismic standards;
- establishment of a program of post-earthquake investigations in USGS; and
- a significant increase in the authorized levels of funding for NEHRP over the three year period of re-authorization.

The 1990 amendment to the Act in large part validates the effectiveness of the paths undertaken by the NEHRP agencies since 1977. It also supplies a definition of the Program purposes and clarification of agency roles and responsibilities. It updates the goals and objectives of NEHRP which now include:

- education of society to the earthquake threat and the means by which to address it;
- development and improvement of design and construction techniques that resist earthquake damages;
- implementation of a system to predict and characterize earthquakes and their effects;
- development of model building codes and land use practices for earthquake hazards reduction;
- improvement of knowledge about and capacity to deal with earthquakes;
- application of research results; and
- development of a mechanism of assuring the availability of affordable earthquake insurance.

The roles and responsibilities of other NEHRP agencies are:

<u>United States Geological Survey</u> - The USGS conducts the research necessary to characterize and identify earthquake hazards, assess earthquake risks, monitor seismic activity, and improve earthquake predictions.

<u>National Science Foundation</u> - The NSF is responsible for funding research on earth science (to improve the understanding of the cause and behavior of earthquakes), on earthquake engineering, and on human response to earthquakes.

National Institute of Standards and Technology - The NIST is responsible for carrying out research and development to improve building codes, standards and practices for structures and lifelines.

The structure of NEHRP has been fine-tuned to better serve the needs of the ultimate users. Toward this end, the Program has specific tasks, objectives and milestones. Planning and executing NEHRP work is complex and involves intra- and interagency input and decision making. Constituents are important and supply both formal and informal input. There is a newly created NEHRP Advisory Committee to obtain non-Federal expert advice, comments and input.

A 2.0 BACKGROUND ON EARTHQUAKES

An understanding of the origins and causes of earthquakes is necessary to gain an insight into seismic hazards. This section describes some of the fundamental concepts behind earthquakes, and presents currently prevailing theories as to their causes. It should be noted, however, that this is a developing science, and the causes of earthquakes are not fully understood.

A 2.1 Definition of Earthquakes

Earthquakes are the sudden and violent release of elastic energy within the earth as the result of movements along geologic structures. This energy is mainly the result of stresses built up during tectonic processes consisting of the interaction between the earth's crust and the interior of the earth.

Global tectonics is a concept based on an earth model characterized by a small number (10 - 25) of large thick plates composed of both continental and oceanic crust. Each plate "floats" on a viscous underlayer and moves independently of the others, grinding against them at the common boundaries. Figure A2-1 shows the boundaries of the major plates, while Figure A2-2 shows the distribution of earthquakes around the world. The plate edges coincide well with the epicenters of most frequent earthquake activity as can be seen by comparing these figures. As a result of the immense pressure and temperature within the inner layers of the earth, the relatively thin outer crust is continually subject to movement. Most movements are gradual and can only be detected by careful measurements. Some, however, are the result of sudden releases of elastic energy as the large plates making up the earth's crust move relative to each other. It is these violent releases that typically cause the phenomenon we call earthquakes.

The location of an earthquake is usually referred to as its epicenter, which is the point on the earth's surface directly above the crustal disturbance. The effect of an earthquake can be very far-reaching as the ground vibrates under the propagation of the generated waves.

The destructive phase of an earthquake may vary in duration from a few seconds to about one minute. It is estimated that throughout the world there are over a million earthquakes every year. The majority are quite weak and many occur in remote unpopulated areas and therefore are noticed only by scientists. It is estimated that a large earthquake (greater then magnitude 6 on the Richter scale) occurs about once every week.

The size of an earthquake can be described in terms of the impact it has on the developed environment of the area (people and structures), a semi-subjective measure, intensity, or it can be classified according to the quantitative measure of the energy released, magnitude. Both methods have their importance.

The intensity scale used in the United States is known as the Modified Mercalli Scale. This scale uses Roman numeric classification from I to XII to describe the intensity based on the impact to the surroundings. For example, Intensity I refers to an event detectable only by instruments, while Intensity XII implies almost complete destruction. Significant building damage begins at intensity VII to VIII. Table A2-1 lists the damage associated with the various Modified Mercalli Intensities. A given earthquake is characterized by its peak intensity, since the intensity for a given earthquake varies with location. Plotting the intensity observed at various locations for a single earthquake indicates the attenuation of the earthquake effects with distance, and also indicates the importance of local soil conditions.

In contrast to the Mercalli Scale, the Richter Scale is a quantitative instrumental measurement of the magnitude, or strength of an earthquake on a logarithmic scale. The magnitude of a local earthquake is defined as the logarithm to the base ten of the maximum amplitude in microns recorded on a Wood-Anderson seismograph located at a distance of 100 kilometers from the earthquake epicenter. The largest magnitude ever measured on the Richter scale is 8.9, and the lowest magnitude that can be felt by people is about 2.

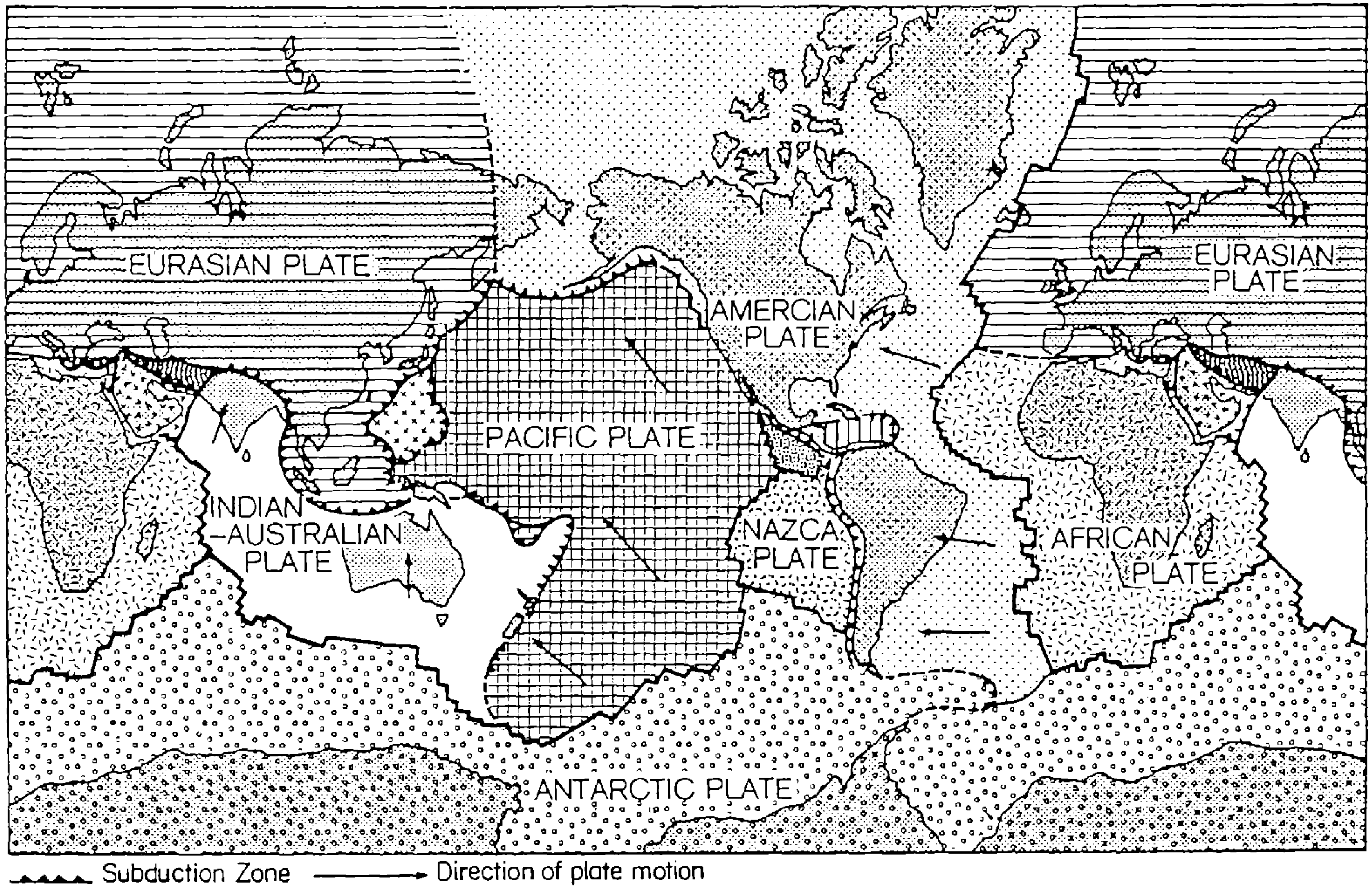

Figure A2-1 - Tectonic Plate Map of the World

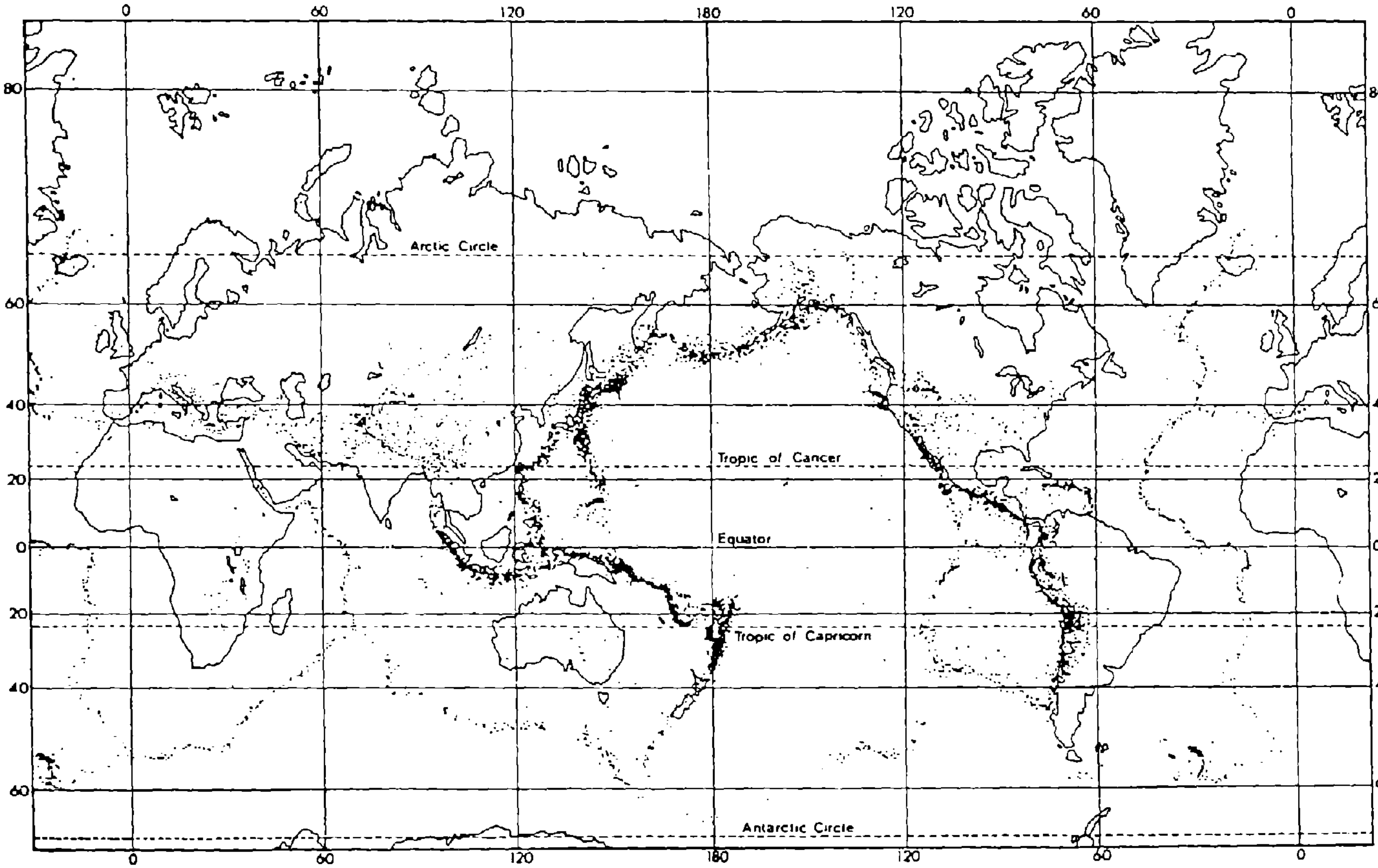

Figure A2-2 - Worldwide Earthquake Distribution

Lower magnitudes, even negative ones, can be measured by instruments. (Negative earthquake magnitudes are possible because of the logarithm of a number less then one is negative.) An increase of magnitude of one, say, from 6 to 7, corresponds to an increase in total energy release by a factor of about 30. The logarithmic nature of the Richter Scale is often overlooked by the general public and news reporters.

TABLE A2-1

Modified Mercalli Intensity Scale

I	Not felt by people, except under especially favorable circumstances.
II	Felt only by persons at rest on the upper floors of buildings. Some suspended objects may swing.
III	Felt by some people who are indoors, but it may not be recognized as an earthquake. The vibration is similar to that caused by the passing of light trucks. Hanging objects swing.
IV	Felt by some people who are indoors, by a few outdoors. At night some people are awakened. Dishes, windows and doors are disturbed; walls make creaking sounds; stationary cars rock noticeably. The sensation is like a heavy object striking a building; the vibration is similar to that caused by the passing of heavy trucks.
V	Felt indoors by practically everyone, outdoors by most people. The direction and duration of the shock can be estimated by people outdoors. At night, sleepers are awakened and some run out of buildings. Liquids are disturbed and sometimes spilled. Small, unstable objects and some furnishings are shifted or upset. Doors close or open.
VI	Felt by everyone, and many people are frightened and run outdoors. Walking is difficult. Small church and school bells ring. Windows, dishes and glassware are broken; liquids spill; books and other standing objects fall; pictures are knocked from the walls; furniture is moved or overturned. Poorly built buildings may be damaged, and weak plaster will crack.
VII	Causes general alarm. Standing upright is very difficult. Persons driving cars also notice the shaking. Damage is negligible in buildings of very good design and construction, slight to moderate in well-built ordinary structures, considerable in poorly built or designed structures. Some chimneys are broken; interiors and furnishings experience considerable damage; architectural ornaments fall. Small slides occur along sand or gravel banks of water channels; concrete irrigation ditches are damaged. Waves form in the water and it becomes muddied.
VIII	General fright and near panic. The steering of cars is difficult. Damage is slight in specially designed earthquake-resistant structures, considerable in all well-built ordinary buildings. Poorly built or designed buildings are damaged; interiors experience heavy damage. Frame houses that are not properly bolted down may move on their foundations. Decayed pilings are broken off. Trees are damaged. Cracks appear in wet ground and on steep slopes. Changes in the flow or temperature of springs and wells are noted.

IX Panic is general. Interior damage is considerable in specially designed earthquake-resistant structures. Well-built ordinary buildings suffer severe damage with partial collapse; frame structures are thrown out of plumb or shifted off their foundations. Unreinforced masonry buildings collapse. The ground cracks conspicuously and some underground pipes are broken. Reservoirs are damaged.

X Most masonry and many frame structures are destroyed. Specially designed earthquake-resistant structures may suffer severe damage. Some well-built bridges are destroyed and dams, dikes and embankments are seriously damaged. Large landslides are triggered by the shock. Water is thrown onto the banks of canals, rivers and lakes. Sand and mud are shifted horizontally on beaches and flat lands. Rails are bent slightly. Many buried pipes and conduits are broken.

XI Few, if any, masonry structure remain standing. Other structures are severely damaged. Broad fissures, slumps and slides develop in soft or wet soils. Underground pipelines and conduits are put completely out of service. Rails are severely bent

XII Damage is total, with practically all works of construction severely damaged or destroyed. Waves are observed on ground surfaces, and all soft or wet soils are greatly disturbed. Heavy objects are thrown into the air, and large rock masses are displaced.

An approximate correlation between Richter Magnitude and Practical intensity is provided in Table A2-2.

TABLE A2-2

Correlation Between Richter Magnitude and Practical Intensity

Richter Magnitude	Practical Intensity
1	Detectable only by instruments
2	Barely perceptible even near epicenter
4.5	Detectable within 20 miles of epicenter
6	Moderately destructive
7	A major earthquake
8	A great earthquake

A 2.2 Potential Impact of Earthquakes

The impacts from earthquakes of most concern in this study concern property damage and loss of life. Historically, the most dangerous aspects of seismic activity have been the effects earthquakes have had on man-made structures. Most deaths in recent times have not been the direct result of the earth's actions, such as ground shaking, earth rupture, volcanic eruption, or tidal waves, but the result of the failure of the man-made structures within which people live and work. Failure of infrastructure facilities such as buildings, highways, and transit systems, exposes the population to direct risks of injury and death. In addition, there are longer term social problems associated with disruption of communications, vital services, and the damage to utilities.

Earthquakes can cause damage in a number of ways. Damage to facilities occur through *primary, secondary,* and *tertiary* hazards. Primary hazards are those which can be directly related to the earthquake. They include such phenomenon as ground vibration and fault rupture. Secondary hazards are those potentially dangerous situations triggered by the primary hazards. These include foundation settlement, landslides, soil liquefaction, and tsunamis. Tertiary hazards result from structural damage caused by the primary and secondary hazards and are often the most serious. These include such events as flooding due to dam failure or fire following an earthquake. In fact, most of the property damage in the 1906 San Francisco earthquake was due to the great fire, not the ground shaking itself. Primary and secondary hazards are the subject of this Appendix. Tertiary hazards are minimized through the accomplishment of sound seismic design, the subject of Appendix C.

The most common and most damaging hazards from earthquakes are as follows:

Ground Shaking: Ground shaking refers to the vibration of the ground produced by seismic waves arriving at a site. Vibrations originate in the bedrock which undergoes explosive movement, releasing the stresses built up from restrained movement at the edges of tectonic plates. The vibrations propagate up through soils overlying the bedrock, causing ground shaking at the surface. The soil undergoes an oscillating acceleration as it moves back and forth. Man-made structures are supported on foundations built in or on soil or rock mass. Movement of the soil or rock, and therefore of the foundations, results in movement of the structure. The associated accelerations induce forces within the structure. Structural damage occurs when the forces exceed the capacity of the structural members. If the damage is extensive enough and the member is critical to the integrity of the structure, total failure or collapse results.

Ground Displacement: Ground displacement causes structural displacement which must be accommodated with allowance for shake spaces between buildings, etc. For underground structures such as tunnels, however, ground displacement is of primary concern. As the soil mass vibrates back and forth it "racks" or tilts. The structure must be capable of assuming this shape.

Surface Faulting: This is the offset or tearing of the ground surface by differential movement across a fault during an earthquake. For long linear systems, typical of many transportation facilities, surface faulting is an important potential hazard that must be considered.

Landslides: Steep earth slopes that are stable under static loads can become unstable during earthquakes. Seismic lateral loads can be sufficient to overcome the internal frictional forces within the soil mass that keep it in place under static loads causing a sliding failure. Facilities located within the effected zone either above, or below such a slope are threatened by this potential hazard.

Slopes fail because of a phenomenon called *progressive shakedown.* For a slope, the driving or destabilizing force is in one direction only, and the stabilizing forces are in the opposite direction. A seismic motion in the stable direction will have no consequence, but a motion or acceleration in the direction of the driving forces can exceed the point of stability. This does not necessarily mean the failure of the slope; rather, the slope is potentially subjected to slippage for the duration of pulse when the driving force exceeds the stabilizing force. If several pulses or acceleration peaks in one earthquake causes slippage (all, of course, in the same direction) additional displacement will accumulate, and the slope may fail.

Liquefaction: Loose granular soils with high ground water levels can *liquefy* during dynamic earthquake loading. This phenomenon occurs when water trapped within the voids between soil particles prevents the compaction and settlement that the soil would otherwise undergo as it is shaken in an earthquake. The soil particles lose contact with each other and the soil mass assumes a liquid-like state This causes the soil to lose its capacity to support loads, resulting in a foundation failure. Structures can float up, sink down, or tilt over depending on the magnitude and distribution of loads.

For liquefaction to occur, the sand must be loose, it must be below the groundwater table, it must have a permeability low enough that the pore water pressures cannot dissipate during the period of shaking, and the shear strain induced by the earthquake must be greater than a threshold value dependent on the sand density.

Settlement: Fill, or loose soils can densify during ground shaking, causing dramatic settlement. Structures located in these areas are susceptible to damage.

Failure from surface faulting, liquefaction, landslides and settlement can often be avoided in new structures with proper siting of the facility since the site's predisposition for such extreme behavior can be determined in advance. It is more difficult to remedy an existing structure already located in a deficient site with the above characteristics. Possibilities include dynamic compaction, vibroflotation, excavation and replacement of substandard soil, grouting and strengthening with long piles.

Ground shaking and displacement are accommodated through proper consideration of seismic forces and movements in the structural design of the facility. Structures are typically designed with allowances for all anticipated forces. The dominant forces are usually those associated with gravity acting on the structure itself, and on the occupants. Fortunately, the magnitudes of such loads are fairly predictable and it is easy to allow sufficient capacity. Seismic loads, however, because of the unpredictable nature of earthquakes, are more difficult to determine in design with the proper allowances. Although historical earthquake records indicate which geographical areas are more likely than others to experience quakes of significant magnitudes, it is not possible to accurately predict the time or location of an event, or its intensity and characteristics. The design for earthquakes must therefore be approached from a probabilistic viewpoint. One method often used in earthquake design is the response spectrum, which defines the likely response of simple structures to a particular earthquake.

A 3.0 SEISMIC REGIONALIZATION

A 3.1 General

Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes. On a large scale, the probability of an earthquake and the severity that can be expected can be predicted depending on the region. This regionality has been determined through a review of historical records and is currently presented on maps providing various ground motion parameters for the various regions of the country.

As mentioned previously, earthquakes most frequently occur along plate boundaries (Figure A2-1 and A2-2). The west coast of North America is the boundary of the Pacific plate and the North American plate, accounting for the significant earthquake activity in California and Alaska. The central and eastern United States (within the North American plate) are not areas which have plate margins. Earthquake activity, including some of the largest earthquakes known to have occurred within the United States were within this plate. These are the New Madrid, Missouri earthquakes of 1811 and 1812 which had Modified Mercalli (MM) intensities (see Table A2-1) of X-XI, IX and X-XI and estimated magnitudes of 7.5, 7.3 and 7.8; the Charleston, South Carolina earthquake of 1886 which had a MM intensity of X and an estimated magnitude of 7.5; and the Cape Ann, Massachusetts earthquake of 1755 which had a MM intensity of IX and an estimated magnitude of 6.0. Geologic structures (faults) have been related to the New Madrid earthquakes, but the others have no known geologic origin. Because of this, no area within the United States can be considered immune to earthquake activity. There is a finite probability of an earthquake occurring anywhere within the United States.

Within the United States, the observation and recording of seismic events has led to the development of maps which categorize areas by their seismic activity and level of risk. An early seismic probability map of the United States (Figure A3-1), prepared by the United States Geological Survey in 1948, graphically depicts historical earthquakes by location and size. The distribution is non-uniform, with the greatest concentration of activity along the west coast. This map also assigned seismic zone categories to areas throughout the United States. This zonation was adopted by many building codes at the time and continued in use for many years thereafter.

Very high seismic activity (both in frequency and magnitude) has been experienced in Alaska and the Aleutian rim, and just off the coast of Canada near Vancouver Island. The state of Washington shows some activity, as does the area around the Idaho and Wyoming borders with Montana. Of the contiguous states, California has been the most active, with high concentrations of activity in the areas south of San Francisco, and the southern part of the state.

Within California the dominant geological feature actively generating earthquakes is the San Andreas Fault System, extending in a north-south direction over the entire length of the state. This system has received worldwide attention because of its history of significant activity and because it passes through some of the most populated parts of the United States.

Western states such as Nevada and Utah show a low to moderate amount of activity, otherwise, the internal parts of the country have been relatively quiet. A concentration of activity which appears out of place is near the borders of Tennessee, Kentucky, and Missouri. This is the area of the New Madrid Fault.

Although earthquakes are more probable in some areas than others, it should be recognized that all states have at least some potential for seismic activity. It is a fallacy to think that some areas are immune. Earthquakes can happen anywhere. The risk can be greatest, in fact, in those areas that historically have experienced few earthquakes where structures have been built without seismic resistant details. The consequences can be catastrophic.

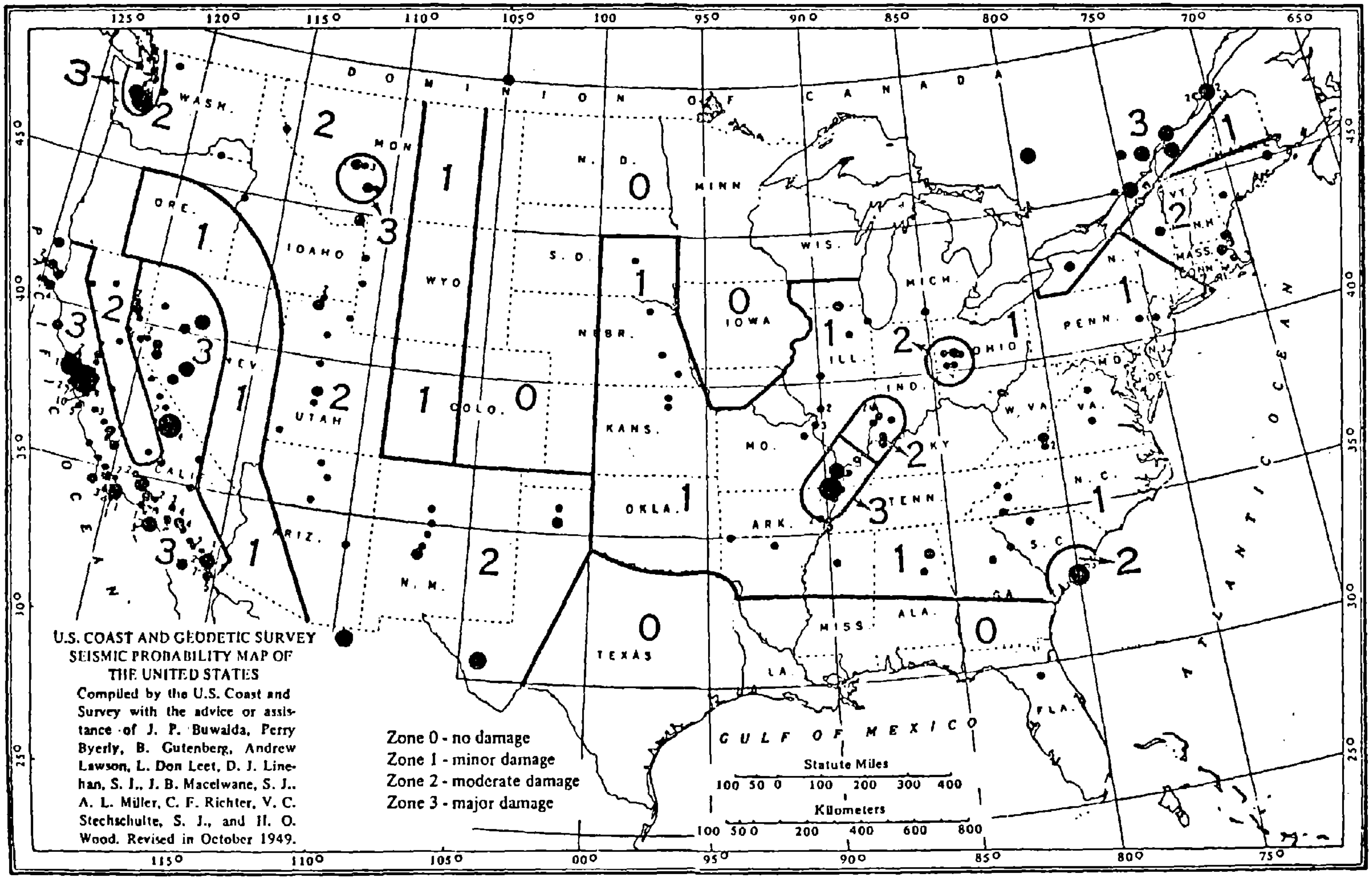

Figure A3-1 - Seismic Probability Map of the United States

A 3.2 <u>Current Code Requirements</u>

Some code provisions provide a basis for hazards evaluation for the transportation facility manager. General code concepts are presented here as an introduction to a method for hazards evaluation.

Seismic design code requirements are presented in a number of codes, governing the design of different types of structures. Most code provisions in the United States addressing the design of structures for earthquakes have the same basic philosophy. They are concerned primarily with life safety. The Applied Technology Council's (ATC) *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC 3-06) published in 1978, for example, states that the objectives of the provisions are to provide buildings with the capacity to:

- resist minor earthquakes without damage;
- resist moderate earthquakes without structural damage, but with some non-structural damage;
- resist major earthquakes without collapse, but with some structural as well as non-structural damage.

These objectives are similar to those of American Association of State Highway and Traffic Officials (AASHTO) *Standard Specifications for Seismic Design of Highway Bridges* (1991), and are applicable objectives for other public-related transportation facilities.

The *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* (1991) goes a little further with its goal of presenting "criteria for the design and construction of new buildings subject to earthquake ground motions in order to minimize the hazard to life for all buildings, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential facilities to function after an earthquake."

Designing a structure to achieve theses objectives and goals, requires the best possible estimate of the earthquake forces to which the structure may be subjected. Current code provisions for estimating seismic design forces generally involve the use of earthquake ground motion parameters such as ground acceleration, ground velocity, response spectra or ground motion time history. As described further in later chapters, these parameters also provide an excellent basis for evaluating seismic hazard.

Appendix C describes in detail the seismic design procedures using the equivalent static force method employed by various agencies. AASHTO's seismic design specifications for highway bridges uses an Acceleration Coefficient, A, which is expressed as a percentage of gravity. This acceleration coefficient indicates the estimated peak ground acceleration that statistically has a 90 percent probability of not being exceeded within 50 years. In areas of moderate seismicity such as New York City or Boston, this acceleration coefficient is approximately 0.15, while in areas of high seismicity in California, it may be higher than 0.40, which translates into significantly higher seismic design forces for structures in this region.

The 1991 Uniform Building Code (UBC) uses a seismic zone factor, Z, in calculating the total base shear. The zone factor ranges from 0.0075 in Zone 1 (areas of low seismicity) to 0.40 in Zone 4 (areas of high seismicity). Similar to the acceleration coefficient employed in AASHTO, the zone factor is more or less directly related to the estimated peak ground acceleration at the structure site with 90 percent probability of non-exceedance in 50 years.

The provisions for seismic design of buildings given in ATC 3-06 (1978), NEHRP 1991, and the 1992 Supplement to the Building Officials and Code Administrators (BOCA) National Building Code employ equations of similar form to those of AASHTO in calculating the total base shear forces.

However, in contrast to the Acceleration Coefficient (based on peak ground acceleration) in AASHTO, ATC 3-06, NEHRP 1991, and BOCA 1992 now use two parameters to more accurately characterize the intensity of design ground shaking for structures with differing characteristics:

EPA (or A_a): Effective peak acceleration coefficient. This indicates the acceleration resulting from a *near field* earthquake, with the epicenter located close to the structure site. It will generally control the design of more rigid structures (those less than 5 stories in height).

EPV (or A_v): Effective peak velocity-related acceleration coefficient. This indicates the acceleration resulting from a *far field* earthquake, with the epicenter located far from the structure site. It will generally control the design of more flexible structures (those greater than 5 stories in height).

These code provisions all require, in one form or another, certain parameters representing the ground shaking intensity which is used to calculate the seismic forces for design. This presentation of ground motion parameters also represents an excellent basis for determining seismic hazard for a given area. Acceleration coefficient is thus far the most commonly used design parameter to represent earthquake hazard. The characterization is commonly presented as earthquake hazard regionalization maps. The following section discusses the maps currently available for the United States, how they are used in current seismic design codes, and how they relate to hazards evaluation.

A 3.3 Regionalization Maps

Regionalization maps indicating ground motion parameters have been steadily evolving since earthquake study began, and they will continue to evolve as more data becomes available. The first national earthquake hazard map for the United States (Figure A3-1) was simply a geometric partitioning of the United States based on the maximum intensities experienced during historic earthquakes (1948). This map ignored the frequency of earthquake occurrence and attempted no characterization of earthquake sources or seismic wave transmission through the earth. With the evolving understanding of earthquake generation processes and the associated uncertainties, several generations of national probabilistic ground motion maps have been produced and incorporated into national design standards.

There are three sets of regionalization maps currently in use in the United States, the NEHRP, UBC, and AASHTO maps. The governing code for a structure determines which map should be used for determination of seismic hazard. In general the NEHRP and UBC maps are used for buildings and the AASHTO maps are used for bridges.

NEHRP Regionalization Maps

Two maps have been adopted by NEHRP to define seismic hazard throughout the United States, for effective peak acceleration (EPA) coefficient, A_a, and for effective peak velocity-related acceleration (EPV) coefficient, A_v (see Figures A3-2 and A3-3). These maps indicate peak horizontal acceleration contours for rock or firm foundation materials. The contours represent the magnitude of acceleration that, statistically, has no more than a 10 percent chance of being exceeded within 50 years. The EPA map was developed based upon the identification of local source zones or source faults, historical seismicity (recurrence rates) in each zone, and attenuation rates for peak ground acceleration. The EPV contour map is constructed from the map for EPA based upon the attenuation data of ground velocity with distance, rather than through a systematic probability-based peak velocity mapping. The NEHRP maps have been adopted for building design by the BOCA and Standard Building Codes.

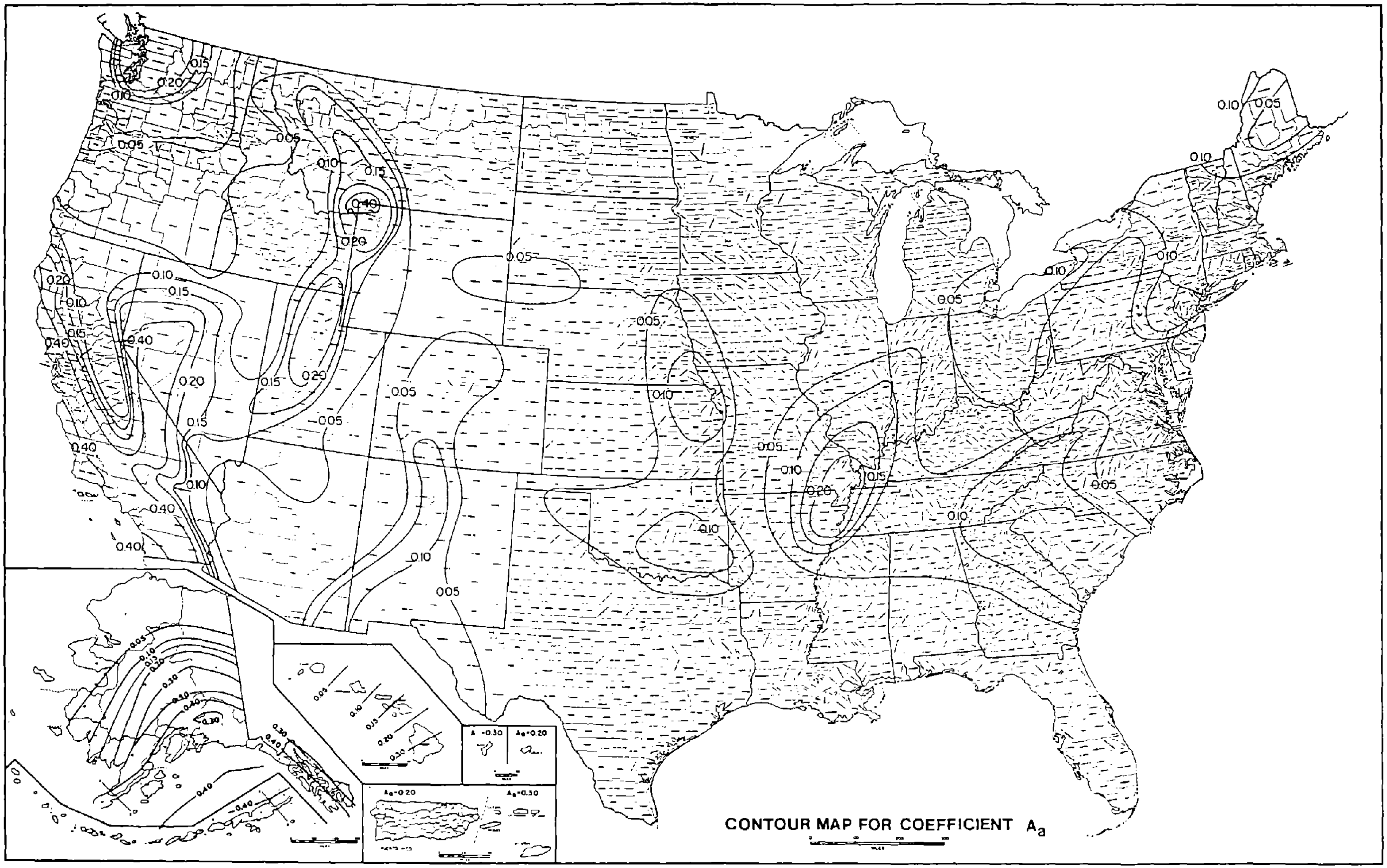

Figure A3-2 - Effective Peak Acceleration (EPA) Coefficient Contour Map

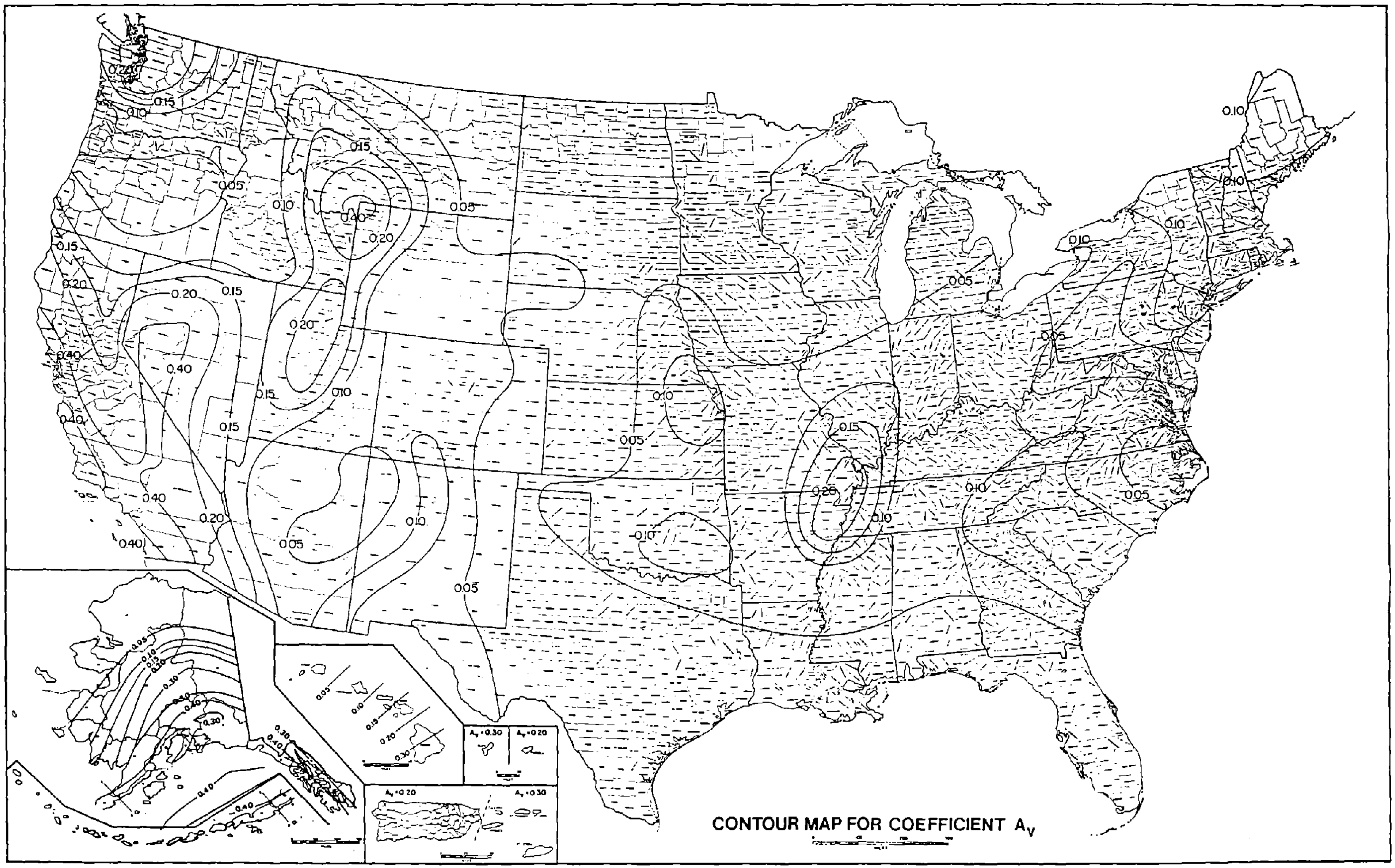

Figure A3-3 - Effective Peak Velocity (EPV) Coefficient Contour Map

UBC Regionalization Maps

The 1991 Uniform Building Code, instead of using the EPA or EPV contour maps, characterizes the earthquake hazard in the United States with a seismic zone map as shown in Figure A3-4. Hazard levels associated with different areas of the United States are expressed through seismic zones ranging from 0 to 4. Seismic zone factor, Z, increases from 0.0 for Zone 0 to 0.4 for Zone 4. The zone factor is then used for design. This seismic zone map, with some minor changes and revisions to accommodate local seismologic and geologic conditions, is essentially a direct descendant of the EPV map. A comparison between the UBC zone map (Figure A3-4) and the EPV contour map (Figure A3-3) reveal the similarity between the two.

AASHTO Regionalization Maps

The 1991 *AASHTO Interim Specifications for Seismic Design of Highway Bridges* utilizes maps having the same basis as NEHRP's EPV maps. They provide more accurate estimates of the ground motion hazard for long-period structures Figure A3-5 shows the map currently used by AASHTO.

It should be noted that although peak ground acceleration is the most commonly used hazards analysis parameter for above ground structures such as buildings and bridges, the peak ground velocity and peak ground displacement have been found to be useful in the evaluation of seismic hazard for underground structures. This is primarily because the seismic response of underground structures are more sensitive to earthquake - induced ground deformations than ground accelerations.

Recent geological and seismological evidence has also suggested that peak ground velocity and displacement in the Central and North-Eastern United States may sometimes be a better indicator of seismic hazard than peak ground acceleration. This is due to the unusual frequency content unique to this area.

In any event, based on the current state of the art, the hazards maps described above are a good indicator of seismic hazard for most facilities.

A 3.4 Hazard Analysis Methodology

Seismic hazard is usually defined as the "expected occurrence of a future adverse seismic event". It is often times mistakenly thought to be synonymous with seismic risk, which is defined as the "expected consequences of a future seismic event". Consequences may be loss of life, economic loss and the socioeconomic impact of the event on the affected region. From these definitions, it is apparent that the various regionalization maps presented in Section A 3.3 are based on hazard analysis rather than risk analysis, although sometimes these maps are referred to as seismic risk maps.

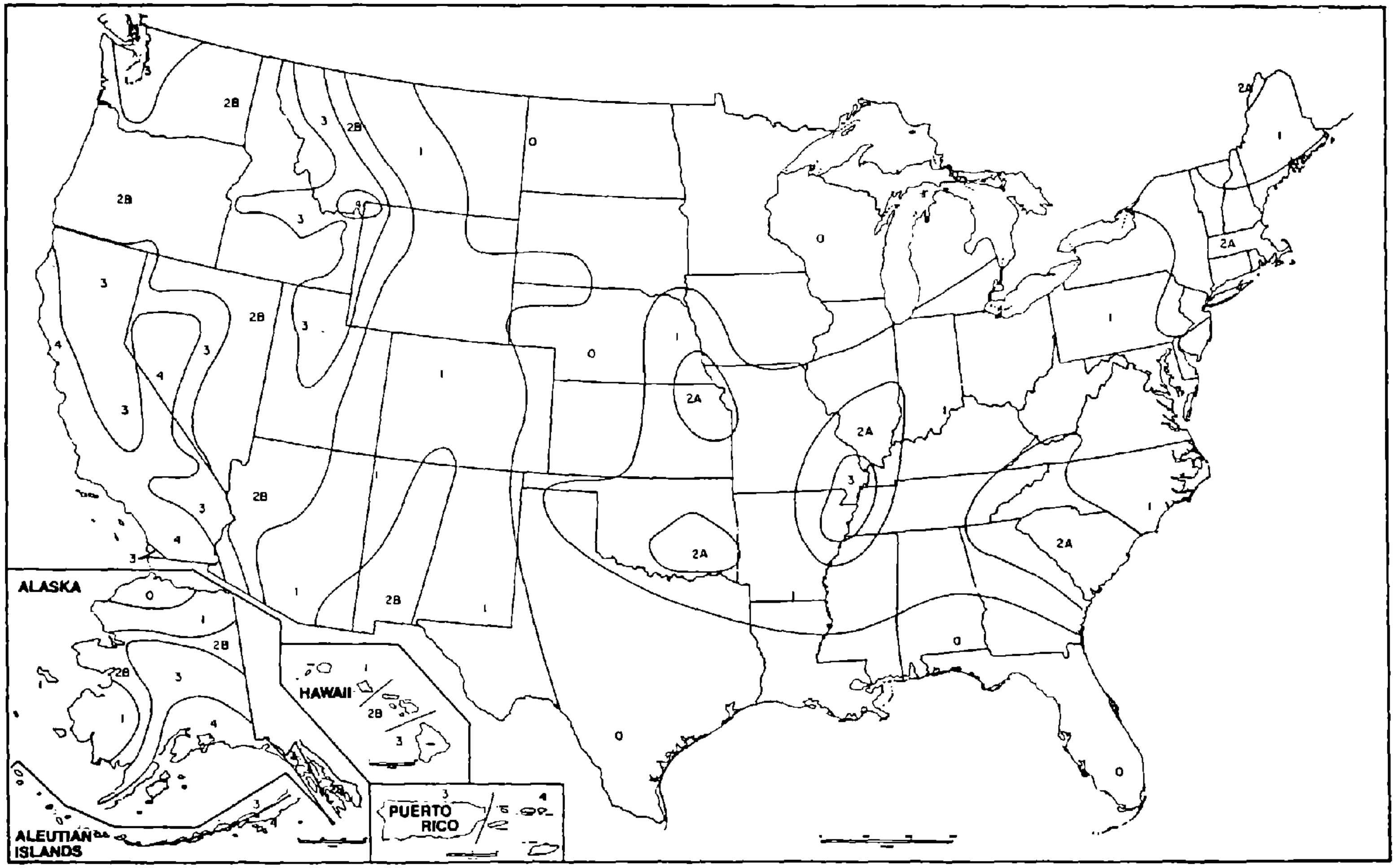

Figure A3-4 - UBC Seismic Zone Map

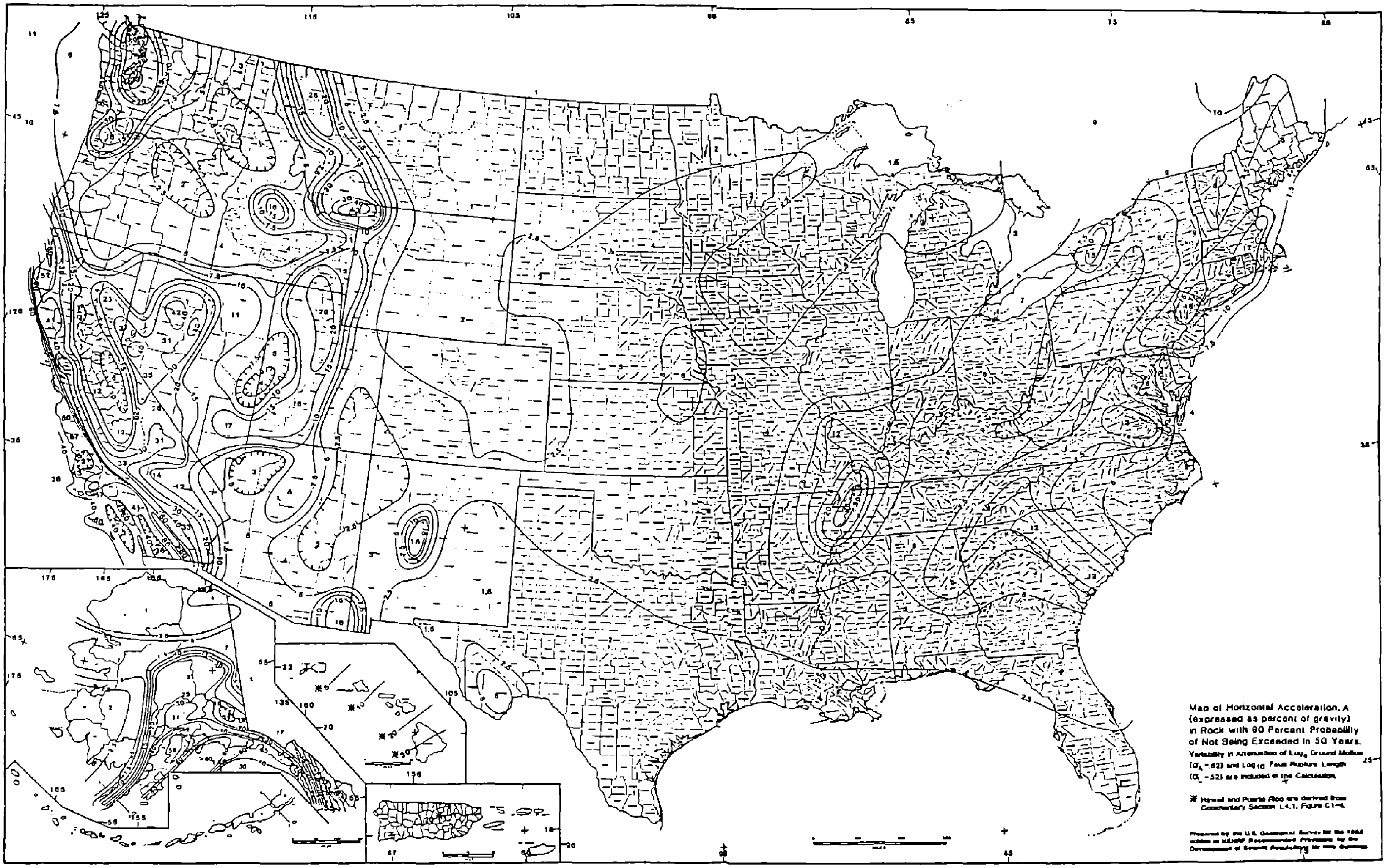

Figure A3-5 - AASHTO Seismic Zone Map

Method for Typical Facility:

For the usual case, the regionalization maps can be used to determine earthquake hazard. Regionalization maps have been developed using a probabilistic approach to indicate the earthquake hazard for a given region. That is, they indicate an estimate of the maximum earthquake intensity that statistically can be expected with a specified probability within a specified time interval. The most widely accepted parameters indicate a 90% probability within a design life of 50 years. All of the maps described above have been developed on this basis. For example, the NEHRP EPA map (Figure A3-2), indicates that there is a 90% probability that an earthquake with an effective peak acceleration coefficient of no higher than 0.40 will occur within 50 years, in San Francisco. The acceleration coefficient is, thus, a simple representation of earthquake hazard, or the expected occurrence of a future seismic event.

Examination of the maps indicates the variation by region. *It should be noted that the latest maps indicate that there are no regions within the United States that are immune from earthquakes.* All areas have some hazard that should be considered by the transportation facility manager.

The facility manager must determine whether the level of risk and design life inherent in the maps is appropriate for the facility(ies) in question (e.g. 90% certainty in 50 years). The maps are intended for typical buildings, bridges or other facilities. If closing a facility due to earthquake damage will disable a key transportation system, or if the facility is critical to emergency preparedness or post-earthquake recovery a lower risk level may be appropriate. A few examples are major water crossings like the San Francisco-Oakland Bay Bridge, key interchanges, and airports. Where the typically accepted risk levels are too low, the following method should be used.

Method for Non-Typical Facility:

In some cases the existing regionalization maps are insufficient or inappropriate for design purpose, making it necessary to perform a project specific or site specific seismic hazard analysis. These situations, among others, may be:

- design of a critical facility, as described above, for which the existing hazard results are judged to be insufficient;
- revelation of new seismological and geological evidence to nullify the existing hazard results;
- design of a facility in an area adjacent to a fault.

This section discusses the general methodology and procedure used in probabilistic hazard analysis. It is through this type of analysis that the regionalization hazard maps were produced. The general procedure used in current practice to determine the seismic hazard at a site can be divided into the following four steps:

1) Seismic Source Modeling: By using a combination of geologic evidence, geotectonic data, historic earthquakes and any other subjective or objective input, seismic sources (such as faults) are identified and modeled as line, area or dipping plane sources. In a region (such as the northeastern portion of the United States) where historic earthquakes can not be convincingly related to known tectonic features, such as an active fault, the region is divided into zones. The definition of a seismic source zone is generally deterministic. In recent years, the representation of probabilistic source-zone boundaries has also been incorporated into hazard analyses.

2) Source Seismicity and Earthquake Recurrence Rate: For each seismic source, the rate of recurrence relating the size (magnitude) of the past events with the observed

frequency is determined. This relationship is usually taken as a log-linear relationship of the following form:

$$\log N_{(m)} = a - b\,m$$

In this equation $N_{(m)}$ is the number of earthquakes per year of magnitude "m" or greater, and "a" and "b" are regression constants. The maximum possible magnitude earthquake for each source can be determined either probabilistically or deterministically using judgment and the historical record, supplemented by available information on active geologic structures.

Most of the available models assume the occurrence of earthquake events are independent and follow a homogeneous Poisson process. Time-dependent occurrence models have also been employed in hazard analyses.

3) <u>Ground Motion Attenuation</u>: For each seismic source, an attenuation relationship is determined which relates the ground motion parameters (such as peak ground acceleration, peak ground velocity and response spectrum amplitude) at a site as a function of the distance from the source to the site and the size of the earthquake. The uncertainties associated with defining these attenuation laws are incorporated into the analysis.

4) <u>Ground Motion Characteristics</u>: Using the above three steps, the ground motion characteristics at a site are estimated. In general, a computer program based on probabilistic principles is used to produce the ground motion parameter values. These parameters are generally in the following forms:

   - peak ground motion parameters, together with ground motion frequency content described by the response spectrum shape;

   - estimated earthquake time history.

Since the seismic hazard is a probabilistic estimate of expected shaking levels (represented by ground motion parameters) at a particular site with a particular probability during a specified time, the design shaking levels for a facility depend on the design life (i.e., a specified time) and the probability of exceedance of future earthquake events. This implies that:

- for a longer design life, the design shaking level for the facility should be greater;

- for an important facility providing critical services the probability of exceedance of future earthquake events should logically be lower, to minimize adverse consequences. Design shaking level, therefore, increases with lower probability of exceedance.

A 4.0 OTHER CONSIDERATIONS FOR HAZARDS ANALYSIS

In Section A 2 of this Appendix the origins and general characteristics of earthquakes in the United States, and some of the effects of earthquakes were discussed. In Section A 3, code requirements for seismic design and a method for seismic hazards evaluation were discussed. In this section, the background for some of these code requirements and the basic requirements for more rigorous seismic analysis are described.

A 4.1 Basic Earthquake Parameters

The various factors used for seismic design according to the codes and guidelines are based on actual experience records, though to a large measure in an empirical way. Because of the complexity and variability of real earthquakes, and of the behavior of real structures during earthquakes, a great deal of complexity in the derivation of seismic design parameters and in the design analyses is usually not warranted. Nonetheless, a better understanding of seismic phenomena is useful in trying to understand seismic performance of structures, and detailed analyses of complicated or expensive structures is often necessary.

Many variables affect the earthquake motion at a particular site; most important are the following:

- Earthquake magnitude
- Distance from epicenter
- Site geologic conditions.

With some knowledge of seismic history and geology, it is possible to obtain estimates of local earthquake motion parameters that are better, i.e., more accurate and representative than those implied by the various codes and guidelines.

A. 4.2 Earthquake Magnitude and Intensity

As earlier discussed, the Richter Magnitude of an earthquake is a measure of the total energy released by the earthquake. Another measure of earthquake magnitude is the seismic moment (M_o). This is a measure of the total elastic strain energy released by the fault rupture, defined by:

$$M_o = G\ A\ D$$

where:

G is the rigidity of the rock

A is the area of the rupture surface of the fault

D is the average fault displacement.

The seismic moment can be estimated, imprecisely, from geologic and other evidence, and it is mostly of scientific interest. It does point out, however, the relationship between fault displacement and length of fault rupture, and earthquake magnitude: the longer the fault rupture, and the greater the fault displacement, the greater the earthquake magnitude.

An idealized curve showing the approximate relationship between fault rupture length and earthquake magnitude is shown on Figure A4-1. For the 8.3-magnitude San Francisco earthquake of 1906, the graph shows about a 250-mile (400 km) length, which agrees with the observed length.

For magnitudes less than about 6, the fault rupture is assumed to be equidimensional; for greater magnitudes progressively greater horizontal extent of the fault rupture is required, because the typical vertical extent of fault rupture is limited to some 10-15 miles.

The focus or hypocenter of the earthquake, vertically below its epicenter, is where the fault rupture is initiated, and that is the part of the rupture that sends out the first seismic waves. The rupture then propagates in all directions along the fault. A large earthquake rupture has to propagate for a long distance, up to several hundred kilometers, which takes some time. Hence, a large earthquake is also characterized by a long duration.

For a small earthquake with a small area of fault rupture, the intensity of the earthquake motion can be assumed to decrease or attenuate equally in all directions, as from a point source, assuming similar geologic conditions in all directions. However, the energy from a large earthquake is spread over a considerable length, and a line source assumption is more appropriate. On the other hand, since the effect of a larger earthquake is spread over a greater length of fault and a greater area of influence, the close-in intensity of ground motion does not increase greatly with magnitude, especially for large magnitudes. Thus, the close-in intensity from a magnitude 8 earthquake may be only some 10 or 20% greater than that from a magnitude 7 earthquake, even if the total energy released is about 30 times greater; the main difference in effect is the areal extent and duration of shaking.

An idealized diagram showing the attenuation of ground motion intensity with distance from the causative fault is shown in Figure A4-2. Actual attenuation observations will be quite irregular because of the effects of geologic conditions and of topography. A number of attenuation curves are found in the literature, some derived from particular earthquakes, others applicable to particular regions. There is considerable variation among these attenuation curves; for example, attenuation appears to occur significantly faster in the western United States than in the central or eastern part of the country.

A 4.3 Ground Motion and the Effect of Local Conditions

Local ground motion is usually recorded by an accelerograph as ground acceleration in one or another direction. Figure A4-3 shows several examples of recorded strong ground motions. A number of differences are apparent among these accelerograms: the peak acceleration, the duration of significant shaking, the frequency content. These are a function of the earthquake magnitude and distance, the degree of attenuation during the trip from the fault to the site, and the local site conditions.

The topography of a site can increase or decrease the amplitude of motion. It is known, for example, that a site on a peak or promontory of the landscape tends to show greater amplitudes of motion than a site on the flats, assuming similar circumstances. This type of focusing of the earthquake effect is also known on a greater scale. Large-scale focusing is partly blamed for high intensity shaking in Mexico City in 1985, even though the epicenter was several hundred kilometers distant.

More important than the topography, however, is usually the geologic conditions at the site. As long as the seismic vibrations travel through competent bedrock, there is little damping, and the attenuation is largely the effect of spatial spreading. A seismic wave traveling through softer material, such as overburden soils, will tend to be modified by this passage, and measured effects at the ground surface can be quite different from the top-of-bedrock motion. The softer materials have a lower shear modulus than the bedrock, and a significant amount of damping occurs. Commonly, travel through a thickness of soft ground tends to dampen out high frequencies so that low frequencies (long periods) dominate. At the same time amplification due to resonance effects can occur. In the thick, soft clays of Mexico City, the 1985 earthquake resulted in ground surface motions with a dominant period around 2 seconds, which unfortunately was also the natural period of a large number of 8-10-story buildings, which were badly damaged.

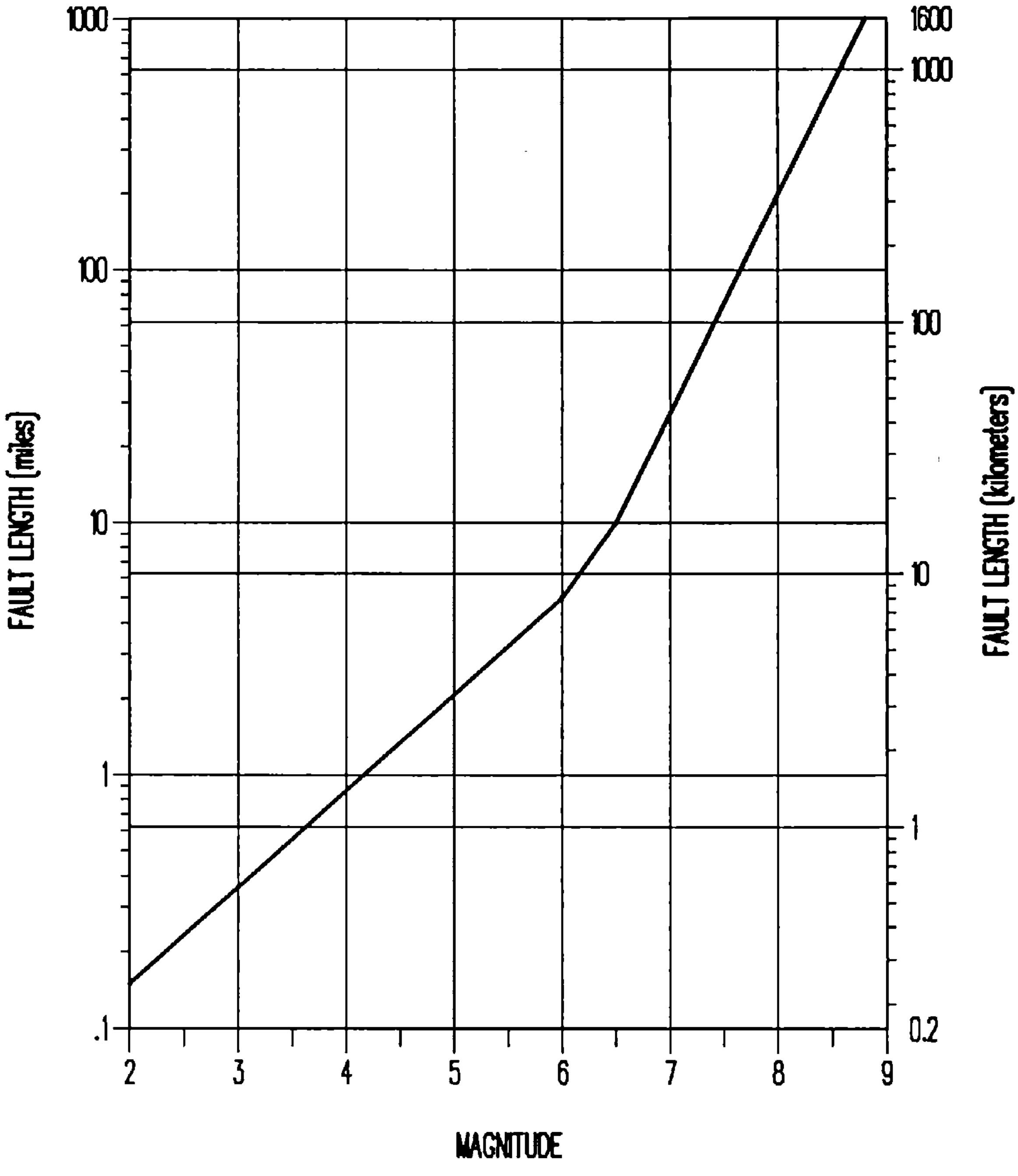

Figure A4-1 - Relationship Between Fault Rupture and Earthquake Magnitude

The amount of modification resulting from travel through a thickness of soil is dependent also on the shear strain suffered by the soil during the earth shaking, which is a function of the amplitude of particle velocity. The lower the soil modulus, the greater the shear strain, and the greater the damping ratio.

Modification of the seismic motion through the soil is now often studied using computer codes such as SHAKE. This code analyzes the soil column as a vertical cantilever of varying rigidity (shear modulus) and with varying degrees of damping, both dependent on imposed shear strain. Given a bedrock seismic input, in terms of a response spectrum or a time history accelerogram, the ground motion at the ground surface or any other depth can be estimated. Conversely, with a known (recorded) surface ground motion, one may deduce the character of the originating bedrock motion.

Thus, if bedrock motion at a site has been approximated by means of estimated earthquake sources (faults), estimated magnitudes of earthquakes, and distance attenuation traveling through bedrock, then the seismic motion at the ground surface can also be approximated.

For these kinds of analysis, an earthquake time history is usually required. Often a recorded time history is used, which has the appropriate frequency content and duration; the acceleration amplitude is usually scaled. When a suitable time history is not available, artificial time histories are often generated using random number generation by computer and other manipulation so as to produce the desired frequency mix, duration and maximum amplitude.

A 4.4 Soil - Structure Interaction

Foundations, piles, piers, or basements modify the seismic motion through soil-structure interaction, just as the seismic motion is modified and amplified as it moves up through a building or other structure. Just how this modification occurs and what results is little known, and these effects are usually ignored, though it is not clear that this is a conservative assumption in all cases. When underground structures are considered, however, this soil-structure interaction is all that occurs, and there is no appreciable amplification based on the natural period of the structure, as there is for surface structures.

The underground structures of concern for this study are subway tunnels and stations embedded in soil, as well as highway tunnels, culverts, and other structures surrounded by soil or rock. What matters for the interaction is the relative rigidity of the structure as opposed to the surrounding geologic material, and the strains imposed on the medium by the earthquake.

Where the soil or rock is of substantial rigidity, and the structure flexible, the imposed seismic deflection will be small, the structure will move with the ground, and there will be no permanent effect of the ground motion. In a much softer ground, a soft clay for example, ground motions can be significant, and a flexible structure, such as a typical, circular subway tunnel structure can suffer minor distress. Larger, more rigid structures are more difficult to assess.

It has been the practice for subway station structures, which typically are relatively rigid box structures surrounded by soil, to estimate the displacement of the soil as if the structure were not there, and then to impose this *free-field displacement* on the structure to calculate stresses and moments. This method works well when the surrounding soils are of good quality and rigidity and the imposed displacement is small. However, in a soft soil, such as typical soft marine or estuarine clays, the seismic displacement can be very large, and the imposition of such displacements on a rigid concrete structure is inappropriately conservative. Here it is necessary to take into account the soil-structure interaction, and it is no longer appropriate to ignore the structure stiffness, because it can be much greater than that of the soil.

Soil-structure interaction problems such as this can be analyzed using computer codes such as FLUSH, which is a finite element program particularly suited to deal with earthquake problems. With this method it has been found that when the structure is in fact more rigid than the soil it displaces, then the displacement forced onto the structure is substantially reduced.

A 4.5 Uncertainty and Complexity

The preceding paragraphs emphasize that a variety of earthquake parameters are required for an assessment of earthquake effects on structures, or for seismic design, depending on the character of the structure. For relatively simple structures or in regions of low seismicity, all that is needed may be a seismic zonation map and a copy of the building code applicable to that area. In other, more exposed cases it may be necessary to explore some or most of the following kinds of earthquake characteristics:

- Earthquake magnitude and distance from facility (one, or several capable faults)
- Regional seismicity
- Attenuation through the bedrock transmission
- Seismic acceleration history (modified recorded history or artificial)
- Modification of the seismic wave through the soil column
- Focusing of the wave
- Peak or effective acceleration
- Peak or effective velocity
- Peak displacement or relative displacement
- Maximum induced shear strain.

For large, important, or critical facilities, the selection of the approach to an "appropriate" design philosophy is largely a combination of blending judgment and existing regulations. For example, the $5 - billion Central Artery/Tunnel (CA/T) Project in Boston created a special set of Seismic Design Criteria for Underground Structures (Appendix B of Section V of the Project Design Criteria). The Table of Contents for this several hundred page document appears in Table A4-1, and illustrates the complexity of addressing this issue on a large, critical project.

Facility complexity, in and of itself, introduces an element of uncertainty into seismic assessments and seismic design, since simplifying assumptions must be made to relate and interconnect various parts of a facility analysis. In addition, the uncertainties of selecting such items as attenuation factors, and acceleration history at a particular site make the assessment of the actual complete facility design level of conservatism, or "risk", a very subjective process. In terms of costs, both design and construction, the uncertainty and complexity factors can have great impacts.

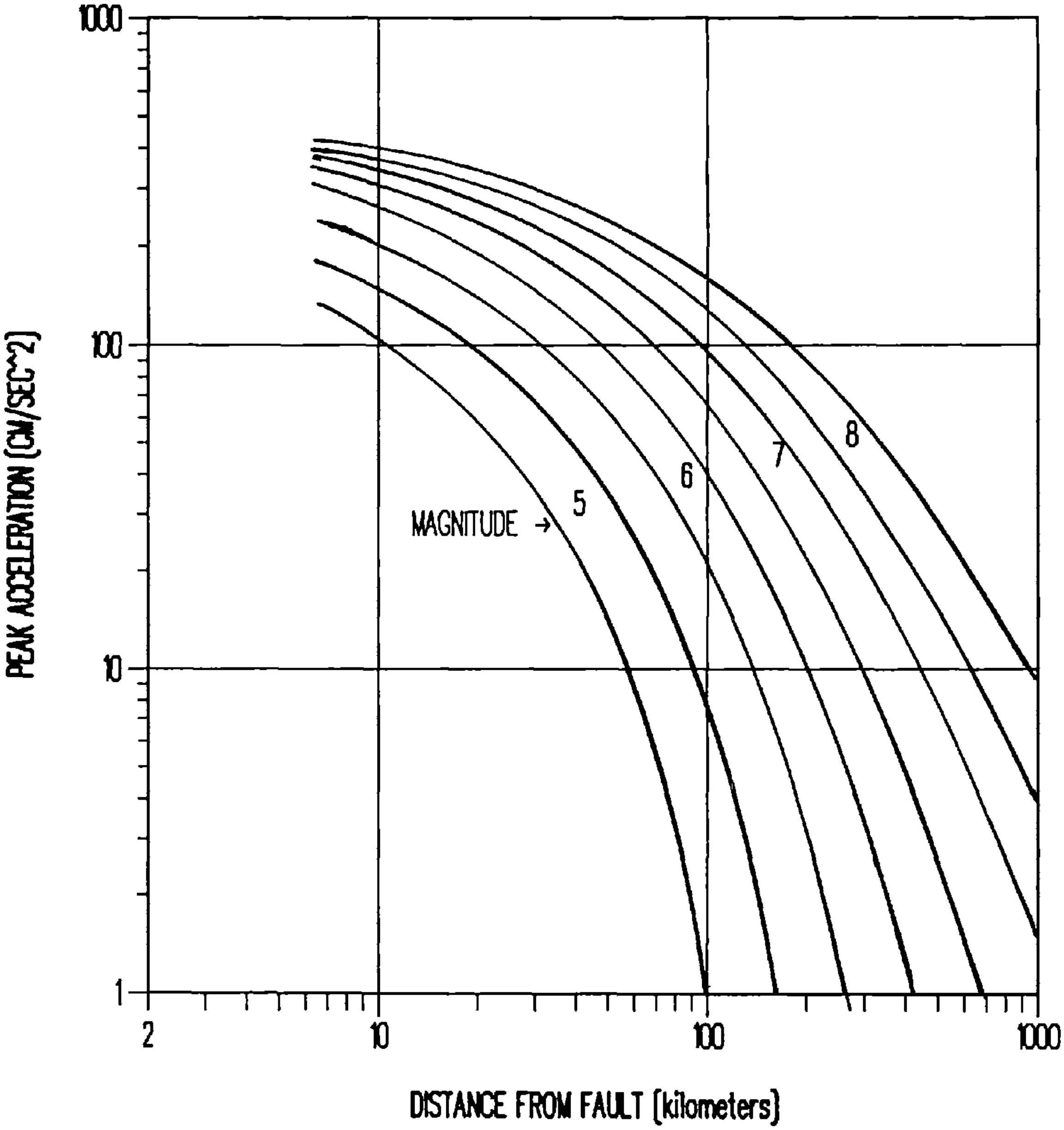

Figure A4-2 - Attenuation of Ground Motion Intensity With Distance From Fault

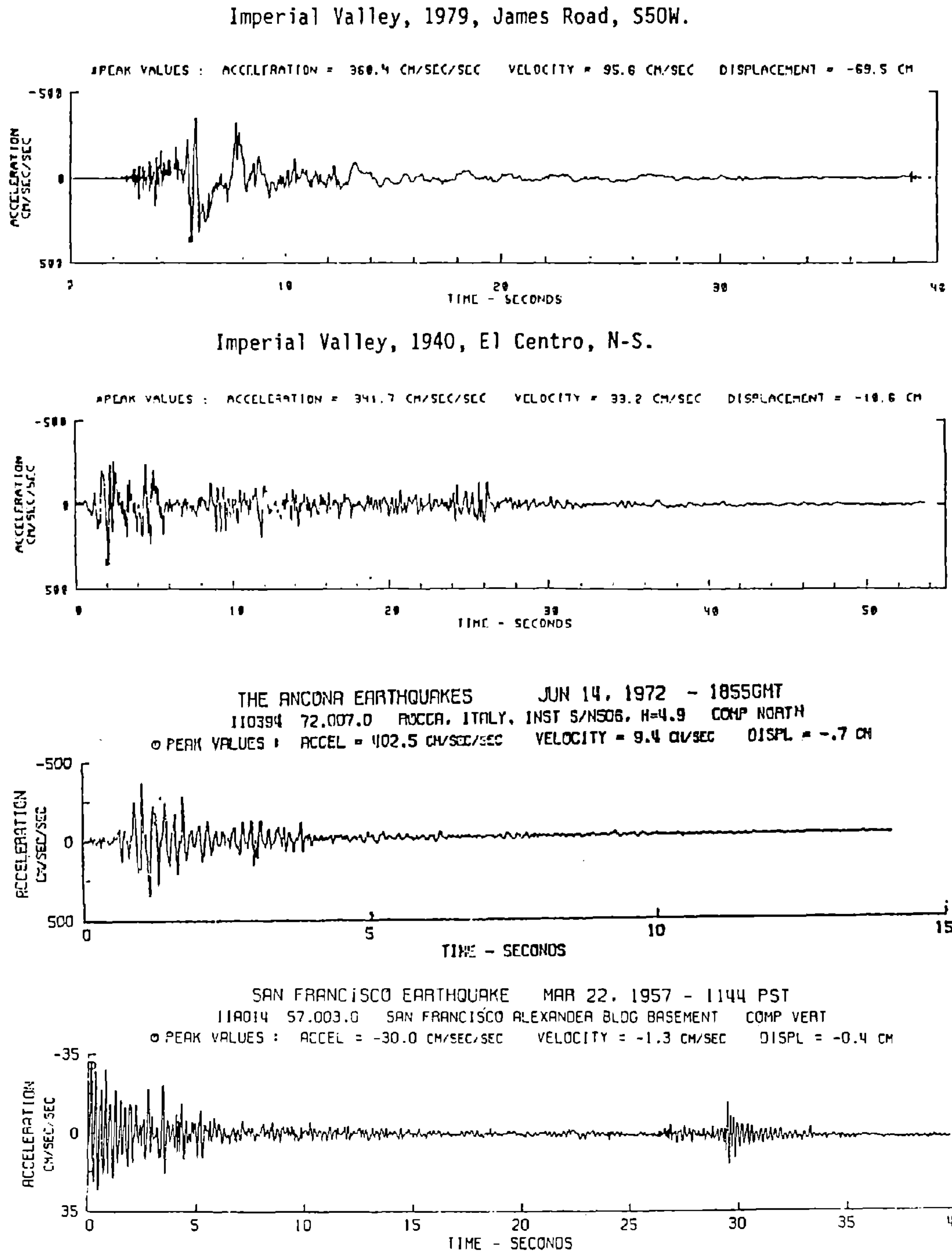

Figure A4-3 - Samples of Recorded Strong Ground Motions

TABLE A4-1

Table of Contents of CA/T Project Seismic Design Criteria for Underground Structures

A 5.0 ONGOING RESEARCH

There is a significant amount of basic and applied research underway in the United States regarding the definition of the geographic variations in levels of seismic hazard potential throughout the country.

Within the continental United States, California and Florida are extreme examples of high seismic hazards to essentially zero seismic hazards. Alaska and Hawaii present unusual problems, but both contain extremely active seismic zones, with the potential for great damage and loss.

There are, of course, controversies and unresolved issues regarding both the origin and cause of all types of earthquakes and the appropriate level of intensity to select for a given geographic location. For example the large Charleston, South Carolina and Cape Ann, Massachusetts quakes of 1886, and 1755, respectively have not been shown to be fault-related. However, given the vast volume of study to date, the FEMA Seismic Regionalization Maps form the best documented basis for selecting the basic "design" earthquake parameters in a certain geographic location.

The effective peak acceleration, EPA, values given on the maps can be used as a basis for an initial evaluation of seismic hazard potential. As noted on the maps, large areas of the United States, between 1/3 and 1/2 of the continental US, would be subject to only nominal values of acceleration, i.e., 0.05 in./sec^2 or less.

The problem of translating the basic seismic parameters from the FEMA maps into a longer list of site-specific design criteria and design parameters in order to complete a seismic evaluation for a new or an existing facility is a more formidable task. Local geologic and geotechnical studies are generally necessary to permit this process to occur. However, significant studies have already been performed in the most active seismic areas, or in areas where large, complex and/or critical projects have been sited.

NEHRP is working on this very issue, i.e., converting the vast amount of existing and on-going research in the seismic area into usable codes and design guidelines. Table A5-1 shows the objectives of selected sub-tasks aimed at achieving a greater degree of seismic safety in Federal buildings. FY 92 efforts include developing regulations to facilitate FEMA's implementation of Executive Order 12699 relative to new buildings, developing standards for existing Federal buildings, and publishing *Recommended Practice 2.1, Guidelines and Procedures for Implementation of Executive Order on Seismic Safety*.

Table A5 - 1

Objectives of Greater Seismic for Federal Buildings

Objectives/Origins/Tasks	1992	1993	1994	1995-96
Objective: **Achieve a greater degree of seismic safety for Federal buildings.**				
Origins: **Earthquake Hazards Reduction Act of 1977, as amended, Sec. 2(7), Sec. 5, and Sec. 8; Executive Order 12699.**				
e. Develop and support implementation of seismic-resistant design provisions, including supporting documents that will enhance the seismic safety of new Federal buildings. (FEMA/NIST)	Provide technical assistance through ICSSC to Federal agencies in implementing Executive Order 12699 on the seismic safety of all activated Federal construction.	Continue.	Continue.	Continue.
	Develop regulations to facilitate FEMA's implementation of Executive Order 12699 on the seismic safety of all new buildings affected by FEMA's programs.	Issue final regulations. Report to the President and the Congress on the development of regulations to implement the Executive Order.		Report to the President and Congress on assessment of the implementation of the Executive Order by Federal agencies.
f. Develop seismic-resistant design provisions, including supporting documents that will enhance the seismic safety of existing Federal buildings and single-family homes. (FEMA/NIST)	Using ICSSC, develop standards for issuance by the President for assessing and enhancing the seismic safety of existing buildings constructed for or leased by the Federal government, and methods for application to federally assisted and regulated buildings.	Continue.	Complete standards by December 1993 for preparation of the President's Executive Order.	Issue Executive Order on standards and report to Congress on methods of application by December 1994.

Table A5 - 1

Objectives of Greater Safety for Federal Buildings (Continued)

Objectives/Origins/Tasks	1992	1993	1994	1995-96
g. Support the ICSSC and its efforts to assist Federal departments and agencies to develop earthquake hazard-reduction measures and to implement Executive Order 12699. (FEMA, NIST)	Publish *Recommended Practice 2.1, Guidelines and Procedures for Implementation of Executive Order on Seismic Safety* and other documents related to implementation of Executive Order 12699 on seismic safety of new buildings. Conduct workshop on preparing Federal agency regulations.	Repeat implementation workshop.	Assess member agency response to Executive Order 12699.	
	Proceed with development of standards for existing Federal buildings by planning and initiating a trial design program.	Incorporate results of trial designs into draft standard, ballot standard by full ICSSC, amend if necessary.	Submit finalized standard to the office of the President by January 1, 1994.	President to sign Executive Order on seismic safety of existing Federal buildings by December 1, 1994.
	ICSSC members to participate in formulating a plan for the development of seismic standards for lifelines.	Contribute to development of lifelines standards for use by Federal agencies, following priorities recommended by national plan.	Continue.	Continue.

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10. "NEHRP Maps", 1991 Edition

Map 1.	ATC	County Map - Coefficient Aa
Map 2.	ATC	County Map - Coefficient Av
Map 3.	ATC	Contour Map for coefficient Aa
Map 4.	ATC	Contour Map for Coefficient Av
Map 5.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 6.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 7.	USGS	0.1 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 8	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 9.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
Map 10.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 11.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
Map 12.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years (California)

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APPENDIX B

SEISMIC VULNERABILITY OF TRANSPORTATION FACILITIES

B

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SEISMIC VULNERABILITY OF TRANSPORTATION FACILITIES

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EXECUTIVE SUMMARY

Appendix B is one of three appendices that provide a technical basis for "Seismic Awareness: Transportation Facilities", a report written for transportation facility managers to educate them to the potential for seismic hazards directly effecting their facilities (new and existing), to present a suggested approach to evaluate facility vulnerability and to address seismic design aspects of new and existing facilities.

Appendix B discusses the *vulnerability* of transportation facilities. Vulnerability here refers to the likely consequences of the expected seismic event on a particular structure. Unlike seismic hazard, vulnerability applies to specific structures. It is dependent on the expected seismic hazard, as well as the structural characteristics of the facility and the *local* geology of the site. Vulnerability is also distinguished from hazard in that hazard is a naturally occurring phenomena that man is unable to affect, while vulnerability is dependent on human factors that we have control over and can change - like the construction of a building or the slope of an earth embankment.

Appendix A, included in a separate volume describes the nature of seismic *hazards* in the United States. Appendix C, also included in a separate volume, summarizes current seismic design and retrofit practices in the United States.

The task of evaluating the seismic vulnerability of transportation facilities is considered in two steps:

1. Preliminary screening

2. Detailed evaluation

Transportation facilities are made up of different types of structures, substructures and equipment. Several broad categories of facility types representing the major functional components of each transportation system are selected for this study:

- *Highway Transportation System*
- *Railway Transportation System*
- *Air Transportation System*
- *Sea/Water Transportation System*

A methodology is presented that can be followed by transportation facilities managers to perform an initial screening of their facilities to determine their vulnerability to seismic events. This methodology uses peak ground acceleration, determined in accordance with Appendix A, and a set of curves developed for this purpose for each of the above categories. The potential damage and loss is read from these curves as a percent of replacement value.

The potential damage and loss figure obtained in this method is a good indication of seismic vulnerability. In one number it summarizes the seismic hazard and vulnerability for a given facility in a given area of the United States and gives an indication of the consequences of a likely earthquake in terms of replacement cost.

The method presented in this section is a valid approach for preliminary screening for seismic vulnerability. It can be useful for large groups of structures/facilities with similar characteristics to give an overall view of the vulnerability of each group as a whole, or to set up priorities of individual structures within the group. However, because of the great complexities and variations of real structures it should not be relied upon to give accurate results for a specific structure. Recommendations are given for a more detailed analysis that should be used in these situations.

B 1.0 INTRODUCTION

B 1.1 Background

Appendix B is one of three appendices that provide a technical basis for "Seismic Awareness: Transportation Facilities", a report written for transportation facility managers to educate them to the potential for seismic hazards directly effecting their facilities (new and existing), to present a suggested approach to evaluate facility vulnerability and to address seismic design aspects of new and existing facilities.

Appendix B discusses the *vulnerability* of transportation facilities. Vulnerability here refers to the likely consequences of the expected seismic event on a particular structure. Unlike seismic hazard, vulnerability applies to specific structures. It is dependent on the expected seismic hazard, as well as the structural characteristics of the facility and the *local* geology of the site. Vulnerability is also distinguished from hazard in that hazard is a naturally occurring phenomena that man is unable to affect, while vulnerability is dependent on human factors that we have control over and can change - like the construction of a building or the steepness of an earth slope.

Appendix A, included in a separate volume, describes the nature of seismic *hazards* in the United States. These hazards include the probability of occurrence of an earthquake in a certain area, as well as its likely intensity. Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes.

Appendix C, also included in a separate volume, summarizes current seismic design and retrofit practices in the United States. It gives some history on seismic design, illustrating the evolution of seismic design technology. It also explains in some detail the methods currently used for the design of different types of transportation facilities and components. Finally, it reviews the economic considerations involved and emphasizes the cost-effectiveness of incorporating seismic design elements into new structures during pre-construction design as opposed to retrofitting existing structures.

B 1.2 Purpose of This Appendix

Appendix B provides an overview of different types of facilities that the various transportation agencies own and operate, describes the types of damage to which transportation facilities are vulnerable and discusses a method of determining vulnerability of a particular facility to a regional or site-specific seismic hazard.

B 1.3 Approach

The task of evaluating the seismic vulnerability of transportation facilities will be considered in two steps:

- Preliminary screening
- Detailed evaluation

It is the goal of this appendix to develop broad, general guidelines to approach the first step, the preliminary screening of those facilities. However, it is our aim that those guidelines will also provide the general basis for the more detailed evaluation. That is, where possible, techniques developed will be structured such that later development will convert them from qualitative screening tools into quantitative evaluation methods.

B 2.0 REVIEW OF PAST EARTHQUAKES

B 2.1 Introduction

Earthquakes are a regular occurrence throughout the U.S., though most occur in the West, (primarily California), Alaska, and Hawaii. The intensity and duration of the events also varies, and very few are major events. Figure B2-1 depicts the scatter of historic earthquakes in the U.S. Figure B2-2 shows a seismicity map for the New Madrid (Missouri) area where one of the largest earthquakes ever recorded in the U.S. occurred in 1812. Figure B2-3 shows the probability of earthquakes that could occur in California in the next 30 years. Table B2-1 lists cities of 2 million population or more that are in high seismic zones.

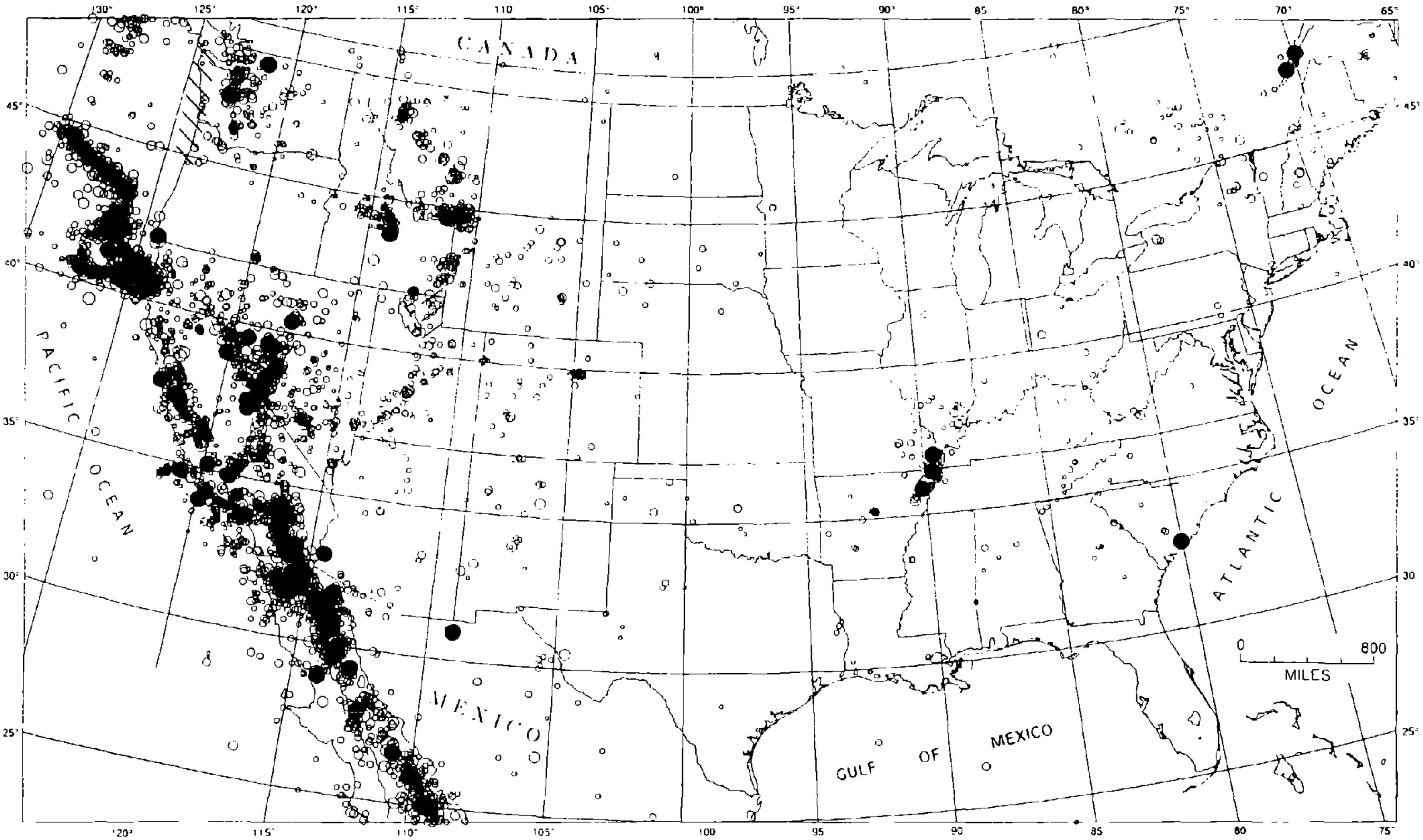
The following sections summarize the effects of five significant events that occurred in the U.S. within the past 20 years, highlighted by the Loma Prieta event of 1989 in which 62 people died and $6 billion in damage was inflicted. These sections review the performance of various kinds of facilities, with property damage, and other adverse effects noted. Also, consideration is given to possible mitigative efforts that, if in place at the time of the event, could have reduced the magnitude of the adverse consequences. This review provides a sampling of some of the potential damage to which structures are vulnerable.

B 2.2 Anchorage (1964)

At 5:36 p.m. on Good Friday, March 27, 1964, Anchorage and all southern Alaska within a radius of about 400 miles of Prince William Sound was struck by perhaps the strongest earthquake to have hit North America within historic time. The magnitude of this great quake has been computed by the U.S. Coast and Geodetic Survey at 8.5 on the revised Richter scale. Its epicenter was about 80 miles east-southeast of Anchorage near the head of Prince William Sound. Reportedly, the quake was felt throughout most of Alaska, including such remote points as Cape Lisburne, Point Hope, Barrow, and Umiat, 600 to 800 miles north of the epicenter on the Arctic Slope of Alaska, and at Fort Randell, 800 miles south-west at the tip of the Alaska peninsula.

The duration of the earthquake at Anchorage can only be surmised owing to the lack of strong-motion seismograph records. Although seismographs have since been installed, none was present in Southern Alaska at the time of the quake. Intense seismic motions seem to have lasted 3 to 4 minutes, possibly longer. Where localized ground displacements occurred, as in or near landslides, strong motions may have lasted appreciably longer, after strong seismic shaking had ceased. The durations at Anchorage, timed by several eye witnesses on wrist or pocket watches, ranged from 4 minutes 25 seconds to 7 minutes. Even longer durations were reported outside the Anchorage area. In some areas people reportedly were thrown to the ground by the force of the acceleration and were unable to regain their footing.

Total earthquake damage to property in the Anchorage area could not be fully evaluated and perhaps will never be fully known. Nine lives are reported to have been lost - five in the downtown area, three at Turnagain Heights, and one at the International Airport. In less than 5 minutes, more than 2,000 people, including apartment dwellers, were rendered homeless, according to press estimates. The loss of life was less in Anchorage than in some of the small coastal towns, where many people were killed by sea waves. But Anchorage, because of its much greater size, bore the brunt of the property damage and property losses reportedly were greater there than in all rest of Alaska combined.



All Historic Earthquakes (Magnitude 5.5 or Larger), Largest Circles; (Magnitude 5 to 5.43 since 1925) - Smaller Circles; (Magnitude 4 to 4.9 since 1962) - still Smaller Circles; and (Magnitude 3.5 to 3.9 since 1975) - Smallest Circles

Figure B2-1 - Historic Earthquakes in the United States

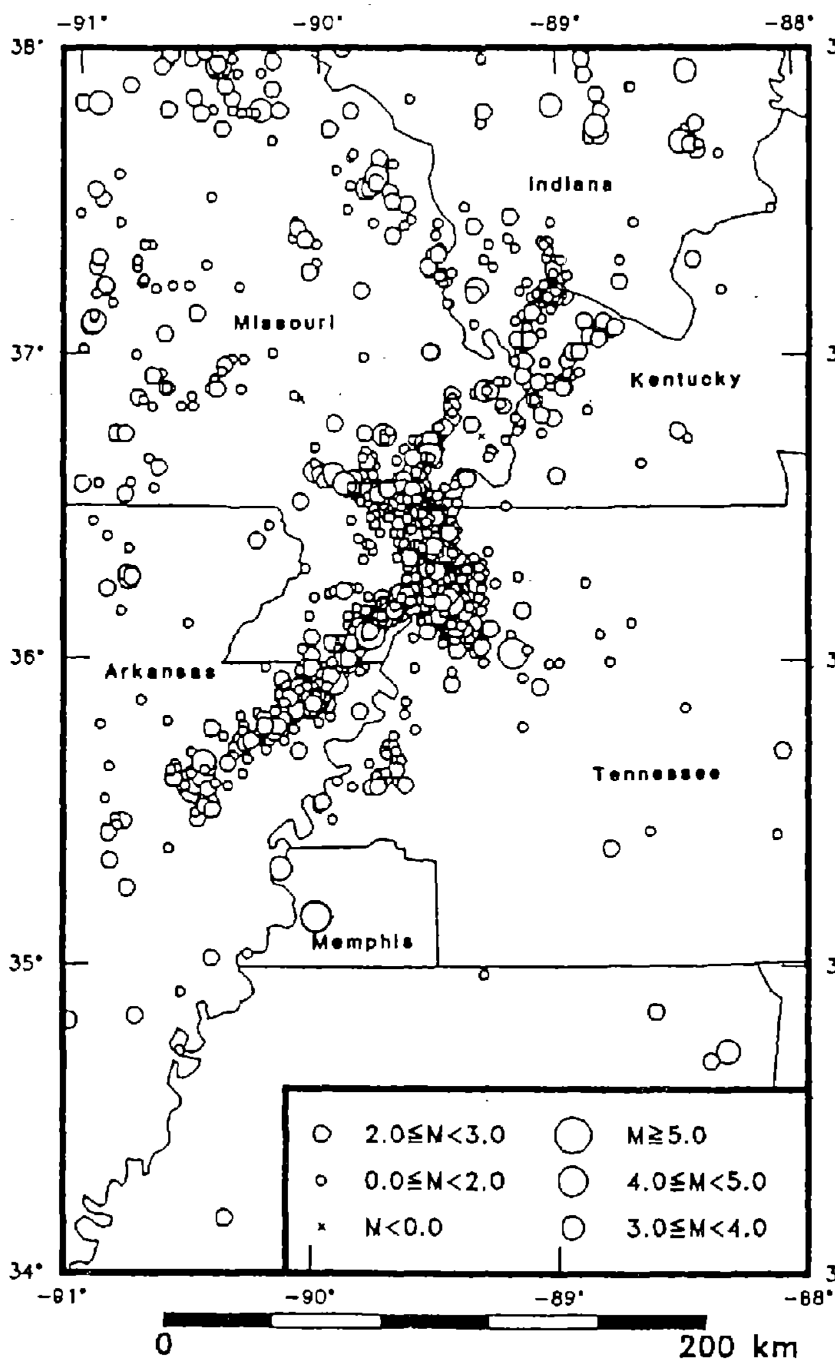


Figure B2-2 - New Madrid Seismicity Map (1974 - 1990)

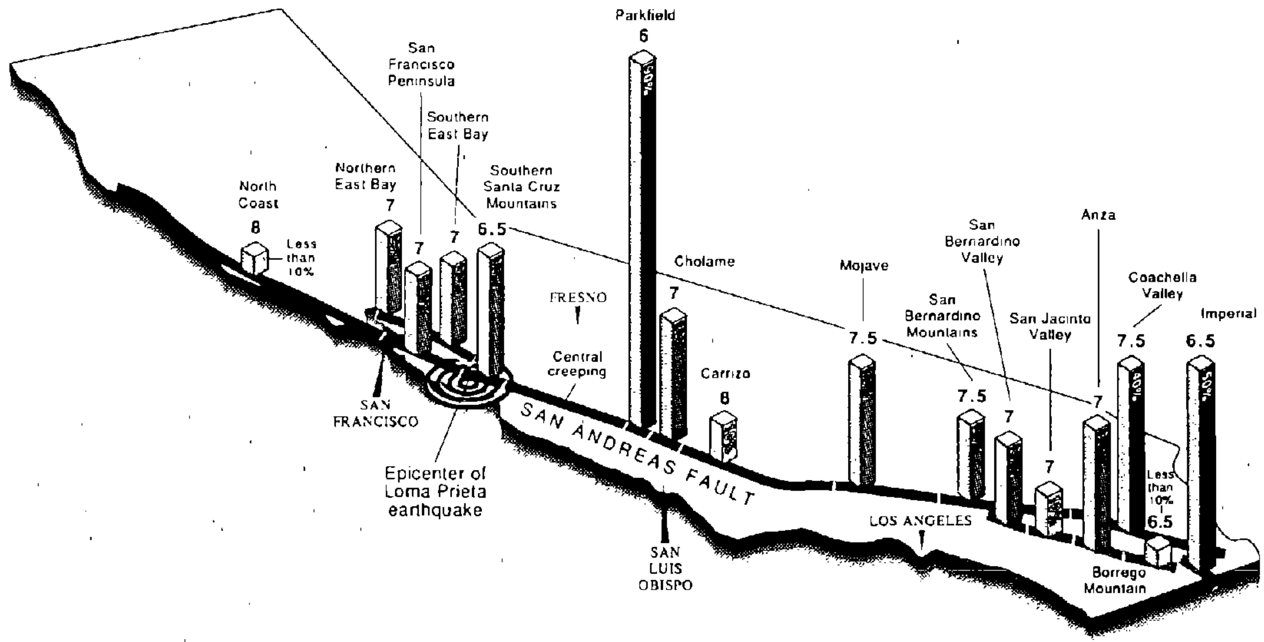


Figure B2-3 - Earthquake Probability in California

Table B2-1

Cities of 2 Million Population or More in Magnitude 7 Seismic Zones

	1975	2000
Mexico City	11.6	26.0
Tokyo	16.4	20.0
Jakarta	5.5	13.0
Los Angeles	9.0	11.0
Beijing	8.9	11.0
Lima	3.7	9.1
Algiers	1.6	5.1
Baghdad	2.7	7.5
Naples	3.8	4.3
San Francisco	3.0	5.0

Early estimates by the Office of Emergency Planning indicated that about 75 percent of the City's total developed worth was measurably damaged. Early estimates of total damage, however, tended to be larger than later ones. According to the Anchorage Daily Times of April 9, 1964, 215 homes were destroyed in Anchorage and 157 commercial buildings were destroyed or damaged beyond repair. At Turnagain Heights alone, 75 or more dwellings were destroyed. Estimates by the Daily Times placed the damage at about $200 million. Later, the Office of Emergency Planning estimated the total damage to Alaska at about $537,600,000, of which about 60 percent was sustained by the Anchorage area. The final total damage estimate for Alaska, exclusive of personal property and loss of income, was about $311 million. Scores of buildings throughout Anchorage sustained damage requiring repairs costing many thousands of dollars.

Roads and railroad facilities were badly damaged. In the downtown area, many streets were blocked by debris, and in landslide areas, streets and roads were completely disrupted. Differential settlement caused marginal cracking along scores of highway fills throughout the Anchorage Lowland. In the Alaska Railroad yards where landslide debris spread across trackage and damaged or destroyed maintenance sheds, an estimated $2,370,700 damage was sustained. Cars and equipment were overturned, and car shops were damaged by vibration. Along the main line of the railroad, bridges failed, fills settled, and tracks were bent or buckled. At Potter, near the south margin of the Anchorage Lowland, several hundred feet of track was carried away in an area that has had a long history of repeated sliding.

At the Anchorage International Airport, the control tower failed under seismic vibration and collapsed to the ground, killing one occupant and injuring another. The airport terminal building, although tied structurally to the tower, was only slightly damaged.

Damage was caused by direct seismic vibration, by ground cracks, and by landslides. Direct seismic vibration affected chiefly multistory buildings and buildings having large floor areas, probably because of the long period and large amplitude of the seismic waves reaching Anchorage. Most small buildings were spared. Ground cracks caused capricious damage throughout the Anchorage Lowland. Cracking was most prevalent near the heads or within landslides but was also widespread elsewhere. Landslides themselves caused the most devastating damage.

Triggering of landslides by the earthquake was related to the physical engineering properties of the Bootlegger Cove Clay, a glacial estuarine-marine deposit that underlies much of the Anchorage area. Most of the destructive landslides in the Anchorage area moved primarily by translation rather than by rotation. Thus, all the highly damaging slides were of a single structural dynamic family despite wide variations in size, appearance, and complexity. They slid on nearly horizontal slip surfaces after loss of strength in the Bootlegger Cover Clay. Some failures are attributed to spontaneous liquefaction of sand layers.

In most translator slides, damage was greatest in graben areas at the head and in pressure-ridge areas at the toe. Many buildings inside the perimeters of slide blocks were susained little damage despite horizontal translations of several feet. The large Turnagain Heights slide, however, was characterized by a complete disintegration and drastic lowering of the prequake land surface. Extensive damage back from the slide, moreover, was caused by contless tension cracks. Geologic evidence indicates that landslides similar to those triggered by the March 27 earthquake have occurred in the Anchorage area at various times in the past.

The very large magnitude of this earthquake, coupled with the soft, loose, and deep soil deposits, combined to produce severe damage that would have been difficult to prevent with reasonable engineering and construction procedures. The extensive mass soil movements reflected the vulnerability of the foundation materials. The town of Valdez, only 45 miles from the epicenter was essentially destroyed and, after some consideration, was rebuilt on a different, more stable area.

Still, much was learned from this event, since many small structures, that were part of mass slides, or other large lateral movements, were not structurally damaged, though because of excessive movement and damage around them (tension cracks in the ground and loss of utilities) their useful function was lost.

Some examples of the damage sustained in this earthquake are shown in Figures B2-4, 5 and 6.

B 2.3 San Fernando (1971)

The San Fernando earthquake, with a Richter magnitude of 6.6, occurred at 6:01 A.M. on February 9, 1971. The earthquake's epicenter was in the San Gabriel Mountains located north of Los Angeles.

The earthquake caused 58 deaths, (47 were due to the collapse of the non-earthquake resistant Veterans Hospital), and over 2,500 hospital-treated injuries in the San Fernando Valley, which had a population of over 1,200,000 at the time of the quake.

Strong ground motion lasted 12 seconds, and peak ground accelerations as high as 1.25g were recorded in the vicinity of the Pacoima Dam. These motions were greater than any previously recorded. The damage to nearby wood frame dwellings and to hospitals indicated that the building codes needed revision.

Direct damage to buildings and other structures exceeded $0.5 billion. This amount was divided about equally between public and private property. Most of the severe damage and major losses were along the southern foothills of the San Gabriel Mountains and along a narrow band of surface faulting (see Appendix A for definition) that runs east-west on the valley floor.

The epicenter was close to four metropolitan freeway routes with numerous bridges. These bridges sustained heavy damage. A total of 62 bridges were damaged, mostly in a zone 5 miles long, located 6 to 10 miles from the epicenter. The observed damage identified many code deficiencies, and the earthquake resulted in profound changes to seismic code provisions.

The collapse of the Foothill Freeway overpass (Figure B2-7) was caused by inadequate support width at the girder supports. The earthquake movement caused the girders to slide off the piers. It was noted that adjacent bridges with wider supports experienced movement, but not collapse (see Figure B2-7).

Other deficiencies were noted in the reinforcing steel of pier columns. Inadequate *spiral* reinforcing, tying the vertical bars together allowed the concrete within to crumble, and the vertical bars to buckle (see Figure B2-8). Also, inadequate embedment of vertical reinforcing bars in concrete footings allowed the bars to pull out of the footing under earthquake loading (see Figure B2-9).

Serious damage also was sustained by buildings considered earthquake-resistant at the time, by dams located up-stream from densely populated areas, and by public utilities and roadways, that are the lifelines of cities (see Figure B2-10). Damage to one-story industrial and commercial structures with wood roof systems was common and severe in the area of strong ground motion. High-rise buildings in the Los Angeles area generally suffered little damage. In this earthquake, health care facilities were especially hard hit.

Buildings that survived the earthquake without collapse met the intent of the building code, however, from an economic viewpoint, many may be considered failures. Facilities that were undamaged or only slightly damaged were able to quickly reopen. Facilities such as Olive View Hospital (Figure B2-11), Pacoima Memorial Lutheran Hospital, and Holy Cross Hospital suffered major damage and required years to fully recover. The resulting loss of market share and revenue far exceeded the property losses.

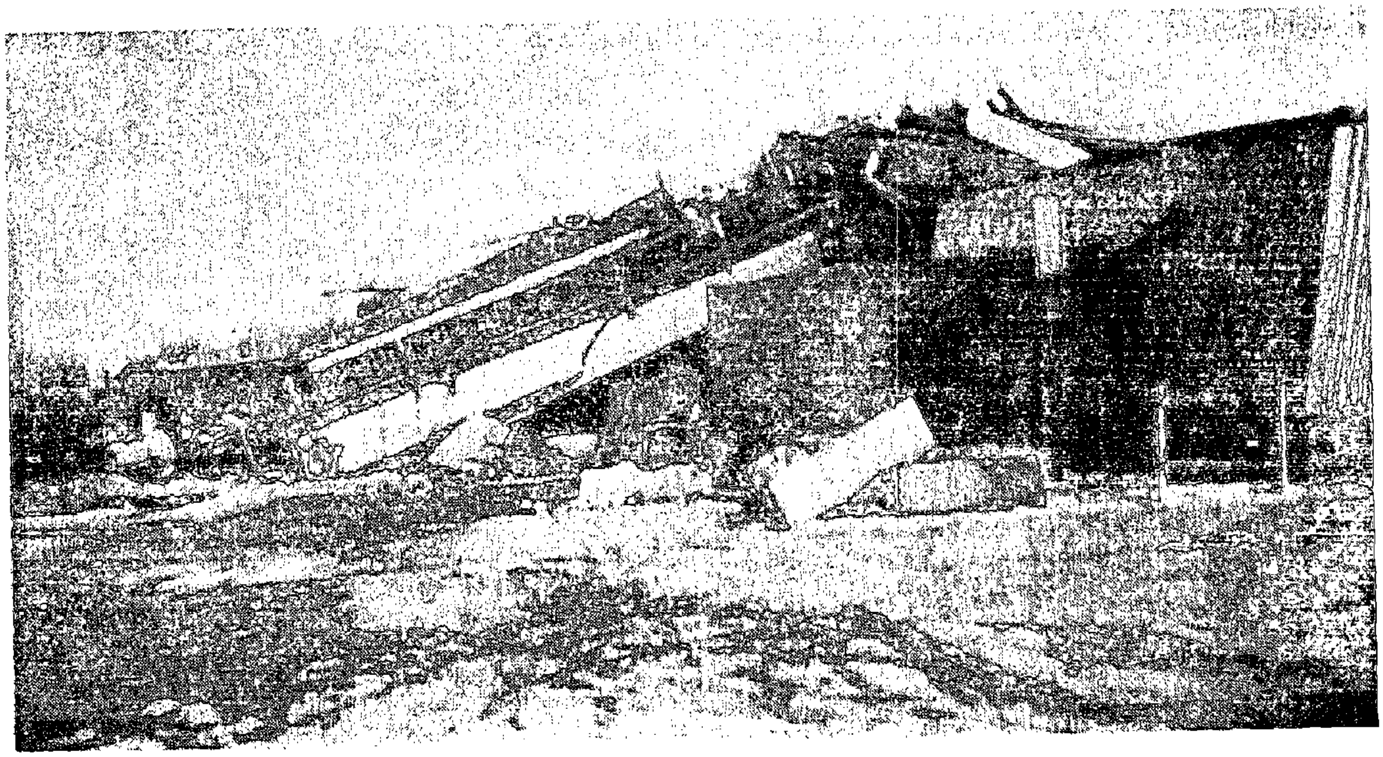


Figure B2-4 - Damage to Air Traffic Control Tower, Anchorage (1964)

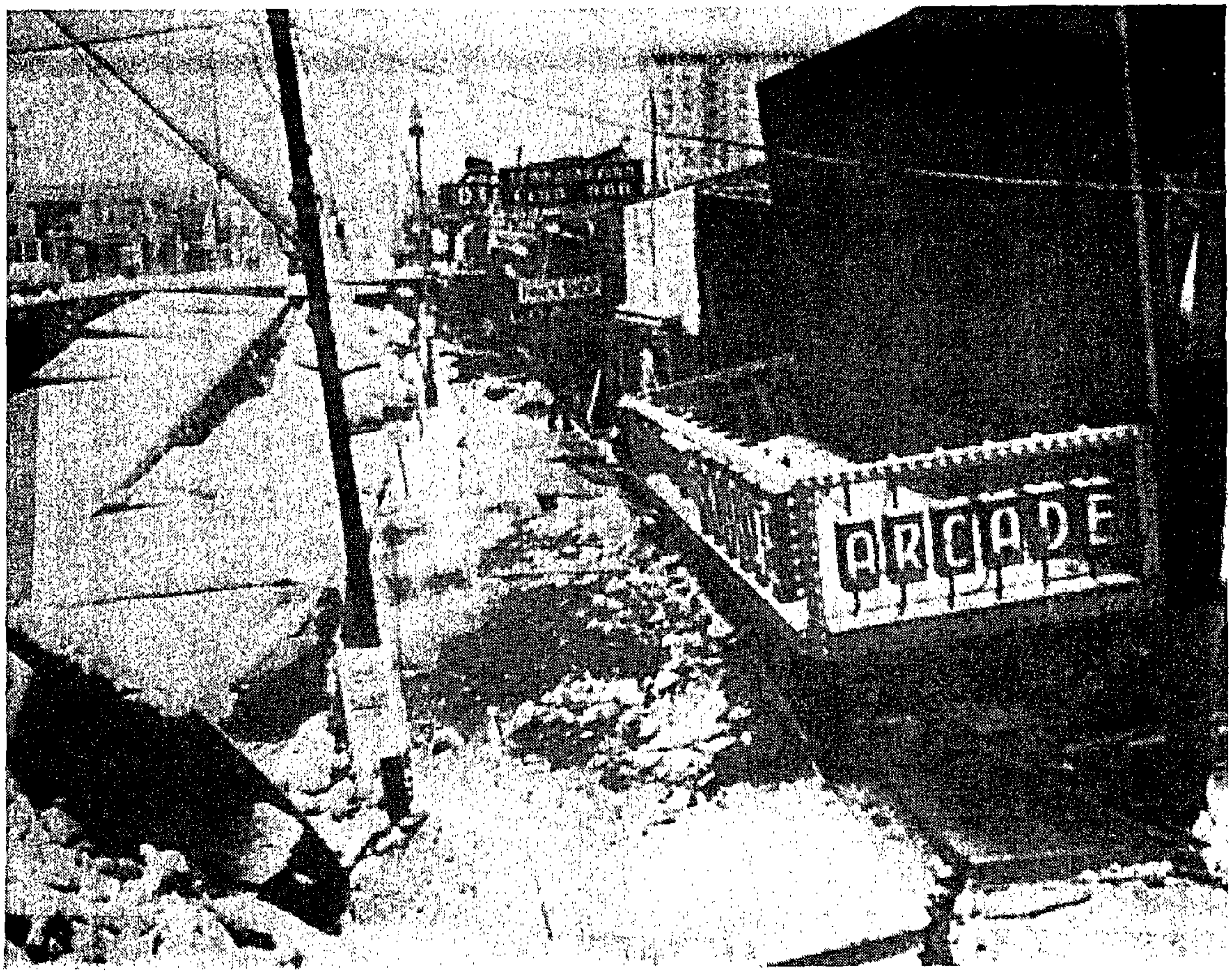


Figure B2-5 - Ten Foot Subsidence of Street at Head of Landslide, Anchorage (1964)

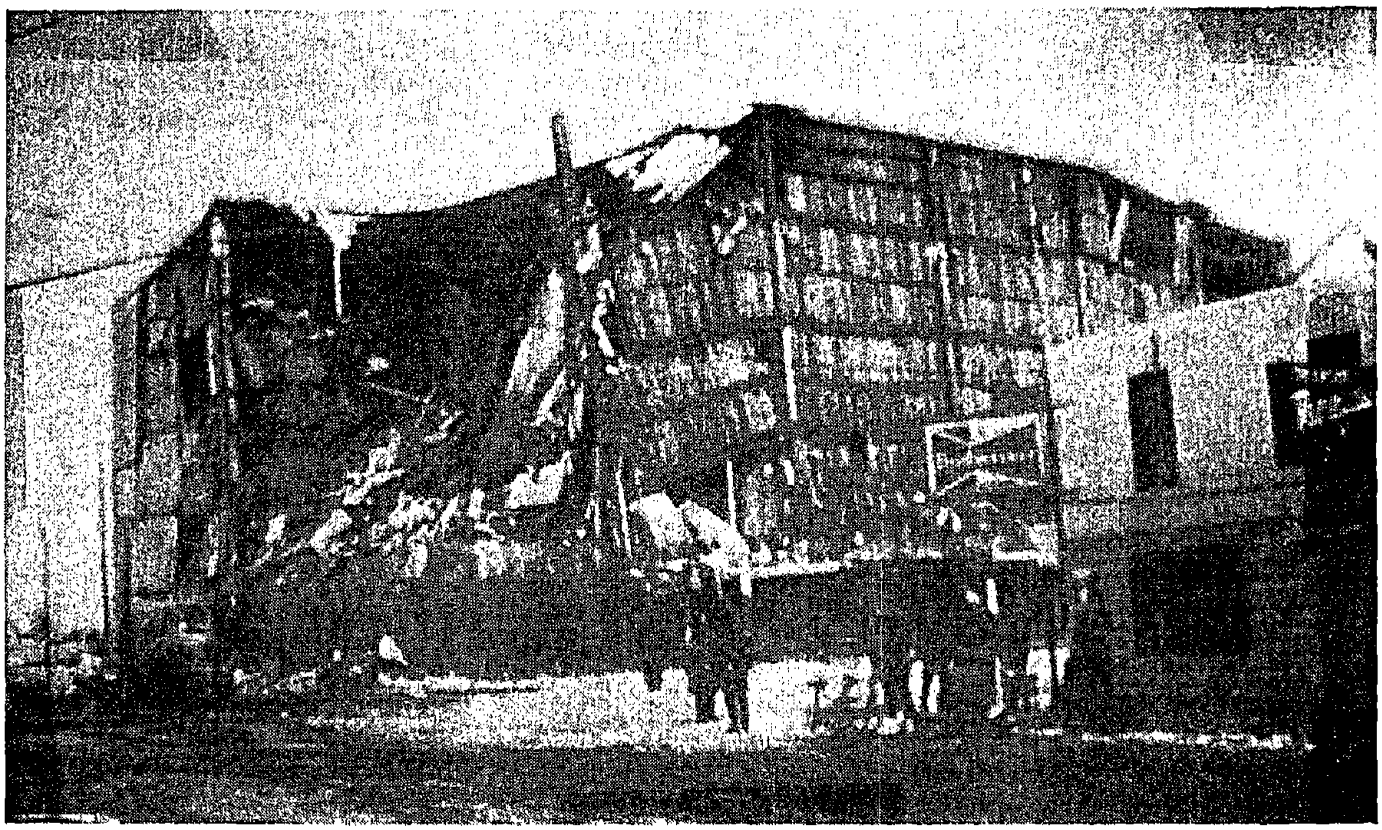


Figure B2-6 - Damage to Department Store Building, Anchorage (1964)

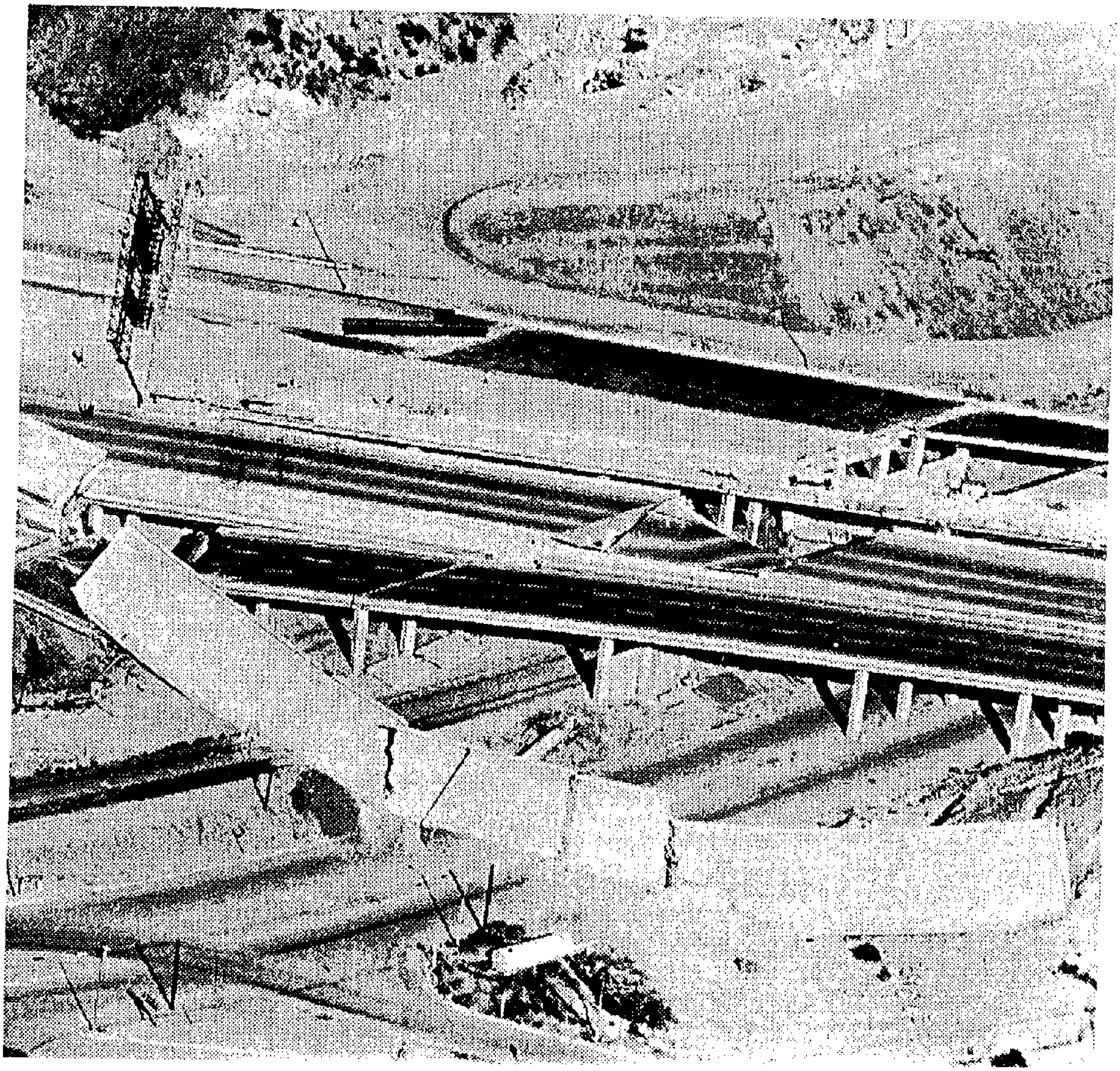


Figure B2-7 - Structural Damage, Foothill Freeway Overpass, San Fernando (1971)

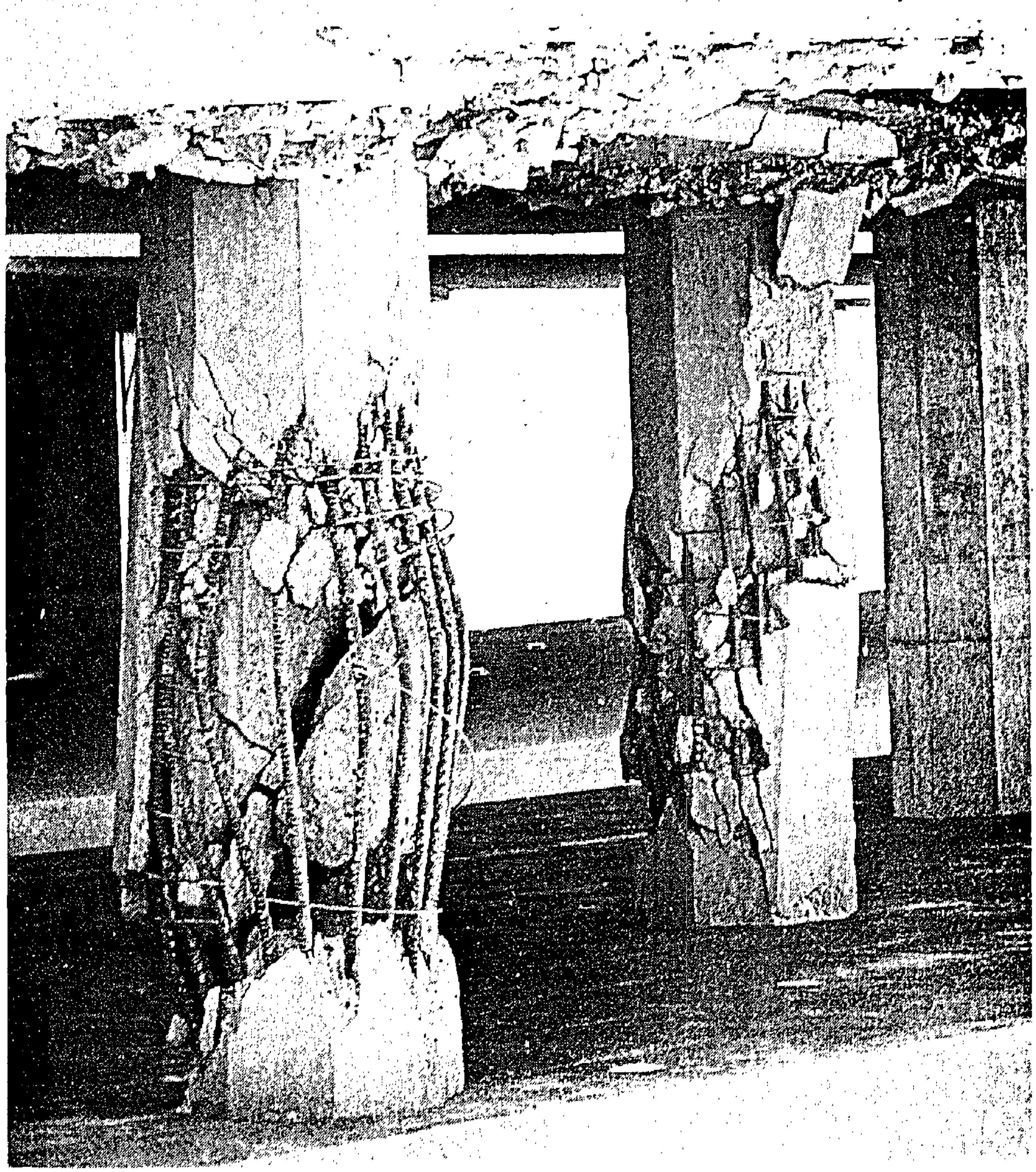


Figure B2-8 - Structural Damage, San Diego Freeway, San Fernando (1971)

Figure B2-9 - Structural Damage, Route 210/5 Interchange, San Fernando (1971)

Figure B2-10 - Building Damage, City of San Fernando, San Fernando (1971)

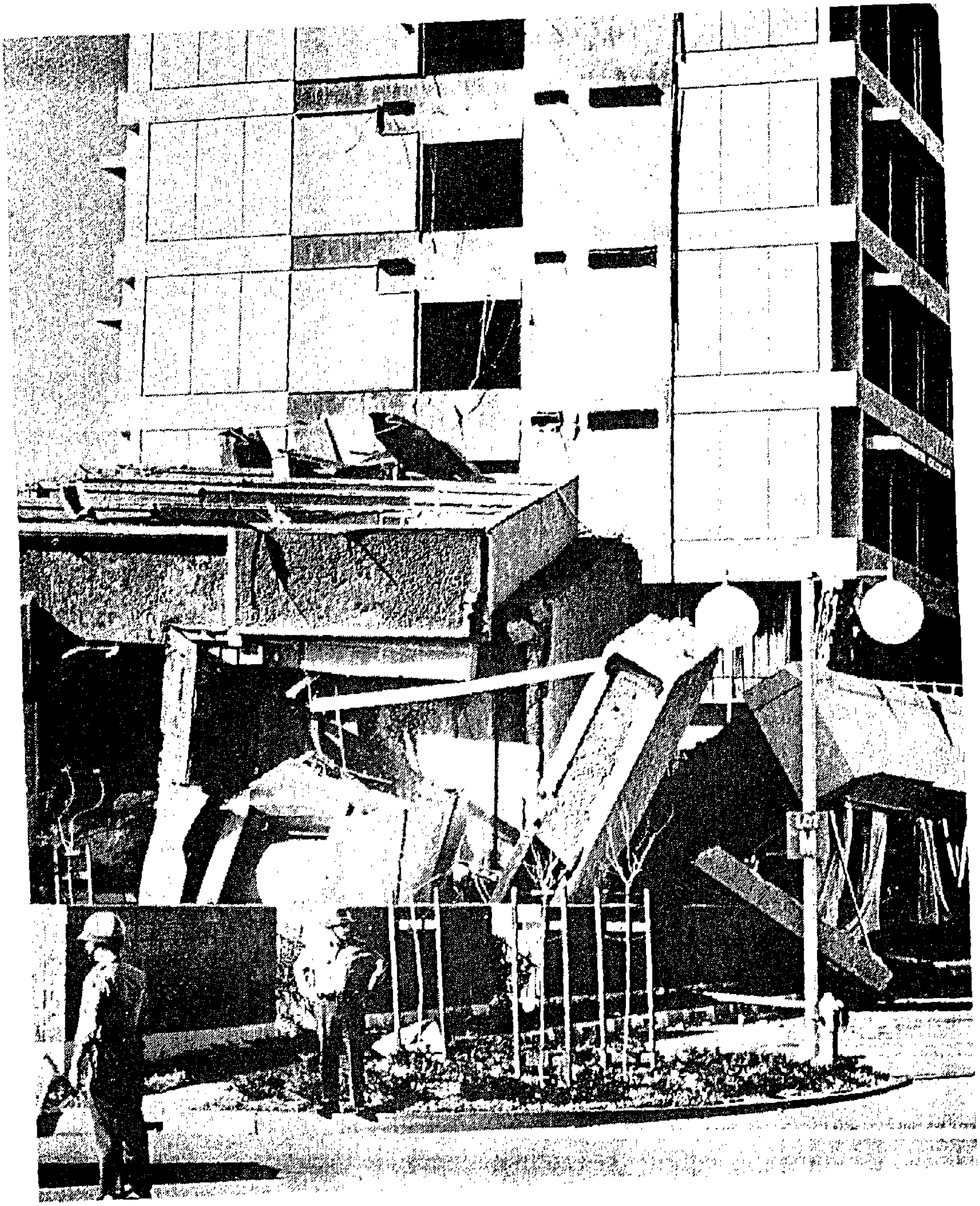


Figure B2-11 - Damage at Olive View Hospita, San Fernando (1971)

Part of the Van Norman reservoir collapsed and 80,000 residents in the Mission Hills area had to be evacuated. Residents were only allowed to return when the water reached a safe level after DPW workers drained the dam.

The San Fernando earthquake, although moderate in energy release and in amount of surface rupture, led to post-earthquake studies that provided significant new data and information concerning the effects of an earthquake on bridges, building structures, on the operations and services of public utilities, and transportation facilities. Human reactions and response to an earthquake, emergency in a metropolitan area, engineering problems related to soils and foundations, and man's knowledge and adjustments to the seismic, geologic and geodetic features of the physical environment were also studied. Following the San Fernando earthquake, CALTRANS initiated a seismic upgrading of bridges and other vulnerable structures. As a result of this quake, bridge and building codes were revised to provide more effective seismic-resistant design, and the seismic safety of dams in California was reexamined.

B 2.4 Northern Kentucky (1980)

Shortly before 3 P.M. on July 27, 1980, an earthquake struck near Sharpsburg, Kentucky, approximately 31 miles northeast of Lexington. The earthquake was followed by about 30 aftershocks centered in Sharpsburg, with several in the surrounding hills. Although this was only a moderate seismic event, several of its features were unusual. It was the largest earthquake in at least 200 years in that area, and the worst damage occurred to structures located in the town of Maysville, some 30 miles from the epicenter.

The area in which the earthquake occurred is part of the Central Stable Region, in which the frequency and relative size of earthquakes are considerably less than in other tectonic zones of the southern U.S.

The magnitude of the July 27 event was 5.3 and the focal depth was estimated at 7.5 miles. According to available information on earthquake history, no event of comparable size has been recorded in this area during the last 200 years. The earthquake was felt from Toronto, Canada, to the Gulf Coast. The maximum Modified Mercalli Intensity (see Appendix A, Table A2-1) based on observed damage was VII. Figure B2-12 shows the isoseismal map of the affected area.

Soil borings encountered artificial fill, ranging in thickness from less than 1 foot to more than 15 feet, overlaying Ohio River alluvium 30 to 40 feet thick. All materials were in unconsolidated to poorly consolidated state. Such material overlying a saturated sand is especially susceptible to earthquake-induced vibrations of low frequency which cause damage to structures.

Although there was a significant amount of damage to structures in Maysville, estimated at approximately $1,000,000, and lesser amounts in towns near the epicenter, there were no structural failures. 86 businesses and residences sustained major damage (loss exceeding $5,000), and 220 structures suffered minor damage. Many of the buildings damaged were built either in the late eighteen hundreds or early nineteen hundreds. Damage consisted primarily of cracked chimneys, cracked masonry walls and plastered ceilings, separated walls at the roof line of buildings, cracks and bulges in concrete slabs on grade, and broken windows. There did not appear to be damage to non-structural building systems. Except in Maysville, where some chimneys were broken off near the roof line, the majority of damage occurred at the top of the older chimneys from dislodged bricks. Cracks in masonry walls appeared to be typical stress concentration cracks, normally found at corners of wall openings. Modern construction survived the earthquake quite well.

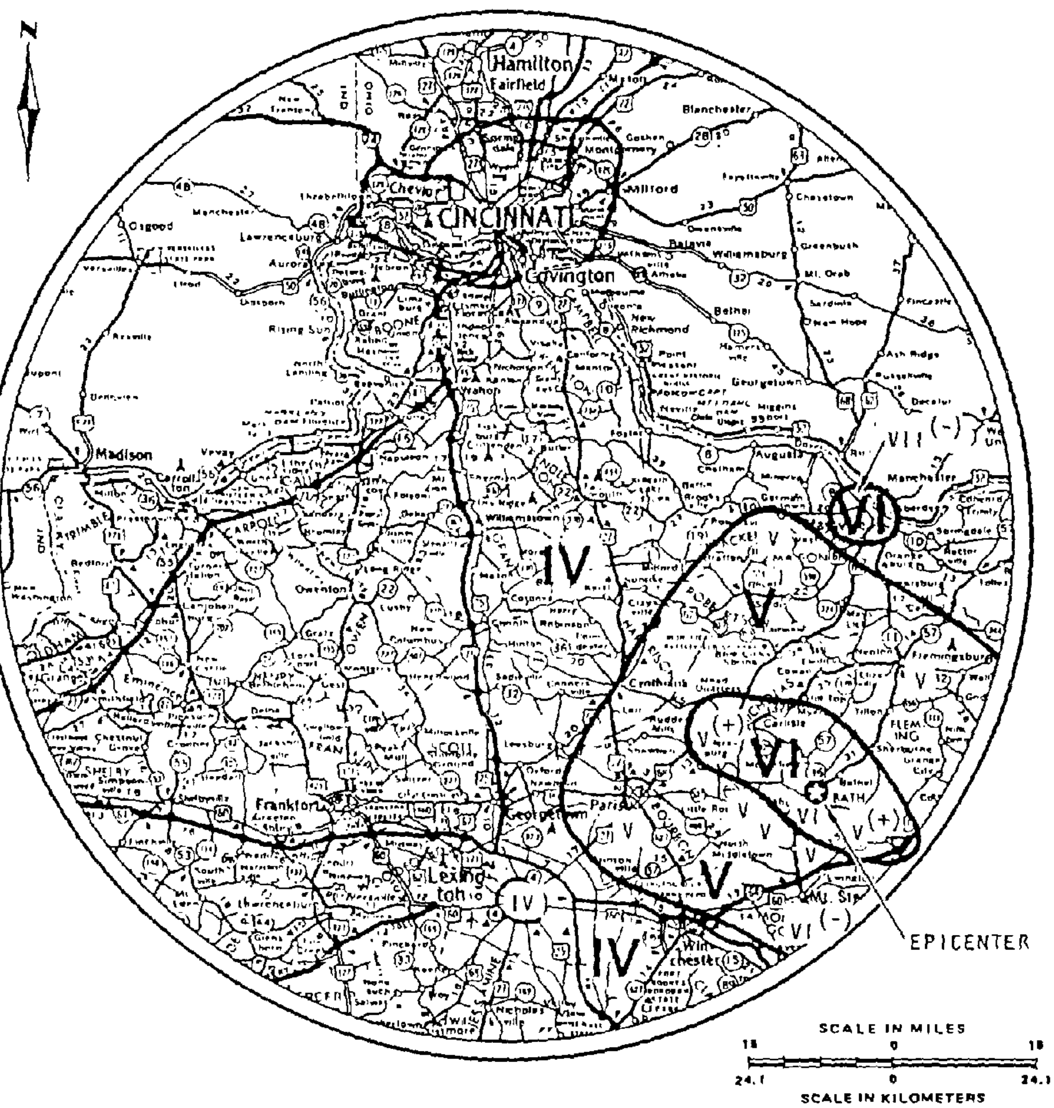


Figure B2-12 - Tentative Isoseismal Map, Northern Kentucky (1980)

The importance of this earthquake lies in the fact that it occurred in an area that, historically, had not experienced much earthquake activity. The area was considered one of low seismic risk. There are many areas that, today are considered to have low seismic risk. The Sharpsburg earthquake taught us, however, that there are no areas that are completely immune and that seismic design is important to all areas of the country, no matter how low the perceived risk. The extent and type of damage caused by the earthquake upon older buildings led to arguments favoring greater lateral force resistance in buildings throughout the nation.

Kentucky, like many other regions of the country, historically has disregarded consideration of lateral forces in the design and construction of their structures. This is explainable because these regions and the people personally have not before felt an earthquake. The construction in northern Kentucky seems to be representative of construction in many other parts of the eastern United States. An extensive amount of structures are old, maintenance varies from neglect to good, and lateral forces have not been considered very much. Attention has to be focused in these areas to the vulnerability of these buildings to the occasional damaging earthquakes and more responsible maintenance and construction practice should be introduced.

B 2.5 Whittier (1987)

At 7:42 A.M. on Thursday, October 1, 1987, a magnitude 5.9 earthquake occurred east of Los Angeles near the city of Whittier, California (see map on Figure B2-13). There were numerous aftershocks, the largest being a magnitude 5.5 in the early hours on October 4, which caused further damage to structures weakened by the main shock.

The earthquake occurred along a previously unrecognized fault at the northwestern end of the Puente Hills, and had a focal depth of 9 miles. No surface rupture due to fault movement was evident. Peak ground accelerations as high as 0.45g were recorded. Strong ground motion was recorded over a wide area. For example peak accelerations of 0.40g were observed in downtown Los Angeles 12 miles from the epicenter. The total duration of significant ground motion was about 15 seconds, but strong ground motion lasted only about 5 seconds.

Although it was a moderate earthquake, damage occurred over a surprisingly large portion of the Los Angeles basin. Damage occurred up to 18 miles from the epicenter, ranging from the Los Angeles International Airport to the city of Burbank.

Four persons were killed and numerous heart attack deaths were also attributed to the quake. Hospitals across the Los Angeles basin treated a total of 1,349 earthquake-related injuries on Oct. 1 and after the largest aftershock on Oct. 4. Estimates of property damage to public and private structures approached $350 million.

The earthquake effects in different communities varied, depending on local subsurface conditions. Areas underlain by more recent, loose, fine-grain soils suffered heavier damage, while areas founded on rock were less strongly affected. In general, modern, engineered buildings performed well in this earthquake, with few well-designed structures experiencing significant damage. The most vulnerable were unreinforced brick buildings, older wood frame, pre-cast concrete tilt-up, and older non-ductile (brittle) reinforced concrete structures. This illustrates once again that these types of buildings have the least strength to resist even moderate, short duration shaking.

A total of 24 bridges were found showing either evidence of movement or damage. Table B2-2 shows the bridges affected by the quake. The seismic upgrading of bridge structures, initiated by CALTRANS after the San Fernando earthquake, apparently was effective in preventing damage to the southern California freeway network. Failure of the I-5/I-605 separator suggests, however, that even with such programs, individual structures may fail and disrupt the regional transportation system.

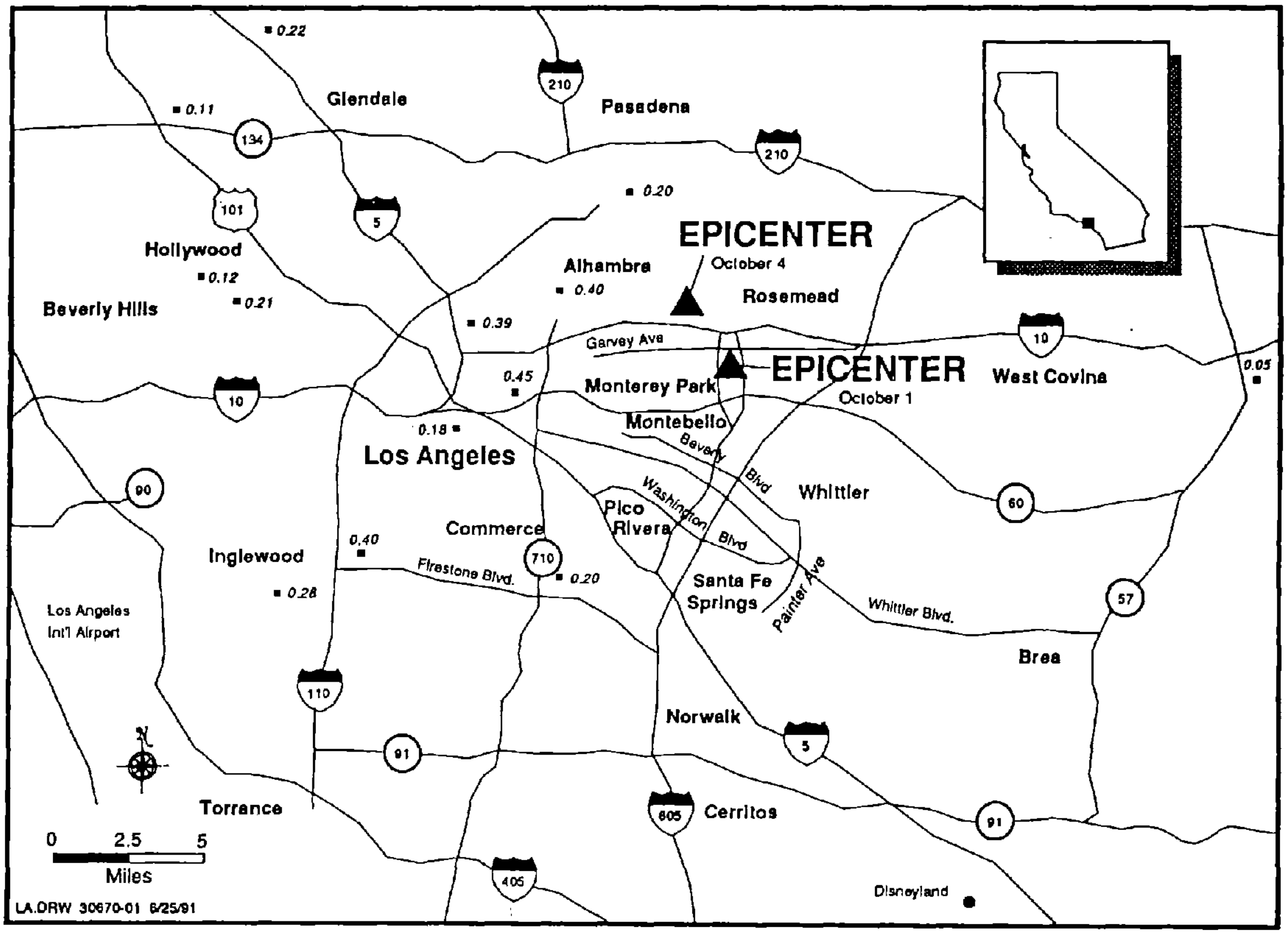


Figure B2-13 - Map of Epicentral Region, Whittier (1987)

Table B2-2

State Highway Bridges Damaged, Whittier (1987)

Bridge Name	Bridge No.	Route	Post Mile	Damage
Rte 605/5 Sep	53-1660	605	9.55	Moderate
E Conn OC	53-1657G	5	6.72	Minor
WB Busway OC	53-2540	10	21.10	Minor
College Busway OC	53-2505L	10	21.35	Minor
Almansor St OC	53-650	10	24.32	Minor
Puente Ave UC	53-0666	10	33.35	Minor
Rte 60/71 Sep	53-2081R/L	60	29.38	Minor
Juarez St UP	53-1007	72	6.81	Minor
Rio Hondo	53-0004	72	8.55	Minor
Burbank Blvd OC	53-1291	405	40.29	Minor
Hoxie OC	53-1652	605	8.23	Minor
Florence Ave OC	53-1656	605	9.35	Minor
West Conn OC	53-1083F	605	9.65	Minor
Dunlap Crossing Rd OC	53-1669	605	12.85	Minor
Walnut Cr	53-1343	605	19.85	Minor
Rte 710/10 Sep	53-1445R	710	26.47	Minor
East Conn OC	53-1447G	710	26.56	Minor
Santa Monica Via	53-1301	10	16.50	No
Cogswell RD POC	53-2085	60	10.50	No
Rte 91/605 Sep	53-1704	605	5.05	No
Florence OR OC	53-1659K	605	9.54	No
Bradwell OH	53-1664	605	11.39	No
Peck Rd OC	53-1477	605	16.65	No
Hobart Yard OH	53-0840	710	22.17	No

B 2.6 Loma Prieta (1989)

On October 17, 1989, at 5:04 P.M., a magnitude 7.1 earthquake (Richter scale) struck the San Francisco Bay area. The epicenter of the earthquake was 60 miles south of San Francisco in the Santa Cruz mountains. The devastating ground shaking produced by the earthquake lasted for approximately 10 seconds and was felt as far away as San Diego and western Nevada. This earthquake was the largest in Northern California since the 1906 great San Francisco earthquake, (whose magnitude was 8.3 on the Richter scale), and can be ranked as one of the more costly natural disasters ever to occur in California, if not the United States.

Seismic shaking, which affected a region of more than 400,000 square miles from Los Angeles northward to the Oregon border, was triggered by rupture of the crust along 25 miles of the southern Santa Cruz Mountain segment of the San Andreas fault.

The Loma Prieta earthquake and its aftershocks resulted in widespread damage to a variety of structures over an area of approximately 3,000 square miles. The California Governor's Office of Emergency Services estimated the damage as follows :

- 62 deaths (42 of which were caused by the collapse of the multiple-deck Cypress elevated highway structure in Oakland)
- 3,757 injuries.
- the San Francisco Bay Bridge was unusable for 1 month
- over $6 billion property damage
- number of homes damaged: 18,306
- approximately 12,000 people were at least temporarily displaced from their homes.
- 376 businesses were destroyed, 2,575 were damaged

The affects of the earthquake, in the form of disrupted transportation routes, for example, continues to test the patience of Bay Area commuters to this date. The most known damages from the earthquake occurred in the San Francisco area. Yet the mountain communities in the epicentral region and the nearby towns suffered the most losses to individuals and small businesses.

Damage resulted from many causes. Intense ground shaking, cracking from lateral extension, and sizable slides occurred in the Santa Cruz Mountains. No surface faulting was observed. Utilities, residences and large structures were not uniformly affected by the earthquake. Liquefaction (see Appendix A for definition) of uncontrolled bay fill and stream alluvium caused considerable losses.

Most buildings of seismically resistant construction survived the earthquake with little damage, typically limited to cosmetic damage to cladding and partitions, and disarray of contents. Older structures were hardest hit, with failure of many unreinforced masonry and some reinforced concrete buildings throughout the effected area (Figure B2-14). The expensive real estate development in San Francisco's Marina District was heavily damaged, caused by locally amplified shaking and by permanent deformation of the ground due to liquefaction of the sands and debris used to fill the former lagoon. Figure B2-15 shows a badly damaged apartment building in the Marina District where three people died.

Damage to the area's transportation infrastucture was extremely heavy. The earthquake caused the collapse of a 50 ft section on the upper deck of the Bay bridge, the collapse of a 3,970 ft section of the Cypress structure, major damage to several bridges, and minor damage to over 100 other bridges (ISSULF 1990). The known extent of the damage to the area's bridges is shown in Table B2-3. Figure B2-16 illustrates the collapsed section of the Cypress structure.

Figure B2-14 - Damage to Masonry Buildings in Oakland, Loma Prieta (1989)

Figure B2-15 - Damage to Apartment Building in San Francisco, Loma Prieta (1989)

Figure B2-16 - Collapsed Upper Deck of the I-880 Cypress Structure, Loma Prieta (1989)

Table B2-3

Extent of Damage to Bridges in Area Affected by Loma Prieta (1989)

Design Date (1)	Responsible Agency (2)	No Damage (3)	Minor Damage (4)	Major Damage (5)
Before 1972	City/County	2,000	N/A	N/A
	State	2,075	89	11
After 1972	City/County	470	N/A	N/A
	State	364	13	1

N/A = Not available

Based on the lessons learned from the 1971 San Fernando earthquake, the California Department of Transportation formulated a seismic retrofitting program for bridges and other key structures in the state. The reason for delay in retrofitting the Cypress structure and other vulnerable structures was primarily due to the lack of appropriated funds. Seismic retrofitting is generally expensive, and does not usually produce physical results that can be easily recognized and identified by the public. Thus, more visible projects often have a higher priority on the government's list than earthquake hazard-reduction activities.

The Loma Prieta earthquake has established a heightened awareness of seismic hazards in the Bay Area, and revealed the inherent vulnerability of facilities in seismic regions. It has also shown that ground failures can occur well outside the limits of mapped fault zones. Efforts to reduce seismic hazards in California since implementation of the Alquist-Priolo Act in 1972 can be credited for the relatively low losses from the earthquake given the dense Bay area population.

B 2.7 Damage Summaries

The preceeding paragraphs indicate two things. They indicate the type of damage that can occur in an earthquake, but more importantly, they indicate that earthquakes can occur anywhere.

Table B2-4 lists the effects of some recent larger earthquakes that have occurred in North America. These estimates give an idea of the scale of damage incurred in past earthquakes, and what can be expected from future earthquakes.

It is acknowledged that earthquakes will occur in the future. It is understood what type of damage can be expected from the various types of construction. It is well documented how to minimize earthquake damage through proper design and retrofit practices. What cannot be predicted is where future earthquakes will occur. An earthquake could strike anywhere. Facility managers would certainly be well advised to take agressive action to ensure that their facilities are designed and constructed appropriately to survive a seismic event.

Table B2-4

Damage Summary - Recent U.S. Earthquakes and Mexico City

Event	Date of Occurance	Magnitude on Richter Scale	Distance from Major City (Miles)	Duration (Seconds)	Maximum Ground Acceleration (Ft./Sec.^{2})	Deaths	Injuries	Temporary Displace-ments	Property Damage (Millions)
Loma Prieta, CA	10/17/89	7.1	60-San Francisco	15	1.00g	62	3,757	12,000	$350
Whittier, CA	10/1/87	5.9	250-Mexico City	15	0.45g	4	1,349	10,359	
Southeastern Illinois	6/10/87	5.6	125-St. Louis						
Northeastern Ohio	1/31/86	5.0	25-Cleveland		0.18g				
Mexico City	9/19/85	8.1	250-Mexico City	180	0.20g	8,000		40,000	
Coalinga, CA	5/2/83	6.7			0.59g				$500
Eureka, CA	11/8/80	7.1							
Northern Kentucky	7/27/80	5.3	31-Lexington	30	0.05g				$1.5
Imperial County, CA	10/15/79	6.6	16-Caluco	11.8	1.74g	0	0	0	$30
San Fernando, CA	2/9/71	6.6	80-Los Angeles	12	1.25g	58	2,500		
Prince William Sound, AL	3/27/64	8.4	75-Anchorage	Several Minutes	0.25g	125			

B 3.0 TRANSPORTATION FACILITY CHARACTERISTICS

The key characteristics and potential seismic exposure of the overall transportation system and its major functional components are described in this section. These characteristics are the essential data in performing a seismic vulnerability study, particularly at a nationwide level. Information presented in this section provides answers to the following questions:

1. What are transportation facilities?

2. Where are these facilities located in the nation?

3. Why and how are these facilities vulnerable to earthquakes?

4. What characteristics make facilities more or less vulnerable?

B 3.1 System Distributions

Transportation facilities, in a broad sense, can be divided into four major categories:

1. Highway Transportation Facilities

2. Railway Transportation Facilities

3. Air Transportation Facilities

4. Sea/Water Transportation Facilities

Mass Transportation, undoubtedly, is another legitimate category because of the numerous existing transit systems as well as some proposed facilities in major urban areas. The major system components and facilities, however, are in many ways similar to those contained in the four broad categories. For the purpose of this study, therefore, the mass transit facilities are not considered as an independent category. Instead, they are integrated into the vulnerability study for the broad categories.

The transportation system in the United States is an expansive and complex network. To evaluate the impact of earthquakes on the system on a national basis, it is necessary to understand the distribution of all major transportation systems in the United States, particularly their locations in relation to the seismic exposure potential (Appendix A). A brief description of the major systems based on FEMA inventory data is given below. Figures are included to illustrate the distribution of transportation facilities in each category across the country. Comparison of the figures with hazards maps (Appendix A) gives an indication of where seismic concern is most critical.

- State and Federal Highway System. There are some 490,000 miles of State and federal highway including over 42,500 miles of interstate highway. The geographic distribution of this system over the nation is shown in Figure B3-1. The majority of this system, which was built in the mid 1950s, now requires upgrading.

- Local Highway System. Specific details on the highway system at the local level were not readily available. A rough estimate, however, can be made by assuming that there is approximate 1 mile of local roadway for every 300 persons. This estimate is based on data from California DOT.

- Federal and State Highway Bridges. Figure B3-2 shows the locations of about 144,800 state and federal highway bridges throughout the United States. Many of these bridges are structurally deficient, with little or no seismic resistance.

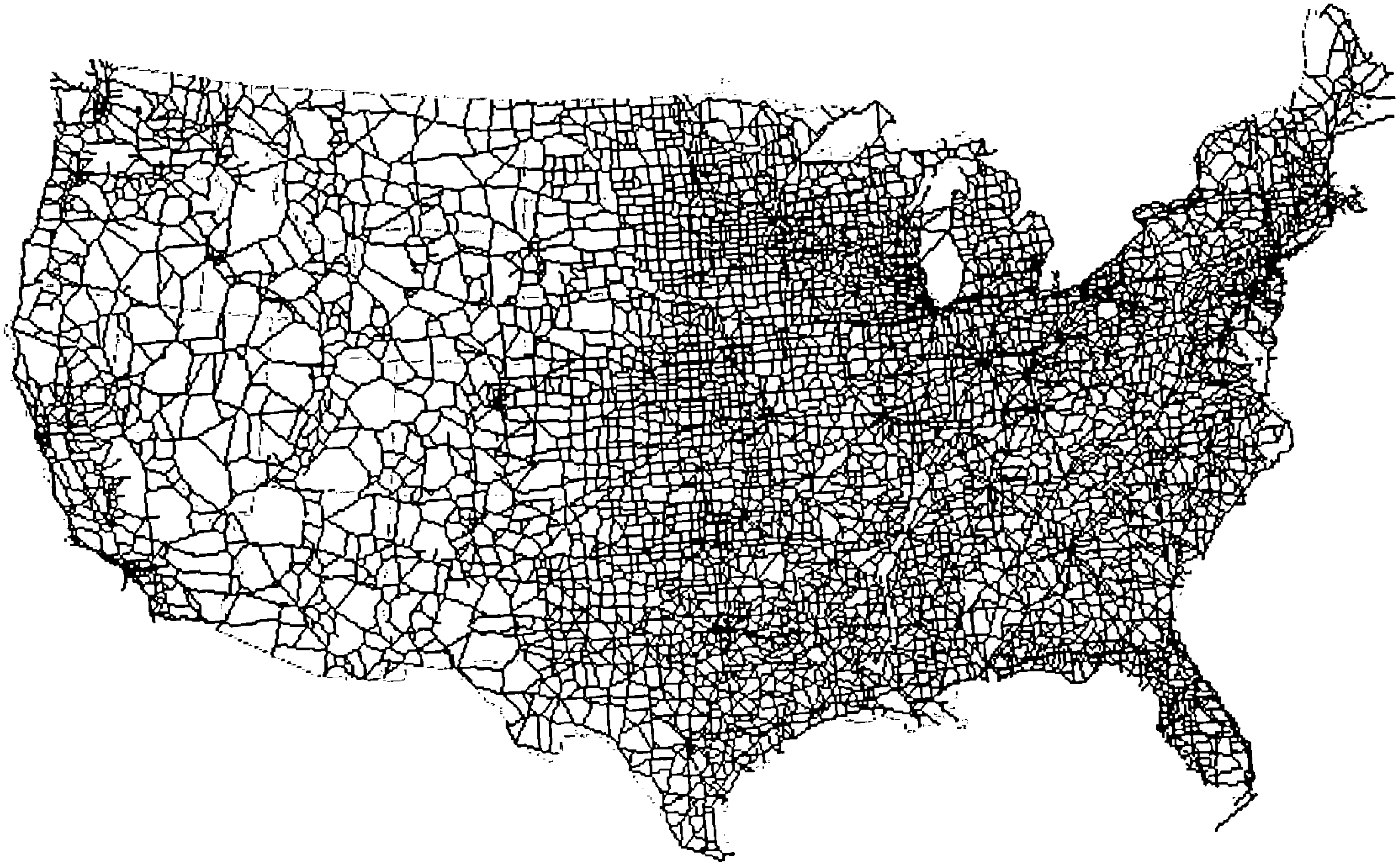


Figure B3-1 - State and Federal Highways

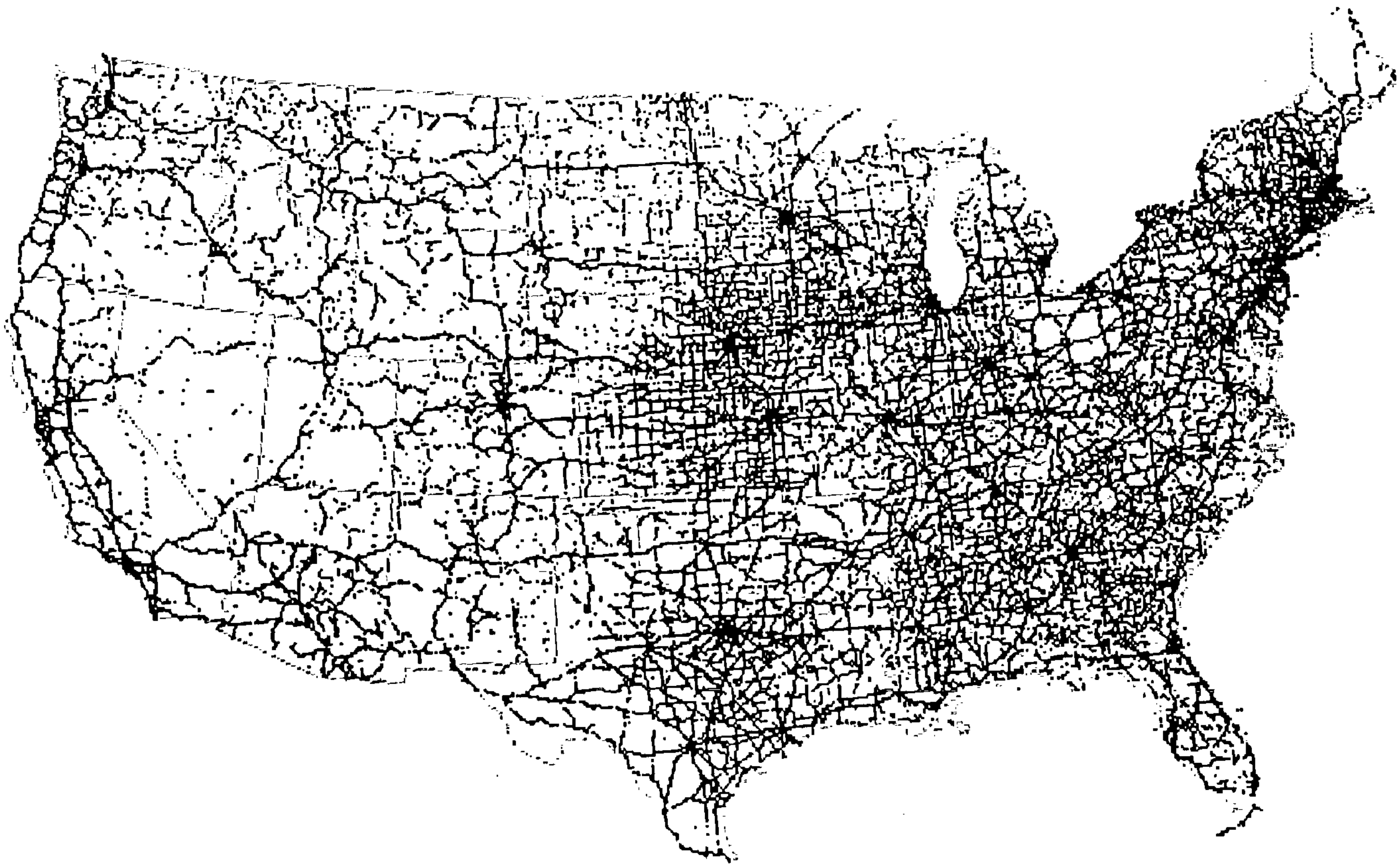


Figure B3-2 - State and Federal Highway Bridges

- Railway Systems. Passenger and freight transportation provided by rail service consists of about 180,000 miles of railways. Figure B3-3 gives picture of the system's network.

- Airports. There are over 17,000 civil and general aviation airports in the United States, through which more than 200,000 commercial and private aircraft travel. The locations of these airports are presented in Figure B3-4.

- Ports and Harbors. Inland waterways carrying tow boats and barges are estimated to be over 25,000 miles in length. Marine terminals providing services to ocean going vessels number over 2,400. Figure B3-5 presents location information for about 2,177 ports and harbors. This figure is based on data from FEMA.

B 3.2. Major Functional Components

Transportation facilities are made up of different types of structures, substructures and equipment. Several broad categories of facility types representing the major functional components of each transportation system are selected for this study. This classification, originally developed for the Applied Technology Council (ATC-13 and ATC-25), is based on functional characteristics rather than structural engineering characteristics. Classifying in this way does not break down facilities directly according to vulnerability characteristics, however, it helps assess the impact from social and economic standpoints, and it helps the transportation facility manager identify the category to which his facility belongs. It serves its purpose for initial screening of vulnerability. A more refined breakdown is necessary for the detailed study.

Transportation facilities can be broken down into the following major functional components:

1. Highway Transportation System
   - Major Highway Bridges
   - Conventional Highway Bridges
   - Highway Tunnels
   - Freeways/Conventional Highways
   - Local Roads
2. Railway Transportation System
   - Railway Bridges
   - Railway Tunnels
   - Railway Track and Roadbeds
   - Railway Terminal Stations
3. Air Transportation System
   - Airport Terminals (including Control Towers)
   - Airport Runways and Taxiways

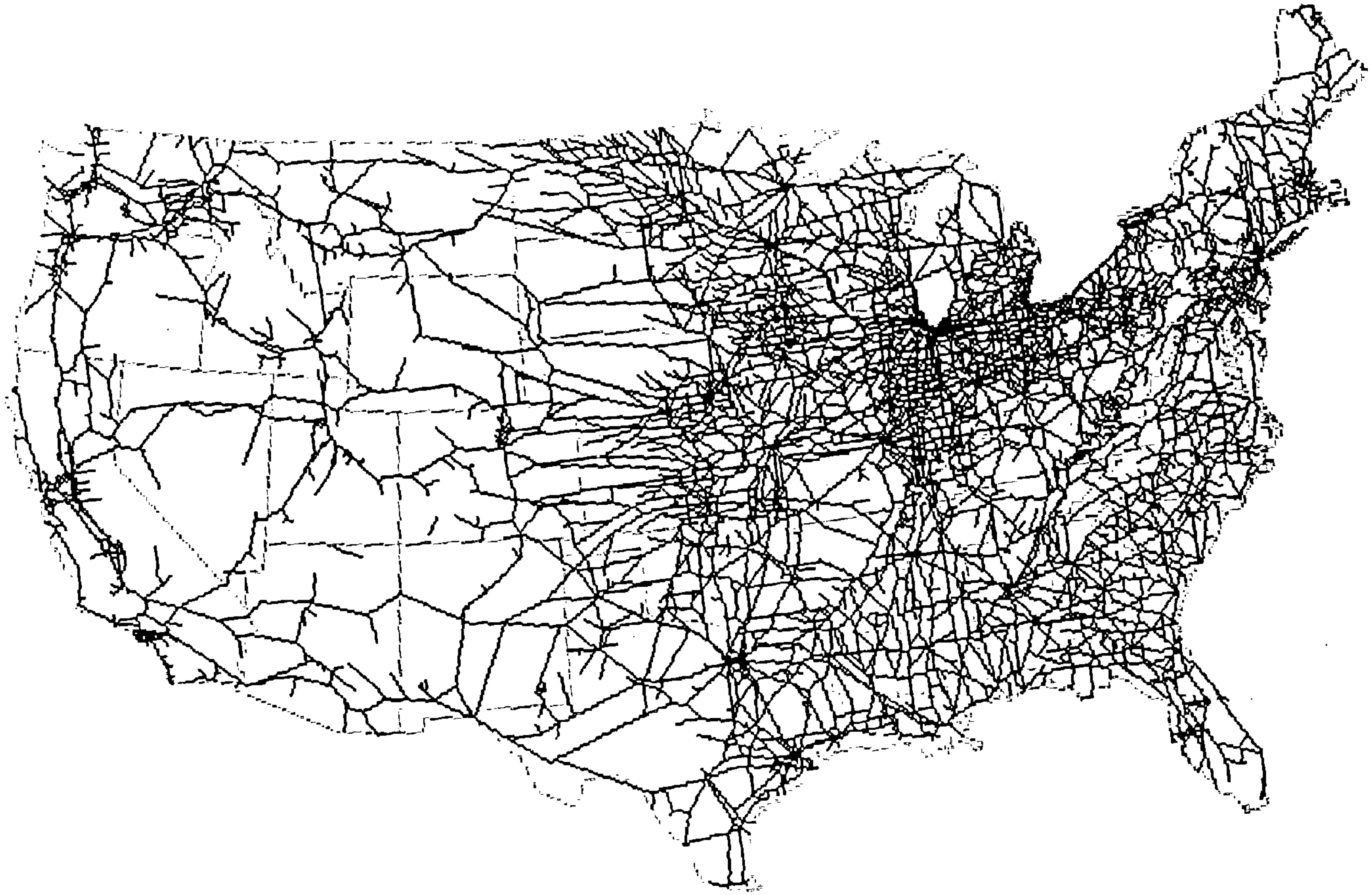


Figure B3-3 - Railroad System

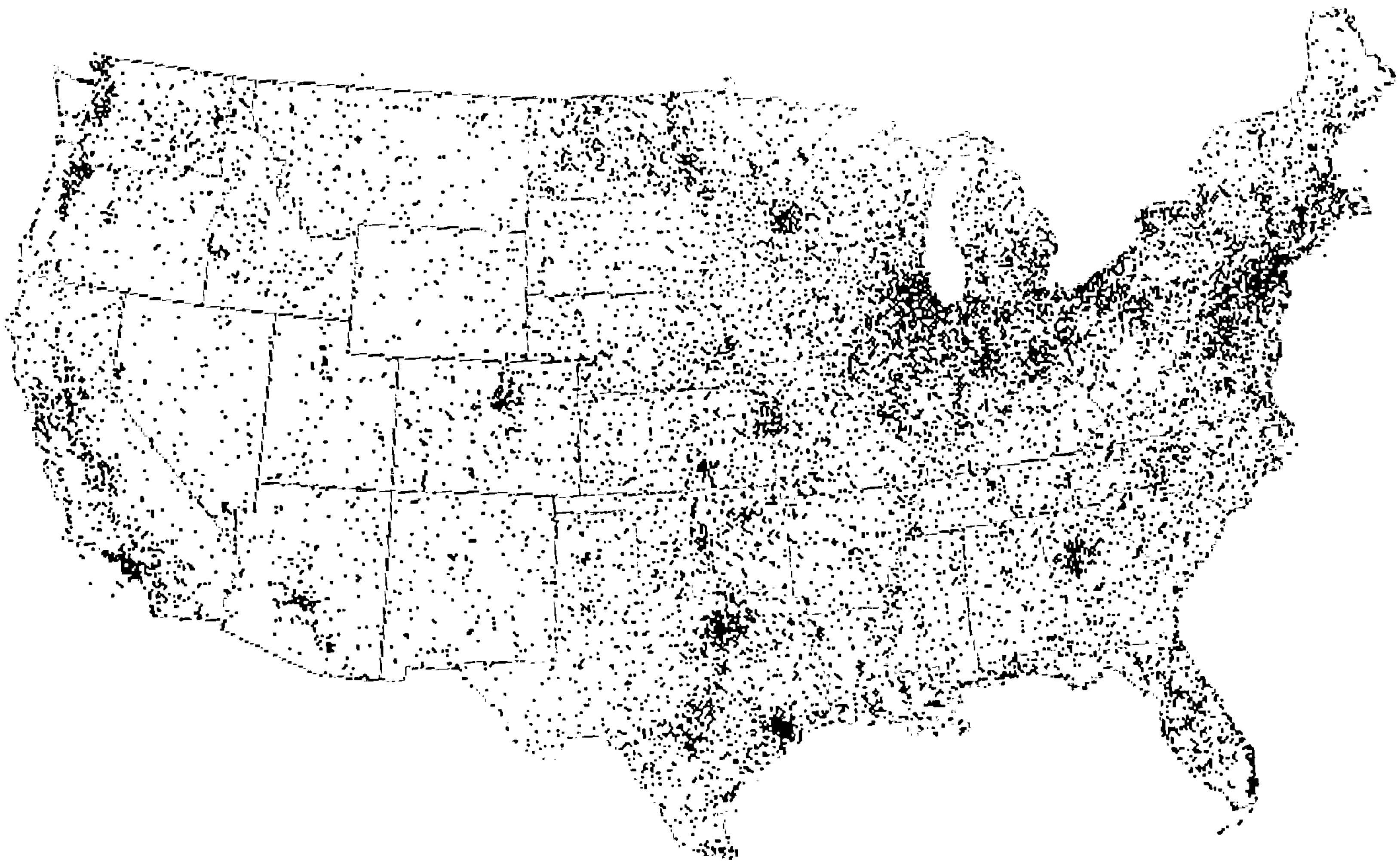


Figure B3-4 - Airports

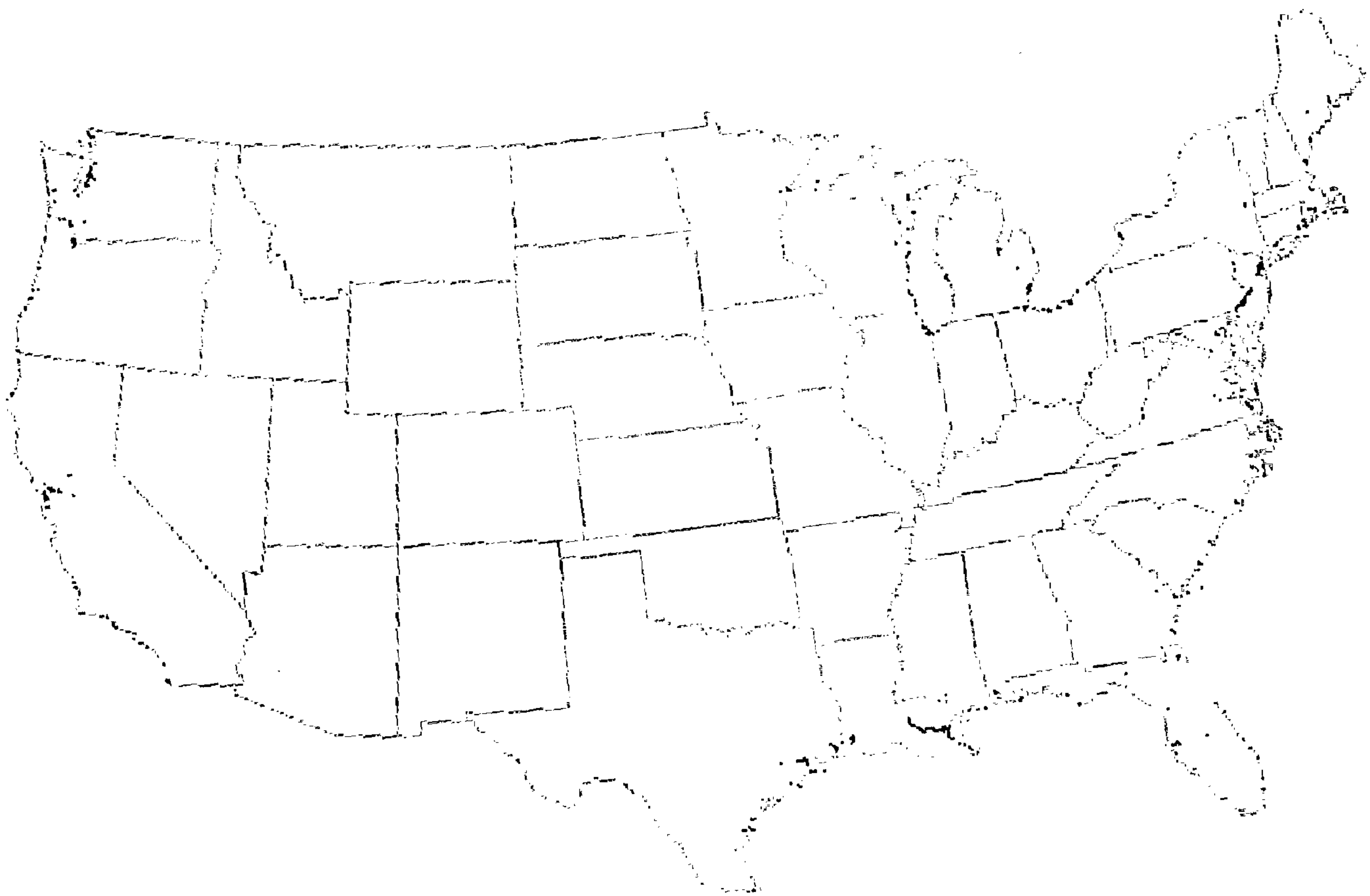


Figure B3-5 - Ports and Harbors

4. Sea/Water Transportation System

- Ports and Harbors
- Cargo Handling Equipment

B 3.3. <u>Earthquake Vulnerability Characteristics</u>

Factors that may affect a structure's or its elements' vulnerability potential to earthquake hazards include the following:

- Construction material
- Structural geometry and configuration
- Load-resisting system (framing system)
- Age
- Construction quality
- Design standard, or building code to which the structure was built
- Soil foundation material (ground condition)

Past experience indicates that design, construction quality and structural detailing play a major role in seismic performance of structures. The factors presented above should all be considered in a vulnerability assessment. Some considerations in identifying vulnerability characteristics for major transportation components are outlined below.

* <u>Highway Bridges</u>: Conventional highway bridges are defined in this study as those with spans less than 500 feet and with regular configurations. Simple and multiple spans are the most common type of construction although continuous spans are also often seen. Typical earthquake effects on these types of bridges are : 1) bridge deck collapse due to insufficient support length, 2) soil/foundation failure of bridge piers and abutments due to poor soil conditions (e.g., liquefaction), and 3) pier, column and beam failure due to insufficient steel reinforcement and inadequate detailing to provide required ductility. Skewed bridges in particular have performed poorly in past earthquakes. Soil and structure amplification effects have also been demonstrated to increase seismic loads on bridge structures and therefore increase their vulnerability.

  Major highway bridges are those with individual spans over 500 feet, and commonly include suspension, cable-stayed, or truss bridges. Long span reinforced concrete arch or prestressed concrete segmental bridges are also in this category. Although seismic loading was generally not considered until the 1970s, these major bridges have historically fared better than conventional bridges, possibly due to the high live loads (wind loads) used in the design. In most cases, damage was limited to ground and structural failures at bridge approaches. Consideration should also be given to potential ground failures (e.g., liquefaction and landslides).

* <u>Railway and Rail Transit Bridges</u>: Most railroad bridges currently in use in the United States were built before most highway bridges were built, with little or no consideration given to seismic design. In general, railway bridges are simple or

multiple span structures rather than continuous. Railway bridge configurations range from small wooden trestles supported on wooden piling to large steel truss bridges on pile supported concrete piers and abutments.

Damage to railroad bridges in the United States has been limited in past earthquakes. In general, they have performed much better in earthquakes than highway bridges. The major damage to railway bridges, including the observed performance during the 1964 Alaska earthquake, was primarily caused by instability of foundation soils. Compared to highway bridges, railway bridges display several favorable design and construction characteristics that may have resulted in their superior performance in the past. According to Pauschke (1990), these characteristics are: 1) most railway bridges were designed as simple span with large seat widths which prevent spans from sliding off, 2) the continuous rails crossing the bridge deck provide longitudinal restraint against superstructure movement, 3) the higher design live load (compared to that for highway bridges), to account for the braking and centrifugal forces may provide large reserve capacity for seismically induced lateral load, and 4) the absence of the highway slab results in a lighter superstructure and hence less dynamic inertia effects.

- Tunnels: Tunnels, for highway, railway, or rail transit systems, may be constructed in rock or soil using various drilling, blasting and cut-and-cover methods. River/channel crossing tunnels may be placed along prepared beds connecting with land tunnels or portal sections. Tunnels may be unlined or lined with brick, reinforced and unreinforced concrete, and steel. Lining using timbers and wood lagging may also be found in some existing tunnels.

  Due to their confined nature, tunnels are in general less vulnerable to earthquakes than above-ground structures. Inertia forces and amplification effects are not as critical to lined tunnel structures as they are to above-ground structures. Rather, the seismic response of tunnels is vulnerable to large ground movement due to fault rupture, sliding soil/rock mass, liquefaction and traveling seismic waves. Landslides at tunnel portals and failing rock wedges in unlined tunnels could also cause major damage and disruption to these facilities. Stress concentrations at soil/rock interfaces, intersections, bends and connections with shafts and ventilation structures also warrant special consideration.

  Figure B3-6 presents the results of observed earthquake performance on 71 underground openings (Dowding and Rozen, 1978). In this figure, "Minor Damage" represents new cracking and minor rockfalls, while "Damage" is an indication of severe cracking, major rockfalls and closure.

- Highway/Local Roads, Rail Transit and Railroads: Highway/local roads include roadways, embankments (including retaining walls where applicable), signs and lights. Roadway facilities may be damaged or disrupted by failure of adjacent embankments and landslides. Instability or structure failures of retaining structures may be caused by dynamic incremental earth pressure, sometimes in combination with excessive pore water pressure build-up or liquefaction behind or underneath the retaining walls. Roadway surface damage may take the form of settlement, cracking or even buckling or roadway slabs.

  Rail roadbed and track consists of ties, rail, ballast, embankment and switches for railroads and rail transit. The most common damage consists of settlement of slumping embankments. Similar to vulnerability characteristics of highway roadways, landslides, retaining wall failures and liquefaction all contribute to the damage potential of rail roadbed.

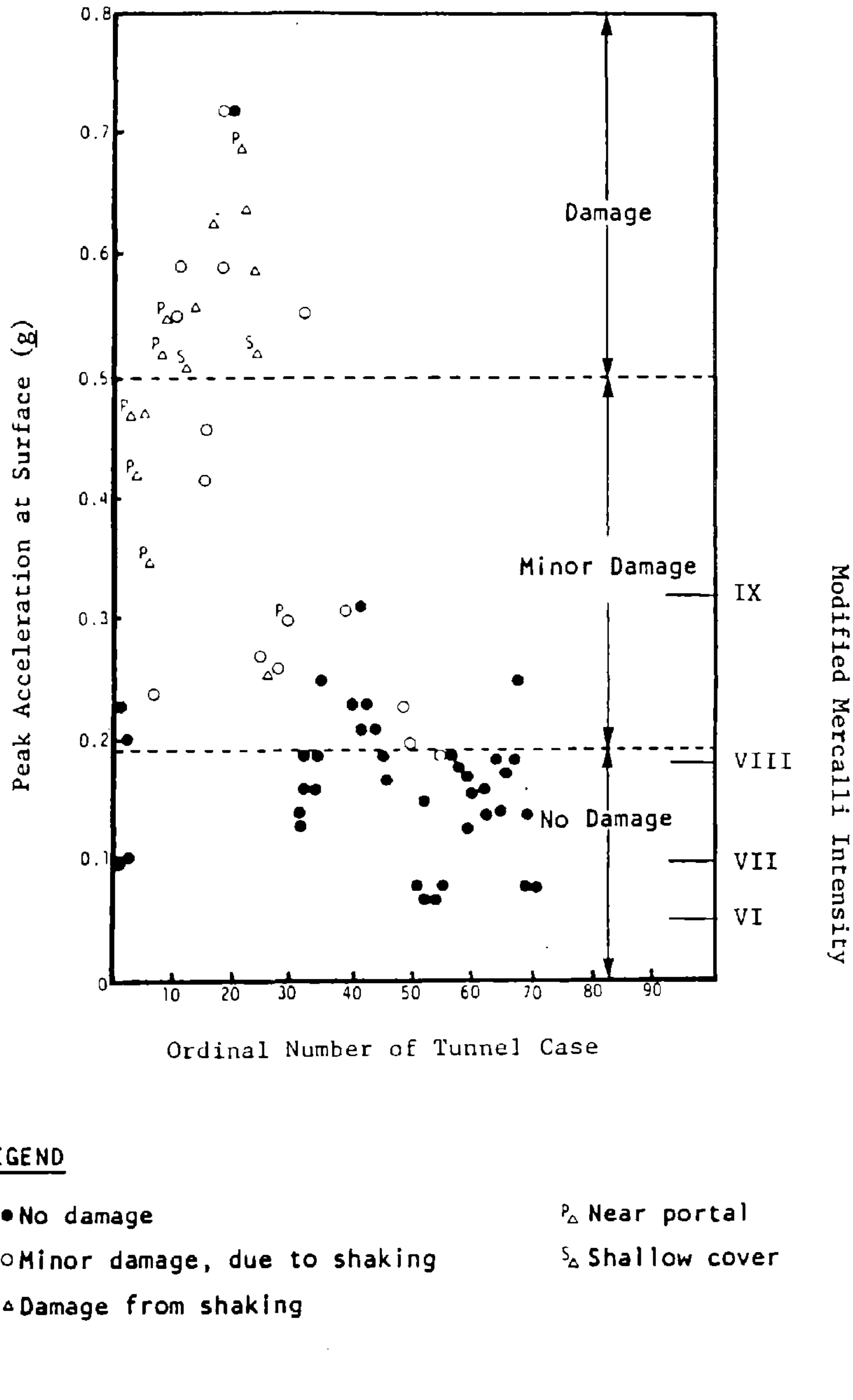


Figure B3-6 -Calculated and Peak Surface Accelerations and Associated Damage Observations for Earthquakes. (Adapted from Dowding and Rozen, 1978)

* Railway and Rail Transit Terminal Stations: Terminal stations may be of any type of structure configuration and construction from steel frame to unreinforced masonry bearing walls. Since the terminal stations are essentially conventional buildings, damage observed in buildings is also typical of the terminal stations.

  Buildings are more vulnerable to earthquakes if they have plan (horizontal), or vertical *irregularities*. Examples of plan irregularities include non-rectangular plan layouts such as L, V, or C shapes, large wall openings and non-parallel vertical bracing systems. Examples of vertical irregularities include stories with strengths, stiffnesses or weights that differ radically from adjacent stories, or where the overall horizontal dimension of the lateral force resisting system in one or more stories varies significantly from the rest.

  The type of seismic resisting structural system has a big effect on seismic vulnerability. Unreinforced masonry and concrete structures have been notorious poor performers in earthquakes. Poorly detailed reinforced concrete structures have also performed poorly. Most commonly problems have been caused by inadequate embedment of reinforcing bars at the ends of beams and columns, and inadequate lateral confinement reinforcing in columns and beams, especially at their intersection. Reinforced concrete structures, if detailed properly, have performed well. The best performance has been observed in structural steel framing systems, especially unbraced moment resisting space frames which are very ductile.

  Various types of mechanical/electrical equipment are housed by these stations. This equipment, including machinery, piping, ductwork, etc. is vulnerable to damage if not anchored properly to the structure.

* Airport Runways and Taxiways: Many airports are located adjacent to bodies of water, often on landfill along waterfront areas. Liquefaction in landfill has caused major damage to runways in the past. Most recently during the Loma Prieta earthquake, several airports in the San Francisco Bay area suffered severe runway damage due to liquefaction. Oakland International Airport lost 3000 ft of the 10,000 ft long main runway. Cracking, settlement, heaving and lateral spreading are the dominating surface manifestations of liquefaction. It took about four weeks to restore the full capacity of the runway. Alameda Naval Air Station also experienced serious runway damage as a result of liquefaction. Numerous sand boils were observed. The most widespread airport damage occurred in the 1964 Alaska earthquake where twelve airports suffered major damage. Many runways experienced significant subsidence and cracking. In some cases parts of the runway sank below sea level and were inundated at high tide.

* Airport Terminals: Terminal buildings, control towers, hangars and other miscellaneous structures are considered in this category. Control towers are typically reinforced concrete shear wall buildings and hangars are either steel or wood long-span structures. Fuel tanks and underground pipelines also serve as a critical part of the facility.

  The most common damage in control towers is broken windows. This type of failure was observed during the 1964 Alaska earthquake, the 1971 San Fernando earthquake, the 1987 Whittier Narrows earthquake and the 1989 Loma Prieta earthquake. The loss of control tower windows severely disrupts tower operations. Inadequate anchorage of critical equipment represents another problem in the airport terminal facility. Although fuel storage tank failure has not been observed frequently at airports, vulnerability potential associated with this type of facility should not be overlooked. Typical modes of failures consist of wall buckling, settlement, ruptured piping, or even fires caused by collapse.

Conventional type buildings exhibit damage similar to that described above under Railroad and Rail Transit Terminal Stations.

* Port/Cargo Handling Equipment: The most critical components of port/harbor facilities are waterfront structures and cargo handling equipment. The waterfront structures include pile supported piers, sheet-pile bulkheads, dikes and gravity type retaining structures such as quay walls. Other important elements consist of administration buildings, warehouses, tanks, pipelines, rail system, conveyors. etc.

  It has been repeatedly demonstrated in past earthquakes that seismic performance of the ground dominates the overall performance of ports/harbors. Waterfront facilities tend to be underlain by loose, saturated soils that are susceptible to pore-pressure build-up and liquefaction. Excess pore-pressure together with dynamic earth thrust often lead to instability of retaining structures and bulkheads. Past experience also suggests the vulnerability of block-type gravity walls due to their tendency to slide between layers of blocks. Liquefaction in the backland area tends to induce subsidence in the area where most of the infrastructures are located. Cargo handling equipment sitting directly on the fill is particularly vulnerable to the earthquake induced ground instability. Tie-back bulkhead walls with anchors embedded in liquefiable soil may fail by anchor pull-out. Piers supported partially by batter piles were demonstrated to be prone to damage at pile cap - deck connections in the recent Loma Prieta earthquake (Port of Oakland). Seismic stability of dikes/embankments depends on the soil performance behind as well as beneath the dikes. Permanent dike/embankment deformations should be limited to the extent not to damage the piled wharf structures on the waterfront. Any unremoved loose soil, such as harbor sediment, underlying the dike is an indication of problem during earthquake.

B 4.0 EARTHQUAKE VULNERABILITY ASSESSMENT

B 4.1 Preliminary Screening for Seismic Vulnerability

Section B.3.3 described, in a qualitative manner, the vulnerability characteristics of the major functional components of transportation facilities. In order to provide a quantitative assessment it is necessary to establish a procedure to estimate the damage and the consequent losses for a given facility or structure exposed to a certain seismic environment.

Unfortunately, while there are a great deal of earthquake performance data, actual quantified earthquake damage and loss data are limited. One way to develop this required data is to draw on the experience and judgment of earthquake engineers. This approach has been used in a study funded by FEMA to produce an earthquake damage evaluation data base for California (ATC-13). This valuable data base, and therefore the approach, has since been used by others to conduct seismic vulnerability and loss study at nationwide level (ATC-25) and regional level (Massachusetts Civil Defense Agency, 1990).

It is recommended that this methodology be followed by transportation facilities managers to perform an initial screening of their facilities to determine their vulnerability to seismic events. This methodology is as follows:

1. **Quantify Seismic Hazard (MMI):**

   The purpose of this task is to identify the earthquake shaking characterization that is most appropriate for estimating earthquake damage and losses. The Modified Mercalli Intensity (MMI) scale has been selected to express the damage - ground motion relationship for facility damage evaluation. The great preponderance of available damage-motion data in the form of MMI prompted this selection.

   The scale (Appendix A, Table A2-1) consists of 12 categories of ground motion intensity from I (not felt) to XII (total damage). Several relationships between the MMI and the peak ground acceleration have been proposed in the literature as presented in Figure B4-1. These relationships allow the assessment of damage/loss through the use of peak ground acceleration, available from seismic hazard maps as described in Appendix A.

2. **Identify Facility Functional Component Classification:**

   Identify the functional component classification from the list presented in Section B3.2.

3. **Identify Non-Standard or Special Construction**

   Identify any deviations from the norm for the structure under consideration. The damage probabilities developed in the ATC studies apply to facilities having standard construction in California. Standard construction includes all facilities except those designated as special or nonstandard. Special construction consists of: 1) California elementary and secondary school buildings, 2) post-1972 California hospitals, 3) railway bridges and 4) any facility determined to have special earthquake damage control features. Nonstandard construction includes those structures that are more susceptible to earthquake damage than standard construction. For example, older facilities designed prior to modern seismic design requirements can be assumed to be nonstandard.

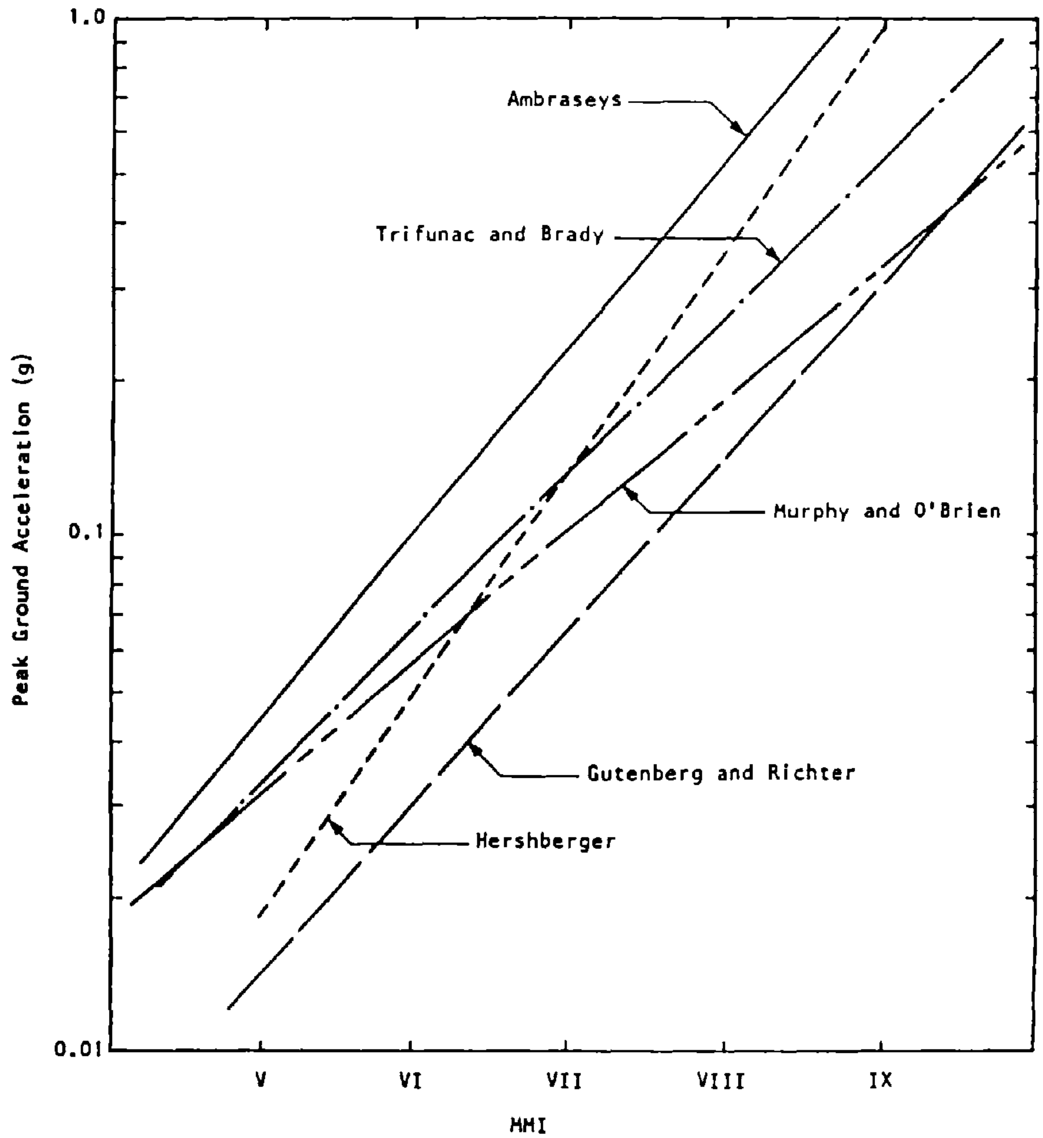


Figure B4-1 - Acceleration Versus Modified Mercalli Intensity Relationships

4. **Identify Regional Classification**

To apply the damage probability data to regions outside of California, one must account for the variation in seismic design practice in different regions. ATC-25 suggests an approach in which the United States is divided into five regions based on the history of seismic design practice. The division is based on a NEHRP seismic map (ATC, 1978) as presented in Figure B4-2. The five regions are listed below.

Region I:	California Map Area 7
Region II:	California Map Area 3-6
Region III:	Non-California Map Area 7
Region IV:	Puget Sound Map Area 5
Region V:	All Other Areas

Region I (California Map Area 7) is considered the only region with a significant history of seismic design for "great" earthquakes. Therefore the damage probability data developed in the ATC studies can be readily applied to this region.

Regions II, III and IV are considered regions with a significant history of seismic design for "major" earthquakes. Region V is assumed to have no significant history of seismic design for major earthquakes. For facilities in Regions II to V, the damage probability data should be modified to account for the increased facility damage potential in these regions. This is done with the use of separate probability curves for the different types of regions as described in step 5, below.

5. **Estimate Vulnerability (Potential Damage and Loss):**

Determine potential damage and loss as a percent of replacement value, using curves developed in the ATC-13 and ATC-25 studies (Figures B4-3 through B4-14). This percentage is read from the curves, given facility type, MMI intensity level and regional classification.

The use of these curves is explained by example. Figure B4-3 shows the results of vulnerability assessment for major highway bridges (bridges with spans greater than 500 feet or special bridges). The vulnerability is expressed in terms of potential damage and loss as a percent of replacement value of the bridges. Three motion-damage curves are shown in the figure, with each representing a different seismic design and construction standard in that region. Clearly, the vulnerability is a function of the locations of the structure as well as the ground shaking intensity. Given the same shaking intensity, bridges designed and constructed with better seismic practice such as the ones in the California Area 7, are expected to perform better than similar bridges built in other areas.

For a specific structure, the vulnerability curves should not be used strictly according to the location of the structure, however. Any special or non-standard construction can also be accounted for. For example, if a structure, say recently built, has been designed and constructed according to the most modern seismic requirements, the damage potential of this structure can be estimated by using the California Area 7 curve even if the structure is not in California. The three curves can generally be classified as indicating high, moderate and low seismic capacity, for the lowest, middle and highest curve, respectively.

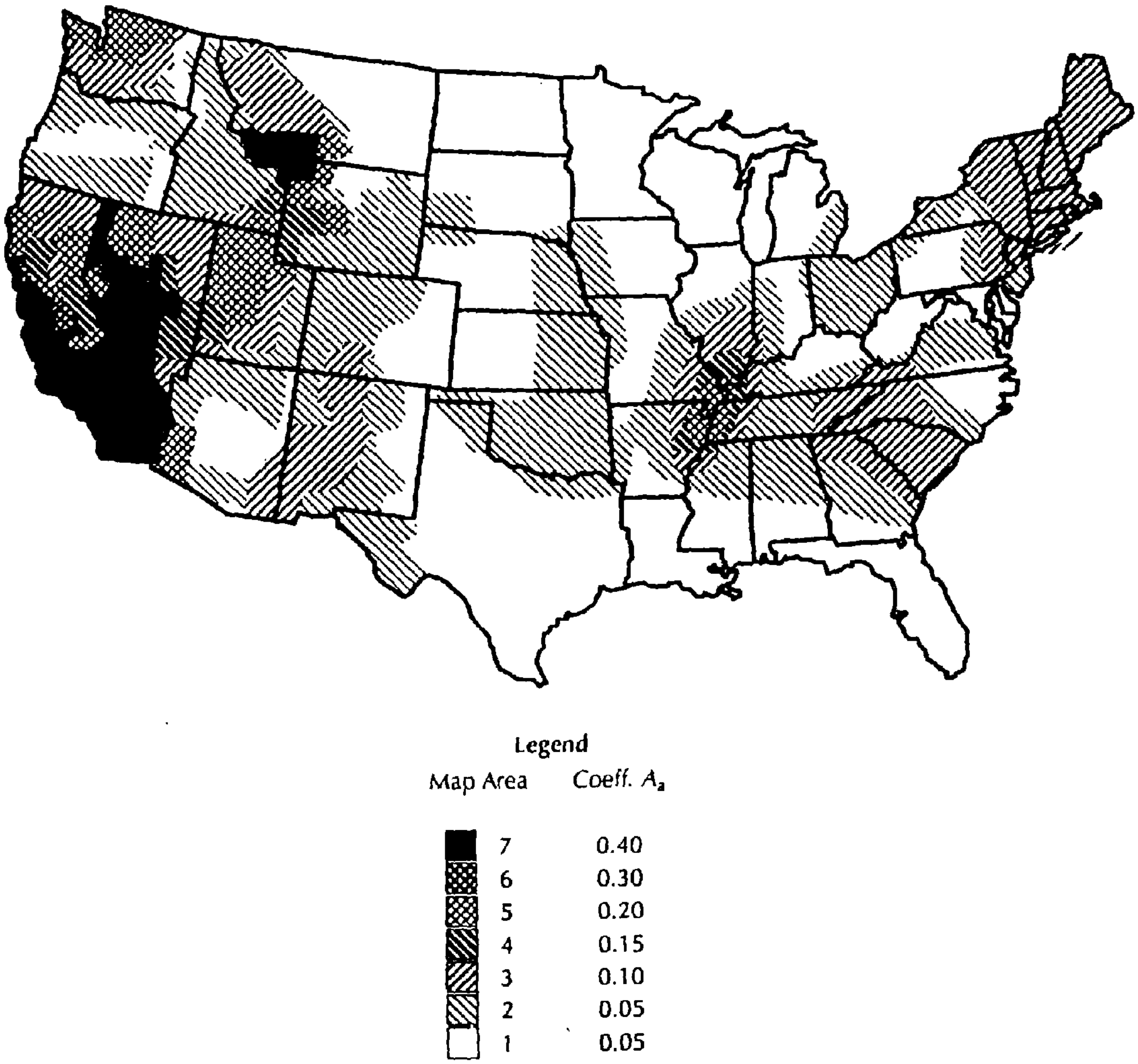


Figure B4-2 - NEHRP Seismic Map Areas (ATC, 1978; BSSC, 1988)

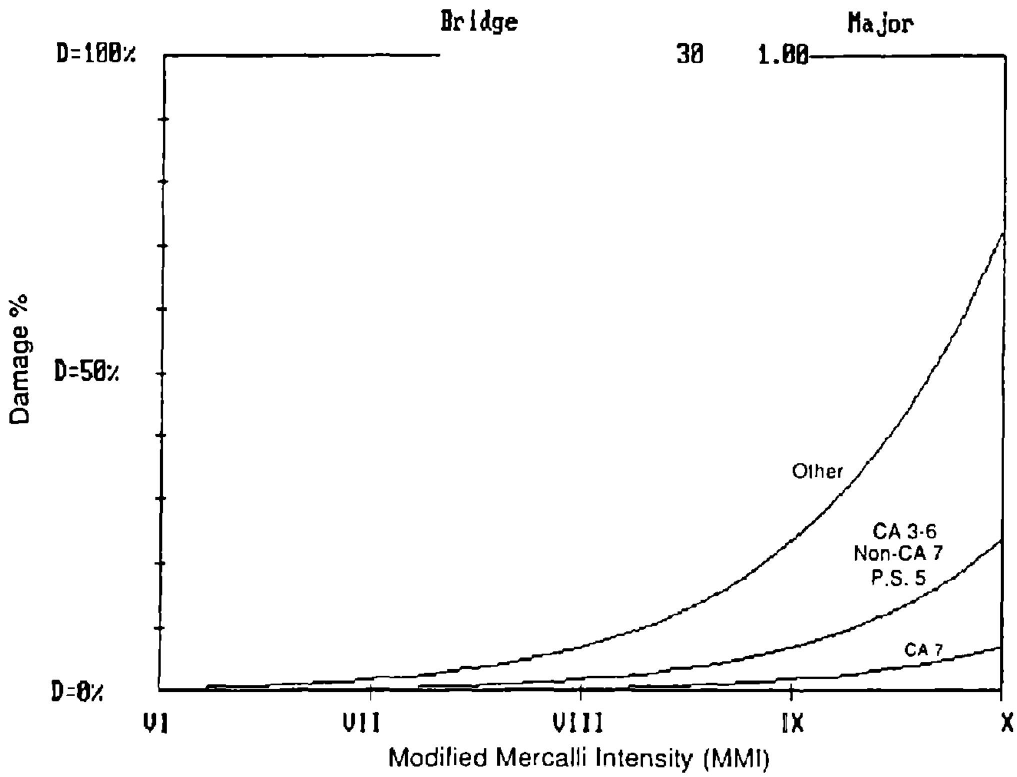


Figure B4-3 - Damage Percent by Intensity for Major Bridges

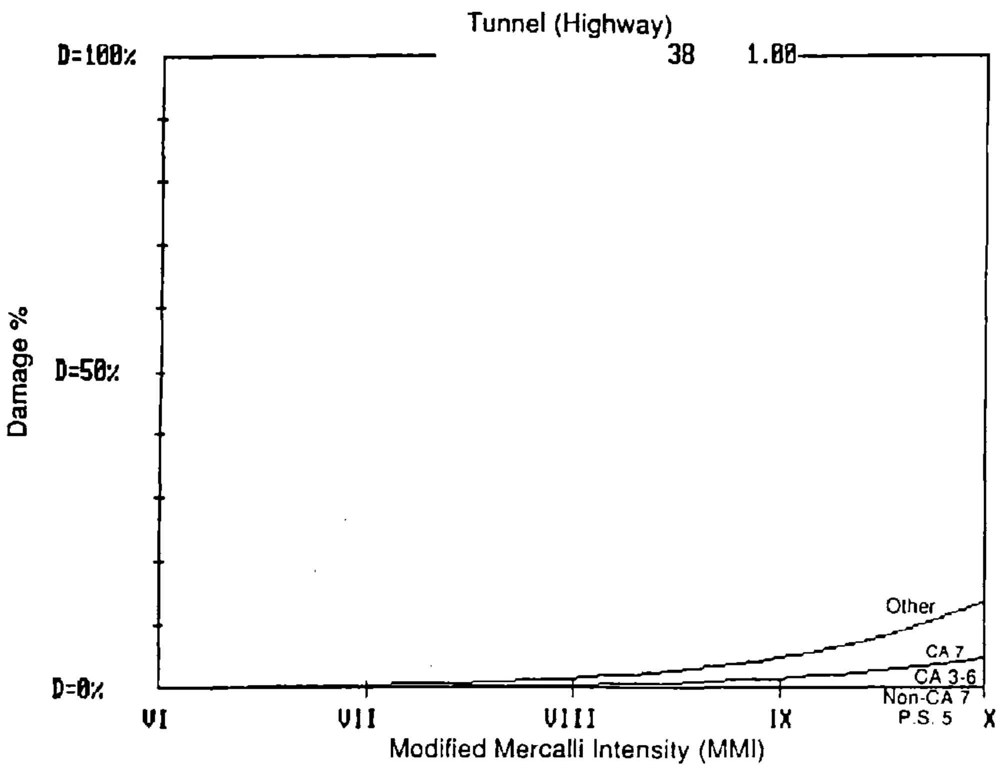


Figure B4-4 - Damage Percent by Intensity for Highway Tunnels

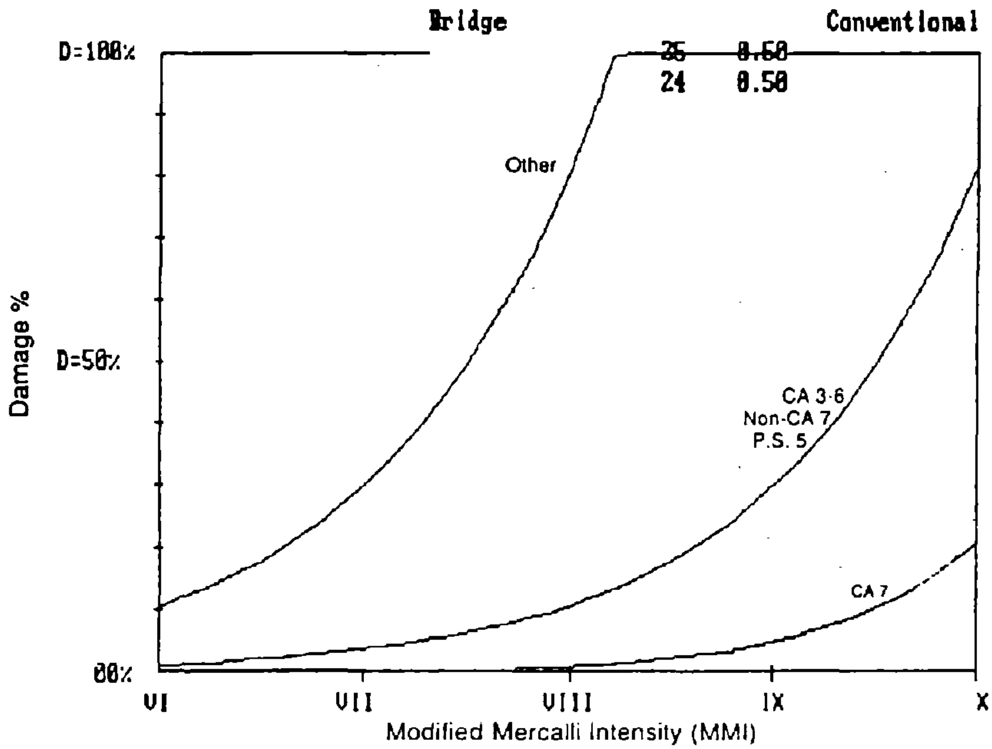


Figure B4-5 - Damage Percent by Intensity for Conventional Bridges

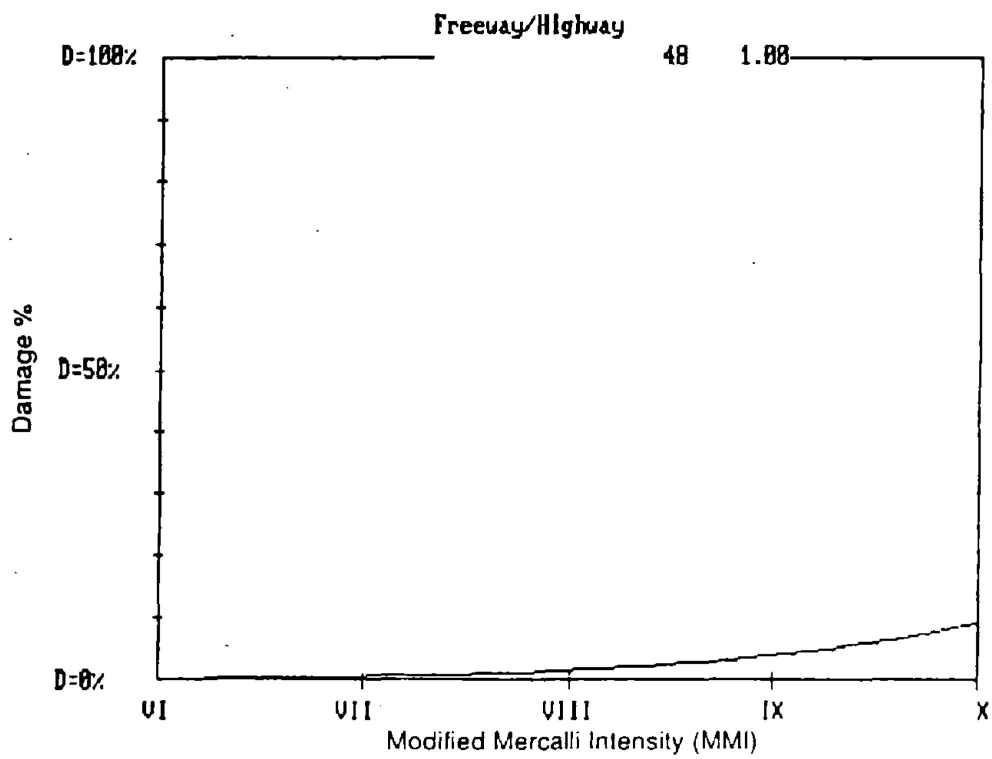


Figure B4-6 - Damage Percent by Intensity for Freeways/Highways

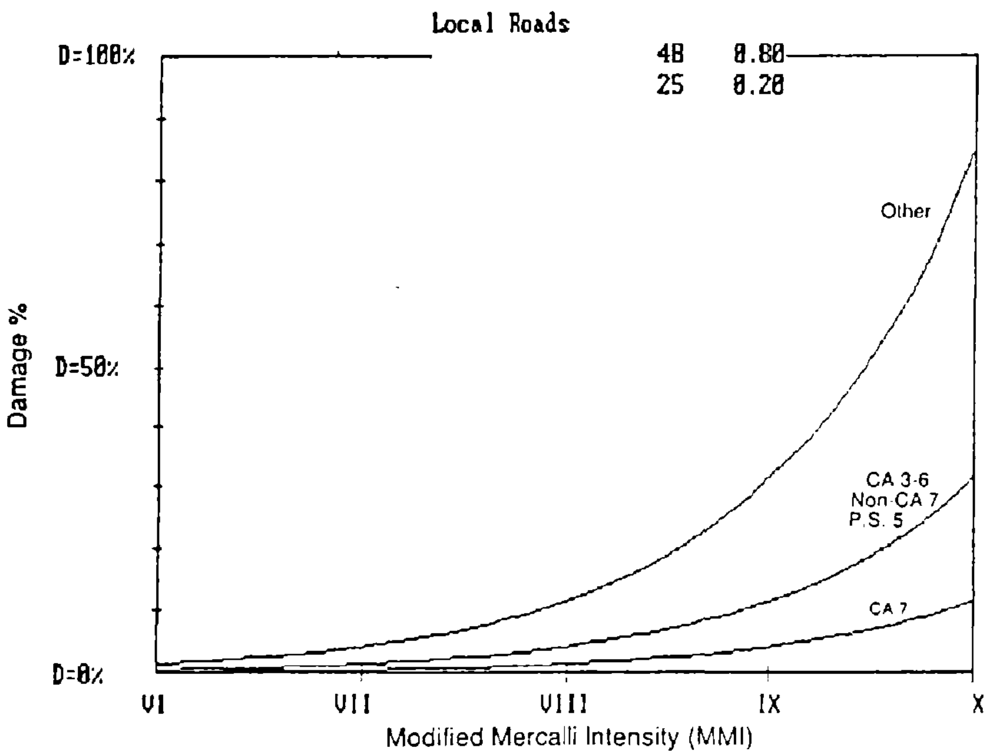


Figure B4-7 - Damage Percent by Intensity for Local Roads

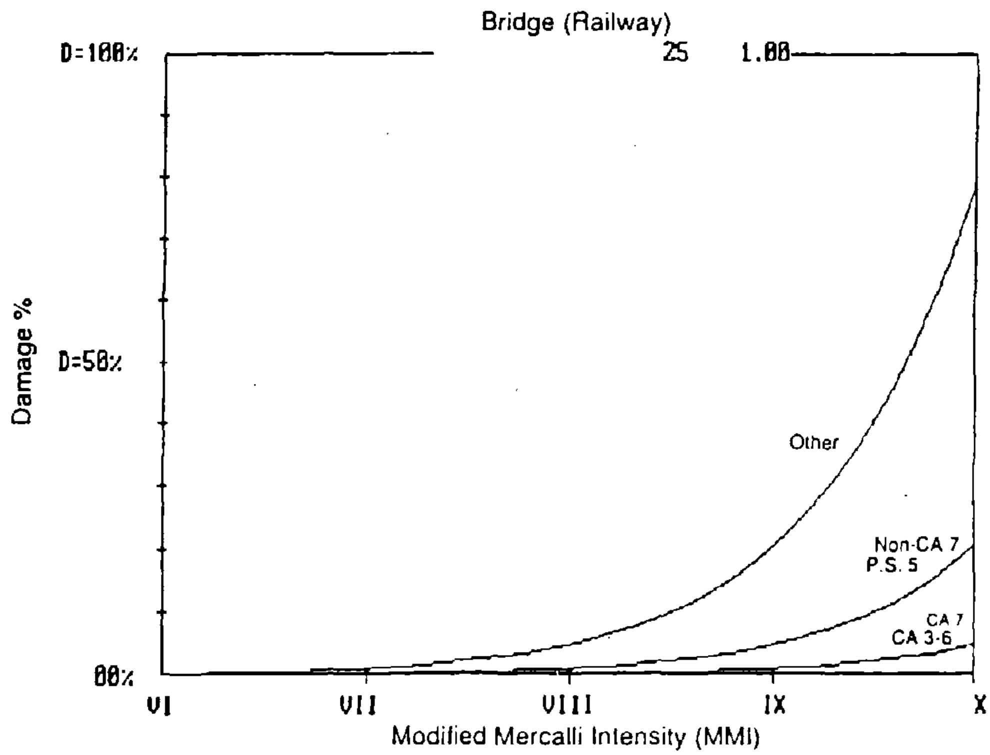


Figure B4-8 - Damage Percent by Intensity for Railway Bridges

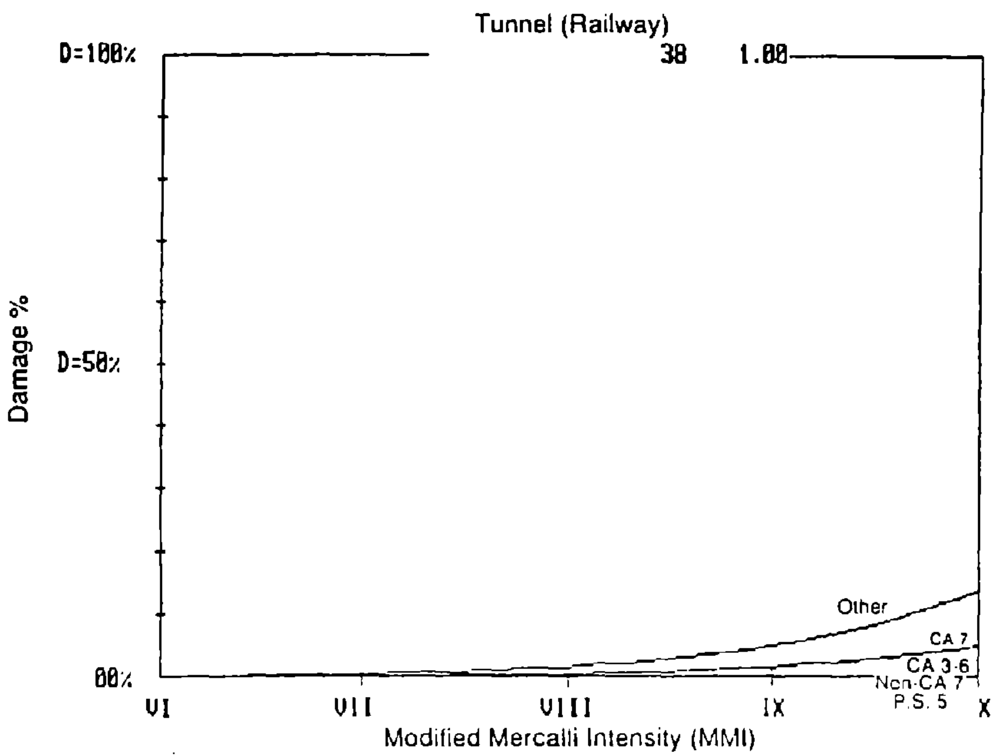


Figure B4-9 - Damage Percent by Intensity for Railway Tunnels

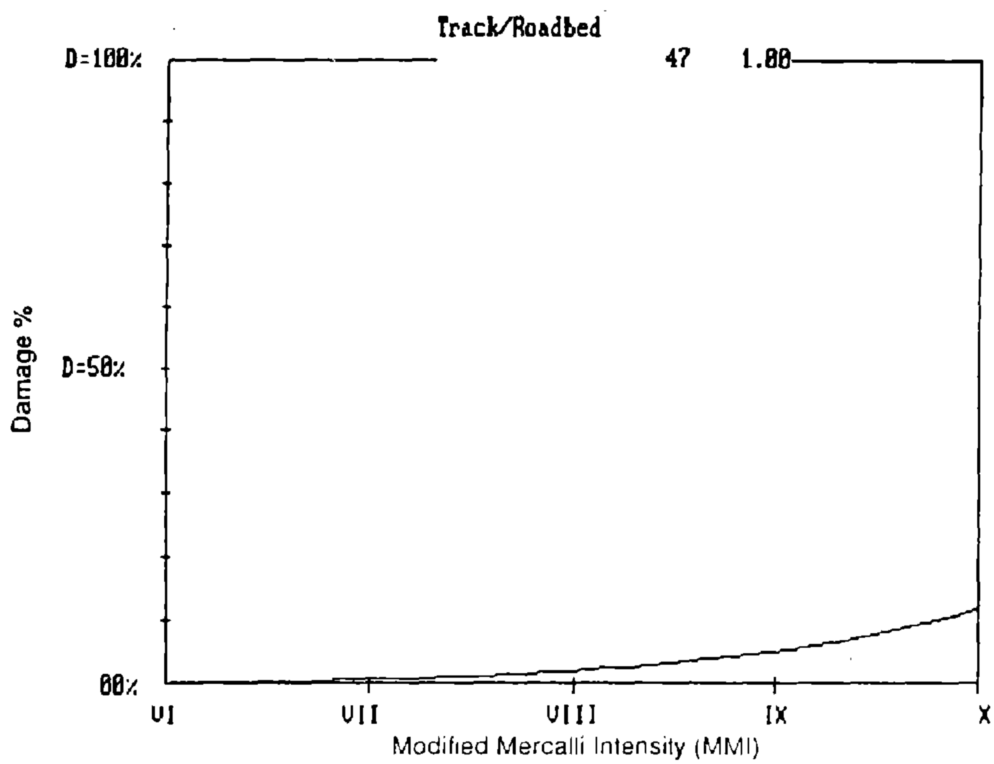


Figure B4-10 - Damage Percent by Intensity for Tracks/Roadbeds

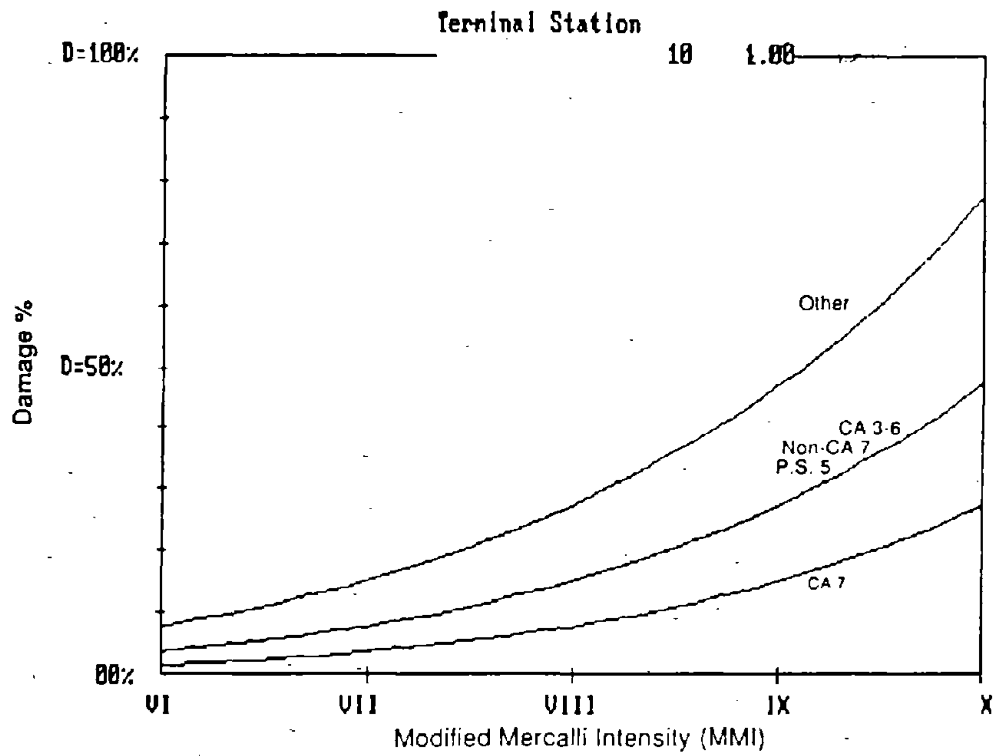


Figure B4-11 - Damage Percent by Intensity for Railway Terminal Buildings

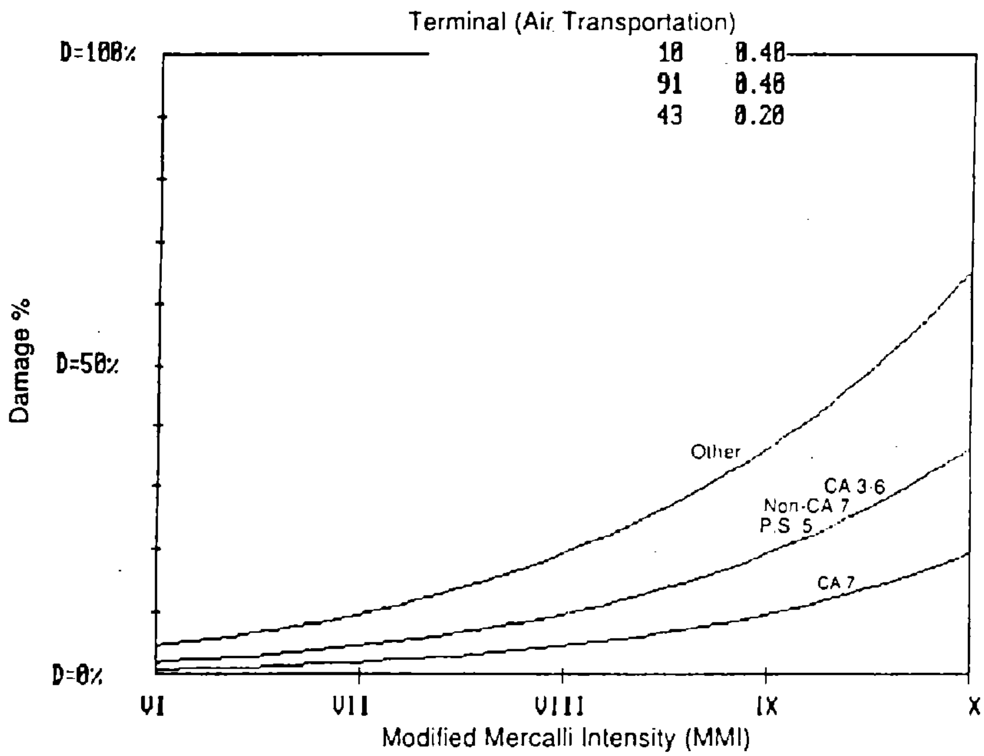


Figure B4-12 - Damage Percent by Intensity for Airport Terminals

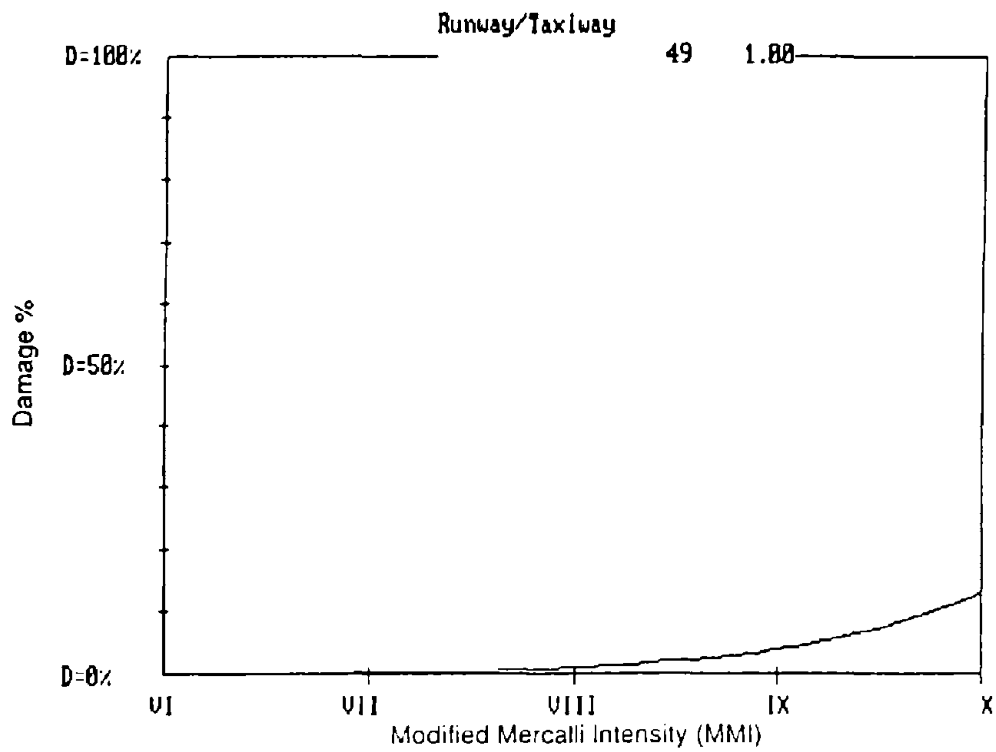


Figure B4-13 - Damage Percent by Intensity for Runways/Taxiways

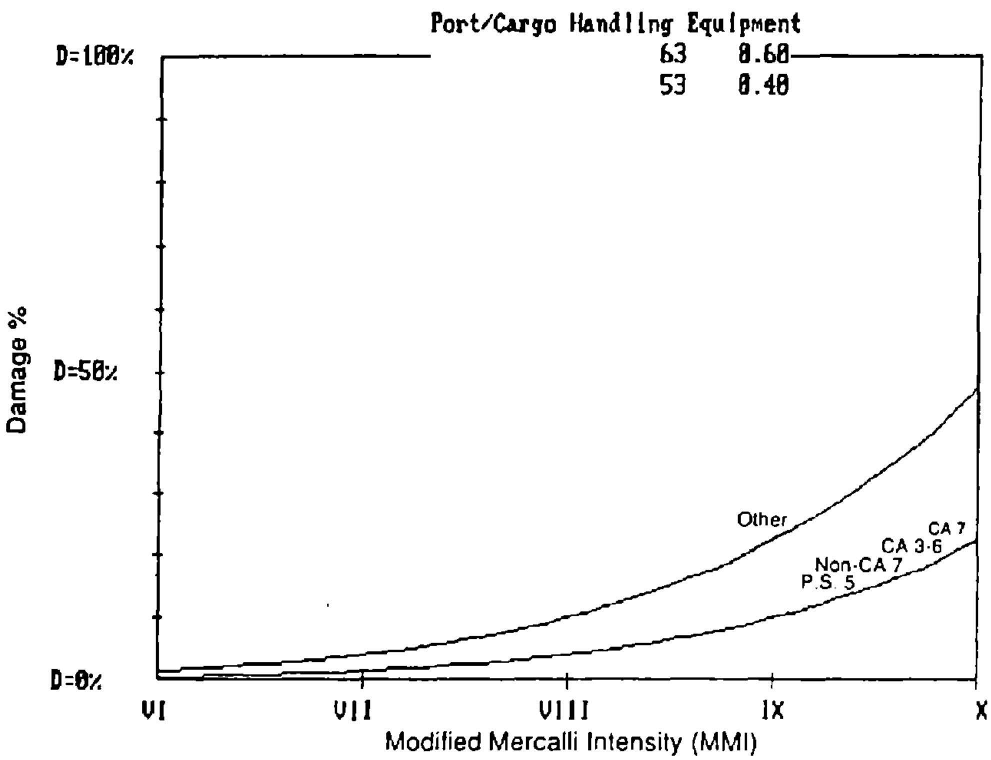


Figure B4-14 - Damage Percent by Intensity for Port/Cargo Handling Equipment

It should also be noted that the motion-damage curves are based on the data developed in the ATC-13 project, which in its original form was represented by damage probability matrices. The damage potential estimated by using these curves, therefore, represents the "expected" or "mean" damage loss.

Figure B4-4 through B4-14 present the motion-damage curves for the rest of the major components of transportation facilities. As some of these figures would indicate, there are cases where only one motion-damage curve is derived for a certain facility. The assumption made is that there is little or minimal variation in construction quality for this type of facility (such as tracks/roadbeds, Figure B4-10).

This potential damage and loss figure is a good indication of seismic vulnerability. In one number it summarizes the seismic hazard and vulnerability for a given facility in a given area of the United States and gives an indication of the consequences of a likely earthquake in terms of replacement cost.

Other factors should be considered by the transportation facility manager in this preliminary screening process. Importance of the facility to life safety, emergency preparedness and post-earthquake recovery, and socio-economic impacts should be considered when making decisions on programming of further study or construction work related to seismic vulnerability.

The method presented in this section is a valid approach for preliminary screening for seismic vulnerability. It can be useful for large groups of structures/facilities with similar characteristics to give an overall view of the vulnerability of each group as a whole, or to set up priorities of individual structures within the group. However, because of the great complexities and variations of real structures it should not be relied upon to give accurate results for a specific structure. This is the subject of the next section, where recommendations are given for the detailed analysis.

Another approach for bridges has been developed by the Applied Technology Council in ATC-6-2. In 1987, this was incorporated by the FHWA in their "Seismic Design and Retrofit Manual for Highway Bridges".

B 4.2 Detailed Analysis for Seismic Vulnerability

B 4.2.1 New Structures

For any new structure, the given state-of-the-art in seismic design presents the opportunity to create a safe facility that will result in acceptable facility performance during and after a seismic event. These design techniques are explained in detail in Appendix C. The vulnerability, assuming proper design methods are used, should be minimal. Specifically, it should behave in accordance with code objectives; it should sustain minor damage, but should not collapse or threaten the life safety of its inhabitants.

Except for extremely unusual structures, e.g., particularly radical geometry, or very heavy, suspended loads, the actual design and construction of a facility to withstand moderate a seismic event is a relatively straightforward process, adding about 1-6% to the total costs of the same structure, but without seismic design aspects. For very severe design seismic events, the premium design and construction costs can escalate very rapidly.

The most difficult decision in the process is the selection of the actual project/facility design earthquake, and the resulting seismic design parameters, i.e., ground accelerations, particle velocities, response spectra, etc. It is in these areas that uncertainty and controversy exist, and where it is most difficult to achieve consensus and agreement, even among experienced professionals. Detailed knowledge (which is costly) of local geology and site specific soil, rock, and groundwater conditions will aid in assessing the "proper" input parameters for this part of the problem, as discussed in detail in Appendix A.

Thus, for new structures, it is concluded that the profession is quite capable of designing and constructing facilities that would perform in and acceptable fashion for a given seismic event with minimal vulnerability.

B 4.2.2 Existing Structures

For existing structures/facilities, the problem of assessing the level of seismic vulnerability and potential damage for a specific facility in detail is much more complex and is beyond the scope of this report. In general, this type of study will require the expertise of structural and geotechnical engineering professionals experienced in seismic analysis and design methods. The general issues that must be addressed, however, are discussed below.

The detailed vulnerability assessment starts with a detailed seismic hazard assessment. The procedure for performing this assessment is covered in Appendix A. It involves consideration of the geographic location of the facility, potential earthquake sources, recurrence rates and geology in the area in order to arrive at seismic ground motion parameters, usually effective peak ground acceleration.

Once the seismic hazard has been determined, the vulnerability of the facility is determined. Many factors must be considered. The assessment must incorporate facility type - whether it is at grade, above ground or below ground. The design of the structural framing system and structural details must be evaluated to determine whether they have any impact on the vulnerability of the facility. Of particular importance is the building code to which the structure was originally constructed, and the adequacy of its seismic code provisions. The local geology and foundation details must be evaluated for vulnerability effects. General attributes of the facility must be taken into consideration including its age, occupancy, use, importance for emergency preparedness and port-earthquake recovery, its replacement costs and potential costs associated with loss of revenue. These issues are discussed in detail in Appendix C.

B 4.3 <u>Large Scale Facility Evaluation</u>

Assessment of seismic vulnerability on a large scale poses its own special problems. It is necessary to evaluate large numbers of facilities from technical, economic, and political viewpoints, with the objective of arriving at conclusions for the need for seismic retrofit, reconstruction, change of use, or other means of achieving a satisfactory level of facility vulnerability.

A comparative analogy with the issue of dam safety in the US can be instructive in this regard. In 1976, the Teton Dam, a new structure in Idaho owned by the Bureau of Reclamation, failed catastrophically upon its initial filling, resulting in loss of life and $6 billion in damages spread over several hundred square miles. This failure called into serious question the safety of existing dams in the US, and instigated the beginning of a US-wide Dam Safety Program.

This effort was initially carried out by the US Corps of Engineers and subcontractor consultants, and moved through Phase I (inventory and cursory evaluation), Phase II (detailed evaluation), and later further studies. After about 8-10 years, the efforts had grown to the point where a strong national association of State Dam Safely Programs were in place and funded to be able to carry out their mission of ensuring the safe design, construction, and operation of dams within their respective jurisdictions. The most shocking aspect of these efforts was that when they began the program in the 1970's , a number of states had no idea how many dams even existed in their area, let alone what condition they were in.

The Phase I investigations, i.e., just creating the inventory data base and performing a very cursory evaluation resulted in immediately taking out of service, hundreds of dams that were a threat to major property damage, and/or loss of life. Later phases of the effort resulted in more refinements to dam design, and resulted in more dam closures as well as retrofit upgrades to others.

A similar approach would be effective for seismic evaluation. Phase I would be a data collection and preliminary screening process using the methods described above. Phase II would be a detailed evaluation of critical facilities identified in Phase I. Phase III would consist of the establishment of remedial rehabilitation to facilities with unsatisfactory vulnerability. This would consist of facility closure, retrofitting, change of use, or new construction.

This can be accomplished by creating a technical database of all facilities under consideration, incorporating details on their vulnerability characteristics. This database could be nationwide, transportation system-wide (for example all interstate highways), regional, or it could be done by each transportation facility manager for all facilities under his/her purview.

There are currently thousands of existing facilities/structures in the transportation network, all under the jurisdiction and control of public and private entities. None have a database with sufficient site data and structural information to conduct a seismic vulnerability assessment.

For such entities to create the required database involves two aspects: 1) collecting data that exists, i.e., drawings, plans, specifications, design or construction reports, soil/rock boring, logs geologic reports, etc., and 2) obtaining new data via physical inspections, borings, testing, etc. For newer structures, say after 1960 or so, the database could probably be assembled with largely (1) while for older structures more of (2) would be required. As the use of (2) increases, the costs generally increase rapidly.

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Map 1.	ATC	County Map - Coefficient Aa
Map 2.	ATC	County Map - Coefficient Av
Map 3.	ATC	Contour Map for Coefficient Aa
Map 4.	ATC	Contour Map for Coefficient Av
Map 5.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 6.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 7.	USGS	0.1 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 8	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 9.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
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Map 11.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
Map 12.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years (California)

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APPENDIX C

SEISMIC DESIGN AND RETROFIT PRACTICE IN THE U.S.

C

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SEISMIC DESIGN AND RETROFIT PRACTICE IN THE U.S.

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EXECUTIVE SUMMARY

Appendix C is one of three appendices that provide a technical basis for "Seismic Awareness: Transportation Facilities", a report written for transportation facility managers to educate them to the potential for seismic hazards directly effecting their facilities (new and existing), to present a suggested approach to evaluate facility vulnerability and to address seismic design aspects of new and existing facilities.

Appendix C summarizes current seismic design and retrofit practices in the United States. It gives some history on seismic design, illustrating the evolution of seismic design technology. It explains in detail the methods currently used for the design of different types of transportation facilities and components. Finally, it reviews the economic considerations involved and emphasizes the cost-effectiveness of incorporating seismic design elements into new structures during pre-construction design as opposed to retrofitting existing structures.

Appendix A, included in a separate volume, describes the nature of seismic *hazards* in the United States. Appendix B, also included in a separate volume, discusses the *vulnerability* of transportation facilities.

Vulnerability to earthquake damage can be reduced by good engineering design. Research is constantly revealing new and more effective methods of designing for seismic resistance. Seismic design and retrofit practice in the U.S. has evolved quickly within the last century. Until the 1950's, there were virtually no seismic provisions in force, except for some limited requirements in California. Today, engineers have the capability of designing structures to withstand earthquakes with a high level of reliability.

Designing buildings and structures to be resistant to earthquakes is a complicated problem. The forces a structure will experience from an earthquake depend on many seemingly unrelated factors including: site soil conditions, the structure's geometry, type of framing, connection detailing (how members are connected together to resist the applied structure loads), and the magnitude, frequency and duration of the earthquake ground-shaking excitation. Often, an iterative approach is necessary, consisting of: estimation of seismic forces using assumed structural member properties, design of members, calculation of revised force estimates using the designed structural member properties, and repetition of the process until a satisfactory design results.

There is more uncertainty in seismic design theory than in other areas of structural design. The accuracy with which earthquake forces can be predicted is approximate at best. Even with extensive subsurface investigation, soil properties can only be estimated, as can the nature and severity of the anticipated earthquake. The seismic design problem is largely one of simplification based on reasonable assumptions. The techniques currently in use, however, have proved to be effective.

The term "seismic retrofit" is used to describe the construction of improvements to improve the performance of existing structures during an earthquake such that their vulnerability can be restored to an acceptable level.

Many structures currently in use as part of transportation facilities were designed and constructed prior to the development of modern seismic design techniques. There are, therefore, many facilities that are vulnerable to serious earthquake damage. Seismic retrofits are accomplished in order to restore them to an acceptable level of safety based on newly developed technology.

Adequate provisions at the time of construction can be made with modest increases in total investment, usually less than 5% of the total facility cost. Retrofitting can expensive, and the financial and social costs of major repair or replacement are sometimes not viable. It is clear that the minor additional cost of building a safer structure is well justified.

APPENDIX C

SEISMIC DESIGN AND RETROFIT PRACTICE IN THE U.S.

C 1.0 INTRODUCTION

Vulnerability to earthquake damage can be reduced by good engineering design. Research is constantly revealing new and more effective methods of designing for seismic resistance. Seismic design and retrofit practice in the U.S. has evolved quickly within the last century. Until the 1950's, there were virtually no seismic provisions in force, except for some limited requirements in California. Today, engineers have the capability of designing structures to withstand earthquakes with a high level of reliability.

The evolution of seismic theory received much stimulus from the occurrence of earthquakes. As major earthquakes occurred, the new data collected prompted reconsideration and revision of the philosophies and codes. Also, increased public awareness at these times provided the political viability of expending resources on these issues. Therefore, the practice of seismic design and retrofit is in a state of flux as new lessons are learned after new major earthquakes.

C 1.1 Background

Many major transportation facilities have been built in the U.S. since the 1960's. These include highways, transit systems, airports, and water ports. Also during this period, awareness of the force, magnitude, and frequency of earthquakes has increased due to recent earthquake disasters worldwide and increased seismic monitoring programs. Two dramatic occurrences in the U.S. which caused increased concern over the seismic resistance of transportation facilities were the San Fernando (1971) and Loma Prieta Earthquakes (1989). These earthquakes caused major damage to many transportation facilities and both forced changes in the way that certain structures are designed for seismic loading.

In large part, structures which fail during an earthquake have been designed with outdated criteria or for smaller earthquakes. Older criteria (previous to 1960) did not provide modern guidance related to detailing of joints and structural ductility. Many structures designed recently (after 1970) consider these important aspects of seismic design and are more resistant to earthquake forces than older structures, as confirmed by the performance of new structures in recent earthquakes.

The seismic design practices of today have deep roots in the practices which evolved years ago, although many new approaches have been proposed in recent times. What has changed significantly over the years is the value of input parameters used and philosophies governing joint details, connections, and ductility demand. New seismic design philosophies tend to be re-evaluated every time there is a major earthquake. Thus, this appendix represents a snapshot of current seismic design and retrofit philosophies, but they will continue to advance and change with time.

C 1.2 Purpose of This Appendix

Appendix C is one of three appendices that provide a technical basis for "Seismic Awareness: Transportation Facilities", a report written for transportation facility managers to educate them to the potential for seismic hazards directly effecting their facilities (new and existing), to present a suggested approach to evaluate facility vulnerability and to address seismic design aspects of new and existing facilities.

Appendix C summarizes current seismic design and retrofit practices in the United States. It gives some history on seismic design, illustrating the evolution of seismic design technology. It also explains in some detail the methods currently used for the design of different types of transportation facilities and components. Finally, it reviews the economic considerations involved and emphasizes the cost-effectiveness of incorporating seismic design elements into new structures during pre-construction design as opposed to retrofitting existing structures.

Appendix A, included in a separate volume, describes the nature of seismic *hazards* in the United States. These hazards include the probability of occurrence of an earthquake in a certain area, as well as its likely intensity. Seismic hazard is dependent on location, geology and the location of subsurface features that can cause earthquakes.

Appendix B, also included in a separate volume, discusses the *vulnerability* of transportation facilities. Vulnerability here refers to the likely consequences of the expected seismic event on a particular structure. Unlike seismic hazard, vulnerability applies to specific structures. It is dependent on the expected seismic hazard, as well as the structural characteristics of the facility and the *local* geology of the site. Vulnerability is also distinguished from hazard in that hazard is a naturally occurring phenomena that man is unable to affect, while vulnerability is dependent on human factors that we have control over and can change - like the construction of a building or the steepness of an earth slope.

C 1.3 Approach

This study provides a review of basic seismic design principles, and comparison of various codes and criteria that have been developed to date. The affect of major earthquakes on the evolution of seismic technology is evaluated and the latest state of the art in seismic design theory is summarized. Engineering and design methods currently being practiced in the United States for the various structure types are described in detail. Finally, the additional construction cost of incorporating seismic resistance is discussed.

There are many different types of transportation facilities, as described in Appendix B. Design of these types of facilities can be conveniently broken down into five categories, as follows:

1. *Buildings:* Railway terminal buildings, rail transit terminal buildings, airport terminal buildings and port facility buildings.
2. *Bridges:* Major and conventional highway, railway and rail transit bridges.
3. *Marine Structures:* Piers and wharves.
4. *Subsurface Facilities:* Highway, railway and rail transit tunnels, retaining walls and bulkheads.
5. *At Grade Facilities:* Freeways, highways, local roads, tracks, roadbeds, runways and taxiways.

Currently, there are extensive code provisions for categories 1 and 2. There is limited guidance for the design of marine structures or subsurface facilities (categories 3 and 4); however the design of these features certainly must incorporate seismic considerations. Usually project specific criteria are developed. At grade facilities (category 5) have minimal impact from earthquakes, and typically seismic design is not considered for this category, except for potential damage from landslides, liquefaction, and surface faulting displacement. This appendix, therefore, only covers design and retrofit practice for the first three categories.

C 2.0 SEISMIC DESIGN PRINCIPLES

C 2.1 Background

Designing buildings and structures to be resistant to earthquakes is a complicated problem. The forces a structure will experience from an earthquake depend on many seemingly unrelated factors including: site soil conditions, the structure's geometry, type of framing, connection details, and the magnitude, frequency and duration of the earthquake ground-shaking excitation. Often, an iterative approach is necessary, consisting of: estimation of seismic forces using assumed structural member properties, design of members, calculation of revised force estimates using the designed structural member properties, and repetition of the process until a satisfactory design results.

There is more uncertainty in seismic design theory than in other areas of structural design. The accuracy with which earthquake forces can be predicted is approximate at best. Even with extensive subsurface investigation, soil properties can only be estimated, as can the nature and severity of the anticipated earthquake. The seismic design problem is largely one of simplification based on reasonable assumptions.

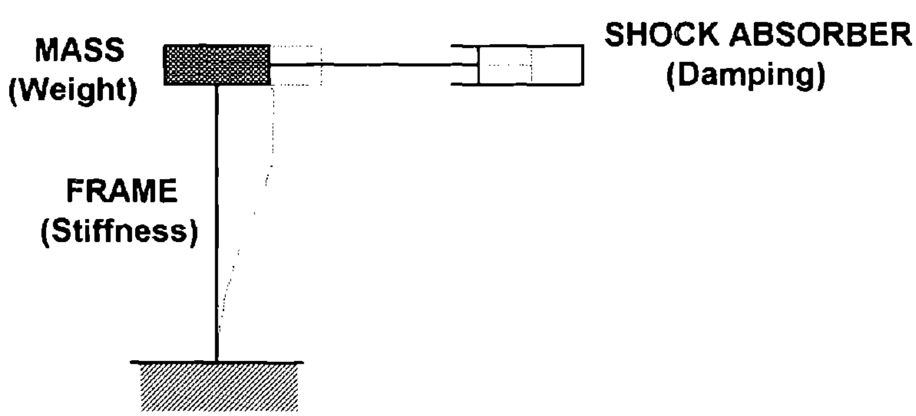
C 2.2 Seismic Engineering Fundamentals

Structural design for earthquake conditions is a dynamics problem. Earthquakes cause ground shaking that in turn shakes the structure, imposing an acceleration on the structure. This dynamic movement causes inertia forces equal to the structure's mass multiplied by its acceleration.

$$F_{eq} = m_b \times a_b$$

where
F_{eq} = Earthquake force on structure
m_b = Mass of structure = Weight of structure/acceleration of gravity
a_b = Acceleration of structure

The biggest problem in seismic analysis and design is in determining the structure's acceleration. To do this, structures are modeled, in simple terms, as shown in Figure C2-1.



- *Mass represents weight of structure.*
- *Frame represents stiffness of structure's framing system.*
- *Shock absorber represents structure's capacity to dampen movement*

Figure C2-1 - Simplified Structural Model.

The structure acceleration is a function of a number of factors: the *ground acceleration* caused by the earthquake, the structure's *fundamental period of vibration*, the structure's *ductility*, and the structure's *damping*. Also, the frequency content of the ground motion, and its *resonance* with the structure are important elements that will determine the magnitude of the earthquake load. These concepts are described below.

Ground Acceleration: The earthquake induced vibration in the earth crust causes an acceleration in the soil or rock surrounding a structure. Values of ground acceleration can be obtained from maps developed statistically based on historical data from past earthquakes. The most commonly used ground acceleration is that which has a 90% chance of not being exceeded in fifty years. Examples of two types of ground acceleration are shown in Figure C2-2 and C2-3 (From BOCA Building Code - 1992 Supplement). The derivation and use of these maps is discussed in detail in Appendix A.

Fundamental Period of Vibration: A period of vibration is the length of time a freely vibrating structure takes to vibrate through one complete cycle (see Figure C2-4). In the case of a tuning fork there is one fundamental period that describes its movement. In reality a tuning fork, building, or any structure, vibrates in a complex fashion with many different periods. There is one period, however, that predominates. This is the fundamental period. The structure's mass and stiffness determine its fundamental period of vibration.

Ductility: A ductile structure is capable of sustaining large earthquake induced movements without fracture. It is distinguished from flexibility in that the ductile structure imparts a high resisting force early in its movement which remains nearly constant throughout the movement. A flexible structure, on the other hand, imparts a low resisting force which steadily increases as the movement increases, until fracture occurs. Throughout ductile movement, energy is absorbed by the member, dissipating the energy of vibration. This is characteristic of structural steel structures and heavily reinforced concrete structures.

In more technical terms, ductile movement of a structure occurs when, as a displacement is imposed on it, stresses within the structural members build up to the point where they exceed the normal elastic capacity, and the material yields. With continued movement, a ductile material will continue to yield, imparting a nearly constant resisting force to the structure (see Figure C2-5). Of course, there is a limit to the amount of movement that the structure can tolerate without collapse.

Damping: This is the physical phenomenon that causes a freely vibrating structure to taper off over time and eventually come to rest (see Figure C2-6). As a tuning fork vibrates, the air dampens the movement through friction, causing it to taper off gradually. A tuning fork immersed in water experiences much more damping, and tapers off almost immediately. Damping in a structure is caused primarily by friction loss within structural components, and by ductility.

Resonance: When a structure's fundamental period of vibration is close to the period of induced vibration (ground shaking), the structure experiences *resonance*. Under these conditions, the structure's vibration increases in magnitude without bound until the structure fails. Damping puts a limit on the magnitude of the vibration that can occur (see Figure C2-7).

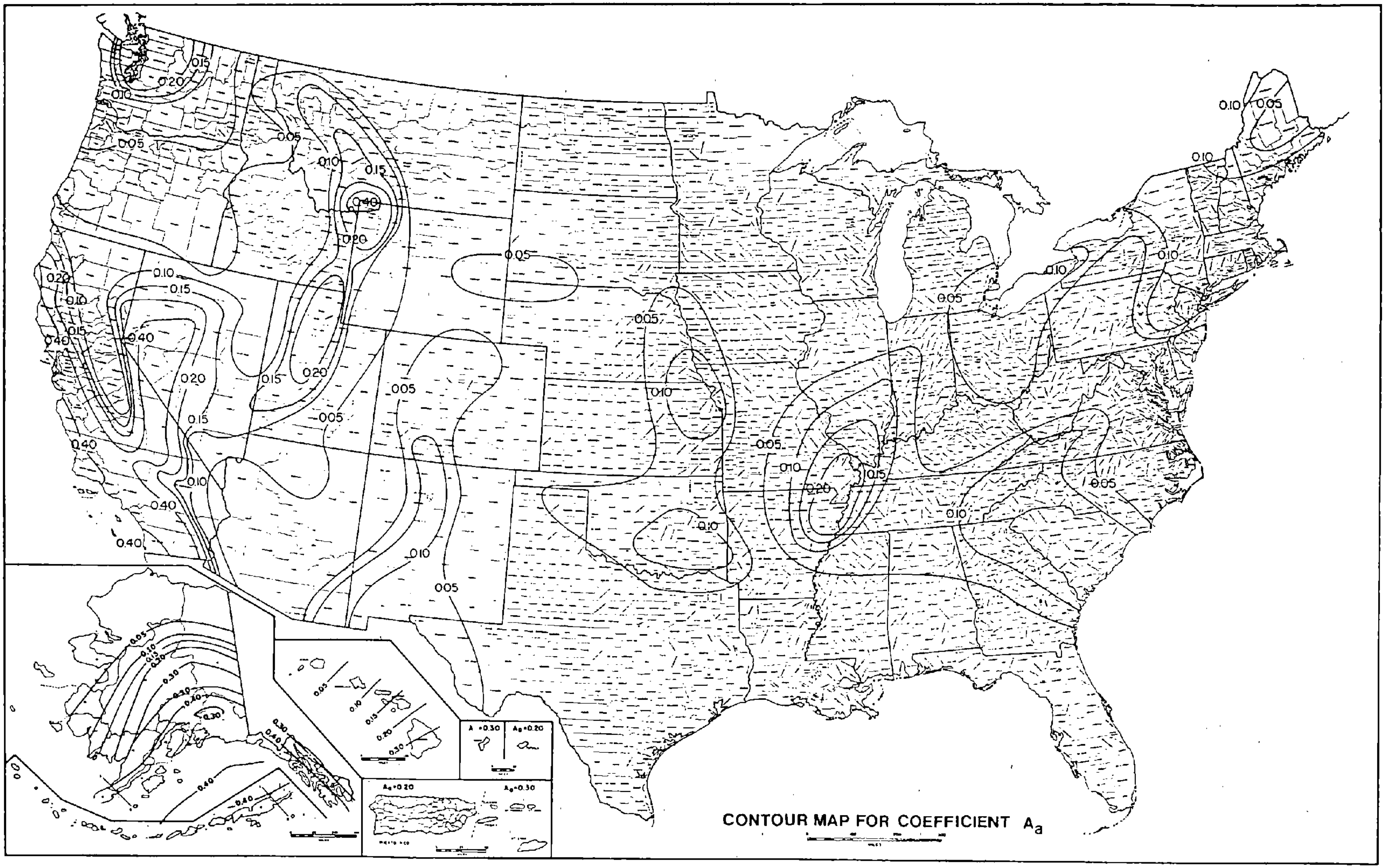


Figure C2-2 - Contour Map of Effective Peak Acceleration Coefficient, (A_a).

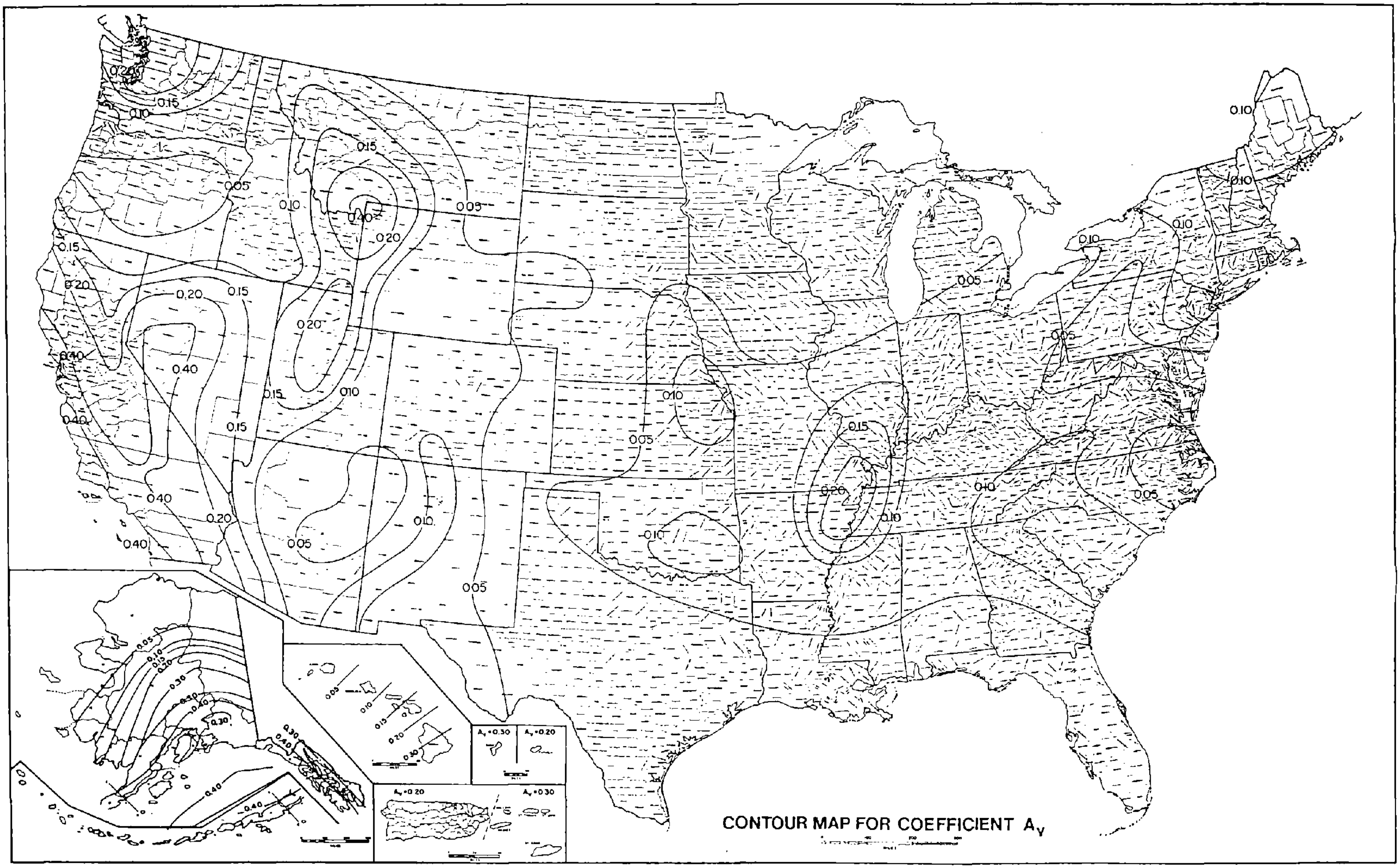


Figure C2-3 - Contour Map of Effective Peak Velocity-Related Acceleration Coefficient, (A_v)

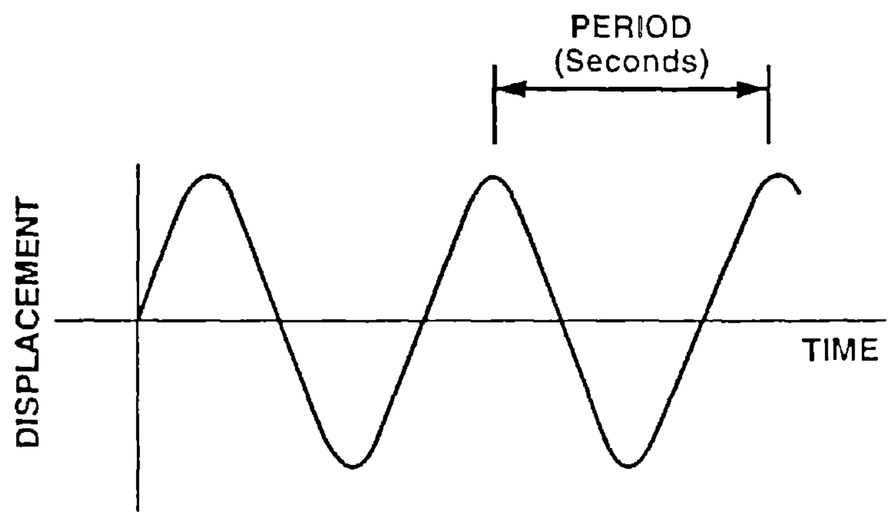


Figure C2-4 - Period of Vibration

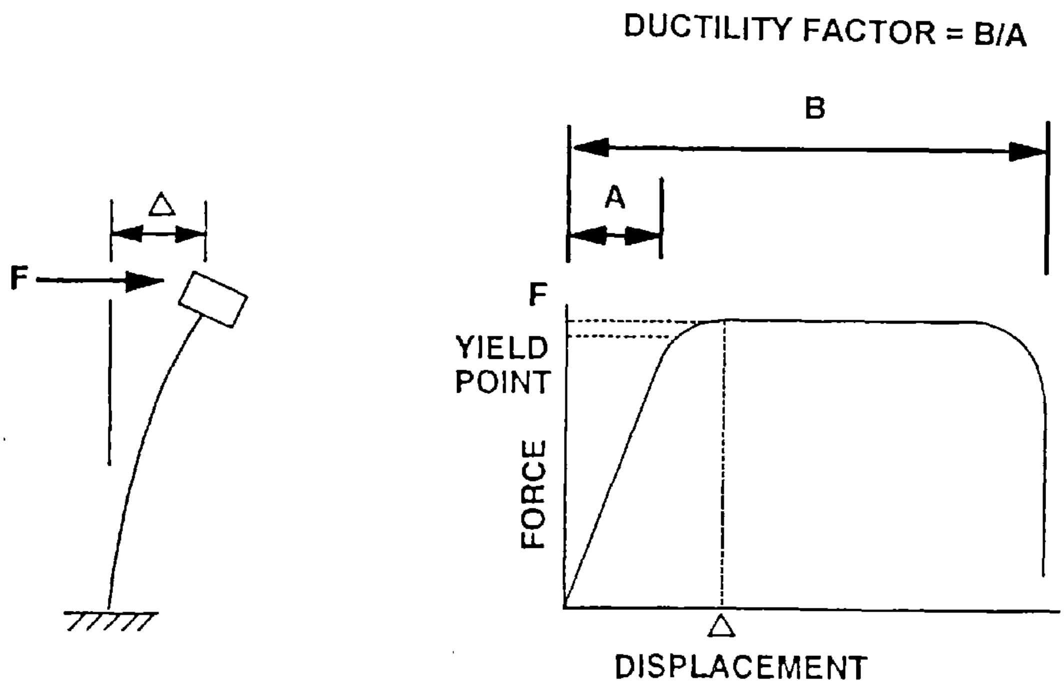


Figure C2-5 - Ductility

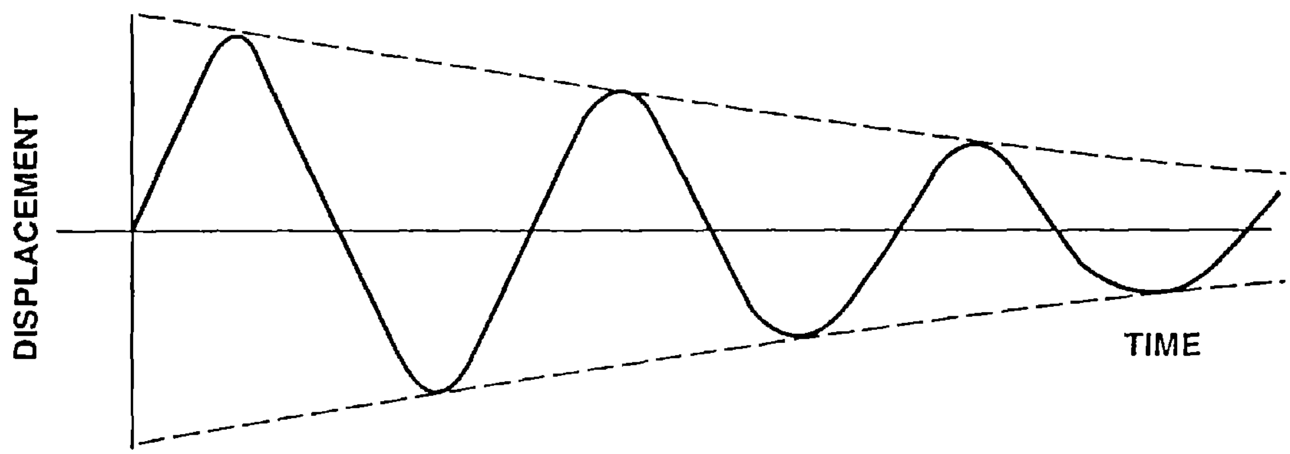


Figure C2-6 - Effect of Damping on Free Vibration

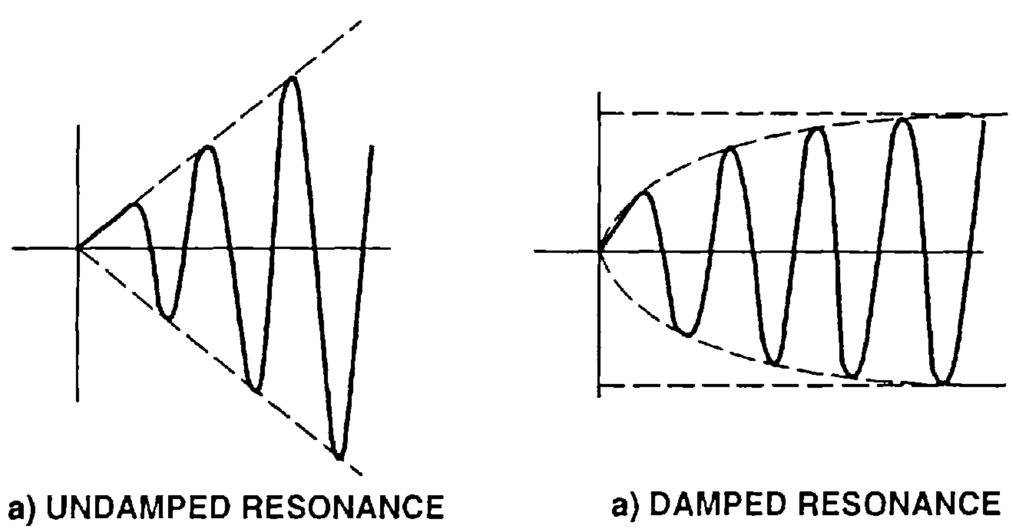


Figure C2-7 - Undamped and Damped Resonance

C 2.3 Governing Codes

There are a number of building codes in existence that reflect various methods of reducing seismic vulnerability. The codes have slightly different approaches to seismic design, but the general philosophy is the same. Structures are designed to resist minor earthquakes without damage. Seismic provisions are not intended to protect the structures from damage from major earthquakes, however, but to protect the life safety of the inhabitants, or users of the structures. Since earthquakes are generally a rare occurrence, codes attempt to prevent collapse, but not damage. Structures would be prohibitively expensive if they had to be designed to withstand major earthquake forces with no damage.

Codes take advantage of the ductility of structures, which results in a reduction of design load. Because of a ductile structure's ability to absorb energy from an earthquake, members can be designed for the yield load that would be sufficient to resist collapse under these conditions, with some safety factor. This member load can be reduced to as little as $^1/_{12}$ of the elastic load.

In order to accommodate ductile movement, special detailing of structural members is necessary. Masonry walls must be strengthened with reinforcing steel, concrete beams and columns must have extra reinforcing steel near the connections at their ends, and structural steel connections must be strengthened.

The codes used for structural design can be split conveniently into two categories, those for buildings, and those for bridges.

Seismic design requirements for buildings are prescribed in building codes. Local jurisdictions usually have their own building codes. Typically, towns or cities adopt the state building code, modified with special provisions for their locality. State codes are usually based on one of three model codes:

The Uniform Building Code (UBC)	Published by the International Conference of Building Officials, Whittier, CA.
The BOCA National Building Code (BOCA)	Published by the Building Officials and Code Administrators International, Chicago, IL.
The Standard Building Code	Published by the Southern Building code Congress International, Birmingham, AL.

The codes require that all buildings, except for small residential buildings in low risk areas and agricultural storage facilities, be designed for earthquake effects. Design requirements vary according to building type and ground acceleration.

The seismic provisions contained in the three codes are based primarily on two source documents. The BOCA and Standard Building Codes are based on the NEHRP (National Earthquake Hazards Reductions Program) *Recommended Provisions for the Development of Seismic Regulations for New Buildings*, while the UBC is based on the SEAOC (Structural Engineers Association of California) *Recommended Lateral Force Requirements and Commentary.*

Highway bridges are covered by the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for the Design of Highway Bridges*. The seismic provisions of the AASHTO code are similar in concept to the provisions in the building codes, but there are some requirements unique to bridges.

The American Railway Engineering Association (AREA) *Manual for Railroad Engineering* is used for the design of railroad bridges. This code currently gives no specific provisions for seismic design. In practice, the AASHTO code is usually used for seismic design of railroad bridges. Work is underway by AREA on the development of criteria for the design of concrete components for bridges in seismic zones. Retrofit criteria and specifications are also being prepared.

C 2.4 Evolution of Seismic Code Provisions

Seismic design theory has been one of the latest developments in structural engineering. Theory for seismic design was basically non-existent prior to 1900. Code-prescribed design requirements for seismic loads have undergone a great evolution in the past century, and they are still evolving. The greatest change has been initiated by and has followed major earthquakes. This is due not only to political pressures and increased public consciousness following a major event, but also because many new things are learned from each earthquake. New ground motion data is obtained, detailing deficiencies are observed, and successful structures are evaluated. This new information is eventually put into practice through new code provisions.

Table C2-1 presents a chronology of the major events within the last century that have brought us to the current state of the art in seismic design.

Table C2-1

Chronology of Seismic Design Theory

Year	Event	Resulting Seismic Design Theory Development
1906	San Francisco, California Earthquake	Equivalent wind load of 30 psf used to account for seismic effects (San Francisco Code)
1911	Messina, Italy Earthquake	Equivalent static inertial force used, equal to 10% of building's weight (dead load only).
1925	Santa Barbara, California Earthquake	US Coast and Geodetic Survey study of strong motion seismology.
1927	Uniform Building Code Published (First Edition)	Equivalent static force used, equal to 7.5% of building's dead plus live load for good soils, 10% for poor soils. Extra allowable stress permitted for earthquake loading.
1933	Long Beach, California Earthquake	Equivalent static force used, equal to 8% of building's dead plus half the live load for good soils, 16% for poor soils. Minimum reinforcement requirements developed for masonry. Special requirements developed for schools and hospitals.
1940	El Centro, California Earthquake (magnitude 7.1)	First time-history record of ground motion obtained from seismographs.
1943	New City of Los Angeles Building Code Published	Seismic design provisions tied to building's mass and dynamic properties. Provisions made for vertical distribution of loads.
1952	San Francisco Joint Committee Provisions Published	Seismic code developed for City of San Francisco. Recommended use of modal analysis and a response spectrum from the El Centro Earthquake. Provisions made for vertical and horizontal load distribution.
1957	Mexico City Earthquake	Superior behavior was noted by the engineering community, of the recently completed 43 story Latino-Americano Tower, which was designed using dynamic analysis principles.
1957	Formation of Structural Engineers Association of California (SEAOC)	The Structural Engineers Associations in southern and northern California formed a joint committee charged with the development of uniform seismic code provisions for the state.

1958	AASHO Specifications for Highway Bridges Published	American Association of State Highway Officials, in their 1958 publication of Specifications for Highway Bridges, specified an equivalent static load approach with seismic force equal to 2%, 4%, or 6% of dead load, depending on soil conditions and foundation type.
1960	SEAOC Seismic Provisions Published	Seismic provisions developed, later adopted by UBC. Seismic base shear force tied to period of building and its mass, consisting only of dead load (and some live load in warehouses and storage use). Factor included for different types of framing systems.
1964	Alaska Earthquake	Severe, destructive earthquake occurred. No ground motion data was obtained.
1964	Niigata, Japan Earthquake	Many instances of liquefaction observed, causing bearing capacity failure of building foundations.
1966	SEAOC Seismic Provisions Published	Concept of ductile concrete frame incorporated. Detailing requirements for concrete columns and beams introduced. Special requirements included for tanks and other non-building structures.
1967	Caracas, Venezuela Earthquake	Damage observed due to rigid non-structural walls located on upper portions of the building, creating soft story in lower portions.
1968	CALTRANS Seismic Provisions Published	The California Department of Transportation developed the CALTRANS Dynamic Characteristics Method. This was an equivalent static force approach for seismic bridge design incorporating factors for different structure types and periods of vibration.
1971	San Fernando Earthquake	Performance of buildings designed according to previous codes studied, problems identified. More ground motion time-history data obtained.
1973	CALTRANS Earthquake Design Criteria Published	Introduced new seismic design criteria for bridges, incorporating seismicity, soil effects, dynamic characteristics, and ductility.
1974	SEAOC Seismic Provisions Published	Specified accelerations increased, importance factor included for hospitals and other life-safety related structures, and site factor included to account for soil conditions and soil/structure interaction. Limitations prescribed for drift, the horizontal deflection of the building resulting from earthquake loading.
1975	AASHTO Interim Provisions Published	Incorporated 1973 CALTRANS provisions into the Specifications for Design of Highway Bridges. Added acceleration coefficient map for entire U.S.
1976	New UBC Code Published	1974 SEAOC provisions adopted.
1976	Applied Technology Council Report ATC 3-06, Tentative Provisions for the Development of Seismic Regulations for Buildings Published	Major development of seismic building design provisions. Realistic accelerations specified using ground motion contour maps. Method developed for determining equivalent static force for elastic response, and reductions to account for ductile behavior. Extensive detailing provisions incorporated.
1977	Earthquake Hazards Reduction Act passed by Congress	National Earthquake Hazard Reduction Program established (NEHRP), funding appropriated for research toward development of building code provisions.
1979	Formation of Building Seismic Safety Council	Group formed as mechanism for review of ATC 3-06, and discussion of seismic building code issues. Review completed, recommendations for improvement developed, and sample designs performed to evaluate costs, and feasibility of code recommendations. Further refinements made as a result.
1981	ATC-6 Seismic Design Guidelines for Highway Bridges Published	Study funded by the Federal Highway Association (FHWA) developed seismic guidelines for bridges, evaluated impact on bridge design, construction, and costs.
1983	AASHTO Guide Specifications for the Seismic Design of Highway Bridges Published	1981 ATC-6 guidelines adopted as AASHTO Guide Specification. Option given for its use instead of 1975 Interim AASHTO Specifications.
1983	ATC-6-2 Report Published	FHWA funded study for retrofitting of bridges completed.

1985	NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings Published	Tentative provisions of ATC 3-06 adopted by the BSSC. Further detailing requirements made, more guidance given for irregular buildings.
1985	Mexico City Earthquake	Extensive damage observed in region with unusually soft clay soils.
1988	New SEAOC Provisions Published	New site factor incorporated to account for soft clay soils.
1988	New NEHRP Provisions Published	New site factor incorporated to account for soft clay soils. New hazard maps generated. New detailing requirements for steel braced frames. More detail on the effect of building configuration.
1988	New UBC Code Published	1988 SEAOC provisions incorporated. Essentially the same philosophy as NEHRP provisions, except working stress rather than ultimate strength methods specified.
1988	Armenia Earthquake	Detailing deficiencies in Soviet Codes illustrated by observed damage.
1989	Loma Prieta, California Earthquake	Tremendous amount of data gathered, validity of latest code provisions and design theory generally confirmed.
1990	New BOCA Code Published	Incorporated NEHRP-85 provisions.
1991	Interim AASHTO Specifications Published	1983 AASHTO Guide Specifications adopted in 1990 and incorporated into 1991 Interim Standard Specifications for Highway Bridges
1991	New NEHRP Provisions Published	New response ordinate maps. More conservative load combinations for sensitive components. New provisions for timber structures, and anchor bolts.
1991	New UBC Code Published	Latest edition.
1991	New Standard Building Code Published	Incorporated NEHRP-85 provisions. Almost identical to BOCA-90.
1992	Supplement to BOCA-90 Published	Latest edition. Incorporated NEHRP-91 Provisions.
1992	Supplement to 1991 Standard Building Code Published	Latest edition. Incorporated NEHRP-88 Provisions. *Note: it is anticipated that the 1993 Supplement will incorporate the NEHRP-91 Provisions.*

C 3.0 CURRENT U.S. SEISMIC BUILDING DESIGN PRACTICE

The following section summarizes the current state of the art in the seismic design of buildings. As described below, the rigorousness of the procedure depends on the complexity of the building to be designed. Building codes give simplified, standard procedures that are appropriate for the majority of buildings. Non-typical, complex structures require a more refined procedure.

C 3.1 General Procedure

Seismic design of buildings follows the following general procedure:

1. **Perform Geotechnical Evaluation:** A subsurface investigation program is carried out to identify potential hazards related to the subsurface conditions, including the following:

   - **Liquefaction:** Some soils can liquefy during dynamic earthquake loading if it is loose, and has a high groundwater table. This causes the soil to lose its bearing capacity, resulting in a foundation failure. There is not much that can be done under these conditions, except to locate the building elsewhere. There are ground modification techniques that are possible, but their high cost is rarely justified. It is necessary, therefore, to identify whether there is potential for liquefaction at the proposed building site.

   - **Slope Instability:** Steep slopes adjacent to the building, or supporting the building could become unstable during an earthquake. These areas are checked for stability under seismic loads.

   - **Settlement:** Fill, or loose soils could densify during ground shaking, causing dramatic settlement. Soil characteristics are checked for susceptibility to this phenomenon.

2. **Develop Preliminary Structural Design:** The structural framing layout is developed along with preliminary member sizes.

3. **Calculate Seismic Base Shear:** The total horizontal earthquake force to be applied to the building is calculated using either an equivalent static force method (described in more detail below), or for more complicated structures, a modal analysis or time-history analysis method (see below). Usually elastic theory is used.

4. **Determine Distribution of Base Shear Force:** The distribution of the seismic base shear to be applied to the building's structural components is determined.

5. **Determine Internal Member Forces:** A structural analysis of the preliminary design of the building frame is performed, combining seismic loads with other loads, to determine the resulting internal member forces for design.

6. **Design Structural Members:** The structural components are designed for the internal member forces calculated as above. The preliminary sizes are modified as required to withstand the calculated internal member forces. If member sizes change significantly, it may be necessary to run another structural analysis with the new sizes. Special detailing is designed for all structures to ensure that the assumed ductility can be accomplished. Additional requirements are made for irregular structures, to avoid problems that have been observed for these structures in past earthquakes. The structural members to be addressed include the following:

- Framing members - beams, columns, and bracing.
- Diaphragms - floor slabs and roofs serving to distribute horizontal loads through diaphragm action.
- Foundations.
- Connections - beam/column connections, bracing connections, diaphragm connections, column base plates, etc.

7. **Detail for Displacement:** Seismic joints, or shake spaces, are provided between adjacent buildings or portions of buildings to prevent impact damage during shaking. Also, structural components are detailed to accommodate anticipated displacements.

8. **Check Drift:** Horizontal deflection is checked for conformance with code limitations, accounting for increased movement resulting from the ductility of the framing system. The framing may need to be redesigned to comply with the prescribed limits.

9. **Design Foundation Components:** Footings, pile caps and walls are designed for the applied loads. They must be capable of sustaining the loads, and also of transferring the loads to the surrounding soil or bedrock.

10. **Design Non-structural Components:** Parapets, cladding, and some architectural, electrical and mechanical fixtures are designed for seismic loads. Though they may not be part of the structure, they can jeopardize life safety if they collapse.

11. **Construction Inspection:** Some codes make special requirements for inspection during construction to ensure that components critical to seismic resistance are constructed properly.

C 3.2 Equivalent Static Force Method

The most common method for seismic design of buildings is the Equivalent Static Force Method. This is a simplified method that can be used for the majority of building structures. It is limited to those structures that can be considered *regular*, with a uniform distribution of mass and stiffness. In this method, seismic forces are calculated based on the seismicity of the region, importance of the facility to life safety and emergency response, the properties of the framing system and soil, and the weight of the structure. It makes assumptions as to damping, and accounts for the ductility of the framing system to dissipate energy. The equivalent static force approach used for calculation of base shears in each of the codes are summarized below.

C 3.2.1 UBC-91, and SEAOC-88

The UBC Code is used predominantly in the western states, including California, where the seismic exposure is the greatest. The UBC presents a seismic zone map of the United States. The seismic design requirements for a particular structure depend on the seismic zone, and the structural characteristics and importance of the structure. The UBC permits design using a static force procedure or one of several dynamic lateral force procedures.

The static force procedure requires only two site parameters to be determined, the seismic zone factor and the site coefficient. The remainder are determined from the characteristics of the structure to be designed. This procedure requires the application of a horizontal force at the base of the structure, the design base shear, and a certain distribution of this horizontal force up through the structure. The resulting horizontal forces are applied as equivalent static horizontal forces at each floor level of a building, and member forces and bending or torsion moments are calculated from the combination of horizontal and vertical forces.

The design base shear is determined from the following formula:

$$V = (ZIC/R_w)W$$

where:

$V =$ The total design force applied at the base of the structure

$Z =$ Seismic zone coefficient, varies from 0.075 to 0.4 according to 5 zones (from map).

$I =$ Occupancy Importance Factor varies from 1 to 1.5 for essential facilities.

$R_w =$ Response modification factor that accounts for the ductility of the type of structural framing system, varies from 4 to 12.

$W =$ Weight of structure including portion of semi-permanent load such as would occur in warehouses or storage facilities.

$C =$ A factor that essentially considers the relationship between the natural period of the site and of the structure. It is determined from

$$C = 1.25S/T^{2/3}$$

where:

$S =$ Site coefficient that depends on the soil conditions, and varies from 1.0 for rock foundation to 2.0 for soft clay.

$T =$ Structure's fundamental period (sec), calculated according to approximate formulae.

C 3.2.2 NEHRP-91, and BOCA 92 (Supplement)

The NEHRP and BOCA provisions are similar to the UBC Provisions, but there are some differences. In general, the NEHRP Provisions are more rigorous. Ground accelerations are given on contour maps, giving a higher degree of variability, over UBC's 5 zones. Maps are presented for two types of acceleration A_v, and A_a, which account for different types of buildings. The design is based on *seismic performance categories* that account for both the seismicity of the site, and of the nature of the building occupancy. Different design methods, as well as detailing requirements, are given depending on the seismic performance category. Structural design is based on ultimate strength or factored allowable stress rather than working stress.

The total seismic base shear is given by:

$$V = C_s W$$

where:

$$C_s = 1.2A_vS/RT^{2/3}, \text{ but}$$

$$C_s \leq 2.5A_a/R$$

where:

A_v = Effective peak velocity-related acceleration for the site (from contour maps). This acceleration will control for shorter structures with shorter periods (less than 5 stories high, approximately).

A_a = Effective peak acceleration for the site (from contour maps). This acceleration will control for taller structures with longer periods (Greater than 5 stories high, approximately).

S = Soil profile coefficient, varies from 1 to 2

R = Response modification factor to account for ductility of framing system, varies from 1 1/4 to 8

T = Fundamental period of building, calculated based on building height, and type of building frame system

C 3.3 Modal Analysis Method

The modal analysis method determines the different modes of vibration of a structure and combines their effect to arrive at a more accurate estimation of base shear. This method is used for more complex structures that cannot be classified as regular structures. These *irregular* structures have significant physical discontinuities in configuration or in their lateral force-resisting systems that make the equivalent static force method inappropriate. There are horizontal irregularities, summarized in Table C3-1, and vertical irregularities, summarized in Table C3-2.

To understand the modal analysis procedure, one must understand the concept of *modes* of vibration. Although structures usually have one predominant mode, or shape, of vibration, they actually have many modes. This is illustrated in Figure C3-1 for the case of a multi-story building. The fundamental, or first mode, is the deflected shape where all masses, associated with the building floors, move in the same direction, say to the right, at the same time. Higher modes occur when the masses move out of phase with each other in opposite directions.

Calculation of seismic base shear force by the modal analysis method consists generally of the following procedure:

1. **Develop Mathematical Structural Model:** Usually, a structural computer analysis model is generated for seismic analysis of the structure. The structure is modeled as a system of masses lumped at the floor levels, connected by structural framing having a specified stiffness.

2. **Determine Modal Characteristics:** The computer model is run to determine the natural modal periods and characteristic modal displacement shapes.

3. **Determine Effective Modal Gravity Load for Each Mode:** The effective modal gravity load is an effective weight that is unique to each mode. This value is calculated as a function of the mass and displacement at each story corresponding to each mode shape.

Table C3-1

Horizontal or Plan Irregularities for Buildings

Irregularity type and description	Referenced section	Seismic Performance Category Application
1. Torsional irregularity — to be considered when diaphragms are rigid in relation to the vertical structural elements which resist the lateral seismic forces. Torsional irregularity shall be considered to exist when the maximum story drift computed, including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.	1113.3.6.4.2 1113.4.3.1	D and E C, D and E
2. Re-entrant corners Plan configurations of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1113.3.6.4.2	D and E
3. Diaphragm discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1113.3.6.4.2	D and E
4. Out-of-plane vertical element offsets Discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements which resist the lateral seismic forces.	1113.3.6.4.2	D and E
5. Nonparallel systems The vertical lateral force-resisting elements are not parallel to, or are not symmetric about, the major orthogonal axes of the lateral force-resisting system.	1113.3.6.3.1	C, D and E

Table C3-2

Vertical Irregularities for Buildings

Irregularity type and description	Referenced section	Seismic Performance Category Application
1. Stiffness irregularity — soft story A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1113.3.5.3	D and E
2. Weight (mass) irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	1113.3.5.3	D and E
3. Vertical geometric irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in an adjacent story.	1113.3.5.3	D and E
4. In-plane discontinuity in vertical lateral force-resisting elements An in-plane offset of the lateral force-resisting elements greater than the length of those elements.	1113.3.6.4.2	D and E
5. Discontinuity in capacity — weak story A weak story is one in which the story lateral strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	1113.3.6.2.4	B, C, D and E

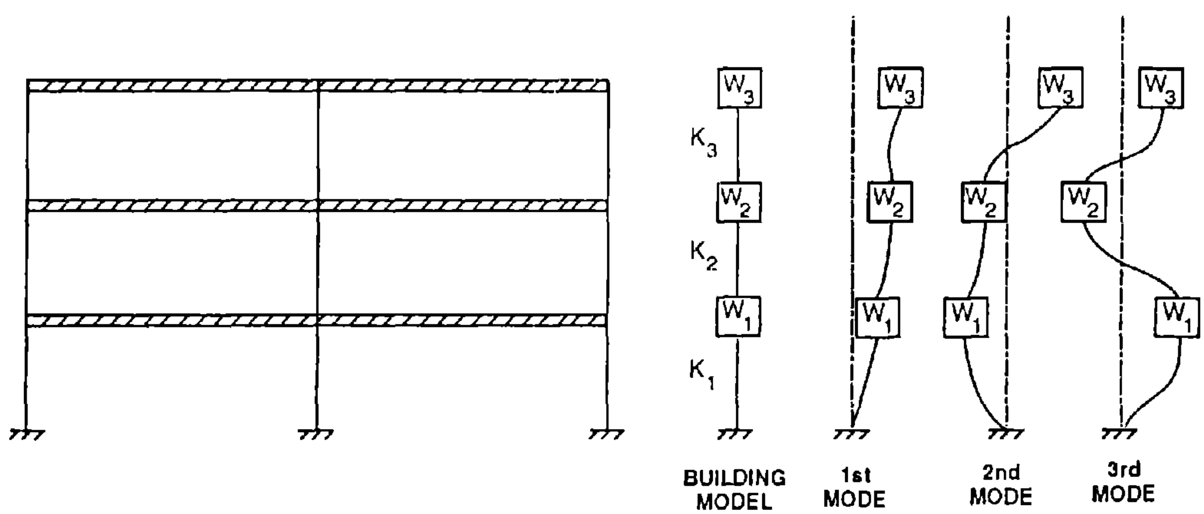


Figure C3-1 - Modes of Vibration for a Three Story Building.

4. **Determine Modal Seismic Design Acceleration for Each Mode:** Some codes present accelerations in the form of a set of *normalized response spectra curves* for different soil types, as a function of structural period. Other codes present accelerations as a series of formulae for various ranges of period, with variables consisting of period, ground acceleration, soil coefficient, and ductility factor.

   Using normalized response spectra, acceleration is determined by reading values from the appropriate curve, given the modal period, and applying an appropriate ductility factor. Otherwise, acceleration is calculated using code formulae and the appropriate values of modal period, ground acceleration, soil coefficient, and ductility factor.

   The response spectrum in the building codes are based on structural and damping characteristics of most ordinary building structures. For other types of structures it may be appropriate to develop a spectrum more suited to the characteristics of the structure (rigidity, complexity, damping coefficient, etc.). It is also sometimes appropriate to modify the spectrum for site specific geologic and seismic conditions.

5. **Determine Base Shear for Each Mode:** The base shear is calculated as the product of the modal acceleration and the modal gravity load.

6. **Combine Base Shears from All Modes:** The total base shear is calculated by combining the base shears for all modes using the square root sum of the squares method.

7. **Check Code Limits for Base Shear:** Some codes give lower limits to the value of base shear that may be calculated using the modal analysis procedure. This limit is calculated, and the total base shear is modified appropriately.

8. **Design and Detail Structural Members:** Members are designed and detailed as described under the equivalent static force procedure.

C 3.4 Time-History Analysis Method

For very complex structures, or those associated with high risk, such as nuclear power plants, an even more detailed analysis may be warranted. This method is based on the development of a ground motion time-history, or record of ground movement over time, that is anticipated at the site. This time-history is developed considering soil and geological formations, as well as seismicity of the area. The ground motion representation must have no greater than a 10 percent probability of being exceeded in fifty years.

With a time-history analysis, a computer is used to model the complete structure, and its dynamic response is determined through each increment of time when the base is subjected to the specified ground motion time-history. Again, an appropriate combination of the responses at the various vibration modes is required.

This type of analysis is very complicated, time consuming and costly. It is not warranted for most transportation facilities.

C 3.5 Seismic Detailing of Buildings

Detailing of structural components refers to the design and configuration of connections, reinforcing steel, and other *parts* of structural members. It involves structural design on a *micro* scale, whereas structural framing design (the sizing and layout of beams and columns) is structural design on a *macro* scale.

Proper detailing is critical to the successful performance of a structure during an earthquake. Well designed detailing will ensure that ductility can be achieved, and that the structure has the ability to withstand the ductile movement necessary to dissipate earthquake energy. Until recently building codes did not have requirements for detailing. This has been one of the latest areas of development in seismic design theory.

Current editions of building codes have specific requirements for detailing that have been established from theory, and also from observations of performance of certain types of structures during recent earthquakes.

Requirements vary according to the unique properties of the various building materials used.

Structural Steel: Structural steel is a very ductile material, and generally performs well in earthquakes. Provisions for extra capacity are necessary, however, for connections of braced frames. Previous earthquakes have shown them to be susceptible to failure. Moment connections should also be strengthened in moment resisting frames.

Concrete: Reinforced concrete, if not detailed properly is a brittle material, and performs poorly in earthquakes. With good detailing, however, it can be very ductile and has proven to resist seismic forces well. There are many detailing requirements, but they generally involve confinement of concrete with additional stirrups in beams, ties in columns, and development of reinforcement at beam and column joints.

Masonry: Unreinforced masonry has been notorious for poor seismic performance. It is very brittle with almost no capacity for ductility. Codes prohibit the use of unreinforced masonry in new buildings, and require that masonry structures have adequate reinforcing to resist seismic loads. Additionally, minimum amounts of reinforcement are specified to ensure ductility.

Timber: Timber structures have performed well in earthquakes, partly because of their light weight, and partly because their connections which slip under high loads create high values of damping. Special requirements are specified, however, for bracing, diaphragms, shear panels and positive connections for columns and other members.

C 3.6 Design of Structures With Base Isolation

Base isolation is sometimes used in areas of very high seismicity, or where building damage from earthquakes cannot be tolerated. This is accomplished with the use of a flexible connection at the structure's base that allows the ground movement to occur with little transfer of movement or force to the structure above.

The most common type of isolation joint is constructed of rubber with an inner core of lead. The lead is designed to be strong enough to resist wind forces, but flexible enough to offer very little resistance to dynamic seismic forces. The lead is also very ductile, and helps dissipate seismic energy.

C 4.0 CURRENT U.S. SEISMIC BRIDGE DESIGN PRACTICE

The Seismic Design Guidelines for Highway Bridges (AASHTO) was originally prepared by the Applied Technology Council in California and encompassed principles similar to those of the UBC, though more specific to highway bridges, piers and abutments. Similar to building codes, the AASHTO specifications were developed with the intention that:

- Small to moderate earthquakes would be resisted within the elastic range without significant damage.
- Large earthquakes would cause some damage, but not cause collapse of the bridge.

But, AASHTO provisions also are designed to:

- Force damage to occur in visible locations of the structure (above the ground surface) so that distress can be easily detected and repaired following an earthquake.

C 4.1 General Procedure

Seismic design of bridges according to AASHTO Specifications follows the following general procedure:

1. **Perform Geotechnical Investigation:** As for buildings, a geotechnical investigation is performed to identify potential hazards related to subsurface conditions, including:
    - **Liquefaction:** Loose granular soils with high water table are identified and checked for the potential for liquefaction.
    - **Slope Instability:** Steep slopes adjacent to the bridge are common, due to the adjacent grade changes. These areas are checked for stability under seismic loads.
    - **Settlement:** Fill, or loose soils are checked for potential settlement.
    - **Increase in Lateral Earth Pressure:** The increase in lateral loading on foundation elements due to earthquake ground motion is evaluated.
2. **Determine Applicability of Standards:** The specifications are for the seismic design of new bridges, and are applicable to conventional steel and concrete girder and box girder type bridges having span lengths not exceeding 500 feet. Seismic design is not required for buried culverts, and minimal requirements are made for single span bridges
3. **Develop Preliminary Design:** Estimate member sizes, and bridge geometry.
4. **Determine Acceleration Coefficient (A):** This coefficient is given in seismic zone maps (see Figures C4-1 and C4-2).
5. **Determine Importance Classification (IC):** An importance classification is given, with essential bridges having an IC of I, other bridges, II.
6. **Determine Seismic Performance Category (SPC):** Four classifications, A through D, are specified, similar to BOCA's five classifications. They are based on the acceleration coefficient and importance classification, as shown in Table C4-1.

Table C4-1

Seismic Performance Category

Acceleration Coefficient	**Importance Classification**	
A	**I**	**II**
$A \leq 0.09$	A	A
$0.09 < A \leq 0.19$	B	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C

Table C4-2

Analysis Procedure

Seismic Performance Category	**Regular Bridges with 2 or More Spans**	**Irregular Bridges with 2 or More Spans**
A	--	--
B	1	1
C	1	2
D	1	2

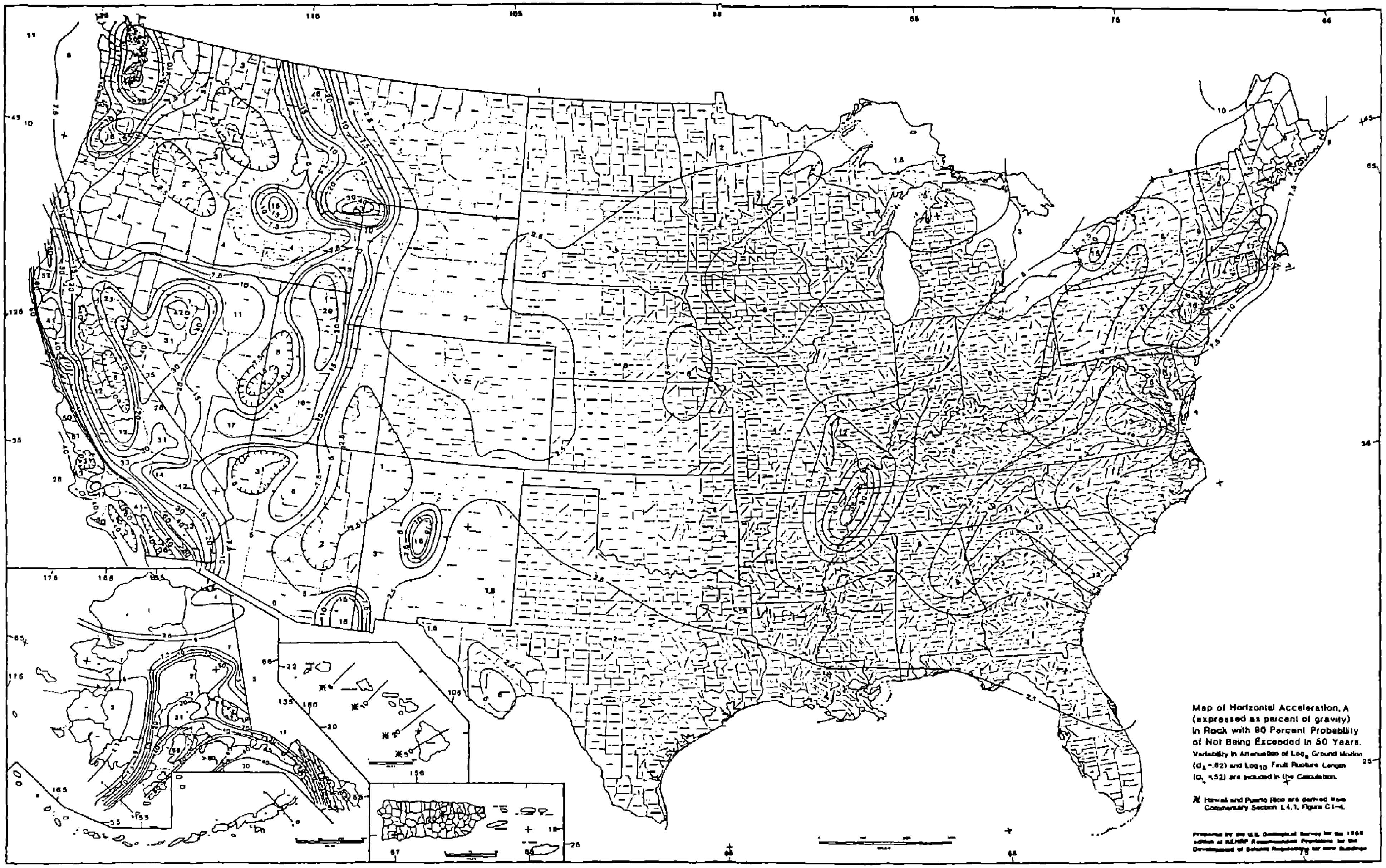


Figure C4-1 - AASHTO Seismic Zone Map

7. **Determine Site Coefficient (S):** A coefficient varying from 1 to 1.5 is specified according to soil and site conditions.

8. **Determine Response Modification Factor (R):** Values of R, ranging from 2 to 5 are given for different substructure types for superstructure and substructure design. A different set of R values, varying from 0.8 to 1.0, are given for connection design.

9. **Determine Analysis Procedure:** Different procedures are specified for different bridge types and different SPC's.

Single Span Bridges:	Connections between the bridge span and abutments are designed for horizontal forces calculated as the gravity reaction multiplied by the Acceleration Coefficient for the site.
SPC A:	Connections between the bridge superstructure and substructure are designed for horizontal forces, in the restrained direction, calculated as the dead load reaction multiplied by a factor of 0.2.
SPC B through D:	Two analysis procedures are given for these bridges: Procedure 1), the Single Mode Spectral Analysis Method, and Procedure 2), the Multi-Mode Spectral Analysis Method. The method to be used depends on the number of spans, the geometrical complexity and the Seismic Performance Category as shown in Table C4-2. The two analysis methods are described in detail in following sections.

10. **Determine Design Forces:** Design forces are calculated in accordance with appropriate analysis procedure, and combined with the static loads. Provisions are made for the application of the earthquake loads in two directions simultaneously.

11. **Design Structural Members:** The structural components are designed for the internal member forces calculated as above. Special detailing is designed for all structures to ensure that the assumed ductility can be accomplished.

12. **Determine Design Displacements:** Displacements for SPC B through D are calculated in accordance with the procedures specified as above. Minimum support lengths of bearing seats are given as a function of the length of bridge deck to the adjacent expansion joint, and height of columns or piers supporting the bridge superstructure. Components may have to be resized to comply with these limitations.

C 4.2 Procedure 1 - Single Mode Spectral Analysis Method

This method is similar to the equivalent static force approach described for buildings. It is appropriate for use in SPC B, and for *regular* bridges in SPC C and D. A regular bridge is defined by AASHTO as one that "has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports". This method, like the equivalent static force approach for buildings accounts for the seismicity of the region, importance of the facility to life safety and emergency response, the properties of the framing system and soil, and the weight of the structure. It makes assumptions as to damping, and accounts for the ductility of the framing system to dissipate energy. Unlike the building codes, the single mode spectral analysis method develops a force that varies along the bridge superstructure's length. The method is summarized as follows.

The equivalent static earthquake loading, $p_e(x)$, is determined from the following formula:

$$p_e(x) = (\beta C_s/\gamma)w(x)v_s(x)$$

where:

x = Distance measured along the length of the bridge

β = Integration over the length of the bridge of bridge dead load, and displacement resulting from unit load on bridge, given by the following formula:

$$\beta = \int w(x)v_s(x)dx$$

C_s = Elastic seismic response coefficient, given by the following formula:

$$C_s = 1.2AS/T^{2/3}, \text{ but no more than } 2.5A$$

γ = Integration over the length of the bridge of bridge dead load and square of displacement resulting from unit load, given by the following formula:

$$\gamma = \int w(x)v_s(x)^2dx$$

w(x) = Bridge dead load per unit length

$v_s(x)$ = Displacement of bridge superstructure resulting from p_o

p_o = Hypothetical load on bridge per unit length equal to 1

A = Acceleration coefficient

S = Soil coefficient

T = The fundamental period of the bridge, calculated based on an integration over the length of the bridge of its mass and stiffness, given by the following formula:

$$T = 2\pi\sqrt{\gamma/(p_o g\alpha)}$$

a = Integration over the length of the bridge of displacement resulting from unit load on bridge, given by the following formula:

$$a = \int v_s(x)dx$$

g = Acceleration of gravity

The force $p_e(x)$ is calculated in two directions - parallel, and perpendicular to the bridge. This force is applied to the bridge, and the internal member forces and displacement of the structure are calculated.

C 4.3 <u>Procedure 2 - Multi-Mode Spectral Method</u>

This procedure is intended for SPC C and D bridges that are *irregular*, those with abrupt or unusual changes in mass, stiffness or geometry along its span and/or with large differences in these parameters between adjacent supports. This method is similar to the modal analysis method described for buildings. The use of a space frame linear dynamic analysis computer program is recommended for this analysis.

Basically, with this method, the formulae for Procedure 1 are used for each of the structure's modes and they are combined using square root sum of the squares method. The elastic seismic response coefficient, however, is now given by the following formulae:

$$C_{sm} = 1.2AS/T_m^{2/3,} \text{ but}$$

$$C_{sm} \leq 2.5A$$

Some exceptions and variations to the above formulae are given in the AASHTO specifications, that depend on soil conditions, acceleration coefficients, and the mode under consideration.

C 4.4 Design and Detailing Requirements

Like in the building codes, there are special detailing requirements provided in AASHTO to ensure that ductility can be achieved, and that the structure has the ability to withstand the ductile movement necessary to dissipate earthquake energy. These requirements are similar to those of buildings, but some are unique to bridges. The requirements vary according to SPC category.

Piers and Columns: Special procedures are specified for the design of piers, columns, and their connections for SPC C and D, to ensure that their total plastic moment capacity can be achieved without shear failure of the column or failure of the joints.

Linkage: Requirements are specified for linking adjacent sections of the superstructure across expansion joints.

Hold Down Supports: Provisions are made for attachment of the superstructure to substructure for certain structures.

Minimum Bearing Support Lengths: Minimum lengths are given for bearing supports, so that girders will not lose support during seismic movement.

Piles: There are many detailing requirements for piles including minimum embedment and anchorage and minimum reinforcing for concrete piles. The design also must account for bending in the upper region of the piles.

Abutment Design: Lateral seismic earth pressure forces are estimated by an inverse triangular load distribution, based on the ground acceleration at the site, the abutment height and various soil parameters.

Approach Slabs: Slabs connected on the back face of the abutment and extending away from the bridge in the direction of the approach roadway are required for SPC D bridges. These slabs are intended to provide structural support between approach fills and abutments.

Reinforced Concrete Structures: Detailing requirements for concrete structures vary with SPC classification. They are similar to the requirements for buildings. Included are requirements for minimum main column reinforcement, column shear and transverse reinforcement, transverse reinforcement for confinement at the top and bottom of columns and pile bents, reinforcing steel splices, minimum shear reinforcement in piers, column connections, and construction joints in piers and columns.

C 5.0 CURRENT U.S. SEISMIC UNDERGROUND STRUCTURE DESIGN PRACTICE

Subsurface facilities include tunnels, culverts, and underground structures and chambers. The predominant type of subsurface structure related to transportation facilities is the tunnel.

Seismic design theory for subsurface facilities is by far the least advanced of all structure types. There is little if any direction given by any of the codes on how to determine seismic loads, or how to approach seismic design. It has been common practice for tunnel designers to develop their own seismic criteria, based on past experience and soil mechanics theory.

C5.1 Types of Underground Seismic Motion

C 5.1.1 Ground Shaking

The magnitude of ground shaking at a given site depends on its distance from a capable fault, the magnitude of earthquake expected at that fault, and the detailed properties and depth of soil overlying bedrock. As described in Appendix A, ground acceleration maps have been developed for all regions of the U.S. These general categorizations are certainly valuable, but they can only be used as guides. The guides may be generally adequate for a relatively simple, isolated structure. For major facilities such as the L. A. METRO and North Outfall Relief Sewer (NORS), however, special studies are necessary and specific values for ground shaking (normally expressed as peak ground acceleration, velocity, and displacement) must be defined. In many cases, it is also necessary to characterize the vibratory frequency characteristics of the design levels of ground shaking. Often these are included in tripartite plots called earthquake spectra. Normally the typical spectra reflect the response of a structure to the ground shaking. Spectra are normally created for discrete values of assumed damping in the system. For underground structures, spectra are often more useful only in design of critical equipment enclosed in the facility.

Some work had been done prior to 1971 indicating that site properties affect the ground shaking (for example, Jennings, 1971). However. the Sylmar Earthquake of February 9, 1971, clearly showed that partial resonance and details of soil/structure interaction can result in disastrous magnifications of seismic motions. An often cited example is the collapse during this 1971 earthquake of the I-210/I-5 interchange north of Los Angeles. Soil-structure interaction is also important in underground structures. Fortunately it is the nature of underground structures to move fully or partially with the seismic motion of surrounding ground: structural resonance is usually not a concern. Nevertheless, amplification of motion often occurs in the soil overburden and, as a result, the motion reaching the structure will often be much higher than that in the underlying bedrock.

C 5.1.2 "Discrete" Displacements on Faults

Whenever a buried structure crosses a fault, there is potential for slippage to occur on the fault as a result of an earthquake. Both the magnitude and recurrence interval are important. Often in cities and suburban areas, development has destroyed all surface evidence of the exact location of the fault. Additionally, development often precludes the use of trenches to uncover the faults and define the history of motion on the fault. Fault displacement being evident in only older deposits and absent in more recent ones is an important factor in dating fault recurrence intervals and the magnitude of slip.

Nevertheless, it may be possible from old records, such as geologic notes on former projects (for NORS, notes from oil field exploration made possible the definition of the surface expression of the Newport-Inglewood Fault) and aerial photography from eras prior to development, to define fault locations accurately. Also, new trenching may be possible in large parking lots, alleys or less trafficked suburban streets. On the other hand, mapping the face in a shield-driven, or tunnel boring machine - (TBM) constructed, tunnel is often not very satisfactory. The face is so restricted that location of the fault is difficult and, more importantly, design modification at the time of construction may be impossible; it certainly would be expensive.

Finding the fault early and defining potential displacement on it are of paramount importance in proper tunnel design. Practical schemes for fault location may require the use of special, often expensive, exploration procedures. However, being able to minimize the need for special construction procedures will often repay, by several times, the cost of the effort to find the fault and gather definitive data on the magnitude and recurrence interval of the fault displacement. This was true for NORS, where the final design required only 400 ft (120 m) of lining system using special backpacking instead of the one mile (1.6 Km) first considered.

Prediction of fault displacement is done using procedures such as those developed by Bonilla (1984). These procedures relate fault displacement to the length of the rupture created by the earthquake in the epicentral area. Typical historical values on major faults vary from a few centimeters to a couple of meters. In the last case, the potential impact on design is obvious!

Although designs for fault displacement, normally for conservatism, assume that the displacement occurs on a single plane, the actual ruptures tend very often to occur along several sub-parallel branches. This tends to mitigate the distress imposed on the buried structure.

C 5.1.3 Lateral Spreading and Liquefaction

Conditions leading to lateral spreading and liquefaction were mentioned in Appendix A. Tunnels and other underground structures are particularly vulnerable to liquefaction. Flotation is often a concern. Liquefaction can be dealt with most effectively by avoiding areas subject to liquefaction in specifying the alignment, placing the structures deeper to take advantage of higher natural confining stresses, using permanent dewatering, or specifying ground consolidation or ground modification. Conditions leading to potential liquefaction must, therefore, be specifically evaluated for underground structures, but often their effects can be avoided or mitigated.

C 5.2 Design Strategies to Mitigate Risk

C 5.2.1 Introduction

Although underground structures have proved much less vulnerable to earthquakes than surface structures, there still is a significant potential for damage to buried structures in strong-motion earthquakes. The actual risk must be assessed on the basis of both seismological and geotechnical evaluation of the site. For this assessment, seismological information includes:

- historical data on earthquake recurrence; magnitudes and associated parameters of ground shaking;
- proximity to faults;
- historical evidence of slippage on the faults and magnitude of actual offsets with their recurrence interval; and required geotechnical information includes:
  - depth to and nature of underlying bedrock;
  - stratigraphic section and properties of the individual components of soil/rock in the overburden;
  - water table, presence of perched water and typical degrees of saturation of the soil;

- geophysical data, especially shear wave seismic velocity in each major segment of the soil/rock horizon.

These data must be carefully evaluated and an appropriate philosophy of design established. In most cases, it is impractical to design a structure to survive the conditions which might develop in the most severe earthquake which statistically might occur during the useful life of the project. Like bridge and building seismic design, a proper tunnel design should resist minor earthquakes without damage, and should sustain major earthquakes without collapse, but with some damage.

C 5.2.2 Concept of Relative Stiffness

In a hand lettered note, Newmark (1959) first proposed the characteristics of the "perfect" tunnel lining, in presenting the design concept for structures in a program called Event HARD HAT. The basic idea was to design a relatively thin lining to have the same load-deformation characteristics as the cylinder of soil or rock displaced by the lining. That concept was, of course, carried forward and presented in detail in Peck, Hendron, and Mohraz (1972). As pointed out still later by Monsees (Merrin, et al, 1985), there are three interesting possibilities:

1. The characteristics of the structure are exactly matched to the free field deformation (load deformation characteristics of the surrounding soil).

2. The structure is more flexible than the surrounding soil.

3. The structure is less flexible then the surrounding soil.

For a given applied, generally-prevailing displacement of the soil, in the first two cases the structure will deform with the soil; in the third case, the structure will deform less than the soil. It is, therefore, always proper or conservative to assume that the structure deforms with the soil. That concept is extremely important, and it allows us to straight-forwardly design any buried structure. We propose violating that concept of design only when it is practical to over-excavate a region of a tunnel in the area where the tunnel crosses a known fault at a known or relatively limited uncertainly in location. In this case, it is possible to surround the structure with highly deformable backpacking materials such that discrete motion on the fault can occur, but both the load and deformation on the structure are controlled and limited.

It must be noted that situations may exist where this design approach might be overly conservative for structures falling in the third case. Obviously, a very stiff structure buried in soft clay may not deform as much as that clay. The research topics discussed at the end of this paper include this case as one requiring additional development. The first step in this direction is being taken by Wang (1992) under a fellowship sponsored by Parsons Brinckerhoff Quade and Douglas, Inc.

C 5.2.3 Shaking Motion

Shaking motion can produce axial strains and flexure in any linear underground structure, such as a sewer tunnel. The shaking motion is made up generally of a complex combination of waves. Methods are available for calculating induced strains, or deflections resulting from shaking motions, given certain characteristics of the design earthquake, and tunnel geometry.

C 5.2.4 Ovalling/Racking

A shear wave traveling along or oblique to a tunnel tends to rack it; that is, it tends to cause a circular section to take an oval shape. If it is a rectangular structure, such as a subway station, it will induce conditions akin to classical sidesway. In either case, it is most important to realize that the embedded structure does not deform entirely freely. If it is very stiff, it may try to, or even separate, from the surrounding soil; before it can collapse, however, it generally will contact the soil and the soil will then mobilize resistance and impede further deformation. As a consequence, we postulate only the mechanisms shown in Figure C5-1 as possible mechanisms for collapse in a rectangular structure. The effect of this racking can be estimated given certain characteristics of the earthquake, tunnel geometry, and material properties of the soil and tunnel.

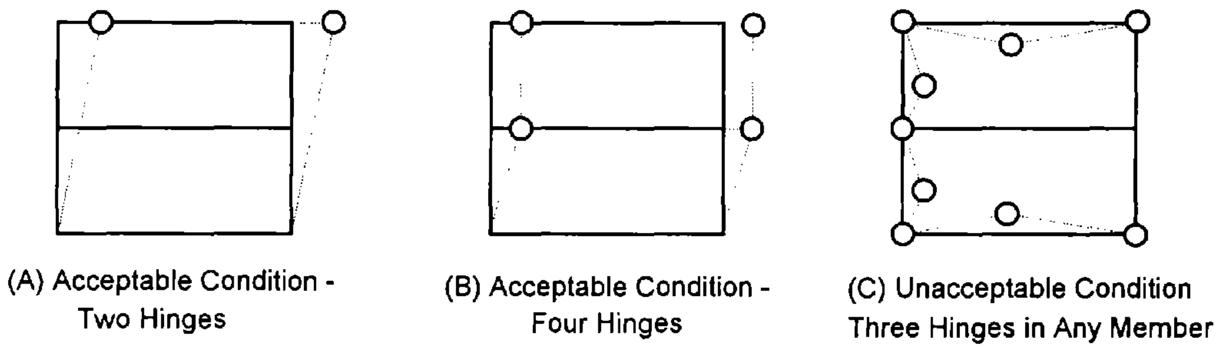


Figure C5-1 - Structural Failure Mechanisms

Another way to account for seismic impacts on an underground structure, is to expose it to incremental dynamic earth pressures. Even this approach is difficult because of the limited ability to correctly model the dynamic pressures. One method which has been used involves the Mononobe - Okabe method, which has been extensively used in Japan. This technique, however, is known to have limitations and can result in unconservative design. This approach can be referred to as the "pressure" method.

If the model used for analysis were perfect, the displacement approach and the pressure approach would achieve the same result. Since this is not the case, it is advisable to design the structure checking both methods. It should be noted that two are not additive, and the design should be based on the worse or controlling case.

The methodology for underground structure design should be as follows:

a. Determine the free-field soil deformation. Generally a simple computer program (such as Shake) can be used to compute the ground shear deformation as a function of the depth of soil. A design earthquake acceleration time history is input from the bottom boundary (e.g. bedrock).

b. Based on the tunnel's location, determine the differential free-field deformation between the top and the bottom of the tunnel.

c. With the given structure geometry and properties, apply a horizontal load such that the structure distorts laterally with that magnitude and determine internal forces in the structural members.

d. Combine the effect of the seismic internal member forces from step c with those obtained from static design, by summation.

e. Determine the effect of seismic soil loads using the Mononobe-Okabe method. This is done by calculating the seismic loads, and running a structural analysis of the tunnel carrying these loads, to determine internal member loads.

f. Combine the effect of the seismic internal member forces from step e with those obtained from static design, by summation.

g. Check the combined internal member forces against the maximum allowable and revise as appropriate for member design.

C 5.2.5 Loosening Loads

Terzaghi with his trap door analogy (1943) addressed the concept of loosening load under "a yielding trap door." He also addressed a somewhat similar phenomenon in rock tunnels 946). In a shield driven tunnel, these loosening loads, as we've chosen to call them, result primarily from the clearance between the tail of the shield and the excavation, and by the care exercised in installing, expanding, and/or grouting the tunnel lining. Good construction practice will reduce these loads generally, but probably never eliminate them. Less attention to detail will increase them. The loosening loads will always be present to some degree; it is hard to conceive of any natural mechanism which will reduce them once they develop. As a result, the effects of the loosening load must be superimposed on other effects. Such loads are usually estimated by usually combining judgment and limited empirical data.

C 5.2.6 Fault Crossings

Location of fault crossings and design for mitigation of their effects are especially challenging. Nyman (1983) indicates that faults should always be crossed, if possible, such that the conduit is placed in tension. However, many structures can not be freely located normal to the fault as may be the case of pipelines running across country (the principal subject of Nyman). For example, seismic analysis of the NORS crossing showed that the structure was in compression. The realignment of the LA METRO is still being studied; thus, the angles of any fault relative to the alignment for the subway are not yet fully known.

In NORS it appeared likely that a single tunnel size would be driven for most of the alignment. Hydraulic conditions dictated a smaller conduit in the region of the potential fault crossing. It was, therefore, practical to consider a smaller conduit in a larger excavation in the early stages of design. When it became clear that the strands of the Newport-Inglewood fault of concern were only near the upstream terminus of the sewer where the conduit was shallow, it may have been practical to construct a specially designed tunnel by cut and cover techniques. However, since the needed length of possible special design was quite limited, it was potentially practical to enlarge this limited reach of tunnel. For these reasons it was decided to consider an articulated (segmented longitudinally) concrete lining surrounded by cast-in-place deformable cellular concrete which could literally accommodate the design value of 8 in. (20 cm) of lateral displacement and limit the compressive load on the conduit to about 20 psi (ISOKA). This proved practical and cost effective.

For LA METRO, data are only now being developed. For the original alignment, the lateral displacements could have been as much as 5 ft (1.5 m) on one fault and 6.6 ft (2 m) on another. A tentative plan was to use a steel lining over the entire reaches where these faults were suspected to lie. Special articulated (longitudinally and circumferential) steel linings were being considered. As an alternative, corrugated metal might have been used in some special situations. Segmented concrete did not appear to be appropriate due to the large displacements, the relative brittleness of concrete, and the need to prevent leakage of methane gas into the tunnel from surrounding deposits.

Due to the need for re-alignment, a new effort will probably be made to locate the faults. Further effort will likely also be expended in defining the design values for displacements on the faults when they are found. All options for design and construction will then be re-examined to develop the appropriate concept for the fault crossing(s).

C 6.0 CURRENT U.S. SEISMIC RETROFIT PRACTICE

C 6.1 Background

The term "seismic retrofit" is used to describe the construction of improvements to improve the performance of existing structures during an earthquake such that their earthquake vulnerability can be reduced to an acceptable level.

Many structures currently in use as part of transportation facilities were designed and constructed prior to the development of modern seismic design techniques. There are, therefore many facilities that are vulnerable to serious earthquake damage. Seismic retrofits are accomplished in order to restore them to an acceptable level of safety based on newly developed technology.

Seismic retrofitting, like seismic design, is a relatively new field. The evolution of seismic retrofitting principles and techniques is in its infancy. Much of the advancement of this technology has taken place in California, particularly in the area of bridge retrofitting.

This field usually receives little attention. The benefits of retrofitting are not glamorous, and politically it is difficult to obtain funding for the accomplishment of retrofits. It isn't until a major earthquake hits that the public appreciates the importance of upgrading vulnerable structures. This is when most funding is obtained and most work initiated.

Some of the most extensive retrofit work has been done in the area of highway bridges. Following the San Fernando earthquake of 1971, California began a program of retrofitting highway bridges, including those which had not been damaged during this earthquake. The emphasis of the retrofitting program was the installation of cable restrainers at expansion joints to prevent beams from falling off of their supports. The Whittier Narrows earthquake of 1987 was responsible for the institution of a second retrofitting program which was aimed at strengthening columns of single-column bents. The Loma Prieta earthquake of 1989 caused a renewed concern with retrofitting of highway bridges and resulted in a review of all California highway bridges, and an assignment of priorities. The Federal Highway Administration has developed a Seismic Design and Retrofit Manual, and Seismic Retrofitting Guidelines which are valuable resources for design of bridge retrofitting.

The Loma Prieta earthquake also resulted in the retrofit design of a number of wharves at the Port of Oakland. In this case the retrofit was related to repair of damaged wharves. One of the wharves at the Port, built in 1980, had been designed to resist earthquake loads with ductile moment-resisting frames, and was undamaged by the Loma Prieta earthquake.

C 6.2 Procedures

The problem is far-reaching. Most structures currently in use were constructed under old building codes with insufficient seismic provisions, and they are vulnerable to serious earthquake damage. No region of the country is immune. Probably the biggest problem facing the transportation facilities manager, and the first step in a retrofit program, is the prioritization of required retrofit work, according to vulnerability, risks and cost/benefits analyses.

California's statewide seismic retrofit effort shows many of the considerations required for seismic retrofit programs. This retrofitting has been legislated to a high priority for the State Department of Transportation, CALTRANS. With numerous bridges to review, it was necessary to prioritize the bridges according to their need for retrofit. Bridges designed prior to 1971 with non-ductile details were at risk. Single-column bent bridges have little redundancy so they were at risk. Bridges adjacent or crossing major faults were at risk. Bridges founded on deep soft soils or sands with liquefaction potential were at risk. Also, it is quite possible that during the next ten years, new developments may show that bridges currently being designed or retrofitted are still at risk. These are the types of issues to be considered in the course of the retrofit program.

C 6.2.1 Prioritization

The criteria for prioritization for seismic retrofitting for any transportation facility should include the following:

- *Vulnerability.* The methods discussed in Appendix B should be utilized to determine seismic vulnerability of the facilities for comparison and ranking.

- *Importance of structure to the transportation network.* If closing a facility due to earthquake damage will disable a key transportation system, then it qualifies as an important structure. Also if the facility is critical to emergency preparedness or post-earthquake recovery, it is an important structure. A few examples are major water crossings like the San Francisco-Oakland Bay Bridge, key interchanges, and airports.

- *Cost for repair/replacement versus cost for seismic retrofit.* The cost of replacing or repairing a structure after an earthquake is usually much more than the cost of constructing a seismic retrofit modification prior to an earthquake. Structures with a high ratio of repair/replacement cost compared to the seismic retrofit cost generally should receive a high priority for retrofit.

- *Adjacency of earthquake faults.* Facilities located near major fault systems are likely to see higher accelerations or larger displacements than those located at greater distances. These facilities are likely highly vulnerable to major damage and should receive special attention.

- *Site soil conditions.* Structures founded on unconsolidated fills, or deep soft soils will experience very different excitation than those founded on rock. If a structure is founded on soils susceptible to major liquefaction, major damage or failure is often likely; alternative sites or routes should be evaluated since retrofitting the structure will have little effect on a liquefaction failure.

- *Redundancy of structural system.* Redundancy in the structural system should be considered. For example multiple-column piers can absorb more damage than single-column piers, and continuous bridges are not as vulnerable to displacements as simply supported bridges. Structures with little redundancy are more vulnerable and should receive higher priority.

C 6.2.2 Strategy

Once a structure has been selected for retrofitting, a *retrofit strategy* must be developed. A retrofit strategy is a plan to provide adequate ductility, strength, and stiffness to a structure. The strategy must consider the complete structure in addition to each of its elements. Some strategies may require only increased ductility, while other strategies may require more strength and stiffness. In some cases a strategy may use lower strength or stiffness to force another structural component to absorb the majority of the earthquake energy, thereby protecting the other members from damage.

One concept important for the understanding of retrofitting is the d*emand-to-capacity ratio.* This is the ratio of dynamic demands on a member to that member's capacity, or the ratio of the seismic force that will be imposed on a member to the maximum force that the member can safely carry. Since the need to retrofit a structural component depends on the demand-to-capacity (D/C) ratio, a deficient structural component requires a seismic retrofit method that reduces the demand-to-capacity ratio.

Either capacity must be increased or demand reduced. Capacity can be increased by strengthening the structural component. Alternatively, demand can be reduced by changing the structure's vibratory characteristics so as to subject the structure to lower earthquake forces (but necessarily accompanied by larger displacements).

Usually, the retrofit strategy involves reducing the demand-to-capacity ratio, but sometimes it can be a matter of prescribing a new *load path*. The load path is the path the seismic loads take as they "travel" from the center of mass of the structure, down to the foundation support. For short bridges like highway crossings, for example, an abutment retrofit may be the best solution to deficient pier columns. By strengthening the abutments, and with proper detailing they become a stiffer element than the pier columns, and the load path will be more through the abutments than through the columns. The demand-to-capacity ratio will be higher because the structure will generally be stiffer, but the seismic resistance problem will be solved.

If a strategy is developed that requires any change in mass or structural stiffness or behavior, the retrofits should be modeled and new demand-to-capacity ratios tabulated. New structural stiffness, damping, joint strengthening, and beam or column strengthening can make significant differences in dynamic response and force distribution.

C 6.2.3 Summary

The procedure for development of a Retrofit Design Program is summarized below:

1. Select structure for retrofitting. ATC-6-2 gives a method and criteria for identification of those structures where retrofitting is advisable.
2. Obtain information on structure design and construction, as well as foundation and site soil conditions.
3. Perform field reconnaissance of structure and site vicinity.
4. Determine liquefaction potential and dynamic settlements of site soils.
5. Determine capacity of existing structure.
6. Determine dynamic demands on structure from design earthquake.
7. Compare dynamic demand with structure capacity.
8. Determine areas of inadequate capacity.
9. Establish a retrofit strategy.
10. Design the retrofit.
11. Construct the retrofit.

Table C7-1 summarizes some seismic deficiencies, along with alternatives for remedial retrofits and comparative costs.

C 7.0 ECONOMIC IMPLICATIONS

An important consideration for the facility manager is the cost of incorporating good seismic design practice into new construction. This is difficult to determine, and there does not appear to be one universal solution which applies to all situations. However, some general observations can be made.

The design philosophy used for a facility clearly has economic implications. A facility could be designed such that it would suffer only minor damage in a major earthquake. However, the cost increase would be considerable. Conversely, it could be designed at a lesser cost, to protect life safety during an earthquake, but with some damage. Thus, in the development of a design philosophy, clearly stated objectives are important to ensure that available resources are spent prudently.

C 7.1 <u>New Structures</u>

In general, the cost of a structure is related to the amount of material used to construct it. For conventional loads, the dimensions of the structure are proportioned relative to that load. Thus, the amount of material and the cost of the structure are generally related to the magnitude of the loads. It is difficult, however, to clearly specify the individual costs contributed by each load because design is based on the controlling combination of all loads.

In the case of earthquake provisions, it is even more difficult to generalize regarding the specific contribution to the cost of a structure. The earthquake loading may or may not govern the design. In California, it may be expected that the specific choice of structural details is controlled by seismic considerations. In other areas this may not be the case. Further, as has been discussed in the preceding sections, an earthquake does not simply contribute a load to the structure (like live or wind load), requiring additional strength, but it makes a demand for displacement requiring flexibility and ductility. Indeed, increasing the size and thus the rigidity of structural members can increase the amount of the earthquake load they attract and make them less able to meet the displacement demands. This fundamental difference is important in defining costs associated with seismic provisions. Also, just as there are differences in the state-of-the-art treatment of aboveground and underground structures, it is expected that the seismic ramifications of cost will differ for each.

C 7.1.1 Buildings

There is a slight increase in building construction cost resulting from incorporating seismic resistance.

Studies were conducted by the Building Seismic Safety Council (BSSC) in 1983-84 to determine the additional costs associated with constructing seismically resistant buildings. This study was intended to determine the affect on construction cost of an amended version of the Applied Technology Council (ATC) Tentative Provisions for the Development of Seismic Regulations for Buildings" (the precursor to the NEHRP Provisions). In this study, 52 trial building designs were developed in different regions of the country for two cases: 1.) without seismic provisions, and 2.) with the tentative seismic provisions. The construction costs of each case were compared to determine the effect of seismic design on first cost.

The results give an indication of the approximate effect on construction cost of incorporation of the tentative seismic provisions. Similar results would be expected for the 1991 NEHRP provisions because modifications made to the earlier versions would have little effect on construction costs. The results are summarized as follows:

- **Cities Without Seismic Provisions:** 29 trial designs were completed in 5 cities that, at the time, did not have seismic provisions in their local building codes (Chicago, Fort Worth, Memphis, New York and St. Louis). The average projected increase in total building construction costs, attributable to incorporation of seismic provisions was 2.1%.

- **Cities with Seismic Provisions:** 23 trial designs were completed in 4 cities that did have seismic provisions in their local building codes (Charleston, Los Angeles, Phoenix, and Seattle). The average projected increase in total building construction costs, attributable to incorporation of the new seismic provisions was 0.9%.

C 7.1.2 Bridges and Other Elevated Structures

Similar studies were performed by AASHTO, and the results were included in their Commentary to the Standard Specifications for Seismic Design of Highway Bridges, dated 1983, including Interim Specifications dated 1985, 1987-88 and 1991.

In this study 21 bridges were evaluated by five state agencies (California, Idaho, New York, Oklahoma and Washington) and four consultants. The bridges were designed first using previous AASHTO seismic provisions and then redesigned using new proposed provisions (these were later adopted into the AASHTO Standard Specifications for Highway Bridges). The states used seismic acceleration coefficients and seismic performance categories appropriate to that state. The consultants evaluated the bridges for four acceleration coefficients (0.1, 0.2, 0.3, and 0.4) and three seismic performance categories (B, C and D). In all cases all loads remained the same except for seismic loads.

The average cost increase was approximately 6%. The percent increase varied with structure type and, of course, acceleration coefficient. One continuous span concrete bridge increased by 45%. All but three were below 10%, however.

C 7.1.3 Subsurface Facilities

No studies were found for increased costs for underground structures. The following discussion gives some indication, however of the increased costs that can be expected for tunnels.

In general, the cost of seismic provisions is associated with the additional reinforcement used in making joints suitably ductile. Typically this might mean additional secondary reinforcement to confine the concrete and to contain the main reinforcement. There may be limitations on the number or location of reinforcement splices, and in general there may be more steel for a given volume of concrete. The additional amount of steel, and possibly the greater difficulty factor associated with placing it at a greater density, can lead to tangible cost increases.

It is difficult to determine the significance of these design changes. It is not likely in the case of a modern tunnel design, that the provisions for earthquakes would lead to a stouter structure requiring more cubic yards of concrete. In the case of a recent metro design in California, it was determined that the cost directly attributable to seismic factors was approximately a 5% increase in the reinforcement cost. To put this figure in perspective, it is necessary to consider the whole project cost.

As a hypothetical example, consider a cut-and-cover tunnel project with the following costs-

Item	Cost
Mobilization	$ 2 ,000,000
Earthwork	$ 1,400,000
Foundation Drain	$ 200,000
Reinforced Concrete	$25,000,000
Mechanical	$ 4,400,000
Ventilation	$ 3,000,000
Electrical	$ 5 ,000,000
Total	$41,000,000

For this example, concrete makes up 61% of the cost of the project. Assuming a unit cost for in-place concrete of $160 per cubic yard (not including reinforcing) and an average of 180 lb of reinforcing steel per cubic yard at $0.50 per lb, the reinforcing steel would make up 36% of the cost of the concrete, and 22% of the cost of the total project. Therefore, if the seismic considerations added 5% to the cost of the reinforcing steel, they would actually be adding 5 % to the above 22 % i.e. they would add 1.1% to the cost of the project. If the above were a transit tunnel, and the actual transportation systems were to be added to the project cost (e.g. the cost of the trains and the power and control equipment) the impact of seismic considerations would be even smaller.

It can be concluded that cost impacts would be low, less than 5%. This is reasonable considering the relatively small demands put on subsurface structures during earthquakes.

C 7.2 Retrofitting

The economic implications of implementing a seismic design/retrofit policy are truly enormous. The United States Department of Transportation Seismic Committee is currently pursuing a seismic retrofit program. Thousands of facilities are involved, and the seismic resistance of many older structures is largely unknown. Just completing an inventory to begin assessing seismic vulnerability for existing structures will be a major undertaking.

In general, the cost of constructing a seismic retrofit to a substandard structure is orders of magnitude higher than the cost of constructing a seismically resistant structure in the first place. Designing and constructing a structure for seismic loads generally adds about 1% to 6% to the cost of the facility, while designing and constructing retrofits can exceed 100% of the replacement cost, in which case abandonment or total reconstruction would be warranted.

The cost of retrofits can vary widely, however. They can range from the replacement of airport control tower glass windows with Plexiglas, to the total reconstruction of bridge abutments. Similarly, the cost/benefit ratio can vary widely. Costs, benefits, risks and vulnerability must all be considered in the development of a seismic retrofit program.

The following table summarizes some seismic deficiencies, along with alternatives for remedial retrofits and comparative costs.

Table C7-1

Seismic Retrofit Costs

Potential Vulnerability	Retrofit Alternatives	Relative Cost[1]
Liquefaction	• Dynamic Compaction • Vibroflotation • Excavate and Replacement • Grouting • Strengthen With Long Piles	• Low • Moderate • Low to High • High • High
Landslides (including embankments and dikes at port facilities)	• Stabilizing Berms • Flattening Slope • Horizontal Drains • Reinforcing Dowels	• Low to Moderate • Low • Low • Moderate to High
Bridge Superstructure Failure	• Cable Restrainers • Increase in Beam Seat Support Width • Base Isolation Bearings • Keeper Blocks at Bearings • Replacement of Rocker Bearings with Elastomeric Bearings	• Low • Low to Moderate • Low • Low • Low
Bridge Column Failure	• Steel Jackets • Column Replacement • Supplemental Columns	• Moderate • High • High
Bridge Substructure/ Foundation Failure	• Base Isolation Bearings • Increase in Footing Size • Installation of Piles or Caissons • Replacement of Substructure	• Low • Low to Moderate • Moderate • High
Fuel and Gas Piping (Airports and Harbors)	• Automatic Shut-off Valves • Independent Regulators	• Low • Low
Moment Resistant Framed Building Failure	• Base Isolation Bearings • Framing Modifications	• Low • Low to High
Unreinforced Masonry Shear Wall Failure	• Abandonment • Total Reconstruction • Supplementary Framing, Foundations • Grouted in Supplemental Reinforcing	• High • High • High • High
Failure of Airport Control Tower Windows	• Replacement of Glass with Plexiglas	• Low
Non-Structural Components: (Mechanical, Electrical, HVAC, etc.)	• Bracing and Anchorage	• Low

Note 1. Relative costs are defined approximately as follows:
- Low: Less than 10% of facility cost.
- Moderate: Between 10% and 50% of facility cost.
- High: Greater than 50% of facility cost.

C 8.0 CONCLUSIONS

C 8.1 Summary

Seismic design/retrofit implementation involves three (3) major elements, as discussed previously:

1) Determination of the seismic hazard potential (Appendix A),

2) Evaluation of the level of vulnerability of a particular facility to a particular seismic hazard (Appendix B), and

3) The actual seismic design of a new facility or retrofitting an existing facility (this section - Appendix C).

Seismic design has evolved rapidly, from almost no criteria up through the 1950s, to quite complex procedures today. At the beginning of this evolution, seismic design measures were deficient, and structures built according to these provisions are vulnerable to severe earthquake damage. Remedial retrofitting of these structures can correct these deficiencies. Current seismic design technology enables engineers to design structures with reduced vulnerabilities. This has been proven by the successful behavior of newly constructed structures exposed to recent earthquakes. This technology will continue to evolve and new improved methods of designing for earthquake will be developed.

C 8.2 Major Uncertainties

As stated before, earthquake engineering, by nature, has many uncertainties. Although the state of the art now provides reasonable solutions for the majority of the situations an engineer is likely to encounter, there are still many areas of uncertainty in seismic design. Research is ongoing in most of these areas, and the state of knowledge continues to expand at a rapid pace.

Some structure types that have historically performed well in earthquakes have received little attention, while those that have proven to be safety issues during earthquakes have received the most scrutiny. For example, masonry buildings have been scrutinized, whereas tunnels have received very little attention. Other uncertainties whose solution are the most complex, have been put aside while equally important, but less costly problems were tackled first.

C 8.2.1 General

Ground Motion Data: One glaring uncertainty in seismic design is in the prediction of ground motion accelerations in different areas of the country. To date, this has been based primarily on historical data. Earthquakes have not been studied long enough to gather sufficient data to be able to make accurate predictions in all regions. There are areas, such as the northeast, where there have not been enough recent earthquakes to gather adequate ground motion data.

Ground motion possibilities are ever changing. Prior to the Mexico City Earthquake, seismic criteria did not account for the type of motion observed there. This motion was unusual in that it had a long period, and long duration. This motion resulted from the presence of unusual soils and geological formations in the area. Subsequent to this earthquake, codes were updated to account for these types of soils. In the future, undoubtedly, other types of ground motion will occur and codes will be revised accordingly.

Unified Approach: A unified approach to seismic design is needed. The various codes use different methods. Design would be simpler if there was a nation-wide consensus on seismic design.

This is one of the main objectives of NEHRP, and there has been recent progress, but more is needed.

The methods now used to design for seismic activity vary according to the agency policies and the judgment of the designer. Often, these methods do not properly account for regional seismic hazards. Additionally, different approaches are taken for different categories of structures, and although there are many codes and guidelines, it is not always clear which should be used for a specific project.

It would be desirable to have a nationally adopted design guideline which would allow a project to be responsive to its specific needs, while still ensuring that a minimum level of attention is paid to the problem of earthquakes. Such a guideline should consider all of the factors in the current codes, such as regional seismicity, categories of structure and levels of risk. Any such guideline should represent minimum levels of attention, and should not preclude project-specific enhancements made on the basis of special studies.

The biggest danger that exists now is that in areas of low-to-medium seismic activity, earthquake considerations may be generally overlooked and thus result in designs of insufficient capacity to safely perform in the event of an earthquake.

C 8.2.2 Surface Facilities

Irregular Structures: The affect that structural irregularities have on a structure's ability to withstand earthquakes is still in the developmental stages. To date, the codes exclude these structures from their force provisions, and require a more complex approach. This could be refined more, so that a simpler approach could be followed, possibly using equivalent static force methods, with a factor based on their specific irregularities.

Detailing: As more earthquakes occur and damage is observed, more deficiencies in detailing are identified. This will be an ongoing effort for some time.

Non-Structural Building Components: To date, architectural, mechanical, and electrical building components have not received the attention they need. These components can be life threatening in an earthquake. This is an area that has fallen through the cracks in the past. These are not structural items, so they have not received much attention from structural engineers. Other disciplines do not have the where-with-all to properly design them. Possibly the use of standard approved details, similar to Underwriters Laboratories' fire resistant details, would be a solution.

Allowable Soil Bearing Pressure: More data is needed for determination of allowable soil bearing pressures under dynamic loading. The building codes currently give little direction on this issue.

Effect of Soil Conditions: More accurate methods of determining soil factors are needed. Currently, the way in which soil and site conditions are considered in seismic design is crude. Usually, there are three or four factors corresponding to general descriptions of subsurface conditions. More categories should be developed, or empirical or theoretical formulae developed that accurately account for subsurface conditions.

Railroad Bridge Design: There are no specific provisions for seismic design in the AREA Code. The methodology presented in the AASHTO Code should be appropriate, generally, for railroad bridges. The AREA Code should be updated to include these, or similar, seismic requirements.

C.8.2.3 Subsurface Facilities

Design Methods: Currently, codes have no provisions for seismic design of tunnels or buried structures. There are lateral earth pressure formulae for retaining walls, which incorporate the passive soil resistance from the deflection of the wall, but no direction is given for tunnels or buried box structures. Procedures have been developed for project-specific requirements in the past. The methods that have been developed have been successful, as evidenced most recently by the performance of the BART tunnels during the Loma Prieta Earthquake. Code requirements should be developed based on these past projects and the current state of the art.

Soil/Structure Interaction: Specific procedures should be developed for determining the effect of soil/structure interaction in tunnel design. Current practice is to assume the tunnel is more flexible than the surrounding soil and deforms as the soil deforms. This is overly conservative for stiff structures in loose soils. Research is currently underway at Parsons Brinckerhoff to quantify these effects.

C 8.3 Conclusions

A brief review of damage and loss of life associated with the world's great historical earthquakes clearly indicates the high cost associated with the failures of inadequately constructed facilities, both private/domestic and large public facilities. Recent events in our own country, including the San Fernando (1971) and Loma Prieta (1991) earthquakes, indicate the private and public costs associated with earthquake damage. Post earthquake analyses of damage from these earthquakes concluded that much of the damage was to older facilities that were constructed without adequate seismic provisions. Had there been a greater awareness of seismic risk at the time of construction, many more structures might have survived with minimal additional investment.

The cost of replacing a collapsed structure, or one deemed unsafe, is obviously very high, both in terms of capital expenditure and in the social inconvenience suffered by the community during the interim period. Also, as indicated by the extensive retrofitting program undertaken after the San Fernando earthquake in California, there are only limited seismic improvements which are practical to install after the fact.

In conclusion, adequate provisions at the time of construction can be made with modest increases in total investment, usually less than 5% of the total facility cost. Also, the financial and social costs of major repair or replacement are not viable. Retrofitting is expensive, and the actual benefits are often of limited effectiveness. It is clear that the minor additional cost of building a safer structure is well justified.

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Map 1.	ATC	County Map - Coefficient Aa
Map 2.	ATC	County Map - Coefficient Av
Map 3.	ATC	Contour Map for Coefficient Aa
Map 4.	ATC	Contour Map for Coefficient Av
Map 5.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 6.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 7.	USGS	0.1 second spectral response acceleration with a 90% probability of nonexceedance in 50 years
Map 8	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 9.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
Map 10.	USGS	0.3 second spectral response acceleration with a 90% probability of nonexceedance in 50 years (California)
Map 11.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years
Map 12.	USGS	1.0 second spectral response acceleration with a 90% probability of nonexceedance in 250 years (California)

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