EARTHQUAKE-RESISTANT STRUCTURES

Design, Build, and Retrofit

Mohiuddin Ali Khan

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MOHIUDDIN ALI KHAN, PH.D., P.E., C. ENG., M.I.C.E. (LONDON)

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PREFACE

Earthquakes in the past few years have rocked the world, causing any number of deaths and destruction, the magnitude of which far exceeds that caused by man-made world wars. It rests on the shoulders of engineers more than anyone else, to find engineering solutions to resist them and mitigate their ill effects. This is possible through science and technology by developing seismic-resistant structure designs governed by effective seismic building codes.

Earthquakes are a global phenomenon associated with the formation of earth billions of years ago. Discontinuities and faults in the earth's crust cause physical changes and tectonic plate movements that result in vibrations and oscillations in the foundations of buildings, bridges, dams, and other structures. Earthquakes often create tsunamis caused by seismic movements of ocean beds.

Seismic-resistant design is a relatively new subject that is now being studied and carried out on a scientific basis. It is also being taken very seriously at the international level. One area of investigation, particularly in less-developed countries, is to make structures seismically safe at reasonable cost.

Buildings are different from bridges in their configurations, orientation, and structural planning. Methods of analysis are unique in each case. The lateral loads generated are dependent on structural mass. For equilibrium, reactions are transferred to foundations from superstructures. It is necessary to ensure stability and sufficient internal resistance in structural members. Constructability and the ability to translate a theoretical model into a practical structure are of vital importance. Since an earthquake affects millions of people and their abodes, casualties are a norm rather than an exception.

The information presented in the following chapters is based on the author's lectures on seismic engineering, both in the United States and abroad, research projects, and planning and detailed design of bridges and buildings for multiple agencies. Publications by the author were compiled from knowledge acquired at international conferences, service on the ASCE Seismic Effects and Methods of Analysis committees, organizing seminars as chairman of the Structural Engineering Institute in Philadelphia,

and investigating and planning post-earthquake reconstruction for the National Academy of Sciences.

In the United States, leadership in specialized education is being provided by the American Association of State Highway and Transportation Officials, the Federal Emergency Management Agency, the International Building Code, the U.S. Geological Survey (USGS), the American Society of Civil Engineers, the Multidisciplinary Center for Earthquake Engineering Research (MCEER), the Applied Technology Council, and many others, both in the United States and abroad.

Basic knowledge of earthquakes is being provided at the school level by organizations such as the Seismological Society of America. However, the number of textbooks, manuals, and handbooks is limited. The *Journal of Earthquake Engineering* is published on a monthly basis in the United States, and similar journals are published in other countries. In addition, informational websites include, among others, those hosted by MCEER, USGS, the British Geological Survey, and the Alaska Public Seismic Network. USGS, MCEER, the National Earthquake Information Center, and NATO have compiled and continually update databases of information relevant to seismic events. The USGS Earthquake Hazards Program FAQ database is very useful for both students and practitioners.

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CHAPTER ONE

Modern Earthquake Engineering: An Overview

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Come forth into the light of things
Let nature be your teacher

An English Poet

1.1 INTRODUCTION AND REVIEW OF LITERATURE

Nature teaches us about the earthquake location and the zone and intensity to expect. Over the last 50 years much progress has been made in earthquake engineering that will save lives, infrastructure, and homes. As the old saying goes, "Necessity is the mother of invention." Seismic-resistant design is not a single subject but a combination of diverse scientific disciplines, and the state of the art is advancing yearly. A book like this one is required just to broadly review new procedures and to cover the field's salient features. These are addressed in the chapters that follow.

A review of scientific literature on the subject can be made by glancing at the research papers being published in relevant structural

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engineering journals, conference proceedings, text books on related subjects, university theses, students project dissertations, seismic design codes and the continued publications by the management and scientific agencies. The available technical information is generally supported by dynamic analysis theory, computer software and laboratory studies on structural components or shake tables testing of typical prototype models.

The bibliography at the end of the chapter includes important code references such as:

American Association of State Highway and Transportation Officials (AASHTO, 1991), Specifications for Highway Bridges, Division1-A (AASHTO, 1996), Standard Specifications for Highway Bridges, 16th ed.; (AASHTO, 1998), Load Resistance Factor Design Specifications for Highway Bridges (AASHTO, 2007), LRFD Specifications for the Design of Bridges. In addition, American National Standards Institute/American Institute of Steel Construction (ANSI/AISC, 2005), Seismic Provisions for Structural Steel Buildings, #341-05.

ANSI (2005), Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, #358-05. American Society of Civil Engineers (ASCE) 2010. #07-2010;

Applied Technology Council (ATC, 1996), Improved Seismic Design Criteria for California Bridges Provisional Recommendations, ATC-32.

Publications of major scientific agencies are listed in the Bibliography such as Federal Emergency Management Agency (FEMA, 2006), Designing for Earthquakes, Providing Protection to People and Buildings, A Manual for Architects, Risk Management Series #454; FEMA/National Earthquake Hazard Reduction Program (NEHRP, 2000), Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, #368/369. (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 Commentary, #450, 2003; (NEHRP, 2003). Recommended Provisions: Design Examples, #451;

(NEHRP). Earthquake-Resistant Design Concepts: An Introduction to the NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, #P-749, 2009; Federal Highway Agency (FHWA) 1987, Seismic Design and Retrofit Manual for Highway Bridges.

Seismic Retrofitting Manual for Highway Bridges (FHWA, 2000).

National Seismological Teachers Association/FEMA 2012. Bulletin of the Seismological Society of America 102,2; National Fire Protection Association (NFPA, 2006); Building Construction and Safety Code.

Structural Engineers Association of California (SEAOC, 1999). Seismology Committee Recommended Lateral Force Requirements and Commentary.

In addition, useful newer publications by the following authors are included in the Selected Bibliography:

Agarwal and Shrikhande (2006), Berlin (1980), Coffman and von Hake (1982), Duggal (2007)

Saadeghvaziri et al. (2001), Spence, Sipkin, and Choy (1989) of U.S. Geological Survey, Stratta (1987) and Wakabayashi (1986)

References to other useful publications are addressed in the text in this chapter and in the chapters which follow.

Management agencies such as U.S. Geological Survey (USGS), the Federal Emergency Management Agency (FEMA), the Federal Highway Administration (FHWA), the National Earthquake Hazards Reduction Program (NEHRP), the American Society of Civil Engineers (ASCE), the California Transportation Agency (Caltrans), the Multidisciplinary Center for Earthquake Engineering Research (MCEER), and the Earthquake Engineering Research Institute (EERI), along with universities and individuals, play an important role in research, in developing design methods such as performance-based seismic engineering, and in developing policies for resisting earthquakes. The work and funding by these organizations and individuals have aided the development of state and regional earthquake design codes, as well as those that have been by many countries around the world. Indeed, USGS hazard maps are used on a global basis and International Building Code (IBC) and American Association of State Highway and Transportation Officials (AASHTO) design specifications are generally referred to in international publications.

In recent years, advances in seismology and soil science have influenced structural concepts and design guidelines, resulting in corresponding code changes. There have been many innovations, such as seismic hazard analysis, geotechnical site response and liquefaction, soil—structure interaction, structural response, structural design and retrofit, short- and long-term remediation, and post-earthquake reconstruction.

The goal of this book is to introduce important concepts in the study of earthquakes as they relate to the construction and retrofitting of structures to be earthquake-resistant. These concepts include a new understanding of uncontrolled natural causes such as tectonic plate movement and the ramifications of structure location near a fault, the distance of a structure to the quake's epicenter, and the soil type on which a structure sits.

We need to look for solutions so that communities living in seismic zones feel safer. This requires seismic evaluation of new and existing projects, which involves the following:

- Accurate seismic data
- Seismic characteristics of engineered systems
- Seismic design criteria and analysis procedures
- Seismic detailing and quality control

It is a challenge to design seismic-resistant structures, especially in a seismic zone. Some of the difficulty is due to uncertainties related to the unexpected nature of seismic activity, the duration of an earthquake's ground movements, the response of the soil below foundations, and the performance of nonstructural members.

This chapter introduces the main seismic concepts and the important subject of performance and response of structures to earthquakes. The scope of presentation is broad and mainly qualitative. A quantitative approach, by way of typical solved examples for seismic forces acting on buildings and bridges and the application of computer software is provided in the appendices.

1.2 THE BASICS OF EARTHQUAKES

According to an English Poet,

Art is long and time is fleeting, And our hearts though stout and brave, Still like muffled drums are beating, Funerals marching to the grave.

Indeed, the art of earthquake engineering is long comprising of several disciplines, while the duration of earthquake is short and fleeting. It requires men with stout and brave hearts to bear the agony of destruction. For example, in October, 2005 earthquake in Northern Pakistan, 80,000 funerals marched to the graves. Those injured seriously or disabled, their lives are far from normal.

The term *earthquake* describes a seismic event. It can be either a natural or a man-made phenomenon and is caused by the generation of seismic waves in soils and rock. Earthquakes generally result from a geologic event and pose hazards for those living in earthquake-prone areas. Several alternate terminologies are used to define an earthquake, including *quake*, *trembling motion of the earth*, *tremor* or *temblor*, and *subsurface seismic activity.*

The earth's crust is made up of a variety of materials, including rock, that form into what are known as *tectonic plates*. Sometimes there are breaks or fractures between various rock layers, which are called *faults*. Faults represent an area of weakness in tectonic plate formation. An earthquake is caused by a buildup of stress along the fault, causing the tectonic plates to crack and slip past each other. The faults open up to release energy in the form of seismic waves that cause the ground to shake. The degree of shaking is based on the vertical distance from the *epicenter*, which is the point on the earth's surface directly above the focal point of the quake. The waves occur below the surface, usually at depths of less than 30 kilometers.

When hit by an earthquake of moderate magnitude, the ground "trembles" and soil-supported structures "shiver" in the region of occurrence. During this quick and abrupt shaking, there is not always enough time for escape from a structure before it fails. The destabilizing action on construction can be from direct ground motion (such as *displacement*, *velocity*, and *acceleration*) or indirect effects such as earthquake-induced landslides, soil *liquefaction*, and tsunamis). Foundations resting even on compact ground, if displaced in any direction, can cause the supported walls and roofs to tilt and collapse.

Aftershocks may occur for several hours, or even several days, after the initial earthquake, but their magnitudes are lower. Continued aftershocks may last longer than wind or hurricane events. Except for structures located near geological faults, the forces generated are comparable in magnitude to those generated by peak wind forces such as on high-rise buildings.

Fire is one of the main results of earthquakes. Fires may occur after the earthquake for any number of reasons, including broken gas mains and downed power lines. The fires generated by the 1906 San Francisco earthquake are a case in point. Depending on the quake's magnitude, essential utilities such as water supply and sewage disposal may be disrupted in Figure 1.1, seismic activity (both natural and man-induced) is summarized in a flow diagram showing wide-ranging primary and secondary effects in 1969 the locations of all earthquakes occurring from 1961 to 1967 were published. Most of them were confined to narrow belts defining the boundaries of the tectonic plates. The interiors of the plates are mostly free of large earthquakes. However, within continental regions which are located, away from boundaries, earthquakes are large and devastating.

1.3 THE MOST DESTRUCTIVE EARTHQUAKES ON RECORD

In 1556, the deadliest earthquake then known struck Shensi province in China, killing about 830,000 people. The New Madrid quakes of 1811–1812 hit Missouri and Arkansas; the largest one made the Mississippi River run backward for a time. No one alive now has memories of the South Carolina quakes of 1886, which toppled 14,000 chimneys in Charleston and were felt in 30 states. The Messina earthquake and tsunami took as many as 200,000 lives on December 28, 1908 in Sicily and Calabria.

One of the largest earthquakes ever recorded was a magnitude 9.5 that occurred on 1960, in Chile, on a fault that is almost 1,000 miles long. This quake caused the deaths of more than 4,000. Extreme examples of entire plate-boundary rupture are the great earthquakes in Chile in 1960 and in Alaska in 1964.

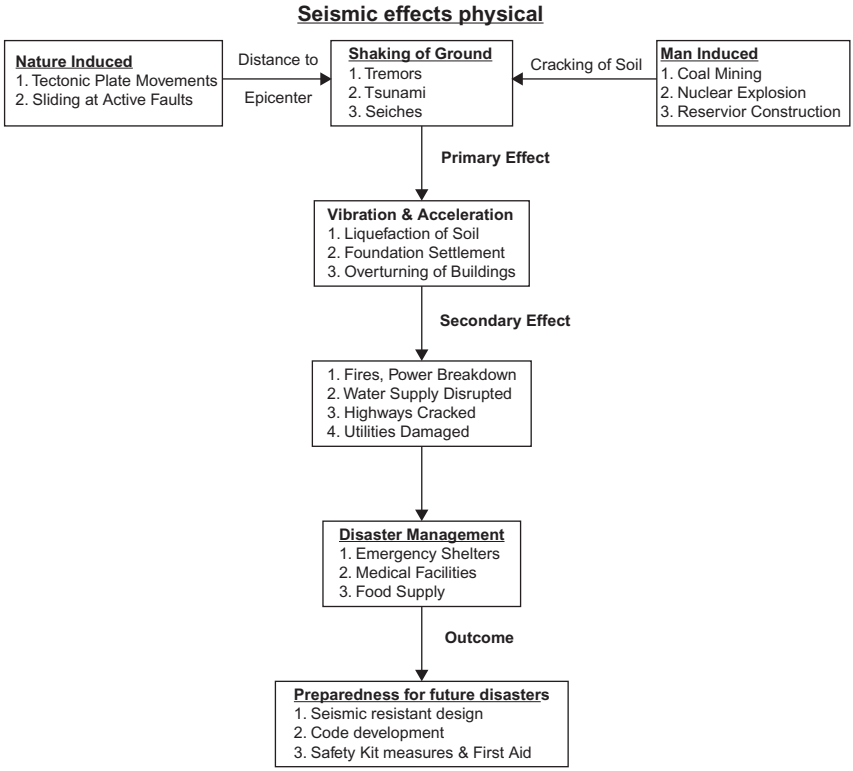

Figure 1.1 Flow diagram of causes and effects of earthquake activity.

In 1970, in Peru, at least 18,000 people died and towns around Ranraharca, including most of Yungay, were destroyed. A resulting avalanche amounting to about 50 million cubic meters of rock, snow, ice, and soil traveled 15 kilometers from the north peak of Huascaran Mountain. The 1976 magnitude 7.8 earthquake that struck Tangshan, China, killed somewhere between 250,000 and 800,000. Kobe, Japan, experienced colossal devastation in 1995, with over 5,000 people killed even though the city was relatively new.

In 2003, the magnitude 6.5 earthquake in Bam, Iran, killed more than 40,000 people. Almost 70,000 lost their lives in Sichuan, China, in 2008, when the Zipingpu Dam increased the pressure on a nearby fault, causing a landslide. This remains the nineteenth deadliest earthquake of all time.

1.4 IS SEISMIC ACTIVITY INCREASING?

The recent devastating earthquakes in Haiti, Chile, Mexico, and elsewhere have many wondering if this activity is unusual. However, six major earthquakes striking in the first four months of 2010 are well within the normal range. Further, from April 15, 2009, to April 14, 2010, there have been 18 major earthquakes, a number also well within the expected variation. Instrumentations used for the measurement of intensity and magnitude of earthquakes are described in Section 1.6.

Although small-magnitude earthquakes are fairly common, as we have seen, there have also been many large-scale earthquakes throughout history. The more recent ones with magnitudes starting at 8.0 are listed in Table 1.1. Table 1.2 shows the very large number of recurring small- and large-magnitude earthquakes and their accompanying physical effects.

Earthquakes took the lives of many people in 15 countries on four continents during 2009, including Afghanistan, Bhutan, China, Costa Rica, Greece, Indonesia, Italy, Kazakhstan, Honduras, Japan, Malawi, Samoa, South Africa, Tonga, and American Samoa. People in 11 additional countries were injured, including the mainland United States, where a magnitude 4.4 earthquake on May 2, 2009, injured one person in the Los Angeles area.

In 2009, 17 earthquakes reached a *magnitude* of 7.0 or higher, and one broke a magnitude of 8.0. These statistics are higher than those of 2008,

Table 1.1 Earthquakes of 8.0 Magnitude and Higher, 1900–1965

Location	Date	Magnitude
Chile	May 22, 1960	9.5
Prince Sound, Alaska	March 28, 1964	9.2
Aleutian Islands	March 9, 1957	9.1
Off-coast Ecuador	January 31, 1906	8.8
Aleutian Islands	February 4, 1965	8.7
India–China border	Aug 15, 1950	8.6
Kamchatka	Feb 3, 1923	8.5
Banda Sea, Indonesia	Feb 1, 1938	8.0
Kuril Islands	Oct 13, 1963	8.5

Table 1.2 Number of Earthquakes per Year and their Physical Effects

Magnitude	Physical Effects	Number (est.)
<2.0	Generally not felt, but recorded; effects near epicenter	600,000
2.0–2.9	Potentially perceptible	300,000
3.0–3.9	Felt by some	49,000
4.0–4.9	Felt by most	6,200
5.0–5.9	Damaging shocks	800
6.0–6.9	Destructive in populous regions	266
7.0–7.9	Major earthquakes with serious damage	18
8.0	Great earthquakes; cause extensive destruction near epicenter	1.4

which experienced only 12 earthquakes over magnitude 7.0 and none over 8.0. Factors such as size, location, and depth relative to population centers, and fragility of buildings, utilities, and roads all influence how communities are affected.

Earthquakes in 2010, 2011, and 2012 in Japan, New Zealand, Chile, Haiti, Indonesia, and Mexico caused much loss of life, enormous damage to man-made infrastructure and buildings, and immeasurable disruption of commerce and industry. Communities exposed to the wrath of nature know "where the shoe pinches."

Although statistically the number of recent earthquakes is within the normal range, as previously noted, the fact that an average of 18 major earthquakes of magnitude 7.0 to 7.9 and one great earthquake of magnitude 8.0 or higher occur per year, emphasizes the need for seismic-resistant structure design much more seriously.

1.5 SOME SEISMOLOGICAL HISTORY

The twentieth century has seen an increased interest in the scientific study of earthquakes. However, long ago, before the advent of seismology, people had other ways of explaining this phenomenon.

For example, many ancient peoples believed that something large and restless lived beneath the earth's surface. Our ancestors imagined that giant snakes, turtles, catfish, or spiders lived underground and it was their movements that created earthquakes. Another ancient belief was that the earth was perched on the horns of a bull. An earthquake resulted when the bull grew tired and shook his head.

Earthquakes were also seen as an act of God, occurring as a warning to people who defied God's instructions and thereby suffered His wrath as punishment. By the same logic, if collective behavior continued to be unsatisfactory, earthquakes might recur with a greater frequency or intensity. Some faiths still hold this belief.

1.5.1 Early Scientific Beliefs

Aristotle was one of the first observers to attempt an explanation of earthquakes based on natural phenomena. He postulated that winds within the earth acting on vertical surfaces or mountains caused the occasional shaking of the earth's surface. Empirical observations of the effects of earthquakes were rare until 1750. Then the Lisbon earthquake of November 1755 struck, causing a huge tsunami that killed an estimated 70,000 people and leveled the city. Prior to this catastrophe, European scholars had looked to Aristotle, Pliny, and other ancient classical sources for explanations, but now the beginning of the modern era of seismology commenced, prompting numerous studies into locations, effects, and timing. These early efforts were spurred by the 1783 Calabrian earthquakes that killed 35,000 people in Southern Italy.

Following a major earthquake in Chile in 1822, the author Maria Graham reported systematic changes in the elevation of the Chilean coastline. Robert Fitz Roy, captain of the *H.M.S. Beagle*, confirmed observations of coastline changes after another Chilean earthquake in 1835. Mark Twain witnessed the 1906 major San Francisco earthquake and for once his fiction turned out to be reality. Jules Verne wrote the novel on "A journey into the interior of earth". The origin of volcanoes and tectonic plate movement is linked to the origin of geology of earth and

physical formation of the solar system and the universe itself. Unfortunately, our knowledge of seismology is still rudimentary and at best a surmise.

1.5.2 The Beginnings of the Science of Seismology

Role of Seismologist Mallet: The notable founders of seismology were Robert Mallet (1810–1881), a civil engineer by profession who designed many of London's bridges, and John Milne (1850–1913), a British mining engineer. After the great Neapolitan earthquake of 1857 in southern Italy, Mallet explained earthquake effects as "masses of dislocated stone and mortar" seen in debris. He established a basic vocabulary that included such terms as *seismology*, *hypocenter* (often called the earthquake focus), and *isoseismal* (referring to contours of equal seismic intensity). To measure the velocity of seismic waves in the earth, Mallet used explosions of gunpowder. His idea was to look for variations in seismic velocity that would indicate changes in the earth's properties. Mallet was also one of the first to estimate the depth of an earthquake underground. His method is used today in oil exploration.

Role of Seismologist Milne: Milne, along with Englishmen James Ewing and Thomas Gray, working at the Imperial College of Tokyo, invented the first seismic instruments sensitive enough to be used in scientific study. The late 1800s and early 1900s also saw scientific inquiry into earthquakes begun by Japanese researchers. Seikei Sekiya became the first person to be named a professor in seismology; he was also one of the first to quantitatively analyze seismic recordings. Another famous Japanese researcher from that time was Fusakichi Omori, who studied the rate of decay of aftershock activity following large earthquakes. His early concepts are still in use.

Earthquakes in Fiction and Film

Addressing the social issues and suffering resulting from earthquakes has given their scientific aspects greater prominence. Stories about earthquakes generally begin with a disaster and then focus on its aftermath. Luckily, significant feedback exists from novelists and filmmakers worldwide about the impacts of any violent ground shaking. The casualties and destruction are magnified and the sense of being out of control emphasizes helplessness.

An earthquake depicted in a story or in a movie is sometimes exaggerated and based on anyone's guess. However, simulations showing patterns of

bridge and building collapses in a movie, for example, may be borrowed from actual events and not entirely falsely represented. They may also emphasize serious and important issues of human suffering and the difficulties faced in a post-earthquake scenario. Still, responsibility eventually falls on the engineer's shoulders to find engineering solutions.

Depicting disasters and their dramatic and adverse impact on society may help indirectly in remediation by focusing attention on the funding of research and code development.

1.5.3 Modern Seismology

Only in the past few decades has some progress been made by seismologists in understanding tremors, earthquakes, and resulting tsunamis. The science related to seismic activity across the globe is now developing rapidly, made possible by advanced techniques such as computer software for dynamic analysis, shake table tests, soil analysis, and GIS zoning maps. From close field observation, documentation, and analysis, it has been established that earthquakes are the result of a variety of natural and man-made factors:

- Subsurface tectonic plate movements
- Ground rupture at geological faults
- Landslides
- Volcanic activity
- Mine blasts
- Underground nuclear explosions

1.6 MEASUREMENT OF EARTHQUAKE INTENSITY AND MAGNITUDE

The basics of the subject were discussed in Section 1.2. Definitions used in physical science seismic theory and applied mathematics are presented in Sections 1.6 to 1.8. *Vibrations* are the waves of energy caused by the sudden breaking of rock within the earth or by an explosion. The record of an earthquake is an instrumental plot of vibrations versus time. The time is marked at regular intervals so that the times of arrival of the first P wave and the first S wave are recorded (P waves and S waves

are described in Chapter 2). *Seismic waves* travel through the earth and are recorded on seismographs. The greater the interval between the arrival of the first primary wave and the first secondary wave, the greater the distance to the epicenter. To understand the source, modern seismology uses

- The P wave's first motions
- The amplitudes of direct P and S waves and surface waves

1.6.1 Intensity

Before the invention of instrument-based intensity estimation, the most widely used tool was the Rossi–Forel scale, invented in the 1870s. A modification was developed by the Italian Mercalli in 1884; its modern version is termed the *Modified Mercalli Intensity (MMI) scale* and is suitable for conditions in the United States. The MMI scale measures the human impact of magnitude, rating felt intensity from I to IX. The information it records helps in seismic zoning, identifying seismic categories, and preparing peak ground acceleration (PGA) maps.

Definition: Intensity is an estimate of the violence of an earthquake at a given site, representing the amount of shaking and damage. The area of highest intensity is the most probable position of the epicenter and the causative fault rupture. Bolt (2003) describes the details of the various intensity measures. Intensity generally decreases with distance from the epicenter and is associated with the physical factors of location, damage to structures, degree of felt shaking, and presence of secondary effects. It reflects seismic wave size, distance from the epicenter, and involved geological structures.

Felt intensities as measured by the MMI scale during an earthquake are shown in Table 1.3.

1.6.2 Magnitude

In 1935 the seismologist Charles Richter introduced a logarithmic scale using an instrumental measure of magnitude from 1 to 10. A logarithmic *Richter scale* measures energy release during plate movements—an increase of 1 in magnitude corresponds to a 10-fold increase in the amplitude of shaking. Determining Richter magnitude involves measuring the amplitude (height) of the largest recorded wave at a specific distance from the earthquake. *Amplitude* is based on distance, depth, and wave period.

Table 1.3 Damage and Physical Effects of Intensity Based on the MM Scale

MM Scale	Perception and Reaction	Physical Effects and Damage
I	Not felt	Felt by very few under special circumstances
II	Felt by a few at rest	Felt on upper floors
III	Quite noticeable indoors	Felt on upper floors
IV	During day felt indoors by many, outdoors by few	As is building hit by a heavy truck
V	Felt by nearly everyone, many awakened	Disturbances of trees, poles, and tall objects noticed
VI	Felt by all; many frightened and run outdoors	Damage slight: heavy furniture moved; few instances of fallen plaster or damaged chimneys
VII	Shock; running outdoors	Damage negligible in buildings of good design and construction; slight to moderate in well-built structures; considerable in poorly built or badly designed structures
VIII	Shock; running outdoors	Fall of chimneys, columns, monuments, walls; damage considerable in ordinary buildings; partial collapse; damage great in poorly built structures
IX	Widespread chaos	Buildings shifted off foundations; ground cracked; damage considerable in specially designed structures
X	Panic	Most masonry/frame structures destroyed; ground badly cracked
XI	Rare	Few masonry structures remain standing; bridges destroyed; broad ground fissures
XII	Very rare	Damage is total; waves appear on ground surfaces; objects thrown into air

Magnitude is measured by ground amplitudes recorded at stations. The main features of Richter measurements are

- *Focus*: the point of origin, typically tens to hundreds of miles underground.
- Location: where the rupture begins.
- Epicenter: the point on the surface directly above the focus.
- *Strain-release pattern*: the complexity at the source and in the earth materials through which the waves pass before they reach a seismograph.

Table 1.4 Largest U.S. Earthquakes on the Richter and MMI Scales

Location	Year	Richter Magnitude	MM Intensity
California (largest)	1857	7.92	IX
California (2nd largest)	1906	7.68	IX
Delaware	1871	Unknown	VII
Georgia	1914	Unknown	V
Maine	1904	Unknown	VII
Maryland	1990	Unknown	V
Massachusetts	1755	Unknown	VIII
New Hampshire	1940	5.25	VII
New Hampshire	1940	5.6	VII
New Jersey	1783	Unknown	VI
New York	1944	5.52	VIII
North Carolina	1916	Unknown	VII
Pennsylvania	1998	Unknown	VI
Rhode Island	1976	2.07	VI
South Carolina	1886	7.02	X
Vermont	1962	Unknown	V
Virginia	1897	Unknown	VIII

Table 1.4 is a historical record of major U.S. earthquakes based on both the Richter and the MM scales. It shows that California has experienced the highest magnitudes among all of the continental states.

1.7 SEISMIC INSTRUMENTATION

The number of earthquakes being reported is greater than in the past (see Table 1.2) primarily because of increased concern and the development of modern instrumentation. The USGS National Earthquake Information Center (NEIC) receives data in real time from nearly 990 stations in 85 countries, including the 150-station Global Seismographic Network, which is jointly supported by USGS and the National Science Foundation (NSF) and operated by USGS in partnership with a consortium of universities known as Incorporated Research Institutions for Seismology (IRIS).

Modern seismic instrumentation such as seismogram, seismograph and seismoscope (described below) uses complex electronics to accurately record ground shaking. Electronics have given rise to high-precision

pendulum seismometers and sensors of both weak and strong ground motion. Electronic voltages produced by the motions of a pendulum are passed through electronic circuitry to amplify the ground motion and digitize the signals for more exact measurements.

Seismograms

A *seismogram* shows the amplitude of body and surface waves, which indicates the amount of strain energy released. Richter magnitude is measured in energy (ergs): $M = \log 10\ (A/A0)$, where A = amplitude on a seismograph, and $A0 = 1/1{,}000$ millimeters. Each increase of 1 in Richter magnitude represents a 31-fold increase in the amount of released energy. Thus, a magnitude of scale-6 intensity = 10 × a magnitude of scale-5 intensity.

The amount of energy released by an earthquake is related to the Richter scale by the equation $\log E = 11.8 + 1.5M$, where

log = logarithm to the base 10
E = energy released, in ergs
M = Richter magnitude.

Seismographs

A *seismograph* records ground motions such as accelerations and displacements as a function of time using the principle of *inertia*. The record of ground shaking helps locate the quake's epicenter and focus. Special arrays of strong-motion seismographs have been installed in areas of high seismicity around the world, both away from and on structures. Strategically placed on structures, they provide information on structural response.

Earthquakes can be recorded up to great distances because seismic waves travel through the Earth's interior. When a vibration reaches the seismograph, the movement of the earth in relation to a stationary mass is recorded. The equivalent energy released can be comparable to that of an atomic bomb. For example, the atomic bomb dropped on Hiroshima released an amount of energy equivalent to a quake of magnitude 5.5.

Seismoscopes

Seismoscopes, usually arrayed in networks, indicate the occurrence of an earthquake. With increases in seismic literacy, the size of seismoscopic networks has increased from about 350 stations in 1931 to many thousands today. The first seismoscope was invented by the Chinese philosopher Chang Heng in 132 A.D.

Shake Maps

Correlations have been worked out between measured characteristics of seismic waves and reported Modified Mercalli intensity. A common correlation is that between the maximum ("peak") ground acceleration, *A*, and the MM intensity, *I*. Peak ground acceleration (PGA) is employed in the current USGS Shake-Maps program, which produces maps showing ground-shaking intensities that are made available online within a few minutes of an event. The geographical distribution of intensity is summarized by constructing isoseismal curves, or contour lines, that separate areas of equal intensity.

1.8 COMPREHENSIVE SEISMIC STUDY

Seismic study comprises of a host of disciplines, including seismology, geotechnical and structural engineering, and architecture. The seismic disciplines are captured in Figure 1.2 as a flow diagram.

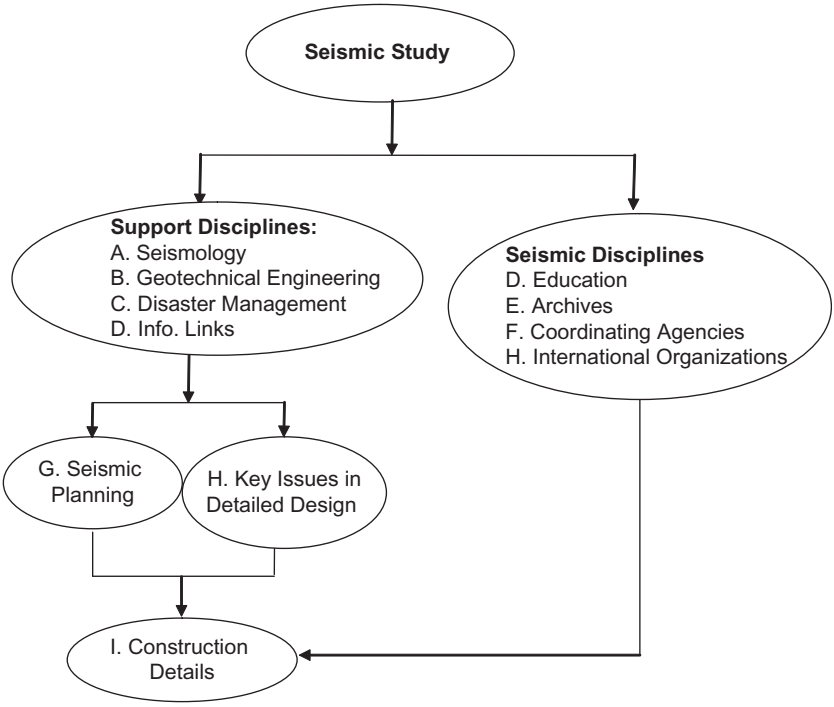

Figure 1.2 Flow diagram of disciplines in seismic study.

1.8.1 Structural Effects of Seismic Events

More Definitions Related to Seismic Analysis and Force Computation: Newton's second law of motion states:

Inertial force (F) = mass (M) × acceleration (A)

When a building shakes it is subject to inertial forces. A horizontal (lateral) force generates the acceleration, or rate of change in velocity, of the waves, setting the building in motion. This force is determined by the percentage of the building mass or weight that shakes as a result. Compared to reinforced concrete buildings, wood frame buildings tend to perform well because the inertial forces that act on them are not as great.

Acceleration Occurs during an Earthquake

Acceleration is measured in terms of gravity or g value. One g is defined as the rate of change in velocity of a body freely falling in space (32 feet per second per second). Acceleration is additive and in progression with time, so a second later the velocity is 64 feet per second. In moderate earthquakes, the vibration waves may last for a few seconds and accelerations may be as high as 0.2 g. Poorly constructed buildings begin to suffer damage at about 0.1 g.

Response of Nonstructural Components and Utilities to Accelerations

Most nonstructural components are fragile and may increase lateral forces and vibrations. They are easily damaged and are costly to repair or replace after an earthquake. Nonstructural components account for over 60 percent of a modern structure. Thus, for example, maintenance of mechanical systems, such as water supply, plumbing, and wiring, is important in the ability of a building to resist seismic forces. Any short circuit of electrical cables or leakage of gas can give rise to fires. These can be avoided by special connections with beams and columns. Pipes and wires can pass through trenches or be hung from the structure so that any large movements might bend and rattle but not break them.

Duration of Earthquake

In general, a number of cycles of moderate acceleration repeated over time can be much harder for a building to withstand than a single high-peak acceleration. Continued shaking weakens a building and reduces its resistance to damage. When accelerations exist over a time *duration*, the impact of forces comes into play and displacements increase.

Natural Period of Vibrations

The natural or *fundamental period* is the rate at which objects move back and forth if given a horizontal push. An important characteristic of ground waves is their period or frequency. If the waves are quick, seismic forces are higher and vice versa.

Damping Helps in Reducing Vibrations

Damping is a measure of decay in amplitude. It is based on internal friction and absorbed energy. Building components such as partitions and exterior façades contribute to damping when made to vibrate. The amplitude of the vibration decays over time and eventually the vibration ceases. The percentage of damping with respect to time is described by *response spectrum* tables or graphs, which generally show acceleration values for 0, 2, 5, and 10% damping. For typical structures, a value of 5% damping is common. For a water tank supported on a cantilever column, on the other hand, a damping value of 0% might be conservatively used in column design.

Ground Amplification

Ground amplification causes an increase in damage especially in areas of soft ground. The 1989 Loma Prieta earthquake in California exhibited six times maximum amplification as a result of long-period shaking. It caused extensive damage to buildings in San Francisco's Marina district, which was largely built on filled ground.

Ductility and Stiffness of Structural Components

Building response is affected by ductility, which is defined as a structure's ability to deform or distort itself to dissipate or absorb seismic energy. Failure in steel and other construction materials occurs only after they experience considerable loss of ductility, which is affected by the following factors:

- Construction material
- Structural system
- Size and height
- Configuration
- Age and quality of construction

Brittle materials, such as unreinforced masonry or inadequately reinforced concrete, fail suddenly with a minimum of deformation, but heavy steel bars embedded in reinforced concrete with heavy and close

spacing can increase ductility considerably. Member proportions, end conditions, and connection details play a part in how much ductility a material exhibits.

Ductility increases the reserve capacity of a material. The stress-strain curve reaches the elastic limit and permanent deformation is caused. Such ductile materials can take higher loading before complete failure.

Lateral forces are distributed in proportion to the stiffness of the resisting member. Stiffness is measured by the extent to which a structural member deflects, which is generally expressed as a fraction of the length of the member.

Induced Torsion Due to Nonsymmetry

Torsion is a twisting action in plane, which results in an undesirable and possibly dangerous concentration of stress. For structures, there is no torsion if the mass or weight is uniformly distributed in plane and the geometric center coincides with the center of mass. Torsional forces are created in a building when the location of the resisting elements and the arrangement of the building mass lack balance. Eccentricity between the center of mass and the center of resistance causes torsion.

1.9 APPLICATIONS OF SEISMIC DESIGN CODES

It has been observed time and again that buildings not designed to seismic code requirements start to collapse sooner in an earthquake than those designed adequately. In contrast, structures designed to seismic codes are likely to perform well and avoid immediate failure. Still, factors such as soil liquefaction may cause their foundations to settle and expose concrete piles, thereby reducing carrying capacity, jeopardizing safety, and increasing life-cycle and maintenance costs.

The International Code Council's *International Building Code 2010* (ICC 2010) and AASHTO's *LRFD for Highway Bridges* (AASHTO 2007)—to name just two examples—are updated at least every five years. Many state design codes are also frequently updated. A review of selected states shows that U.S. design practice comprises the combined codes of many states. Indeed, the objective of state codes is to supplement national codes, and national code-drafting committees have state representatives and circulate draft codes to state officials for comments before final

adoption. Still, a state code is applicable to the engineering and construction practices in its state only, even though it may be adopted by other states or incorporated in national codes.

1.9.1 Retrofitting Needs

A major event causes many casualties and great damage. These are a high price to pay for failing to incorporate seismic retrofitting into a disaster preparedness plan. For safety reasons, at least structures with high importance should be retrofitted to a certain level of safety until they can be replaced with modern seismic-resistant structures. *Retrofit* methods may vary for different types of structures. Also, separate codes may be needed for retrofit as opposed to new design.

New code formulas are likely to be based on seismology studies, soil types, and liquefaction effects. Semi-empirical design formulas must be backed up with laboratory tests on well-instrumented scaled models using shake tables as well as historical failure observations in the field. Unfortunately, few structures built in the past one hundred years were designed using scientific criteria such as those recommended in the latest design codes for buildings and bridges. This is a problem because seismic retrofit can be expensive, especially in areas where earthquakes are infrequent and of low intensity.

Notable Scientific, Research, and Code Organizations

These may be listed as follows:

American Association of State Highway and Transportation Officials (AASHTO)
American Concrete Institute (ACI)
American Institute of Steel Construction (AISC)
American Railway Engineers Maintenance Association (AREMA)
American Society of Civil Engineers (ASCE)
Applied Technology Council (ATC)
Broadband Seismic Data Collection Center (ANZA network)
California Geological Survey (CGS)
California Seismic Safety Commission (CSSC)
Consortium of Organizations for Strong Motion Observational System (COSMOS)
Consortium of Universities for Research in Earthquake Engineering (CUREE)
Earthquake Engineering Research Institute (EERI)
European Strong Motion Database (ISES)

Geological and Planetary Services of Caltech (GPS)
Global Seismic Hazard Assessment Program (GSHAP)
International Association of Earthquake Engineering (IAEE)
International Association of Seismology and Physics of Earth's Interior (IASPEI)
Multidisciplinary Center for Earthquake Engineering Research (MCEER)
National Center for Earthquake Engineering Research (NCEER)
National Earthquake Hazards Reduction Program (NEHRP)
National Earthquake Information Center (NEIC)
National Fire Protection Association (NFPA)
National Information Service for Earthquake Engineering (NISEE)
National Oceanic and Atmospheric Administration (NOAA)
National Seismological Teachers Association (NSTA)
Quick Epicenter Determination (QED)
Southern California Earthquake Center (SCEC)
Structural Engineering Association of California (SEAOC)
Structural Engineering Institute (SEI)
U.S. Army Corps of Engineers (USACE)
U.S. Department of Interior/National Park Service
U.S. Geological Service (USGS)
University of California Pacific Earthquake Research Center (UCPEER)

Figures 1.3 and 1.4 provide web addresses for major U.S. and international seismic organizations (see www.isc.ac.uk/ for others).

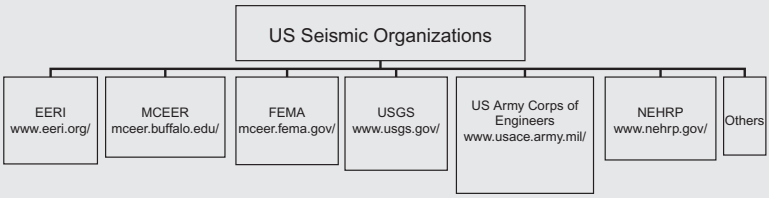

Figure 1.3 Website addresses of U.S. seismic organizations.

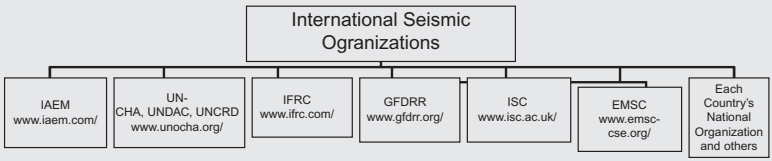

Figure 1.4 Web addresses for international seismic organizations.

1.10 THE ROLE OF THE U.S. GEOLOGICAL SURVEY

The U.S. Geological Survey is responsible for recording and reporting seismic activity nationwide. Statistics on size and frequency are made available on its website. Users of USGS information include the following:

- Local planners and building officials setting appropriate building and retrofitting standards.
- Professionals conducting detailed site assessments.
- Researchers engaged in engineering and seismologic research.
- State and local governments carrying out regional assessments of earthquake hazards.
- Partner institutions that use real-time links between USGS offices and worldwide seismographic stations to continuously monitor, detect, and locate earthquake activity.

1.10.1 Earthquake Monitoring and Reporting Capabilities through ANSS

Full implementation of the Advanced National Seismic System (ANSS) will result in nearly 7,000 new high-tech instruments in the ground or in buildings. Once in place, ANSS will provide near real-time information on intensity and distribution of ground shaking that can guide emergency response efforts. Similarly, information on building shaking will equip engineers with the data they need to improve building designs.

Funding Initiatives for Increased Seismic Resistance Activity

The National Earthquake Hazards Reduction Program (NEHRP), set up by Congress in 1977 to mitigate the effects of earthquakes, commissioned a body of scientists in 2008 to draw up a 20-year action plan for reducing earthquake hazards in America. Specific infrastructure projects for retrofit, bridge repair, public-transit expansion, and port development (based on seismic codes) are expected to be funded by federal, state, and local grants or loans, the municipal bond market, and, in some cases, the Highway Trust Fund or international banks.

Much of the hundreds of billion dollars in funds must come from alternate sources, most likely through cooperation, creativity, and leadership, according to a report by the Urban Land Institute. Public–private partnerships, in which

a private firm or joint venture builds and manages a road, a port, or other infrastructure, are another possibility. These partnerships will not privatize a project; rather, they will allow state and local governments to leverage local sources. The report suggests the following:

- Explicitly linking development and infrastructure
- Exploiting the possibilities of collaboration and partnership
- Tapping available federal funds to maximize investment opportunities
- Using infrastructure to its maximum potential of demand

Seismic Retrofit and Restoration

In a post-earthquake scenario, multiple large-scale projects will be carried out. Funds will be required and must be specially created, just as they are in a time of war for the country's defense. Besides a special federal budget, funds will come from voluntary organizations such as ASCE and many others. For International projects they will come from organizations such as NATO and the U.S. Agency for International Development (AID), which went to the aid of Pakistan after its 2005 earthquake, and the U.S. government, including the Nuclear Regulatory Commission (NRC), and ASCE, which offered technical assistance to Japan in the wake of the 2011 Tohoku tsunami.

1.11 CONCLUSIONS ON THE STATE OF ART

There have been notable developments in the science of earthquakes and in the technology of seismic design. These advances draw from many disciplines, including geology, seismology, soil science including liquefaction, and structural engineering using ductility principles of connections and reduction of mass of components. Zoning maps of a region reflect past seismic activity, location of a fault and distance to epicenter. Progress is being made in the development and refinement of seismic design codes and it must be continued. To accomplish this, engineers, engineering students, builders, and government officials need to keep abreast of updates in the seismic design provisions of organizations such as AASHTO, International Building Code and ASCE-07 Seismic Loads. New computer analysis and design software is now available. Instrumentation methods for measurements of essential test data is available from laboratory models. It is their responsibility to see that these provisions are implemented and enforced for safety, an essential requirement.

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Chen and Seawthorn 2003
Chopra 1995
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Doyle 1995

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Guha 2000
Paz 1994
Taranath 2005
Towhata 2008
Federal Emergency Management Agency (FEMA) 2006
Manuals and Handbooks
Federal Highway Agency (FHWA) 1987

CHAPTER TWO

Seismology and Earthquake Effects for Engineers

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2.1 INTRODUCTION

Earthquake science is a rapidly developing field. Its concepts are still rudimentary, and the accuracy of its theories cannot be ascertained or guaranteed as fully correct. Nevertheless, dynamic analysis procedures provided in structural codes of practice are based on the assumption that current fundamental theories are accurate. Thus, for an effective seismic-resistant design it is important to know the seismicity of the area under study.

Earthquake construction refers to seismic design that enables structures to survive anticipated earthquake exposure up to their builders' expectations and in compliance with applicable building codes. It requires the study of seismology and structural and geotechnical engineering, which is an important supporting discipline of geology. Structural engineers are not always familiar with seismic zoning, epicenters, and the influence of soil type, but a certain knowledge of seismology is helpful in planning, design,

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and site selection. Following are the general aspects of earthquakes for which geotechnical and structural engineering solutions are required:

- Ground shaking (USGS 2009)
- Liquefaction (Bozorgnia and Bertero 2004a,b)
- Landslides (Bozorgnia and Bertero 2004a,b)
- Fault offset distances (State of California 2010; California Geological Society 1999)

This chapter broadly reviews procedures related to seismology, which are directly linked to structural design engineering.

A detailed literature review shows contributions by the following organizations:

Fault, Rupture, Hazard Zones and Guidelines for Evaluating and Mitigating Seismic hazards in California (California Geological Society, 2002)

Financial Management of Earthquake Risk (EERI Endowment Subcommittee, 2000)

Understanding Your Risks, Identifying Hazards and Estimating Losses (Federal Emergency Management Agency (FEMA, 2001))

Standards of Seismic Safety for Existing Federally Owned or Leased Buildings (National Institute of Standards and Technology (NIST, 2002))

Seismology Committee Recommended Lateral Force Requirements and Commentary (Structural Engineers Association of California (SEAOC, 1999))

Earthquake Research at Parkfield, California for 1993 and Beyond (U.S. Geological Survey (USGS, 1994))

National Earthquake Prediction Evaluation Council Working Group.

Seismic Design Maps and Tools for Engineers (USGS/National Earthquake Hazards Reduction Program (NEHRP, 2009))

The following authors have contributed to several seismological aspects which are unique and based on research. Latest selected publications up to 2012 and the state of art related to seismology is presented here:

Empirical response spectral attenuation relations for shallow crustal earthquakes (Abrahamson and Silva, 1997)

Prediction probabilities from foreshocks (Agnew and Jones, 1991)

New ground motion relations for Eastern North America (Atkinson and Boore, 1995)

Earthquakes (Bolt, 1992)

Earthquake Engineering from Engineering Seismology to Performance Based Engineering (Bozorgnia and Bertero, 2004a,b)
Dynamics of Structures (Chopra, 1995)
Dynamics of Structures (Clough and Penzien, 1993)
Seismology (Doyle, 1995)
Real-time seismology at UC Berkeley: The Rapid Earthquake Data Integration Project (Gee et al., 1996)
Induced Earthquakes (Guha, 2000)
The Global Seismic Hazard Assessment Program (Giardini and Basham, 1993)
International Handbook of Earthquake and Engineering Seismology (Lee et al., 2002)
An Evaluation of the Seismic-Window Theory for Earthquake Prediction (McNutt and Heaton, 1981)
Practice of Earthquake Hazard Assessment (PEHA) (McGuire, 1993)
Earthquake Spectra and Design (Newark and Hal, 1982)
Earthquake spectra (Naomi, 2005)
Circum-Pacific Seismic Potentia (Nishenko, 1991)
An evaluation of the animal-behavior theory for Earthquake prediction (Schall, 1988)
National Landslide Hazards Mitigation Strategy, A Framework for Loss Reduction (Spiker and Gori, 2000)
Geotechnical Earthquake Engineering (Towhata, 2008)
Predicting Earthquakes (Watson and Watson, 1997)

In addition to above publications, relevant individual references are discussed in the text.

2.2 BASIC SEISMOLOGY

Earthquakes are part of a global tectonic process, based on geological formation, magnitude, and intensity of shaking. While many natural causes and environmental effects are responsible for earthquakes, sometimes lesser known human activity is at fault and cannot be ruled out. Examples are disturbances in the geology of the earth due to mining and changes in reservoir loads.

They occur in the crust or upper mantle at depths ranging from the earth's surface to about 500 miles. When strain along a fault within the

earth's crust slowly accumulates, the energy stored in elastically strained rocks is suddenly released. This causes intense ground shaking in the area near the source and sends waves of elastic energy, called *seismic waves*, throughout the earth.

Ground rupture, debris flows, and landslides are caused by earthquakes. Their occurrence and extent are determined by the following:

- Soil type
- Water content (degree of saturation)
- Gradient (slope angle)
- Triggering events

2.2.1 Some Terminology

Over the years, the terminology related to seismology and seismic-resistant design has grown. Common concepts such as *fault plane*, *epicenter*, and *focus* are illustrated in Figure 2.1. Other common terms are defined here:

- *Seismology*: the study of earthquakes and seismic waves that move through and around the earth; also the behavior of these waves.
- *Seismic waves*: vibrations of the surface of the earth caused by a rapid release of energy.
- *Focus* or *hypocenter*: the point of initial rupture.
- *Epicenter*: the point at ground level directly above the hypocenter.

2.2.2 Modern Theories of Earthquakes

Reid's Definition: Harry Fielding Reid, following Grove Karl Gilbert's study of the 1872 Owens Valley, California, earthquake, and after examining the fault trace of the 1906 San Francisco earthquake, deduced that earthquakes are the result of a gradual buildup of stresses in the earth

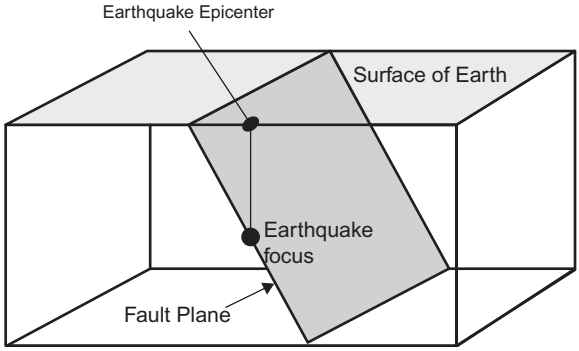


Figure 2.1 Earthquake focus in relation to earth's surface.

occurring over many years, leading to slippage along a fault that causes the fault to rupture. Such stresses are due to distant forces and are eventually violently released during an earthquake, allowing the earth to rapidly rebound after years of accumulated strain.

When a fault ruptures, elastic energy stored in the rocks is released, partly as heat and partly as elastic waves. Known as *elastic rebound*, this was believed by Reid to be the immediate cause of an earthquake, and his explanation has been confirmed over the years. Like a watch spring that is wound tighter and tighter, the more the crustal rocks are elastically strained, the more energy they store.

Elastic Rebound

As stated previously, when the slippage along a fault zone is hindered because of friction caused by the immense weight of a fault's tectonic plate, the elastic strain energy builds up in the deforming rocks on either side of the fault. When frictional forces are exceeded, slippage occurs and the release of energy causes an earthquake. According to Tarbuck, if the fault creeps, it will produce frequent *micro-earthquakes*. If it binds together and then slips, it will produce large earthquakes. Stress will then quickly be released, offsetting the sides of the fault.

According to Yeats, rocks rebound to their initial state of stress—hence, the *elastic rebound theory*. However, this theory is approximate. Earthquakes do not produce the large drop in stress required for rocks to rebound or to return to their initial state of stress.

Alternate Seismic Gap

The *alternate seismic gap theory* states that strong earthquakes are unlikely in regions where weak earthquakes are common. The longer the quiescent period between earthquakes, the stronger the earthquake will be when it finally occurs.

Plate Tectonics

Origin of Earthquakes: The geological model known as *plate tectonics* provides a coherent explanation for the majority of earthquakes. It combines the continental drift theory proposed in 1912 by Alfred Wegener in Germany. Alfred Wegener proposed that the continents had once been massed together in a single supercontinent and then gradually drifted apart. Continental drift and the more recent science of plate tectonics are now the bedrock of modern geology. The sea-floor spreading

concept was proposed by Harry Hess of Princeton University in November 1962.

Each tectonic plate extends to a depth of about 100 to 200 kilometers and includes the earth's outermost rigid rocky layer, called the *crust*. Plates appear to be in constant motion, and where they interact, along their margins, geological processes such as the formation of mountain belts, earthquakes, and volcanoes take place. The plates slide over the uppermost layer of the mantle. Seismologists describe earth's rigid outer shell (the *lithosphere*) as composed of fairly stable plates. Because the lithosphere completely covers the planet, ocean plates are involved in the process of *sea-floor spreading*.

All types of tectonic earthquakes are caused by the sudden release of elastic energy when a fault ruptures, with the fault's sides rapidly slipping in opposite directions. This slip does work in the form of heat and wave radiation and allows rock to rebound. Most of the geological action of earthquakes, faulting, volcanoes, and the formation of mountain and rift valleys is due to different interactions at tectonic plate boundaries (USGS, http://www.gldss7.cr.usgs.gov/neis/plate_tectonics/rift_man.html). Hazard studies have been published for collision zones of mountain ranges formed by continent-to-continent collisions. Examples are the Himalayan, the Zagros (Iran), and the Alpine ranges. Another example, in the United States, is the Cascadia Zone in western Washington, where the Juan de Fuca Plate slides under the American Plate. Ruptures along this zone have resulted in very large earthquakes that occur about every 500 to 600 years.

When an earthquake relieves the stress in one fault, it may pile additional stress on another one in the network. This contradicts the seismic gap theory because a series of small earthquakes in an area can increase the probability that a large quake will follow.

2.2.3 Faults

Types of Faults: *Dip-slip* and *thrust* faults are inclined fractures where the blocks have mostly shifted vertically. Normal faults are dip-slip faults and occur mainly in areas where the crust is being extended as a divergent boundary. A thrust fault is a reverse fault with a dip of 45 degrees or less. Reverse or thrust faults are dip-slip faults that occur in areas where the crust is being shortened, such as at a convergent boundary. *Oblique-slip faults* have components of both dip-slip and strike-slip faults. *Strike-slip* faults are steep structures where the two sides of the fault slip horizontally past each other.

A *fault scarp* is a line of cliffs formed by faulting and fracturing of the earth's crust. Grove Karl Gilbert, as mentioned, studied the fault scarp from the 1872 Owens Valley, California, earthquake and concluded that its faults were a primary, not a secondary, feature.

If the rock mass above an inclined fault moves downward, the fault is termed *normal*, whereas if the rock above the fault moves upward, the fault is termed *reverse*. Some faults are identified as active. The Emerson Fault in California is one example (Figure 2.2).

Earthquake storms are similar to aftershocks, but occur on adjacent segments of faults over the course of years. Where earthquakes strike a fault in clusters, each cluster is triggered by the shaking or stress redistribution of previous earthquakes.

Deep-located tremors, also called *tectonic tremors*, are a newly discovered type of seismic signal. They are less hazardous than earthquakes and occur at greater depths. They cause relatively weak ground shaking and, while they may last longer than earthquakes, do not represent any danger.

According to the research on earthquakes performed at German Karlsruhe Institute of Technology, earthquakes and tremors result from the relative movement on fault surfaces, a result of the motion of the tectonic plates. Researchers have recorded and are studying 2,600 tremor events.

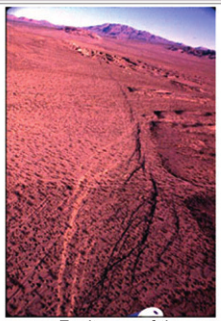
While earthquakes typically occur at depths of approximately up to 10 miles below the earth's surface, tectonic tremor signals are generated at depths ranging approximately from 10 to 18 miles. They differ from earthquakes, which makes them difficult to detect. The tectonic tremor generated on the deep part of a fault may provide clues about the more shallow parts of a fault where more damaging earthquakes occur.

2.2.4 Crustal Plates

According to Zebrowski, the earth's crust is part of a collection of well-defined crustal plates that grind past each other, move under and over each other, and recede from each other. There are seven major crustal plates up to 80 kilometers thick, which can be subdivided into a number of smaller plates moving at rates varying from 10 to 130 millimeters per year; they are all in constant motion relative to one another.

2.2.5 Interplate Earthquakes

Earthquakes in tectonically active boundary regions are referred to interplate earthquakes. The 1964 Alaska earthquake was in such a region and so far is



The Emerson Fault, one of the segments that ruptured in the m7.2 1992 Landers, Calfornia, earthquake. (Photo by Kerry Sieh, Caltech)

Active Fault

An active fault is a fault that is likely to have another earthquake sometime in the future. Faults are commonly considered to be active if they have moved one or more times in the last 10,000 years.

Figure 2.2 Aerial view of the active Emerson Fault in California.

the largest recorded event in the United States. The belts of very hazardous shallow-focus earthquakes in South America (Chile, Peru), America (California, Southern Alaska, and the Aleutians), Asia (Alpine-Caucasian-Himalayan, Japan, Taiwan, the Philippines, and Indonesia), the eastern Caribbean, Central America, Southern Mexico, and New Zealand are of the *plate-edge* type. In these regions, plates slide past each other along strike-slip or *transform* faults. In California, for example, all of the known damaging earthquakes have been shallow focus with depths of less than 230 feet. Shallow-focus earthquakes wreak the most devastation, and they contribute about three-quarters of the total energy released throughout the world.

2.2.6 Seismic Zones

Using past seismic events, plate tectonic activity can be classified into four seismic zones: Zone 1 follows the line of mid-ocean ridges. Seismic activity is low and occurs at very shallow depths. Zone 2 is associated with shallow-focus events unaccompanied by volcanic activity. The San Andreas Fault is a good example, as is the Anatolian Fault in Northern Turkey. In these faults, two mature plates scrape by one another. The friction between them can be so great as to cause very large strains that are periodically relieved by large earthquakes. The 1906 San Francisco earthquake was caused by breakage only along the northern end of the San Andreas Fault.

Zone 3 includes oceanic and continental plates. This type of earthquake can be shallow, intermediate, or deep, according to its location on the down-going lithospheric slab. In this zone one plate is thrust, or subducted, under another plate to produce a deep ocean trench. Along the Peru—Chile trench, the Pacific Plate is being subducted under the South American Plate, which responds by crumpling. The Andes mountain range was formed in this way.

Zone 4 occurs along the boundaries of continental plates. Typical of this zone is the broad swath of seismicity from Burma to the Mediterranean, crossing the Himalayas, Iran, and Turkey to Gibraltar. Here shallow earthquakes are associated with high mountain ranges where intense compression is taking place. Intermediate- and deep-focus earthquakes also occur, as in the Himalayas and the Caucasus.

2.2.7 International Seismic Belts

Earthquake-prone geographic regions tend to be narrowly concentrated. History shows that earthquakes occur in the same general pattern over time, but are principally located in three large *seismic belts.*

The *Circum-Pacific seismic belt*, familiarly known as the "ring of fire," is the world's most active zone. It is found along the rim of the Pacific Ocean, where the majority of the world's largest earthquakes are known to occur. The geographical spread encompasses

- South of equator Chile northward along the South American coast
- Through Central America and Mexico
- The West Coast of the United States and the southern part of Alaska
- Through the Aleutian Islands to Japan
- The Philippine Islands and New Guinea
- The island groups of the Southwest Pacific
- New Zealand

The *Alpide belt* is the second most important belt, accounting for the world's most destructive earthquakes. Its geographical spread follows

- Java to Sumatra
- Through the Himalayas
- The Mediterranean
- Out into the Atlantic

The submerged Mid-Atlantic Ridge is the third seismic belt, in which tremors and shocks are scattered in various areas.

Occasionally damaging shocks occur outside of these three belts, whenever sleeping faults become active. Well-known examples in the United States are the nineteenth-century earthquakes in New Madrid, Missouri, and Charleston, South Carolina. Decades to centuries usually elapse between such destructive shocks.

2.3 INDUCED SEISMICITY

There are other causes of seismicity besides tectonic plate movements. Induced earthquakes of varying magnitudes between 5 and 7 have been recorded. Such seismicity results from an increase or decrease in soil pore pressure that may cause limited disturbance to the earth's substrata and result in soil movement local to structural foundations. The epicenter is likely to be at a shallower depth than that for a natural earthquake. Generally, induced quakes seem to act independently from those generated by natural causes.

Induced earthquakes can result from both natural and man-made events (Bolt 1992). They are described in the following sections.

2.3.1 Naturally Induced Earthquakes

Earthquakes Associated with Volcanic Activity: Earthquakes can occur in volcanic regions because of tectonic faults and the movement of magma. Earthquake swarms can serve as markers for the location of flowing magma throughout the volcanoes. These swarms can be recorded by seismometers and tiltmeters (devices that measure ground slope) and used to sense imminent or upcoming eruptions.

Most volcanic activity is likely to generate earthquake tremors of varying magnitude. Such volcano-induced seismic activity has been observed at the following locations:

- Kilauea in Hawaii: Kilauea's current volcanic eruption dates back to January 3, 1983, and is by far its longest-lived historical period of activity, as well as one of the longest-lived eruptions in the world; as of January 2011, the eruption has produced 3.5 cubic kilometres of lava and resurfaced 123.2 km^2 of land.
- Mount St. Helens in Washington: Mount St. Helens is most notorious for its catastrophic eruption on May 18, 1980, the deadliest and most

economically destructive volcanic event in the history of the United States. Fifty-seven people were killed; 250 homes, 47 bridges, 15 miles of railways, and 185 miles of highway were destroyed. A massive debris avalanche triggered by an earthquake measuring 5.1 on the Richter scale, caused an eruption, reducing the elevation of the mountain's summit from 9,677 ft to 8,365 ft and replacing it with a 1 mile wide horseshoe-shaped crater.

- Pavlof, Alaska: Pavlof Volcano is a strato-volcano of the Aleutian Range on the Alaska Peninsula. It has been one of the most active in the United States since 1980, with eruptions recorded in 1980, 1981, 1983, 1986–1988, 1996–1997, and an eruption starting on August 15, 2007 and ending on September 13, 2007.
- Long Valley Caldera in California: In May 1980, a strong earthquake swarm that included four Richter magnitude 6 earthquakes struck the southern margin of Long Valley Caldera associated with a 10 in dome-shaped uplift of the caldera floor. This ongoing unrest includes recurring earthquake swarms and continued dome-shaped uplift of the central section of the caldera (the resurgent dome) accompanied by changes in thermal springs and gas emissions.

  In 1982, the United States Geological Survey under the Volcano Hazards Program began an intensive effort to monitor and study geologic unrest in Long Valley Caldera. The goal of this effort is to provide residents and civil authorities in the area reliable information on the nature of the potential hazards posed by this unrest and timely warning of an impending volcanic eruption, should it develop. Most volcanic eruptions are preceded and accompanied by geophysical and geochemical changes in the volcanic system. Common precursory indicators of volcanic activity include increased seismicity, ground deformation, and variations in the nature and rate of gas emissions.
- Mounts Vesuvius and Etna and the Phlegraean Fields in Italy: Mount Vesuvius is best known for its eruption in AD 79 that led to the burying and destruction of the Roman cities of Pompeii and Herculaneum. An estimated 16,000 people died due to hydrothermal pyroclastic flows.

  Vesuvius has erupted many times since and is the only volcano on the European mainland to have erupted within the last hundred years. Today, it is regarded as one of the most dangerous volcanoes in the world because of the population of 3,000,000 people living nearby and its tendency towards explosive eruptions. It is the most densely populated volcanic region in the world.

Lunar Tidal Earthquakes

Earthquakes may be caused by changes in lunar tidal forces at new and full moons. Tide-generating forces are maximum when the moon is at the zenith and minimum when it is at the horizon. Examples of such seismic activity were observed in Trans-Caucasia between 1917 and 1950.

2.3.2 Man-Made Earthquakes

Human activity can initiate earthquakes through increased weight or pressure on soils, contour disturbance, or holes deep in the soil layers. These earthquakes are generally minor and have been documented in a few locations in the United States, Japan, and Canada. Examples of man-made earthquakes and induced seismicity are listed below:

Reservoir-Dam-Induced Seismicity

Deepwater reservoirs created by damming exhibit induced seismicity, the extent of which is influenced by the rate of filling, hydrogeology, fracture pattern, and fault zone. Reservoirs in volcanic terrain with deep waters are more prone to earthquakes. Examples include

- The Aswan Dam in Egypt: On July 21, 1970, the Aswan High Dam, a water reservoir holding about four times the amount of water in the Hoover Dam, was completed. The dam has served as a producer of hydroelectric power for the Nile region, where 95% of the population of Egypt resides. Rapid changes in the water surface elevation can cause tremors.
- The Kurobe Dam in Japan: The Kurobe Dam, Japan's largest dam, is on the Kurobe River in Toyama Prefecture on the island of Honshū. It generates electricity for the Kansai Electric Power Company. It stands 186 metres high and holds 200,000,000 cubic metres of water. Its construction, completed in 1963 at a cost of 51.3 billion yen, claimed the lives of nearly two hundred people.
- The Kariba Dam in Zambia: The double curvature concrete arch dam was constructed between 1955 and 1959 at a cost at a total cost of $480,000,000. 86 men lost their lives during construction.
- The Koyna Dam in India: The dam, after it was strengthened in 2005–06, can withstand a quake as strong as 7 on the Richter Scale. At both the Kremasta Dam in Greece (1965) and the Kariba Dam in Zimbabwe-Zambia (1961), the generating mechanism was dip-slip on normal faults. The dam has withstood many earthquakes in the recent past, including the devastating 1967 Koynanagar earthquake,

resulting in the dam developing some cracks. After the disaster grouting of the cracks was done. Also internal holes were drilled to relieve the hydrostatic pressures in the body of the dam. Indian scientific establishment has formulated an ambitious project to drill a deep bore hole in the region and intensely study the earthquake activity.

- The Hoover Dam in the United States: Hoover Dam, once known as Boulder Dam, is a concrete arch-gravity dam in the Black Canyon of the Colorado River, on the border between the US states of Arizona and Nevada. It was constructed between 1931 and 1936 during the Great Depression and was dedicated on September 30, 1935, by President Franklin D. Roosevelt. Its construction was the result of a massive effort involving thousands of workers, and cost over one hundred lives. The dam was controversially named after President Herbert Hoover. Hoover Dam impounds Lake Mead, and is located near Boulder City, Nevada, about 25 mi Southeast of Las Vegas, Nevada.
- The Zipingpu Dam in China: The 7.9 magnitude quake on May 12, 2008 caused some damage to the dam, with the dam having a few cracks and fissures. Geologists now believe that the dam could have led to the massive earthquake.

In May 2008, an earthquake in China's Sichuan Province resulted in 69,227 fatalities and was the 19th deadliest earthquake of all time. The Zipingpu Dam is believed to have fluctuated the pressure of a fault 1,650 feet (503 meters) away; this probably increased the pressure on the fault and accelerated its rate of movement.

Seismicity Associated with Energy Technologies

Overdepletion of large built-in volumes of oil and gas has been found to trigger soil movements, causing damage to structures and oil installations. Examples of such seismic activity are the following:

- Coalinga, Oil Field California
- Sleepy Hollow Oil Field, Nebraska
- Lacq Oil and Gas Field, France
- Gasli Gas Field, Uzbekistan

Researchers have known since the 1920s that injecting or withdrawing fluids from the ground can cause earthquakes, but the issue has come to the fore as states such as Arkansas, Ohio, and Texas have experienced small tremors associated with wastewater disposal from oil and gas excavation.

The total balance of fluid injected into or removed from the ground is the biggest factor in causing earthquakes related to energy development.

Soil excavations from deep wells for oil and gas exploration trigger seismicity, causing damage to structures and oil installations in waste disposal sites and oil and gas fields. Pore pressure and proximity to fractures and faults contribute to seismicity. Examples include:

- The Rocky Mountain Arsenal in Colorado
- The Reef Oil Field in Texas
- The Calhio Injection Well (near the Perry Nuclear Power Plant) in Ohio

A 2012 report from a National Research Council study examined the risk associated with these energy technologies—including shale gas recovery, carbon capture and storage, geothermal energy production, and conventional oil and gas development—to cause earthquakes. "Although induced seismic events associated with these energy technologies have not resulted in loss of life or significant damage in the United States, some effects have been felt by local residents and have raised concern about additional seismic activity."

Regarding the seismicity associated with energy production generally, among the National Research Council's findings are the following:

- Fracking does not pose a high risk for seismic events; however, a 2.3 magnitude earthquake in Blackpool, England, was officially linked to fracking for shale gas.
- While wastewater disposal wells pose seismic risks, earthquakes resulting from them have been rare and typically less than 5.0 in magnitude.
- More research is needed to better quantify the risks associated with energy production; best practices should be developed to help prevent or mitigate seismic events.

Seismicity Associated with Underground Carbon Dioxide Storage

Storing carbon dioxide underground as a way to curb climate change could cause greater stress on faults and subsequently result in earthquakes. (Zoback 2012).

Rising pressure from the enormous amounts of CO_2, which would have to be stored for centuries to a few thousand years, could trigger seismic activity, but it could also be strong enough to crack rock above the formations used for storage, providing pathways for the buoyant

CO_2 to leak back into the atmosphere. The volumes that would have to be injected are so enormous that CO_2 storage may well be impossible because of the triggered-earthquake problem in many parts of the world.

Mining-Induced Seismicity

The magnitude of mining-induced earthquakes depends upon faults, mine depth, and production rate. Examples of such seismic activity include

- A silver mine in Idaho
- A quarry in New York
- Virginia coal mines
- The Klerksdorp Gold Mine in South Africa
- The Kolar Gold Field in India

The greatest earthquake in Australia's history was induced by coal mining. The city of Newcastle rests on a large sector of coal mining areas. This earthquake was spawned by a fault that reactivated as a result of the removal of millions of tons of rock in the mining process.

Micro-Seismicity Related to Underground Nuclear Explosions

Underground nuclear explosions trigger fault displacements and movements and thus the release of tectonic strain energy. Examples of such seismic activity were observed in the Boxcar and Benham nuclear explosions in Nevada and on Amchitka Island. Above-ground nuclear detonations have not been linked to seismic activity because energy from nuclear blasts dissipates quickly along the earth's surface.

The effects of induced seismicity are currently not considered in seismic codes, although they should be investigated for buildings and bridges. Ground motion and fault movement need to be monitored, especially at bridge locations, by seismographs or other sensitive instruments.

2.4 WAVE GENERATION AND COMPOSITION

Seismic ground motions in solid rock or soil during earthquakes generate an array of body waves that create shaking that can be felt and that cause damage. These waves are of three types. *P waves* (shock or pressure waves) are the first to occur and always travel longitudinally. *S waves*

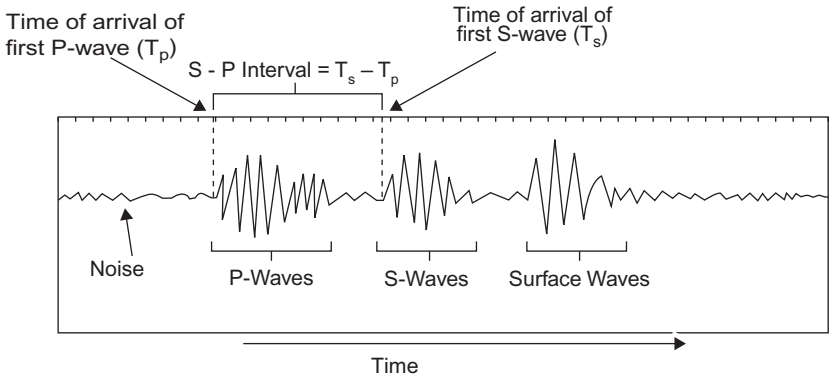


Figure 2.3 Sequence of seismic wave generation.

(secondary or shear wave waves) are the second to occur and travel transversely. *Surface waves* (Rayleigh and Love waves) occur after the P and S waves and travel only through the crust.

Phenomenon: Every tremor produces different types of seismic waves, which travel through rock with different velocities (Figure 2.3). P waves, just like acoustic waves, can travel through solid rock such as granite as well as alluvium soils and liquids such as volcanic magma or lake and ocean water.

Earthquake waves are much affected by soil elastic properties and by the three-dimensional nature of underground geological structures. In weathered surface rocks, alluvium, and water-saturated soil, the relative sizes of P, S, and surface waves can vary significantly, depending on wave frequency, as they propagate through surficial nonhomogenous geological structures.

Because S waves depend on elastic shear resistance, they cannot propagate in liquids such as water bodies and are confined near the earth's surface. The velocity of *S waves* ranges from 3 to 5 feet per second in light sediments, from 6 to 8 feet per second in the crust, and up to 12 feet per second in the deep mantle. S waves are of a lower frequency than P waves and are easily distinguished on a seismogram.

Love Wave: The first type of surface wave is called a *Love wave*, after A. E. H. Love, a British mathematician who worked out the mathematical model for this kind of wave in 1911. The fastest surface wave, the Love wave moves the ground from side to side in a horizontal plane parallel to the earth's surface but at right angles to the direction of propagation. Its motion is the same as that of S waves that have no vertical displacement.

Rayleigh Wave: The second type of surface wave is called a *Rayleigh wave*, named for Lord Rayleigh, who mathematically predicted the existence of this kind of wave in 1885. It is the cause of most of the shaking felt, which can be much greater than that caused by the other waves. A Rayleigh wave rolls along the ground just as a wave rolls across a lake or an ocean. Because it rolls, it moves the ground up and down and side to side in the same direction of its movement.

As seismic body waves (P and S waves) move through layers of rock or soil, they are reflected or refracted at the layer interfaces. Surface waves travel more slowly than P and S waves, and Love waves travel faster than Rayleigh waves in the same geological formation.

Love and Rayleigh waves are almost entirely responsible for the damage and destruction associated with earthquakes epicenters located close to the earth's surface. Deeper earthquakes lessen the strength of these waves and thus reduce this damage.

2.4.1 Travel Time of P and S Waves

In the earth's interior, P waves travel much faster than S waves. The approximate ratio of speed is 1.7 to 1 mile per second. The propagation velocity of these waves ranges from approximately 3 to 8 miles per second, depending on the density and elasticity of the medium. P waves are known to alternately compress and expand material in the same direction they are traveling. They are also known as compression waves because of their pushing and pulling action.

Soil particles, when subjected to a P wave, move in the direction of the wave, which is the direction in which the energy is traveling. In solid rock, P waves are known to travel at 4 to 5 miles per second. Velocity can increase within the deep mantle to 8 miles per second.

2.5 EARTHQUAKE PREDICTION AND FORECASTING

Locating epicenters and wave generations underground has not been successful. Although it is now known *where* earthquakes are likely to occur, there is currently no reliable way to predict exactly *when* in any specific location. Scientists estimate earthquake probabilities by studying both the history of large earthquakes in a specific area and the rate at which strain accumulates in rock, but these have not proved to be

effective indicators. Unfortunately, accurate diagnosis of the strain state of faults and the precise timing of large events may continue to elude us.

Current forecasting techniques can be used for well-understood faults, such as the San Andreas. In the 1980s, for example, an earthquake near Parkfield, California, was predicted likely within a decade. No forecasts, however, can be made for poorly understood faults, such as those that caused the 1994 Northridge, California, and 1995 Kobe, Japan, quakes.

One well-known successful prediction was the Haicheng, China, 7.3 magnitude earthquake in 1975, for which an evacuation warning was issued the day before. In the months prior to the event, changes in land elevation and groundwater levels, widespread peculiar animal behavior, and many foreshocks had been observed. In unfortunate contrast, there was no warning for the 1976 Tangshan magnitude 7.6 earthquake, which caused an estimated 250,000 fatalities.

Earthquake Prediction by Clusters

According to a report by catastrophe modeler *Eqecat*, evidence indicates that great earthquakes occur in clusters over a period of years and that a significant event is on the horizon. Thenhaus developed seismotectonic models as a member of the USGS National Seismic Hazards Mapping Team.

For example, from 1950 to 1965, three earthquakes of magnitude 9 or higher occurred:

- 1952: Kamchatka, magnitude 9
- 1964: Prince William Sound, Alaska, magnitude 9.2
- 1960: Chile, magnitude 9.5

According to Thenhaus et al., giant earthquakes of magnitude 8 or higher may occur in clusters over a decade or more. Such occurrences, they argue, "cannot be attributed to chance," noting that the giant magnitude 9.1 Andaman-Nicobar (Sumatra, Indonesia) earthquake of 2004 began "a new cycle of global great earthquake activity."

If future earthquakes follow the pattern of the 1900s, a third major earthquake is a very real possibility. So far this century, there has been the Indonesian magnitude 9.1 earthquake in 2004 and the magnitude 9 Tohoku earthquake and tsunami in 2011.

2.5.1 The Gutenberg-Richter Law in Predicting Earthquakes

Movement of Tectonic Plates: For millions of years, the forces of plate tectonics have shaped the earth, as huge plates that form the earth's

surface slowly move over, under, and past each other. Sometimes, however, the plates are locked together, unable to release the accumulating energy. When this energy grows strong enough, the plates break free.

In 1954, an important development in the study of the frequency and energy of earthquakes was the publication of the second edition of *Seismicity of the Earth and Associated Phenomena*, by Beno Gutenberg and Charles Francis Richter (1954), which introduced the Gutenberg-Richter law. This law gives the approximate relationship between the magnitude and total number of earthquakes in any given region. Thus,

$$\log N = a - bM$$

$$N = 10^{a-bM}$$

where N is the number of events having a magnitude equal to or greater than Ma, and b represents constants. The a value simply indicates the total seismicity rate of the region. The constant b is typically equal to 1.0.

Although, according to the Gutenberg-Richter law, earthquakes can occur almost anywhere, roughly ten times as many larger than magnitude 4 as those larger than magnitude 5 will occur in a particular time period. For example, for every single magnitude 4.0 event, there will be four magnitude 3.0 quakes and four magnitude 2.0 quakes. Because the Gutenberg-Richter relationship is exponential, larger earthquakes occur less frequently. In any case, more events are being reported than in the past not because of an increase in actual occurrence but because of the vast improvement in instrumentation networks.

Start of the Golden Era of Earthquake Forecasting

Plate tectonics may tell us where 90 percent of the major earthquakes are likely to occur but not when. In the 1960s, UCLA professor emeritus of statistics Vladimir Keilis-Borok studied seismic waves from underground nuclear explosions and compared them with those of earthquakes. This led to a boom and start in prediction studies in the 1970s to this most significant development in preparedness and mitigation.

In 1985, Keilis-Borok forecast that a quake would strike in the near future along the San Andreas fault. On October 17, 1989, the earth moved, interrupting a World Series game at San Francisco's Candlestick Park, collapsing a freeway in Oakland, and leaving 63 people dead. This was the Loma Prieta

earthquake. Keilis-Borok spent the ensuing 30 years with Ahab-like determination, trying to better what he considered a breakthrough. Essentially, Keilis-Borok's team of international scientists developed mathematical algorithms to try to discern patterns. Keilis-Borok believes that there is a trail leading up to strong quakes, and that it is a question of identifying those footsteps and how they differ from normal seismic activity.

More than a decade after the Loma Prieta quake, Keilis-Borok gained international attention. His group had appeared to score a hit when it predicted that a magnitude 6.4 or larger quake would strike between Fort Bragg and Cambria, California, a 310-mile stretch, before September 2004. In December 2003, a magnitude 6.5 quake struck six miles northeast of San Simeon, on the southern edge of the prediction boundary, killing two people. The prediction provoked chatter among scientists. The Seismological Society of America conference was in Palm Springs that April, and Keilis-Borok was invited to speak.

If the Mojave prediction had come true, some scientists said at the time, they would have no choice but to take Keilis-Borok's methods seriously. There has lately been a renewed interest in the idea of trying to predict quakes. However, a UCLA project has shown that predictions of underground activity cannot be true 100 percent of the time.

In 2007, Japan installed a network of 1,000 seismic stations as part of a national warning system to detect P waves, which are the first to be emitted by an earthquake and travel at twice the speed of the more destructive S waves. The objective is achieving valuable seconds of warning depending on the distance from the epicenter.

2.6 EARTHQUAKE-TRIGGERED TSUNAMIS

Definition: A tsunami is essentially a rapid rise in coastal sea level. Tsunami waves, which have a very long wavelength and travel at approximately 500 miles per hour in the open ocean, can cause unpredictable, high, and rapidly changing tidal-like inundation from 1 to 30 feet above the tide. As the waves approach shallow waters, their speed and wavelength decrease and their height dramatically increases.

The initial wave is followed by secondary waves for periods lasting up to eight hours. These secondary waves can be higher than the first wave and carry debris from the initial inundation, creating a lethal combination of flooding and battering.

Tsunamis, particularly the 2011 Tohoku, Japan, disaster, will be discussed in a later chapter. It is sufficient to say here that they are caused by

- Offshore earthquakes that displace the ocean bottom
- Earthquake-triggered or natural submarine landslides and slumps
- Volcanic eruptions
- Meteor strikes (very infrequently)

Tsunamis have been generated by distant earthquakes on the Pacific Rim, by volcanic eruptions or undersea landslides on near-coast continental shelves, and by near-shore earthquakes. They are not limited to the Pacific Coast, Hawaii, and Alaska. Earthquakes and volcanic eruptions can generate tsunamis along the U.S. southeast and gulf coasts and the Caribbean as well.

2.7 SEISMOLOGY-RELATED HAZARDS

Most large earthquakes occur in long fault zones around the margin of Pacific Ocean. The zones that ring the Pacific are subdivided by geologic irregularities into smaller fault segments, each rupturing individually. Friction controls the movement of tectonic plates at a fault. The lower the friction, the weaker the fault and the easier it displaces. Medium friction produces small earthquakes. High friction produces a fault that will slip, occasionally generating large earthquakes. The USGS and many state geological surveys have produced maps of these major active faults, all of which have ruptured within last 11,000 years.

Seismic risk assessment has traditionally involved peak ground acceleration (PGA), velocity, and displacement as functions of frequency or period. For example, using a set of assumptions about fault mechanics and rate of stress accumulation, the USGS predicted that an earthquake of about 6.0 magnitude would occur in Parkfield, California, between 1988 and 1992. Although this quake did not materialize until 2004, long after the prediction window had expired, estimating an earthquake using a dense network of instrumentation was a significant accomplishment and gave new insights on the mechanics of fault rupture.

2.7.1 Probabilistic Seismic Hazard Analysis

Definition: A seismic hazard is the probability of occurrence of a particular earthquake characteristic such as PGA. Based on geological and seismological studies, probabilistic seismic hazard analysis (PSHA) estimates

the likelihood of a hazard, considering the uncertainties in magnitude and the location of earthquakes and their resulting ground motions that are likely to affect a particular site. For statistical reasons, risk involves probabilistic values that are greater than expected.

2.7.2 USGS and UN Global Seismic Hazard Assessment Programs

The USGS and its partners in the multi-agency National Earthquake Hazard Reduction Program (NEHRP) are working to improve monitoring and reporting capabilities via the USGS Advanced National Seismic System (ANSS). Another project, the UN Global Seismic Hazard Assessment Program (GSHAP), a demonstration project of the UN/International Decade of Natural Disaster Reduction initiative, was carried out from 1992 through 1998. The GSHAP Global Seismic Hazard Map joins GSHAP regional maps to depict global seismic hazards as PGAs with a 4% chance of exceedance in 50 years, corresponding to a return period of 475 years. *Exceedance* refers to a range of values to be exceeded.

Landslide Hazard Maps

The USGS and the California Geological Survey (CGS) have prepared landslide hazard zone maps indicating areas where slope, soil type, and seismic risk could trigger landslides. Because downslope slides can undermine foundations and cut off utilities and access, rendering a structure nonoperational and structurally unsafe, construction in the Landslide Hazard Zone requires assessment by a geotechnical engineer.

2.7.3 Shake Maps and Response Spectra

Definition: A *shake map* is useful in seismic zoning. This technology is rapidly evolving as new advances in communications, earthquake science, and user needs drive improvements. Shake maps have become a valuable tool for emergency response, public information, loss estimation, earthquake planning, and post-earthquake engineering and scientific analyses.

Use of GIS: The USGS S*hakeMap* program produces a computer-generated representation of ground shaking. It represents a major advance for scientific and engineering purposes (see http://earthquake.usgs.gov/shakemap). Automatically generated shaking and intensity maps combine

instrumental measurements of shaking with information about local geology and earthquake location and magnitude to estimate shaking variations throughout a geographic area. The results are rapidly available online in a variety of map formats, including geographic information system (GIS) coverage.

Site Response Spectra

Structures with different periods or frequency responses react in widely differing ways to the same earthquake ground motion; conversely, any structure acts differently during different events. Thus, for design purposes the *site response spectrum* represents a structure's range of responses to ground motion of different frequencies for the peak accelerations.

Definition: A site response spectrum graph plots the maximum response values of acceleration, velocity, and displacement against period and frequency. It enables the engineer to identify the resonant frequencies at which a structure undergoes peak accelerations. According to NEHRP (1997), structural design might be adjusted to ensure that the building period does not coincide with the site period of maximum response.

Structural Response Methods

Besides the site response spectrum, other methods of evaluating structural response are step-by-step integration, equivalent lateral force, and soil–structure interaction. Depending on seismic intensity, structural response is evaluated by different mathematical methods. Methods specifically for the evaluation of buildings and bridges include base isolation and energy dissipation.

Objectives: Engineers and building code developers use site response evaluation methods to improve the safety of new and existing structures, thereby reducing ultimate risk. New design codes ensure that new structures are built with sufficient resistance to lateral forces and sufficient flexibility of movement.

2.8 SEISMOLOGY SOFTWARE

Structural engineers can appreciate the developments in seismology through the variety of seismological software available, some of current software is described here. With the availability of high-speed and large-storage computers, theories of seismology and seismic effects based on

many parameters are being programmed. The various types of seismic software include

- 3D velocity modeling
- Estimated ground motions for building design
- Locations and magnitudes
- Seismic hazard
- Engineering response
- Seismic travel time
- National seismic hazard maps and shake maps

The following software is representative of such developments and is being revised with time, as it is based on an approximate science with assumptions subject to change. For this reason, it is recommended that, if feasible, more than one software program be used for verification of results.

2.8.1 Seismic Risk Analysis and Software

REDARS 2 provides the basic framework of the seismic risk analysis (SRA) methodology and its modules. The main modules of REDARS 2 include hazards, components, systems, and economics. The northern Los Angeles highway system is used as a demonstration application of the SRA methodology (MCEER 2006). REDARS 2 Methodology and Software for Seismic Risk Analysis of Highway Systems, S.D. Werner, C.E. Taylor, S. Cho, J.P. Lavoie, C.K. Huyck, C. Eitzel, H. Chung and R.T. Eguchi MCEER-06-SP08 | 8/31/2006.

2.8.2 3D Velocity Modeling

Simulps12.for is used for 3D velocity model determination and hypocentral location with local data.

Ground Motion Parameter Calculator calculates estimated ground motions for building design. Hazard curves, uniform hazard response spectra, and design parameters are available for sites in all 50 states, Puerto Rico, and the U.S. Virgin Islands.

HYPOINVERSE is USGS earthquake location program that locates earthquakes and determines magnitudes in a local or regional seismic network.

HYPOINVERSE2000 determines earthquake locations and magnitudes from seismic network data such as P and S wave arrival times, amplitudes, and coda durations. Its latest version is in routine use by the Hawaiian Volcano Observatory, The Northern California Seismic Network, and many

other organizations. Because it is the standard location program supplied with the Earthworm seismic acquisition and processing system, it is in wide use. Multiple crustal models cover different regions and either flat layer or flat layer with linear velocity gradients.

USGS supported *MacR1D* is a one-dimensional seismic travel-time calculator for Macintosh. Travel time versus distance is calculated for P and S wave arrivals for sources at any depth and may be compared to observed travel times. MACR1D provides a quick way to define 1D velocity models from observed arrival pictures and can be down loaded.

OpenSHA is an object-oriented, web- and GUI-enabled, open-source code for conducting seismic hazard analyses (SHA). The goal is to provide a framework in which any arbitrarily complex earthquake-rupture forecast, ground-motion, or engineering-response model can "plug in" for analysis.

2.8.3 Probabilistic Seismic Hazard Calculation Software

SATSI (Spatial And Temporal Stress Inversion) is a modified version of Michael's (JGR 1984, 1987) code (http://gcmd.nasa.gov/records/USGS_SATSI.html) that inverts focal mechanism data for a spatially and/or temporally varying stress field. The inversion finds the least complex stress field model that is consistent with the data, and uses an adaptive smoothing method that discriminates variations that are and are not strongly required by the data, retaining only those that are well resolved.

ShakeCast "Lite" delivers automated shake maps of affected areas, facilitating use of ShakeMap products. Areas of interest can be defined, and shaking thresholds can be set to trigger automatic notifications.

2.9 CONCLUSIONS ON SEISMOLOGY AND EARTHQUAKE EFFECTS

It is a complex subject for geologists and seismologists. For engineers the relevant issues in static seismic analysis and practical seismic-resistant design can be summarized as seismology, induced-seismicity underground waves, reservoir and mining induced seismicity and Seismicity Associated with Energy Technologies have not received enough attention so far. Seismic belts, faults, earthquake prediction and forecasting, seismic hazards, shake maps, tsunamis and acceleration response spectra.

Continuing seismological research is essential to the development of the most effective seismic designs. Investment in earthquake prediction and forecasting will yield dividends, as will more specialized information and data on location of faults, identification of seismic belts, natural and man-made—induced seismicity, and comprehensive seismic design.

Continued refinement of seismic hazards maps, shake maps, response spectrum graphs, and seismic zone maps is equally important, along with improvements in seismology software.

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CHAPTER THREE

Seismic Response of Structures to Liquefaction

Chapter Outline

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3.1 INTRODUCTION

This chapter broadly reviews procedures related to soil behavior during earthquakes and liquefaction in some types of soil, which are directly linked to foundation design engineering.

A detailed literature review shows contributions by the following organizations and design codes:

American Railway Engineering and Maintenance-of-Way Association (AREMA, 2012)

International Code Council (ICC, 2003) Metropolitan Transportation Authority (MTA, 2003)

The following authors have contributed to several soil aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to soils response in earthquakes is presented here:

Ground Improvement (Moseley and Kirsch, 2005)

General Theory of Earth Pressures (Okabe, 1926)

A study on Probabilistic Evaluation of Soil Liquefaction (Chi and Ou, 1999)

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The Geology of Earthquakes (Yeats et al., 1997)

Seismic Analysis and Design of Continuous Prestressed Concrete Bridges (Yost et al., 2004)

Proceedings of 1st International Conference on Earthquake Geotechnical Engineering, Tokyo, 1995.

In addition to above publications, relevant individual references are discussed in the text.

Definition: Liquefaction is the process by which a soil suddenly loses strength, most commonly as a result of ground shaking during a large earthquake. Because not all soils liquefy in an earthquake, soil type plays an important role in seismic activity and thus in the development of seismic design codes. The focus of this chapter is on liquefaction as an effect of seismic activity.

Earthquake waves are generated underground, and the vibration effects are transmitted through soil layers above hard rock. As they pass from rock to soil, their velocity decreases but their size increases. Certain soils greatly amplify shaking, and different kinds of geology behave differently according to magnitude. For example, shaking at a site with soft sediments can last three times as long as that at a stable bedrock site such as one composed of granite. A soft, loose soil shakes more intensely than hard rock, even though both may be located at the same distance from the same earthquake. Particle size defines the basic soil types, which are characterized as sand, silt, and clay. Biological and chemical agents, weather and climate, parent material, time, topography, and vegetation determine soil type.

3.2 CHARACTERISTICS OF SOILS AND SITE EFFECTS

Soil characteristics and their effects on structure sites are governed by amount of moisture; amount of pore space filled with air; color; chemical composition; grain size, shape, and hardness; and permeability. A soil profile is a cross-section of soil layers with different characteristics. The four profile types vary in strength from very firm (Type I) to soft clay (Type IV) and are classified as follows:

Type I (rock of any description). Either shale-like or crystalline, or stiff where the soil depth is less than 200 feet. Soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Type II (stiff cohesive soil or deep cohesionless soil). Soil depth exceeds 200 feet. Like Type I, Type II soils overlying rock are stable deposits of sands, gravels, or stiff clays.

Table 3.1 Soil Profile Type Based on Shear Wave Velocity (BSSC, 1994, *NEHRP Recommended provisions*)

Profile Type	Shear Wave Velocity (ft/sec)
Hard rock	>5,000
Rock	2,500–5,000
Soft rock	1,200–2,500
Stiff soil	600–1,200
Soft soil	<600

Default Type II. Any soil whose properties cannot be sufficiently determined or that do not fit any of the four other types.

Type III (soft to medium-stiff clays and sands). Characterized by 30 feet or more of soft to medium stiff clays with or without intervening layers of sand or other cohesionless soil.

Type IV (soft clays or silts greater than 40 feet in depth). Characterized by a shear wave velocity of less than 500 feet per second and may include loose natural deposits or man-made, non-engineered fill (Table 3.1).

3.3 SOIL TYPE AND THE PROCESS OF LIQUEFACTION

Classification of Soils: The soils that can potentially liquefy are sands and silts that rest quite loosely in the ground. They are found in areas of unconsolidated alluvium; near coastlines, rivers, and streams; and where groundwater is about 10 feet beneath the surface. Such soils do not stick together the way clay soils do. They are below the water table, so all of the spaces between the grains are filled with water, as shown in Figure 3.1. (Dry soils above the water table do not liquefy.) When seismic shaking is rapid and violent, sand and silt grains act to compress the spaces filled with water, but the water pushes back and pressure builds up until the grains "float." Once that happens, the soil loses its strength and liquefies.

Soil that was once solid behaves as a fluid and cannot support the weight of whatever is lying above it, whether the surface layers of dry soil or the concrete floors of structures. The liquefied soil under that weight is forced into any cracks and crevasses it can find, including those in the dry soil above and those between concrete slabs. It flows

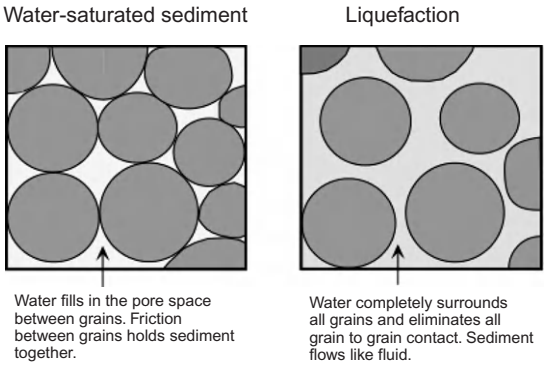

Figure 3.1 Saturation and liquefaction in grains.

out onto the surface as boils, sand volcanoes, and rivers of silt. Water-saturated soils, sands, or gravels, like liquids, flow laterally or vertically. Lateral liquefaction flows typically result in long tears and rips in the ground surface that look like classic fault lines. In some cases, the liquefied soil flowing up a crack can erode and widen it to a size that can accommodate a car.

Void Ratio: Depending on the initial void ratio, soil material can respond to loading that is either strain-softening or strain-hardening. Strain-softened soils—loose sands, for example—can be triggered to collapse either monotonically or cyclically if the static shear stress is greater than the ultimate or steady-state shear strength of the soil. Some examples are quicksand, quick clay, and turbidity currents. The resistance of the cohesionless soil to liquefaction depends on density, confining stresses, soil structure (age and cementation), magnitude and duration of cyclic loading, and extent of shear stress reversal.

Quicksand: Quicksand forms when upward-flowing underground water is trapped in loose sand, causing agitation. The trapped water creates liquefied soil that can no longer support weight. Objects in the liquefied sand sink to the level at which their weight is equal to the weight of the displaced sand/water mix; buoyancy causes them to float (see Figure 3.2).

Quick Clay: Quick clay is found only in northern areas such as Russia, Canada, Alaska, Norway, Sweden, and Finland. Also known as Leda clay in Canada, it is a unique form of highly sensitive clay, with the tendency to change from relatively stiff to liquid when it is

Sand ejected through a crack forming a series of sand boils along the railroad tracks adjacent to Deschutes Parkway in Olympia. (Photo courtesy of geomatrix)

Figure 3.2 Quicksand.

disturbed. Although a solid, quick clay has very high water content, up to 80%. When the structure containing it is broken by a shock, the clay reverts to a fluid. Quick clay has been the underlying cause of many deadly landslides—more than 250 mapped landslides in Canada alone.

After the 1811–1812 New Madrid, Missouri, and 1886 Charleston, South Carolina, earthquakes, widespread liquefaction resulted in severe damage. In the 1964 Alaskan and 1971 San Fernando earthquakes, it led to substantial structure settlement and caused major disruptions of services and utilities. In 1987, the magnitude 5.9 Whittier Narrows earthquake pounded an apartment building in Monterey Park, California, shifting its superstructure about 10 inches to the east on its foundation.

3.4 LIQUEFACTION AND STRUCTURAL INTEGRITY

Not all of a structure's foundations may be affected by liquefaction, but those that are may subside (settle) or be pulled sideways by lateral spreading. Liquefaction is the most significant cause of damage to bridge structures in this manner and has been related to lateral movement of soil at bridge abutments and to loss of lateral and vertical bearing support of

bridge pier foundations. The foundations can tilt or sink into liquefied deposits.

3.4.1 Evaluation of Potential for Liquefaction

NCHRP has defined six seismic detailing requirements (SDR 1 through 6) and four seismic hazard levels. If peak ground acceleration is less than 0.15 g, the magnitude causing liquefaction is generally less than 6 and the duration of ground shaking will be relatively short. Liquefaction in such cases develops slowly at most sites and its potential is low for SDR levels 1 and 2. Evaluation of potential for liquefaction is not required in these cases.

The potential for liquefaction in SDR levels 3 through 6 is higher. However, a liquefaction evaluation may be required only if directed by the site owner or if certain conditions related to magnitude, SPT blow count, and 75-year event obtain.

3.4.2 Liquefaction Susceptibility

Methods: Liquefaction susceptibility can be determined from USGS liquefaction hazard maps, site geologic investigations, and review of geologic/soil maps and borehole logs. The effects of liquefaction on foundations as well as structures must be thoroughly investigated and mitigation measures undertaken if the impact analyses yield unacceptable performance results such as

- Post-liquefaction (post-earthquake) stability and deformations of embankments
- Loss of bearing capacity and settlements of shallow footings
- Down-drag and lateral/vertical resistance of deep foundations
- Increased lateral pressures and buoyancy forces on walls and underground structures
- Lateral spreading along gentle slopes

The resistance of a structure to liquefaction depends on foundation type, structure configuration, structural material, and construction detailing. For example, bridges with discontinuous superstructures and non-ductile supporting members have a higher liquefaction potential than do those with continuous superstructures. The probability of liquefaction of the soil underlying bridges is determined as follows:

- Earthquake-induced bedrock accelerations
- Soil profile type

- *N*-value from soil borings
- Remaining life of the structure

Lateral forces induced by lateral movement in soil by flow sliding or lateral spreading can be measured for either of two hazard levels: operating design earthquake (ODE) and maximum design earthquake (MDE), which form the basis of the design criteria of the New York State (NYDOT) and New York City departments of transportation for seismic design and retrofit of critical bridges. They are defined as follows:

- ODE: Return period of 500 years; no damage to primary structural members or interruption in service; damage to secondary structural members is minor and repairable.
- MDE: Return period of 2,500 years with a small probability of exceedance; strength and ductility must be adequate to maintain life safety and avoid collapse; any structural damage is easily accessible and can be readily repaired.

There can be significant variation in the results based on these hazard levels, which can be obtained by simplified methods or site-specific analyses. Useful and practical examples of relationships between site class and spectral acceleration values are given in Table 3.2 for MDE and in Table 3.3 for ODE. (Note that in the tables and elsewhere, profile types A through F correspond to the profile types I through V described previously.)

Table 3.2 NYDOT Spectral Acceleration Values for MDE-Level Design

Period (sec)	Site Class A (hard rock) (g)	Site Class B (rock) (g)	Site Class C (soil) (g)	Site Class D (soil) (g)	Site Class E (soil) (g)
0.0 (PGA)	0.24	0.31	0.34	0.37	0.37
0.04	0.58	0.72	0.80	0.87	0.87
0.1	0.51	0.64	0.80	0.87	0.87
0.2	0.40	0.50	0.80[a]	0.87[b]	0.87
0.5	0.20	0.24	0.43	0.59	0.87[c]
1.0	0.10	0.13	0.22	0.30	0.44
2.0	0.05	0.06	0.11	0.15	0.22
5.0	0.02	0.02	0.04	0.06	0.09

Note: For site class F (soil) a site-specific study is required. PGA = peak ground acceleration.
[a]Constant up to 0.27 sec.
[b]Constant up to 0.34 sec.
[c]Constant up to 0.51 sec.

Table 3.3 NYDOT Spectral Acceleration Values for ODE-Level Design

Period (sec)	Site Class A (hard rock) (g)	Site Class B (rock) (g)	Site Class C (soil) (g)	Site Class D (soil) (g)	Site Class E (soil) (g)
0.0 (PGA)	0.06	0.08	0.09	0.12	0.19
0.04	0.15	0.18	0.22	0.29	0.45
0.1	0.15	0.18[a]	0.22	0.29	0.45
0.2	0.10	0.13	0.22[b]	0.29[c]	0.45[d]
0.5	0.05	0.06	0.14	0.20	0.28
1.0	0.03	0.04	0.07	0.10	0.14
2.0	0.02	0.02	0.03	0.05	0.07
5.0	0.006	0.007	0.01	0.02	0.03

Note: For site class F (soil) a site-specific study is required. PGA = peak ground acceleration.
[a]Constant up to 0.14 sec.
[b]Constant up to 0.31 sec.
[c]Constant up to 0.34 sec.
[d]Constant up to 0.31 sec.

Elastic Seismic Response Coefficient

Description: A ground-motion seismic hazard study for New York City was sponsored by the New York City and New York State departments of transportation and the Federal Highway Administration (FHWA). The purpose was to develop ground motion hazards (on rock) applicable to New York City and the surrounding region using the response coefficient C_{sm}, which is considered very important for the *m*th mode of vibration:

Table 3.4 Seismic Zones

Acceleration Coefficient	Seismic Zone
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.19 < A \leq 0.29$	3
$0.29 < A$	4

Source: AASHTO Table 3.10.4-1.

Table 3.5 Site Coefficients

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

Source: AASHTO Table 3.10.5.1-1.

$$C_{sm} = \frac{1.2\ AS}{T_m^{2/3}} \leq 2.5\ A$$

T_m = period of vibration of the mth mode (sec)

A = acceleration coefficient specified

S = site coefficient specified

C_{sm} need not exceed 2.0 A

For bridges on soil profiles III (D) or IV (E) and in areas where the coefficient A is not less than 0.30, and for modes other than the fundamental mode that have periods (T) less than 0.3 sec, $C_{sm} = A(0.8 + 4.0\ T_m)$

If the period of vibration for any mode exceeds 4.0 sec, the value of C_{sm} for that mode is

$$C_{sm} = \frac{3\ AS}{T_m^{\frac{4}{3}}}$$

Calculate the period of the bridge, T_m, using the expression

$$T_m = 2\pi\sqrt{\frac{W}{gK}}$$

where

g = acceleration of gravity (ft/sec^2)

W = dead weight of structural elements and pier caps, abutments, columns, and footings

Live loads may be included for high live-to-dead load ratios where traffic congestion occurs.

For AASHTO-defined bridge acceleration coefficients and site coefficients see Tables 3.4 and 3.5.

According to AASHTO, site effects are to be included in the determination of seismic loads for bridges. The site coefficient S is to be based upon soil profile types defined in AASHTO Sections 3.10.5.2 through 3.10.5.5.

3.4.3 Liquefaction Assessment Methodology

Extensive major bridge evaluation studies were carried out on the Garden State Parkway by the New Jersey Department of Transportation (NJDOT). Foundation design performance based on AASHTO and FHWA retrofitting specifications was independently verified.

Cyclic Stress Ratio: In the evaluations, a factor of safety against liquefaction was used, known as the cyclic stress ratio (CSR), which is dependent on level of shaking and site soil conditions. Thus,

$$\mathrm{CSR} = T_{av}/O'_v = 0.65 * (T_{h,\max}/O'_v)$$

where

τ_{av} = average horizontal shear stress (determined from site response analysis)

O'_v = vertical effective stress

$T_{h,\max}$ = maximum horizontal induced shear stress

In areas of moderate and high seismicity, liquefaction hazard at a bridge site can be assessed by field study of site response spectra, seismological parameters, and soil susceptibility. Geotechnical field testing can assess cone penetration, standard penetration, and seismic crosshole and downhole.

3.5 FOUNDATION DESIGN CODES APPLICABLE TO LIQUEFACTION

Foundation design should allow for soil interaction. This is the most important part of a structural code because the entire structure is supported on soils. Type of soil and water table elevation influence the magnitude of acceleration of the upper layers that results in subsidence and soil liquefaction.

Resistance to failure has to do with the interaction of the foundation and the soil on which it rests. Stability depends on the soil's response to forces and moments. Since no two supporting soils are alike, the dynamic behavior of a structure will be different in each seismic event; however, the primary behaviors are settlement and/or tilting.

Foundation movements can be avoided in the following ways:

- Distributing mass on the supporting soil so that the soil-bearing capacity under the column and wall foundation is not exceeded.
- Ensuring stability of the foundation: The size of the spread footing should ensure that the eccentricity of the resultant force is within an acceptable range. Deep foundations such as piles may be considered in place of shallow footings.
- Improving soil classifications and responses through laboratory testing and in-situ seismic measurements of soil deformation and strength parameters so that tables of engineering properties of nonliquefied and liquefied soil profiles can be prepared.

To summarize, the following engineering procedures for analysis of soil properties are required:

- For the two hazard levels (such as MDE and ODE), analysis of bedrock response spectra and acceleration time histories.

- Creation of databases such as for soil type and *N*-values from soil borings performed at the site.
- Site response analysis along the alignment of the foundation and estimation of appropriate response spectra for each hazard level.
- Investigation of soil liquefaction potential using the site database.
- Determination of the appropriate foundation design based on the results of analyses and findings.

3.5.1 Design Procedure for Site Conditions

Seismic lateral forces are calculated based on soil profiles (time versus acceleration history plots of a 475-year return period) and site coefficients. Different soils, varying in composition ranging from soft clay to hard rock or a mixture of clay, silt, sand, gravel, and so forth, can be classified as representing typical site conditions.

ASTM/AASHTO Specifications for Testing of Soil Properties

A static soil analysis computes small strain stiffness with the application of small unloading and reloading cycles. A dynamic analysis determines shear wave velocity from the time difference between input and output wave signals. Shear stress levels may be computed from approximate methods, using dynamic response programs such as SHAKE or nonlinear programs.

After subsurface explorations of specific sites, laboratory tests are performed in accordance with standards recommended by ASTM or AASHTO. Comparisons of profiles and properties can be made with studies on controlled specimens, for similar types of liquefied soils. Typical methods include

- Standard penetration test
- Static cone test
- Field vane test

Seismic design and soil testing have a more direct influence on AASHTO 2007 bridge foundation specifications, in which soil properties play an important part. Based on borehole test results, ASTM standards for testing of soils can provide guidance in design and construction of bridge foundations which occur for the following reasons:

- Settlement of bridge approaches
- Ground failure due to liquefaction or excessive soil deformation
- Excessive shear or flexural demands
- Soil failure behind abutments

For foundation design, laboratory tests of soils may be grouped into two general classes:

- Classification tests on either disturbed or undisturbed samples
- Quantitative tests for permeability, compressibility, and shear strength

According to AASHTO 2007, the following are examples of laboratory soil tests normally required:

- Water content
- Specific gravity
- Grain size distribution
- *Permeability*
- Consolidation

Rock Properties and Testing

Laboratory and field tests on rock samples are required to determine significant rock properties, such as:

- Compressive strength
- Shear strength
- Hardness
- Compressibility
- Permeability

In-situ rock tests may include

- Deformability and strength of weak rock
- Determination of direct shear strength of rock discontinuities
- Stress in rock using hydraulic fracturing

Ground Response Coefficients

Definitions: The ground response coefficients C_A and C_V are a function of various parameters—for example, zone factor (Z) and soil profile—that affect the amplification of the ground vibration. The initial time interval at which a structure vibrates is called the primary period. It determines which of the two coefficients will govern a structure's seismic design. The acceleration-based coefficient C_A controls for a shorter period up to approximately 1 second; the velocity-based coefficient C_V controls for longer periods.

Soil profile type A (or I) is described as hard rock and mainly occurs in eastern states. Soil profile type B (II) is found mainly in western states. For Type B, the ground response coefficients are identical to the effective peak acceleration value (Z).

3.5.2 Foundation Modeling

Foundations are modeled for analysis as nonlinear elements for translation and rotation. With respect to rotation, this is done to simulate "rocking" of the footings when pile axial capacities are exceeded. If rocking occurs prior to the limiting deformation in the column, the foundation will act as a fuse that may spare the columns significant damage.

Translational yielding of piles can mean destruction of pile heads. This can result in significant vertical settlements of foundations, although complete collapse is unlikely. Foundation systems can be designed to reduce damage from ground failure. The geotechnical engineer can provide recommendations for appropriate foundation design.

If an existing structure foundation is near a fault, this factor of geological and soil conditions can be planned for in new construction. Also, it must be taken into account that adjacent structures with separate foundations may move differently. Poorly anchored wood frame structures tend to slide off their foundations, which means that tying structures to foundations is required.

3.5.3 Mitigation Options for Liquefiable Sites

Better liquefaction assessment will lead to improved performance during seismic events. However, for existing structures that are seismologically vulnerable because of soil conditions, it is necessary to perform one or more of the following remedial or retrofit measures:

- Strengthening the foundations
- Replacement of soil, dewatering, and compaction
- Damping at the foundation level
- In large structures, use of foundation base isolators (justified because significant historic materials do not have to be removed, replaced, or replicated)

For new construction, foundations should be designed to resist increased tension or settlement by soil liquefaction in the following ways:

- Symmetry and proportion of frames and wall foundations to resist seismic forces in two directions
- Deep foundations for increased stability and to guard against soil erosion from floods
- Soil improvement

Various treatment techniques are described below for soil improvement and treatment. They improve the condition of poor or unstable ground by altering the nature of the soil. For example, where loose sands

or man-made fill are encountered, deep densification methods can be used to increase the density of the material to a degree that allows for construction of shallow foundations or, in the case of loose sands, mitigates potential for liquefaction during a seismic event.

The various techniques that may be employed include:

- Jet Grouting
- Chemical and Permeation
- Grouting
- Compaction Grouting
- Soil Mixing
- Preloading/Surcharging
- Geosynthetics and Soil
- Reinforcement
- Cement/Lime Stabilization
- Vibro-flotation: Ground vibration engineering is used for soil compaction by introducing vibro-floats. The floats apply pendulum like probe oscillation. They penetrate wet soil and waves are propagated by vibro-floatation resulting in soil compaction.
- Stone columns: Stone columns act as vertical drains. They are often installed to relieve excess pore pressures before they can build up to dangerous levels. Drains also continue dissipation rapidly after shaking is abated.
- Wick drains: Prefabricated Vertical Drains (also called wick drains) are inserted into the saturated soil under a preload to create a drainage matrix within the soil. This allows the water of consolidation to escape easily.

3.5.4 Soil Improvement and Treatment

Ground modification has been used on many construction sites to densify granular material and reduce potential settlement and susceptibility to liquefaction (see Xanthakos et al 1994). Methods include

- Soil stabilization—mixing with cementitious materials
- Compaction—applying mechanical energy
- Consolidation—preloading the soil
- Grouting—pressure-injecting grout into the soil
- Soil reinforcement—adding a stronger material

Mackiewicz and Camp evaluated the relative effectiveness of soil improvement methods by comparing the pre- and post-construction tip resistance with cone penetrometer test soundings. They also analyzed the

amount of fines observed in subsurface profiles. Results indicate that densification achieved by vibro-compaction and vibro-replacement is a function of the initial density and fines content of the material. As the fines increase, the densification generally decreases. The type of fines (i.e., clay versus silt) also affects the densification performance.

The deep dynamic compaction process is not as influenced by the fines content of the material. Mackiewicz and Camp found that the majority of the materials densified by this process had a fines content of less than 10%. Additional evaluations should be carried out to verify this conclusion.

Dynamic compaction or deep dynamic compaction, use of mini-piles, and compaction grouting are as yet theoretical soil treatments.

The best method for a given site depends on a number of factors, such as

- Type and degree of modification required—to improve settlement, slope stability, bearing capacity, and the like
- Area, depth, and total volume of soil requiring improvement
- In-situ soil type and initial properties
- Environmental constraints, including effects of adjacent structures

Availability of equipment, materials, and experienced contractors, as well as cost, time, accessibility are additional factors to consider.

Grout Injection

Requirements: If a bridge site is placed in soft sands and liquefaction is a probability, stone columns or grout injection can be used. Stone columns are constructed by drilling large-diameter holes around the foundation and packing them with rocks. In a major event, the column will relieve water pressure so that the water spurts up the column, similar to a sand boil. Grout injection is a series of pressure-grouted holes that densify the local areas around foundations to prevent liquefaction.

3.5.5 Elevated Structure Foundation Method

The elevated structure foundation (ESF) method is a seismic vibration control technology that is made an integral part of a superstructure. Anticipated lateral and vertical shaking can be shielded by configuring ESF for

- Structure dimensions
- Site-specific construction
- Local soil conditions
- Appropriate structure materials
- Connections of vertical elements to the foundation
- Embedment of the foundation in the soil

Case Study of the Design of Seismic-Resistant Spread Footings

Although the calculations to design the spread footing were performed for three groups of columns in the building, the A3 and B3 groups carry the largest amount of load.

There are a number of software programs available to carry out the design. An alternative is to create spreadsheets using ACI or AASHTO code equations. For continuous mat footing for weaker soils or when columns are closely spaced, the MATS program is recommended.

Simplified Method of Calculations:

The design of the foundation involved calculations that could be summed up as the calculation of the column loads and the calculation/design of the spread footing. First, the summations of the column's dead and live loads were calculated and then used to design the spread footing.

Effective depth of footing (d_e): The depth of the footing, d_e, is an assumed measurement that was compared to the depth required for shear (i.e., depth required for one way/two way). A footing slab depth between 18 and 15 in. was selected based on ACI guidelines.

Effective soil pressure (q_e): The effective soil pressure, q_e, measures the stress on the soil and determines its ability to resist shear stresses. It was calculated using the equation

$$q_e = q_a(\text{psf}) - d_e(\text{ft}) * W_c(\text{pcf}) - d_s(\text{ft}) * W_s(\text{pcf})$$

The result was an effective soil pressure of 1,680 psf.

Area of footing required: The area of the footing for each column was then calculated by summing the dead and live loads on the column and dividing the result by the effective soil pressure, q_e.

A_f required $= (DL + LL)/q_e$

Based on the result, a 5 ft $\times$ 5 ft footing was selected.

Bearing pressure for strength design (q_u): The bearing pressure, q_u, measures the amount of pressure the spread footing can support. It was calculated by multiplying the dead load and live load by load factors (given in ACI guidelines), summing up the product, and dividing the sum by the area of the footing:

$$q_u = (1.2)(DL) + (1.6)(LL)/A_f$$

The bearing pressure was calculated to be 2.2 ksf.

There were two shear conditions taken into consideration. They were one-way shear (beam shear) and two-way shear (punching shear). These conditions are typically considered when designing spread footings.

Depth required for two-way or punching shear: Two-way shear is a condition in which a column punches through a footing slab because of the compression load from the column and the opposing diagonal tension in the footing. Thus, it was important to design the footing for strength to avoid this type of failure.

The computations for the depth required for two-way shear used the following equations from the ACI Code:

$$d_{s2} = V_{u2}/\varphi 4\sqrt{(f'_c b_o)} \quad \text{(ACI Equation 11–35)}$$

$$d_{s2} = V_{u2}/(\varphi((\alpha_s * d)/b_o + 2)\ \sqrt{(f'_c b_o)}) \quad \text{(ACI Equation 11–34)}$$

After calculating the depths using the two equations, the larger of the two depths was selected and compared to the effective depth (d_e) of the footing.

Depth required for one-way shear: To calculate one-way shear, we used the following equation:

$$d_{s1} = V_{u1}/\varphi 2\sqrt{f'_c b_w}$$

The depth calculated from this equation was compared to the two-way depth equations and was less in value than the two-way shear depths, so it did not control the design of the footing. The minimum calculated depth of footing was 16 in. with a cover of 3.5 in. and was unaffected by the depth for one-way shear.

Bending Moment on Footing

Another important step in designing the footing was for bending moment, calculated using the equation

$$M_u = (l_f/2 - a_c/2) * (l_f) * q_u * (l_f/2 - a_c/2)/2$$

The calculated moment was 4.23 ft-kips, which was then used to arrive at the amount of reinforcement needed in the foundation.

Reinforcing Steel Area

After calculating the bending moments, the number of bars and the bar size that would be used for each spread footing were determined. The following equation was used:

$$M_u/\varphi b d^2 = \rho. \text{ Checked against } \rho_{\min} \text{ for flexure}$$

The result was then checked with the minimum flexure found in a table from McCormac, J. C., and J. K. Nelson, Reinforced Concrete (6th ed.).

Calculating ρ Value

For a single-story building footing, ρ was slightly less than the minimum established by the ACI code. Using the following two equations and taking the larger of the two values,

$$200 b_w d/f_y = \rho$$

$$3\sqrt{(f'_c b_w d)}/f_y = \rho$$

$$A_s = (\text{larger } \rho \text{ value}) * L_f(\text{in})(d_e - c)(\text{in.})$$

Finally, using Table A.4 in McCormack, the number of bars and the bar diameter were calculated. Following the table, we used 5 #9 bars spaced at 13 in. centers or 7 #8 bars at 9 in. centers. These bars were to be used in both directions within the footing.

Development Length for Bars

$$\frac{c + Ktr}{db} = < or > 2.5$$

$$\frac{ld}{db} = \frac{3}{40} = \frac{fy}{\sqrt{f'_c}} \frac{\alpha\beta\gamma\lambda}{\frac{c + Ktr}{db}}$$

The result of the equation, $\frac{c + Ktr}{db}$, was greater than 2.5; thus the control value of 3.5 was used as recommended by the ACI code.

The Mononobe-Okabe Equation for Seismic Earth Pressure Using a Correction Factor

The Mononobe-Okabe method was developed in the 1920s by Mononobe and Okabe (1929) to take into account the vertical and horizontal accelerations induced by an earthquake. Kapila (1962) modified it to consider passive pressures. The method is widely used for the design of bridge abutments and other earth-retaining structures. AASHTO 2007 (A11.1.1.1) recommends it for computing lateral soil pressure for seismic loading. NCHRP (12-49) addresses the effects of probable amplification of active earth pressure and mobilization of passive earth masses.

For the calculation of the seismic soil forces acting on a bridge abutment, the Mononobe-Okabe method is an extension of the Coulomb sliding-wedge theory, taking into account horizontal and vertical inertia forces acting on soil. The analysis is described in detail by Seed and Whitman (1970) and Richards and Elms (1979).

Assumptions are as follows:

- The abutment is free to yield sufficiently to enable full soil strength or the mobilization of active pressure conditions.
- If the abutment is rigidly fixed and unable to move, the soil forces will be much higher than those predicted by the analysis.
- The backfill is cohesionless, with a friction angle of ϕ.
- The backfill soil is unsaturated so that liquefaction is avoided.

Active Lateral Force on Abutment

Equilibrium considerations of the soil wedge behind the abutment lead to a value, E_{AE}, of the active force exerted on the soil mass by the abutment, and vice versa. When the abutment is at the point of failure, E_{AE} is given by the expression (Figure 3.3)

$$E_{AE} = \frac{1}{2}\gamma H^2(1k_v)K_{AE}$$

where the seismic active pressure coefficient $K_{AE}(1)$ is

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta\cos^2\beta\cos(\delta + \beta + \theta)} \times \left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^{-2}$$

where

γ = unit weight of soil (kcf)
H = height of soil face (ft)
ϕ = angle of friction of soil (deg)
θ = arc tan $(k_h/(1-k_v))$ (deg)
δ = angle of friction between soil and abutment (deg)
k_h = horizontal acceleration coefficient (dim)
k_v = vertical acceleration coefficient (dim)
I = backfill slope angle (deg)
β = slope of wall to the vertical, negative as shown (deg)

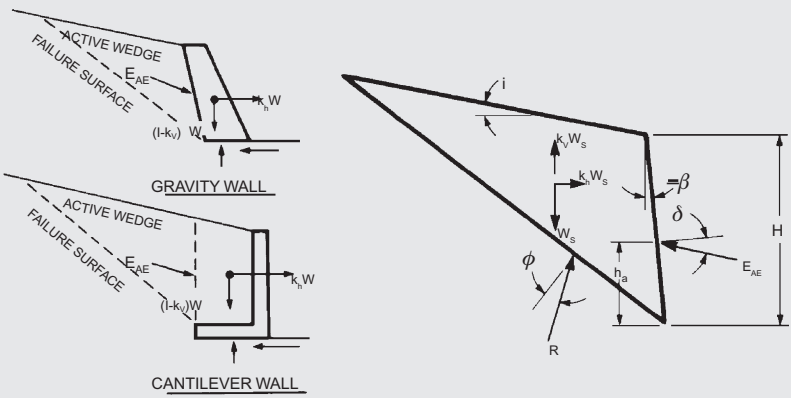

Figure 3.3 Active wedge force.

Equivalent Expression for Passive Force (1)

If the abutment is being pushed into the backfill,

$$E_{PE} = \frac{1}{2}\gamma H^2(1 - k_v)K_{PE}$$

where

$$K_{PE} = \frac{\cos^2(\phi - \theta + \beta)}{\cos\theta \cos^2\beta \cos(\delta - \beta + \theta)} \times \left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta+i)}{\cos(\delta-\beta+\theta)\cos(i-\beta)}}\right]^{-2}$$

As the seismic inertial angle θ increases, the values of K_{AE} and K_{PE} approach each other and, for vertical backfill, become equal when $\theta = \phi$.

The Mononobe-Okabe method will be useful for the design of bridge piers if the uniform load method or single modal spectral analysis method is used for seismic analysis. The equation tends to overestimate the passive pressure when there is wall resistance. In developing his original equation, Coulomb assumed that the failure surface is plane. However, when the angle of friction is high, the passive failure surface is curved. Therefore, Coulomb's equation predicts unrealistically high pressures.

A correction factor, R, is proposed by Yost et al. (2004) (Figure 3.4), given by

$$E_{PE(\text{Corrected})} = R(E_{PE})$$

This correction factor for passive pressure is of great significance to the seismic design of monolithic abutments, sheet pile walls, retaining walls, and other structures. With proper analysis and detailing, it is possible to design the abutment stems as columns bending in the weak direction, which will result in a practical design for the abutment stem and its footing.

When used with the proper choice of input parameters and suitable safety factors, this venerable equation provides a sound basis for the design of many structures.

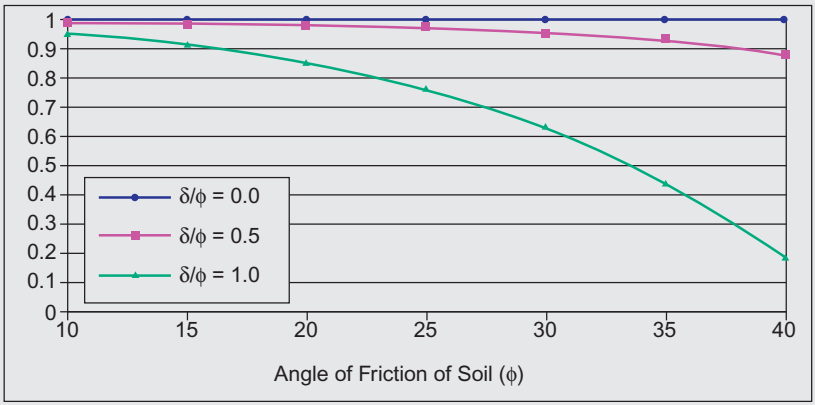

Figure 3.4 Mononobe-Okabe correction factor, R, for passive pressure.

3.6 COMPUTER SOFTWARE FOR LIQUEFACTION ASSESSMENT AND MITIGATION

NovoLiq is designed for soil liquefaction analysis and supports multi-layer as well as single-layer stratigraphy. The results of the analysis are presented as safety factors against soil liquefaction, probability of soil liquefaction, and post-liquefaction movements, including

- Reconsolidation settlements
- Lateral movements (spreading)
- Residual strength

OpenSees, developed by the University of California, Berkeley, allows integration of structure and soil models and can investigate soil–structure–foundation interaction. Analyses of shallow and deep foundations and liquefiable soils are available.

3.7 CONCLUSIONS FOR SEISMIC RESPONSE OF STRUCTURES TO SOIL TYPE AND LIQUEFACTION

After location in a seismic zone, soil type is the most important influence on a structure's seismic vulnerability. Depending on type and profile, certain soils liquefy as a result of ground motions in an earthquake, making them unable to support the weight of overlying structures. The foundations of these structures are thus subject to settling and tilting. To determine a soil's tendency to liquefy in response to ground motions, it is necessary to obtain soil profiles by laboratory tests.

If a structure has moved as a result of liquefaction, retrofit measures will be ineffective. However, if there has been no movement, strengthening the foundation, replacing or compacting the soil, dewatering the site, and damping and base isolation are some of the available retrofit options.

For new construction, foundations can be designed in many ways to resist increased tension or settlement by liquefaction. The performance of deep foundations such as piles and drilled shafts as compared with that of shallow foundations should be investigated. Soil improvement methods depend upon the type of soil, type of foundation and the seismic zone. Although expensive, are useful and effective in many cases and should also be considered.

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CHAPTER FOUR

Major Earthquakes as the Basis for Code Development

Chapter Outline

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4.1 INTRODUCTION

A Causerie of Earthquake Disasters: This chapter broadly reviews the types of damages caused to property and life, based on which it recommends further development of a seismic resistant code, for retrofit and rehabilitation and for enhancement of design of new structures.

A detailed literature review shows contributions by the following organizations and design codes:

Earthquake Engineering Research Center (1984)

Earthquake Engineering Research Institute (1999, 2002, 2004, 2005)

U.S. Geological Survey (1964, 2005).

The following authors have contributed to several soil aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to the study of latest earthquakes is presented here:

Bilham et al. (2001); Bolt (1975); Comerio (1998); Cooper James et al. (1994); Godavitarne et al. (2006); Khazai et al. (2006); Murata (2003); Shedlock et al. (2004); Wald et al. (1993); Zoback (2012).

In addition to the above publications, relevant individual references are discussed in the text.

Most Recent and Historical Earthquakes: In August, 2011. In the east coast of the United States was rocked by a magnitude 5.9 earthquake.

Earthquake-Resistant Structures
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Centered near Washington, D.C., it shook the entire East Coast, with shocks felt as far south as Florida and as far north as Rhode Island, Maine, and Ontario (Ward and Plage 2011, FEMA 2011, Bolt 1971). In Washington the Washington Monument and the National Cathedral sprouted cracks. The tremors were felt in the ASCE Washington office and in ASCE headquarters in Reston, Virginia.

This event will be discussed in some detail later. Its significance here is the subject of this chapter: the study of major earthquakes in the development of seismic-resistant design codes. By analyzing past earthquakes, engineers and seismologists can develop better seismic-resistant design for structures and their foundations. For example, engineers have determined that structures can be prevented from collapsing not just by making them stronger but by making them more flexible so they sway and slide above the shaking ground rather than crumbling. Using a technology known as "base isolation," skyscrapers have been built that float on systems of ball bearings, springs, and padded cylinders. Because they do not sit directly on the ground, they are protected from shocks. In the event of a major earthquake, they sway back and forth a few feet. "Moats," or buffer zones, prevent them from colliding with other structures.

Around the world, there are as many as half a million earthquakes each year. Forty thousand can actually be felt, although not all of them cause damage. (See Figure 4.1 and Tables 4.1 and 4.2). Still, in its publication *Historic Worldwide Earthquakes*, the USGS (1964) has listed 21 quakes throughout recorded history that have killed over 50,000 people in a single tremor. It estimates that since 1900, there have been on average 18 major earthquakes of magnitude 7.0 to 7.9 and at least one of magnitude 8.0 or greater per year. These statistics point up the need for seismic-resistant design given the world's rapidly growing population, so much of which is centered in areas of high seismic risk such as Mexico City, Tokyo, Karachi, and Tehran. Indeed, some seismologists are warning that a single quake may claim the lives of up to 3 million people.

As a reminder, the extent and seriousness of the damage (Figure 4.2) from seismic activity are determined by the following factors:

- Seismology, including rate of tectonic plate movement
- Epicenter location and distance from the epicenter to locations where the shaking can be felt
- Magnitude, intensity, and duration of shaking
- Soil liquefaction and soil profile

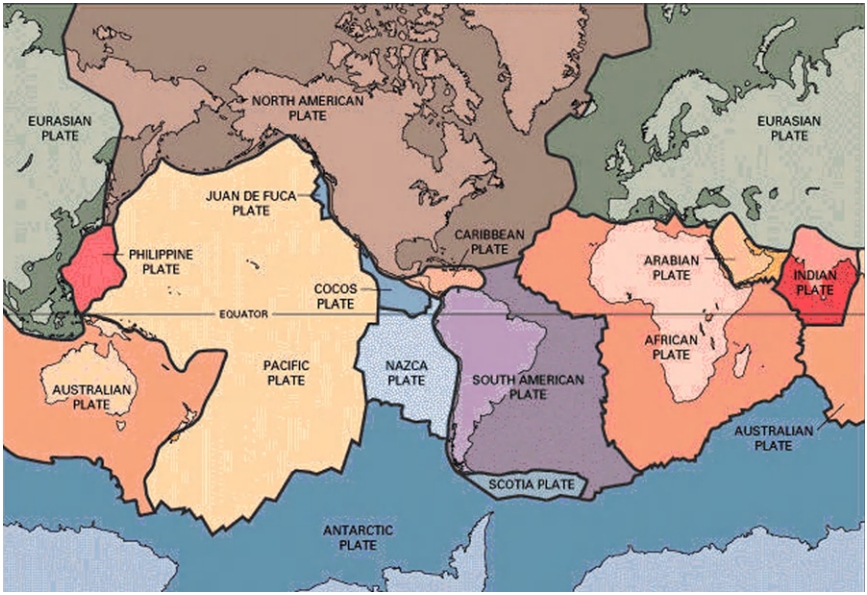

Figure 4.1 Tectonic plate distribution across the earth. *Source: courtesy of USGS.*

Table 4.1 World's Most Destructive Earthquakes (1700–1995)

Location	Year
Cascadia, California	1700
Lisbon, Portugal	1755
New Madrid, Missouri	1811–1812
San Francisco, California	1906
Daly City, California	1957
Alaska	1964
Haicheng, China	1975
Tangshan, China	1976
Loma Prieta, California	1989
Northridge, California	1994
Kobe, Japan	1995

4.2 EARTHQUAKES IN THE UNITED STATES

In the United States alone, earthquakes pose significant risk to 75 million people in 40 states. Alaska registers the most in a given year, with

Table 4.2 History of Major Earthquakes in the Recent Past (2010–1998)

Location	Year
Chile	2010
Haiti	2010
Italy	2009
Pakistan	2008
Kyrgystan	2008
China	2007
Japan	2007
Sumatra	2007
Peru	2007
Solomon Islands	2007
Sumatra	2007
Java	2006
Java	2006
Pakistan	2005
Sumatra	2005
Japan	2005
Iran	2005
Sumatra	2004
Iran	2003
Algeria	2003
China	2003
Iran	2002
Afghanistan	2002
Turkey	2002
Peru	2001
United States	2001
El Salvador	2001
India	2001
El Salvador	2001
Sumatra	2000
Turkey	1999
Taiwan	1999
Turkey	1999
India	1999
Afghanistan	1999
Colombia	1999
Indonesia	1998

California placing second. Tremors pose a continuous risk to Californians because of the density of its population. The total dollar value of losses since 1971 is estimated to be over $55 billion.

Figure 4.2 Examples of failure of poorly designed versus well-designed buildings.

A Timeline of U.S. Earthquakes

1737—New York City, magnitude 5.5
1755—Cape Ann, Massachusetts, magnitude 6.0–6.3
1811–1812—New Madrid, Missouri, magnitude as high as 8.1
1884—New York City, magnitude 5.5
1886—Charleston, South Carolina, magnitude 6.5–7.3
1906—San Francisco, magnitude 7.8
1925—Santa Barbara, California, magnitude 6.8
1944—New York, between Massena and Cornwall, Ontario, magnitude 5.8
1964—South Central Alaska, magnitude 9.2
1971—California, San Fernando Valley, magnitude 6.6
1983—New York, Blue Mountain Lake, magnitude 5.3
1989—California, Loma Prieta, magnitude 6.9
1994—California, Northridge, magnitude 6.7
2002—New York, Au Sable Forks, magnitude 5.1
2002—New York, Plattsburgh, magnitude 3.6
2002—New York, Redford, magnitude 3.3
2003—Alaska, Denali, magnitude 7.9
2009—Alaska, Fox Islands, magnitude 6.5
2009—California, Owens Valley southeast of Lone Pine, magnitude 5.2
2011—Southern Colorado, magnitude 5.3
2011—Arkansas, Greenbrier, magnitude 4.7.

The 1964 Great Alaskan earthquake, one of the most powerful seismic events recorded in U.S. history, was felt across south-central Alaska for nearly

four minutes. Loss of life was estimated at 143 resulting from ground fissures, collapsing buildings, and tsunamis. Soil liquefaction and landslides caused major damage to or total destruction of structures and infrastructure. A 27-foot tsunami destroyed the village of Chenega, killing 23 inhabitants. Post-quake tsunamis also severely affected Whittier, Seward, Kodiak, and other Alaskan communities, as well as British Columbia, Oregon, and California. They caused damage as far away as Hawaii and Japan.

In 2011 in Colorado the strongest natural earthquake since 1973, with a magnitude of 5.3, hit the southern portion of the state near the New Mexico border. It was preceded and followed by a series of smaller temblors. Fortunately, there were no injuries and only minimal damage.

The magnitude 4.7 earthquake centered at Greenbrier, about 40 miles from Little Rock, in 2011 was felt in Oklahoma, Missouri, Tennessee, and Mississippi. It was the largest quake to hit Arkansas in 35 years and the largest of more than 800 quakes to strike the area within a year—in what is now being called the Guy-Greenbrier earthquake swarm.

Although activity varies, several small quakes can occur daily in the Guy-Greenbrier swarm. No major damage has been reported, but gradual damage, such as cracks in walls and driveways, has been observed. Researchers are studying a possible connection to the natural gas drilling in the area, which employs the injection of pressurized water to create fractures deep in the ground as a method of freeing the gas.

Distribution of Earthquakes in the United States

Of the 48 contiguous states, California leads with at least eleven earthquakes of magnitude 7.0 or higher in the last 200 years. In the fall of 1865, while in the city of San Francisco, Mark Twain experienced his first earthquake. His experiences were captured in the book, *Roughing It*. Other major events include

- The Arkansas–Missouri border, four between 1811 and 1812
- Lake Chelan, Washington, in 1872
- Pleasant Valley in 1915 and Dixie Valley in 1954, both in Nevada
- Hebgen Lake, Montana, in 1959
- The "Big Island" of Hawaii, two in 1868, one in 1871, and one in 1975
- Near Borah Peak, Idaho, in 1983
- Alaska, at least twelve in the past 30 years

4.2.1 California Seismology

The Pacific Plate extends from western California to Japan and includes much of the Pacific Ocean floor. The North American Plate comprises most of the North American continent and parts of the Atlantic Ocean. The Pacific Plate is grinding northwestward past the North American Plate along the San Andreas Fault (at a rate of about two inches per year). Thus, Southwestern California is moving horizontally northward toward Alaska. The dividing point, or margin, between the two is the San Andreas Fault system, which extends from the Salton Sea in the south to Cape Mendocino in the north—an 800-mile stretch almost the entire length of California.

The San Andreas Fault is the most tectonically active area in the Western United States (USGS 1993; also see fingerquake@gldfs.cr.usgs.gov) and is responsible for one magnitude 8 or higher earthquake per century (see Table 4.3 for probabilities of future California shocks). It was San Andreas that caused the 1906 San Francisco earthquake (magnitude 7.8), which killed more than 3,000 people and destroyed much of the city (Figure 4.3 illustrates how this earthquake would have been measured). It was also the cause of the 1989 Loma Prieta, California, earthquake (magnitude 6.9).

Table 4.3 California Earthquake Probabilities in the Next 30 Years

Location	Probability	Magnitude	Organization
The San Andreas Fault	21%	6.7 or greater	SCEC/CGS/CEA
San Francisco Bay area	62%	6.7 or greater	2002 Working Group
Greater Bay Area	63%	6.7 or greater	SCEC/CGS/CEA
Southern California	60%	6.7 or greater	
Hayward-Rodgers Creek Fault system (last major earthquake in 1868)	31%	6.7 or greater	
Calaveras Fault in the East Bay	7%	6.7 or greater	SCEC/CGS/CEA
San Gregorio Fault along the San Francisco Peninsula coast	6%	6.7 or greater	SCEC/CGS/CEA
East Bay, near the Central Valley	3% or less	6.7 or greater	SCEC/CGS/CEA
The Greenville Fault	3% or less	6.7 or greater	SCEC/CGS/CEA
The Mt. Diablo Thrust	3% or less	6.7 or greater	SCEC/CGS/CEA
Concord-Green Valley Fault	3% or less	6.7 or greater	SCEC/CGS/CEA

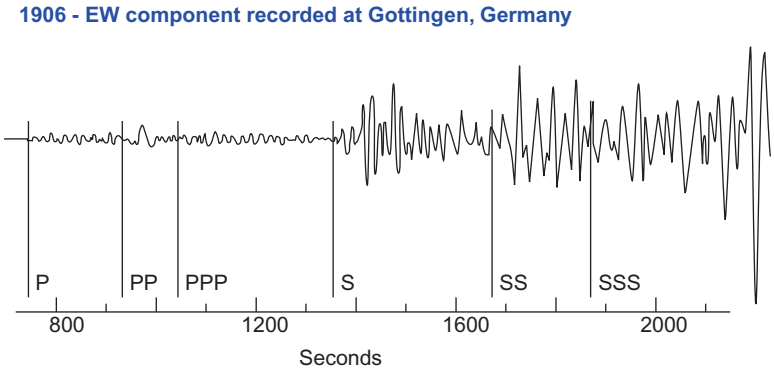

Figure 4.3 Early earthquake recording of P and S waves.

The Polaris Fault, named after a former mining town that sits above it, is a fraction of the size of the San Andreas Fault, at 22 miles long. Because it is located near the Martis Creek Dam at Truckee, however, it poses a major threat of an earthquake bursting the dam and potentially flooding a plain that stretches 35 miles to Reno, Nevada, a city of some 222,000 people.

Unlike Japan, California faces a low risk of tsunamis from its quakes. However, the state's real problem is the condition of its structures, many of which are of substandard construction. The Japanese are ahead of California seismic-resistant design codes have done a better job of new construction and retrofit for seismic survivability.

During the 1971 San Fernando (Figure 4.4), the 1989 Northridge (Figure 4.5), and the 1994 Loma Prieta (Figure 4.6) earthquakes in California, several concrete buildings constructed before the introduction of modern codes collapsed or were catastrophically damaged. According to a new estimate, 17,000 older concrete buildings in California could be vulnerable to seismic activity. Not all of the buildings identified are collapse hazards or prone to severe damage. However, an estimate includes a number of schools, state and local government centers, hospitals, and public safety facilities.

Analysis of Extent and Types of Structural Damages: A more careful study of specific buildings in order to better understand which are the riskiest, is necessary. Unlike unreinforced masonry buildings, which uniformly do not perform well in earthquakes, there is tremendous variability in older concrete buildings. Understanding what makes them vulnerable and quickly weeding out which ones, may need more detailed

Figure 4.4 Two examples of damage caused by the San Fernando earthquake.

Figure 4.5 Northridge earthquake damage.

evaluation and possible retrofit. It is one of the goals of a National Science Foundation—funded research project sponsored at the Pacific Earthquake Engineering Research Center (PEERC).

In a recent 2008 earthquake of magnitude 7.8 on San Andreas Fault in Southern California, damage to property and infrastructure included:

- Collapse of thousands of concrete buildings
- Bridge collapses and destroyed roads

Among the reasons for the failures were these:

- Poor performance of older shear-critical columns and short seat widths in bridges

Figure 4.6 Two examples of damage caused by the Loma Prieta earthquake.

- Major weakness in nonductile concrete buildings.
- Torsional response associated with skewed geometries in skewed bridges
- Substandard construction
- Instability from soil liquefaction
- Concrete frames or walls without sufficient reinforcing steel.

Caltrans Administrative and Technical Remediation Measures

The California Department of Transportation (Caltrans) has always compiled its own seismic design criteria. It uses the latest design criteria, as outlined in the ATC draft document of "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations" (ATC 1996) for reduction of elastic spectral forces:

- Use site-specific ARS spectral curves, where available.
- Verify displacement capacities by inelastic pushover analyses, based on moment-curvature assessment of inelastic members.
- Consider vertical ground accelerations in the design of bridge superstructures.
- Minimize the number of expansion hinges to reduce unseating problems.
- Improve design of skewed bridges by eliminating skew, where feasible and lengthening seats.
- Avoid large skews in the plan geometry of abutments, bents, and expansion hinges.
- Use prismatic bridge columns throughout the column length.
- Reduce torsional response of frames and overloading of stiff bents, by balancing stiffness between bents in individual frames.
- Avoid damage to below-ground structures by oversizing, to accommodate possible future strengthening of columns and/or superstructure should future seismic design requirements exceed the current requirements.

- Wrap the concrete columns with carbon-fiber mesh, to hold the concrete column together when the building is shaken.

Caltrans developed improved reinforcement details: Closer spacing and improved detailing of column transverse reinforcement, requirements for top reinforcement in footings and pile caps, and controls on column longitudinal reinforcement splice location.

4.2.2 East Coast Virginia Earthquake

The Virginia Earthquake of August 2011 was relatively small (magnitude 5.9), but it was the largest recorded, for the state of Virginia in the last 114 years and its shocks were felt up and down the entire East Coast from Florida to Canada. Its impact continues as engineers analyze the damage it caused, in order to develop new seismic codes. Buildings constructed in the Capital since the 1960s are required to follow seismic codes, but limited efforts have been made to reinforce many of the region's historic structures. Most of the buildings dating from the early 1900s are built of stone, brick, and mortar, which makes them vulnerable to shaking.

The Event

According to USGS, the temblor struck 87 miles southwest of the nation's capital (California Department of Conservation, Division of Mines and Geology 1983) at a depth of 3.7 miles, releasing an amount of energy the equivalent of 10,676 tons of TNT. Residents from Richmond to Washington reported shaking at VI–VII on the Mercalli scale, which is significant enough for causing minor to moderate building damage. The shaking was felt at the White House and parts of the Pentagon as well. The White House was evacuated, along with other buildings in the area. There were reports of minor injuries from falling bricks.

Tremors were felt in downtown Pittsburgh and led to evacuations at some universities and businesses. Philadelphia also felt some shaking (Tobin and Montz 1997); tremors could be felt in New York. As a precaution, the control towers at the JFK and Newark airports were evacuated by the Federal Aviation Administration. The quake was felt at least as far west as Michigan, where it hit during a simulated emergency response exercise on the Detroit River.

The Damage

Most of the immediate damage in Maryland was centered in Baltimore, where the dense concentration and older buildings, made for a vulnerable

combination. In Washington, the central tower of the Washington National Cathedral, the highest point in the city, suffered serious damage. There was minor structural damage around the entire structure, and cracks appeared in the flying buttresses on the east end. A crack appeared as well near the top of the marble Washington Monument. Three chimneys on the Ecuadorean Embassy in northwest Washington, D.C., collapsed, sending rubble into the street. Finally, the Armed Forces Retirement Home suffered major damage to its dining room and 600 residents were evacuated.

Authorities inspected numerous bridges in the region after the event. In Maryland, the Harry Nice bridge on Route 301 over the Potomac River was closed temporarily for an inspection. The Woodrow Wilson Bridge on I-95 over the Potomac was inspected as well but remained open.

The U.S. General Services Administration (Washington's largest building owner) follows a combination of local codes and international building codes for new construction. For existing structures, GSA has adopted the Federal Standards of Seismic Safety of Existing Buildings, developed by the Interagency Committee on Seismic Safety in Construction and the National Institute of Standards and Technology NIST located in Gaithersburg, Maryland. These programs fund research programs designed to improve structural resistance to the forces exerted both by earthquakes and high winds. Southern California Edison SCE has taken the lead in efforts to see the programs reauthorized to ensure that the research continues. The Virginia earthquake was well recorded due to its proximity to Federal Capital and for the hub activity of a relatively dense and prosperous population. The event showed that East Coast region is also vulnerable to seismic activity and certain improvements for safety of buildings and infrastructure are desirable.

4.3 EARTHQUAKES WORLDWIDE

Global Tectonics: There has been a resurgence of earthquakes in the Middle East and Southeast Asia, with repeated major earthquakes in Turkey, Iran, India, and Pakistan. Chile and Haiti have also suffered recent devastation. Some of the world's major rivers, which generate high population density because of abundant water supplies and irrigation facilities, are located in earthquake zones. These include the Yangtze and the Mekong in China; the Ganges and the Brahmaputra in India; and the Indus in Pakistan. Combined, the drainage basins of these rivers are home to almost half of Earth's population some 3 billion people.

4.3.1 Seismology of the Himalayas

Tectonic Plate Movements: The Himalayan mountain range separates the Indian subcontinent from the Tibetan Plateau and consists mostly of uplifted sedimentary and metamorphic rock. One of its peaks, Everest, is the highest in the world. The tectonic plates underlying the Himalayas are still moving, causing India and southern Tibet to converge at nearly one inch per year (see Figures 4.7 and 4.8). Massive earthquakes tend to occur along these plate boundaries.

The major earthquakes that have occurred along the Himalayas since 1800 have differed in dimensions (See Tables 4.4 and 4.5). Great earthquakes are likely to have occurred in Nepal in the thirteenth century and in Kashmir in the mid-16th century. Some destroyed vast regions along the front of the range.

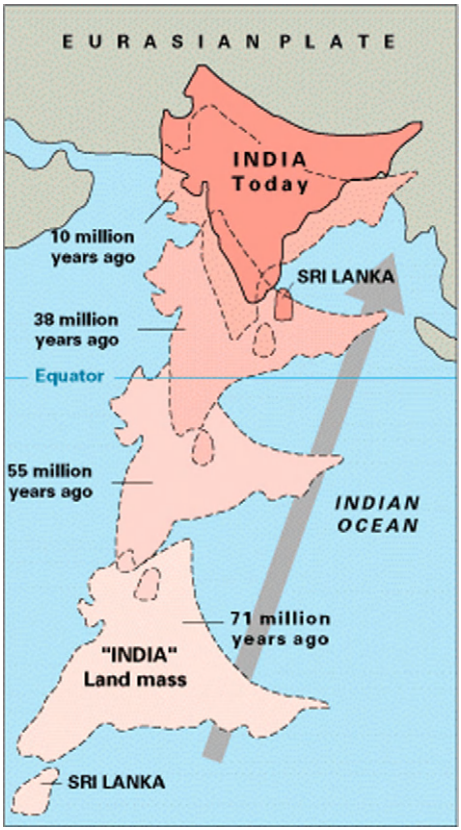

Figure 4.7 71 Million-year movement of the Indian Plate.

Figure 4.8 Formation of Everest on the Indian side (*left*) and formation of K2 on the Pakistan side (*right*) of the Himalayan range as a result of continental collision.

Table 4.4 Nineteenth-Century Himalayan Earthquakes

Year	Location	Magnitude	Casualties
1819	Kutch	7.7 ± 0.2	2,000
1833	Kathmandu	7.7 ± 0.2	500
1869	Cachar	7.5	15
1885	Sopor, Kashmir	7.0	3,000
1897	Shillong	8.1 ± 0.1	1,542

Today, about 50 million people are at risk from Himalayan earthquakes, many of them in towns and villages on the Ganges plain. The capital cities of Bangladesh, Bhutan, India, Nepal, Pakistan, and other cities with nearly a hundred million inhabitants are vulnerable to damage from future tremors.

Following is a sample of Himalayan earthquakes:

- 1934 Bihar-Nepal, magnitude 8.1 (at least 30,000 dead and whole towns demolished)
- 1950 Assam-Tibet, magnitude 8.6 (1,526)
- 1988 Nepal, magnitude 6.6 (approximately 1,467 dead and more than 16,000 injured)
- 1935 Quetta, Pakistan, magnitude 7.9 (approximately 60,000 dead)

4.3.2 Twenty-First-Century Earthquakes on the Indian Subcontinent

India, once a separate land mass, has been moving across the Indian Ocean during the last few million years. It is currently striking southern Asia. The energy from the collision that raised the Himalayas, radiates out

Table 4.5 Twentieth-Century Himalayan Earthquakes

Year	Location	Magnitude	Casualties
1905	Kangra	7.8 ± 0.2	19,500
1918	Srimigal, Bengal	7.6	–
1930	Dhubri, Assam	7.1	–
1934	Bihar/Nepal	8.2 ± 0.1	10,500
1943	Assam	7.2	–
1947	Assam	7.7	–
1950	Arunchal Pradesh	8.5	1,542
1956	Anjar	7.0	113
1967	Koyna	6.5	177
1970	Broach	5.4	30
1975	Kinnaur	6.2	60
1988	Manipur	6.6	35
1988	Udaypur	6.4	6,500
1991	Uttar Kashi	6.6	769
1993	Killari, Latur	6.3	7,610
1997	Jabalpur	6.0	39
1999	Chamoli	6.8	103

from there, often causing destructive seismic events. Five major earthquakes have hit the subcontinent only in the past decade. This frequency is dangerous, given that the population of India has doubled since 1950.

Gujarat, India

The 2001 Gujarat earthquake, magnitude 7.6–7.7, lasted over two minutes. This was an intraplate event, occurring far from any plate boundaries, which means that the region above the epicenter was unprepared for the violent shaking. The Modified Mercalli scale registered the maximum felt intensity (X). In the Kutch district alone, the death toll was 12,290 and around 20,000 were injured; in all, nearly 400,000 homes were destroyed.

Like many of the regions in which earthquakes on the Indian subcontinent occur, the Kutch district of Gujarat has seen its population increase by a factor of ten in recent history, this increase is significant. By way of comparison, an earthquake in Kutch in 1819 resulted in 2,000 fatalities whereas the 2001 event caused 18,000 confirmed and possibly 30,000 unconfirmed deaths. This appears to be a slight improvement in percentage relative to the population, but in real numbers the deaths resulting from the recent quake were much greater.

In 2001, most of the buildings that collapsed were the older ones. Without recent enforcement of new seismic codes, the death toll would have been higher. Moreover, the 2001 Gujarat earthquake occurred in an identified zone of heightened seismic hazard. Given its current population, just one of the possibly several probable overdue Himalayan shakings could yield 200,000 fatalities. Such an estimate may still be too low should a great earthquake occur near one of the megacities in the Ganges Plain.

Unreliable Seismology Data: Studies of earthquake recurrence intervals show that large earthquakes occur in northwestern Gujarat every 266 to 533 years. However, surface ruptures have not been found for any of these events, so there are no geological constraints on recent ruptures. For this reason, geologists are concerned that paleo-seismic investigations of Himalayan surface faults may yield misleadingly long recurrence intervals. Also of concern is that repeat surveys of trigonometric points installed before the 1905, 1934, and 1950 earthquakes have yet to be recalibrated with modern techniques. There is a need to investigate the geology and seismology of the region on an accurate basis. Unexpected earthquakes not just kill large number of people but also shatter the economy of India.

Pakistan

The 2005 Kashmir earthquake, magnitude 7.6, caused the deaths of 80,000. The severity of the damage was attributed to severe upthrust coupled with poor construction. Landslides and rock falls damaged several mountain roads and highways, cutting off access to the region for several days. Liquefaction and sand blows were also a major cause of destruction, which included thousands of building collapses (Figure 4.9).

A few years later, 2008 Pakistan earthquake, magnitude 6.4, hit near Ziarat in Baluchistan. A second quake, almost as strong, followed about an hour later, and several strong aftershocks came after. Fatalities totaled 215, and 50,000 were left homeless, some as a result of landslides. In 1935 an earthquake in the same area killed 30,000 people.

Myanmar, Tibet, and Sikkim

Major damage and Destruction: In 2011 the Myanmar (a country located South of Assam, India and Southeast of Bangladesh) earthquake, magnitude 6.8, killed at least 74 people in Burma and one in Thailand, and injured over 100. The epicenter was a remote area near Burma's eastern border with Laos and Thailand, but shocks were felt as far away as

Figure 4.9 National Academy of Science investigative team members observing the destruction of a town near K2 resulting from the 2005 Kashmir earthquake in Pakistan (the author is on the right).

Bangkok and Hanoi. Hundreds of houses in Shan State, home to the Shan minority, were destroyed or damaged, along with schools and monasteries.

The 2008 Tibet earthquake was followed by three aftershocks above magnitude 5. In all, 7 people died and 22 were injured, all in the border region of Yadong. Hundreds of landslides disrupted traffic, power and water supplies, and telecommunications. Casualties and damage would have been much worse if Yadong had not been sparsely populated.

The 2011 Sikkim earthquake, magnitude 6.9, was centered near the border of Nepal and Sikkim, but was felt across the northeast of the subcontinent. At least 111 people were killed. Numerous landslides made rescue efforts difficult.

4.3.3 Indonesia the Worst Hit Country

The 2004 Indian Ocean (or Sumatra-Andaman) earthquake, at a magnitude of 9.1–9.3, was the third largest earthquake ever recorded on a seismograph and lasted between 8 and 10 minutes. The quake's epicenter was located off the west coast of Sumatra. The loss of life totaled 170,000 from the quake itself. The resulting tsunamis hitting the coasts of most of the Indian Ocean, accounted for 230,000 more dead in 14 countries. This was one of the deadliest natural disasters in history.

Another Sumatran earthquake, magnitude 7.7, occurred off the western coast of Sumatra, Indonesia, in 2010. The ruptured fault that caused it was the same one behind the 2004 Indian Ocean event. A large resulting tsunami struck the Mentawai Islands, causing 435 deaths and the loss of about 4,000 homes.

4.3.4 Haiti Damages Not Far Behind Indonesia

On January 12, 2010, a magnitude 7.0 earthquake hit Port-au-Prince and surrounding regions. It lasted 10 to 20 seconds, leaving around a quarter of a million dead, more than 300,000 injured, and more than 1.5 million homeless. A well recorded damage survey of 47 buildings just in downtown Port-au-Prince indicated that 28 percent had collapsed and another 33 percent were damaged enough to require repairs. A similar survey of 52 buildings in Léogâne found that 62 percent had collapsed and another 31 percent required repairs. The United Nations sent emergency shelter materials to house 1.5 million, safe water was distributed to more than a million, and a million more benefited from cash-for-work programs.

History and Causes

The geologic fault that caused the Port-au-Prince earthquake is part of a seismically active zone between the North American and Caribbean plates. Haiti had suffered similar devastating events in the historical past (in 1701, 1751, 1770, and 1860), but despite this knowledge of historical seismicity, it had no seismograph stations; thus, it was impossible to estimate accurately the intensity of the event's ground motions.

According to the USGS, the following factors led to catastrophic losses of life and property:

- Soil liquefaction, landslides and rockslides in cut slopes, and road embankment failures
- A lack of detailed knowledge of soil conditions (for example, lithology, stiffness, density, and thickness)
- Aftershock activity, which continued for many months (Table 4.6)
- A lack of earthquake-resistant design codes, substandard design, inadequate materials, and shoddy construction practices

Damage Details

Most bridges in Port-au-Prince are simple, unengineered structures that suffered extensive damage. Multi-span bridges on primary routes

Table 4.6 Probabilities of Future Aftershocks

Magnitude	One-Year Period
5 or greater	95%
6 or greater	25%
7 or greater	3%

experienced some damage but remained serviceable. At the main port in Port-au-Prince liquefaction caused the collapse of the North Wharf as well as the westernmost 400 feet of the South Pier. The collapse of a pile-supported pier at the Varreux Terminal resulted in the deaths of about 30 people.

Public safety facilities, schools, government buildings, and churches suffered much destruction, bringing with it the loss of important records, which hampered rescue, recovery, and reconstruction efforts, as did the barely functioning telecommunications infrastructure.

Rebuilding Haiti

The most important factor in the rebuilding of Port-au-Prince is the development of effective land use standards and seismic-resistant design codes for retrofit and new construction. Currently, Haiti has none of these in place. Apart from the lack of proper codes for seismic design and the general lack of standards in construction throughout the country, major engineering flaws, the use of inferior materials, and corruption in the building trades were contributing factors to casualties and property losses. Government intervention will be required in the implementation and enforcement of codes and in the general raising and enforcement of construction standards.

USGS Support: Seismic codes and standards are based on USGS research on geologic conditions, seismic hazards assessments, and site-specific studies of rock and soil conditions and landslide susceptibility. In Haiti, USGS scientists have installed seismic monitoring stations on hard rock and in the softer sedimentary basins to measure the location, frequency, and severity of shaking, giving them the ability to assess the most dangerous and vulnerable areas. In these areas, rebuilding will have to be to the highest standards of seismic resistance or in some cases may not be allowed at all (reference USGS Newsroom, USGS Scientists, Research Help Haiti Reconstruction dated 3/3/2010.).

Possible Causes: Historically, heavy rains and landslides are known to unleash temblors near the fault location. If mass is shifted from the earth surface above an active fault, it may become more active and lead to increased shaking, for several miles from the the fault. This is one of the subjects of interest to Geophysicists in America.

Caribbean Seismicity: The Caribbean is seismically active due to relative motion between tectonic plates. The catastrophe in Haiti has revealed the need for a better understanding of earthquake and tsunami hazards in this region, which is a small-scale "ring of fire" similar to the original ring of fire, the Circum-Pacific seismic belt. Historical events greater than magnitude 7 have occurred in Puerto Rico, Jamaica, the Dominican Republic, Martinique, and Guadeloupe. Along the northern coast of Venezuela, the juncture of the Caribbean and South American plates has caused damaging earthquakes in the vicinity of Trinidad and Tobago. The historical pattern of earthquakes in Haiti indicates that an earthquake of magnitude 7 or larger could strike southern Haiti near Port-au-Prince at any time.

4.3.5 Chile Earthquakes

Chile has a history of large earthquakes. For example, in the 1960s it was struck by a magnitude 9.6 event, the highest recorded in instrumental history (Figure 4.10). On February 27, 2010, disaster struck again, this time a massive magnitude 8.8 earthquake that hit Concepción, killing at least 400 people and destroying much of the city. Interestingly, the Haiti earthquake that occurred a month and a half earlier was less powerful than

Figure 4.10 Fallen tall building in Chile.

Chile's, and yet it killed more than 200,000. The much lower death toll in Chile can be attributed to the country's more stringent building codes and seismic resistant construction.

Damage

Many major bridges collapsed and many stretches of embankment heaved, cracked, or sank. Much road pavement also collapsed and sheared off. Liquefaction, internal erosion, landslides, and faulty construction were the primary reasons for these failures. Making embankments earthquake-resistant is expensive, but this matter of economics may have to take a backseat to the future earthquake survivability of Chile's bridges.

4.3.6 New Zealand's Earthquake Frequency

New Zealand's second-largest city, Christchurch (population 377,000) was hit by an earthquake in February 2011 that measured magnitude 6.3. This followed a 7.1 tremor in September 2010. The loss of life numbered 185, more than half the result of the collapse and burning of the Canterbury Television Building.

The February quake was not as powerful as the one in September, but it was much shallower, leading to greater damage exacerbated by the weakened state of buildings and infrastructure hit by the earlier event and its aftershocks. The epicenter was 10 kilometers southeast of the city, apparently in the middle of the harbor.

The shallowness of the February quake, only 5 kilometers, is why the damage was much worse than that from September's temblor, which occurred about 10 kilometers below the surface (and 40 kilometers to the west in an area of mostly farmland). The existence of a geological fault under Christchurch had been unknown until it ruptured.

Modern Building Codes Helped to Some Extent: There was, of course, much damage to the city and surrounding areas, but New Zealand has some of the strongest building codes in the world, so the casualties and destruction were not as great as they would have been in a city of the same size in a country with few seismic-resistant design standards. Indeed, New Zealand's codes set a world standard in seismic building regulations and are incorporated into the building codes of several countries.

Still, parts of the city had to be abandoned, and thousands of homes, too damaged by liquefaction to be repaired, were demolished. Electricity,

communications, water, and sewer systems broke down, with broken water mains flooding the streets already blocked by debris. Roads were damaged by liquefaction, which also caused buildings to sink or tip.

Older, unreinforced masonry buildings were most vulnerable to shaking, but several recently seismically retrofitted buildings also collapsed, including the Christchurch Cathedral, because of failure of the retrofit methods. Christchurch's only earthquake-resistant structure, Southern Cross Hospital's endoscopy building, was unscathed.

4.3.7 Europe an Seismic Activity

Eastern European countries are not behind the rest of the world in being hit by earthquakes. In July 2012 a magnitude 5.8 earthquake hit the Emilia-Romagna region of north-central Italy. Its epicenter was about 25 miles northwest of Bologna, according to the USGS. Casualties totaled 16 dead and about 350 injured, with 14,000 left homeless. This was the second event to occur in the region in two months. On May 20, 2012, a magnitude 6.0 quake left 7 dead and 5,000 homeless. The last major quake in Italy, in April 2009, was a magnitude 6.3 event that hit the city of L'Aquila in central Italy and caused 295 deaths. Prior to that, in November 1980, a 6.9 magnitude event hit Southern Italy, killing 3,000.

The May 2011 Lorca earthquake, magnitude 5.1, occurred at a very shallow depth of 1 kilometer near the town of Lorca, on the heels of a magnitude 4.4 foreshock that had occurred two hours before. The two quakes resulted in 9 deaths and dozens of injuries, as well as substantial damage to many older structures. The quakes occurred in a seismically active area near a large fault beneath the Mediterranean Sea where the European and African plates brush past each other.

4.3.8 Earthquake Prone Turkey

Turkey is a tectonically active country that experiences frequent destructive events. Movements are controlled by the collision of the Arabian and Eurasian plates, with large, translational fault systems extending across much of Turkey's central and western regions (Figure 4.11).

The devastating Izmit earthquake of 1999 had a magnitude of 7.6. It killed 17,000 people, injured 50,000, and left 500,000 homeless. Approximately 70 kilometers from Izmir an earthquake of 7.3 magnitude had occurred in November 1976 near the Turkey–Iran border, killing several thousand people. A magnitude 7.8 struck Erzincan in 1939,

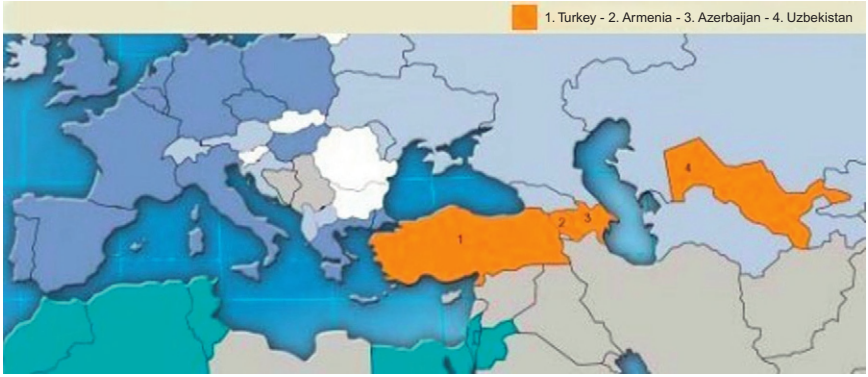

Figure 4.11 Locations of Turkish and Western Asia recurring earthquakes.

leaving an estimated 33,000 dead. The author had a chance to obtain first hand information in 1980s after the Erzurum earthquake and participated in a conference organized by UNESCO at the Middle East Technical University in Ankara. Considerable progress has been made in design and construction methods and research activity has increased. A more seismic resistant building code has been developed and casualties or damage in a future earthquake is likely to be much less.

4.3.9 China an Active Seismic Region

Between August 19 and September 3, 2008, a series of major earthquakes ranging in magnitude from 4.1 to 5.9 struck Yingjiang County in southwestern China's Yunnan province near the border with Myanmar. At least 25 people died and 250 were injured, and more than 1,000 structures were destroyed. Chinese engineers are now paying more than casual interest in making buildings and dams as seismic resistant.

4.4 ANALYSIS OF OBSERVED DAMAGE

At this point some generalizations can be made about the reasons for the damage caused by the earthquakes discussed in this chapter. No two earthquakes are alike and there are many unknowns in the causes and effects. As we have seen, such damage can be most extreme for several reasons, among them growing populations in areas of high seismic activity, lack of seismic monitoring, lack of seismic design codes, shoddy construction using inferior materials, and old, weakened bridges atop failing embankments.

Specific factors include the following:

- Improper construction sites, such as foothills or location close to a fault
- Poor detailing of column reinforcement
- Soft-story collapse (caused by the absence of adequate shear walls at ground level)
- Seismically weak soft stories on first floors
- Long cantilevers with heavy dead loads
- Shifting and sliding of foundations due to liquefaction or structural rigidity
- Lack of a base isolation system
- Poor mortar
- Inadequate roof-to-wall ties
- Lack of reinforcement or ductile joints.

4.4.1 Nonlinear Structural Response and Torsion

Unfortunately, buildings are not uniformly damaged, and the change in structural properties after damage (nonlinear response) often concentrates seismic displacement in one location. Torsion in a building is a twisting in plane caused by an imbalance between the location of the mass and the resisting elements. Older buildings often concentrate lateral strength and stiffness on one end, an elevator or stair tower, for example and a small wall or frame at the other end to prevent torsion. Current codes contain many rules to minimize configurations that could cause nonlinear response, as well as special design rules for elements potentially affected (e.g., columns supporting discontinuous shear walls).

Limitations: Buildings meeting current codes do not suffer complete or partial collapse nor do they lose massive portions of their structure or cladding. However, internal damage may be severe, including local structural and nonstructural damage that poses a risk to life safety. In fact, many buildings in this damage state are a complete economic loss, but because they do not collapse, injury and loss of life can be minimized.

4.4.2 Engineering Seismic Design

An Innovative Approach: Once lateral loads for seismic design have been determined by evaluating soil type, building natural period, importance

level, and working life, the following may be incorporated into a seismic-resistant design to isolate a building from ground movement:

- Base isolation
- Glass walls that act as "frames within frames"
- In unreinforced masonry (URM) buildings, walls, floors, and roof tied together with mechanical fixings, including steel rods and dowels, to make the building behave like a rigid box. This may however be an expensive solution.
- Resilient brick walls using post-tensioning (the insertion of steel rods vertically in walls or attached to the outside, which are then pulled tight)
- Thin membranes and coatings, such as fiber-reinforced polymers applied to internal walls so they hold together and strengthen mortar
- Extra support where the column joins the floor and column and beam joints to resist sideways movement (jacketing columns with light-weight membranes, bolting walls to columns, and steel connections between column and beam)

New Developments: The most earthquake-resistant framing methods known worldwide include the precast seismic structural system (PRESSS), which uses concrete, and New Zealand's version of PRESSS, prestressed laminated timber building system (Pres-Lam). Buildings that incorporate PRESSS and Pres-Lam flex and rock on their foundations and thus withstand powerful shaking.

4.5 CONCLUSIONS TO EARTHQUAKE DAMAGES AND MEASURES BEING TAKEN

Because of modern seismographs and sensors, more seismological and site-specific acceleration response spectral data are available nowadays. Using these data and analyses of observed damages and structural failures from seismic events, such as those discussed in this chapter, has led to the development of modern, effective seismic codes, which should be introduced, implemented, and enforced worldwide, particularly in countries that are seismically vulnerable. These codes incorporate many technological innovations designed to keep buildings from collapsing by making them stronger or by making them more flexible so that they sway and slide above the shaking ground.

Much is to be learned from studying past earthquakes, but it is important to remember that each disaster is a subject unto itself because of

seismology, soil conditions, and the structural responses it involves. Each earthquake is like an open book to study. Structural engineers would continue to rely for dynamic analysis and design methods on the expertise of geologists and seismologists. Although progress has been made worldwide, in mitigation and disaster preparedness but forecasting accurately exact location or timing of the event is still not possible. "A journey of a thousand miles begins with a single step" and the first step has been made. Increased interest in this most sensitive engineering subject, at professional and at top administration levels would be desirable and helpful in meeting the great challenge.

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San Francisco: USGS, http://earthquake.usgs.gov/regional/nca/1906/18april/index.php
Loma-Prieta: ASCE/SEI; Ward and Page 1990
Haiti: Eberhard et al. 2010
New Madrid and Charleston: Fillingim 1998; Spence et al. 1993
Northridge: Holmes 2000; Cooper et al.
Turkey: Gulkan 2003
Skopje, Yugoslavia: Marjorie Greene 1987
Iran: Khazai and Hausler 2005; Tierney 2005
Indian Gujarat Earthquakes: Government of Maharashtra, Maharashtra Emergency Earthquake Rehabilitation Programme 1997, 1998; American Society of Engineers (ASCE)/Structural Engineering Institute (SEI) 1994; Gujarat State Disaster Management Authority (GSDMA) 2002; National Information Centre of Earthquake Engineering (NICEE) 2005

CHAPTER FIVE

Risk Assessment, Mitigation, and Remediation

Chapter Outline

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5.1 INTRODUCTION

This chapter broadly reviews the types of risk assessment in terms of damage and destruction of property and loss of lives, based on which preparedness or mitigation and the type of remediation can be recommended.

A detailed literature review shows contributions by the following organizations and design codes:

Architects and Earthquakes: American Institute of Architects (AIA, 1975)

Earthquake Resistance of Highway Bridges: Applied Technology Council (1979)

International Building Code: International Code Council (2000)

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Proceedings of International Conference on Earthquake Engineering (2006), UET Lahore, Pakistan.

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The following authors have contributed to several soil aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to the study of latest earthquakes is presented here:

Seismic Behavior of Structural Concrete Linear Elements (Beams, Columns) and their Connections, Proceedings of the ATCAP-CEB Symposium on Structural Concrete under Seismic Actions: Bertero (1979)

Strength and Deformation Capacity of Buildings under Extreme Environment: Bertero (1980)

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On Seismic Design of Eccentrically Braced Steel frames: Kasai et al. (1984)

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Need for Risk Assessment, Mitigation, and Remediation:

In the United States there is great awareness of the dangers of earthquakes, which is manifested in increased allocation of funds for research. Organizations such as USGS have prepared detailed information on seismology, soil conditions, and dynamic effects. Others, such as ICC, AASHTO, IHEE, and the U.S. military have developed seismic codes that are currently in use for new construction as well as retrofit for bridges and buildings, nuclear power plants, dams, and the like. Seismic retrofit methods, such as base isolation, are now available.

The current difficulty we are facing, however, is that most existing structures are not seismic-resistant and so are at great risk. Lack of enforcement of building codes, unsafe land use patterns, and poor construction practices contribute to high human and economic losses. Moreover, changes in climate, demography, and urbanization will increase future losses from disasters.

Consider the damage forecast prepared in 2008 for the USGS by geophysicists and engineers, which envisions a magnitude 7.7 quake rupturing the San Andreas Fault in the desert east of Los Angeles and radiating into the nation's second-largest metropolitan area. The projected effects include

- 2,000 dead, 50,000 injured, and 250,000 homeless
- 1,500 buildings toppled, 300,000 badly damaged
- $200 billion in property and infrastructure losses
- 1,600 fires, some growing into conflagrations engulfing hundreds of city blocks

According to the forecast, the biggest long-term economic disruption would come from damage to water-distribution systems that would leave some homes and businesses without running water for months. However, it does not foresee damage to the nearest of the state's two nuclear power plants, the San Onofre station between Los Angeles and San Diego. In fact, both Edison and Pacific Gas & Electric, owners of the Diablo Canyon plant to the north at San Luis Obispo, say their facilities are built to withstand quakes far greater than nearby faults are capable of producing.

The report recommends an 18-point plan for implementing the U.S. Strategic Hazards Reduction Program, which was established by Congress in 1977. Some highlights:

- Better public education
- Short-term forecasting and early warning systems
- Improved building design for earthquake resilience
- Better techniques to retrofit old buildings
- Enhanced simulations and evaluation of critical infrastructure, including electricity, highways, and water systems

These recommendations encompass the three aspects of earthquake response: physical risk assessment and mitigation, which take place before an earthquake occurs, and remediation, which takes place after an event, in both the short and the long term.

Mitigation, hazard assessment, and remediation require answers to the following questions:

- When will the next event take place?
- Where are seismic zones and where will the next event takes place?
- What magnitude and intensity of forces and deformation will the next event generate and how will they be resisted?
- Who is responsible for taking remediation measures?

The answers to these questions will involve short-term forecasting and early-warning systems, better structural design for earthquake resilience, better retrofitting techniques for existing structures, and enhanced simulations and evaluation of critical infrastructure, including electricity, highways, and water systems.

Physical risk assessment refers to the process of identifying and evaluating earthquake hazards. Hazard-specific risk combines both the probability and the level of impact of a specific event. Assessment of risk involves the following:

- Seismic risk/vulnerability: the potential of future earthquakes, including the identification of natural causes besides tectonic plate movement, such as changes in climate and demography, and soil liquefaction effects
- Avoidable deficiencies in the design of structures to increase their seismic resistance
- Enforcement of building codes and investigation of unsafe land use patterns, urbanization, and poor construction practices

Definition: Mitigation is the most cost-effective method for reducing the impact of earthquakes because it focuses on long-term measures for reducing or eliminating risk. Mitigation efforts attempt to prevent earthquakes from developing into disasters altogether or at least reduce their potential effects

Definition: Remediation is the correction of defects or deficiencies, leading to improvement based on lessons learned from the analysis of seismic events, as well as quick and effective coordination of post-earthquake rehabilitation. In this respect, earthquakes may be considered a wakeup call for safer design and construction of structures. New Zealand, for example, looking to rebuild following the earthquake and aftershock in Christchurch, has had the opportunity to rebuild smarter infrastructure and buildings out of the debris and rubble.

Seismic-Resistant Mobile Crèches In Mumbai

Safety Considerations in Earthquake: The Mumbai Mobile Crèche Organization provides schools (Figure 5.1) for the children of laborers living on temporary construction sites in India. As construction workers move from project to project, the crèches' superstructure components can be dismantled and brought along. However, Mumbai is located in an active seismic zone where two major

earthquakes have occurred recently, and the crèches currently in use (Figure 5.2) are nonengineered and non-seismically resistant.

Local Government Requirement for Education of School Going Children: The author visited several construction sites with a team of architectural students and subsequently designed a precast 3-pinned portal frame building and a precast roof panel (Figures 5.3 and 5.4). Structural members and connections are ductile moment–resisting; precast concrete segments are readily available in India.

Figure 5.1 Temporary mobile crèches serving as schools for children living on temporary construction sites.

Figure 5.2 (*left*) Nonengineered self-standing tin wall serving in a Mumbai mobile crèche school; (*right*) children assembled in a temporary mobile crèche.

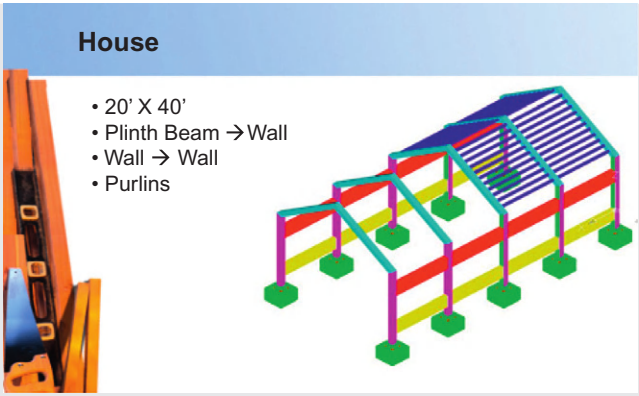

Figure 5.3 Seismic-resistant design of crèche detachable components.

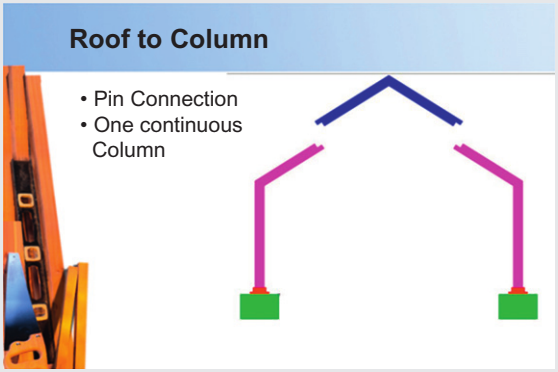

Figure 5.4 Connections between precast elements designed to resist seismic forces in mobile crèches.

5.2 EARTHQUAKE PREDICTION FOR RISK ASSESSMENT AND MITIGATION

Some progress in seismology is being made in understanding the fault mechanics that generate earthquakes. Movements of tectonic plates are being monitored, and very small movements associated with a causative fault can be located with greater accuracy than before. In many cases

the movement patterns near faults are consistent with past patterns at these locations.

Many regions live with the seismic risk of a rare, large intraplate earthquake, whose cause is often uncertain. The fault where tectonic plate moves may be deeply buried and sometimes cannot even be found.

It is difficult to calculate the exact seismic hazard for a given city, especially if there is only one prior event to analyze. Before it was equipped with seismic sensors, southern California faced problems. During the 1971 Sylmar earthquake (also known as San Fernando Earthquake located near Los Angeles), authorities first focused on the mountains north of the San Fernando Valley, which was the epicenter of the quake, before they realized that the fault ruptured southward toward populated areas. Limited success has come with detailed micro-seismic monitoring, which involves dense arrays of seismometers.

5.2.1 Early Identification of Risks

Seismologists may not be able to foretell if a quake is likely to hit any day. However, seismo-tectonic analysis may indicate that similar or larger ones in the same or a neighboring regions are possible. The signs and indications of vulnerability may derive from repeat events, records of tectonic plate movements, and the presence of fault regions.

5.3 RECENT DEVELOPMENTS IN SEISMIC SCIENCE AND TECHNOLOGY

A Review of Recent Progress: Innovations in seismic science have been achieved from the lessons past earthquakes have taught us. The sorrow of so many who have lost so much has not been in vain.

Unfortunately, these innovations are not quick fixes, but will slowly accumulate as older structure—both bridges and buildings—designed according to older nonseismic-resistant codes are either replaced or retrofitted. It is a sad irony that this will come for many as the result of destructive earthquakes. Happily, new structures can and will be "seismically designed" according to current codes. Moreover, important structures such as major bridges, civic buildings, hospitals, and schools can be retrofitted according to their locations in seismic zones.

5.3.1 Discovering New Faults

Scientists believe there may be hundreds of unknown faults around the world. Using state-of-the-art seismic techniques, they are searching for and discovering many new faults, some of which are active, meaning that the danger of a rupture is significant. Identifying areas of earthquake potential is the first step in developing and implementing earthquake protection measures.

A new fault near Memphis, under the area surrounding New Madrid, Mississippi, in the Lower Mississippi Valley was revealed by scanning sediments located thousands of feet beneath the Mississippi River bottom. The fault is a mile deep and stretches at least 25 miles (from Meeman-Shelby Forest State Park to the Ensley Bottoms area). It lends credence to seismologists' belief that quake activity "migrates" within the valley. This seismic zone is found to trigger hundreds of small quakes each year. The 1811–1812 New Madrid earthquake generated a series of massive shocks greater than magnitude 7 in strength.

Researchers now know that seismic hazards extend beyond the New Madrid zone, migrating within the larger Mississippi Embayment, a former inland sea filled with sediment deposits. The migration results from the transfer of stress by one earthquake to another part of the embayment, which explains why some faults are dormant while others are active.

A new earthquake-producing fault, the Polaris, was discovered in California, already one of the most seismically active regions in the world. It came as a surprise to scientists, who thought that all of the state's seismically active regions had been mapped. The U.S. Army Corps of Engineers found the fault by accident, using an airborne laser-imaging technology known as LiDAR to obtain high-resolution topology maps. Once a layer of heavy pine trees was stripped off the maps, experts saw evidence of the fault sitting just 200 yards from the Martis Creek dam.

The Polaris Fault is 22 miles long. Although its most recent seismic activity is estimated to have been 15,000 years ago, it is powerful enough to cause a magnitude 6.9 earthquake, which could destroy the Martis Creek dam and flood the surrounding valley. According to a study published in the *Bulletin of the Seismological Society of America*, because the Polaris connects to two other faults near the dam, the magnitude could be even higher if all three ruptured at the same time.

Throughout the east face of the Sierra Nevada Mountains, there are probably quite a few undiscovered systems that may be responsible for a large portion of tectonic movement.

5.4 RECENT INNOVATIVE SOLUTIONS

An Update on Progress Made: Earthquakes cannot be prevented, but we can know where they are likely to occur by mapping and monitoring geological faults. The next step, of course, is to make the regions where the faults are known to exist as earthquake-resistant as possible. Seismologists and engineers, working together, are developing new construction and retrofitting techniques to minimize the casualties and property losses that are the inevitable result of a major seismic event.

Rapid advances have been made in earthquake response modification technologies for structures, most notably in base isolation and energy dissipation. Many recent developments are improving a structure's chances of surviving ground shaking. Among these are high-performance concrete and high-performance steel for lighter weight and greater resistance; seismic bracing systems; and isolation bearings and dampers that reduce peak seismic forces and moments.

Other innovations include accelerometers for recording magnitudes in hard-rock environments where seismic events have higher-frequency content, geophones in soft rock or sandy environments, where seismic events have lower-frequency components. Geophones are often paired with strong-ground-motion systems to monitor large events.

5.4.1 Base Isolation

An Effective Technique in Dissipating Seismic Forces: The technology for preventing structural collapse works essentially in one of two ways: Structures can be made stronger, or they can be made more flexible so they sway and slide above the shaking ground rather than crumbling. The latter technology, known as base isolation, is a major advance in seismic-resistant construction. It allows skyscrapers to float on systems of ball bearings, springs, and padded cylinders, which absorb shocks. In a major earthquake, buildings can sway up to a few feet, bending but not breaking. This technology was first developed in Japan and New Zealand in

response to major earthquake damage; it is now used in the United States, China, Russia, Italy, Chile, and other countries.

The Imperial Hotel of Tokyo, built by the architect Frank Lloyd Wright, is a prototypical example of base isolation. Built over a sand bed, it floated like a ship during an earthquake in 1923 and resisted ground shocks. San Francisco's City Hall is at present the largest isolated building on earth. Seriously damaged by the Northridge earthquake, it underwent a major structural rehabilitation using state-of-the-art base isolation. Another base-isolated building, the hospital of the University of Southern California in Los Angeles, survived a 1994 earthquake. A neighboring building that did not use the isolation technology suffered considerable damage.

Engineers are developing electronic sensors to improve on base isolation technology. The sensors detect seismic shaking and "tell" the building how to react. If the shaking exceeds a certain level, a shock-absorbing damper system goes into action to reduce it.

5.4.2 Damping

Use of New Materials: Perhaps the innovations with the greatest potential for earthquake resistance are advancements in damping technology. Damping reduces vibration to return the structure to its original position. For example, a super-elastic ferrous alloy developed by Japanese researchers may protect the integrity of buildings during earthquakes by stretching to absorb shocks and resist vibration and stress, and then returning to its original shape. This material is one of a family of smart materials called shape memory alloys (SMAs), the unique properties of which significantly reduce the damage associated with severe earthquakes by deforming to absorb the energy from ground shaking, thus allowing the building to maintain its stability. Nitinol, an alloy of nickel and titanium, is another SMA that works in a similar way. Still in the experimental stage at the University of Newcastle, Nitinol will work in a series of dampers at the bottom of buildings, absorbing and dissipating the energy of the shaking earth to prevent it from tearing the building apart. Nitinol and other SMAs are at the cutting edge of earthquake-proofing technology.

5.4.3 Building Materials

Buildings constructed to current building design standards might withstand collapse and thus prevent loss of life, but they would suffer significant damage from an earthquake of significant magnitude. However,

innovative building materials have been shown to mitigate or even prevent such damage. For example, after the 2011 Christchurch, New Zealand, earthquake, the only unscathed building was Southern Cross Hospital's endoscopy center. The reason for its survival was a new timber technology known as Pres-Lam, developed by a research consortium, Structural Timber Innovation Company, led by Canterbury University and the New Zealand building industry. Pres-Lam is an engineered prestressed laminated timber, almost as strong as concrete, that allows buildings to flex and rock on their foundations in an earthquake.

Pres-Lam is a version of one of the most earthquake-resistant building systems known worldwide, the precast seismic structural system known as PRESSS. PRESSS allows controlled rocking of joints, which softens the blow of an earthquake, springing the building back to upright. The technology is similar to base isolation but at a fraction of the cost.

Seismic Design for Bridges

Research by CALTRANS: As a major highway agency in an active seismic zone, the Calfornia Department of Transportation (Caltrans) has been at the forefront of innovative seismic-resistant bridge design and retrofit. A recent Caltrans publication describes the development of strategies to improve bridge performance through problem-focused research and advances in bridge seismic design and retrofit standards. These advances include new materials, structural systems, and construction methods.

The current era of Caltrans seismic design practice began with lessons learned from the 1971 San Fernando earthquake. Prior to this event only rudimentary consideration was given to designing bridges for ground motions. Caltrans created a Post-Earthquake Investigation Team (PEQIT) that investigates damage to bridges after all earthquakes and makes recommendations. The following paragraphs describe a few examples of PEQIT's investigations.

On October 1, 1987, the Magnitude 6.0 Whittier Narrows earthquake shook the Los Angeles area, causing significant shear damage to the I-605/I-5 Separation. Based on PEQIT's investigation, Caltrans began developing contract plans to retrofit bridges with single-column bents (a substructure unit supporting each end of a bridge span). It continued the process of retrofitting over 1,000 bridges with restrainer cables to resist collapse due to unseating at hinges and bent supports.

On October 17, 1989, the Magnitude 7.1 Loma Prieta earthquake hit the San Francisco Bay Area, resulting in the collapse of one section of the

east span of the San Francisco-Oakland Bay Bridge on I-80, portions of the double-deck Cypress Viaduct structure on I-880 in Oakland, and the Struve Slough Bridge on Highway 101. Other double-deck structures, including the I-280 Southern Viaduct, suffered significant joint damage, but did not collapse. Much of the damage occurred at sites with deep cohesive soils where ground shaking was amplified, causing increased displacements of structures with longer periods (time interval required for one full cycle of a seismic wave).

On January 17, 1994, the magnitude 6.7 Northridge earthquake occurred in the same general location as the 1971 San Fernando earthquake in southern California. Five bridges collapsed and an additional four bridges had major damage. Bridges designed to modern standards and those that had been seismically retrofitted performed well; those that had been neither seismically constructed nor retrofitted suffered, particularly those with irregularities in their configuration, which pointed up the fact that geometric effects could adversely affect bridge performance and cause permanent damage. As a result, design innovations were developed during the subsequent rebuild of damaged or collapsed structures. These included the use of double-cantilever seatless hinges and providing isolation wells to extend the effective length of short columns.

5.4.4 Caltrans Seismic Retrofit Program

With the replacement of the east spans of the San Francisco-Oakland Bay Bridge, the Caltrans Seismic Retrofit Program is nearly complete. For state bridges, a nontoll seismic retrofit program is used. Local agency bridges in California that are not on the state highway system received an influx of funding with the passage of Proposition 1B in November 2006. Most of the 1,235 local agency bridges have been retrofitted, with many currently in the design phase.

The Evolution of Caltrans Seismic Design Standards

In February of 1971, Caltrans released a new "Memo-to-Designers" modifying its design standards to increase the amount of transverse column reinforcement and requiring the inclusion of a top mat of reinforcement in footings and pile caps. In addition, details were provided to add hinge restrainers to existing hinge seats, and hinge seat lengths were increased to minimize the risk of unseating. These new details were to be applied to all new designs and incorporated into bridges under construction.

In 1973, Caltrans worked with the California Division of Mines and Geology (since renamed California Geological Survey) to develop a statewide fault map and began designing for ground motion accelerations tied to bridge site location. To address what was seen as the most serious seismic vulnerability, Caltrans embarked on a seismic retrofit program to install cable hinge restrainers to limit the risk of superstructure unseating.

In 1981, Caltrans seismic design specifications were updated again following publication of the results of ATC-6 by the Applied Technology Council.

As familiarity and use of displacement ductility methods grew during the decade following the Loma Prieta earthquake, Caltrans' design philosophy and accompanying criteria evolved. A team was formed and the new methodology was documented with the December 1999 publication of the new displacement-based Caltrans Seismic Design Criteria (SDC). The philosophy of the SDC is to design structures incorporating the following elements:

- Displacement-based methodology
- Adequate confinement to ensure ductile response of columns
- Capacity protection of the superstructure and foundation to force plastic hinging into the well-confined ductile columns
- Balanced geometry and mass/stiffness compatibility to share seismic protection among the ductile columns and avoid the concentration of damage in just a few locations
- Redundancy such that the overall bridge system performs well even if an individual component may be significantly damaged
- Adequate support length to accommodate anticipated displacements
- Column retrofit using steel casings
- These provisions incorporated results published in ATC-32 and NCHRP 12–49, results of Caltrans' seismic research program, and design practices developed through the experiences of the Seismic Retrofit Program.

An example of Caltrans' innovative thrust is its use of machines that simulate a large truck that can be used to conduct stress tests on the rubber joints between bridge segments. The swaying is picked up by motion detectors, which relay the data to a Caltrans control center in Sacramento.

5.4.5 USGS National Seismic Hazard Maps, Data, and Documentation

USGS's National Seismic Hazard Maps display ground motions for various probability levels across the continental United States and are applied in seismic building codes, insurance rate structures, risk assessments, and other public policy. Recent updates incorporate new findings on ground

shaking, faults, seismicity, and geodesy. The resulting maps are derived from seismic hazard curves calculated on a grid of sites across the country that describe the frequency of exceeding a set of ground motions.

Global Seismic Hazard Assessment Program (GSHAP)

The Global Seismic Hazard Assessment Program (GSHAP), a demonstration project of the UN/International Decade of Natural Disaster Reduction was conducted in 1992–1998 with the goal of improving global seismic hazard assessment. The GSHAP Global Seismic Hazard Map was compiled by joining the regional maps produced for different GSHAP regions and test areas; it depicts the global seismic hazard as peak ground acceleration (PGA) with a 10% chance of exceedance in 50 years, corresponding to a return period of 475 years.

5.4.6 New Quake Warning System Installations

Need for Early Warning System: In terms of mitigation, early warning systems could save thousands of lives by giving people at least a fleeting chance of surviving a major event. For example, Japan, which is far ahead of the United States in early warning technology, built a simple system in the 1960s to shut down power to high-speed trains during earthquakes. At the epicenter of a quake, a warning may provide only a second or two of notice. However, the farther from the epicenter, the longer the window of opportunity to prepare,—according to USGS this could be from several seconds to more than a minute. Even a few seconds' warning allows people to dive under a sturdy table to avoid falling objects or to step back from a pot of boiling water. The system does not predict earthquakes; rather, it detects quake primary waves, or P waves, and sends alerts if they exceed a threshold. After the P waves, the more destructive secondary, or S, waves arrive, traveling at about 2.5 miles per second.

Warning Systems in the United States

A network of sensors along the West Coast gathers real-time seismic data, but the challenge is swiftly spreading the word. Traditional warning systems include emergency broadcasts and community sirens. However, new, faster methods are being developed. For example, USGS, UC Berkeley, and two other universities are testing a prototype that would sound an alert seconds to a minute before a major quake. Companies like Google and Deutsche Telekom are exploring how to almost instantly disseminate warnings via computers and cell phones, whose microprocessors are extremely sensitive

to motion. Two cell phone carriers already send early warnings to their 21 million customers, and a third one plans to start soon. With access to nationwide networks of seismic stations, broadcasters can deliver warnings seconds after an event begins. BART, San Francisco's public transit system, is an early tester of the West Coast prototype.

Installing Quake Warning Sensors in Mexico

U.S., Mexico to install quake sensors in two border city areas. The devices, to be set up in the Mexicali and Tijuana regions, will let scientists swiftly identify worst-hit areas after a quake and tell authorities where to send emergency crews. During the Mexicali Earthquake a wide fissure was caused (Figure 5.5) by the 7.2 earthquake centered south of Mexicali that struck on April 4, 2010. Outdated quake detection equipment in Mexico meant that hours went by before officials knew exactly where the worst-hit areas were. The Mexican quake system will be linked with satellites to keep data flowing after the Big One. The Mexican government is spending $50 million to upgrade its own monitoring system.

In the recent Sierra El Mayor 7.2 magnitude valuable warning time was lost as U.S. scientists could only guess exactly where the epicenter was rather than the precise locations that would have shown which fault was responsible for the quake. Only as the aftershocks developed, officials could see which direction the aftershocks were headed.

Figure 5.5

5.4.7 Improved Seismic Design Codes

Seismic Codes Revised Every Few Years: Current technologies prevent well-designed buildings from cracking when the ground shakes beneath them. These are being incorporated into the building codes of many developed nations. Recall that the difference in the devastation caused by the earthquakes in Haiti and Chile was the result of Chile's better designed and implemented codes. This highlights that the real issue is not so much the technologies themselves (many of which require only simple changes to building materials or composition) but their adoption and enforcement. Updated seismic-resistant design codes are needed, and implementation must be enforced at all governmental levels. Of note is the fact that the cost of constructing new earthquake-safe buildings is about 5 to 10 percent more than the cost of constructing those without any precautions.

According to International Code Council (ICC), a nonprofit group that develops global building standards,

> *The countries that have not adopted the codes tend to be poorer countries and perhaps the degree of sophistication or commitment to code enforcement is also an issue in these countries. It would be a start for more developing countries to adopt building codes that include measures about resistance, but that wouldn't fix everything. People all over the world, and with all job types, from city planners to construction workers, need to be aware of technologies and building methods that prevent buildings from collapsing. You can write a really good code, but you'd better have capacity to enforce it, you've got to have people on ground that are trained and certified in codes and are willing to enforce codes.*

Specialized Training Receivers

Continuing Education: The need for training in seismic analysis, design and retrofit, and preparedness to mitigate the effects of disasters is crucial. Apart from engineering issues, social issues must be a focus of mitigation, which is why the stakeholders in earthquake preparedness range far beyond engineers and construction professionals. They include

- Designers and architects
- Development agency officials
- Skilled laborers such as masons
- Policy makers, urban and rural planners, community leaders, nongovernmental organizations, public representatives.

5.4.8 Seismic Retrofitting for Mitigation

Performance of Older Buildings: ASCE seismic committees have been developing new retrofit standards, which will be ready for release as this book goes to press, under the title *Seismic Evaluation and Retrofit of Existing Buildings.* Two of the more significant changes include updated liquefaction and other site hazard provisions and updated foundation-rocking provisions.

Retrofitting old buildings to modern standards is much more expensive than constructing new earthquake-safe ones. Consider the city of San Francisco's retrofit of its city hall with base-isolation technology. That project, which included other improvements, cost nearly 300 million dollars. High costs keep countries such as Haiti from adopting the latest building and retrofitting techniques and technologies. However, not all retrofitting need be complex and costly. For example, making buildings more basic might actually make them stronger and may cost less than high-tech upgrades. Simple square buildings that are relatively stout do very well in shaking. Simple retrofitting techniques exist, such as reinforcing concrete buildings with steel rods and bolting wooden buildings to their foundations to help them withstand ground shaking.

5.4.9 Seismic Prioritizing

Priority Fixing: The higher the risk, the more urgent it is to target hazard-specific vulnerabilities of existing structures through mitigation such as retrofitting. If there is no vulnerability (say, in a desert with no inhabitants), there is no risk. For this reason, there must be a method of determining which structures should be retrofitted and when, using various rating schemes.

5.4.10 Catastrophe Modeling

Catastrophe modeling uses computer-assisted calculations to estimate the losses from a catastrophic event such as an earthquake that could be sustained by a portfolio of properties. It is especially applicable to analyzing risks in engineering, meteorology, and seismology.

One aspect of catastrophe modeling is performance damage levels. By way of example, descriptions of FEMA performance levels (FEMA, 2000), which cover the full range of performance, are given here:

Operational. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural and nonstructural components. Although possibly impaired, they are suitable for normal

occupancy and use, with power, water, and other required utilities provided from emergency sources. The risk to life safety is extremely low.

Immediate Occupancy. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural elements and only minor damage to their nonstructural components. Although immediate reoccupancy is possible, some cleanup and repair may be necessary and there may be a wait for utilities to be restored to normal functioning. The risk to life safety is very low.

Life Safety. Buildings meeting this performance level may be extensively damaged in both structural and nonstructural components. Structural repair may be required before reoccupancy, and the combination of structural and nonstructural repairs may be deemed economically impractical. The risk to life safety is low.

Retrofitting Priorities

Retrofitting priorities are rated on the basis of a structure's vulnerability: girder connections, footings, and weak soils; seismic and geotechnical hazards; and importance. Failure types can be classified as catastrophic, partial, and structural damage.

Vulnerability is evaluated and classified as follows:

High

- Spread footings not tied in both directions
- Simply supported girders supported on corbels. Corbels only offer simple gravity supports to girder ends. Without continuity connection the applied bending moment from forces is shared by girder alone. Connection needs to be much stronger to transfer bending moment from seismic forces.
- Lack of shear walls
- Minimum number of girders, causing poor redundancy and little resistance to collapse
- Lateral resistance lost at an edge
- Lack of symmetry with irregular frames or skew in plane

Moderate-high

- Tendency of high-rise buildings to overturn

Moderate-low

- Single-storied multi-span structures, columns, and foundation types
- Spread footings on rock or pile foundations

Low

- Single-storied, single-span structures with connections limited to beams and columns
- Substructure tilting, settlement of column foundations, and shear and flexure

The quantitative portion of the seismic rating system consists of structural vulnerability and seismic hazard. The qualitative portion is an overall priority index consisting of importance factor, remaining useful life, nonseismic deficiencies, and redundancy. Retrofit priority may be defined as $P = f$ (R [importance, nonseismic and other issues]), where R = rank based on

- Structural vulnerability and seismicity
- Column vulnerability due to shear failure
- Column vulnerability due to flexure failure at column reinforcement splices
- Column vulnerability due to foundation deficiencies

Vulnerability Rating

An Important Guide: The vulnerability rating scale (VRS) is based on previous studies and as reported in the literature. It can be summarized as follows:

- Immediate priority for remedial work
- High priority for remedial work
- Priority for remedial work based on capital program
- Low priority based on inspections and monitoring
- No action if likelihood of failure is remote
- No exposure to specific type of vulnerability.

It is important to know how seismic-resistant the structure is. In this sense, VRS can be summarized as follows:

- Not applicable (no exposure to specific type of vulnerability)
- No action (remote likelihood of failure)
- Inspection program action (low priority based on inspections and monitoring)
- Capital program action (priority for remedial work based on capital program)
- Safety program action (high priority for remedial work)
- Safety priority action (immediate priority for remedial work)

Liquefaction Vulnerability Rating

Structures with discontinuous superstructures and nonductile supporting members have a higher vulnerability than continuous superstructures.

A high liquefaction vulnerability rating (LVR) is assigned for severe liquefaction; a moderate LVR, for moderate liquefaction; a low LVR, for low liquefaction. Liquefaction retrofit methods include

- Isolation bearings
- Dampers
- Restrainers
- Galvanized wire mesh wall reinforcement
- Snubbers

5.4.11 General Considerations for Retrofit

Cost and funding, functional requirements, soil conditions, geometry, constructability, alternate design, aesthetic requirements, new technologies, and innovative methods are all involved in the decision of whether to go forward with a retrofit project. The decision process requires the following steps:

- Based on safety rating, prepare design and recommend repairs and retrofit to achieve required factors of safety.
- Based on inspection and rating, recommend demolition of all unsafe buildings beyond repair because of weak foundations or connections.

5.5 SIMULATIONS AS ANALYSIS AND DESIGN TOOLS

Simulation is understood as a re-creation of local effects of strong earth shaking. Dynamic experiments on bridges and buildings may be physical, such as shake-table testing, or virtual in which the real structure is simplified and natural time history of seismic acceleration is assumed. Actual time histories from the natural site as recorded by seismometers are preferred. There is a strong incentive to use simulation in which the seismic input possesses only the most essential features of a real event and simulation is therefore not as accurate as the data from real event. The stress results may be on the higher side.

Early simulations were performed by statically applying horizontal inertia forces. Equivalent static forces were based on scaled peak ground accelerations to a mathematical model of a structure. With the development of computational technologies, static approaches began to give way to dynamic ones such as shake tables (the most prominent is now E-Defense Shake Table in Japan) and multiple degrees of freedom in which time-dependent varying displacements and accelerations are applied.

5.5.1 Kinematic Modeling

A building model is said to have similarity with the object it represents if the two share geometric similarity, kinematic similarity, and dynamic similarity. The most vivid and effective type of similarity is the kinematic. Kinematic similarity exists when the paths and velocities of a model's moving particles and those of its prototype match. The ultimate level of kinematic similarity is kinematic equivalence in which time histories of each story's lateral displacements in the model and those in its prototype are the same.

Representing Structural Likeness or Similarity of Prototypes

Limitations: While experimental structural likeness or similarity is relatively easy to reproduce in a laboratory (by using physical scaled effects of the prototype), theoretical models are based on many mathematical assumptions. Similarity may be defined as some degree of analogy or resemblance between the prototype and laboratory models. However, the concept of structural likeness or similarity has its limitations:

- In a seismic event, the accelerations may not follow a well-defined pattern and may be random. This can be seen from the plots of acceleration versus time recorded by accelerograms.
- Actual accelerations are site-specific and depend on soil conditions, liquefaction effects, and the strain rate of tectonic plates (close to the epicenter).
- The physical interaction of nonstructural items is complex. Their effects are generally neglected in a theoretical model. In scaled models, it is possible to simulate in-filled frames, for example. There is resistance to any movement from heavy furniture, cladding, utility pipes, and floor finishes, the dead weight of which is expected to contribute to generation of seismic forces.
- The many connection details between floors and beams, beams and columns, shear walls, elevator shafts and columns and footings cannot be fully represented in laboratory models.

The following structural groups can be classified into different categories for modeling:

- Soft-story effects in high-rise buildings: twisting of columns, stress concentrations accompanied with the tendency to overturn
- Irregular geometry due to unequal spacing of columns and unequal beam stiffness

- Girder end conditions: moment connections sometimes replaced by simply supports; girders supported on corbels or having partial moment connections
- Spread footings not tied in both direction so that liquefaction effects may cause settlement of one or more of them
- Buildings without shear walls: frames having poor redundancy because of minimum number of girders, leading to collapse if lateral resistance is lost at a girder edge
- Buildings skewed in plan, resulting in overstress at skewed connections

5.6 INVESTING IN RESEARCH FOR EFFECTIVE SEISMIC RESISTANCE

Increase in Research by US Organizations: The foundation of the three aspects of earthquake response—mitigation, risk assessment, and remediation—is knowledge based on solid research. Research in both in the field and in the laboratory is required to develop seismic engineering technologies. Technology transfer is a valuable byproduct of research. For example, in a country such as Pakistan, which suffered a devastating earthquake in 2005, a precast concrete industry might be established to manufacture prefabricated elements to be used for quick construction of replacement dwellings. Mass production of beams and columns leads to better quality control and compensates for the shortage of masons in many remote areas.

Research centers devoted to seismic engineering are up and running in the United States and around the world, manned by seismic experts whose job is to provide the science behind updated building codes, improved low-cost construction technology, new building design, and better building materials. The following are some examples:

- Asian Pacific Network of Centers for Earthquake Engineering Research (ANCER): center-to-center coordinated and cooperative research and outreach activities to reduce seismic hazards worldwide
- Beijing Institute of Technology: joint research on multi-hazard mitigation for urban and public infrastructure
- Chiba University, Japan (50): Joint research on remote-sensing technologies and international workshop

- Institute of Engineering Mechanics, China Earthquake Administration: publication of the *Earthquake Engineering and Engineering Vibration Journal*
- International Center for Disaster-Mitigation Engineering (INCEDE), University of Tokyo: Post-earthquake response and reconstruction strategies for urban regions
- National Center on Research for Earthquake Engineering (NCREE), National Taiwan University: steel plate shear walls, joint research, and reconnaissance (e.g., following the 1999 Chi Chi earthquake)
- State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University: joint research and workshops on special bridges
- Waseda University, Japan: Joint research and workshops on resistant design of lifeline facilities and countermeasures against soil liquefaction

In the United States, the National Science Foundation's Hazard Mitigation and Structural Engineering (HMSE) program supports research on new technologies for improving the and response of structural systems to earthquakes. Among its interests are the following:

- Analysis and model-based structure simulation
- Soil—structure interaction
- Design for structural performance and flexibility
- Large high-performance outdoor shake tables
- Control techniques for structural systems

NSF also supports the George E. Brown, Jr., Network for Earthquake Engineering Simulation (NEES), which investigates tsunami loss reduction and oversees and monitors experimental simulation techniques and instrumentation in a network of 15 experimental sites equipped with shake tables, geotechnical centrifuges, tsunami wave basins, and other field-based equipment.

The role of research programs includes the following:

- To provide the science and technology that supports seismic building codes, construction technologies, structural designs, and building materials
- To develop training requirements in seismic design and construction
- To partner with universities in teaching relevant courses at the graduate level
- To propagate new research through, for example, scientific and professional journals and conferences

The Benefits of Research: The Use of Electronic Skin to Detect Structural Faults

Civil engineers at the Massachusetts Institute of Technology working with physicists at the University of Potsdam recently proposed a new method for the electronic, continual monitoring of structures. The research group says a flexible skin-like fabric with electrical properties could be adhered to areas of structures where cracks are likely to appear, such as the underside of a bridge, and detect cracks when they occur.

The prototype of the sensing skin is made of soft stretchy thermoplastic elastomer mixed with titanium dioxide that is highly sensitive to cracks, with painted patches of black carbon that measure the change in the electrical charge of the skin.

Different types of rectangular patches of this "sensing skin" could be glued to the surface of a structure for detecting the type of crack likely to form in a particular part of a structure. A sensing skin formed of 3.25×3.25 in. diagonal square patches, for instance, could detect cracks caused by shear, the movement in different directions of stacked layers. Horizontal patches could detect the cracks caused when a horizontal beam sags.

The electronics behind the patch consists of a computer system attached to the sensing skin that would send a current once a day to measure the capacitance of each patch and detect any difference among neighboring patches. The formation of a crack would cause a tiny movement in the concrete under the patch, which would cause a change in the capacitance—the energy it is storing—of the sensing skin.

The computer would detect the flaw within 24 hours and know its exact location, a task that has proved difficult for other types of sensors proposed or already in use, which tend to rely on detecting global changes in the entire structure using a few strategically placed sensors, according to the researchers.

5.7 EDUCATION IN RISK ASSESSMENT, MITIGATION, AND REMEDIATION

Need for Continuing Education: Without highly trained professionals, there can be no research into improved earthquake risk assessment, mitigation, and remediation. Without the proper education, there can be no highly trained professionals to carry out this critical work.

5.7.1 Undergraduate and Graduate Education

The technology of seismic engineering has undergone a great deal of advancement in the past ten years. The concepts of structural behavior and design methodology and solutions have undergone major changes. Undergraduate and graduate programs in seismic engineering would address notable developments in theory and practice and familiarize students with new techniques of dynamic analysis, including, among others, SAP 2000, STAAD-Pro, and SEISAB. For example, as a subspeciality of a graduate structural engineering program, earthquake engineering would focus on

- Creating sustainable and earthquake-resistant communities by institutionalizing the appropriate principles and techniques
- Minimizing the effects of natural hazards through an integrated approach to planning
- Developing the seismic knowledge and skills required for pre- and post-disaster response
- Highlighting the role of the built environment by studying community development patterns
- Studying transportation and utility design and configuration, relationships between built and the natural environments, patterns of open space, housing and neighborhood design, and building group configuration and location
- Developing the ability to analyze the seismic vulnerability of existing structures, particularly nonengineered structures, and educate the public on modular design and construction
- Retrofitting and strengthening existing weaker buildings
- Developing and using the latest seismic software

Practical topics would include

- Seismic-resistant designs
- Structural codes for buildings and infrastructure
- Risk vulnerability and disaster preparedness in the built environment

Students would learn advanced concepts in applied mathematics, especially structural dynamics, which is one of the most difficult aspects of mechanics. Familiarity with differential equations, matrix methods of analysis, and finite elements modeling would be required, as would be the ability to solve nonlinear equations and perform eigenvalue solutions. Finally, students would acquire the ability to use new software for dynamic analysis.

A Sample Degree Syllabus:

- History of Earthquakes in Seismic Regions
- Plate Tectonics, Fault Maps, Wave Propagation, Seismographs
- Liquefaction, Site Effects, Soil Structure Interaction
- ASCE-7, IBC, AASHTO LRFD and AREMA Codes; Handbooks & Technical Journals
- Examples of Building and Bridge Failures, Economic Considerations in Design
- Deterministic and Nondeterministic Analysis, Degrees of Freedom; Equations of Motion, D'Alembert's Principle; Duhamel Integral for Damped Systems
- Rayleigh's Method, Response Spectra, Spectral Displacement, Velocity and Accelerations
- Site Coefficients and Liquefaction Potential of Soil
- Structural Planning for seismic-resistant design, shear walls, multi-rotational and seismic isolation bearings
- Concepts of Ductility, Performance-Based Seismic Engineering, Pushover Analysis
- Capacity Spectrum Design as applied to low rise buildings, buildings with in-filled frames, and bridges
- Computer Methods; SAP 2000, STAAD-Pro, SEISAB, OPENSEES and other software
- Code Applications; Case studies for seismic analysis of building frames, shear walls, and bridge substructures; Seismic Detailing
- Disaster Management: Vulnerable Populations, Risk Assessment, Loss Estimation (loss of life, property, and commerce is the root of this problem)
- Disaster Planning, Implementation, Mitigation, Recovery, Insurance

5.7.2 Professional Training

For professional engineers, designers, architects, construction managers, and the like, short training courses on seismic-resistant design construction techniques are critical. These might cover disaster vulnerability, risk assessment, strengthening and retrofitting of building systems and other infrastructure, basic principles of modular design, and ductility behavior in structures. An important adjunct to this course would be training of masons and other skilled craftsmen in seismic building techniques.

Another short course would be geared to environmentalists, anthropologists, and rural, town, and urban developers and planners, with a focus on the role and essential design features of the built environment in disaster. An important feature of such a course would be training in rapid damage assessment of existing structures for possible use with or without retrofitting.

5.8 CONCLUSIONS

This chapter discussed earthquake risk assessment, mitigation, and remediation. According to the U.S. Strategic Hazards Reduction Program established by Congress in 1977, these entail

- Short-term forecasting and early warning systems
- Improved building design for earthquake resilience
- Better techniques to retrofit old buildings
- Enhanced simulations and evaluation of critical infrastructure, including electricity, highways, and water systems

These are technological measures, but preparedness also requires public education, legislation, land use planning, and insurance programs—which are crucial as a way of seeing that updated seismic design codes and technological innovations are implemented.

There have been many advances in the science of seismology such as identification of new faults, seismic inventories, and early warning systems, as well as much progress in seismic engineering, such as base isolation, shape memory alloys, and shake-table analysis of structural components. All of these have contributed to the increasing sophistication of seismic design codes for new construction and retrofit of buildings and bridges. They have also contributed to systematic seismic evaluation of structures to determine earthquake vulnerability and mitigation measures, and rating systems for seismic retrofit. These rating systems often determine that it may be less expensive to replace a highly vulnerable structure than to retrofit it.

Investment in pre-event mitigation makes a great deal of engineering sense. However, the work of preparedness cannot continue without ongoing research, and much is being done in universities and earthquake engineering centers here and abroad. This research will be aided by advanced education and training of scientists,

engineers, construction professionals, craftsmen, public officials, and others—anyone who is involved in making communities safe from the devastation that earthquakes have so often proven themselves capable of.

There is a need for continuing research in seismology and seismic engineering. This research will be supported through specialized education and specialized training.

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CHAPTER SIX

Tsunamis, Earthquakes, and Nuclear Power

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6.1 INTRODUCTION

This chapter broadly reviews the performance of nuclear power plants hit by earthquakes and powerful tsunamis generated by less severe earthquakes.

2011 Japanese disaster resulting from tsunami has shown that the billions of dollars of damage to the power plant is peanuts compared to the enormous loss of life and sickness from long term radiation exposure of the dense city population.

Power plants location close to sea makes them extremely vulnerable to malfunction when hit by a tsunami. Leakage in piping system and melt down of steel lining of dry well of reactor is always an uncontrollable hazard even due to accidents other than a tsunami.

Hence it is important for design and maintenance engineers to know the vulnerability of the types of power plants and their operating systems.

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This chapter gives an insight into the elementary description of nuclear power plants and how tsunamis are generated and likely to damage a reactor.

Present day reactors and its components are designed for a high magnitude earthquake but do not have any defense against high velocity waves of a tsunami. This subject has been neglected so far but the Japan's power plants inability to cope with the onslaught of tsunami has opened the eyes of nuclear engineers responsible for maintenance. Both the heating and cooling systems can break down.

A detailed literature review shows contributions by the following organizations and design codes:

Engineering Research Institute (EERI) (2012)

(EERI) (2001)

Institute of Human Settlement and Environment (2005)

The following authors have contributed in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to the study of latest earthquakes is presented here:

Goldman, J. G. (2011), Impact of the Japan Earthquake and Tsunami on Animals and Environment

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In addition to above publications, relevant individual references are discussed in the text.

In the fifteenth century, a giant wave is said to have swept away a hilltop hall housing the Daibutsu, a huge bronze Buddha, in Kamakura, a town south of Tokyo. An offshore quake in 1707 is said to have caused a tsunami that hit the island of Shikoku, leaving several thousand people dead. The tsunami that followed the Great Alaskan earthquake of 1964 caused 143 deaths; southeast of Anchorage, areas near Portage dropped as much as 8 feet, requiring reconstruction and fill to raise the Seward Highway above the new high tide mark.

These are a few examples of the power of a tsunami, but in none of them was there a real risk of radiation exposure as there was in the aftermath of the Tohoko tsunami in 2011. The effects of tsunamis and earthquakes on nuclear power plants is the subject of this chapter.

6.1.1 What Is a Tsunami?

Description: A tsunami is a set of ocean waves instigated by disturbances of the sea surface, often triggered by submarine earthquakes. It is not one giant wave, but a succession of waves. After one has passed, many more may follow. Tsunami waves travel through the ocean at speeds up to 400 to 500 miles per hour and follow each other at intervals of 5 to 60 minutes in a so-called "wave train" reaching up to one hundred feet in height. (See Figure 6.1.)

Tsunami waves are not like normal sea waves, but may initially resemble a rapidly rising tide, which is why they are often referred to as *tidal waves*. The velocity and dimensions of the waves change as they begin to drag the bottom and slow down. Height increases as speed decreases. Velocity, momentum, and run-up elevation vary depending on topography, and the water's retreat may be more violent than the initial assault. The waves break as they near shore, sending a turbulent bore racing 10 meters per second or faster across land.

Causes of Tsunami Generation: Tsunamis occur at boundaries between tectonic plates, which slowly move in relation to each other, causing displacement of water. Other causes are submarine landslides, which result from underwater volcanoes, creating debris that triggers water movement and eventually tsunamis. Though very rare, a tsunami can occur when a meteorite striking water causes water displacement.

Figure 6.1 Tsunami whirlpool effects in coastal areas.

Although the impact of tsunamis is limited to coastal areas, their destructive power can be enormous and they can affect entire ocean basins. In fact, their impacts are far greater than those of wind or earthquakes. On December 26, 2004, an earthquake with an epicenter off the west coast of Sumatra generated a series of tsunamis that devastated coastlines in Indonesia, Thailand, India, and Sri Lanka (Figure 6.2), inundating surrounding communities with huge waves up to 100 feet high. It was among the deadliest natural disasters in human history, with over 230,000 people killed in 14 countries bordering the Indian Ocean.

Depending on their height, speed, and strength, tsunamis can cause massive property loss, untold casualties, and great damage to the environment. The effect of tsunami forces is different from that of earthquakes on structures, with failures resulting from hydrostatic and hydrodynamic forces, liquefaction, and foundation scouring.

There have been tsunamis in most oceans of the world, but most notably in the Pacific Ocean. It is typical for the Pacific to experience one locally damaging tsunami every year or two and one ocean-wide destructive tsunami perhaps twice in a century. In the United States, most tsunamis occur in California, Alaska, and Hawaii. Worldwide, Japan—known to have the highest frequency of tsunamis and to have experienced several devastating disasters over the years—is widely acknowledged

(a)

(b)

Figure 6.2 (a) Locations of recurring Indian Ocean tsunamis; (b) devastation caused by tsunami waves after the 2004 Indian Ocean earthquake. *(Source: photo courtesy of USGS.)*

to be one of the most earthquake-prepared nations. It has had to be because of its perch atop several converging tectonic plates. For all its safeguards, the risks posed by tsunamis are severe, as the massive Tohoku event of 2011 has shown.

Japanese Design and Construction Codes for Buildings

A Review of Japanese Code: Most of the buildings built over the last 30 years in Japan are subject to very tight seismic regulations. They are designed for quite high lateral forces and have additional features such as damping devices. Some are constructed on springs, which decouple the superstructure from ground vibrations.

Tokyo felt the devastating power of the 1995 Kobe earthquake despite being about 380 kilometers away from its epicenter. Skyscrapers swayed, some buildings caught fire, and many commuters were left stranded in various locations across the city. Post mid-eighties buildings in Tokyo performed well, whereas those built in the sixties and seventies fared badly.

Hidden inside the skeletons of Japan's high-rise towers are extra steel bracing, giant rubber pads, and embedded hydraulic shock absorbers. Such seismic construction technology makes modern Japanese buildings among the sturdiest in the world. Instead of vibrating in an earthquake, they sway, absorbing and dissipating the destructive seismic forces. Stringent regulations like these, considerably tightened in the 1980s, may render new regulations for quake-proofing unnecessary in the aftermath of Tohoku.

As a precaution, if an earthquake above a certain magnitude strikes, the Japanese bullet trains stop and nuclear and other power plants automatically go into temporary shutdown.

6.2 THE TOHOKU TSUNAMI

Japanese Disaster of 2011: The Tohoku tsunami of March 2011 in northeast Japan resulted from a magnitude 8 earthquake that occurred about 230 miles northeast of Tokyo at a depth of about 17 miles. It produced waves exceeding 60 feet in height that inundated 200 square miles of land. Nearly 25,000 people were killed, many swept away with the receding water. Authorities evacuated 210,000 people from the area. The city of Sendai was utterly destroyed.

The Japanese government estimated that 130,000 low and mid-rise light-framed residential buildings fully or partially collapsed as a result of surges, debris impact, and debris moving at high velocity. In some areas

soil erosion was considerable and liquefaction occurred, causing foundations to settle. Property losses were valued at over $310 billion—the most expensive damage in recorded history.

Aside from the destruction of underground utilities and treatment plants, otherwise functioning systems were rendered inert because there was no electricity to run pumps. A week after the event, millions of people still lacked drinking water as relief efforts were hampered by fuel and water supply shortages, the ongoing nuclear crisis, mangled roads, and extraordinarily cold weather.

6.2.1 Fukushima Daiichi

Description of Nuclear Power Plants: At the time of the tsunami, three of the Fukushima Daiichi Nuclear Power Plant's reactors (see Figure 6.3) were offline for maintenance. When the earthquake struck, reactors 1, 2, and 3 were shut down and emergency generators were deployed to keep electronic equipment going and to maintain the crucial cooling systems for the spent fuel rods. The tsunami then hit the plant, flooding the lower floors where the generators were located and putting them out of commission. Without electricity, the cooling systems could not operate and the cooling water began to boil away. The fuel rods quickly began to heat up and melt down. Reactors 1, 2, and 3 went into full meltdown, causing hydrogen explosions, one of which blew the roof off of Unit 1. In fear of radiation exposure, evacuations were carried out within a 12-mile radius of the installation.

Japanese authorities classified the event at Fukushima Daiichi's Unit 1 reactor as a level 4 "accident with local consequences." However, observers thought that number was too low. The level was later raised to 5 and eventually to 7 (major accident), the maximum scale value. Detectors

Figure 6.3 Location of nuclear power plants at Fukushima Daiichi.

showed 11,900 microsieverts of radiation—The SI derived unit used to measure the amount of radiation necessary to produce the same effect on living tissue as one gray of high-penetration x-rays. The sievert is named after Swedish physicist Rolf Sievert (1896–1966)—(the limit is 100,000) three hours after the blast, up from just 73 beforehand. There were no immediate deaths due to direct radiation exposure, but at least six workers tested as having exceeded lifetime legal limits and more than 300 tested as having received significant doses.

Adverse Effects of Exposure to Radiation

Health Issues: Exposure to high levels of radiation can result in radiation sickness, which produces a range of symptoms. Exposure to radiation dose of 4 gray will typically kill about half of all healthy adults. For comparison, radiation therapy for cancer typically involves several doses of between 1 and 7 gray at a time.

Description of Reactor 1: The Unit 1 reactor had been scheduled for decommissioning because of age, but Tokyo Electric had been given permission to keep it going for another 10 years. When the power station was built in the late 1960s, its designers failed to anticipate that a tsunami would disable the backup systems that were supposed to stabilize the reactor. A seawater reservoir at a higher elevation than a tsunami can reach could have been the source of cooling water, but no such reservoir was built. It had extended the life of Unit 1 despite warnings against doing so.

6.2.2 ASCE's Post-Tsunami Investigation

ASCE Offers Technical Aid: As it did after earthquakes in Pakistan, Chile, Haiti, and New Zealand in the past decade, the American Society of Civil Engineers sent an investigative team to Tohoku to aid the Japanese government in its damage assessments. The team took photographs, measurements, and rebar and concrete samples for analysis to validate water height, velocity, momentum over land, and impact-force of water and debris. According to team member Ian Robertson of the University of Hawaii at Manoa, research and fundamental knowledge in seismic activity is crucial: "There is a tendency to just do what's in the code, but the code has been idling. It's time to bring it up to speed." The destruction wreaked by Tohoku is 10 times the losses in Chile, which totaled $30 to $40 billion.

The ASCE delegation consisted of seven teams, each assigned to focus on particular areas:

- Structural engineering
- Coastal engineering
- Ports and harbors
- Critical infrastructure
- Dams and embankments
- Industrial structures
- Nuclear structures

The team's terrestrial LiDAR expert analyzed the tsunami's loads and the performance of structures and coastal engineering. Altogether, the team collected an estimated 4 billion data points, 100 gigabites of data, in 10 days.

Methodology

The team analyzed video and satellite imagery of the damage area, then searched for specific structures and reference points to validate estimates of wave height, velocity, land momentum, and water and debris impact. It observed that in Natori, just south of Sendai, a broad coastal plain allowed the bore to smash through many kilometers of coastal communities.

Targets of Investigations and Observations

Several concrete structures that were apparently damaged but remained standing were observed. The team visited those sites to collect concrete and rebar samples, measuring deformations so that they could be accurately modeled and loads that caused that level of damage calculated. According to a team member, "There will be a lot of hypotheses on the ground. But if we take enough hard data, we can test those hypotheses."

At the container port of Sendai, one goal was to examine strike damage to structures from wave-borne shipping containers.

Targets of opportunity were found as well. They consisted of any structure that could be measured and modeled that had been deformed but not destroyed by the rush of water or the impact of identifiable debris. A cargo truck stuffed into the bent frame of a steel building or a line of metal flag poles folded down against the ground by the press of rushing water became objects for analysis.

Debris

Validating formulas for calculating impact loads of water-borne debris was of particular interest. Debris loads are a source of tremendous damage but are difficult to test. Calculations also are complicated by the damming effect of the striking objects. The force of the water driving forward adds to the momentum of the debris. Some of the damage is expected to provide clear examples of the pure force of the water pressure and velocity by considering not only onrushing waves, but the force of the returning seas as well.

Japanese and American forensic engineers combed the debris and data from the March 11, 2011 earthquake and tsunami to study how the wounded and surviving structures differ from those that were destroyed. The ASCE tsunami loads and effects subcommittee is preparing to publish approximately 350 pages filled with engineering analysis and case studies from the debris.

Comparisons of Fukushima Daiichi with Chernobyl and Three Mile Island

Japanese Reactors Subjected to Major Damage: Japan has a total of 55 reactors spread across 17 complexes nationwide. The GE-designed reactors in Fukushima have 23 sisters in the United States. The reactor that exploded at Chernobyl (Sawyer 2011) sent a cloud of radiation over much of Europe, killing 4,000 people either directly or from cancer. It was not housed in a sealed container as were those at Daiichi. Also, Japanese reactors do not use graphite, which burned for several days at Chernobyl.

The INES scale is used to consistently rate the safety significance of events associated with sources of radiation. It runs from 0 (no safety significance) to 7 (major accident).

The event at Fukushima's Unit 1 reactor was rated initially as a level 4 "accident with local consequences," but was raised to a level 7 later. The 1979 Three Mile Island accident was rated a level 5 "accident with wider consequences." The 1986 Chernobyl disaster was rated a level 7.

The Fukushima Daiichi reactors are similar to those at Three Mile Island. The cores are probably also similar—partially melted. Engineers pointed out several key differences in the aftermath of the accidents, however. In Japan, four separate reactors were damaged, and fixing each one is complicated by the presence of its leaking neighbors. Repair also requires a major infusion of equipment to replace parts far from the reactor's core, such as pumps and switchgear that were destroyed by the tsunami.

Comparison of the Tohoku Tsunami with the Kobe Earthquake of 1995

Comparison of Tsunami Effects With Major Japanese Earthquake: In 1995 a magnitude 7.3 earthquake devastated the Japanese port city of Kobe in western Japan. Some 6,400 people were killed and more than 400,000 were injured. Highways were toppled and thousands of buildings were damaged. Fires blazed across the city. (See Figure 6.4.)

Cost estimates of the damage caused by the Tohoku tsunami exceed those of Kobe's damage. The Great Hanshin earthquake, as it is called, is still listed in the *Guinness Book of Records* as the costliest natural disaster to befall any country, causing $102.5 billion in damage, or 2.5% of Japan's GDP.

In Kobe, no building was unaffected and most were full of rubble. The city had never recovered its position as Japan's premier shipping port and one of the world's busiest, but the economic impact of the quake was limited and short-lived. Manufacturing was virtually back to normal within 15 months.

Japan's recovery from Kobe was much speedier than Haiti's has been from its 7.0 magnitude quake just over a year ago. Japan's building regulations have been significantly tightened since 1995.

Already there are dire predictions for the global nuclear industry, with both China and India already announcing that their nuclear power plants will be reviewed and protests in Europe starting up against plans to build reactors in Germany.

Figure 6.4 Damage to a road in Satte, in Saitama Prefecture, from the Kobe earthquake.

A huge earthquake like the one that caused the 2004 Indian Ocean tsunamis can shake the entire planet. According to estimates, this quake had the power of 475 million tons of TNT. The amount of energy released depends on how long the strain has been building up and on the geometry and composition of the plates involved. Depending on conditions, speed, geographic reach, and intensity can vary from nearly undetectable at a short distance to catastrophic for hundreds of kilometers.

6.3 A PRIMER ON NUCLEAR POWER—ADVANTAGES AND DISADVANTAGES

Advantages: There are great advantages to nuclear power for generation of electricity at reasonable cost. According to physicists, one kilogram of uranium-235 (U-235) converted via nuclear processes releases approximately 3 million times more energy than a kilogram of coal burned conventionally (7.2×10^{13} joules per kilogram of uranium-235 versus 2.4×10^{7} joules per kilogram of coal). Fifty-nine nuclear plants are under construction around the world, with 23 in China alone, according to the Nuclear Energy Institute.

The United States and the rest of the world attempt to reduce pollution from power generation, particularly coal plants, and the future of the nuclear industry is being debated because of the dangers of radiation. Nuclear power produces no greenhouse emissions during operation. In the United States, more than 20 companies have announced plans to submit applications for new nuclear power plants, although only one is now under construction. Nuclear, wind, and solar energy are more expensive than electricity from natural gas at its current prices, which is also less polluting than coal. But future increases in demand for power will drive up the prices of gas and coal and the nuclear industry will then have an edge.

The author is familiar with the planning and design of boiled water reactors (BWR) and pressure water reactors (PWR) which are more common in USA. having spent many years as engineer in their seismic analysis, structural analysis, and mechanical analysis and in the power plant construction. These projects included ARS, finite element software for seismic analysis and design, retrofit, and analysis of large-diameter piping and pipe supports.

A nuclear reactor is a device to initiate and control a sustained nuclear chain reaction. Most commonly it is used for generating electricity. Usually heat from nuclear fission is passed to a working fluid (water or gas), which runs through turbines that power generators. Just as conventional power stations generate electricity by harnessing the thermal energy released from burning fossil fuels, nuclear reactors convert the thermal energy released from nuclear fission.

The reactor core generates heat in a number of ways:

- The kinetic energy of fission products is converted to thermal energy when these nuclei collide with nearby atoms.
- Some of the gamma rays produced during fission are absorbed by the reactor, their energy being converted to heat.
- Heat is produced by the radioactive decay of fission products and materials that have been activated by neutron absorption. This decay heat source will remain for some time even after the reactor is shut down.

6.3.1 Electrical Power Generation by Superheated Steam

The energy released in the fission process generates heat, some of which can be converted into usable energy. A common method of harnessing this thermal energy is to use it to boil water to produce pressurized steam, which then drives a steam turbine that generates electricity.

Cooling: a Sensitive but Essential Requirement in Daily Power Plant Operation

A nuclear reactor coolant is usually water but can be a gas, a liquid metal, or molten salt. The coolant is circulated past the reactor core to absorb the heat that it generates. Heat is carried away from the reactor and is used to generate steam. Most reactor systems employ a cooling system that is physically separated from the water that will be boiled to produce pressurized steam for turbines. In some reactors the water for the steam turbines is boiled directly by the reactor core, for example, the boiling water reactor.

Classification by Coolant

In thermal nuclear reactors (LWRs), the coolant acts as a moderator that must slow down the neutrons before they can be efficiently absorbed by the fuel.

Of the 104 operating reactors in the United States, 69 are pressurized water reactors (PWRs). A primary characteristic of PWRs is a

pressurizer—a specialized pressure vessel. Pressurized heavy-water reactors are a subset of pressurized water reactors, sharing the use of a pressurized, isolated heat transport loop, but using heavy water as coolant and moderator for the greater neutron economies it offers.

BWRs are characterized by boiling water around the fuel rods in the lower portion of a primary reactor pressure vessel. A BWR uses U-235, enriched as uranium dioxide, as its fuel. The fuel is assembled into rods that are submerged in water and housed in a steel vessel. The nuclear fission causes the water to boil, generating steam. This steam flows through pipes into turbines. The turbines are driven by the steam to generates electricity. During normal operation, pressure is controlled by the amount of steam flowing from the reactor pressure vessel to the turbine.

The Dangers of Stored Spent Fuel a Real Health Concern

Radiation from Spent Fuel: At the Fukushima Daiichi plant in Japan, the spent fuel pools have emerged as one of the gravest dangers because they are not stored in metal and thick concrete structures as the reactors are. After it has been used to generate heat in the reactor, fuel is kept in pools of water that contain the radiation. Water is also circulated to remove the residual heat from the assemblies, which hold the nuclear fuel rods, for several years.

The U.S. utility industry and Nuclear Regulatory Commission are re-evaluating the notion of a centralized interim storage operation. In the United States, there are less than 100 tons of spent fuel stored at power plants, according to the Union of Concerned Scientists (UCS).

Because there isn't a central repository in the United States for storing spent nuclear fuel, many nuclear plants have a lot of spent nuclear fuel on site. Space for 20 years' worth of spent fuel storage in pools is generally provided but then space is needed to build a dry cask storage facility on site. The nuclear waste is covered in steel and concrete.

A UCS study noticed that storing fuel in dry casks, rather than in pools, is safer from errors, natural disasters, and attacks.

An MIT report recommends that existing spent fuel stored on site be brought to centralized sites and stored in seismic-resistant concrete dry casks suitable for 100 years of storage. For long-term storage of thousands of years, geological formations are suitable for safe storage.

One of the conclusions from the spent-fuel study is that light-water reactors will remain in place for decades unless there is a sharp increase in the use of nuclear power. Technology transitions in the nuclear industry take decades

and there is not a strong economic incentive to move away from light-water reactors because of the supply or cost of uranium according to the Nuclear Regulatory Commission. Interim storage can be done so that countries have the flexibility to retrieve spent fuel from light-water reactors for use as a fuel in so-called fast reactors.

Comparison of Nuclear Plants to Offshore Oil Platforms

Special Structures for Drilling Oil with Nuclear Power Generation: There is some similarity between offshore platforms hit by tidal waves and nuclear power plants hit by tsunamis. The author has been involved in structural analysis and design for fixed and floating offshore platforms. Two examples are the fixed-leg Maureen Hi-Deck Platform and Conoco's floating Tension-Leg Platform, both of which are located in the North Sea (north of Scotland). API and DNV structural codes are applicable, unlike NRC codes for nuclear power plants.

One notable difference is that, for offshore structures, there is adequate warning from the weather bureau of the magnitude and direction of any impending wind storms generating high tidal waves. Advance warning allows personnel to evacuate prior to a storm. In the case of a tsunami, a power plant can be hit with no warning at all. There is the added danger of nuclear meltdown and leakage of high dosages of radiation, affecting marine life and populations hundreds of miles away.

The design of offshore platforms allows for pontoon effects and flexibility of movements of the supporting legs, which are anchored to the sea bed and thus minimize the impact of wave forces. A containment structure in power plants, on the other hand, is rigidly supported on drilled shafts, piles, or firm ground.

6.4 NUCLEAR REACTORS AND TSUNAMIS IN THE UNITED STATES

In the United States, as around the world, nuclear plants may be close to the epicenter of a quake or far from it. Seismic design of these installations is always a problem given the need to maintain sensitive coolant systems, pipe connections, and supports carrying high-pressure steam. In the absence of tsunamis, power plants have generally done well in quakes. However, they may be located at or near a harbor for operational demands of water for cooling, which puts them at greater tsunami risk.

Major U.S. organizations dealing with tsunamis include the National Oceanic and Atmospheric Administration (NOAA), which leads the National Tsunami Hazard Mitigation Program (NTHMP), a federally funded and state-funded program established to help minimize casualties and property loss. The Federal Emergency Management Agency (FEMA) assists the NTHMP in multi-state programs. It also conducts training in planning, public awareness, warning, and evacuation. NOAA, NTHMP, and FEMA join the International Tsunami Information Center in tsunami investigations.

6.4.1 Code Provisions and Need for New Tsunamis Design Code

A penny spent in safety against tsunami strike is worth many times in the life insurance. Not all nuclear reactors are likely to be hit by a tsunami due to their elevated locations. FEMA P646 design approaches can be applied to investigate tsunami causes and effects. However, the provisions developed by IBC and ASCE 7 do not fully address these issues—a situation that should be rectified. Generally, tsunami seismic design should consider the following:

- Regions of high risk in the United States
- Demands generated onshore
- Hydrostatic, hydrodynamic, and uplift forces
- Impact of debris

Analysis and design of nuclear power plants are addressed in detail in the relevant structural seismic and NRC codes (Cuypers 2004, Caltrans 1994). Dynamic methods and finite element software have been developed for analysis. Accelerations and seismic forces are generated based on seismic spectra generated by the response of each structure. Power plant foundations, substructures, and superstructures, including mechanical equipment and large-diameter piping, have been successfully designed to withstand large-magnitude seismic events. Also, because care has been taken in locating power plants away from active faults, no significant damage has resulted from the recurring earthquakes of small and medium magnitudes to which some of these plants have been subjected.

New Design Philosophy for Power Plants

Current procedures preclude the inclusion of any tsunami effects in the load combinations for design of individual components. In light of the Indian

Ocean tsunami of 2004 and the 2011 tsunami in Japan it will be prudent to review the design philosophy for all coastal structures, including power plants, which are subjected simultaneously to tremors and resulting water waves.

6.4.2 Probabilistic Risk Assessment

Risk Analysis: In the United States, as part of its mission to ensure the safety of nuclear power, the Nuclear Regulatory Commission's (2010) goal for radiation leads resulting from a seismic events (NOAA 2012) is that the public's risk should not exceed one-thousandth of the risk of accidental death from all sources; risk of fatal cancer likewise should not exceed that amount. The commission has ruled that reactors will meet that standard if there are meltdowns with offsite consequences—only once in 100,000 years of operation. With 104 American reactors now running, that means that such an event would occur about every 1,000 years. And there is at present no effective defense mechanism in place.

Using a well-known mathematical method, a risk analysis looks at the chance of failure for any piece of equipment, or at the chance of other failures leading to the component failure numbers. These numbers entail tremendous uncertainty: They might be off by a factor of 10 for radiation leakages resulting from conventional seismic failures alone. Moreover, the historical record is not sufficiently long to determine the size of the worst natural hazard. Because the Richter scale is only about 75 years old, experts have studied old newspaper accounts, if available, for earlier records.

A sophisticated seismic risk analysis method for nuclear power plants is currently available. Its use is recommended for all power plants located near coastlines—their vulnerability to an earthquake or tsunami during their projected remaining life must be evaluated (Sawyer 2011). As the Fukushima Daiichi disaster has shown, the damage from a tsunami is many times greater than that caused by environmental disasters. The cost of remediation for Tohoku could run into nearly half a trillion dollars.

6.5 THE NUCLEAR REGULATORY COMMISSION'S RESPONSE TO FUKUSHIMA DAIICHI

Lessons Learnt for Safety of U. S. Power Plants: As a result of the Fukushima disaster, the U.S. Nuclear Regulatory Commission is evaluating

the probability of damage to nuclear plants from tsunamis. This evaluation will be aided by the ASCE Tsunami Reconnaissance Team's detailed report of findings from its Fukushima Daiichi investigation. It may be too late for any structural retrofits and repairs in anticipation of a possible tsunami. Still, certain precautions can be considered and implemented for long-term safety. Moreover, according to industry and government officials and academic experts, American reactors may already be better protected than those in Japan because of precautions against terrorist attacks implemented after September 11, 2001. For example, American nuclear plant operators are required to have diesel fuel and pumps on site, or readily available nearby, to provide backup power and cooling capacity.

Immediately after the Fukushima crisis, American plants were ordered to check that they had the required disaster preparedness equipment on hand and that it was in working order. The details are classified, but it is known that the industry has emergency supplies of pumps, hoses, and generators, and that the U.S. Air Force will be deployed in an emergency to move equipment.

6.5.1 Tsunami Risk Maps

An Important Step: Using new USGS seismic risk maps based on 2008 data (Caltrans 1971), the NRC in August 2010 published new estimates of risk at nuclear power reactors in eastern and central states. Besides the proximity, severity, and frequency of earthquakes, the new estimates take into account design standards used at each plant along with the type of rock or soil on which the plant is built.

Geologists and seismologists, remembering what they learned about rocks, are steadily raising their estimates of the risk of severe quakes based on these new maps. New faults are found, and new computer models change predictions for how the ground shakes. Of special note, according to the USGS, is an allowance for large waves in the New Madrid, Missouri, fault area, roughly centered on the state's Bootheel, as well as inclusion of offshore faults near Charleston, South Carolina, and new data from the mountains of east Tennessee. With each new map, the areas of only negligible risks have grown smaller.

Interestingly, the reactor with the highest risk rating is 24 miles north of New York City, in the village of Buchanan, New York, at the Indian Point Energy Center, which generates up to one-third of the electricity for New York City. There, on the east bank of the Hudson, Indian Point

nuclear reactor 3 has the highest risk of earthquake damage in the country, according to new NRC risk estimates. Indian Point 2 does not rate as risky, with 1 chance in 30,303 each year.

A ranking of the 104 U.S. nuclear reactors was prepared by the NRC based on estimates of risk of catastrophic failure caused by earthquakes. The chance of serious damage from a quake ranges from Indian Point (1 chance in 10,000 each year) to the Callaway nuclear plant in Fulton, Missouri, which the NRC ranked as the least risky (1 chance in 500,000 each year). Installations in Massachusetts and Pennsylvania rank second and third, respectively, followed by those in Tennessee, another plant in Pennsylvania, then plants in Florida, Virginia, and South Carolina. At sixth and seventh riskiest are, respectively, California's Diablo Canyon and Pennsylvania's Three Mile Island.

The risk calculations take into consideration two main factors: the chance of a serious quake, and the design of a plant. Surprisingly, nuclear plants built in earthquake zones, such as the California coastline, have a lower risk of damage since they were built in anticipation of a major quake and sufficient safeguards were provided. Power plants in the East, South, and Midwest, where seismic design standards are considered lower, now find themselves at the top of the NRC's danger list because of upgraded acceleration coefficients and seismic zone reevaluation.

6.6 CALIFORNIA'S SEISMICITY AND NUCLEAR POWER

West Coast Issues: The colossal California quake considered inevitable and long overdue is most likely to strike along the southern end of the famed San Andreas Fault and register a magnitude of 7.4 or greater, many times less powerful than the 9.0 temblor that rocked Japan. Nevertheless this quake, when it comes, will put California's nuclear power plants at considerable risk.

In March 2011, California announced plans for new research on tsunami dangers that will result in statewide planning maps of tsunami risk to help coastal communities better prepare for dangerous waves. In 2009 the state released tsunami inundation maps for events likely to occur every 1,000 years, such as tsunamis spurred by undersea landslides. The new maps employ scenarios of 500 or fewer years. They indicate a worst-case scenario in which potential flooding along the coast could reach as far as two miles inland in Huntington Beach and Newport Beach.

The worst-case inundation maps are intended to prepare for evacuations. The planning maps can be used by communities for siting schools, hospitals, and public safety facilities. The maps also show where hazard zones lie within harbors and ports, and safe areas offshore where boats can be taken when a tsunami is imminent.

Evidence of violent breaks in the feature known as the Cascadia Subduction Zone, or CSZ, has been found in sediment layers left by prehistoric tsunamis. In April 1992, a 7.2 magnitude earthquake with an epicenter at the south end of the CSZ in northern California produced a modest tsunami that struck the shore. That event confirmed the zone's potential to produce not only strong earthquakes but locally sourced tsunamis that could be ashore within 10 minutes.

The Northwest is much less prepared for tsunamis than is California, and California is much less prepared than is Japan. But Oregon's geography resembles that of northeast Japan, with many isolated coastal communities separated by a mountain range from a valley with highways. Like Japan, Oregon faces an undersea subduction zone in which an oceanic plate pushes beneath a continental plate.

6.7 EARLY-WARNING SYSTEMS

Quest for Precautions: Bruce Parker (2012), in his book *The Power of the Sea*, emphasizes the need for prediction of tsunamis, storm surges, and rogue waves. He says, "When the sea turns its enormous power against us, our best defense is to get out of its way. But to do that we must first be able to *predict* when the sea will strike." He goes on:

> *Over 300,000 lives would not have been lost if the Indian Ocean tsunami on December 26, 2004, could have been predicted. Millions would not have been killed by the storm surges that have ravaged the coasts of India and Bangladesh over the centuries if they could have been predicted. Thousands of ships would not have disappeared at sea if we could have predicted when and where 100-foot rogue waves would suddenly appear.*

Sophisticated international warning systems can detect impending tsunamis with delicate sensors. How helpful they are depends on where the tsunami strikes—if the strike is close by, an early warning cannot be early enough. Because the Tohoku tsunami was very close to the coast, authorities could provide only between 5 and 10 minutes of warning before

impact. For earthquakes, even a 10-second warning can be vital, according to a seismologist Roger Musson at the British Geological Survey. It is enough time to turn off a gas stove to prevent your house burning down or to take cover.

The Japanese government has invested heavily in such systems. Founded in 1952, the Tsunami Warning Service is operated by the Japan Meteorological Agency (JMA) and monitors activity from six regional centers, assessing information sent by seismic stations both on- and offshore known collectively as the Earthquake and Tsunami Observation System. The Japanese network of sensors, launched as the world's first in 2007, records P waves and relays that information to computers that calculate the size of the quake. Unfortunately, many of the sensors in Japan's network were quickly knocked out by the Tohoku quake and tsunami, and, because of the intensity and number of shocks, the sensors that worked provided an overwhelming amount of information that was difficult to interpret.

There is only one Pacific Tsunami Warning System (PTWS) on the West Coast of USA. In operation since 1968, it has two centers, one in Hawaii and other in Alaska. PTWS helps to detect, locate, and determine the magnitude of potentially "tsunami-genic" earthquakes occurring in the Pacific so that communities can receive at least some notice of the predicted tsunami's arrival and its wave height. PTWS has given warnings for all five of the major tsunamis over the last 50 years and has falsely predicted 15 others.

Taiwan has more than 100 quake sensors, making it one of the world's best-equipped countries for earthquake monitoring. The country is regularly hit by earthquakes, as it lies near the junction of two tectonic plates. Taiwan's seismology center began building its first undersea earthquake sensor in a project designed to provide earlier warnings of the quakes and tsunamis that frequently hit the region. The goal is a few seconds or even minutes of extra time ahead of an earthquake and an extra 10 minutes of warning for tsunamis.

In Pakistan, which suffered major loss of life and damage in the earthquake of 2005, the Karachi Nuclear Power Plant is located on the coast and is therefore vulnerable to tsunamis. The World Conference on Disaster Reduction (WCDR) and the United Nations have recommended a tsunami warning system, the installation of which would require adequate electric power, radio/television links, transport, and

Table 6.1 Earthquakes Originating in the North Arabian Sea, Sind, and Baluchistan

Date	Magnitude	Intensity at Epicenter	Remarks
September 1930	6.0	8	Felt at Karachi
August 1931	7.4	10	
April 1932	6.0	8	
July 1933	6.0	8	
May 1935	6.2	8	
February 1943	6.3	9	Felt at Karachi
November 1945	8.0	11	Felt at Karachi
February 1953	6.0	7	Felt at Karachi
July 1956	7.0	9	Felt at Karachi
January 1972	5.9	7	Felt at Karachi
October 1974	5.7	8	Felt at Karachi
December 1985	5.4	7	Felt at Karachi
January 1992	5.5	7	

Table 6.2 Distances from Karachi Coastal Region and Warning Time

Earthquake Region	Distance of Epicenter from Karachi (km)	Tsunami Arrival Time (min)
Murray Ridge	100–200	25–45
Runn of Kacch	200–300 km	45–60
Makran Coast	300–400 km	60–90

Note: Peak speed of waves assumed 250 km/hr.

communication. This warning system would reach out to 18 million people living in the Karachi coastal belt (Tables 6.1 and 6.2).

6.8 U.S. NUCLEAR SITES: PREPARING FOR THE UNLIKELY

U.S. Government Initiative: In 1995, Congress directed the National Oceanic and Atmospheric Administration (NOAA) to form a federal and state working group called the National Tsunami Hazard Mitigation Program (National Public Radio 2011). Many of its members are officials in Alaska, Hawaii, and the Pacific Northwest, where tsunami risk is the highest. As mentioned, a magnitude 7.4 or higher California

quake is sure to come, and the threats it poses that are considered most serious by nuclear engineers are loss of power to run cooling systems for the reactor core and the spent fuel rods. Loss of power was what led to the explosions and release of radiation at Fukushima Daiichi.

Description: American nuclear facilities have backup power systems and backups to their backups. All plants are required to have batteries to provide power in the event of a loss of electricity and failure of backup generators. In the United States, 93 of the 104 operating reactors have batteries capable of providing power for four hours; the other 11 have eight-hour batteries. Fukushima had eight-hour batteries, which weren't enough.

Past History: 2011 Virginia Earthquake—The 2011 magnitude 5.8 Virginia earthquake created a state of emergency at the North Anna Nuclear Power Station in central Virginia, which lost electricity and automatically shut down, although generators restored power. The four diesel generators at the plant, 40 miles northwest of Richmond, were able to furnish enough electricity to run vital safety systems indefinitely at the two-reactor site, especially the coolant systems for the reactor cores and the spent fuel rods. Twelve additional nuclear plants in Virginia, Maryland, Pennsylvania, New Jersey, North Carolina, and Michigan declared "unusual events," resulting from this quake, the lowest of four emergency situations. They included Calvert Cliffs, about 50 miles from Washington, D.C., which remained at 100 percent capacity throughout the event, according to its owner, Constellation Diablo Canyon.

At the Diablo Canyon Nuclear Power Plant in San Luis Obispo County, California, nuclear safety regulators (according to an NRC study published in August 2010), using a complex mathematical technique, determined that the simultaneous failure of both emergency shutdown systems designed to prevent a core meltdown was so unlikely as to happen once every 17,000 years. However, a fault line discovered in 2008, called the Shoreline Fault, runs about half a mile from the front door of Diablo Canyon. Opponents want new seismic studies before the plant's license is renewed, but the plant's owner, PG&E, the Nuclear Regulatory Commission, and other experts argue that the fault poses no threat that the nuclear facility cannot handle.

Turkey Point: In 1992 the Turkey Point Nuclear Power Plant on Biscayne Bay 24 miles south of Miami, took a direct hit from Hurricane Andrew, causing a loss of offsite power for more than five days. Backup

systems allowed operators to keep the reactors cool until power could be restored. However, in the wake of the 2011 disaster in Japan, concerns were quickly raised about the potential for a hurricane storm surge simultaneously severing grid power and inundating backup generators—precisely the recipe that crippled Fukushima.

Reactors in Southern New Jersey: In 1983, this potential was realized twice in four days at a pair of nuclear reactors in southern New Jersey. The New Jersey accidents did not result in any core damage or release of radiation, but they show that no one can predict what might upend all computer models, emergency planning, and backup systems designed to eliminate those narrow theoretical probabilities—or at least mitigate their effects.

Illinois Plants: The Dresden nuclear facility in Morris, Illinois, and the nearby Quad Cities plant in Cordova, Illinois, are located north of the New Madrid seismic zone. This area registered quakes greater than 7.0 in magnitude in 1811 and 1812, and it is known for somewhat more regular temblors of lesser intensity.

Indian Point: Fears of an earthquake near the Indian Point nuclear power facility, about 30 miles north of New York City, were stoked in 2008 when researchers at the Lamont-Doherty Earth Observatory at Columbia University discovered a pattern of small but active faults in the area, suggesting that earthquakes near the plant were more common than once thought. During the 2011 Virginia earthquake some minor shaking was felt at Indian Point, but the facility remained online and operating at full power. A spokesman for Entergy, the company that owns the Indian Point nuclear plant 50 miles north of New York City, reported "no issues" there after the quake.

6.8.1 Safety Measures for Future Design

Options: Nuclear plants can be located away from tsunami zones by storing cooling water on higher ground. Instead of using a once-through ocean coolant system, seawater can be pumped to reservoirs located a safe distance away.

A proper seawall to deflect a direct tsunami hit and diesel-powered backup generators situated high enough not to be inundated by water will ensure power plant safety. Such a system was not in place in Fukushima and the consequences were disastrous.

6.9 WHAT CAN WE LEARN FROM JAPAN'S MISFORTUNE?

Tsunami Early Warning System: As noted, Japan is the best-equipped nation to deal with shocks and tsunamis. Even considering level of devastation, the Tohoku/Fukushima disaster would have been much worse, experts say, had Japan not invested in shock-absorbing foundations for buildings and in tsunami early warning systems (Gusterson 2011). New construction in the United States should adopt Japan-style quake-proofing in vulnerable regions. And we should be examining aging nuclear reactors to judge their resistance to natural disasters.

Seawall Construction: We should also consider nonstructural mitigation measures such as seawalls, which are curved barriers that can be as high as 5 meters and are structured in a way to block the wave and redirect the water back to sea; and tsunami water gates, which detect seismic activity and close when they sense tremors, thus preventing potential tsunami waves from flowing freely.

Updated Design Code: In Georgia two new Westinghouse AP1000 reactors are being built at a cost of $14 billion. Their design is based on the latest structural advances and the most up-to-date seismic codes. The hazard data according to which the plants are being constructed are more extensive and accurate, and the concrete technology being used is the most advanced. NRC approval for construction has been obtained, and the reactors should begin producing electricity within four years. Let's hope that the lessons of Fukushima Daiichi will not have been forgotten in the interests of time and costs.

Architectural Planning of Buildings to With stand the Impact of Tsunamis

Non-nuclear Structures: Although this chapter's focus is on the very real, frightening risk to nuclear power installations, a note on mitigation of tsunamis' impact on other structures is appropriate.

Structures can be constructed to survive tsunami waves with repairable damage. For example, Dr Gopal Madabhushi of Cambridge University co-authored a study on tsunami-resistant designs for houses after the 2004 Indian Ocean disaster that devastated coastal areas in Indonesia, Thailand, Sri Lanka, and elsewhere.

The goal of any design is to protect the building and make it easier to reconstruct afterwards, on the assumption that the occupants have fled to higher ground. Researchers using a large wave tank came to the following conclusions:

- Instead of obstructing the wave, a building should be designed to allow it to pass through, causing minimum damage. Large doorways and windows offer a clear path through which the wave can pass. Internal doorways are aligned rather than staggered, to prevent it from becoming "locked" in. Destroyed doors and windows are easy to replace.
- Houses on piles might not be desirable in all regions for practical reasons.
- Buildings with ground floors left clear like a parking bay act effectively like a building on stilts, as is required in many areas subject to flooding from hurricanes and other disasters, allowing the wave to pass through below the main structure.
- Load-bearing walls should not be placed so as to be struck head-on by the wave, which will cause the collapse of the structure.
- Japanese Glass-walled Box Buildings: Collaboration between world-renowned architect Toyo Ito and creative engineer Mutsuro Sasaki resulted in the construction of the Mediatheque in Sendai, Japan, in 2001. The building, a "glass-walled box" supported by spiraling hollow steel tubes and a thin floor plate ribbed with steel, survived the March 2011 Tohoku earthquake.

6.10 CONCLUSIONS ON TSUNAMIS, EARTHQUAKES AND NUCLEAR POWER

Protecting a nuclear power plant against a tsunami is a new subject. Risk assessment is required since power plants can last a long time through regular maintenance. Fukushima Daiichi is a cautionary tale for operators and builders of nuclear power plants. Its Unit 1 reactor was old, ready for decommissioning, but was given a 10-year extension to operate. There was no elevated reservoir and protective wall to ensure backup in the event of the loss of cooling water, and—perhaps most telling—its operators admitted to serious lapses in necessary maintenance.

Worldwide, older power plants, particularly those in coastal areas, are most at risk from tsunamis, especially since many are located in areas of high population density. An accident like that at Fukushima Daiichi

would be a catastrophe on a monumental scale, with thousands dead and sick from radiation exposure and billions of dollars in property losses. It is even possible to imagine entire regions, like Chernobyl, made uninhabitable for many years. Seismic retrofitting of older installations is critical.

It is hoped that new construction of nuclear plants, like those in Georgia, are employing the most up-to-date seismic codes and siting and construction technologies specifically designed to withstand any seismic activity. These include seismo-architectural planning, base isolation and dampering, sensors to detect the very first rumblings of an earthquake, and elevated, protected cooling-water reservoirs. Developing early warning system for tsunamis and earthquakes would be most helpful in saving lives. For minimizing damages to nuclear and non-nuclear structures many practical options are proposed. Further research is needed to make these options financially feasible. There is an old saying that "every cloud has a silver lining" and Japan's loss is nuclear industry's gain.

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Nuclear Power
Massachusetts Institute of Technology 2003
Sovacool 2011

CHAPTER SEVEN

Post-Disaster Engineering: The Pakistan Earthquake of 2005

Chapter Outline

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7.1 INTRODUCTION

Recent Earthquakes in Pakistan and Turkey: This chapter broadly reviews the types of damages caused to property and life, based on which it recommends further development of a seismic resistant code, for retrofit and rehabilitation and for enhancement of design of new structures.

A detailed literature review shows contributions by the following organizations and design codes:

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Construction Manual for Earthquake-Resistant Houses Built of Earth: Pakistan Meteorological Department, Geophysical and Seismological Division (2005)

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The following authors have contributed to several aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to the study of latest Pakistan and Turkey earthquakes is presented here:

Benouar (2003); Bozorgnia and Bertero (2004); Chopra et al. (2011); Khan (1987); McGuire (1993); Godavitarne (2006); Greene (1987); Gulkan (2003); Khan (1986); Khazai (2006); Olsen et al. (1995); Tierney et al. (2005)

In addition to the above publications, relevant individual references are discussed in the text.

Most earthquakes take the communities in which they occur by surprise. The shock of the event needs to subside before a strategy for dealing with the destruction can be developed. There must be administrative machinery in place, using guidance from earlier disasters, to ensure readiness in the face of potential quakes and so minimize their effects. Readiness is achieved from hard lessons taught by previous quakes.

The aim of the recovery phase in disaster management is to restore the affected area to its previous state. Recovery efforts are concerned with issues and decisions that must be made after immediate needs are addressed. They are primarily concerned with rebuilding destroyed

property and repairing essential infrastructure, an important aspect of which is taking advantage of a "window of opportunity" for implementing measures that might otherwise be unpopular. Residents of affected areas are more likely to accept changes when a recent disaster is in fresh memory.

The target groups of a post-disaster recovery project include policy makers, urban planners, rural developers and community leaders, NGOs, master trainers, homeowners, developers, public representatives, professional engineers, designers, executing and maintenance agencies, work supervisors, overseers, engineers, and skilled and semiskilled construction workers. The objective of such a project is proper enforcement of building codes, safe land use patterns, and "Build Back Better" construction practices. The ultimate goal in long-term post-disaster engineering is the creation of sustainable and disaster-resistant properties through integrated planning.

This chapter takes as its subject the 2005 Kashmir earthquake in Pakistan, outlining the post-disaster engineering measures set up by international teams of researchers and Pakistani government ministries (Connors, 2007). The recommendations of these groups can serve as a model for remediation for other countries dealing with the catastrophe of an earthquake. The focus here is on technical measures, which include new construction to replace damaged buildings, selection of construction materials, restoration, repairs and retrofit, and development of new design and construction codes, as well as some long-term planning recommendations.

7.2 CASE STUDY: 2005 PAKISTAN EARTHQUAKE

The earthquake hazard in Pakistan is high, deriving from the collision of Indian and Eurasian tectonic plates. Densely populated cities like Islamabad, Rawalpindi, Quetta, and Karachi are most vulnerable. Karachi, with a population of 15 million, is at considerable seismic risk from the Pubb Fault as well as the Allah Bund Fault, which caused a major earthquake in 1819. The 1935 Quetta earthquake occurred on the Chaman Fault, where the Arabian plate slides beneath the Iranian plate along the Makran coast. In 1945 a magnitude 7.9 earthquake resulted in a tsunami with waves as high as 40 feet (Boen and Jigasu, 2003).

The 7.6 magnitude earthquake that struck northern Pakistan on October 8, 2005, was the most devastating natural disaster in the country's history. Its impact was spread over an area of 27,000 square kilometers across Azad, Jammu, and Kashmir (AJK) and the North West Frontier Province (NWFP). An estimated 80,000 people were killed and some 3 million people were rendered homeless. More than 1,000 aftershocks were recorded in the area, ranging from 5.0 to 6.0 in magnitude. In five districts of NWFP and four districts of AJK, public buildings, private housing, infrastructure, social services, livelihoods, and businesses were damaged or destroyed. The rugged geography of the affected areas, combined with a scattered settlement pattern, posed significant challenges for both humanitarian and reconstruction efforts. Landslides and rock falls damaged several mountain roads and highways, cutting off access to the region for several days. Liquefaction and sand blows occurred in the western part of the Vale of Kashmir and near Jammu. The impact was the greatest on private housing, transport, education, agriculture, and livestock.

It is estimated that as many as three-quarters of the fatalities from the earthquake were caused by the collapse of small dwellings. According to the joint assessment by the Asian Development Bank (ADB) and the World Bank, 203,579 housing units were destroyed and 196,575 were damaged. Some 84 percent of the total housing stock was damaged or destroyed in AJK, while in NWFP, 36 percent of the total housing stock was damaged or destroyed. Schools were especially vulnerable, with some 17,000 students perishing.

In the Muzaffarabad area, entire villages were destroyed. (See Figure 7.1.) At least 32,335 buildings collapsed in Anantnag, Baramula, Jammu, and Srinagar. Collapses also occurred in Abbottabad, Gujranwala, Gujarat, Islamabad, Lahore, and Rawalpindi. Several high-rise building

Figure 7.1 (a) Houses along the terrace rim and on flatland, north of Muzaffarabad, with cracks opening along the rim; (b) post-disaster debris disposal.

were damaged, and many small local medical facilities were reported badly damaged or destroyed.

Ninety percent of the destroyed or damaged buildings were in rural areas where there was a lack of safety standards in construction. Virtually none of the housing in these areas had incorporated seismic-resistant considerations in their design. In addition to non-seismic construction, there was generally poor quality of construction and maintenance.

Luckily, the quake occurred at a time of rapid advances in the science of earthquakes and in engineering and policy measures to reduce risk. It thus drew both national and international attention to the need for recognizing and assessing geologic hazards and for undertaking long-term programs for risk reduction.

The losses in the October 2005 earthquake are similar to the losses that can be expected in other regions. Managing this risk requires a long-term, comprehensive plan, reliable mechanisms for information sharing, and policy advice independent of agencies responsible for carrying out risk management programs. Several national and provincial governments and academic and private organizations of Pakistan had a stake in the post-earthquake planning.

Disaster Comparison of Northern Pakistan and Eastern Turkey Earthquake

On October 23, 2011, a magnitude 7.1 (compared to magnitude 7.6 of Pakistan earthquake) struck that was felt throughout eastern Turkey as well as in the neighboring countries of Armenia, Georgia, Azerbaijan, northwestern Iran, and in parts of Iraq, Syria, Lebanon, Jordan, and Israel. Seven aftershocks were greater than magnitude 5. As of October 31, 2011, there were more than 2,000 aftershocks greater than magnitude 2.

Details of Disaster: In the Ercis-Tabanli-Van region, at least 534 people were killed and 2,300 were injured; 14,618 buildings and homes were destroyed or severely damaged, and 18,000 were moderately or slightly damaged. At least 125,400 people became homeless. Telecommunications, electricity, and water services were disrupted. (Reports by USGS and Eastern Turkey Earthquake Clearinghouse)

Soft-story collapse regularly occurred in Turkish apartment buildings with the bottom story height higher than the stories above. Three- to seven-story concrete-frame buildings with concrete flat slabs collapsed around the urban centers of Ercis and Van. Pancake when columns of multiple levels failed.

Weak column–beam connections and weak building materials such as under-strengthened concrete impacted building damage ratios.

Long History of Earthquakes Similar to Those that Hit Pakistan: Turkey is a tectonically active country that experiences frequent destructive events. Movements are controlled by the collision of the Arabian and Eurasian plates. Large, translational fault systems extend across much of the central and western regions. The worst earthquakes may be summarized as follows:

- The Izmit earthquake of 1999 had a magnitude of 7.6. It killed 17,000 people, injured 50,000, and left 500,000 homeless.
- Approximately 70 kilometers from Izmir, an earthquake of 7.3 magnitude occurred in November 1976 near the Turkey/Iran border killing several thousand people.
- A magnitude 7.8 struck Erzincan in 1939, killing an estimated 33,000 people.

The Pakistan earthquake of 2005 had similar destruction of low-rise buildings that took place in Erzurum, eastern Turkey, in 1987 (visited by the author). Detailed recommendations prepared after the 2005 Pakistan earthquake and experience gained from the 1987 Turkish earthquake should be applicable to other countries to Izmit as well as to earthquake-struck regions in India, Iran, and Haiti.

7.3 PAKISTAN DISASTER RESPONSE

The Pakistan National Calamities Prevention and Relief Act of 1958 and the Local Government Ordnance of 2001 provide a framework for response and preparation for disasters nationwide. Relief following a disaster is managed through the Emergency Relief Cell (ERC) of the Cabinet Division and the Pakistan Army. Pakistani agencies both in Islamabad and in Azad Kashmir were fully mobilized and worked around the clock to meet emergency requirements. Local organizations involved in the post-disaster recovery efforts included the Meteorology Department, The Aga Khan Planning and Building Service, Architect Yasmeen Lari, NESPAK, NWFP University of Engineering and Technology in Peshawar, and the Pakistan Earthquake Relief and Rehabilitation Agency (ERRA).

Foreign relief and rehabilitation teams included the National Academy of Sciences, ASCE (represented by the author), the Asian Development Bank, the World Bank, and the U.S. Agency for International Development.

The loss of 80,000 lives in October 2005 is being taken seriously and with the assistance offered by US AID. As a result of the 2005 earthquake, preparedness is now an integral part of Pakistan's overall planning efforts, and considerable progress has been made in the recovery. On the recommendations of the USAID and NAS teams, the Pakistani government drafted a much needed seismic code in 2007 that is specific to current Pakistani materials and construction practices. It is encouraging also that an Earthquake Engineering Center has been established in quake-damaged Peshawar. Two similar centers have been recommended, one in Karachi and one in Peshawar. UET Lahore have a 3m × 3m shake table using latest measurement techniques. NED University Earthquake Research Center has also developed proficiency in testing scaled models of buildings. Incidentally, the location of shake table is not far from a Fault. Any movements are recorded by a seismograph. Students projects in progress have a direct bearing on revising the seismic code.

On a recent visit by the author to UET Lahore and NED UET, it was observed that the new centers are being managed by professors having research experience in seismic design.

However, given the magnitude of the damage and the difficult geographic setting, many challenges currently remain, among them capacity building in various fields, hazard assessment, seismic design and retrofitting, and strengthening of existing buildings in a cost-effective and culturally sensible manner.

7.4 POST-DISASTER INVESTIGATION

After a detailed field study, performed by the author and a team put together by the National Academy of Sciences, following the 2005 quake (Figure 7.2), comprehensive reports were prepared. The team members had experience post-earthquake reconstruction in California, Yugoslavia, Nepal, India, and Turkey, which helped them in identifying problems and in finding solutions. Many issues were similar to those associated with other earthquakes worldwide.

Existing buildings in many developing countries, Pakistan among them, are made of stone masonry, cement blocks, burnt bricks, and timber; roofs are thatched or covered with G.I. metal sheets, and floors and frames are often constructed of reinforced concrete. While adobe and

Figure 7.2 NAS team sent to survey post-disaster Pakistan to assist in reconstruction (author on the left).

earth houses are the most economical building solutions (and the most popular worldwide), they are socially unacceptable in Pakistan.

The majority of the damaged low-rise buildings appeared to have been nonengineered. The joints in rubble masonry walls were irregular and without the proper mortar mix. The walls usually rested on a small strip of footing and had little continuity with the roof, thus making the system unstable. Even some engineered construction may not have met the latest seismic design standards.

7.4.1 Methodology: Seismic Risk and Damage Assessments

Loss estimations are referred to as *assessments of seismic risk*. The assessment process involves determining the probability of various ground motions coupled with the possible damage of the building or its vulnerability under those ground motions. The results are defined as damage costs versus cost of new building and expressed as a percent of building replacement value.

After the 1987 earthquake in Turkey and the Pakistan earthquake in 2005, the author carried out damage assessment at sites in both countries, particularly of structures falling into the first two categories, which had a far greater number of failures than those in the third category. There was

much adobe construction in Turkey that had not been strong enough to resist the magnitude and duration of shaking. In Pakistan there was a mixture of nonengineered and engineered masonry and reinforced concrete structures designed to older seismic codes.

These types of failures were similar to those encountered in earthquakes in India, Iran, Indonesia, Haiti, and T.Chile. However, a case-by-case approach to damage assessment needs to be adopted based on the volume of aftereffects and the magnitude of aftershocks. A systematic, building-by-building damage assessment is a time-consuming yet a critical early step in recovery after an earthquake. This information can be used to help design repair programs. Based on seismic resistance and structural damage, the following four damage assessment categories predominanted:

- Non-engineered buildings and bridges
- Engineered buildings and bridges designed to old and defunct codes
- Engineered buildings and bridges designed to recent seismic codes

One of the first orders of business in the research team's investigation was a field survey of building types, which revealed the following:

- Low-rise (two-storied) buildings not designed to be seismic resistant
- Low-rise (two-storied) buildings not designed for seismic resistance with one or more unauthorized upper floors
- Low-rise (two storied) buildings designed for two-story seismic resistance but with one or more unauthorized floors added for which the original seismic design was not adequate
- High-rise buildings of three or more stories not designed to be seismic-resistant
- High-rise buildings partially designed to be seismic-resistant but without lift shafts and shear walls
- High rise buildings of three or more stories designed to be seismic-resistant but with unauthorized floors for which the original seismic design was not adequate

Using standard seismic computer software based on the latest seismic building codes, seismic and nonseismic analyses were carried out for the following purposes:

- Evaluation of safety factors and safety rating of buildings
- Classification of buildings in categories of structural deficiency
- Determination of suitable seismic retrofit based on type of structural deficiency

- Cost estimates for rehabilitating classified buildings against future earthquakes

Based on safety ratings, it was determined that many buildings would have to be demolished because of weak foundations or connections that represented a safety hazard to occupants (see Table 7.1). The three options are to build new building to new codes, repair and retrofit existing buildings or to replace/reconstruct as original building.

Although damage surveys were carried out in a few specific areas, a systematic survey conducted to document damage and to classify the types of buildings damaged in the entire zone would be needed. Data on damage and intensity distribution are crucial for determining earthquake source and fault rupture, and for understanding the relationship between the damage and the fault rupture and other geologic or geomorphic parameters, such as sediment properties and slope instability. This information is used guide reconstruction in the area and in other areas that

Table 7.1 Feasibility Studies in Construction Planning

Serial Number	New Using New Design	Repaired as Originally Constructed	Reconstructed (Completely Destroyed) as Originally Constructed
1	Seismic-resistant features	Assessment of necessary repairs and financial/ material assistance to be provided	Assessment of seismic-resistant features to be included
2	Site-specific features and modifications	Incentives necessary to repair as seismic-resistant (according to appropriate code)	Incentives necessary to rebuild as seismic-resistant (according to appropriate code)
3.	Should adhere to appropriate code	Information necessary about seismic repair	Information necessary about seismic rebuilding
4	Should be properly implemented	Materials to be used for seismic repairs (wood, steel, concrete, innovative materials, etc.)	Materials to be used for seismic repairs (wood, steel, concrete, innovative materials, etc.)
5	Verification of implementation/ compliance	Supervision to ensure compliance	Supervision to ensure compliance

may suffer similar events. A survey such as this should be undertaken before clean-up efforts destroy perishable data.

7.4.2 Findings

The investigative team found that many structures were in violation of building codes because of the addition of one or more floors on the original foundation and with the original column design. The added dead loads generated huge seismic forces, which led to unavoidable collapse. It was recommended that these buildings be demolished in the interest of public safety. Also, no solid "pours" of concrete were noted, and repairs appeared to have been made using materials added in small patching quantities. Longitudinal expansion joints reduced seismic resistance significantly.

Observed buildings appeared to have been lightly reinforced. Columns had few, widely spaced ties, and the masonry walls did not extend to grade, creating a soft-story condition. Some concrete work indicated a poor quality of sand, generally not washed or well graded. Construction appears in many cases to have been completed with inadequate cement and insufficient steel. It most likely had been undertaken by poorly trained and unskilled labor with little or no enforcement of building codes and little quality control of materials.

Local Traditions in Reconstruction

In any rebuilding scheme local traditions and designs are taken into account. In Pakistan, ventilation and cooling are important considerations. The front of the dwelling, including the doors and windows, usually opens to the west, facing the wind (which generally blows in a westerly direction). A verandah also helps in interior cooling and ventilation, and the exterior walls are painted white or are whitewashed to reflect the sun's rays. The house height is typically at least 12 feet in order to minimize the radiation of heat from the roof. Low-rise buildings are preferred to multi-storied buildings because they are safer in a seismic event and allow quicker evacuation before collapse occurs.

Designs for rebuilding incorporate local traditions and techniques as well as seismic resistance. As for the latter, because of the large number of parameters involved in a seismic event, choosing the best type of seismic-resistant structural model can be problematic. The solution is that selected models be tested using shake tables to determine each model's ductility

and resilience before adopting it for mass production. The most economical model is selected provided it has adequate seismic resistance as determined from shake-table tests. Time to construct is also taken into consideration as is cost. Effective low-cost techniques are based on the principles of engineering economics. Only engineered buildings would be considered in new construction or repair.

7.5 RECOMMENDATIONS

The following sections discuss the recommendations made by the NAS-ASCE investigative team for post-disaster reconstruction.

7.5.1 Dwellings and Other Small Buildings

Within the parameters of local traditions, the following options were considered: build a new house (new design), repair a house (original design), or reconstruct a destroyed house (original design). The condition and size of footings determined whether an existing building is demolished, retrofitted, or repaired. If the building was fully damaged, a new one would be built. However, even if the damage was major, a retrofit was still possible. The majority of such houses could be salvaged using modern retrofitting methods. One effective method of retrofit is to build a buttress column in the middle of the wall and connect the top of it to the roof and the bottom to the footing.

7.5.2 Suitable Construction Techniques and Materials

Deep piled foundations, though desirable, might not be feasible. The cost of steel piles and pile driving far exceeds superstructure cost. Besides, pile-driving equipment might not be easily available, and for security reasons there may be few piling contractors prepared to work in remote areas or where a different language is spoken.

Because of the shortage of traditional materials for the large-scale demands of post-earthquake construction, indigenous materials would have to be developed that are both lightweight and durable. For example, lightweight concrete and timber are suitable for seismic zones. It is not easy to do away with established construction materials, but investment in factories to produce precast concrete components, fiber-reinforced concrete, Ferro-cement, and lightweight aggregates could be considered.

If required, U.S. manufacturers might be persuaded to establish local factories in Pakistan in convenient locations. New materials such as LYTAG should be introduced.

Enclosing non-engineered walls in perimeter wire meshes for improving ductility can be tested before mass use. Walls and roofs would be made of local building materials whenever possible given availability and suitability. The following locally available material types are commonly used in Pakistan and are considered options because of their low cost:

- Natural coarse aggregate and pebbles found along the banks of streams
- Stone masonry, both dressed and undressed
- Cement blocks, both solid and hollow
- Burnt bricks, both solid and hollow
- Timber for frames and roofs
- Galvanized iron for roof sheets
- Reinforced concrete for frames and roof slabs, both cast in place and precast panels (but only if connected to walls)

Local stone masonry is very strong in transferring loads to footings. If properly constructed, the footings are seismic-resistant and have been known to last for centuries. Horizontal bed and vertical joints must be pointed with mortar. Burnt bricks (more expensive given transportation costs from brick kilns to the site) must be added. Standard-size solid cement blocks can be made at the site itself and tend to be cheaper. Hollow cement blocks require special molds and are made in a precast factory. They are lighter than solid blocks and perform better in a seismic event because of their smaller mass. Hollow blocks are also more thermally efficient.

Portland cement is readily available at reasonable prices, since there are many cement factories in Pakistan taking advantage of the abundance of silica and lime, which are essential for cement manufacture. In contrast, timber is usually in short supply and is rarely used, except for supporting galvanized iron roof sheets. A thin screed overlay can be added to the sheets to increase thermal insulation. For health reasons, asbestos roofing sheets are not to be used.

Reinforced-concrete slabs are heavier and generate higher seismic forces in an event. Such slabs left unconnected to walls caused many deaths in the 2005 quake from school roof collapses. If reinforced concrete were to be reused, strong connections would have to be made between roof and walls. Any use of concrete for construction would have

to take into consideration the different requirements of its manufacture in cold and hot weather.

Cold- and Hot-Weather Concrete Manufacture

The American Concrete Institute offers guidelines and general requirements for producing satisfactory concrete during cold weather. In particular, it discusses recommended temperatures of concrete, temperature records, temperature of materials, preparations prior to placement, duration of protection period, methods for determining in-place strength, form removal, protective insulating covers, heated enclosures, curing methods, and accelerating admixtures. References are included that provide supplementary data on the effects of curing temperature on concrete strength.

The standard specification for cold-weather concreting includes cold-weather requirements for preparations prior to placement of concrete and protection of concrete.

Hot-weather concreting techniques are for temperatures exceeding 75 °F. The duration of curing needs to be adjusted. The concreting practice in Pakistan shows that the correct water to cement ratio is being maintained.

Current practice shows that gaps in timber formwork through which water drips constantly leads to incomplete chemical action. Concrete thus produced tends to be weaker than required. Similarly, cold-weather concreting methods should be introduced when applicable.

The issue of temporary housing and the reuse of materials should be considered, depending on the following factors:

- The geologic, geotechnical, and seismic evaluation of rocks and soils of these cities
- Determination of seismic design parameters for seismic-resistant construction
- The analysis of instrumental seismic data for the provision of realistic PGA values
- Field studies shall also be focused to collect necessary raw data required for geologic mapping and delineation of fault rupture zones.

Generally, any recycled use of existing materials would have to conform to seismic design and seismic detailing.

To improve the quality and uniformity of construction and materials, a construction management system would be needed on the pattern of effective factory production procedures, including the comparison of preconstruction shop and as-built construction drawings. Also,

third-party validation, in the form of independent monitoring and inspections, would have to be performed regularly by independently appointed engineers, probably every six months. Financial audits might also be necessary.

Unsuitable Construction Techniques and Materials

Nonengineered adobe or unburned mud block would not be an ideal construction material. Adobe requires a certain percentage of clay in the soil to act as binding material, and this type of soil is not prevalent in Pakistan's rugged northern areas. The use of mud mortar would also be discouraged because of its low resistance to tension caused by seismic forces.

Conventional framed construction in reinforced concrete and steel, although desirable, would be beyond the reach of many Pakistanis. Rolled steel, though very good for construction, is in short supply in Pakistan and is thermally inefficient. The use of combustible materials such as Styrofoam and plastics would be discouraged because new construction and retrofit should meet minimum two-hour fire resistance criteria.

A method of retrofit using metal straps around the perimeter of the building is sometimes suggested. However, in Zone 4 seismic events, footings move. If they are supported on soft soil, they settle vertically and develop cracks. In such cases, wall straps may not be very effective.

7.5.3 Nonengineered Owner-Constructed Buildings

Reconstruction and retrofit goals would be furthered if consensus-backed material standards were developed. Many proposed building plans require materials and methods that may be expensive and impractical, particularly in remote mountain locations far from any roads. The concern is that individual owners might simply ignore the recommended plans and rebuild their homes with little or no improvement over the originals.

The solution would be to convene a working group of structural engineers familiar with masonry construction, especially stone with weak mortar, to work with Pakistani engineers on designs for rural housing and on recommendations for designs and construction techniques that balance safety, constructability, and cost.

This working group could also address the "engineered" building plans prepared by National Engineering Services Pakistan (NESPAK) to

determine if they might be made easier and cheaper while still providing a level of performance acceptable to Pakistani recovery officials and outside donors.

7.5.4 Schools and Hospitals

Many schools and hospitals throughout the region were damaged or destroyed (see Figure 7.3). The same systematic damage assessment of such buildings as for dwellings would be carried out to determine those that could be repaired and those that needed to be demolished and completely rebuilt.

American school officials, engineers, and policy experts collaborated with Pakistani colleagues in developing a comprehensive school seismic safety program that would consist of evaluating resistance of existing school buildings, establishing preparedness programs, improving earth science and disasters curriculum, and establishing capacity to construct future school buildings to appropriate standards. Reconstruction of some model schools based on seismic-resistant design was carried out under the supervision of the Seabees, an organization of the U.S. Navy. School projects were completed by the Seabees in Honduras and Afghanistan. Other types of reconstruction work were performed by them in Ethiopia and Guatemala.

Figure 7.3 Improvised classes held in the open air after destruction of school buildings.

In the long term it would be easier to rebuild or repair and retrofit schools, hospitals, public safety facilities, and other public structures on their original locations using seismic-resistant designs. Schools, for example, require larger lot sizes than most dwellings. A good starting point would be to look at existing plans, if available. Taking schools as an example, boards of education and private school owners could be contacted to obtain plan copies. Rebuilt schools would typically be not less than 14 feet high for thermal reasons, and rooms would be able to accommodate at least forty students. Roofs for schools, and other public buildings, should be constructed of masonry, but reinforced-concrete arches and domes might be used in keeping with traditional Islamic architecture.

7.5.5 Infrastructure

Infrastructure includes dams and nuclear power plants, highways and bridges, utilities, water mains, sewerage, and communications. Developing infrastructure, such as paved highways and airfields, is vital to support the construction of new townships. A feasibility study would be needed for new road construction or for repair or widening of existing roads. A span of up to 12 meters (two lanes with a shoulder and sidewalk) would be considered. Precast concrete bridges would be used for quick construction.

Repair or replacement of ruptured water supplies would be a top priority. As for power, it was suggested that the Pakistani government consider purchasing it from Central Asian countries, with transmission lines passing through the remote areas of Afghanistan. If this were to materialize, Muzaffarabad, Balakot, and other towns could be placed on a national grid, boosting the rural economy.

In the absence of a national grid, an alternative would be developing alternative power sources such as solar panels to take advantage of Pakistan's abundant sunlight. Solar panel manufacturing organizations might be contacted for this purpose. The use of wind energy techniques for small power generators would be another alternative and might lead to the development of a small local industry.

7.5.6 Existing and Future Buildings

Pakistani specialists indicated a need for advice and training on building structural retrofit and strengthening methods and in procedures to compare

the cost-effectiveness of alternative approaches. In the United States, the Federal Emergency Management Agency (FEMA) has produced a number of publications that, with minor modifications, could be helpful with regard to the evaluation and strengthening of the types of buildings prevalent in Pakistan. It was suggested that a workshop or short training program conducted by American specialists could also be useful to launch and direct a more comprehensive educational program that may be necessary to gain working knowledge of the state of the art in seismic retrofit.

It was also suggested that USAID sponsor projects to improve the capacity of Pakistani engineers to analyze and retrofit existing buildings and infrastructure. Experience in the United States over the last twenty years provides valuable insight into engineering and policy issues involved in retrofit efforts.

U.S. Aid Training for Pakistani Engineers

Study of Traditional Details and Materials: Traditional details, space, planning, spatial organization and materials. Proposed designs will be evaluated against appropriate criteria and recommendations.

Cultural Planning Aspects: Prepare designs for cultural suitability of space planning and sizes, material suitability, and skill availability. Most likely one design cannot fit all types. Also create cultural awareness and guidelines for designers; for example, the location of bedrooms to ensure privacy, windows to face wind direction for air circulation.

Seismic Design Aspects: Seismic detailing of components. Connection details should provide continuity. Instead of lintels, bond beams are required. Symmetry of structural components is required; a low-rise building is more stable in a shaking event than a high-rise building. Spread footings need to be tied with reinforced concrete beams in two directions.

Quick Construction Aspects: In place of cast-in-place concrete, precast segmental construction may be used to eliminate expensive formwork and save time for erection of formwork, site manufacturing efforts, and casting of concrete and curing of over four weeks.

Brainstorming: Regular meetings with members of the Institute of Architects Pakistan for planning low-cost houses in seismic zones while maintaining cultural traditions.

Functional Planning Aspects: Post-occupancy evaluation of already-built shelters to improve future designs.

Designs for Difficult Terrain: Areas of Bagh, Rawalakot, and Azad Kashmir, where low density and distances on steep slopes create special challenges for reconstruction.

Model Homes: Survey low-cost house designs offered by various organizations to be utilized in the new design to ensure a rational planning and cost criteria. Some of their concepts can be refined or incorporated in designs to be developed.

7.5.7 Urban Rebuilding

Low-cost schemes would be developed for the planning of new towns and cities based on economical but comfortable architectural designs and layouts. Towns would be planned around existing damaged buildings that had been repaired or retrofitted. Owners of destroyed homes might receive turnkey units, but they would be responsible for any extra costs.

If not feasible to build on the location of the original house because of soil conditions or proximity to geological faults, the buildings would be relocated. In one scheme of land distribution, a competent authority would negotiate with owners and issue the title of a new plot in return for an existing unbuildable lot. Such alternatives might be discussed at community meetings with owners and local administration officials.

In a row of houses, the collapse of one house triggers the collapse of all the others. A gap should be provided approximately every three houses to allow seismic movements. A flexible approach would include some basic seismic-resistant design principles but with more variation to allow for individual family preferences in terms of number of rooms and layout. Demonstration or model buildings would be very useful in promoting awareness of needed changes in construction practice and in encouraging acceptance of them.

It was recommended that streets be widened and straightened to avoid blockage by debris in a future event, which would require readjustment of property lines. After the Gujarat earthquake in 2001, such readjustments were conducted in each of the four towns that had to be rebuilt. This was done so that wider and safer roads could be built. Some Gujarat property owners were motivated to relinquish land through the offer of a new lot at a relocation site.

In any new city or town planning, provision is usually made for open air markets and for conducting business after sunset or in the mornings. Other amenities such as parks, community halls, and entertainment centers would be beneficial.

7.5.8 Reforestation and Preservation of Heritage Buildings

In seismic events, loose topsoil can trigger landslides. Because tree roots act as binding material for topsoil, the cutting of trees, especially on mountain slopes, would be banned and every attempt made to plant trees and promote reforestation. It was recommended that certain evergreen plants, which require a minimum amount of water, be introduced.

A program to identify and encourage seismic retrofitting of heritage buildings would be necessary. Pakistan is home to six UNESCO World Heritage Sites: the archaeological ruins at Mohenjo Daro; the Buddhist ruins of Takht-i-Bahi and the neighbouring city remains of Sahr-i-Bahlol; Taxila; the fort and Shalamar Gardens in Lahore; the historical monuments of Thatta; and the fort of Rohtas.

Many other sites of significant national and international cultural and historical importance might be vulnerable to earthquakes and should be assessed and prioritized for seismic retrofit. Heritage buildings have a special historical and cultural significance and are subjected to special design criteria for long-term survivability in a disaster of any type.

7.5.9 Seismic Hazard Mapping

A better understanding of seismic hazard is essential. Seismic hazard maps can be drawn to identify areas susceptible to future natural disasters including earthquakes, landslides, and erosion/settlement. They identify needs such as for special foundation treatments and localized resiting where appropriate They also provide guidance on applying this information in practical decision-making—for example, on questions such as moving towns or villages and prohibiting construction in their previous sites or in other vulnerable locations. A better understanding of future seismic risk is critical to decisions regarding seismic codes, building design, and construction practices and materials, as well as to changes in public policy. Successful policy enforcement depends on reaching decisions through consensus building, and care must be taken to avoid the perception that policies are being made on the basis of insufficient information.

More work is needed both in developing accurate and comprehensive seismic hazard maps covering strong shaking, fault rupture, landslides, and other ground failures and in involving the broader community in this task. Joint Pakistani-American workshops and studies could be helpful in reviewing the available data and clarifying the issues on such topics as

probabilistic hazard mapping, microzonation, and policy regarding the use of zone maps.

Developing better baseline hazard maps is a long-term, time-consuming endeavor that should be strongly encouraged. It requires data supplied by networks of seismic instruments located throughout the country that detect strong ground motion, field observation, and detailed analysis of historic records to project the frequency and intensity of future earthquakes. The investigation of seismic risk facing Pakistan's vulnerable areas includes geologic, geotechnical, and seismic analysis of rocks and soils, as well as determination of seismic design parameters for seismic-resistant construction as addressed by ERRA (2005. Infrastructure Reconstruction, Monitoring & Transparency Mechanism. Departmental Report).

Data generated by global positioning systems (GPSs) are the basis of the development of seismic hazard microzonation maps, using remote sensing (RS) and geographical information systems (GISs) that indicate different seismic hazard microzones. For example, GPS and other scientific instruments installed in different areas of Baluchistan indicate that a powerful earthquake, perhaps a magnitude 7.5, resulting from fast accumulation of underground stresses and movement of mountainous areas to the south, may hit Quetta, Murree, and Chitral. Indeed, thousands of people died in the powerful earthquake that hit Quetta in 1935. Other vulnerable cities include Chaman, Harnai, Shahrag, Loralai, Nushki, Khuzdar, and coastal areas of Makran.

7.6 BROAD RECOMMENDATIONS

As a result of the work of many investigative and research teams and observers after the 2005 Pakistani quake, as well as Pakistani and international agencies, several broad recommendations were developed. The following investigation based Pakistan's recommendations should be adopted by all countries of the world for which the risk of earthquakes and the devastation they create are looming. A bullet list is presented here:

- Building codes to be revised, implemented, and enforced.
- Cultural needs and traditions to be considered.
- All new structures to be designed to the latest seismic criteria.
- All existing structures to be evaluated for adequate seismic resistance and retrofitted to meet the new criteria laid down in seismic codes.

- All utilities and pipelines such as water, electricity, gas, and communications to be made seismic-resistant.
- Precautionary measures to be communicated to the public.
- Pakistan's seismological monitoring network to be modernized to perform a country-wide seismic risk analysis.
- Multi-hazard risk assessment and mapping to be carried out and investment in risk reduction to be made.
- An appropriate disaster management mechanism to be developed.
- Professional education and civil servant training to be carried out to improve risk management.
- A property insurance program to be instituted.
- Roads and bridges leading to hospitals and schools to be designed to remain open for access at all times.
- High priority to be given to medical facilities, schools, and public buildings with high occupancy and to bridges on important routes.

7.7 REMEDIAL WORK BY ERRA

The overall long-term objective of ERRA's rural housing reconstruction policy is to ensure that an estimated 400,000 houses that were either destroyed or damaged be rebuilt according to seismic-resistant design codes through grant assistance from the government to eligible households. ERRA (2005) has established the following principles as bullet list:

- Seismic microzoning that delineates the soil and terrain failure (landslide) zones where construction is prohibited. The earthquake hazard assessment is on a national scale as a long-term process supported by research and nationwide instrumentation.
- Damage and eligibility assessments that categorize the level of damage to each house to establish lists of eligible beneficiaries of government rebuilding aid. The assessments establish criteria for houses needing reconstruction and those needing only economically feasible restoration/retrofitting.
- A review and approval mechanism for construction guidelines and structural designs, as well as oversight of reconstruction/restoration activities, that ensures quality and seismic-resistant construction and compliance with social and environmental risk mitigation.

- Owner-driven rebuilding with assistance of the government, including a comprehensive community briefing on key elements of the reconstruction program prior to release of advance payments and government inspection through partner organizations.
- Minimum structural design standards that meet internationally accepted requirements for low-cost earthquake-resistant housing using traditional and current building techniques and easily accessible materials.
- Structural design standards that include thinner walls, lighter roofing, and well-connected structural systems, and that exclude *katcha* (makeshift) construction.
- For uniformity and consistency, construction drawings, bills of quantity, fabrication drawings, material schedules, and illustrated construction manuals for each type of wall and roof system employed, with standard design principles for different wall and roof systems common to each area.
- Rebuilding in situ whenever and wherever possible, addressing land ownership and availability issues.
- Minimum population relocation—only when necessary from sites where risks or hazards remain very high due to seismicity, topography, other environmental factors.
- Program sustainability maintained through parallel efforts in rehabilitation of livelihoods, physical and social infrastructure, and so forth.

In partnership with local authorities and community organizations ERRA provides technical assistance with the following Bullet List:

- Developing and providing Hazard risk maps
- Damage and eligibility assessment
- Categorizing level of damage to each house
- Verifying eligible beneficiaries
- Estimating the material requirements for reconstruction to contribute to supply chain management
- Seismic-resistant housing solutions with standard design principles for different wall and roof systems common to each area
- Facilitation of the building materials market particularly in rural committees and for community-based organizations to avoid material shortages and maintain prices at reasonable levels.

ERRA also supports local authorities and partner organizations through information dissemination and formal and informal training programs:

- Social mobilization, village profiles to guide reconstruction, community housing reconstruction, procurement, community monitoring
- Community land adjudication, relocation, and replanning; land and property rights
- Environment degradation reduction including environmentally sustainable reconstruction techniques

Longer-term education goals include academic scholarships provided at the university level for specialization in earthquake engineering. This might require bringing specialized contract staff from countries like the United States to teach in Pakistani universities, which have little in the way of seismic curriculum or faculty trained in seismic engineering.

7.8 CONCLUSIONS FROM PAKISTAN EARTHQUAKE STUDY

Northern Pakistan is located at the foothills of the Himalayas, the geologic formation of which is based on millions of years of tectonic plate movement and continental drift. The region happens to be one of the most active zones. It is not free of high magnitude earthquakes in the future. In spite of the sparse population density of Northern Pakistan, 80,000 people perished. It shows a non-engineered construction area utilizing only the local building materials, such as rubble masonry and limited timber roofing. The damage is of a similar scale to that of Haiti earthquake. Much needs to be done in the housing sector such as training of the masons or supplying precast buildings which are seismic resistant. The newly formed ERRA Agency is making progress in some of the new construction and relocation issues. NESPAK consulting engineers have framed a new seismic code in 2007 with the help of NAS of USA.

A great majority of structures in the Pakistan earthquake were designed according to older seismic codes and unfortunately could not take advantage of new developments in seismic resistance. For new structures, it will be possible to improve planning based on new codes and seismic design. Important existing structures such as high-rises, hospitals, schools, and other public buildings may be retrofitted. Residential and other buildings

not designed to seismic codes may be designated for early replacement. Property insurance against earthquakes is recommended in all cases.

Disaster remediation experiences come from many affected countries. A realistic approach is to interpret results these countries' seismic failure studies.

A reexamination of local seismic code provisions is necessary and new guidelines are required because of the unique materials and construction methods being used in reconstruction efforts, including those directed at nonengineered buildings. Recently developed innovative solutions such as base isolation, dampers, sensors, and shape memory alloys should be encouraged. For example, Indian building codes for brick buildings might be modified to fit the needs of Pakistan. Some research is needed to determine their effectiveness in the context of observed construction.

The ultimate goal in post-disaster engineering for long-term recovery is the creation of sustainable and disaster-resistant communities by minimizing the effects of natural hazards through integrated planning. This involves developing the seismic knowledge and skills of the community about disaster response and mitigation through education in the following bullet list:

- The role of the built environment in relation to the natural environment, patterns of open space, housing, and neighborhoods
- Transportation and utility design and configuration
- Design and building group configuration and location
- Seismic vulnerability of existing structures, particularly nonengineered structures and the use of modular design and construction
- Retrofitting and strengthening existing weaker buildings

The remediation and reconstruction model developed in Pakistan after the October 2005 earthquake has the potential to be applied in other affected countries, such as Haiti and Chile. The issues are common and optimum solutions are similar to a large extent—for example, safe seismic-resistant, low-cost housing that incorporates ductility and low weight. Low cost is essential in large-scale reconstruction efforts in less-developed countries.

For example, the author has developed three-segment precast lightweight concrete bent-frame models, which utilize varying depths of beam members. They can be used for single-story or two-story construction with a sloping roof. This technology is used in Mumbai in the construction of mobile crèche schools for children of laborers living near temporary construction sites.

Lightweight aggregate concrete is less than 50% of normal concrete weight. Bent shape and varying depth of beam result in the optimum size of members. Span lengths for residential buildings are 12 ft; for clinics, 15 ft; for hospitals, 18 ft (requires superior design as health unit); and for school buildings, 24 ft (requires special design for child safety from collapse).

Roofing members are precast tiles using ferrocement. Double wall panels with a cavity between the panels serve for insulation.

To enhance these efforts worldwide, particularly in less-developed countries where the need is greatest, the following are just some of the steps that should be taken:

- Training and education at all levels in seismology, seismic engineering, seismic-resistant construction, and post-disaster remediation
- Monitoring of earthquake activity to provide early warning
- Improvement of soil response to minimize liquefaction
- Performance-based seismic engineering—for example, base isolation and floating foundations
- Updating, implementation, and enforcement of seismic codes for retrofit and reconstruction
- Symmetric planning on micro and macro levels
- Construction of smarter structures ("Build Back Better")

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CHAPTER EIGHT

Seismic Bridge Design

Chapter Outline

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8.1 INTRODUCTION

Importance of Seismic Codes: Code provisions for the geometry of a bridge structure play a primary role in its performance during an earthquake. These methods are constantly changing with innovations in construction technology (such as accelerated bridge construction/pre-assembled bridges) or in configuration (such as integral abutment bridges). Although certain responses are similar between buildings and bridges—the two types of framed structures—the geometry (span length and height) is significantly different. For example, the use of reinforced concrete shear walls is in two directions in buildings with intermediate columns to distribute seismic forces on foundations; in bridges, only transverse wall piers are used. Both bridges and buildings consist of foundations, columns, and composite beam and slab systems. Building columns are closely and evenly spaced compared to bridge piers. For tall buildings, close, even spacing helps to reduce loads on foundations. Parapets have a significant role in bridges.

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This chapter broadly reviews the need for seismic design and based on which it recommends further development of a seismic resistant code, for retrofit and rehabilitation and for enhancement of design of new structures.

A detailed literature review shows contributions by the following organizations and design codes:

Applied Technology Council (ATC, 1996)

Manual for Railway Engineering (AREMA, 1999)

Liquefaction Study Report, Prepared for TRB,NRC (ATC/MCEER Joint Venture, 2001)

Regulations for Seismic Design: A World List (IAEE, 1996)

Comprehensive Specification for the Seismic Design of bridges (NCHRP, 2002)

Computer Software for Earthquake Engineering (NISEE, 1993)

Seismic Design Criteria (PCI, 2004).

The following authors have contributed to several aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to seismic design is presented here:

Building Configuration & Seismic Design (Arnold and Reitherman, 1982)

ADINA Software for Inelastic Models of Steel components of the Carquinez Bridges (Astaneh-Asl, 1997)

Seismic Retrofit of Steel Bridges (Malik, 1997)

Seismic Design Codes and Procedures (Berg, 1983)

Modeling Pile Behavior in Large Pile Groups under Lateral Loading (Dodds and Martin, 2004)

Seismic Analysis and Design of Bridge Abutments (Fishman and Richards, 1997)

Seismic Analysis for Design or Retrofit of Gravity Bridge Abutments (Fishman et al., 1997)

Bridge Retrofit Construction Techniques (Hipley, 1997)

Earthquake Design Criteria (Housner and Jennings, 1982)

Seismic Analysis of Bridges (Imbsen Associates, 1998)

Structural Details to Accommodate Seismic Movements of Highway Bridges and Retaining Walls (Imbsen et al., 1997)

Seismic analysis of Bridge Substructures Using AASHTO and AREMA Code Methods (Khan, 2002)

Comparative Study of Seismic Design Codes With The Proposed NCHRP Code 12-49 (Khan, 2003)
Modeling and Seismic Analysis of Integral Abutments (Khan, 2004)
Recent Developments in the Design of New Jersey Bridges Using Accelerated Bridge Construction Concepts (Khan and Dunne, 2008)
R-Factor Parameterized Bridge Damage Fragility Curves (Mackie and Stojadinovi, 2007)
Practice of Earthquake Hazard Assessment (PEHA) (McGuire, 1993)
Seismic Column Reinforcement Study (Memari et al., 2001)
Seismic Fragility of Existing Conventional Reinforced Concrete Highway Bridges (Mullen and Cakmak, 1997)
Performance Based Seismic Design for Bridges (Murillo et al., 2007)
Development of Analysis and Design Procedures for Spread Footings (Mylonakis et al., 2003)
Force-Deformation Behavior of Isolation Bearings (Pradeep and Paul, 2007)
Seismic Design and Retrofit of Bridges (Priestley and Calvi, 1996)
Seismic Design Criteria for Bridges and Other Highway Structures (Rojahn et al., 1997)
Seismic Vulnerability of Timber Bridges and Timber Substructures (Sharma et al., 2008)
Fragility Considerations in Highway Bridge Design (Shinozuka et al., 2007)
Fundamental Concepts of Earthquake Engineering (Villaverde, 2008)
Performance Estimates for Seismically Isolated Bridges (Warn and Whittaker, 2007)
Ductility of Rectangular Reinforced Concrete Bridge Columns with Moderate Confinement (Wehbe et al., 1996)
Seismic Design of Buildings and Bridges (Williams, 2003)
Evaluation of Seismic Retrofit Methods for Reinforced Concrete Bridge Columns (Wipf et al., 1997)
FHWA Bridge Seismic Retrofit Manual and Its Applications in Missouri Highway Bridges (Yen et al., 2001).

In addition to the above publications, relevant individual references are discussed in the text.

The configuration of structural members in bridges differs considerably from that of building frames. Bridges are seldom more than one floor, while tall buildings can exceed one hundred floors. Span lengths in bridges are many times greater than closely spaced building columns.

Also the use of bearings is uncommon in buildings. Finally, building height makes a difference to the seismic moments acting at the foundation levels (Khan, 2005).

Analysis of the static and dynamic behavior of bridges and buildings can be applied in a somewhat similar way. Structural engineers are expected to be familiar with the seismic analysis and design of both systems. The most important seismic effect on either structure is the low or high magnitudes of ground shaking and relative distance to the epicenter. Peak ground acceleration (PGA) and maximum considered earthquake (MCE) are based on the seismicity of the region in which the structure is located.

Conceptually, the fundamental equilibrium equation of base shear forces ($V = CsW$) is applicable to both bridges (AASHTO 2007) and buildings (ICC 2003). However, the factor Cs is based on common or different considerations. Both structural types have unequal multi-degree-of-freedom systems that affect their response to seismic activity. Single modes and multiple modes are possible due to relative flexibility (which is higher in a tall building compared to a bridge).

8.2 PHYSICAL EFFECTS OF SEISMIC ACTIVITY

Parameters Affecting Analysis and Design: The effects of seismic activity on a structure result from the following variations:

- Relative location in a seismic zone
- Soil response such as liquefaction
- Degree of ductility
- Function and occupancy

Uncommon variations affecting magnitude of displacement or drift, frequencies, and periods of vibration result from

- Acceptable level of seismic safety and maximum considered earthquake ground motion
- Different return period for bridges and buildings based on probability
- Different stiffness
- Different span lengths of bridges
- Structure height (bridges are single-story structures)
- Inherent structural redundancy in the structure (where large numbers of hinges form)
- Type of connection between beam and column

- Use of nonstructural items
- Response modification factors (R) for reduction in seismic forces based on inherent ductility level of the structural system

  The R factor depends on
- Predicted performance of the system when subjected to peak ground acceleration
- Vulnerability to failure of the gravity load-resisting system
- Reliability of system inelasticity with multiple yielding elements (if system redundancy and element over strength can be reached, a higher value of R is necessary)
- Potential backup frame resistance

It is important that in seismic-resistant design the dynamic forces in a structure be controlled by proper selection of its structural system and by the amount and distribution of its reactive masses that react to shaking. The smaller the reactive masses, the smaller the inertia forces. AASHTO LRFD Specifications have developed a comprehensive flow diagram for design procedures (Figure 8.1) based on seismic zones 1 to 4 (Table 8.1).

8.3 FHWA SEISMIC DESIGN AND RETROFIT

From knowledge gained through analysis of the 1971 San Fernando earthquake, the Federal Highway Administration (FHWA) published its *Seismic Design and Retrofit Manual for Highway Bridges* in 1987. Further knowledge of seismic phenomenon was gained from the 1989 Loma Prieta and 1994 Northridge earthquakes. Each major earthquake seems to point out structural deficiencies. Hence codes are keeping pace with improvements in technology and use of new materials.

Seismic Design and Retrofit incorporates a basic theoretical approach to bridge design that involves first and foremost a thorough grasp of the basic terminology of seismic hazard, deterministic and probabilistic seismic hazard analysis, and seismic zones. The theoretical model is based on multi-degree-of-freedom systems, pseudo-velocity and acceleration, quasi-static analysis, response spectrum analysis, and time history analysis. It employs both D'Alembert's principle and Newton's second law of motion to produce mathematical equations expressed both as nonlinear matrices, eigenvalues, and eigenvectors and as deformation response spectra.

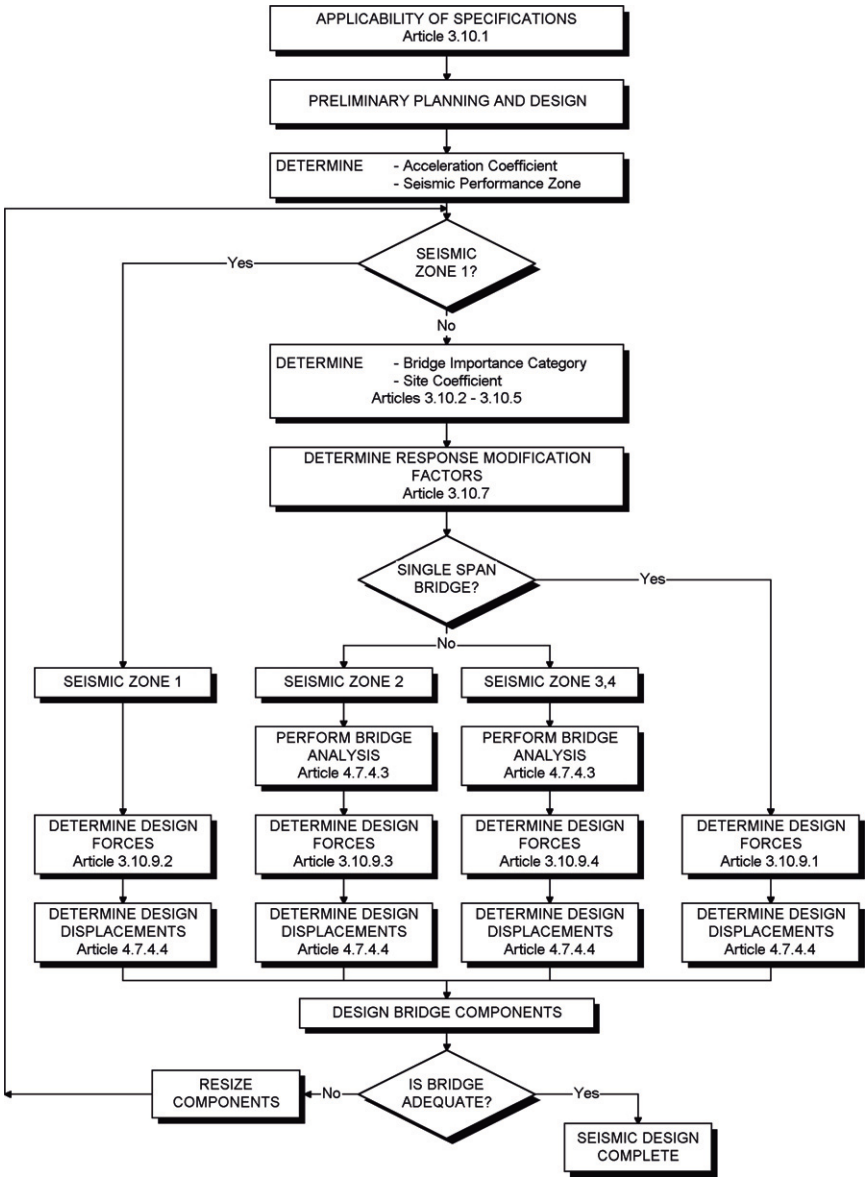

Figure 8.1 AASHTO flow diagram of seismic design procedures.

8.3.1 Methods of Structural Analysis

Three-dimensional structural models allowing for six degrees of freedom need to be developed to perform multi-mode response spectral analysis

Table 8.1 Seismic Zones

Acceleration Coefficient	Seismic Zone
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.19 < A \leq 0.29$	3
$0.29 < A$	4

using SEISAB, GT-STRUDL, or STAAD-Pro software. The parameters for seismic category, response modification factors, site coefficients, and the methods of analysis currently being used appear to be semi-empirical.

Most substructure response modification factors (R) have decreased in AASHTO's LRFD code, resulting in higher design moments and forces, given that design moment or force = moment or force from seismic analysis/R. Designs based on LRFD code result in increased member size or in reinforcement steel.

The current LRFD code reclassified sites as Seismic Zones 1 through 4 in place of A through D in the old LFD code, but there is no change in the range or magnitude of acceleration coefficients. Studies show that in current codes sufficient attention has not been paid to ductility or seismic detailing. The design needs to be performance-based.

A solved example using the single-mode spectral analysis method is given in the AASHTO standard specifications (Supplement A of *Seismic Design Commentary to Section 4, Analysis and Design Requirements*). A plot derived from SEISAB software is given in Figure 8.2 for a 2-span bridge with a transverse cross-frame comprising multiple columns. Load Case 2 is for longitudinal forces.

8.3.2 Caltrans Seismic Design

California has lost many bridges in the past due to recurring earthquakes. There are over 12,000 bridges in the state highway system in California. In 1975 the Federal Highway Administration contracted with the Applied Technology Council to develop updated seismic bridge design specifications, resulting in the publication of MCEER Joint Venture-6 in 1981. Force design methods were used in which structures were analyzed elastically and seismic forces were reduced for ductility and risk by a reduction factor, Z, which varied based on the structure's period and redundancy of its components.

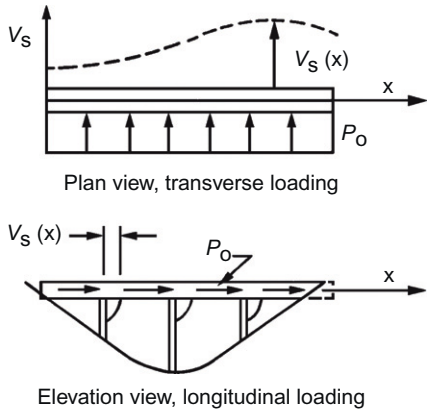

Figure 8.2 Bridge deck subjected to transverse and longitudinal loading.

The lateral earthquake loading specified in the *Standard Specifications for Highway Bridges* (AASHTO 2007) to the San Fernando earthquake used a simple force based equation:

$$EQ = C \times D$$

where C varies from 2 to 6% of gravity depending on the foundation type.

Elastic loads were determined based on one of a set of four standard acceleration response spectra, which were dependent on soil conditions at the bridge site. Typically bridges were analyzed using three-dimensional elastic dynamic multi-modal analysis. The flexural capacity of the columns was designed for these elastic seismic loads, reduced by the reduction factor Z, and the designer ensured that minimum prescribed transverse reinforcement requirements were met.

Variations in ductility were based on physical parameters such as

- Transverse confinement
- Geometry
- Strength and stiffness of adjacent components
- Foundation flexibility
- Column aspect ratios
- Fixity conditions
- Locations of plastic hinges (verified by considering relative strengths of components meeting at joints)

Because capacity design principles were not used, unrealistic loads were indicated, making the ATC specifications inadequate. The seismic design criteria initially used for the column seismic retrofit program were based on this same force design method.

Displacement Design

A substantial design shift occurred with the California Department of Transportation's (Caltrans') adoption of displacement design methods (as Memo to Designers 20-11) outlined in a report issued by the University of California, San Diego. Caltrans' seismic design criteria (SDC) incorporated these methods in an innovative seismic performance approach. In the SDC design philosophy there is a shift from a "force-based assessment" of seismic demand to a "displacement-based assessment" of demand and capacity. This newly adopted approach is based on comparing the elastic displacement demand to the inelastic displacement capacity of the primary structural components of a bridge, while ensuring a minimum level of inelastic capacity at all potential plastic hinge locations.

Displacement ductility methods were used initially for the retrofit of the Santa Monica Viaduct on Route 10 in Los Angeles, which was being retrofitted as part of the Caltrans Seismic Retrofit Program. Displacement design methods were incorporated into other seismic retrofit projects and ultimately became the de facto seismic retrofit design methodology. Caltrans' seismic design criteria incorporate the following elements:

- Displacement-based methodology
- Adequate confinement to ensure ductile response of columns
- Capacity protection of the superstructure and foundation to force plastic hinging into the well-confined ductile columns
- Balanced geometry and mass/stiffness compatibility to share seismic protection among the ductile columns and to avoid the concentration of damage in just a few locations
- Redundancy such that the overall bridge system performs well even if an individual component is significantly damaged
- Adequate support length to accommodate anticipated displacements

These provisions incorporated results published in ATC-32 and NCHRP 12-49, results of Caltrans seismic research program, and design practices developed through the experiences of the Seismic Retrofit Program.

Seismic design requirements may include data on ground stabilization underneath the structure since heavily shaken ground breakup may lead to

collapse. These data are obtained with shake tables and kinematic building models. (In general, a building model is said to have similarity with the real object if the two share geometric similarity, kinematic similarity, and dynamic similarity; this concept is explained in Chapter 5.) Currently the most advanced shake table is the Japanese E-Defense Shake Table.

8.4 DEVELOPMENT OF SEISMIC BRIDGE DESIGN CODES

Theoretical advancements take a long time to be translated into applicable design codes, which need to cover a wide variety of seismic zones and soil types. The purpose of a seismic code, whose application is mandatory, is to protect life and property, to develop a quality structure, and to implement uniformity in construction. An example is the International Building Code (63).

Seismic design of bridges in the United States is very much streamlined and regulated by the provisions of AASHTO 2007. Because of the specialized nature of seismic analysis and design, resulting from the variety of bridge systems, and analytical modeling in dynamic analysis (with different shaking modes and physical parameters such as seismic zones and soil conditions), it is mandatory that the code's stated methods be followed. However, they must be supplemented by the specifics of state codes such as calculations based on peak. While AASHTO specifications provide guidelines and concepts, state codes provide detailed drawings and plates to be used as samples for developing construction drawings. This approach maintains standardization and similarity for the thousands of bridges owned and maintained by state highway agencies.

Seismic zones are of fundamental importance. According to AASHTO 3.10.4, "Seismic loads shall be assumed to act in any lateral direction. Each bridge shall be assigned to one of four seismic zones, in accordance with AASHTO Table 3.10.4-1.... [The] appropriate R-factor shall be applied for reducing moments and forces both orthogonal axes of the substructure. Zone 1 structures are exempted from in-depth seismic design. Similarly, design of a single-span bridge is less complex than that for multiple spans.

Specifications are more descriptive than design codes and are less rigid. An example is AASHTO's *LRFD Specifications for the Design of Bridges*. Guidelines have greater flexibility in interpretation than do specifications. An example is AASHTO's *Guide Specifications for Seismic Isolation Design*.

Seismic Analysis and Design of Bridge Substructures

Because substructures take the biggest impacts during earthquakes, the following design criteria apply:

Gravity-Reinforced Concrete Walls

AASHTO LRFD Specifications Chapter 12 governs the design of the abutment's structural concrete and foundation. Abutments are designed to conform to the requirements for retaining walls and to resist all vertical and horizontal forces from the bridge superstructure and bridge approach slab. The use of fixed bearings upon abutments should be avoided wherever possible.

Abutments are analyzed as retaining walls, subject to seismic forces from the backfill, using the Mononobe-Okabe method. For multi-span bridges, piers act as transverse frames and may be modeled as walls, column bents, or pile bents.

Abutments are designed for a construction condition where the earth fill is fully placed before the superstructure is erected. The design for this condition includes a surcharge load for construction equipment. For this condition, the resistance factors may be increased.

Alternate Abutment Types

Examples of alternate-type abutments are mechanically stabilized earth (MSE) and prefabricated modular wall proprietary systems. Use of these systems should be compared with other abutment types to determine which option best meets project objectives such as structure cost, functionality, construction schedule, and aesthetics. For other project-specific parameters, analysis and recommendations should be included in the bridge type study and foundation report. Proprietary type abutment systems shall be designed based on a 100-year life.

The design of proprietary-type wall systems supporting abutment caps takes into account the anticipated movements and loads transmitted from the abutment caps. Plans and/or specifications for the wall system should clearly state the additional loads and/or movements that will be imposed on the wall system.

Approach Slabs as Structural Members

Approach slabs are provided for all abutments and are constructed for the full width of the roadway including shoulders. Two conditions of support for the approach are considered in the abutment design. For integral abutment bridges, the approach slab is designed as a structural slab resting on grade. Relief and sleeper slabs are provided parallel to the backwall. At "begin" bridge and "end" bridge hinged or partially fixed connections with backwall and at the far ends, transverse expansion joints with the roadway slab will be provided.

8.5 RECENT DEVELOPMENTS IN BRIDGE SEISMIC DESIGN

With the benefit of gains in knowledge of seismic engineering made over the past century, original seismic codes such as the original AASHTO for bridges undergo significant changes every few years. The most current code is AASHTO 2007, which has 14 chapters with hundreds of formulae covering the major disciplines in structural engineering. Section 3 deals with loads and load combinations (Table 8.2); Section 4, with seismic analysis methods. Sections 5 through 8 deal with detailed design of concrete, steel, aluminum, and wood structures, respectively; Section 9, with deck and deck systems. Seismic design has a more direct influence on the contents of Section 10, dealing with foundations, and Section 11, which treats abutments, piers, and retaining walls. Buried structures such as box culverts are addressed in Section 12. Section 14 deals with important design aspects of bearings and joints. Section 1 is introductory, Section 2 describes general design features, while Section 13 deals with railings only.

8.5.1 Computing Lateral Forces

In the higher seismic zones, vertical uplift or downward forces may be generated, but their effects are much smaller than those created by lateral forces. Lateral forces acting on bridges are proportional to the mass of the superstructure and substructure. They generate shaking and cause large bending moments on foundations. Their effect in generating bending moments is greater for vertical members than for horizontal members due to the lever arm component. Hence, as in tall buildings, the taller the bridge abutments and piers, the greater the displacement at the top, causing a physical overturning effect.

The important aspects of seismic design include calculations of lateral forces (based on seismicity, mass of structure, type of soil, and available ductility) (AASHTO 1996, 2007). Thus, the effects of forces generated on the ductility and sizes of slab and beams may not be as much as on the sizes of abutments and piers and their foundations or their respective reinforcements. Seismic designers have the choice of alternative types of lateral force—resisting systems. The system must be selected at the outset of the architectural design process. These basic systems have a number of variations, mainly related to the

Table 8.2 AASHTO LRFD Important Load Combinations and Load Factors

Load Combination										***Use One of These at a Time**			
	DC												
	DD	*LL*											
	DW	*IM*											
	EH	*CE*											
	EV	*BR*					*TU*						
	ES	*PL*					*CR*						
Limit State	*EL*	*LS*	*WA*	*WS*	*WL*	*FR*	*SH*	*TG*	*SE*	**EQ*	**IC*	**CT*	**CV*
STRENGTH-I (unless noted)	γ_p	1.75	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-II	γ_p	1.35	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-III	γ_p	–	1.00	1.40	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-IV	γ_p 1.50	–	1.00	–	–	1.00	0.50/1.20	–	–	–	–	–	–
STRENGTH-V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
EXTREME EVENT-I	γ_p	γ_{EQ}	1.00	–	–	1.00	–	–	–	1.00	–	–	–
EXTREME EVENT-II	γ_p	0.50	1.00	–	–	1.00	–	–	–	–	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
SERVICE-II	1.00	1.30	1.00	–	–	1.00	1.00/1.20	–	–	–	–	–	–
SERVICE-III	1.00	0.80	1.00	–	–	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
SERVICE-IV	1.00	–	1.00	0.70	–	1.00	1.00/1.20	–	1.00	–	–	–	–
Fatigue – LL, IM & CE ONLY	–	0.75	–	–	–	–	–	–	–	–	–	–	–

Source: AASHTO Table 3.4.1-1.

structural materials used and the ways in which the members are connected.

Notations used by AASHTO for Table 8.2 Load Combinations:

The following permanent and transient loads and forces shall be considered:

Permanent Loads

DD = downdrag

DC = dead load of structural components and nonstructural attachments

DW = dead load of wearing surfaces and utilities

EH = horizontal earth pressure load

EL = accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning

ES = earth surcharge load

EV = vertical pressure from dead load of earth fill

Transient Loads

BR = vehicular braking force

CE = vehicular centrifugal force

CR = creep

CT = vehicular collision force

CV = vessel collision force

EQ = earthquake

FR = friction

IC = ice load

IM = vehicular dynamic load allowance

LL = vehicular live load

LS = live load surcharge

PL = pedestrian live load

SE = settlement

SH = shrinkage

TG = temperature gradient

TU = uniform temperature

WA = water load and stream pressure

WL = wind on live load

WS = wind load on structure

8.5.2 Structural Systems to Resist Lateral Forces

Seismic designers have the choice of alternative types of lateral force—resisting systems. The system must be selected at the outset of the architectural design process. These basic systems have a number of variations, mainly related to the structural materials used and the ways in which the members are connected.

Moment-Resistant Frames. A moment-resistant frame is one with no diagonal bracing in which the lateral forces are resisted primarily by bending in the beams and columns mobilized by strong joints between columns and beams.

Diaphragms. Diaphragms are generally provided by the floor and roof elements of the building. They are horizontal-resistance members that transfer lateral forces between vertical-resistance elements, namely shear walls or frames.

Shear Walls in Buildings. Shear walls run from the top of the building to the foundation, with no offsets and a minimum of openings. They are designed to receive lateral forces from diaphragms and transmit them to the ground. They are used in combination with frames and relieve the shear forces (lateral seismic forces) from columns and help reduce column thickness.

Braced Frames in Buildings. Braced frames act in the same way as shear walls; however, they generally provide less resistance but better ductility depending on their detailed design.

The variety of structural systems and construction materials for bridges is not limited to slab and beam. AASHTO 2007 covers alternate systems that include truss, arch, segmental, suspension cable, and cable-stayed. Beams come in a wide variety, including I-girders, T-beams, adjacent box beams, spread box beams, and beamless solid slabs.

The footprint of bridges also varies, which changes their performance. Examples are straight, skew, and curved. The substructure of a bridge can vary as well: abutments are full height, stub, spill-through, and integral or semi-integral. Piers can be wall types, hammerhead, or columns and pier cap.

AASHTO's Section 4 methods of analysis (based on over ten load combinations) require equivalent static or dynamic analysis. When subjected to lateral seismic forces the vertical members (abutment and piers) will sway and drift. Besides magnitudes of resulting moments and forces, the displacements need to be finite. Optimization of member sizes will lead to economical solutions.

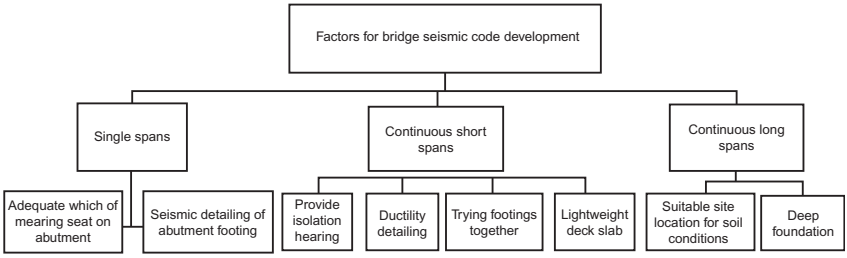

Figure 8.3 Important seismic planning of lightweight deck, isolation bearings, and deep foundations.

Figure 8.3 shows important issues in analyzing a bridge prior to design, such as width of seat, seismic detailing, and ductility. One particular issue is gravity load path. Ground accelerations act primarily on foundations but are distributed to superstructure as vibrations, in proportion to the mass. The load path causes horizontal sway of the structure and originates from the foundation upwards. For gravity loads such as dead and live loads, load path is applied from top down, that is, from horizontal deck slab and beams to vertical walls and columns through connections and transferred to footing resting on grade or on vertical piles.

Recall from Chapter 2 that plate tectonic activity can be classified into four seismic zones: Zone 1 follows the line of mid-ocean ridges. Seismic activity is low and occurs at very shallow depths. Zone 2 is associated with shallow-focus events, the result of friction between two mature plates that can be so great as to cause very large strains that are periodically relieved by large earthquakes. (The San Andreas Fault is a Zone 2.) Zone 3 includes oceanic and continental plates. This type of earthquake can be shallow, intermediate, or deep, according to its location on the down-going lithospheric slab. In this zone one plate is thrust, or subducted, under another plate to produce a deep ocean trench. Zone 4 occurs along the boundaries of continental plates, where high mountain ranges undergoing intense compression produce intermediate- and deep-focus earthquakes

These seismic zones reflect the variation in seismic risk and are of fundamental importance. AASHTO 2007 mandates that every bridge be assigned to one of them to classify a range of accelerations; to permit different requirements for dynamic methods of analysis; to aid development of substructure, pier, and column design details, as well as foundation and abutment design; and to allow minimum support lengths. For example,

bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, minimum requirements, as specified in AASHTO, apply

Table 8.3 shows requirements for analysis for single- and multiple-span bridges based on seismic zones. The design of single-span bridges is less complex than that for bridges with multiple spans. Table 8.4 lists requirements for span lengths and relative stiffness for multi-span bridges.

Because of the specialized nature of seismic analysis and design resulting from the variety of bridge systems and their analytical modeling in dynamic analysis (with different shaking modes and physical parameters—namely, seismic zones and soil conditions), it is mandatory to use AASHTO's methods. They need to be supplemented by specifics of state codes such as those based on peak ground acceleration.

8.6 SIMPLIFIED STEPS IN SEISMIC DESIGN OF BRIDGES

Bridge designers are expected to perform the following iterative steps in preliminary design and initial sizing:

1. Develop a geometry.
2. Assume member sizes.
3. Select loads and load combinations for strength and serviceability.

Table 8.3 Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single-Span Bridges	Multi-Span Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		Regular	Irregular	Regular	Irregular	Regular	Irregular
1	No seismic analysis required	⋆	⋆	⋆	⋆	⋆	⋆
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

Source: AASHTO Table 5.7.5.3.1-1.

Table 8.4 Span Ratio and Stiffness Ratio Requirements

Parameter	Value				
Number of spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutments	—	4	4	3	2

Source: AASHTO Table 5.7.5.3.1-2.

4. Apply equilibrium methods and conventional static or dynamic analysis formulae.
5. Solve resulting linear or nonlinear equations to obtain displacements, moments, forces, and reactions.
6. Assume rectangular stress blocks across the depth of section.
7. Equate external moments with moments of resistance based on rectangular blocks to obtain required steel section and reinforcement.
8. Check maximum displacement and reactions.
9. Revise member sizes for optimum solutions.

8.6.1 Local Site Effects

The severity of local effects on a given bridge depends on earthquake magnitude, distance from the epicenter, and local geological and geomorphologic conditions. Specific local geological, geomorphologic, and geostructural features can amplify or reduce ground wave propagation. AASHTO 2007 includes a simplified uniform load method that calculates the equivalent static earthquake loading:

$$p_e = \frac{C_{sm} W}{L}$$

where C_{sm} = the given dimensionless elastic seismic response coefficient, and p_e = the equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration (in kip/ft). The displacements and member forces for use in design are calculated either by applying p_e to the structure and performing a second static analysis or by scaling the results of the first step by the ratio p_{er}/p_o. Longitudinal restrainers complying with article skew of support measured from line normal to span must be provided.

Compared to AASHTO's earlier specifications, there is no change in earthquake return period, as shown in Table 8.5. Nor is there any change

Table 8.5 Importance Category on a Functional Requirement Basis

Importance Category	E.Q. Return Period	Open to All Traffic	Emergency Vehicles	Security/ Defense Use
Critical	2,500 years	Yes	Yes	Yes
Essential	450 years	No	Yes	Yes
Other	<450 years	No	No	No

Table 8.6 Substructure Response Modification Factors *R* Revised

Response Modification Factors	AASHTO LRFD	Old ASD/LFD
Wall-type pier	1.5 critical 1.5 essential 2.0 other	2
R.C. pile bents with vertical piles	1.5 critical 2.0 essential 3.0 other	3
Single column	1.5 critical 2.0 essential 3.0 other	3
Multiple-column bent	1.5 critical 3.5 essential 5.0 other	5

in response modification (*R*) value for superstructure to abutment or piers (Table 8.6). Seismic load combinations in two-dimensional forces also remain the same:

- (DL + 100% longitudinal + 30% transverse)
- (DL + 30% longitudinal + 100% transverse).

8.6.2 Testing of Scale Models of Structures

AASHTO 2007 requires that scale models of bridges be tested to establish force effects and/or load-carrying capacity under various conditions of traffic and/or environmental loads. The tests are applicable to important bridges and to bridges located in higher seismic zones include dimensional and material properties, boundary conditions, and all loads. These measured force effects are used for the following:

- To project fatigue life
- To serve as a basis for similar designs

- To establish permissible weight limits
- To aid in issuing permits
- To establish a basis of prioritizing rehabilitation or retrofit

The following is to be modeled as accurately as possible:

- The dimensional and material properties of the structure
- Its boundary conditions
- All loads

Dynamic analysis includes inertial scaling, load/excitation, and damping functions. Also required is a factored dead load to be simulated for strength limit state tests. Existing bridges may be instrumented as long as the instrumentation does not significantly influence the model response.

8.6.3 Improved Seismic Planning of Structural Members

Basically, lateral forces on a structure during a seismic event lead to displacement, velocity and accelerations, and oscillations in transverse and longitudinal directions. Thus, the two most important requirements in seismic planning of bridges that will withstand lateral forces are ductility (by proper connections between structural members and foundations) and light weight (by use of lightweight aggregates and materials). Ductility is a crucial factor in the response of structures to seismic activity because a ductile structure can absorb much more force before it fails than a nonductile structure can. Conversely, nonductile structures such as unreinforced masonry or inadequately reinforced concrete are very dangerous because of the possibility of brittle failure.

Along with ductility and light weight the following are also required:

- Low height-to-base ratio to minimize the tendency to overturn.
- Symmetrical plan shape minimizes torsion. Identical resistance on both axes would make the building equally strong in all directions. Optimized structural configuration results in placing seismic resisting elements at perimeter for maximum torsional resistance.
- Short spans for low unit stress in members. Multiple columns provide redundancy and loads can be redistributed if some columns are lost.
- Continuous load path—uniform loading of columns and walls and no irregularity.

In a model design, the lateral force—resisting elements are placed on the perimeter of the building, which is the most effective location. As the

building characteristics deviate from the symmetric model, the building becomes increasingly irregular. It is these irregularities that affect the building's seismic performance.

- No large cantilevers, which may increase vibrations.
- Equal or nearly equal spans and heights of piers and abutments. Bridges on fast-flowing rivers causing soil erosion should preferably have pile foundations to be able to resist seismic movements.
- Substructures, subject to the most severe impacts of earthquakes, to be designed to conform to the requirements for retaining walls and to resist all vertical and horizontal forces from the bridge superstructure and approach slab.

8.6.4 Analysis and Design Computer Software

Modern seismic design involves a large number of interconnected formulae that are not easy to solve without computer algorithms. These formulae involve dynamic behavior and consist of a large number of variables, such as geometry, seismic category, and geotechnical considerations. The common theoretical approach involves

- Seismic hazard and damage assessments
- Earthquake magnitude, intensity, and duration
- Deterministic probabilistic seismic hazard analysis
- Seismic zone, site class, soil type, and importance factor
- Figures 8.4 and 8.5 are flow diagrams for simplified methods of analysis of single- and multi-span bridges. Several software programs are available, such as *SAP 2000*, *STAAD-Pro*, *ANSIS*, *NASTRAN*, and *OpenSees*.

The cost of a bridge increases significantly when seismic effects are included in the design criteria. It makes economic sense to incorporate the latest state-of-the-art in design codes and to provide the engineer access to this rather complex aspect of structural mechanics. After the 1971 San Fernando earthquake, a great deal of interest was generated in this discipline and as a result, FHWA published *Seismic Design and Retrofit Manual for Highway Bridges* in 1987. The 1989 Loma Prieta and 1994 Northridge earthquakes, and more recent earthquakes in California alone, have revealed safety concerns even for structures designed in conformity with the latest codes.

Seismic codes, whether new or old, involve a large number of variables such as seismic category, dynamic behavior, bridge geometry, and

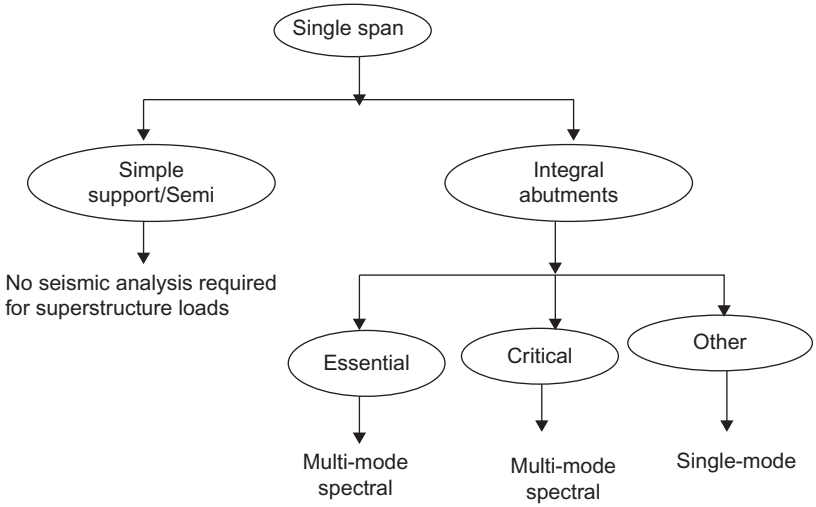

Figure 8.4 Flow diagram for single-span bridges spectral analysis.

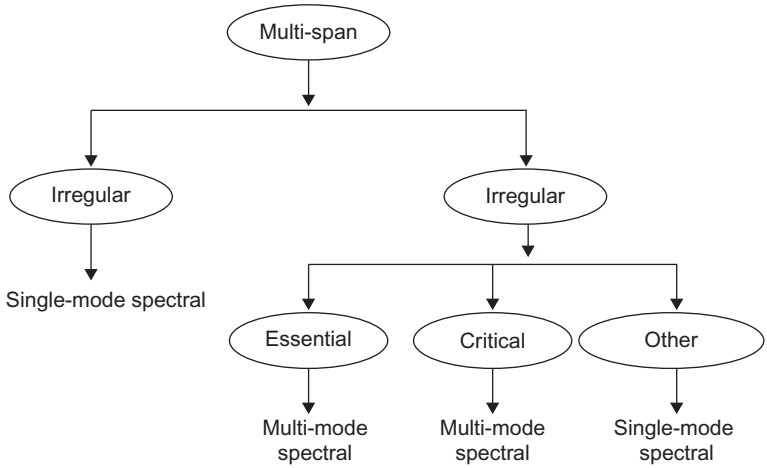

Figure 8.5 Flow diagram for multi-span bridges spectral analysis.

site suitability. Similarity seems to exist between extreme seismic impact effects on structures and those due to blast loading; a greater understanding of the latter will also help in the better understanding of the former.

8.6.5 AASHTO's Recommended Analysis Methods

The following are AASHTO's recommended analysis methods for multi-span bridges based on seismic zone. Refer to the section numbers in parentheses for more information in AASHTO 2007.

- Method 1: Single-mode spectral analysis (9.7.9.3.2b)
- Method 2: Uniform load (9.7.3.2c)
- Method 3: Multimode spectral analysis (9.7.9.3.3)
- Method 4: Time-history method (9.7.9.3.4)

8.7 SEISMIC PLANNING AND DETAILED DESIGN

It is convenient to perform design in two stages, preliminary and final. Initial design is based on limited data about topography, geometry, and soil information. It is used for planning purposes, selection of alternatives, and cost comparisons running into millions of dollars. Once the geometry and footprint are set, preliminary design is refined by site information from field surveys and soil tests. Coordination is required with the client and supporting disciplines such as highway, hydraulic, geotechnical, and environmental engineering in evolving the final design.

NEHRP Response Criteria

According the National Earthquake Hazard Reduction Program (NEHRP), the following bridges are to be designed using the NEHRP response spectra:

- Single span
- Bridges with less than 50,000 square feet in deck area or those with an overall length less than 1,000 linear feet
- Bridges with a deck area exceeding 50,000 square feet or an overall length exceeding 1,000 linear feet (these shall require a site-specific evaluation)

Site-specific procedures may be used for any bridge, but shall be used where any of the following apply:

- The bridge is located on Profile Type F soils (as defined by NEHRP).
- A time-history response analysis will be performed as part of the design/retrofit.
- The Seismic Zone at the site shall be defined according to Table 3.10.4-1 in the AASHTO LRFD Bridge Design Specifications. These seismic zones reflect the variation in seismic risk across the country and are used to

permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures. The value of "A" shall be taken as the peak ground acceleration (PGA) from the latest USGS maps for the 2,500-year event at the site.

For engineers the major issues to be considered in seismic-resistant planning and design are presented in a bullet list:

- The causes of earthquakes and concepts such as epicenter and relative location in relation to tectonic plates.
- The appropriate LRFD design method for determining member sizes, connection details, and foundation sizes.
- The best retrofit for bridges not originally designed for seismic events or those designed to substandard criteria and so unable to resist any earthquake.
- The type and profile of the soil on which the bridge is to be built.
- Displacements, oscillations, vibrations, and computed shear forces and bending moments during a seismic event.

Once these critical issues have been resolved, the engineer's responsibilities include the following:

- Identifying critical facilities and lifelines
- Advising the client on seismic criteria and costs
- Structural planning
- Selecting locations and types of bearings (rotational/translational)
- Compliance with relevant seismic codes
- Studying past structural design
- Selecting computer software for seismic analysis
- Knowing and employing National Earthquake Hazard Reduction Program (NEHRP) response criteria if applicable to the bridge being constructed
- Applying seismic detailing
- Preparing construction drawings and guidelines for the contractor
- Training design engineers in seismic design procedures
- Following the QA/QC procedures for the project and keeping seismic design costs to a minimum

8.8 IMPORTANT DESIGN DEVELOPMENTS

The following sections discuss the latest developments in seismic bridge design approach:

8.8.1 Performance-Based Seismic Engineering

Performance-based seismic engineering (PBSE) establishes performance objectives such as those based on elastic design or inelastic design criteria:

- No damage—seismic category 1
- Moderate damage—seismic category 2
- Safety—seismic category 3
- No collapse—seismic category 4

The first three categories are more applicable to the northeast United States; the fourth, to the high seismic risk regions of the western states and Alaska. PBSE can also be based on analytical methods such as single-mode, multi-mode, or response spectrum analysis.

8.8.2 Integral Abutments

Integral abutments are a new application for substructure types. As an example, a modification to AASHTO Table 3.4.1-1 (also shown above as Table 8.2 for reference) for general load combinations is listed in Table 8.7 for Pennsylvania's increased live loads (PHL93) (a 25% increase in truck axle loads). In different states, the use of unique integral connection details exhibits minor differences. Massachusetts code requires a less thick approach slab than do New Jersey and Pennsylvania. Other differences are in detailing of reinforcement steel and may affect bridge behavior.

The parameters to be considered in the theoretical model are as follow:

Type of Beam. For beams spanning rivers, where corrosion is a problem, and for bridges with small spans, concrete (spread) box beams may be used. In other cases, steel beam is preferable because of improved thermal performance.

Beam Spacing. Adjacent box beams are to be avoided, since larger seismic forces would result from the increase in mass created by closer spacing.

Pile Spacing. Bridge stability is dependent on spacing of the single row of piles. Although pile spacing is determined by frame analysis

Table 8.7 PennDOT Design Manual (DM-4) Load Combinations for Integral Abutments
Load Factors

Load case	SERV I		STR I		STR IP		STR II		STR III		STR V		Extreme II	
	max	min	max	min	max	min	max	min	max	min	max	min	max	min
γ_{DC}	1.00	1.00	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90
γ_{DW}	1.00	1.00	1.50	0[a]	1.50	0[a]	1.50	0[a]	1.50	0[a]	1.50	0[a]	1.50	0[a]
γ_{EV}	1.00	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00
γ_{EH}[b]	1.00	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
γ_{ES}	1.00	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
γ_{LS}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50	0.50
γ_{LLIM}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50[c]	0.50[c]
γ_{PL}	0.00	0.00	0.00	0.00	0.75	0.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
γ_{WS}	0.30	0.30	0.00	0.00	0.00	0.00	0.00	0.00	1.40	1.40	0.40	0.40	0.00	0.00
γ_{WL}	0.30	0.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.40	0.40	0.40	0.00	0.00
γ_{CE}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50	0.50
γ_{TU}[d]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Design vehicle	PHL-93		PHL-93		PHL-93		P-82		—		PHL-93		PHL-93	

and soil considerations, the minimum of 2.5 times the diameter of the pile is allowed by AASHTO. For simplicity of construction, a pile should be placed at the centerline of every beam end or at alternate beam ends.

Pile Length and Column–Soil Interaction. An equivalent pile height can be determined from analysis of single-pile programs such as L-Pile or COMP624P. The buried length of pile at displacement/moment reversal location can be obtained from computer output. Effective length of pile for frame analysis is the sum of unsupported length of about 5 m (pile length encased in pipe) and the buried length computed.

Pile Orientation. Since freedom of movements of top of pile is the main consideration, the minor axis of pile should be placed perpendicular to direction of beam.

Types of Connection Details. Figures 8.6 and 8.7 show the types of connections between abutment cap and beam ends and abutment cap and structural approach slab. Each state has its own requirement. The details are similar as they use stub abutments on a single row of piles.

8.8.3 Multi-Hazard Design

Design of Highway Bridges against Extreme Hazard Events: Issues, Principles and Approaches (by MCEER in 2008) is the first in a series of publications dedicated to multi-hazard design principles for highway bridges. The report's eight contributors offer perspectives from academia, private practice, and government agencies. Along with new detailed seismic maps, the report includes the following proposals:

- More definitive design for earthquakes and for performance objectives and damage states.
- Design and construction incentives for performing a more sophisticated "pushover analysis" that provides information on expected deformation demands of columns and foundations, leading to greater understanding of expected performance.
- New spectral shapes that remove conservatism and provide a spectral shape that decays as $1/T$.
- Use of earthquake-resisting systems (ERS) and earthquake-resisting elements (ERE) that are not permitted in current AASHTO provisions.

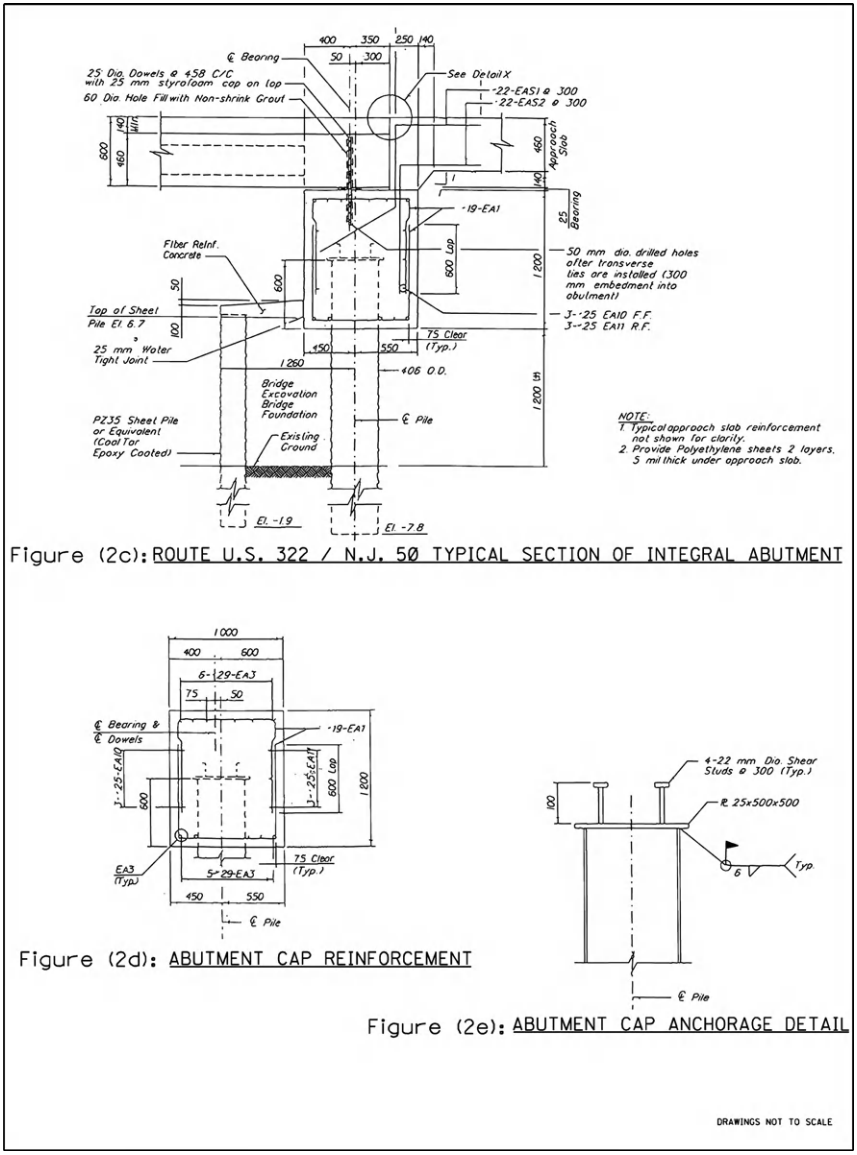

Figure 8.6 Connection details for integral abutments used in New Jersey.

The most significant proposal offered by *Design of Highway Bridges* is the use of new soil site factors as they impact the level of seismic design forces. Soil site factors increase with decreasing accelerations as a result of the nonlinear response effects of soils.

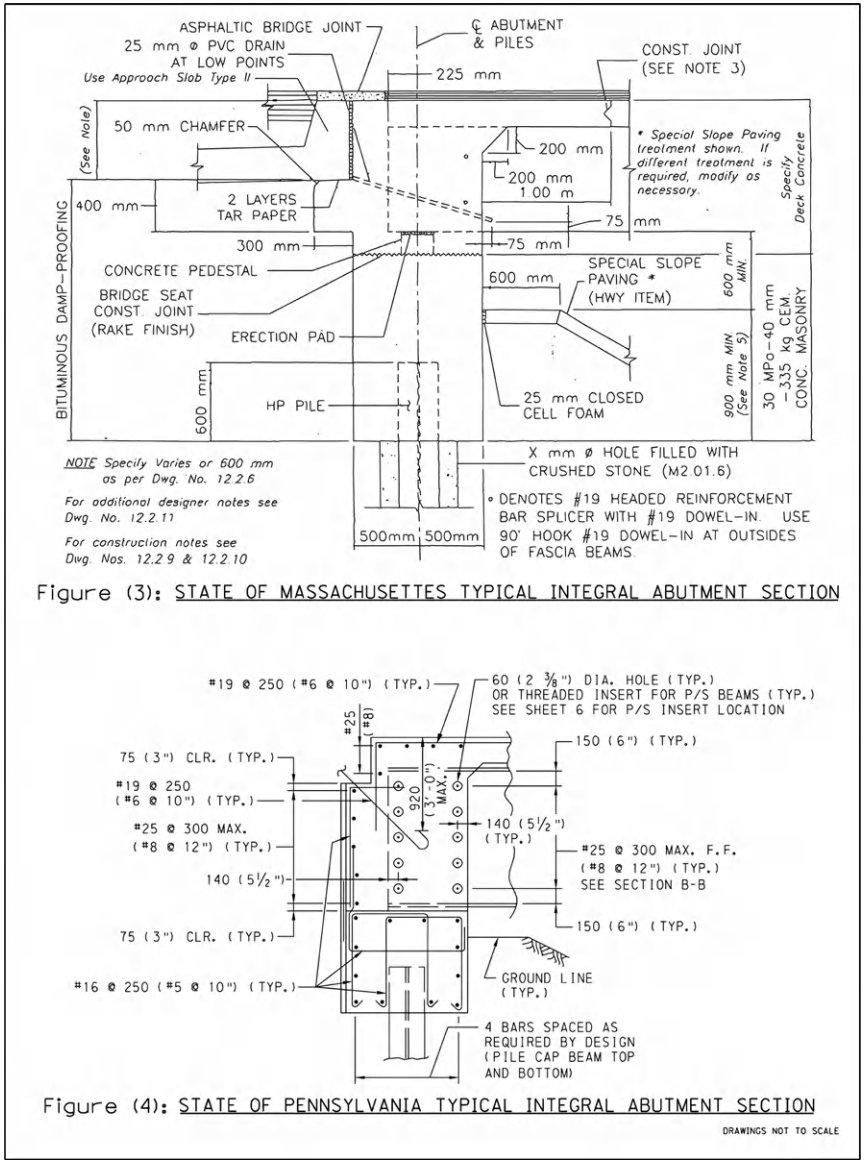

Figure 8.7 Connection details of integral abutments in Massachusetts and Pennsylvania.

8.8.4 Steel and Concrete Bridge Design Requirements

A comprehensive set of special detailing requirements for steel components, (AASHTO LRFD Code 2007, Section 6) which are expected

to yield and dissipate energy in a stable and ductile manner during shaking, has been developed. These include provisions for ductile moment-resisting frame substructures, concentrically braced frame substructures, and end-diaphragms for steel girder and truss superstructures. For concrete bridge design, the minimum amount of longitudinal steel has been reduced from the current AASHTO requirement of 1.0 percent (AASHTO LRFD 2007 Code to the new requirement of 0.8 percent; this is expected to result in significant material cost savings when used with capacity design procedures.

8.9 COMPARISON OF HIGHWAY AND RAILWAY BRIDGES

Railway bridges have higher risks due to concentration of passengers and are more conservatively designed than highway bridges and have historically performed well in seismic events with little or no damage for the following reasons:

- The track structure functions very effectively as a restraint against longitudinal and transverse movement.
- The spans are small.
- Heavy concrete decks are not present and dead load inertia forces are smaller.
- The types of damage that are permissible are very limited compared to highway bridges.
- Post-seismic event operation guidelines put restrictions on train traffic and speeds depending on the intensity of the event, until proper inspection is carried out.
- Level 1 ground motion has a smaller return period for railway bridges: 100 years as compared to 450 years for highway bridges. Acceleration coefficients are expressed as a percent of gravity for return periods of 50, 100, 250, 475, and 2,400 years.
- Risk factor is an integral part of seismic design for railway bridges. For level 2 and 3 ground motions, risk criteria are based on a high passenger train occupancy rate.
- The seismic response coefficient (*Cm*) is multiplied by a damping adjustment factor usually greater than 1, resulting in higher seismic force.

- Analysis is based on serviceability and ultimate and survivability limit states, unlike LRFD analysis for highway bridges.
- The substructure response modification factor (R) is smaller for level 3 ground motion, resulting in higher seismic design moments.
- Train live load is combined with dead load to give higher horizontal forces.

A number of bridges located in New Jersey, New York, and Pennsylvania were analyzed by the author. The findings with regard to analysis procedures have contributed to a chapter, "Seismic Analysis and Design" in the proposed *New Jersey LRFD Bridge Design Manual*. By way of illustration one study is discussed in the following section. All four bridges have acceleration factors of 0.18.

8.9.1 Case Study: Route 46, New Jersey

Figure 8.8 shows a plan of U.S. Route 46 over Peckman's River in New Jersey. The bridge replacement was required due to scour from Hurricane Floyd. In New Jersey, all counties are classified as Zone 2. Hence, a quasi-static method rather than a dynamic analysis, and such response spectrum is not required.

The uniform load method is applicable to regular bridges with acceleration coefficients of 0.10 and 0.18. The method is described in AASHTO (AASHTO 1996, 2). The procedure used is described in detail in a paper by Khan (2004).

- From single-mode coefficients, static displacement $Vs = \rho \quad L/k;$ $\gamma = \beta Vs$
- Compute the time period $T = 2\pi(\gamma/\rho.\alpha.g)^{0.5}$
- Elastic seismic response coefficient $Cs = (1.2AS)/(T)^{2/3}$ where A = acceleration coefficient, S = site coefficient, and T = period of oscillation.

  A limiting value of 2.5 for A was used.
- For single-mode analysis, using deck weight Vs and single-mode coefficients $(\alpha),(\beta),(\gamma)$ factors
- An equivalent static earthquake loading $pe(x) = \beta(Cs)(w(x) \quad Vs(x)/\gamma$ was computed and modified by the response modification factor R.
- Seismic force = $pe(x)L/R$.

  Seismic load combinations used:
- (DL + 100% longitudinal + 30% transverse)
- (DL + 30% longitudinal + 100% transverse)

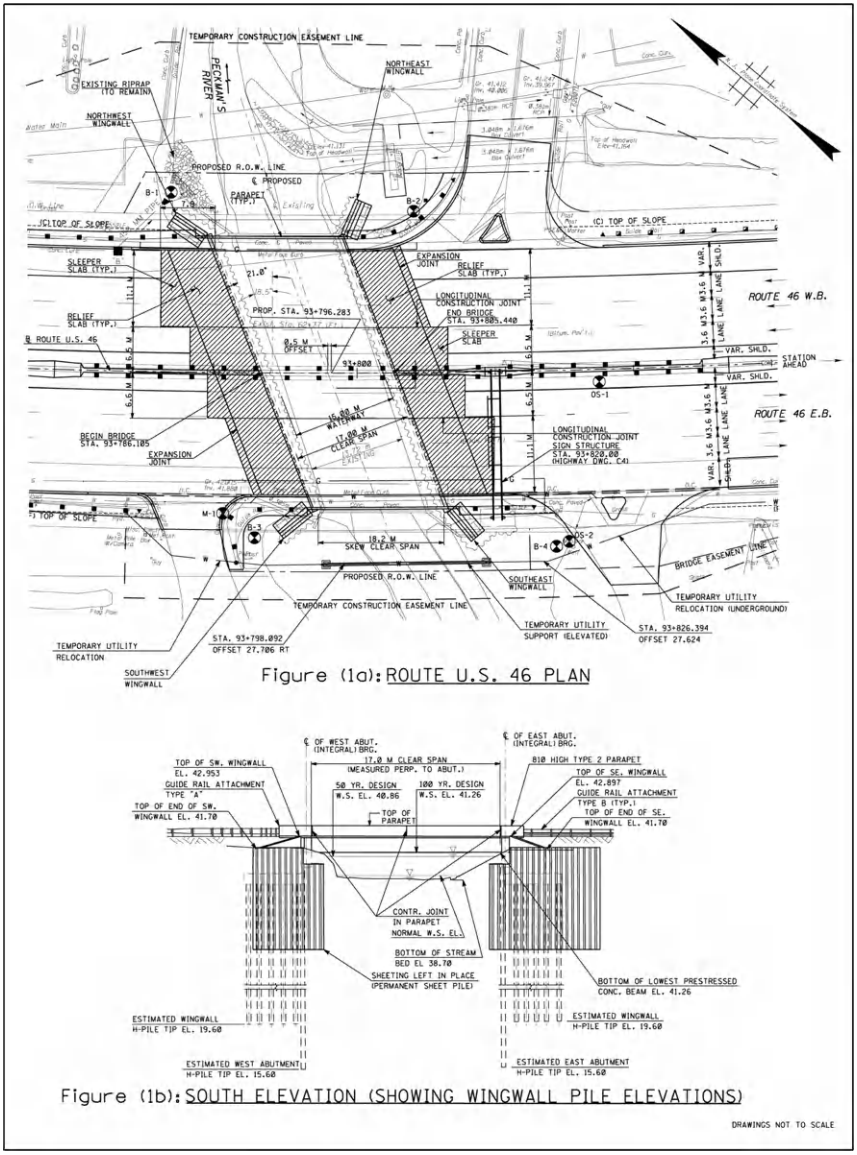

Figure 8.8 Case study of an integral-abutment bridge in New Jersey designed by the author.

Unlike conventional single-span bridges where seismic forces are neglected, there is a need to consider seismic behavior in single-span bridges with integral abutments to aid in the design of the buried abutment piles.

Freedom of top of pile movement for thermal and seismic loads is provided by placing the top unsupported 5 m length in a steel pipe. Only a single row of piles is used. For pile design, seismic moments and forces are combined with dead load and partial live load results. Since the pile minor axis is placed perpendicular to the direction of beam, it is important to check the total unsupported pile length of pile, for axial buckling.

For bridge stability, efficient details of integral connections between pile cap (with single row of piles) and beam ends are required. Connection details from various state bridge design codes were studied and compared.

The following observations resulting from the preceding study and the others can be made:

- Seismic performance is typically better in systems with regular configurations than in those with skew, unequal pier heights, and curved bridges.
- The equivalent static analysis method is suitable only for preliminary design. Final design moments and forces should be checked by spectral methods. AASHTO's LRFD code specifies *R* Factors, which are rounded off, that appear to be approximate. Since this factor scales down the design moments, future research should focus on arriving at a more accurate assessment of the R Factors.
- An independent code committee should verify the suitability/accuracy of each standard software program used in this type of analysis.

8.10 ALTERNATE METHODS OF SEISMIC ANALYSIS

The following sections discuss alternate methods of seismic analysis.

8.10.1 Pushover Analysis

Future bridge codes are likely to include pushover analysis (Ballard et al. 1997b), which estimates nonlinear behavior under lateral loads resulting from higher modes of vibration. So far, most applications of this method of analysis have been for buildings rather than for bridges. However, it has great potential for bridge seismic design and retrofit, particularly for critical long-span, segmental, and railroad bridges, given that normal engineering analysis practice assumes linear-elastic behavior for structural members, which fails to reliably account for redistribution of forces due to member nonlinear behavior and dissipation of energy due to material yielding.

Case Study: Carquinez Bridge in California Using Pushover Analysis

The Carquinez Bridge in California was completed in 1958. It has two 153-meter anchor spans, one 46-meter center tower span, and two 336-meter interior spans. Each interior span comprises two 102-meter cantilever spans simply supporting a 132-meter-long suspended span.

The state of California contracted with South Carolina Solutions to perform an analysis of a retrofit design for the bridge to evaluate important nonlinear behaviors for its structures. The purpose was to determine the structural response of the bridge steel members in a postulated seismic event.

Tower Collapse Analysis

A key issue in evaluating the behavior of the Carquinez Bridge was tower collapse, or "push-over," (Figure 8.9) (Ballard et al. 1997a). The analyses provided valuable insight to the structural characteristics of the as-built towers, as well as the effects of various retrofit alternatives on their displacement capacities. After identifying deficiencies in displacement capacity as well as vulnerability of key members to net section fracture and early local buckling, retrofit measures for the tower were investigated via subsequent pushover analyses.

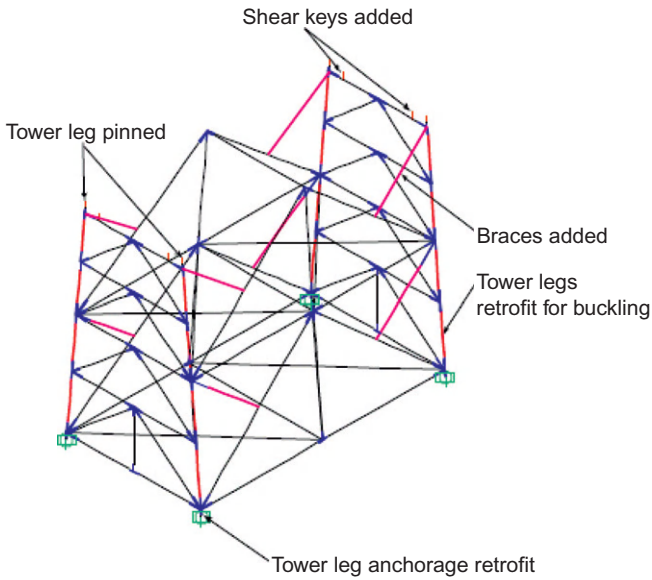

Figure 8.9 Final retrofit push model for the 136-foot tower of the Carquinez Strait Bridge.

In addition to the overall retrofit goals for the bridge, certain behavioral characteristics of the retrofitted towers were sought:

- Increased ductility of the tower members
- Ability of Tower 3 to resist all of the longitudinal seismic reactions of the superstructure
- Significantly increased displacement capacity for Tower 3 without increased force resistance

The pushover analyses demonstrated that the retrofit measures greatly improved displacement ductility. Most notably, displacement capacity in the longitudinal direction for Tower 3 increased over 650%. The results of this study demonstrated the usefulness of nonlinear pushover analysis for determining the most efficient retrofit details for steel bridge structures. It showed that a refined method such as pushover analysis is recommended for a more accurate understanding behavior of continuous span bridges utilizing integral piers.

Pushover Analysis of Integral Abutments and Integral Piers

Integral abutments and piers are not regular rigid-frame structures. Assumptions need to be made for partial fixity between beam end and pile cap or beam end and pier cap. Moreover, the length of buried pile acting as a column needs to be independently determined from soil-pile interactive analysis using L-pile or COMP62P software. A refined method such as pushover analysis is recommended for a more accurate behavior of continuous-span bridges utilizing integral piers (see Khan 2004).

8.10.2 OpenSees Software

The Pacific Earthquake Engineering Research Center (PEER) at the University of California, Berkeley, has developed a powerful finite element analysis software for system modeling. The original software Open System for Earthquake Engineering Simulation (Opensees) is developed by Mckenna, by Fenves and Fillipou.

PEER's modeling software is most suitable for important structures and is ideally poised to replace time-history analysis methods by allowing the creation of finite element applications for simulating the response of structural and geotechnical systems. *OpenSees* is a collection of modules for implementation of models and simulation procedures for structural and geotechnical engineering. Unique among seismic engineering software, *OpenSees* allows integration of models of structures and soils to interaction.

In addition to improved models for reinforced concrete structures, shallow and deep foundations, and liquefiable soils, *OpenSees* is designed to take advantage of the latest developments in databases, reliability methods, scientific visualization, and high-end computing.

The author served as a member of ASCE Seismic Effects Committee and Methods of Analysis Committee for six years, during which time alternate computational mechanics applications were often discussed. Professor Fillipou served as chair of the committee.

Several software programs such as *SAP 2000*, *STAAD-Pro*, *ANSIS*, *NASTRAN*, and *OpenSees* are available for system modeling for a large variety of bridges. One program widely used is *PennDOT LRFD*. *BridgePBEE* (Performance-Based Earthquake Engineering) analysis is the latest software approach in performance-based seismic design. The analytical options available in *BridgePBEE* include pushover analysis, base input acceleration analysis, and PBEE analysis.

8.11 CONCLUSIONS FOR SEISMIC BRIDGE DESIGN

Seismic design aspects are dependent upon local geology and seismicity. Familiarity with seismic terminology and conventional dynamic analysis is necessary, as are innovative methods such as performance-based seismic engineering (PBSE) and pushover analysis.

It is crucial that structural engineers be familiar with the latest AASHTO LRFD code, which addresses seismic hazard assessment, ductility, and detailing for bridges. Changes in load factors and resistance factors for seismic conditions will lead to more accurate design than earlier code. Also, new concepts, such as soil-related response spectra and liquefaction potential of weaker soils, must be mastered.

Structural engineers should be trained in structural dynamics, geotechnical engineering, computer software, and ductility detailing, as well as in shake-table technology as a way to predict the structural behavior of unusual bridges not covered by existing codes. Continuing education requirement by the licensing boards of many states, is a step in the right direction to hold short courses, lunch time or evening seminars and webinars for the benefit of introducing new technology.

AASHTO's code does not cover all aspects of seismic retrofit. A separate code for this aspect of seismic design, which includes technologies such as base isolation, is required as a supplement.

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PennDOT 1998
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CHAPTER NINE

Bridge Seismic Retrofit Methods for Seismic Events

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9.1 INTRODUCTION

Retrofits More Economical than Replacements:

In continuation of previous chapter for design of bridges, this chapter broadly reviews the need for seismic retrofit based on which it recommends further development of seismic analytical methods. Analytical methods are similar to that required for a replacement bridge. While repairs are an inherent part of maintenance, retrofits involve upgrades and deviation from the original design. Detailed seismic resistant specifications for retrofit and rehabilitation, on similar lines to designing new structures are required. Both 475 year and 2250 year events are typically considered.

An ATC review of bridge design criteria is also included.

A detailed literature review shows contributions by the following research organizations and those responsible for developing design codes:

Standard Specifications for Highway Bridges, Division1-A (AASHTO, 1991)

Earthquake-Resistant Structures
ISBN: 978-1-85617-501-2
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Seismic Hazard Methodology for Central and Eastern United States (EPRI, 1986)

1994 Seismic Retrofitting Manual for Highway Bridges (FHWA, 1995)

Guide Specifications for Seismic Isolation Design (1999)

The following authors have contributed to several aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to seismic design is presented here:

Bridge Security and Blast Resistance Design (Agrawal et al., 2004)

Utilizing Pushover Analysis for Seismic Performance of Steel Bridge Structures (Ballard et al., 1997)

Earthquake Engineering (Bartero, 1997)

Earthquake Engineering (Bozorgnia and Bertero, 2004)

Constructible and Economical Seismic Retrofits for Existing Bearings (Buchheit and Capers, 2005)

Performance Based Seismic Engineering:Past, Current and Future (Freeman, 2000)

Garden State Parkway Seismic Prioritization Program (New Jersey Turnpike Authority, 2002)

Guide Specifications for Seismic Isolation Design (1999) (New Jersey Turnpike Authority, 2002)

Seismic Design with Supplementary Energy Dissipation Devices (Hanson and Soong, 2001)

Bridge Design, Construction and Maintenance (Lark, 2007)

Liquefaction Assessment Methodologies for Seismic Bridge Design and Their Applications to New York City Projects (Nikolaou et al., 2003)

Design Guidelines for Assessment, Retrofit, and Repair of Bridges for Seismic Performance (Priestley et al., 1992)

SEISAB Software, Seismic Analysis of Bridges (Imbsen Associates, 1998)

Seismic Response and Evaluation of MSSS Bridges in New Jersey (Saadeghvaziri et al., 1999)

Bridge Engineering (Tonias, 1994)

Yen et al., Bridge Seismic Retrofit Manual and Its Applications in Missouri Highway Bridges

A Simple Pendulum Technique for Achieving Seismic Isolation (Zayas et al., 1990)

In addition to the above publications, relevant individual references are discussed in the text.

There are two types of existing bridges. Those designed according to seismic codes are newer and generally should survive a major earthquake. Those not designed according to seismic codes tend to be older but can be retrofitted. The retrofitting of bridge structures is primarily to prevent collapse in large seismic events. As a by-product of collapse prevention, an economic benefit can be realized because a retrofit bridge will experience limited damage. Also, it is much less expensive to replace a column or repair a hinge than it is to replace the entire structure.

Engineered bridges may have been designed on the basis of earlier seismic codes, which were less stringent. Such bridges are more vulnerable during earthquakes and may need improvised retrofitting not covered by current codes. Moreover, differences in the seismicity of the U.S. east and west coasts require different bridge retrofit techniques.

All bridges that are scheduled for rehabilitation are evaluated with regard to seismic failure vulnerability. The purpose of this evaluation is to assess seismic retrofit measures and to incorporate into the rehabilitation plans measures deemed warranted to eliminate or mitigate such failure vulnerability. The target is not to exceed 25% to 50% of the of bridge population to fix and simultaneously convert to seismic-resistance.

Routine non-seismic rehabilitation of superstructure and substructure for daily live loads and wind loads has priority over retrofits for probable earthquake load occurrence. Combining seismic rehabilitation with routine maintenance will reduce overhead costs. There may be regulations for earthquake insurance that may offset financial loss and costs to replace a bridge.

A feasibility report is prepared for review by all stakeholders prior to implementation.

9.2 RETROFIT PRIORITIZATION

Either a partial or a full retrofit may be necessary, and for reasons of maintenance and budget control, priorities need to be established. Prioritization requires a pilot study to select a diverse group of bridges as a sample population. This study is based on the following physical parameters:

- Traffic volume
- Detour length

- Bridge function (highway overpass, local, ramp, waterway, railroad)
- Structure vulnerability (substructure condition, seat length, rusting of bearings, etc.). Bearing lengths are defined in AASHTO (2007) Code or in the State Codes and computed based on the formulae.
- Seismic hazard assessment
- Soil conditions or potential for soil failure

In choosing bridges for retrofit, USGS seismic hazard maps, inspection reports, soil data, including boring logs, and contract documents are necessary for obtaining data on bridge geometry, age, and condition. Construction drawings provide important information on substructure vulnerability, such as number of girders and bearings; bearing types and anchor bolt arrangements; seat length vulnerability; and substructure details including wall pier, multiple column bents, and integral abutments.

The first priority in retrofitting is major bridges that are at risk. These can have multiple lanes or long spans, or they may carry heavy truck traffic. Some bridges are generally identified as more important than others, such as historic bridges and those on hospital, school, and national defense routes. Risk is calculated in one of two ways:

Consequences of damage or failure × probability of bridge damage or failure

Vulnerability rating × probability of damage or failure in remaining life of bridge

Since the same yardstick is used for all bridges, there is no relative error in using either method. This calculation considers the following factors:

- Seismic hazard levels (e.g., 500-year and 2,500-year return period)
- Liquefaction potential and site response
- Structure response (based on finite element models to identify critical elements)
- Maintenance and protection of traffic (MPT) during construction and alternates for detour

A bridge inspection for retrofit prioritization uses a seismic coding guide to rate a bridge's superstructure, substructure, components, and bearings, as well as the underlying soil.

Compute Ratings for Bridge Importance. Some bridges are identified as more important than others, such as historic bridges and those on hospital and school routes or located on defense routes.

Collect Seismic Vulnerability Data. Bridge inspectors inspect each bridge component, superstructure, substructure, bearings, and soil. A

database is created for seismic deficiencies and inventory data are fed into a computer program for relative ranking based on points allocation.

Compile Vulnerability Ratings. Total bridge vulnerability rating is based on individual component vulnerability rating either as the total or as the highest rating value. Aspects considered are magnitude of earthquake forces and capacity of components to resist lateral forces.

Prioritize Bridge Population. Risk is based on relative importance and vulnerability for each bridge.

Probability of Damage or Failure. The methodology is based on site location and seismic hazard, inherent soil vulnerability, and structural vulnerability.

Consequences of Damage or Failure. Consequences are determined from the functional importance of the bridge in the transportation network and on disruption to daily traffic.

Develop Conceptual Retrofit Measures and Associated Costs. Typical examples are fixing connections and isolation bearings, extending bearing seat length, column jacketing, and soil improvement.

9.2.1 Diagnostic Approach for Retrofit

Superstructure retrofits are generally recommended for live load maintenance (e.g., increased traffic and gravity loads). Seismic retrofit methods are recommended mainly for substructure components, foundations, and bearings, due to lateral seismic loads which may act at the bearings but would cause bending moments on substructure components due to the long lever arms to the foundation elevation which are discussed in the following paragraphs.

Foundations. During major earthquakes, the main cause of bridge failures is foundation damage. This occurs when the bearing capacity of the soil is exceeded and lateral spreading occurs. In moderate earthquakes, foundations usually perform well, but bridges over rivers may experience scouring of their foundations, causing settling.

Columns. Bridge columns designed prior to 1950 are under-reinforced. To ensure ductility, AASHTO 2007 recommends a 20% capacity increase over the original cracking moment specification. Japanese and European codes require a larger increase. However, most columns have large cross-sectional dimensions and perform elastically for the functional FEE hazard level.

Pier Caps. Hammerhead and pier bents act as deep beams due to closely spaced columns. The load path from bearings is dispersed into column tops. Anchorage of positive reinforcements may not be of adequate lengths, and transverse reinforcement lap splice may fail in the inelastic range for the SEE hazard level. On the Route 322-50 bridge in New Jersey, the author extended the height of the long concrete piles to directly frame into the cast-in-place caps. Pile cap was thus avoided, improving the frame action.

Bearings. Comparisons of benefits and costs offered by alternate retrofit schemes need to be performed. The appropriate seismic rehabilitation/retrofit measures are selected on merit as the best able to sustain new seismic loads. The following types are considered:

- Bearing
- Dampers, brace dampers
- Diagonal bracing with dampers
- Dampers with base isolation

Bearing Toppling

Seat Lengths: Bearing toppling is a concern with tall expansion rocker bearings. Conventional rocker bearings generally have adequate capacity for an FEE event but cannot resist the seismic shear demands from an SEE event.

At fixed bearings, forces transferred from girders are higher. Connections between girders and supporting bearings must be stronger than connections between girders and expansion bearings. The number of anchor bolts and their length and diameter should be adequate to resist seismic forces for an SEE hazard level. Comprehensive bearing retrofit may be required for the SEE level if plastic hinging could occur, and bearings must be capable of transmitting large lateral forces.

Seat length may not be adequate for an SPC C earthquake, and seat length extension may be required to prevent shear failure of bearings. SPC C (also designated as Zone 3) is based on geology and seismology of the area while FEE or SEE are based on recurrence of earthquake, whether a 475 year earthquake or a very rare 2250 year event. Seat extension should be made for the extreme case of 2250 year earthquake event and for the resulting seismic forces. Use of isolation bearings aids in an optimal distribution of lateral loads between piers. Isolation bearings permit a measure of force control compared to other retrofit measures.

The use of isolation bearings aids in an optimal distribution of lateral loads between piers. Isolation bearings permit a measure of force control compared to other retrofit measures.

The MCEER manual by Buckle, Constantinou, Dicleli, and Ghasemi, (MCEER, 2006, Seismic Isolation of Highway Bridges, SP07) presents the principles of isolation for bearings. It develops step-by-step methods of analysis, explains material and design issues for elastomeric and sliding isolators, and gives detailed examples of their application to standard highway bridges. The manual is a supplement to *Guide Specifications for Seismic Isolation Design* published by AASHTO in 1999. MCEER-06-SP07.

According to Washington State DOT memorandum dated September 2011, use of seismic isolation bearings is not recommended for conventional short and medium length bridges, or bridges with geometrical complexities. However, they may be used for seismic retrofit and bridge widening projects to reduce the demands of dynamic forces and as a viable alternative to strengthening weak elements or a non-ductile bridge substructure members. Isolation bearings and dampers are being used for very vulnerable structures or extremely important structures such as toll bridges. Seismic isolation bearings isolate a structure from the ground motion produced by an earthquake. They are designed to absorb seismic energy. Such devices currently undergoing evaluation are designed for both retrofit and new bridge design.

Developed after extensive tests at several research centers, fluid viscous dampers provide complete protection for bridges and elevated freeways that are subject to damage. Once the bearings fail, dampers isolate the structure from seismic energy and are used for important structures in which much energy can be dissipated with large movements, thus limiting extreme movements and preventing more damage.

After the removal of the existing bearing, the new bearing is slid into place and shimmed up tightly to the bottom of the girder. Nonshrink grout is then pumped under the bearing and allowed to cure prior to removing the jacking system. Most isolation devices require the top and bottom plates to be bolted to the bridge members.

Since movements can shift a bridge in any direction, universal joints are recommended on each end of the damping device to prevent the shaft from being bent during movements other than axial. The damper should be set to the bridge in its mid-stroke position or a position that takes full advantage of the free movements. The dampers are brought to the job

site a bit longer than the mounted length and the piston is compressed slowly until the pin can be set into position.

Friction pendulum seismic isolation bearings protect seismically isolated buildings, bridges, and industrial facilities. They are a specialty-engineered product that uses a sliding mechanism to control a supported structure's seismic response and to dissipate earthquake energy.

EPS's advanced, multi-stage Triple Pendulum bearings can be designed to protect not only the structure from damage during the most severe earthquakes, but also its contents and non-structural components.

Cross Bracing Between Girders. In older bridges axial forces are due to wind loads. During earthquakes, buckling may result. Since bracings serve as secondary members, their failure will not result in collapse.

ATC-18 Report on Seismic Design Criteria for Bridges

Review of Design Practice Worldwide: The overall scope of work on the ATC-18 project consisted of a review of current design practice and criteria, ongoing research in seismic design criteria development, and philosophies behind the design of seismic-resistant highway structures, including bridges, tunnels, abutments, retaining structures, and foundations. Prepared as part of NCEER Project 112 on new highway construction, ATC-13 (ATC 1985) considered bridges, tunnels, abutments, retaining wall structures, and foundations.

The report reviews current domestic and foreign design practice, philosophy, and criteria, and recommends future directions for code development. This involved a review of existing U.S. standards along with the latest codes for Japan, New Zealand, and Europe. Guidelines developed for the transportation corridor agencies for Orange County, California, and in-progress work, such as the new AASHTO *Load and Resistance Factor Design (LRFD) Bridge Specifications* were also reviewed.

Issues addressed in the ATC-18 report include performance criteria, importance classification, definitions of seismic hazard for areas where damaging earthquakes have longer return periods, design ground motion, duration effects, site effects, structural response modification factors, ductility demand, design procedures, foundation and abutment modeling, soil–structure interaction, seat widths, joint details, and detailing reinforced-concrete for limited ductility in areas with low-to-moderate seismic activity.

Two-level Hazard Design Approach: The ATC-18 report concludes that a two-level design approach should be included in future codes, as a minimum, for important bridges in the higher seismic zones. The report

provides a detailed discussion on the advantages and disadvantages of a two-level design approach. It also provides lengthy discussion on future directions for code development and recommended research and development topics.

Publication ATC-32 (ATC 1996): The report was sponsored by Caltrans and provides recommended revisions to the current Caltrans Bridge Design Specifications (BDS). The issues addressed are

- Seismic loading
- Structural response analysis
- Component design

Special attention is given to issues related to reinforced-concrete components, steel components, foundations, and conventional bearings. Research was conducted in the field of bridge seismic design and the performance of Caltrans-designed bridges in the 1989 Loma Prieta and other California earthquakes prior to 1996. These reports served as a preliminary study for proposed codes NCHRP/MCEER Project 12-49 and NCHRP Report 472 and were made a part of latest AASHTO LRFD (2007).

Detailing of reinforced concrete for limited ductility in areas with low-to-moderate seismic activity was addressed in the ATC-18 report.

For important bridges in the higher seismic zones the ATC-18 report recommends that a two-level design approach be included in future codes. Standard details for ductility will be applied.

9.3 IMPROVING SEISMIC-RESISTANT SYSTEMS (17), (18)

A 3-D computer model is shown in Figure 9.1 using finite elements. Simpler computer models are permitted by AASHTO LRFD Chapter 4. Figure 9.2 shows primary flexure and torsional modes. The subject covers the following relevant aspects presented in bullet list:

- Basic terminology
- Definition of seismic hazards
- Deterministic and probabilistic seismic hazard analysis
- Selection of seismic zone
- Pseudo-velocity and acceleration
- Concepts of quasi-static, response spectrum, and time-history analysis
- Deformation response spectra

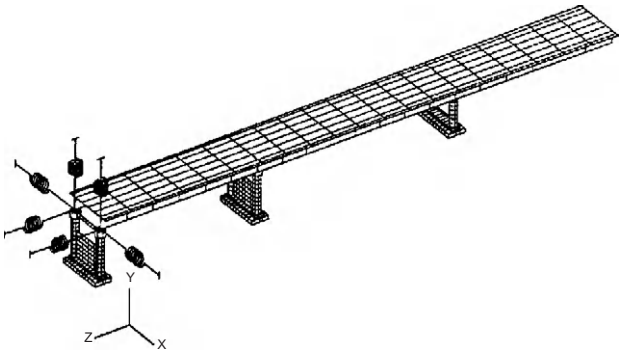


Figure 9.1 Finite element analysis of a 3-span bridge with fixed bearing at left and expansion bearing at right abutment.

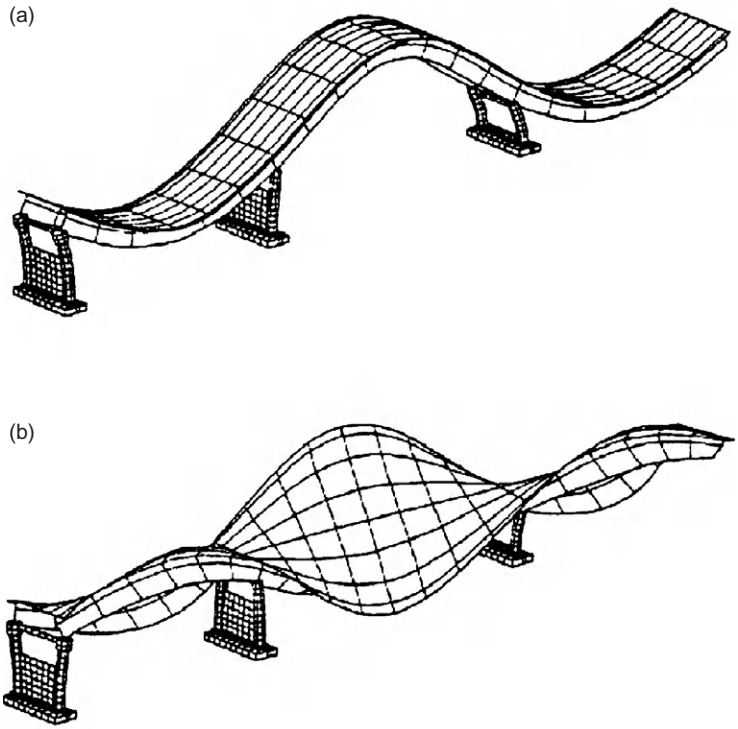


Figure 9.2 (a) Slab-beam bridge subjected to flexure and (b) torsion.

9.3.1 Retrofit Schemes Based on Seismic Zones

In California, seismic retrofit for bridges follows the Federal Highway Administration (FHWA) Seismic Retrofitting Manual for Highway Bridges (FHWA 1995), which is based on applicable seismic zones. These zones require different approaches to seismic rehabilitation. Seismic activity in Zones 1 and 2 is usually not severe. Analytical requirements and seismic details are accordingly lenient, and seismic load effects are not likely to supersede those of other load combinations. Seismic activity in Zones 3 and 4 can be severe, and analytical requirements and seismic details are accordingly stricter.

For some designs, even where the acceleration coefficient is low, seismic loads can supersede other load cases. For example, a long continuous superstructure may be supported on a system of sliding bearings at all piers except over one stiff pier, where fixed bearings are used. The fixed bearings in such cases must be designed to transmit most of the calculated seismic forces.

9.3.2 Subsurface Exploration for Substructure Seismic Retrofits

Developing Site Specific Response Spectra: Subsurface exploration, which entails soil borings, shear wave velocity measurements, and laboratory testing, is necessary to determine substructure conditions, to obtain data for site response analyses, to evaluate liquefaction potential, and to ascertain foundation flexibility. As part of a subsurface exploration program, deterministic analysis estimates ground motions based on an assumed earthquake magnitude and a known distance from a fault. Such analyses can provide general guidelines for modeling design ground motions. Earthquake magnitudes selected are greater than the maximum observed magnitudes given in the historical record. Probabilistic analyses are typically used in the northeast United States because seismic risk is generally lower in this region.

Based on existing subsurface information and the results of a subsurface exploration, site response analyses are performed to establish site-specific response spectra for each hazard level. The bedrock time histories used in site response analyses are the recommended time histories developed. A detailed assessment is performed to determine the effects of soil–structure interaction of pile bents and the soil supporting them. Response spectral analyses should be augmented by lateral pushover

analysis to determine the ultimate strength of the substructure, which should account for local yielding of foundation elements as well as non-linear soil response in pile bents.

A vulnerability assessment may require a dynamic response evaluation using a spectrum synthesized from response spectra developed for each of the bridge's soil profiles.

Influence of Soil Conditions on Seismic Performance of Bridges

Liquefaction and Site Effects: Two major impacts during shaking are soil liquefaction (i.e., the reduction in soil strength due to buildup of pore pressure) and site effects or modification of earthquake-induced bedrock motions. Estimates of ground motions are primarily a function of rate of earthquake occurrence and rate of attenuation of ground motions away from the epicenter. They require the following steps:

Step 1: Review the seismicity of the area and select seismic hazard levels.

Step 2: Obtain probabilistic and deterministic ground motion analysis results. Select a bedrock acceleration response spectrum and develop acceleration time histories consistent with it.

Step 3: Estimate key soil parameters such as unit weight and shear wave velocity. Develop representative soil profiles.

Step 4: Obtain acceleration response spectra at the bridge foundation site via site response analyses for soil profiles. USGS national seismic hazard maps may be used.

Step 5: Obtain seismic vulnerability from the spectra.

9.3.3 Factors Influencing Structural Behavior

Structural factors influencing performance include following bullet list:

- Condition of damaged and deteriorated structures
- Proportions/sizing/mass distribution
- Performance characteristics of lateral load—resisting systems
- Types of foundations and soil—structure systems to resist ground motions
- Span arrangement and intermediate pier-bearing selection (where applicable)
- Energy-dissipating devices
- Base isolation.

9.3.4 Modes of Failure Resulting from Earthquakes

From the studies of bridge failures it is seen that following modes of failure are likely to occur:

- Settlement of bridge approaches
- Ground failure due to liquefaction or excessive soil deformation
- Shear cracking of column bents
- Large skews and unusual geometries
- Cracking or spalling of concrete abutments and concrete column covers
- Displacements of steel or neoprene bearings

In a preliminary assessment, abutments are not evaluated because the potential for liquefaction and lateral spreading and for interaction between the abutment and the soil is neglected at this stage. Foundations are modeled as infinitely rigid for translation and rotation.

FEE and SEE Hazard Levels: A bridge's seismic response is dominated by fundamental dynamic modes, which are on the order of 1 to 2 seconds. Single-mode analysis is generally applicable to short- or moderate-span bridges. Multi-mode analysis is applicable to longer spans that are more flexible. Higher modes contribute to overall response.

A similar two level hazard approach as used for the design of nuclear power plants is adopted for bridges. The nuclear plant is kept functional after the assumed level of earthquake (known as operating based earthquake).

The alternate is to design for a very high level assumed earthquake. The cost of construction increases but not all components need to be designed for the extreme event and the plant is shut down until repairs to some damaged secondary components are made. This level of design is known as safe shutdown earthquake. Since bridges are also very important for going to work or hospital or school, the first seismic hazard level is also known as a functional evaluation earthquake (FEE). A 10% probability of exceedance in 50 years (475-year return period) is assumed. FEE represents low earthquake shaking. For this hazard level the structure should be retrofitted so that little or no damage occurs, according FHWA retrofitting guidelines and AASHTO 2007 requirements. The second seismic hazard level, a safety evaluation earthquake (SEE), expects the structure to sustain limited damage but not fail. A 2% probability of exceedance in 50 years (2,250-year return period) is assumed.

Relative Costs of Design Levels: For older bridges, FEE retrofit costs can range from $30,000 to $900,000. However, SEE retrofit costs may be many times higher than FEE retrofit costs. Based on vulnerability assessment, to limit the potential for damage, upgrading the seismic performance is strongly recommended.

9.3.5 Methods of Structural Analysis

Three-dimensional structural models allowing for six degrees of freedom need to be developed to perform multi-mode response spectral analysis using *SEISAB*, *GT-STRUDL*, SAP 2000, ADINA or *STAAD-Pro* software and other approved analysis software. The parameters for seismic category, response modification factors, site coefficients, and methods of analysis currently being used, appear to be semi-empirical.

Most substructure response modification factors (R) have decreased in the LRFD code, resulting in higher design moments and forces, since design moment or force = moment or force from seismic analysis/R. Designs based on LRFD code result in increased member size or in reinforcement steel.

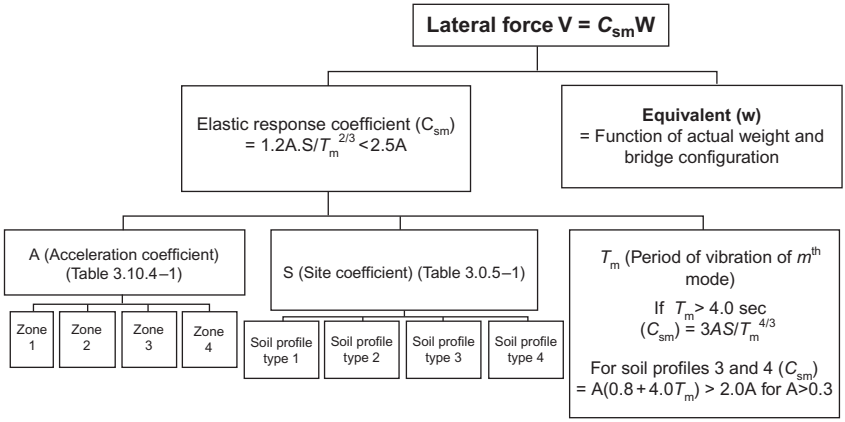
Zones Classification: LRFD code reclassified sites as Seismic Zones 1 through 4 in place of A through D in the old LFD code, but there is no change in the range or magnitude of acceleration coefficients. The studies show that in current codes sufficient attention has not been paid to ductility or seismic detailing. The design needs to be performance-based.

Figure 9.3 shows a method of calculation for lateral loads V acting on a substructure from weight of superstructure and live loads based on AASHTO LRFD specifications. Figure 9.4 shows a flow diagram for standard methods of seismic analysis for bridges. The procedure is common for both new bridges and retrofit of existing bridges.

A solved example using a single-mode spectral analysis method is given in AASHTO *Standard Specifications* (Supplement A of *Seismic Design Commentary to Section 4, Analysis and Design Requirements*). A plot from *SEISAB* software is given in Figure 9.5 for a 2-span bridge with a transverse cross frame comprising multiple columns. Load Case 2 is for longitudinal forces.

Suggested Procedure

The preliminary assessment phase involves linear spectral analysis. For bridges of limited ductility capacity (built prior to 1980s), neglecting nonlinear behavior is acceptable since large displacement effects cannot



Note: Seismic Moments and forces obtained from analysis shall be devided by Response Modification Factor (*R*).

Figure 9.3 Method of calculation of lateral loads *V* acting on a substructure.

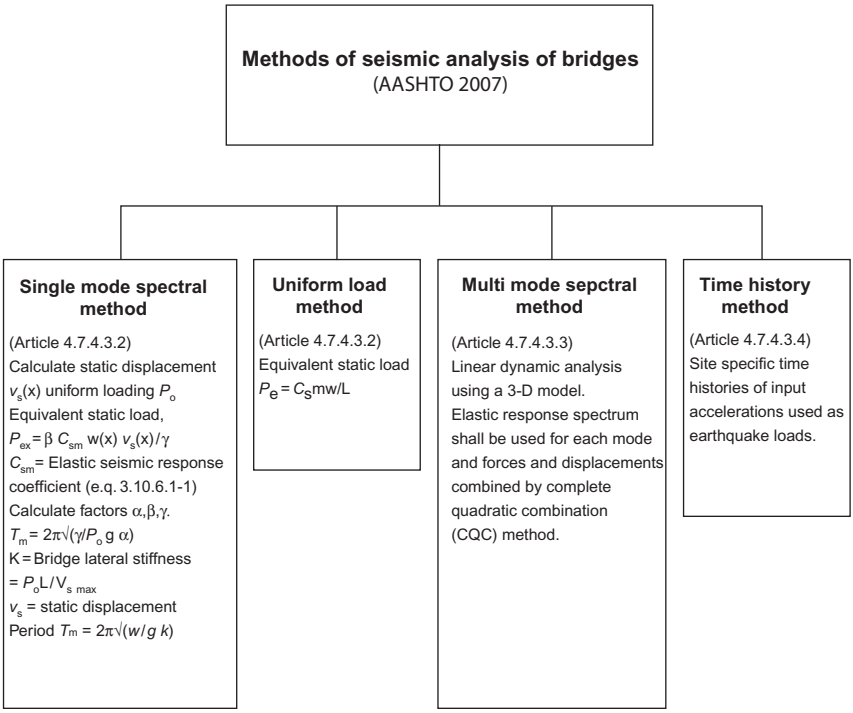


Figure 9.4 Flow diagram for standard methods of analysis for bridge design and retrofit.

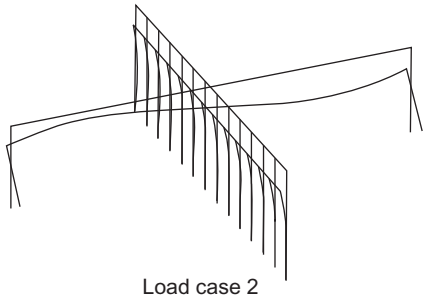


Figure 9.5 Graphic plot of a single-line idealization of a bridge with a single transverse frame using SEISAB.

be directly assessed. Nonlinearities can be incorporated by modifying the response spectral input to better reflect them. Differences in motion at different supports (multi-support excitation) are not considered. An alternative to linear spectral analysis is time-history analysis. AASTO notations are assumed for mathematical equations.

For detailed assessment, a response spectral analysis can be supplemented by a pushover analysis for ultimate strength evaluation of substructure units and to take into account nonlinear soil response and yielding of foundation elements.

An Innovative Retrofit in California

Example: The retrofitted 354-foot Spanish Creek Bridge (Figure 9.6) is one of the longest conventionally reinforced concrete spans in the state of California, with a carrying capacity of 360 kips that quadruples the capacity of the old bridge. Funded primarily through the federal American Recovery and Reinvestment Act of 2009, the retrofitted bridge replaces one built in 1932 during the Great Depression as a Works Progress Administration project. It connects Quincy, the Plumas County seat, to state Highway 70 through the Feather River Canyon.

Using the software program *STAAD-Pro*, a model of the bridge was generated and then analyzed with two types of frames (flat roof and sloped roof), each with three different configurations. Two different methods, equivalent static analysis and dynamic analysis, were used. Also, comparisons were made of the density of lightweight concrete and regular concrete since lateral forces are dependent upon the basic density of materials and overall mass.

Geometry and Data: In equivalent static analysis, the frame's span was 20 ft wide and 15 ft high; the dead load was approximately 1 kips/ft and the

live load was about 0.3 kips/ft. The lateral force caused by seismic activity was taken to be 0.9 kips.

The sloped roof was provided with a two-pinned connection. At 12 ft high it made a slope at an angle of 30 degrees, which formed the roof. The pins were positioned 5.83 ft from the point of inclination on both sides. The base of the frame was fixed.

The compressive strength of concrete was taken to be 3,000 psi; the yield strength of steel, 60,000 psi. The final size of the beam chosen was 12 by 24 in., and the column size was 12 × 14 in. for the loads applied.

The advantage of the slope roof was its arch effect during deflection, which would not heavily compromise the look of the structure. The flat roof had a higher deflection. The slope roof would be good for rain and snowfall since it allows rain and snow to run off and reduces the amount of weight the roof must carry.

Figure 9.6 Long-span concrete arch Spanish Creek Bridge in California.

Update of New Jersey Department of Transportation Manual for Seismic Retrofit of Bridges

Case Study: The author was assigned the task of upgrading Section 45 of *NJDOT Design Manual for Bridges*, Section 1.410.3, "Seismic retrofit of existing highway structures," which addresses important issues related to seismic retrofit of and modifications to existing bearings.

The minimum seismic performance zone as indicated in AASHTO Specifications Table 3.10.4-1 shall be SPC B for the entire State of New Jersey.

The AASHTO LRFD Bridge Design Specifications apply to bridge spans not greater than 150 meters. For Importance categories, definitions of Critical, Essential or Other bridges apply. A seismic design is usually not required for buried structures or culvert structures.

Two-hazard Levels Considered for New Jersey: Critical bridges must remain open to all traffic after the design earthquake of a 2,500-year return period event. Essential bridges must be open to emergency vehicles and for security/defense purposes immediately after the design earthquake of a 475-year return period event. Bridges that are designed as critical require the approval for that classification.

A percentage of live load may be considered in computing seismic forces, depending on the importance of the bridge.

Seismic ductility design must be provided at the locations where plastic hinges form on all new structures.

New Jersey seismicity is shown in Figure 9.7. Locations of past earthquakes are shown in Figure 9.8.

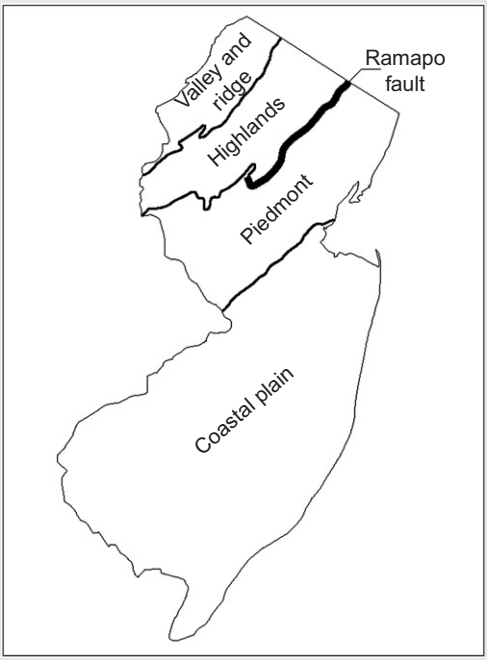


Figure 9.7 Seismicity of New Jersey.

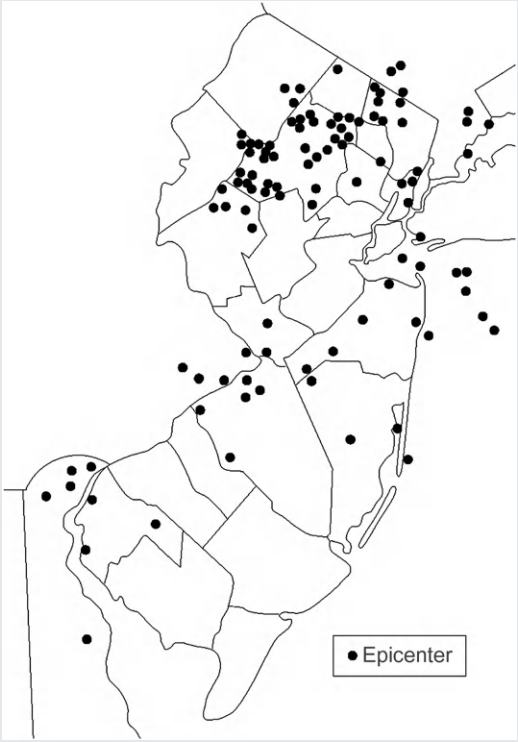


Figure 9.8 Location of epicenters of major earthquakes in New Jersey.

Acceleration coefficients for 21 counties are tabulated (Table 9.1). Acceleration coefficients varied between 0.10 and 0.18, with the higher values for northern New Jersey.

The author developed the sections of *NJDOT Bridge Design Manual* for seismic effects and bridge scour (NJDOT 2003) and supervised senior design projects on earthquake engineering.

9.4 PREPARING SEISMIC RETROFIT FEASIBILITY REPORTS

9.4.1 Field Reconnaissance Reports

A field reconnaissance report documents the performance of structures designed according to AASHTO specifications and assesses the adequacy

Table 9.1 Acceleration Coefficients for New Jersey Counties

Acceleration	County	Location Coefficient
$A = 0.10$	Atlantic, Cape May, Cumberland, Salem	South
$A = 0.15$	Burlington, Camden, Gloucester, Monmouth, Ocean	South and east
$A = 0.18$	Bergen, Essex, Hudson, Hunterdon, Mercer, Middlesex, Morris, Passaic, Somerset, Sussex, Union, Warren	North and central

of the standards used at the time of construction. For example, *Damage to the Highway System from the Pisco, Perú Earthquake of August 15, 2007*, by O'Connor, Mesa, and Nykamp, is the product of a field investigation undertaken in September 2007 after a magnitude 8.0 earthquake. It provides a brief description of the event and the consequential damage to the highway system.

9.4.2 Design Office Procedures

Developing a Retrofit Feasibility Report: A seismic retrofit report provides a determination of a bridge structure's eligibility for a seismic retrofit. It is based on prioritization criteria and cost and must justify a retrofit on the basis of safety and preparedness against a disaster during the structure's remaining life. The planned work is carried out in conjunction with major repairs to the superstructure, such as deck replacement, superstructure rehabilitation, and repairs to bearing seat areas and rusted or malfunctioning bearings. The bridge must be of high importance, and its proposed repair must be cost-effective; also, the feasibility of simultaneous repairs to adjoining bridges in the highway system must be determined.

Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges, by Ho et al., presents the state of the practice as of 2005 for retrofitting steel truss bridges in the United States. Using a performance-based seismic retrofit philosophy, the guidelines cover all major aspects pertinent to seismic retrofitting with a focus on superstructures. The guidelines supplement the 2006 FHWA publication *Seismic Retrofitting Manual for Highway Structures*, which is applicable to "unusual" or "long-span" steel trusses.

9.5 APPLICABLE RETROFIT DESIGN FOR EXISTING BRIDGES

Practical Considerations: All bridges need to be checked for retrofit except those that are due for replacement. Examples of the former include those with

- Longer spans generally in excess of 150 ft
- Continuous steel or concrete girders
- Steel or concrete box girders
- A sharp curve radius generally less than 1,000 ft
- Concrete segmental steel cable supports or complex or unusual geometry
- Connections between the superstructure and substructure that need to be designed for seismic forces

The single-mode spectral method, or any higher-level analysis, is used to calculate the seismic forces to be resisted by the bearings or other anchorage devices. The connections between the superstructure and substructure need to be designed for seismic forces.

9.5.1 Bearings Retrofit

Bearings are the Weakest Components: Several methods of seismic retrofit are outlined for bearings and expansion joints within the FHWA *Retrofit Manual*. The methods are recommended for consideration in order of preference. If applicable, a recommendation as to the proposed treatment of a bridge structure should be included in the Seismic Retrofit Report.

- Modify existing bearings to resist seismic loads or to prevent toppling of existing bearings by installing longitudinal displacement stoppers.
- Use longitudinal joint restraints (outlined in Subsection 5.2.1, FHWA *Retrofit Manual*).
- Increase size, number, or embedment of anchor bolts.
- Increase outer diameter of the pin head.
- Increase the width of the expansion rocker. Increase top and bottom dimension of the pintle detail for increased movement.
- Use expansion joint restrainer.
- Use pin piles for increasing structure—soil interaction (Figure 9.9).

Details of Bearings Retrofit

During an earthquake, bearings are subjected to displacements, rotation, and lateral forces in various directions, resulting in brittle failure of the

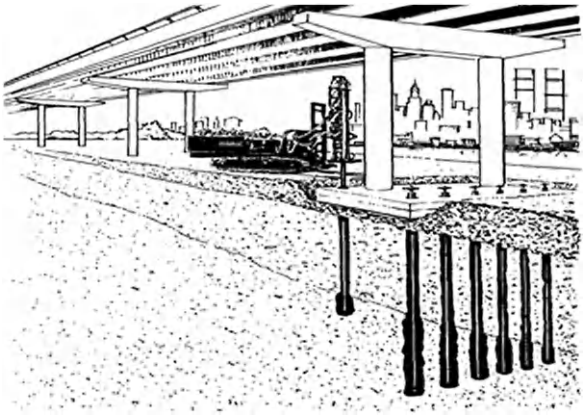


Figure 9.9 Footing retrofit using pin piles.

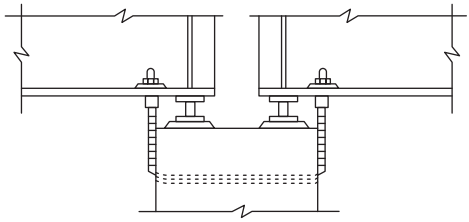
unidirectional steel high rocker and low-sliding bearings. Replacing these bearings with ductile, multi-rotational, and multi-directional bearings provides safety against potential unseating of the superstructure.

Old Technology Exists: The expansion rocker bearings or pin/hinge-type bearings located on bridge abutments and piers have been known to perform poorly due to bearing toppling and out-of-plane rotation. A solution is either to provide cable restrainers (Figure 9.10) or to use a catcher system that is designed to prevent loss of elevation based on bearing toppling. A built-up assembly of welded steel plates is attached to the girder or the bridge seat. The assembly allows normal movement, but supports the girder in the event of excessive longitudinal movement.

Several methods of seismic retrofit are outlined for bearings and expansion joints in the FHWA *Retrofit Manual*. Of these methods the following are recommended for consideration in order of preference. If applicable, a recommendation as to the proposed treatment of a bridge structure should be included in the Seismic Retrofit Report with cost evaluations and comparisons. A bullet list is presented here:

- Modify existing bearings to resist seismic loads or to prevent toppling of existing bearings by installing longitudinal displacement stoppers.
- Use longitudinal joint restraints as outlined in Subsection 5.2.1 of FHWA *Retrofit Manual*.

  Modifications to existing elastomeric bearings include
- Secure bearing against movements in any direction.
- Modify pad thickness.
- Modify plan area.



Cable restrainers are only to be used with the approval of the chief bridge engineer

Figure 9.10 Typical cable restrainer for uplift.

- Reduce reactions by *R* factors in lower seismic zones.
  Modifications to steel bearings include
- Increase expansion rocker width.
- Increase size, number, or embedment of anchor bolts.
- Increase pin head diameter.
- Increase top and bottom dimension of pintle detail for increased movement during an earthquake.

Bearing Types

A bearing is ideally required to provide six degrees of freedom. In recent years considerable progress has been made in the mechanical design of the fixed and moving components. Bearing types include

- No bearing required (integral or continuous frame-type bridge)
- Modern isolation bearings AASHTO 2010, Guide Specifications
- Disk-type bearings

Rocker and Roller Bearings. Readjust all rocker and roller bearings to restore their required function. Clean, paint, and lubricate (roller bearings only) as warranted. For deck replacement or other major bridge rehabilitation projects, rocker bearings should be replaced and roller bearings should be considered for replacement unless seismic criteria are met.

Multi-Rotational Pot Bearings. The neoprene material is adequately contained in the pot, and the gap between the top of the pot and the piston-bearing plate is fairly uniform under dead load. Also, sufficient end distance should exist to the stainless steel plate (mirror plate) for expansion and contraction at extreme temperatures.

Unguided and Guided Bearings. Unguided bearings can slide in any direction and are in use in Zone 2 seismic movements.

Elastomeric Bearings (Stanton 1990). Secure the bearing against horizontal and vertical movement. Modify the plan area and/or thickness of the elastomeric bearing to reduce seismic forces to the substructure.

Neoprene Bearings. Problems such as major uneven deformation or walk-out must be corrected.

Bearings for Temporary Construction. Appropriate bearing types and restraining connections are designed to endure construction and traffic loads.

Other Metal Bearings. Restore the required function of these bearings, as warranted, by repairing or replacing worn-out parts. The types of bearings are as follows:

- Sliding plate
- Pin
- Spherical
- Rocker plate

9.5.2 Bearing Retrofit Alternatives

Old Fashioned Bearings: Maintenance has been a real drawback in keeping the older bearing types functional. The vulnerability of rocker and other bearings in a seismic event has become obvious. Replacement of rocker bearings in regions of moderate seismicity is recommended in current retrofitting guidelines and state standards. In addition, the resetting of rocker bearings represents a rather substantial maintenance cost. From both seismic and life-cycle cost perspectives, rocker bearing replacement is warranted for more vulnerable and important bridges.

Extend Bearing Seats. Bearing seat lengths must meet minimum support lengths as per design specifications. This must be addressed on rehabilitation projects.

Provide Cribbing for Vulnerable Bearings. While it is desirable to eliminate vulnerable bearings (i.e., rocker and roller bearings), this is not always possible or cost-effective. Cribbing is used as a temporary measure until an economical bearing replacement can be performed (i.e., during a deck or bridge replacement).

Add Shear Blocks and/or Pedestals. Structures that are deficient in areas such as seat length and bearing instability, or that have inadequate connections between superstructure and substructure, may be retrofitted by the addition of shear blocks and/or dowel bars, or by

construction of concrete pedestals that act as shear blocks and alleviate bearing instability.

During rehabilitation work, existing structures are jacked to remove the existing steel rocker or steel sliding bearings. Due to the height differences between the elastomeric bearings and the existing rocker bearings, existing pedestals are built up to a higher elevation, as recommended in the *Seismic Retrofitting Manual for Highway Bridges* (FHWA RD-94) (Figure 9.11).

Recent earthquakes, including Kobe, indicate that the failure of the steel bearings causes substantial damage to the superstructure. Depending on the capacity of the existing substructure, it is prudent to investigate seismic demand on the substructure. When warranted, the seismic demand can be reduced by adjusting the bearing configurations in one of the following manners:

- Options Available: Providing all expansion bearings with transverse restraints, thus reducing the transverse force demand by distributing it to all the substructures through expansion and fixed bearings. Providing conventional laminated elastomeric bearings at the expansion supports and using a lead core base isolation bearing at the fixed support will reduce the seismic demand.
- Providing guided pot bearings at the expansion supports and a lead core base isolation bearing at the fixed support: Since the coefficient

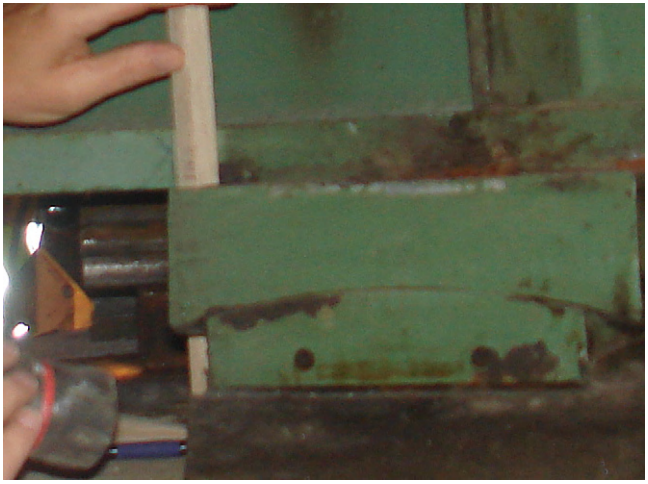


Figure 9.11 Height of existing bearing being measured by author for replacement.

of friction for expansion pot bearings is less than for elastomeric bearings, it will further cut down the demand on the substructure.

- Providing a fixed bearing at one of the abutments and expansion bearings at the pier and the other abutment, which will help to reduce the seismic demand at the pier.

Alternate methods include

- Replacing high rocker and roller bearings.
- Extending bearing seats: Bearing seat lengths must meet the minimum support lengths as per the design specifications. This must be addressed on rehabilitation projects. Seat extensions should be provided.
- Evaluating seismic loadings resisted by bearings and their connections in accordance with AASHTO LRFD Specification and Subsection 2.2.10.

Elastomeric bearings are preferred where their use is practical. These are least expensive and easier to maintain. Selection will depend upon the magnitude of reactions from dead and live loads etc. and from seismic forces. Other commonly used types include Pot, disc, sliding, seismic isolation, and other alternative bearing types should be evaluated on a case by case basis.

According to Article 12.3 of the Guide Specifications for Seismic Isolation Design, bearing stability must be evaluated with a safety factor of 3; while Article 14.10.10.3.6 of the AASHTO Standard Specifications states the method for the computation. For reinforced elastomeric bearings, the bonded dimensions, instead of the gross dimensions, are used in stability computations. Bearing stability is evaluated using the lower bound shear modulus (not the average). Performance measures for the bearings are presented on contract drawings and specifications.

9.6 RETROFIT OF SIMPLE MULTI-SPAN BRIDGES

Uniform Distribution of Seismic Forces: By providing continuity coupled with replacement of bearings, the lateral resistance of a superstructure will be enhanced and seismic loads will be distributed among all the substructure elements. The length of superstructure that can be made continuous is a function of thermal movement.

When connecting the unrestrained ends of adjacent girder spans, it is important to provide a complete splice between the flanges and the webs. Similarly, shear blocks should be designed to resist the movement of the superstructure beyond the anticipated thermal movement. A detailed

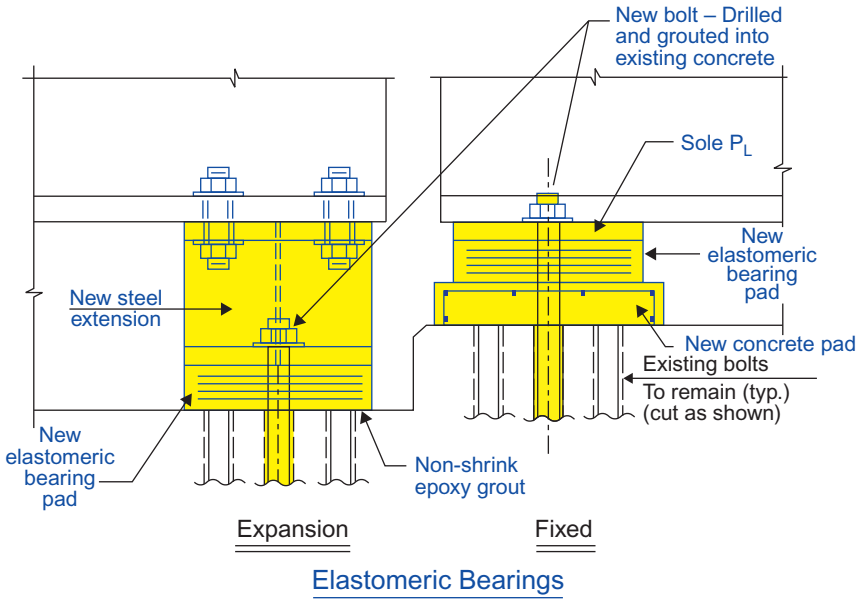


Figure 9.12 Elastomeric bearing retrofit.

method is provided in the FHWA manual for designing the restrainers and shear blocks.

Connections. All connections and anchor bolts should be designed for a minimum lateral force of 19% of the dead load plus live load reactions at the support. Anchor bolts should be anchored into the bridge seat to resist uplift. Figures 9.12 through 9.15 show alternate options to select for retrofit of bearings.

9.7 SUBSTRUCTURE DETAILING AND RETROFIT

9.7.1 Columns and Casings

Besides the bearings, the most seismically vulnerable aspect of a bridge is the column. Columns hold the structure up, and if they shear or are weak in flexure, a catastrophic failure is possible. When columns are to be replaced, the new seismic details in Figures 9.16 and 9.17 are recommended. Since bearings and columns are the most vulnerable to ground motion, alternate retrofits such as in-fill walls, catcher blocks, and foundation work are available.

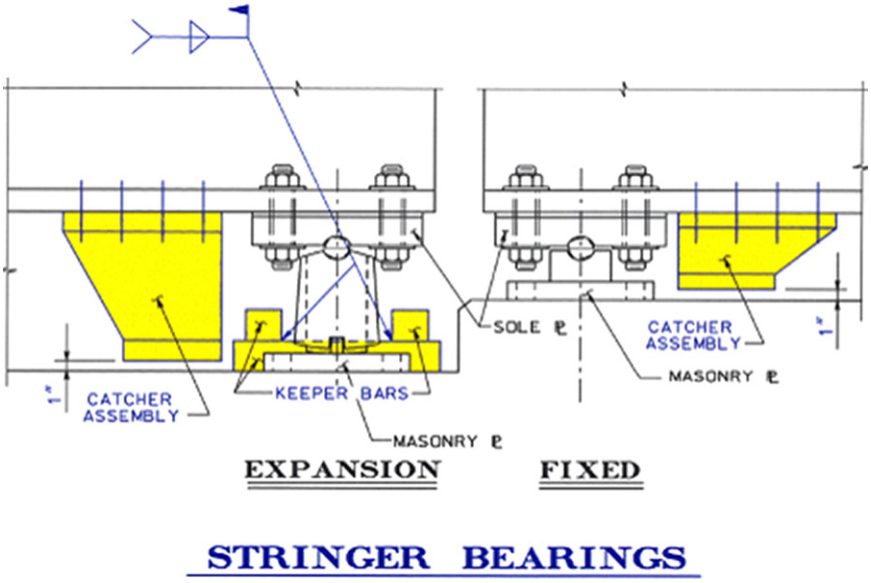


Figure 9.13 Stringer bearing retrofit.

Composites are now in use as an alternative to steel casings in limited situations and show much promise in longevity and ease of installation. The use of composites is a good tool if only the tops of the columns need retrofit because it is glued to the column. For a historic structure composites can also be used where any change to the shape of the members is undesirable.

Columns are retrofit utilizing welded steel casings ranging from 1/4 in. to 5/8 in. in thickness (see Figure 9.18). Steel casings are made circular or as close to circular as practical using an elliptical shape for rectangular columns. The shells are pre-manufactured in two or more sections and then field-welded to encircle the column. For circular casings, a minimum 1-in. gap is left all around and the void is filled with grout. Grout is pumped from the bottom up to guarantee that no air gaps are created. Injection ports for the pump need to be spaced vertically depending on the capacity of the pump used.

9.7.2 In-Fill and Strut Walls

An effective and relatively inexpensive means (cheaper than casings) of retrofitting multi-column bents for column shear are in-fill walls (Hipley

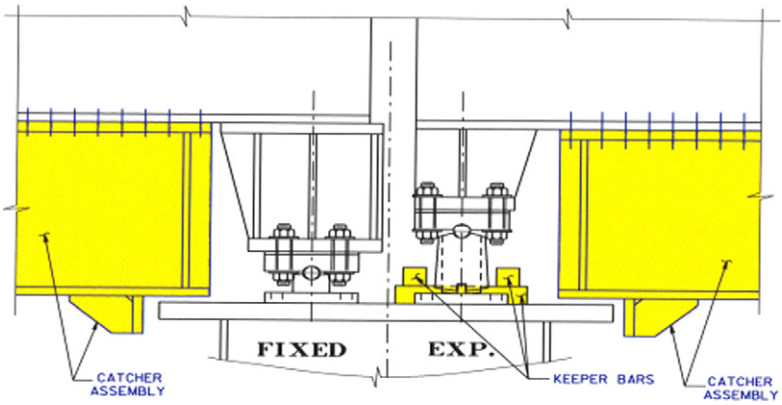


Figure 9.14 Stringer bearing and cross-girder retrofit.

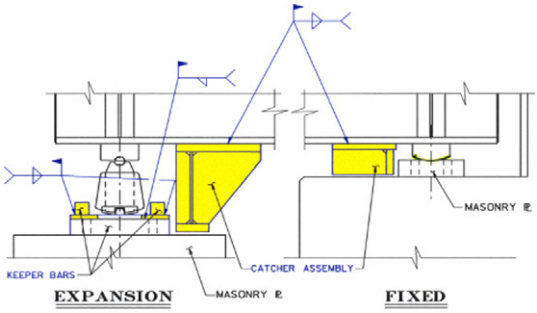


Figure 9.15 Box girder bearing retrofit.

1997). Strut walls are used for pedestrian over-crossings. They create stability, mostly transversely, but add some longitudinal resistance as well.

The wall is sloped from the superstructure to the footing in which new piles or spread footings are used to create a base for the walls. Rows of dowels are placed on the column faces and the existing footing top to create a monolithic system.

9.7.3 Recommended Abutment Reinforcement

Absolutely Minimum Reinforcement for Seismic Resistance: The vertical compression reinforcement of all abutment stems and walls must be doweled into the footing with minimum 5/8-in.-diameter bars. These dowels should have 180-deg hooks on both ends. Minimum reinforcement is 5/8-in. bars at 18-in. centers or less.

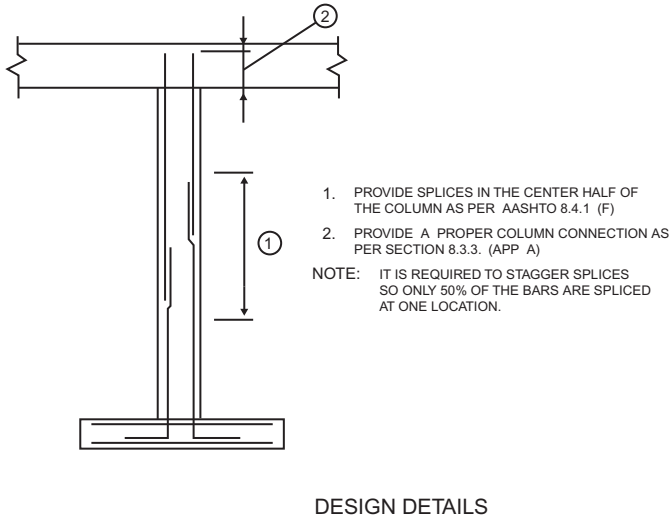


Figure 9.16 Seismic column detailing.

The recommended top layer of bridge seat reinforcement for steel girders, prestressed-concrete I-beams, and spread prestressed concrete box beams is 1-in.-diameter bars. For adjacent prestressed concrete box and slab unit structures, the top layer of bridge seat reinforcement is bars at 8-in. centers. The minimum vertical reinforcement is bars at 18-in. centers. The entire capacity of these bars is developed by embedment or lapping.

Sequence of Retrofit Work: The first order of work is to excavate to the footing top and around the sides if piles are to be added. A better method is to remove the entire edge of the footing vertically as recommended by researchers. This can be done by placing expansive grout in a line of drilled holes or hydraulic rock wedges. This concrete debris is removed.

Pile work will be the next order of work. The pile type used will reflect the most practical situation for the soil conditions. In many cases driven piles are selected for low overhead situations that may be encountered due to the height of the bridge. All piles should be positively anchored to the new footing to create uplift resistance. The bottom mat reinforcement (if widened) can be extended by welding or the use of mechanical couplers.

By having a vertical face on the footing, the mechanical crimping device can be lowered from the footing top directly. Also, if the bars are welded, the X-ray machine is easier to manipulate into position with a clear vertical footing edge to verify a proper weld.

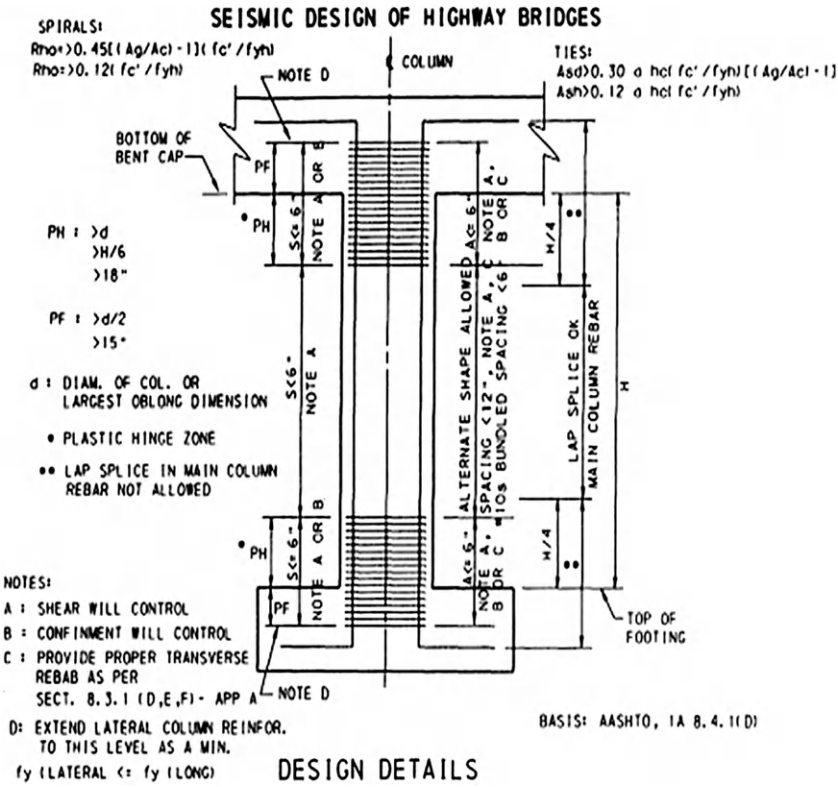


Figure 9.17 Locations of increased transverse reinforcements in columns and piers.

Dowels are added to all faces that will receive new concrete. Typically, these dowels consist of #5 or #6 bars embedded 12-in. with magnesium phosphate grout. The holes for horizontal dowels are drilled downward 15 to 20 deg so the grout will not spill out.

Tie-Backs. The author worked on strengthening the abutments of a railway bridge using the tie-back techniques shown in Figure 9.19.

9.7.4 Upgrading Bent Caps

Bent cap upgrades: These are uncommon because the caps tend to be stronger than the columns and are thus undamaged by large movements. They are basically the addition of two new beams on each side of the existing cap. These beams can be doweled into the existing caps and if needed tied together with HS rods through cored holes. If the bent upgrade is for a box girder bridge, holes can be drilled near the bottom

Figure 9.18 Column retrofit using steel casings. (*Source:* Courtesy of Michael Keever)

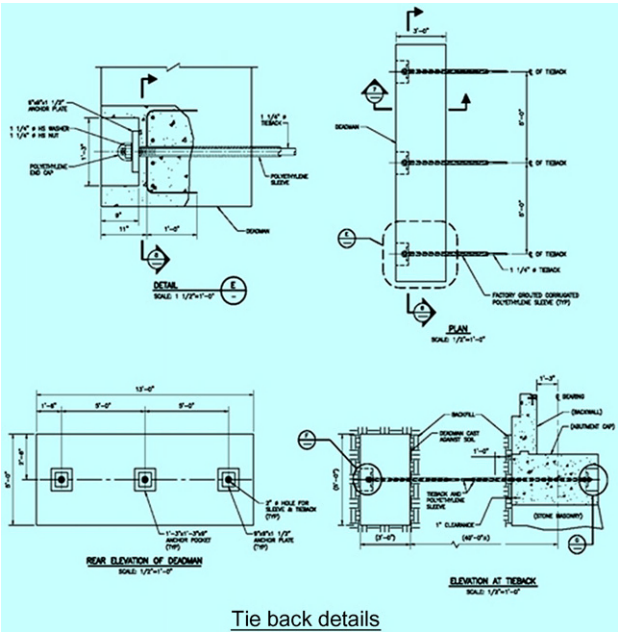


Figure 9.19 Strengthening existing abutment walls with tie-backs.

of the girders so continuous bars can be passed through the holes. These bars can be mechanically coupled or lapped in cells for continuity.

9.7.5 Retrofit of Footings

Preventing Footing Failure During Earthquake: Retrofit of footings tends to be the most difficult and costly of all retrofit measures because large excavations and new pilings are needed. Many times access roads need to be built just to get to the footing with heavy equipment. If footing reinforcement is necessary, it should designed for the applied loads but follow minimum requirements to maintain integrity in the event of seismic loading.

Minimum Reinforcements

Recommended Practice: As recommended in the literature, the top reinforcement for a continuous pier footing is 3/4-in.-diameter bars in transverse and longitudinal directions. The minimum top reinforcement for an individual pier footing is not be less than 50% of the area of the designed bottom reinforcement in both directions. Top and bottom reinforcement in footings in both directions is provided with horizontal or vertical hooks at both ends.

The bottom reinforcement mat in footings with piles is placed 2 in. to 3 in. clear above the tops of the piles. In special cases, where design requirements dictate and the pile pattern permits, the bars may be located between piles. In this case, a minimum clear distance of 2 in. to 3 in. shall be maintained between the reinforcing bars and the piles.

9.8 UNCOMMON RETROFIT CONCEPTS

9.8.1 Steel Girder Continuity Plates

An Option: Steel plates can be used to sandwich the steel girder webs of simple-span bridges. The plates are bolted to each web thus creating continuity across the joint over the bent. One side of the plate utilizes slotted holes to allow for thermal or rotational movements. The plates are typically 3/8 to 1/2-in. thick, and many HS bolts are placed on each web. The fixed side bolts can be torqued and the expansion side (slotted holes) nuts can be snug tight and the threads locked. New stiffener is applied prior to removal of old stiffener.

9.8.2 Anchor Slabs

Performance during Earthquake: An anchor slab replaces the approach slab in which the new slab is anchored firmly with new piles or is simply a large concrete block. As the superstructure moves away from the abutment or tries to rotate in plane due to the transverse swaying mid-structure, the new anchor slab is engaged and resists this motion. Thus the stiffness of the superstructure, by being fixed at the ends, increases the transverse stiffness of the system.

This retrofit measure saves a lot of work to the substructure, but is limited to short or moderately long bridges. The nuts for the bearing plates at the new bolsters should be backed off and locked to accommodate the thermal movements of the bridge.

9.8.3 Hinges

Limited Use of Hinges: The most destructive failure mode for bridges is the unseating of in-span hinges, which are not designed to cantilever. Thus, that entire length of span will drop, possibly causing a domino effect that will destroy much of the structure (Hipley 1997).

The first order of work, after providing access, is to clear out any formwork left in the work area and then core the holes for the cables and pipes through the existing diaphragms. These cored holes should be placed a few inches above or below the hinge seats' main transverse reinforcement and centered in the bays. The cored holes should clear the hinge seat faces by at least 6 in. to avoid cutting any transverse bars or the bearings.

Once the retrofit is complete, a shim is placed between the nuts and the bearing plate to allow for thermal movements, and the cables are tightened snug. The shim thickness may vary depending on the season as the hinge retrofit is completed. The shims are then knocked out.

Substructure Movement: Settlement of fill under and behind abutments is frequently accompanied by horizontal movement of the abutment top, and small rotations of tall piers result in appreciable displacement of the bearings. Sufficient expansion capacity must be provided in the bearings to accommodate the substructure movement, and so minimize the need to reset them.

Structural Details to be Approved for Catcher Blocks: The simplest and least expensive means of allowing the superstructure to "float" over the substructure in a large event is to place seat extenders (catcher blocks) under the girders. Once the bearings fail or shear, the weight of

the girders transfers to the catcher blocks to ride out the earthquake without a drop failure. The rods can be embedded in the catcher blocks with anchor plates. This will allow better shear resistance of the blocks and prevent "zippering" of the dowels.

9.8.4 Retrofit of Existing Movable Bridges

At the time when most movable bridges were constructed, there were generally no seismic codes in effect. Hence, it is important to evaluate their seismic vulnerability in both open and closed positions and in positions in between. According to AASHTO specifications, the seismic load used for the open position may be reduced by 50% if the bridge is in that position for less than 10% of the time. However, counterweights on bascule bridges represent large seismic inertias, whose effects on the girders needs to be explored, as do seismic acceleration forces on the span drive braking machinery when the span is raised. If the bridge structure falls into Seismic Zone 2, an acceleration coefficient (A) of 0.15 is applicable. The guidelines stated in FHWA *Seismic Design and Retrofit Manual for Highway Bridges* in conjunction with the *Highway Agency Design Manual* need to be followed as a minimum.

9.8.5 Seismic Retrofit Details for Movable Bridge Bearings

Many movable bridges exist over rivers which are navigable. These require special attention and retrofitting, as the cantilever arms are lifted several times a day. A multiple-span movable bridge has a large number of bearings (with two lines of bearings on each pier). It is important that all bearings allow movement during a seismic event; therefore, they require regular maintenance.

If the last cycle inspection report indicates that several bearings, sliding plates, and anchor bolt nuts exhibit moderate to severe rust and material loss or that some bearings are fully rusted, necessary retrofits are required. Problems with some of the damaged bearings appear to be locking up or freezing such that no movement is possible. This situation results in overstress at the end of the girder leading to possible failure. Shear cracks may be caused by a frozen bearing. Similarly, roller action may be blocked due to dirt. Water and salt penetration gives rise to rust and corrosion. Eventually, the girder may settle. Any existing damaged or seismically

deficient bearing that does not allow rotation and translation during a seismic event must be retrofitted or replaced.

Comparative Study with Bearing Replacement: Options available for upgrading the support system for seismic resistance are bearing retrofit or bearing replacement, including,

- Modification of the existing bearings to resist seismic loads
- Preventing toppling of bearings by installing longitudinal displacement stoppers
- Installation of longitudinal joint restraints as outlined in Subsection 10.2.1 of the FHWA *Retrofit Manual*.

Several methods of seismic retrofit are outlined for bearings and expansion joints in the FHWA *Retrofit Manual*. Primarily, the bearings, joint restrainers, and minimum seat widths for Seismic Zone 2 need to be assessed for a Seismic Zone retrofit.

Because of stringent operational and mechanical tolerance requirements, movable bridges need to be evaluated not only for stress but, more important, for displacements. Efficient functioning of the expansion bearings is therefore important. In bascule bridges, there are heavy counterweights that significantly affect the seismic behavior of the long structure. Recommendations on the following will be included in the Seismic Retrofit Report:

9.9 COMPUTER SOFTWARE

Computer software, such as *ADINA, GT-STRUDL*, *SAP 2000*, *SEISAB*, and *STAAD-Pro* and other approved software, has become a valuable tool for the structural engineer. However, even the best computer software cannot replace good engineering judgment and design practice. It is good to remember also that programs such as developed by PENNDOT are written by and for a particular state highway agency and have built-in defaults that apply only to that agency's design criteria. Thus engineers should review and evaluate such programs and make the necessary modifications to ensure that any retrofit conforms with AASHTO's design criteria. Generally, AASHTO requires that all program input and output be checked by a second engineer and that all results be verified by hand calculations. Also, moments and forces should be checked by dynamic analysis for the final design, and laboratory models of complex bridge sub-structures

need to be tested, by simulating earthquakes using shake-table displacements and forces.

It is encouraging to note that the software is being updated to comply with the revised AASHTO and State Codes. Also, new software is being developed giving the designer a free hand in its selection. However, many Agencies would like the software to be approved ahead of the computation. Available design software for modeling and analysis includes the following. Programs for modeling design and analysis and for substructures are given in Tables 9.2 and 9.3, respectively.

OpenSees. Finite element software developed by PEERC, University of California, Berkeley, for analysis based on proposed codes (millen@ce.berkeley.edu)

Table 9.2 Available Design Software for Modeling and Analysis

Program I.D.	Authors	Telephone No.	Website/e-mail
SAP 2000	CSI Berkeley, CA	510 845 2177	info@csiberkeley.com
RISA-3D	Risa Technologies	1-800 332 RISA	www.risatech.com
OpenSEES	Pacific Earthquake Eng. Res. Center UC Berkeley	510 642 4020	millen@ce.berkeley.edu
WINSEISAB	Imbsen Associates	212 279 9677	info@imbsen.com
STAAD.PRO 2002	Research Engineers Inc.	1 800 FOR RESE	www.reiworld.com
OPIS (Under Preparation)	AASHTOW are	1 800 231 3475	www.aashtoware.org
Pushover Analysis	SC Solutions		www.scsolutions.com

Note: URLs and phone numbers may change.

Table 9.3 Available Substructure Seismic Design Software

Program I.D.	Authors	Telephone No.	Website/e-mail
RC-Pier LA	Leap Software	800 451 5327	www.leapsoft.com
MERLIN-RCWALL	OPTIMATE	610 867 4077	
SAM	Bestech System Ltd.	800 347 4171	www.bestech.co.uk/ Bridges@bestech.co.uk
ABLRFD	PennDOT	717 787 7214	www.dot.state.pa.us

Note: URLs and phone numbers may change.

BRIDGEPEEB. Based on *OpenSees* by PEER; capable of pushover analysis incorporating liquefaction.

CONSPAN LA. Analysis and design program for prestressed-concrete girders, box beams, and voided slabs on single- or multiple-span bridges; allows LFD or LRFD design.

RC-Pier LA. Analysis and design of reinforced-concrete piers based on AASHTO LFD and LRFD codes; easily switched between English and metric unit systems.

SEISAB. Analysis of simply supported or continuous deck girder-type bridges for seismic response; contains AASHTO single-mode and multi-mode response spectrum analysis techniques.

GTSTRUDL. Structural analysis for static, dynamic, p-delta, nonlinear, buckling, or cable analysis; capable of steel, concrete, and timber design.

STAAD-Pro. Structural analysis for static, dynamic, p-delta, nonlinear, buckling, or cable analysis; accepts truss, plane, floor, and space structural types; capable of steel, concrete, and timber design.

9.10 CONCLUSIONS ON SEISMIC ANALYSIS, PRIORITIZATION AND RETROFIT

Several parameters have an important effect on the seismic response of bridges, such as soil—structure interaction, impact between adjacent spans, and behavior of steel bearings. As a means of analyzing these effects and mitigating their negative aspects, seismic resistance technology has improved significantly. Examples include pushover analysis, performance-based seismic engineering, and inelastic analysis allowing plasticity at pier columns. Some common analytical methods originally developed for buildings are now applicable to bridge structures. Both 2D and 3D models need to be employed in equivalent static analysis and time-history dynamic analysis.

As a first step in a retrofit program, bridges need to be prioritized based on physical parameters such as seismic vulnerability, function and traffic volume, and detour length. As part of this process, the designer must indicate all deficient seismic items and provide preliminary details. The first priority is major bridges that are at risk and those that carry heavy truck traffic. Some bridges are generally identified as more important than others, such as historic bridges and those on hospital, school, and national defense routes. Two hazard levels of FEE and SEE will be adopted in design.

There are a number of ways to retrofit a bridge structure to prevent collapse. Each method usually has a unique design unto itself and a unique style of construction. All methods of retrofitting a bridge should be compared, and the best method chosen should also be the most economical and have the least impact on the traveling public.

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Priestley 2002

CHAPTER TEN

Seismic Design for Buildings

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10.1 INTRODUCTION

Design of Buildings against Earthquakes: This chapter broadly reviews the need for seismic retrofit based on which it recommends further development of seismic analytical methods. Analytical methods are similar to that required for a replacement of buildings. While repairs are an inherent part of maintenance, retrofits involve upgrades and deviation from the original design. Detailed seismic resistant specifications for retrofit and rehabilitation, on similar lines to designing new structures, are required. Both 475 year and 2250 year events are typically considered.

A detailed literature review shows contributions by the following research organizations and those responsible for developing design codes:

American Institute of Architects (AIA)/Association of Collegiate Schools of Architecture (ACSA), Council on Architectural Research (1992), Buildings at Risk: Seismic Design Basics for Practicing Architects

Timber Construction Manual: American Institute of Timber Construction (AITC, 2012)

Earthquake-Resistant Structures
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Communicating with Owners and Managers of New Buildings on Earthquake Risk. FEMA389: American Technology Council (ATC)

NEHRP Guidelines for the Seismic Rehabilitation of Buildings. FEMA273: American Technology Council (ATC, 2003)

Earthquake Damage Evaluation Data for California, ATC 13: American Technology Council (ATC, 1985)

NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings. FEMA 274: American Technology Council (ATC, 1991)

NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part I: Provisions, and Part II, Commentary, 2000ed. FEMA 450: Building Seismic Safety Council (BSSC, 2004)

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Recommended Provisions and Commentary for Seismic Regulations for New Buildings and Other Structures. FEMA 450: National Earthquake Hazards Reduction Program (NEHRP, 2003)

Building Construction and Safety Code: National Fire Protection Association (NPFA, 2002)

Standards of Seismic Safety for Existing Federally Owned or Leased Buildings: National Institute of Standards and Technology (NIST, 2002)

Controlling Disaster: Earthquake-Hazard Reduction for Historic Buildings (National Trust for Historic Preservation, 1992)

The primary intent of all seismic code provisions is to protect the life safety of building occupants and the general public through the prevention of structural collapse from a natural event such as an earthquake. This goal encompasses the following objectives:

- To control the severity of damage in small or moderate earthquakes.
- To resist a minor level of ground motion without damage.
- To resist a moderate level of ground motion without structural damage or only some nonstructural damage.

- To resist a major level of ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage.

In accomplishing these ends through codes, many variables involved in seismic design must be taken into consideration:

- Variability in ground motion
- Soil type
- Dynamic characteristics
- Material strength properties
- Quality assurance and control
- Construction practices

According to the National Earthquake Hazard Reduction Program (NEHRP), because of these complexities, its code, like others, is intended as a reference and not as a model. Its provisions are designed to enhance the expected performance of higher-occupancy structures more so than normal-occupancy structures such as dwellings.

Objectives of the International Building Code These are:

- To establish the minimum requirements to safeguard the public health, safety and general welfare through structural strength,
- Means of egress, facilities, stability, sanitation, adequate light and ventilation, energy conservation,
- Safety to life and property from fire and other hazards attributed to the built environment and 4. To provide safety to fire fighters and emergency responders during emergency operations.
- To provide minimum design criteria for structures appropriate to their primary function and use, considering the need to protect the health, safety, and welfare of the general public by minimizing the earthquake-related risk to life, and
- To improve the capability of essential facilities and structures containing substantial quantities of hazardous materials to function during and after design earthquakes.

History of Seismic Codes: Seismic codes in the United States did not develop until the 1920s, and a mandatory code was not enforced, in California, until 1933. The first modern seismic code was not consistently applied until the 1950s, and then only in identified zones of higher seismicity. (See ICC 2002.)

The modern era of code development might be said to have begun in the late 1940s when SEAOC (Structural Engineers of California) (SEAOC 1991, 1995) responded to the inadequacy of seismic design codes by embarking on the first edition of its *Recommended Lateral Force Requirements and Commentary*, which became known as the *Blue Book*. Since that time seismic codes have continued to evolve based on the input of many organizations, including the International Council of Building Officials (ICBO), which adopted SEAOC's recommended seismic design provisions for new buildings in its 1961 Uniform Building Code (first published in 1927); the American Technology Council; FEMA, which established the National Earthquake Hazard Reduction Program (NEHRP) in 1978; the Building Seismic Safety Council (BSSC); the SBCCI (Southern Building Code Congress International); CUREE (Consortium of Universities for Research in Earthquake Engineering); MCEER (Multidisciplinary Center for Earthquake Engineering Research); and many others, including engineering research centers and professional societies and associations.

Role of NEHRP: The seismic provisions of most current building codes are largely based on the NEHRP provisions, supplemented by industry materials association standards. A significant difference between NEHRP and earlier model codes is its introduction of provisions that relate design forces to ground motions at a building site. These required designers to consider dynamic effects and resulted in larger deformations for some kinds of structures.

Seismic codes are based on the work of many scientific disciplines such as physics, applied mathematics, geology, seismology, and materials science. They are specialized documents, and the research needed to develop them takes time. For this reason, many countries have adopted U.S. codes for seismic design with some modifications. Table 10.1 illustrates this worldwide distribution.

10.2 DEVELOPMENT OF U.S. SEISMIC BUILDING CODES

The evolution of seismic concepts has resulted from three factors:

- **Study of Past Events.** New and better ideas frequently flow from observed damage which are well documented. Learning from past

Table 10.1 Old and New Seismic Codes in Use around the World*

Country	RSD	SUP	IHEE	PEHA	SDB	UBC	AASHTO
Afghanistan		X	X	X			
Argentina	X		X	X	X	X	
Australia	X		X	X	X	X	
Brazil			X	X	X		
Brunei				X	X		
Bulgaria	X		X	X	X	X	
Burma				X	X		
Burundi				X	X		
Cameroon				X	X		
Canada	X		X	X	X	X	X
China	X		X	X	X	X	
Cuba	X			X	X	X	
Greece	X	X	X	X	X	X	
India	X		X	X	X	X	X
Iran	X		X	X	X	X	X
Israel	X	X	X	X	X	X	
Italy	X	X	X	X	X	X	
Japan	X	X	X	X	X	X	
Mexico	X		X		X	X	X
New Zealand	X		X	X	X	X	
Pakistan					X	X	X
Philippines	X				X	X	X
Turkey	X	X	X	X	X	X	X
USA	X	X	X	X	X	X	X
Venezuela	X	X	X		X	X	X

*Local building codes may also exist.

performance has evolved successful seismic structural systems that continue to be used; unsuccessful systems are eventually abandoned. Certainly, California's 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge earthquakes had much to contribute in terms of lessons learned.

- **Research Data.** New ideas for structural concepts are frequently developed jointly by design engineers and university research laboratories. These systems are physically tested and analytically studied.
- **Building Code Developments.** Code-designed structures have a better chance of surviving earthquakes. Seismic codes such as IBC and ASCE-7 have come a long way from the codes of the 1970s.

Rapid Code Changes in Design

Seismic codes to be more flexible than non-seismic codes: In these times of rapidly evolving technology such as precast concrete segments and high-performance materials, codes are usually created five to ten years after new ideas are developed. Provisions may not be current or at the cutting edge of new thinking, which is why overly specific codes may stifle and delay innovations (Table 10.2), particularly given that structural systems that are listed in building codes are adopted by many engineers as gospel. It is for this reason that code interpretation should be flexible.

Wind Codes Replace Seismic Codes

San Francisco Earthquake emphasized needs for design code: Even with fatalities estimated by the USGS (2006) and others of between 700 and 2,100, and the destruction of 27,000 buildings, the 1906 San Francisco earthquake did not directly stimulate an explicit seismic code response. A wind load requirement was implemented and was assumed to be sufficient to resist equivalent earthquake forces. This approach was in keeping with the technology of those days.

Concepts of dynamic response gained interest and in the early 1930s when studies of structural dynamics with analysis and models were initiated at Stanford University. This ultimately led to a design approach that acknowledged the importance of building periods and dynamic rather than static design concepts.

Buildings catching fire an added hazard: Investigators of the 1906 San Francisco earthquake reported that 80% to 95% of the damage in the most affected areas was caused by fire, leaving only 5% to 20% caused by shaking. The event was thus interpreted as a great fire rather than earthquake. Differences in building performance based on construction, configuration, and soil conditions were observed, but there was no systematic investigation of the performance of seismic-resistant systems, which were voluntarily implemented by engineers in the earlier last four decades of the 19th century.

Engineers' awareness of seismic risk increased, resulting in voluntary efforts as seismic-resistant design in the early twentieth century, but codes did not specifically direct the design community to address related hazards. In fact, performance observations of selected steel-frame and concrete structures were used to justify the removal of some code restrictions concerning building height.

Table 10.2 Research Requirements for Improving Future Codes*

Actual or Potential Failure Type	Corresponding Code Provision	Seismic Event	Research Required
Dynamic response of structures not well predicted by static lateral force equations	Use dynamic analysis procedures to determine seismic design forces in irregular and tall structures	Future earthquake	Dynamic response analysis
Failure of wall-to-diaphragm ties	Design forces for anchorage of diaphragms larger than seismic design forces	Future earthquake	Nonlinear dynamic analysis of instrumented buildings
Extreme cyclic loads cause concrete crushing	Confinement reinforcing in beams, columns, and joints of concrete in moment-resisting frames	Future earthquake	Cyclic lab testing of beams, columns, and joints
Rocking of walls and splitting of sill plates under lateral forces	Use plate washers under heads of anchor bolts on sill plates of wood walls	Future earthquake	Cyclic lab testing of shear wall panels
Brittle fracture of welded connections in steel frames	Use notch-tough weld filler metals	1994 Northridge	Further research required
Failure of column braces	Component forces resulting from yielding of tributary braces to be resisted by steel columns	1985 Mexico City	Further research required
Failure of columns in building	Provide adequate size columns beneath discontinuous frames or discontinuous walls	1979 Imperial Valley	Further research required
Separation of load-bearing precast walls from roof	Strong connection required between walls and diaphragms and continuous ties in diaphragms	1971 San Fernando	Further research required

(*Continued*)

Table 10.2 Research Requirements for Improving Future Codes*—(Continued)

Actual or Potential Failure Type	Corresponding Code Provision	Seismic Event	Research Required
Many unreinforced masonry bearing wall fail	Unreinforced masonry bearing walls not be used in seismic design categories D	1933 Long Beach	Further research required
All masonry buildings fail	Buildings higher than 160 feet require load-carrying space frame for categories D–F	1906 San Francisco	Further research required

*Performance of new materials such as HPC, HPS and reinforced plastics and glass need to be investigated.

10.3 SEISMIC EFFECTS ON FOUNDATIONS AND SUPERSTRUCTURE

The following authors have contributed to several aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to seismic design is presented here:

Earthquake Architecture Explorations (Charleson et al., 2004)
Earthquake Resistant Design (Dowrick, 1981)
20 Tools That Protect Historic Resources after an Earthquake: Lessons learned from the Northridge Earthquake (Eichenfield, 1996)
Fundamental Rules for Earthquake Resistant Design (Flesch, 2006)
Design of Shallow Foundation (French, 1991)
Performance-Based Seismic Design of Tall Buildings, Why and How? (Gerges et al., 2012)
Relationship of the NEHRP Provisions and 2006 IBC (Hamburger, 2006)
Background and History of the California Hospital Seismic Safety Program (Holmes, 2002)
Seismic Retrofit Policies: An Evaluation of Local Practices in Zone 4 and Their Application to Zone 3 (Hoover, 1992)
LomaPrieta: An Engineer's Viewpoint (Kariotis et al., 1990)
Behaviour of Reinforced and Prestressed High Strength Concrete Beams (Khan et al., 1994)
Shear Design of High Strength Concrete Beams (Khan et al, 2000)

Elastic full Composite Action in Slab-Beam Systems (Khan et al., 1969)
Elastic Composite Action in Slab-Frame Systems (Khan, 1970)
Structural Deficiencies of Houses in Katchi Abadis (Non-engineered Construction of Buildings) (Khan, 1987)
Seismic Design of Building Structures (Lindeburg et al., 2001)
The Seismic Retrofit of Historic Buildings (Look et al., 1991)
Bridge PBEE: OpenSees 3D Pushover and Earthquake Analysis of Single-Column 2-span Bridges (Lu et al., 2012)
General Principles for a Good Design (Maini, 2006)
Materials for Civil and Construction Engineers (Mamlouk et al., 1991)
Comparison of Recent U.S. Seismic Codes (McIntosh et al., 1997)
Bricks, Mortar, and Earthquakes (Langenbach, 1981)
Seismic Design of Reinforced Concrete and Masonry Buildings (Paulay and Priestly, 1992)
Consulting Engineers (1990) Seismic Retrofit Alternatives for San Francisco's Unreinforced Masonry Buildings: Estimates of Construction Cost & Seismic Damage (Rutherford et al., 1990)
Fire Following Earthquakes (Scawthorn et al., 2005)
Structural Evaluation of Historic Masonry Buildings (Schuller et al., 1995)
Uniform Code for Building Conservation (Schuller, 1991)
Vision 2000: Performance Based Seismic Engineering of Buildings (Schuller et al., 1995)
Seismic Design of Building and Bridges (Williams, 2003)
Radical Reconstruction (Woods, 1997)

In addition to the above publications, relevant individual references are discussed in the text.

The following direct physical phenomena impact building foundations as they are transmitted from ground to substructure to superstructure:

- Surface faulting or fault rupture
- Vibration of soil from seismic waves
- Ground cracking
- Liquefaction
- Ground lurching
- Differential settlement
- Lateral spreading

Data required: To analyze and design a building structure to resist seismic loading arising from these phenomena, several important factors must be taken into consideration. This step requires gathering great amounts of information about ground accelerations; soil conditions; overall mass, geometry, and dimensions of components, both structural and nonstructural; shape and height; and connections between columns and foundation and columns and beams.

10.4 SITE CONDITIONS AND GEOTECHNICAL REPORT

Ground accelerations and soil type relate to the relative seismic hazards of certain building sites, such as those near faults and/or exhibiting poor soil conditions. Thus, the first step in the design of a proposed structure should be the geotechnical analysis of site suitability using seismic hazard maps and soil analysis.

10.4.1 Seismic Hazard Maps Based on Seismology

The U.S. Geological Survey (USGS) National Seismic Hazard Maps display ground motions and accelerations. They are used to formulate seismic building codes based on risk and are derived from seismic hazard curves calculated on a grid of sites across the United States that describe the frequency of exceeding a given set of ground motions. The first maps, developed in 1975, were replaced in 1996 to include 500-year events covering the 1811–1812 New Madrid, Missouri, and 1898 Charleston events. In 2002, the maps were updated with 2,500-year occurrences. The were revised once again in 2008.

According to a fact sheet posted on the USGS website, the 2008 versions supersede those released in 1996 and 2002. These maps are the basis for seismic design provisions of building codes, insurance rate structures, earthquake loss studies, retrofit priorities, and land-use planning. Their use in the design of buildings, bridges, highways, and critical infrastructure allows structures to better withstand earthquake shaking, saving lives and reducing disruption to critical activities following a damaging event. The maps also help engineers avoid costs from over-design for unlikely levels of ground motion.

Effects of Changes in Maps: The current maps are not based on hazard but on unifying risk-based magnitudes and probabilistic ground motions.

The net changes for the western United States are moderate (10%) but are greater in cities where the USGS has provided new seismic hazards data. Small changes are shown for the central United States.

10.4.2 Effective Peak Acceleration

The USGS 2008 seismic hazard maps identify effective peak acceleration (EPA), (based on seismology of the area) which reflects light, moderate, and severe shaking risks as a percentage of the acceleration of gravity that can be expected in an area. Seismic codes use EPA calculations to proportion member sizes of buildings to resist resulting seismic shear forces and bending moments and prevent damage.

Two Hazard Levels: One approach to satisfying basic safety objectives using EPA maps is to evaluate a structural design according to two levels of shaking hazard: the functional evaluation event (FEE) and the safety evaluation event (SEE). If a structure meets FEE following an inspection, immediate operation is allowed and minimal damage is permissible. This hazard level has a 10% probability of exceedance in 50 years (i.e., a return period of approximately 500 years). For SEE, significant disruption to service is permissible as is significant damage. This level has a corresponding 2% probability of exceedance in 50 years (amounting to return period of approximately 2,500 years).

10.4.3 Acceleration Coefficients and Seismic Category

Acceleration coefficients measure the horizontal accelerations moving through the ground. They are expressed in terms of percentage of gravitational acceleration (*g*). Seismic performance category is a function of acceleration coefficient and relates to seismic zone. A location having an acceleration coefficient rate of 0.19 is considered to be in a high category. Imperial County in California, for example, has an acceleration coefficient of 0.80, which is the maximum in the United States. Seismic Zones were introduced into the building codes in 1935 and have been continually updated over the years. A seismic zone is solely dependent on the probable site ground motion for a region. This was developed through experience of where earthquakes were most likely to happen and to what severity. United States is divided into four seismic performance zones. Areas in Zone 1 face little risk of a significant event; for areas in Zone 4, the risk is high. Each risk zone has a corresponding acceleration coefficient.

International Building Code (IBC) classifies structures into Seismic Design Categories (SDC): this is different from the Uniform Building Code (UBC) which classified them into Seismic Zones. Seismic Design Categories include classifications of A, B, C, D, E, and F and are based on the following three basic criteria.

1. Probable Site Ground Motion
based on FEMA maps, the maximum spectral acceleration and the design acceleration response.

2. Soil (Site Class):
Site class is based on the type of soil. Soil classifications are A, B, C, D, E, or F defined below.

3. Building Occupancy Use:
Building occupancy groups are broken into four types.
Type IV: Agricultural Buildings
Type III: Essential Buildings
Type II: Structures that represent a substantial hazard in the event of collapse or failure
Type I: For special buildings that do not fit Type II, III or IV

Seismic categories, based on elastic design or inelastic design performance criteria, follow:

- Category A: no damage
- Category B: moderate damage
- Category C: life safety
- Category D: no collapse

10.4.4 Soil Analysis by Geologist

The second aspect of a geotechnical site evaluation is soil analysis. Ground shaking is governed by shear wave velocity: the higher the velocity, the less ground shaking will occur. Velocity is determined by soil type and is greater in soft soil than in hard soil or rock. According to the International Code Council (ICC, 2009), estimates of average shear wave velocity in the top 100 feet of soil are based on type and engineering properties as A, B, C, D, E, or F.

Classification: Of the types, A is hard rock found in eastern states; B is rock found mainly in western states; C is soft rock or very dense soil. D is Stiff Soil or by default when soil properties are not sufficiently known. E is soft soil; F is sensitive clay or peat. Ground response coefficients (Cv and Ca) are parameters that reflect potential amplification of

ground vibration caused by different soil types. Cv is increased by 230% in soil type E. For the soil type F, a hazard evaluation is required to determine ground response.

NEHRP requires site-specific soil data to establish the seismic design category of a building. More extensive geotechnical investigations of the building site can be used to determine the appropriate code-defined soil classification and provide the design team with critical information that will affect seismic design requirements.

10.5 STRUCTURAL RESPONSE

Once the geotechnical site evaluation is completed, the designer/engineer turns to structural response analysis of the proposed structure, which determines the effect of lateral seismic forces on a structure as a result of ground vibrations.

Seismic analysis methods can be divided into two groups: Linear and nonlinear. (See Figure 10.1.) In general, nonlinear methods are not useful for verifying compliance with minimum code provisions. Linear analysis will suffice.

10.5.1 Linear Dynamic Analysis

A *linear dynamic analysis* is useful for evaluating irregular or dynamically complex (e.g., flexible) buildings, which have a nonuniform distribution of mass or stiffness, such as those with intricate space planning requirements or asymmetrical configurations. The structure–ground shaking interaction is usually modeled using a response spectrum analysis.

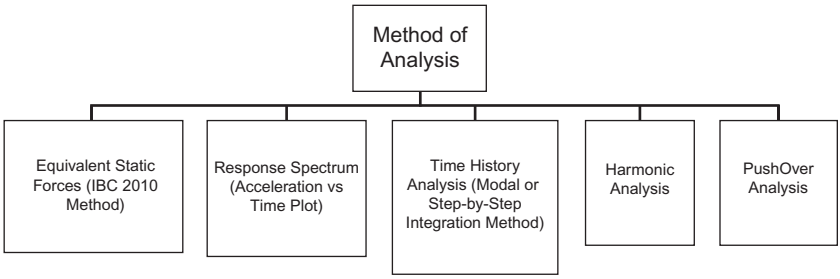

Figure 10.1 Methods of seismic analysis as recommended by building design codes.

Linear analysis can be static or dynamic. The IBC's equivalent lateral force procedure is a simple version of a linear static analysis, which is well suited to buildings with regular configurations that have response to ground motion dominated by the fundamental mode.

Modal Analysis: The dynamic version of a linear analysis is known as modal analysis and is frequently used to create a more accurate picture of how irregular structures perform. Seismic codes allow and, in some cases, require designers to use a modal analysis, which is based on an idealized site response spectrum. It takes into account motions that are influenced by higher mode shapes to provide more information. Displacements expected under different modes can be added together to identify critical design behaviors.

10.5.2 Nonlinear Dynamic Analysis

Nonlinear dynamic analysis or time history analysis is the most sophisticated and time-consuming analysis method, requiring physical data based on detailed knowledge of building properties and ground motions. It is relatively new to design practice and is used primarily in research and experimentation. The IBC accepts this analysis procedure, provided that an independent design review is performed. Nonlinear analysis can be used to design new buildings, but it is more common in evaluation and retrofit of existing structures.

Response Time History Dynamic Analysis

Time-history analysis examines modal response using actual ground motion data. The code requires to it consider several different ground motion records to ensure that response accounts for future unknown ground motion patterns. Standard dynamic analysis software (Figure 10.2) can perform a response history analysis on a structure subjected to time-

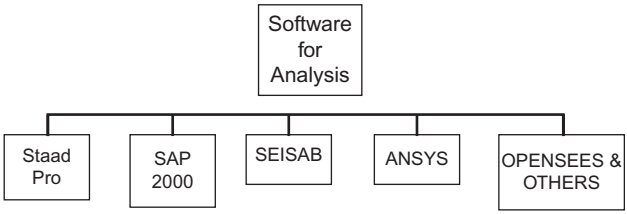

Figure 10.2 Commonly used building and analysis and design software.

varying forcing function loads at the joints and/or a ground motion at its base. This analysis is performed using the modal superposition method.

Pushover Equivalent Static Analysis

Pushover analysis is a static procedure that uses a simplified nonlinear technique to estimate seismic structural deformations. Structures redesign themselves during earthquakes. As individual components of a structure yield or fail, the dynamic forces on the building are shifted to other components. A pushover analysis simulates this phenomenon by applying loads until the weak link in the structure is found and then revising the model to incorporate the changes in the structure caused by the weak link. A second iteration indicates how the loads are redistributed. The structure is "pushed" again until the second weak link is discovered. This process continues until a yield pattern for the whole structure under seismic loading is identified.

Pushover analysis is commonly used to evaluate the seismic capacity of existing structures and appears in several recent guidelines for retrofit seismic design. It can also be useful for performance-based design of new buildings that rely on ductility or redundancies to resist earthquake forces.

10.6 ESTIMATING LATERAL FORCES

The lateral forces exerted on the structure by ground vibrations may be determined by the static or equivalent lateral force procedure (ASCE 2003a,b, ASCE 2000/FEMA 2000). Base shear is an estimate of the maximum expected lateral force on the base of the structure due to seismic activity. It is calculated using the seismic zone, soil material, and building code lateral force equations (Figure 10.3). Notations in IBC/UBC are used in mathematical equations.

Base shear value (V) is determined by combining the following physical factors:

- Soil conditions at the site
- Proximity to potential sources of seismic activity (such as geological faults)
- Probability of significant seismic ground motion
- Level of ductility and overstrength associated with various structural configurations and total weight

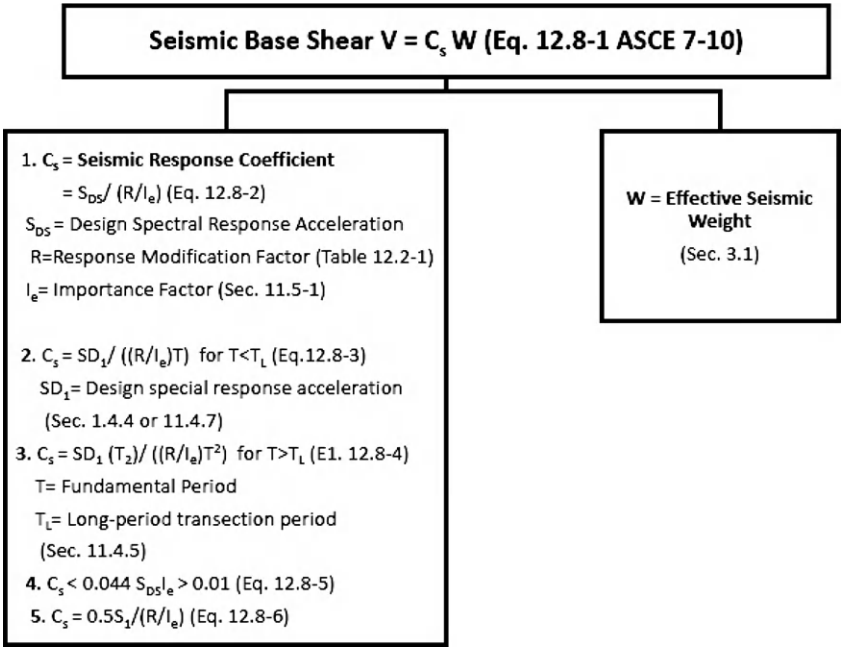

Figure 10.3 Flow diagram for computing base shear for buildings.

- Fundamental (natural) period of vibration of the structure when subjected to dynamic loading

10.6.1 Fundamental Mode

The tendency for a building to sway from side to side in response to ground motion produces greater accelerations in its upper parts. This back-and-forth motion is called the *fundamental mode* and dominates most structural seismic response. It is influenced by soil type, structural stiffness, and building use.

Period of Vibration

Mode shapes are movement patterns that occur naturally in structures that have been set in motion by ground shaking. For seismic-resistant structures each structure has a unique natural or fundamental period of vibration, which is the time required for one cycle of free vibration.

Parameters: The factors determining the fundamental period include structure stiffness and height. The equation provided in the Uniform

Building Code (UBC) for determining the natural period of vibration is (refer to UBC for notations used in formulae).

$$TimePeriod$$
$$T = Ct(hn)\hat{}(3/4)$$

where hn is typically 12 feet and Ct is 0.030 for reinforced concrete moment−resisting frames. After inputting the value, the time period = 0.193 seconds.

Seismic Response Coefficient Cs

The ground response coefficients C_A and C_V are a function of various parameters, for example, zone factor (Z) and the soil profiles S_A to S_E. The parameters, which affect the potential amplification of the ground vibration, are caused by different soil profile types. The initial time interval at which the structure vibrates is called the primary period. It determines which of the two coefficients C_A or C_V governs the seismic design of the structure. The acceleration-based coefficient C_A controls for a shorter period up to approximately one second, and the velocity-based coefficient C_V controls for longer periods.

Soil profile type S_A is described as hard rock and mainly occurs in eastern states as stated earlier. Soil profile type S_B is found mainly in the western states described as rock and ground response coefficients. C_A and C_V are identical to effective peak acceleration value (Z). When a soil's characteristic is unknown, in accordance with UBC and IRC, soil profile S_D may be assumed as default. Table 10.3 shows the wide range of shear wave velocity.

The seismic response coefficient Cs represents the design elastic acceleration response of a structure to the input ground motion and is calculated as

$$Cs = Cv\ I/RT$$

where $Cv = 0.54$ from the ground response coefficient table for Zone 3 and stiff soil (soil type D) and I = 1. The form of this expression indicates that the response coefficient increases as the importance factor increases and as the response modification factor and natural period reduce. The effect of the importance factor is to increase the seismic response coefficient by 25%. The maximum value of the seismic response coefficient, $Cs = 2.5\ Ca\ I/R$. The expression controls for a shorter period up to 1 second.

Table 10.3 Comparative Study of Load Combinations in Different Codes

AISC Code	ASCE Code	ACI Code	International Building Code (NJ Edition)
1.4D	1.4D	1.4(D + F)	1.4(D + F)
1.2D + 1.6L + 0.5 (Lr or S or R)	1.2D + 1.6L + 0.5 (Lr or S or R)	1.4(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)	1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
1.2D + 1.6(Lr or S or R) + (0.5L or 0.8W)	1.2D + 1.6(Lr or S or R) + (L + 0.5W)	1.2D + 1.6(Lr or S or R) + (1.0L or 0.8W)	1.2D + 1.6(Lr or S or R) + (f1L or 0.8W)
1.2D + 1.6W + 0.5L + 0.5(Lr or S or R)	1.2D + 1.0W + L + 0.5 (Lr or S or R)	1.2D + 1.6W + 1.0L + 0.5 (Lr or S or R)	1.2D + 1.6W + f1L + 0.5 (Lr or S or R)
1.2D ± 1.0E + 0.5L + 0.2S	1.2D + 1.0E + L + 0.2S	1.2D + 1.0E + 1.0L + 0.2S	1.2D + 1.0E + f1L + f2S
0.9D ± (1.6W or 1.0E)	0.9D + 1.0W	0.9D + 1.6W + 1.6H	0.9D + 1.6W + 1.6H
	0.9D + 1.0E	0.9D + 1.0E + 1.6H	0.9D + 1.0E + 1.6H

D = dead load; L = live load; Lr = roof live load; W = wind load; S = snow load; E = seismic or earthquake load; R = rain load or nominal load due to initial rainwater or ice exclusive of the ponding contribution; H = loads due to weight and lateral earth pressure of soils, groundwater pressure or pressure of bulk materials; T = total effect of temperature, creep, shrinkage, differential settlement and shrinkage-compensating concrete; f2 = 0.7 for roof configurations (such as saw tooth), and 0.2 for other roof configurations; f1 = 1 for floors in places of public assembly, for live load in excess of 100lbs/ft2 and for parking garage live load, and 0.5 for other live loads. F = load due to fluids with well defined pressures and maximum heights.
Note: Applicable Code needs to be consulted for any changes in the load factors or load combinations.

Response Modified Factor an Engineering Approach

The probability of occurrence of the most powerful event is small and uncertain. For this reason both IBC and UBC allow the use of a response modification factor (*R*) in scaling down peak design moments. *R* values are based on a building's energy absorption and dissipation capacity. Larger values are assigned to highly damped structures constructed of ductile materials.

Factors Affecting *R* Values: The planning and design of a seismic-resistant structure must consider such issues as configuration, shape, and height; foundation and framing system; materials; construction methods; and nonstructural components. In resolving these, the following facts must be remembered:

Building Shape Influences Behavior. Geometric shapes such as squares or rectangles usually perform better than buildings in the shape of the letters L, T, U, H, O, or a variation of these.

Building Material Influences Behavior. Ductile materials perform better than brittle ones. Examples of ductile materials include steel and aluminum. Examples of brittle materials include brick, stone, and unstrengthened concrete. Concrete, the most widely used construction material in the world, is made of sand, gravel, and crushed stone bonded with cement and needs to be reinforced with steel bars.

Height Affects Behavior. Buildings of different heights shake at different frequencies. Soil response to shaking is a factor.

Function Affects Design. Seismic-resistant criteria may differ for hospitals, fire stations, office buildings, and the like.

Proximity Affects Design. Close proximity to adjacent buildings influences.

10.7 STRUCTURAL COMPONENTS

The structural components in a building include the foundation, the framing system, walls, connections, and roof and floor slabs.

- Frame types include concrete, steel, and braced steel.
- Walls can be concrete, unreinforced masonry infill and noninfill, reinforced masonry, and wood stud.
- Roof and floor slab types include metal decks, wood, concrete, and precast.

Simplified Approach: The analysis and design of building frames and footings may be summarized as a series of steps, each of which is governed by an applicable seismic code.

1. Planning geometry, floor heights, and beam spans
2. Selecting steel, reinforced concrete, prestressed concrete, timber, or masonry
3. Selecting foundation type
4. Computing lateral forces
5. Finalizing member design
6. Finalizing ductility connection details

10.7.1 Construction Materials

Except for its emphasis on light weight and ductility, seismic design utilizes the same relatively small number of basic structural elements that nonseismic design employs for an economical design.

Timber

Among traditional structural materials, timber, masonry, concrete, and metals (steel and aluminum), the most efficient seismic-resistant material for low-rise buildings is timber. Timber framing dates back thousands of years and has been used in localities where timber was in good supply and building stone was not. In the United States, Canada, and Europe, good-quality timber, such as sawn lumber, CLT and glulam, is available for residential and public building construction. If properly engineered, it lends itself to seismic survivability because of its light weight as well as its flexibility. In this type of framing system, seismic resistance comes from the following:

- Rigid plywood shear walls
- Wood structural panel diaphragms
- Drag struts or collectors to distribute shear along a diaphragm length
- Adequate lateral bracing and tying components together, from roof to foundation

Timber buildings should be carefully designed and constructed. For example, large garage openings greatly reduce the area of walls available to resist lateral and gravity forces, and introduce large eccentricities that lead to significant torsional loading. During the magnitude 6.5 San Fernando, California, earthquake in 1971, numerous modern split-level and two-story wood frame houses were severely damaged because their garage walls were inadequately braced.

Cross-Laminated Timber (CLT)

Developed in Switzerland and Austria in the 1990s, cross-laminated timber (CLT) has gained popularity and now has the potential for use in tall buildings higher than 10 stories. CLT buildings are lightweight and relatively quick to erect compared to buildings of steel or reinforced concrete.

CLT is made from layers of dimensional lumber, each stacked at right angles to the adjacent layers and glued to form solid elements. Boards are kiln-dried prior to lamination. By varying the number of layers as well as the lumber species, grade, and thickness, excellent bonding is obtained compared to sawn wood. CLT panels can be used for walls, floors, roofs, stairways, and elevator shafts.

Two full-scale models of three- and seven-story structures were tested at a magnitude of 7.2 on the world's largest shake table in Miki, Japan. Both models showed good ductile behavior and energy dissipation due to mechanical connections between beams and columns. In an earthquake, failure is localized at the connections and is relatively easy and cost-effective to fix by substituting new connections.

Light-frame structures have environmental benefits. They usually gain seismic resistance from rigid plywood shear walls and wood structural panel diaphragms.

In North America, CLT product information is available from the Engineered Wood Product Association (www.apawood.org), the American Wood Council (www.awc.org), and the Timber Research and Development Association (TRADA, www.trada.co.uk), and from FPInnovations (www.fpinnovations.ca).

AITC Timber Construction Manual has new material on the role of glulam in sustainable building practices. It includes the latest timber design and construction codes, including the 2012 National Design Specification for Wood Construction, AITC 117-2010, and examples featuring ASCE 7-10 and IBC 2009.

Masonry and Cement Mortar

In 1988, the Masonry Standards Joint Committee (MSJC) placed limitations on the use of cement mortars in lateral force—resisting structural members in areas of high seismic risk because of their lower flexural bond strength. Before that time these had been used as alternatives to traditional Portland cement-lime mortar. Based on the latest research, however, at Drexel University by Professor Ahmad Hamid et al. (2006) and University of Texas by Richard Klingner et al. (2010), "Structural formulation of fully grouted reinforced masonry walls in response to earthquake

forces is dominated by grout and reinforcement; it is unaffected by mortar formulation." Shake-table tests using ground motion records from the 1994 Northridge earthquake bear this out. Fully grouted participating elements in high seismic areas can be built with mortar formulated with masonry cement and provide acceptable performance. Other universities performing research on the subject include University of Colorado at Boulder, by Benson Shing (1997), who also served as member of ASCE Methods of Analysis Committee.

Reinforced Masonry

The devastating Long Beach earthquake in 1933 revealed that masonry construction was dangerously seismically inadequate. The California State Code made reinforced masonry mandatory. The Field Act introduced after the earthquake in 1933 banned unreinforced masonry construction. (NISEE, University of California, Berkeley, The Long Beach Earthquake of 1933.) To achieve ductile behavior in masonry, it is necessary that the shear strength of the wall be greater than the tensile strength of the reinforcement so that the wall does not fail in shear first and to ensure a kind of bending failure by yielding of reinforcement.

Reinforced Concrete

Reinforced concrete is generally the answer for all types of construction, small, medium, or large. Whether seismic or nonseismic, it combines the best of concrete and steel, although, like timber or masonry, its use is restricted to small buildings. A traditional reinforced concrete frame should have ductile joints.

Fiber-Reinforced Polymer (FRP) Concrete. Fibers strengthen brittle concrete material to increase durability and reduce cracking.

High-Strength Concrete (HSC). A study carried out by the author has shown that high-strength reinforced concrete beams are lighter (12), (13). A reduction in dead weight up to 50% makes HSC more suitable for lateral forces in seismic design than normal-weight concrete.

Prestressed Concrete. Prestressing overcomes concrete's natural weakness in tension, making it suitable for long spans and for resisting high loads. High-strength tendons (generally of high-tensile steel cables) are inserted to provide a clamping load, producing a compressive stress that offsets tensile stress.

A Brief History of Reinforced Concrete Frameworks

Reinforced concrete framing became popular in California immediately after the 1906 San Francisco earthquake. The early years of seismic codes, about 1908 to 1915, featured nonductile frameworks of reinforced concrete column—girder moment frames. The early use of wood floor infills between concrete floor girders was followed rapidly by all-concrete floors. It was not until the early 1960s that nonductile frames were recognized as an unsafe collapse mechanism because of inadequate shear capacity or because of poorly confined concrete.

Developing Ductile Frames: Early steel frames for gravity load support were initially encased in masonry cladding, soon to be changed to reinforced concrete frame infill walls, during the period from 1910 to 1930. This was a simple change that added definable strength around the steel.

Soon after bare concrete frames were developed, they were in-filled with concrete shear walls for lateral stability. Not all of these buildings were well conceived structurally, and because of the ease of adding or deleting walls, torsional problems became common. Significant reinforced concrete shear walls acting as bearing walls evolved in the 1920s, without moment frames.

It was not until the late 1960s and early 1970s, after substantial research at the University of Illinois, that the benefit of confined concrete columns and ductile concrete frames was recognized and adopted for dependable seismic resistance.

In the 1970s and 1980s, a dual system of ductile concrete moment frames coupled with confined concrete shear walls was developed. This concept works best with a perimeter moment frame and an interior shear wall core, or the reverse, using a perimeter ductile wall and an interior ductile moment frame. Finally, a dependable concrete dual system evolved.

Shear walls coupled with yielding shear links were developed in New Zealand in the 1980s after successful tests carried out by Park and Paulay (1975). This coupled-wall system is another excellent example of creative research to develop a low-cost mechanism for seismic energy dissipation.

Active Code Provisions

Upgrades of California Buildings: Two large-scale examples of active seismic code provisions were started by California. The first was a program to reduce the risk from URM buildings. Over 10,000 URM buildings have been brought into compliance with local ordinances, most by retrofit but some by demolition (SSC, 2003). Active code provisions result from policy decisions to reduce community seismic risk by requiring seismic upgrading of certain buildings known to be particularly vulnerable to unacceptable damage.

The second program, created in 1994 following the Northridge earthquake, gave California hospitals until 2030 to retrofit or replace their buildings to comply with state law. The program's intention is to enable buildings to be functional following an earthquake.

Mud Brick Adobe Buildings

Nearly half of the global population lives or works in buildings made of earth, which is one of the oldest building technologies in the world. One reason for its continued use is the insulating properties of thick mud walls against extremely hot and cold weather.

Adobe bricks are still one of the most widely used building materials. Their use is very common in some of the world's most hazard-prone regions:

- Latin America
- Africa
- The Indian subcontinent
- The Middle East and Turkey
- Southern Europe

Adobe buildings are considered very vulnerable during strong quakes. However, multiple methods of seismic strengthening are available (Lu 2012).

The author presented a paper at a UNESCO conference at Middle East Technical University in Turkey in 1987. Key factors presented for improved seismic performance of adobe construction include

- Experienced masons
- Quality of construction
- Use of straw to resist tensile forces
- Seismic reinforcement
- Compact, box-type layout

10.7.2 Primary Framing Systems

The engineering profession had progressed fairly slowly until the early 1980s from the basic framing concepts that were first evolved in the early 1900s. When the concerns about seismic performance and energy dissipation became paramount, researchers and design engineers investigated mechanisms and configurations to supplement the basic rectangular grid framing in use for over 100 years.

Steel Moment -Resisting Frames

Steel structures are considered the most seismic-resistant, allowing smaller member sizes compared to reinforced concrete. However, after the 1994

Northridge earthquake in southern California a number of welded-steel moment–resisting frame buildings, which looked seismic-proof, had experienced brittle behavior and been hazardously damaged. However, steel structures have progressed from a simple steel frame, braced laterally by unreinforced masonry, to complete moment frames with full lateral load resistance. As a consequence, the American Institute of Steel Construction introduced AISC 358, "Pre-Qualified Connections for Special and Intermediate Steel Moment Frames."

Concrete Dual System Frames

Reinforced concrete framing became popular in California immediately after the 1906 earthquake. Significant reinforced concrete shear-wall buildings without moment frames evolved in the 1920s. It was not until the late 1960s and early 1970s, after substantial research at the University of Illinois, that the benefit of confined concrete columns and ductile concrete frames was recognized and adopted for dependable seismic resistance.

In the 1970s and 1980s, a dual system of ductile concrete moment frames coupled with confined concrete shear walls was developed. This concept works best with a perimeter moment frame and an interior shear wall core, or the reverse, using a perimeter ductile wall and an interior ductile moment frame. Finally, a dependable concrete dual system was achieved.

Couple Walled System for Energy Dissipation Shear walls coupled with yielding shear links were developed in New Zealand in the 1980s after successful tests carried out by Park and Paulay (1975). This coupled-wall system is another excellent example of creative research used to develop a low-cost mechanism for seismic energy dissipation.

10.7.3 Substructure

Soil and bed conditions affect substructure seismic resiliency in the following ways:

- Building construction on steep slope or on an improper construction site on a foothill.
- Shifting or sliding off foundation (for example, during the 1987 Whittier Narrows earthquake).
- Lack of base isolation system: If a superstructure is not mounted on a base isolation system, its shifting on the basement should be prevented.

- Failure of retaining wall due to ground movement (example is Santa Cruz Mountains—area retaining walls during the Loma Prieta earthquake).

Lateral Spreading and Liquefaction

This mode of ground failure, termed lateral spreading, is a principal cause of liquefaction-related damage. Ground shaking—triggered soil liquefaction in a subsurface layer of sand produces differential lateral and vertical movement in sand and silt. This was observed in the aftermath of the Loma Prieta earthquake.

Settlement of Landfill leads to beam and column diagonal cracking, as was seen in the 2008 Sichuan earthquake.

10.8 PERFORMANCE LEVELS

FEMA has defined four discrete structural performance levels for buildings and two intermediate structural ranges for meeting rehabilitation objectives. As reported in ASCE/SEI 41-06, structural performance levels (SPL) and structural performance ranges (SPR) are defined as follows:

- S-1 **(Immediate Occupancy)** SPL: The structure is designed to remain safe or sustains little damage.
- S-2 **(Damage Control)** SPR: Damage level between S-1 and S-3 SPLs.
- S-3 **(Life Safety)** SPL: Significant damage but some margin of safety against partial or total collapse.
- S-4 **(Limited Safety)** SPR (between S-3 and S-5 SPLs).
- S-5 **(Collapse Prevention)** SPL.
- S-6 Structural Performance is not considered.

Hence Structural Performance Levels are S-1, S-3, and S-5 only. Structural performance ranges are S-2 and S-4 only.

The applicable maximum damage level for each component is selected from

- Collapse prevention (Type S-5 SPL)
- Life safety (Type S-3 SPL) and applicable minimum damage level for
- Immediate occupancy (Type S-1 SPL)

With the application of UBC, ASCE-07, IBCO, and California seismic codes the structural performance of buildings has improved considerably. Some of the planning factors contributing to safety are diagnosis, case studies, and detailed investigations—causes of building failures in major earthquakes like Loma Prieta, Northridge, or other well-documented events. Also contributors are dynamic loads, distribution of seismic loads, and improved assessment of seismic zones and PGA values through advanced seismology leading to higher magnitude of dynamic loads and vibrations. A bullet list is presented below:

- Improved formulae for equivalent lateral load for Zones 2 and 3
- Releasing seismic forces by providing isolation bearings and snubbers
- Allowing for symmetry in plan of buildings
- Avoiding discontinuity: Providing bond beams around doors and window openings
- Box formation: Adding shear walls along building perimeter to provide a tube effect
- Unified behavior of horizontal and vertical members: Improving strength of connections between roof and walls and floors and walls

Key performance issues: These are elastic behavior, inelastic behavior, and the related cyclic behavior resulting from pushing a structure back and forth as stable, nondegrading, predictable, and capable of dissipating a large amount of seismic energy. Contribution from non-structural members is not considered.

Elastic Design in Linear Systems. The simple building code approach to seismic design requires diminishing an acceleration spectra plot by use of an *R* value (defined earlier). *R* value is used to modify the acceleration spectral value to a simple seismic design force based on the nature of short duration load and ductility of the materials.

Post-Elastic Design for Nonlinear Drift. Nonlinear drift impacts structural and nonstructural behavior. For significant seismic energy dissipation, the drift should be large, but for favorable nonstructural or content behavior this drift should be small. Inelastic design is a better indication of realistic lateral drift or deflection that results from real motions.

Cyclic Behavior. A good measure of seismic performance is stable cyclic hysteretic behavior. The plot of load vs. deformation of an element, for motion in both directions, represents cyclic behavior. A stable system that is ductile has sufficient capacity to deliver a constant level of energy dissipation during the shaking imposed.

10.8.1 Performance of Low- and High-Rise Buildings

For low- and medium-rise buildings, displacements are primarily in the fundamental mode; for tall buildings, they are excited by ground shaking. There are significant differences in the behavior of buildings of different heights that influence seismic design. For example, wind design may control in tall buildings and strength at the base may be controlled by flexural moments from wind distribution, although shear forces may increase from higher modes. For serviceability, minimum stiffness is controlled by wind forces. As for low-rise buildings, the overturning moment is controlled by a common height not exceeding 20 feet. For high rise buildings, dual system of shear walls and moment resisting frames as described earlier should be utilized.

The dynamic forces can be controlled to some extent by the use of a lightweight structural system. The smaller the mass, the smaller the lateral forces. Therefore, any mass in the building should be necessary and seismically functional. (See Figure 10.4.). The location of columns in the building were optimized to minimize seismic forces for a research study project. Number of columns or frame spacing were varied for an economical design.

Figure 10.4 Planning a low-cost lightweight single-story building.

10.9 CONCLUSIONS ON SEISMIC DESIGN OF BUILDINGS

For seismic resistance, the size and scale of a building should determine the appropriate structural solution. As building mass increases with the use of heavy concrete floors, the mass increases many times. The greater the number of floors, the higher is the mass factor. For example, the same frame or brace that works well at two stories no longer works well at ten stories. Also, structural components when simply scaled up, no longer behave in the same fashion.

Avoiding Irregular Planning: Seismic codes are oriented toward an economical building design. They impose increased cost and poor performance penalties on the use of irregular configurations due to torsion and increased design forces. For larger buildings, more advanced analytical methods would give fewer cost penalties. Only two irregularities are banned outright in essential buildings in high seismic zones: extreme soft stories and extreme torsion.

A design that has attributes of the ideal configuration should be used under the following conditions:

- It includes design and analysis for code conformance.
- It utilizes simplicity of seismic detailing.
- It utilizes repetition of structural component sizes and placement conditions.
- The most economical design and construction are needed. An approved software should be used for seismic analysis.
- When the best seismic performance for the lowest cost is needed
- When maximum predictability of seismic performance is desired.

Damping Effects: Appropriate systems do not degrade in strength and have ample sustainable damping. There are several ways to introduce damping into a structure.

An increase of damping is the most positive solution for reducing seismic demand. It reduces vibration amplitude similar to hydraulic shock absorbers in an automobile and reduces structural demand.

Redundancy: It is also important to add redundancy or multiple load paths to a structure to improve seismic resistance. If carefully selected,

multiple systems can each serve a purpose such as limiting deflection or drift, providing strength, and increasing damping. Multiple systems also serve to protect the entire structure by allowing failure of some elements without endangering the total building.

Introducing Base Isolation: Seismic energy dissipation is the ultimate test of good seismic-resistant design. Large building displacement is required for good energy dissipation, while minimum displacement is required to protect the many brittle nonstructural components in a building. Only seismic base isolation adequately solves both aspects of this problem. Seismic isolation is an ideal solution for irregular buildings and unusual or creative building forms that are difficult to design with conventional structural systems.

Reclassify Some Older Buildings: Due to an increase in understanding of seismic response of buildings and combined with parallel changes in code requirements, many older buildings may be reclassified as seismically deficient.

Impact of Non-structural Members: Composite action of various non-structural members in distributing seismic forces to structural members needs to be investigated. Refer to IBC 2010, Chapter C13 for requirements for piping and other attachments to primary members and adequacy of fire prevention, short-circuit and gas leakages etc. during an earthquake should be ensured.

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CHAPTER ELEVEN

Performance-Based Design and Retrofit Methods for Buildings

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11.1 INTRODUCTION

While new design deals with future construction, retrofit deals with existing buildings. Some newer buildings may require seismic redesign in their lifetime because of changing codes and technology. The primary owner-designer issue is the relative cost of retrofit versus replacement, which is based on condition, importance, and risk factors. Hence, risk-based priorities must be established on a building-by-building basis. Also, local regulations and construction permits have legal constraints. Options available for existing, even new, buildings are rehabilitation, retrofit, renovation, and repair (refer to Figure 11.1). For existing buildings, retrofits are desirable either in anticipation of an earthquake or in a more expensive post-earthquake upgrade.

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Figure 11.1 Earthquake damage of a confined masonry building during 1999 Tehuacan earthquake (as reported by EERI, 1999).

This chapter broadly reviews the need for seismic retrofit based on which it recommends further development of seismic analytical methods. Analytical methods are similar to that required for a replacement of buildings. While repairs are an inherent part of maintenance, retrofits involve upgrades and deviation from the original design. Detailed seismic resistant specifications for retrofit and rehabilitation, on similar lines to designing new structures, are required. Both 475 year and 2250 year events are typically considered.

A detailed literature review shows contributions by the following research organizations and those responsible for developing design codes:

Earhquakes: A new look at cracked masonry (Langenbach, 1992)

Buildings at Risk: Seismic Design Basics for Practicing Architects (AIA/ACSA Council on Architectural Research, 1992)

Seismic Evaluation and Retrofit of Concrete Buildings (ATC, 1996)

Building Seismic Safety Council (1997)

Recommended Provisions for Seismic Regulations for new buildings and other structures, part 1

NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings (EERI, 1992)

Reducing Risks of Nonstructural Earthquake Damage (FEMA, 1994)

Uniform Code for Building Conservation (International Conference of Building Officials, 1991)

Advancing Bridge Technologies in Research, Design, Construction and Preservation (MCEER, 2006)

OpenSees: The Open System for Earthquake Engineering Simulation (PEER, 2010)

Seismic Rehabilitation (SEAOC/FEMA, 1996)

The following authors have contributed to several aspects in the form of research papers and text books, which are unique and based on research. Latest selected publications up to 2012 and the state of art related to seismic design is presented here:

Design of multi-Storey Reinforced Concrete Buildings for Earthquake Motions (Blume et al., 1961)

Performance-Based Seismic Engineering: Past, Current and Future (Freeman, 1998)

IEP and CSCE, Post Construction Analysis of Lateral Beams and Walls of Reactor Building of a Nuclear Power Plant (Khan, 1986)

Bricks, Mortar, and Earthquakes; Historic Preservation vs. Earthquake Safety (Langenbach, 1981)

The Great Hanshin-Awaji Earthquake: The Experience and Reconstruction (Masahiko, 2003)

Comparison of Recent U.S. Seismic Codes (McIntosh et al., 1997)

Structural Evaluation of Historic Masonry Buildings (Schuller and Atkinson (Noland))

Design of Concrete Structures (Winter and Nilson, 1986)

In addition to the above publications, relevant individual references are discussed in the text.

11.1.1 Performance-Based Seismic Design

A start toward performance-based seismic design (PBSD) has been made in seismic retrofit projects. PBSD is the latest trend in seismic design philosophy. PBSD was introduced by ATC 40 in 1996 and has been accepted by FEMA 358 (currently ASCE 41-06) and ASCE 7-10.

The scope of FEMA 349 includes development of structural, non-structural, and risk management guidelines supported by a development plan and a stakeholder's guide. As research results are made available, there will be greater refinement of applications of PBSD in future codes.

PBSE can be a statement of performance objectives or analytical methodologies. It facilitates innovation in design in the following ways:

- It makes it easier for designers to propose new building systems (not covered by existing code provisions).

- It extends the use of existing systems beyond code limitations.
- It considers two earthquake performance levels, elastic design level and limit-state inelastic design level.
- It provides a unified basis for comparison of design alternatives and gives decision makers a consistent means of quantifying risk.

Building codes can be considered a special type of PBSE. They too provide design forces for elastic design and detailing requirements for post-yield inelastic behavior (even though indirectly).

Recent seismic performance design has forced engineers to look at the full range of structural behavior, from linear to nonlinear to failure. Years of physical testing and corresponding analytical studies have furthered understanding of earthquakes, materials, and assemblies. Moreover, recent advances in engineering understanding of building performance during past earthquakes, laboratory tests, and structural analysis methods have been incorporated into design guidelines. PBSE covers dynamic analysis, inelastic response of structures, energy concepts (pushover analysis), and ductile reinforced-concrete detailing. Table 11.1 lists PBSD criteria.

11.2 A DIAGNOSTIC APPROACH TO RETROFIT

The first task in a proposed retrofit project is assembling a team to consist of an architect, a structural engineer, a code specialist, a geotechnical engineer, an environmental engineer, and a building contractor. The goal of the architect should be symmetry of planning and avoidance of soft-story effects and other dangerous conditions. The structural engineer selects materials and loads, performs detailed analysis and design, and outlines construction alternatives. The code specialist performs QA/QC to make sure that the applicable seismic design code provisions are followed.

Table 11.1 PBSD Criteria

Earthquake Levels	Objectives	Mean Return Interval (years)	Probability of Exceedance	Damping	Ductility
I	No damage	25	87%	5%	1.0
II	Moderate damage	72	50%	15%	2.0
III	Life safety	475	10%	30%	3.5
IV	No collapse	2,475	2%	40%	5.0

Note: Refer to FEMA 273, Chapter 26.

11.2.1 Retrofit Criteria

Retrofit criteria include unreinforced-masonry buildings (Rutherford and Chekene 1990), historic buildings (National Trust for Historic Preservation 1992, 2012; Look 2001), and important older buildings not designed to seismic standards. This includes the majority of buildings constructed before the 1970s. Because damage can be cumulative, it is important to analyze a building: structural capacity.

Retrofit criteria include

Duration of Earthquake. Aftershock tremors may result in crack formation.

Existing Building Condition. Earlier buildings constructed of concrete are often inadequately reinforced, inadequately tied, or both, and are thus susceptible to earthquake damage. Well-maintained buildings, even without added reinforcement, survive better than do buildings weakened by lack of maintenance. Cyclical maintenance, which reduces moisture penetration and erosion of materials, is essential. The capacity of the structural system to resist earthquakes may be severely reduced if previous alterations or earthquakes have weakened structural connections or if materials have deteriorated from moisture, termite, or other damage. Furthermore, in historic unreinforced-masonry buildings, deteriorated mortar joints can weaken entire walls. Over time, structural members can become loose and pose a major liability.

Building Construction Details. Construction materials, structural systems, and plan configuration may not offer an optimum solution. Rigid construction techniques may have seismic deficiencies.

Infilled Masonry. While structural frames may survive an earthquake, masonry infill can crack and in some cases dislodge.

Building Configuration. Smaller round, square, or rectangular buildings generally survive an earthquake because their geometry allows for equal resistance of lateral forces in all directions. The more complex and irregular the plan, the more likely that buildings, especially tall ones, will be damaged because of uneven strength and stiffness in different directions.

11.2.2 Assessing Building Condition and Risk Hazards

The retrofit team assesses the general physical condition of the building's interior and exterior and identifies areas vulnerable to seismic damage. Figures 11.2 and 11.3 show the need for laboratory testing. This often

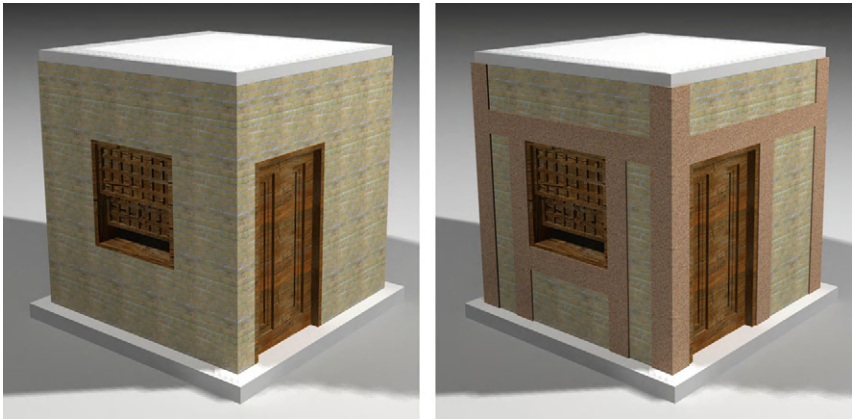

Figure 11.2 Prototypes of unconfined and confined masonry single room house built for shake table testing.

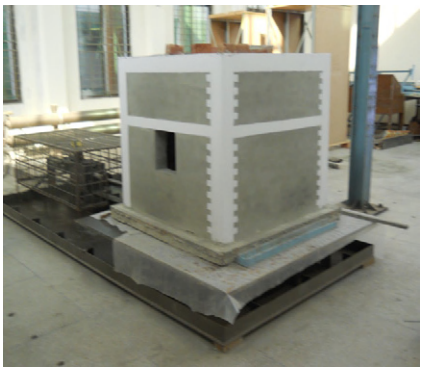

Figure 11.3 A scaled model of unconfined masonry single room house placed on shake table at UET Lahore Testing Facility.

requires determination of the strength and durability of materials and connections.

11.2.3 Compiling Documentation

All available documentation related to design must be reviewed. Original plans and specifications as well as those showing alterations through time often detail structural connections. Photographs of construction or of a building before and after previous earthquakes are invaluable. Base maps for geological or seismic studies and utility maps showing the location of water, gas, and electric lines should be also identified.

The more standardized building evaluation methods essentially require drawings. If detailed structural drawings are not available, simple evaluations of some model building types (wood, tilt-ups, and sometimes unreinforced masonry) can be performed based on layout drawings or from data prepared from field visits. Extensive field work is necessary to produce enough structural data to create a reasonable set of construction documents.

11.2.4 Assessing the Cost of Seismic Retrofit

Cost plays a critical role in selection of the most appropriate retrofit measure. It is always best to undertake retrofit measures before an earthquake occurs, when options are available for strengthening existing members. Once damage is done, the cost will be substantially higher, and finding engineers, architects, and contractors will be more difficult. The cost for a fully retrofitted building can offset the potential post-event loss of income, relocation, and rebuilding.

Some municipalities and states provide low-interest loans, tax relief, municipal bonds, or funding grants targeted to seismic retrofit. Planned seismic retrofit work may add between $50 and $150 per square foot to the cost of rehabilitation work depending on the level of intervention, the condition of the building, and whether work will be undertaken while the building is occupied. In some cases it may be less expensive to replace the damaged building since a building constructed to new technology lasts longer than an old one.

11.2.5 Developing a Retrofit Plan

The initial task of the project team is to develop a retrofit plan that may require alternatives to be considered. There are significant advantages to completing a seismic survey and analysis even if the resources for implementing a retrofit are not yet available. Long-term restoration solutions with future phases may be considered as funding becomes available.

Once the retrofit plan is finished, the project team will have a document by which to proceed. Some partially completed retrofit measures have left buildings more rigid in one area than in others, thereby contributing to more extensive damage during an ensuing earthquake. If construction is phased, its impact on the entire building should be understood.

Options for the level of seismic retrofit generally fall into the following classifications, depending on the expected seismic activity and the desired level of performance:

Basic Life Safety. Upgrades may include bracing and tying the most vulnerable elements, such as parapets, chimneys, and projecting ornamentation.

Enhanced Life Safety. Upgrades follow a flexible approach to building codes for moderate earthquakes. Inherent deficiencies found in older buildings, such as poor floor-to-wall framing connections and masonry walls, would be corrected. After a design-level earthquake, some structural damage is anticipated, such as masonry cracking, and the building will be temporarily unusable.

Immediate Occupancy. This option is intended for designated hospitals and emergency preparedness centers so that they can remain open and operational after a major earthquake. Most buildings, particularly commercial ones, can incorporate seismic strengthening during other construction work.

11.2.6 Seismic Strengthening

Seismic strength within buildings is achieved through the reinforcement of structural elements such as

- Anchored ties
- Reinforced mortar joints
- Braced frames
- Bond beams
- Moment-resisting frames
- Shear walls
- Horizontal diaphragms

11.3 SEISMIC EVALUATION OF INDIVIDUAL BUILDINGS

The majority of evaluations are now tied to the general procedures of ASCE 31-03, *Seismic Evaluation of Existing Buildings* (ASCE 2003), which has become a national standard and is in compliance with the 1973 Uniform Building Code (UBC) or equivalents. There are three basic categories of retrofit measures:

- Modification of global behavior, usually decreasing deformations (drifts)
- Modification of local behavior, usually increasing deformation capacity

- Adequate wall-to-floor ties
- Connectivity

Connectivity consists of ensuring that individual elements do not become detached and fall, ensuring a complete load path, and ensuring that the force distributions assumed by the designer will occur. Careful placement of new lateral elements may minimize a connectivity issue such as diaphragm deficiency.

11.3.1 Modification of Global Behavior

Modification of global behavior normally focuses on deformation. This may take the form of adding strength and/or stiffness with shear walls or braced frames. A soft or weak story is usually eliminated by evenly spreading the deformation demand over the height of the structure. Seismic isolation provides redistribution of deformation to bearings placed at the isolation level that are specifically designed for such a response. The bearings ensure elastic response of the superstructure for maximum loads. Isolation also controls large movements of nonstructural systems and contents.

11.3.2 Modification of Local Behavior

A local retrofit can be done in a variety of ways. Columns in frames and connections in braces can be strengthened. Concrete columns can be wrapped with steel or concrete to provide confinement and shear strength. Concrete and masonry walls can be layered with reinforced concrete, plate steel, and other materials. Composites of glass or carbon fibers and epoxy enhance shear strength and confinement in columns and provide strengthening to walls.

A method to protect against collapse posed by excess drift is to provide a supplementary gravity support system for concentrated wall-supported loads. This is a requirement in California standards for retrofit of unreinforced masonry buildings.

Deformation capacity can be enhanced locally by placement of vertical saw cuts in unreinforced masonry walls to create slots between spandrel beams and columns.

11.3.3 Failure Modes

Typical earthquake damage to historic buildings results from

- Poor ductility
- Poor structural connections between walls, floors, and foundations

- Heavy weight and mass of materials
- In masonry buildings that have not been seismically upgraded,
- Parapets, chimneys, and gable ends may dislodge and fall to the ground during a moderate earthquake.
- Walls, floors, roofs, skylights, porches, and stairs, which rely on tied connections, may fail.
- Interior structural supports may partially or totally collapse.
- Unreinforced masonry walls between openings may exhibit shear or diagonal cracking.
- Upper stories may collapse onto under-reinforced lower floors with large-perimeter openings or atriums.
- Unbraced infill material between structural or rigid frame supports may dislodge.
- Ruptured gas and water lines often may cause fire and water damage.

Three levels of evaluation are briefly described below:

Initial Evaluation (ASCE 31 Tier 1)

The ASCE Tier 1 evaluation is similar to FEMA's rapid evaluation in that it is based on model building type and certain building characteristics. The significant difference is that structural drawings, or data equivalent to structural drawings, are required to complete it. After identifying the appropriate FEMA building type, a series of prescriptive requirements are investigated. If the building is found to be noncompliant with any requirement, it is potentially seismically noncompliant or deficient. (See ASCE 2003.)

Intermediate Evaluation (ASCE 31 Tier 2)

The ASCE intermediate level of evaluation, called Tier 2, is similar in the level of effort of historical nonstandardized methods. Normally, an analysis of the whole building is performed and the equivalents of stress checks are made on important lateral force—resisting components. ASCE 31 includes the requirements for both Life Safety and Immediate Occupancy performance levels for a Tier 2 evaluation.

Detailed Evaluation (ASCE 31 Tier 3)

The most common method used in Tier 3 is a performance evaluation using FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA 2000) based on simplified nonlinear pushover analysis. This method approximates the maximum lateral deformation that the

building will suffer in a design event, considering the nonlinear behavior created by yielding and damage to components. The level of deformation of individual components is compared with standard deformations preset to performance levels of Collapse Prevention, Life Safety, and Immediate Occupancy.

Efforts are being made to more realistically relate the damage states of all components to a global damage state. The ultimate goal is to simulate the movements of full buildings during an earthquake, including the constantly changing properties of the structural components due to yielding and damage. The overall damage to various components is then accumulated and the global damage state thereby surmised.

11.3.4 Data Required for Seismic Evaluation

For methods depending on FEMA building types, the building type must be known. Crude expectations of performance and therefore comparative evaluation can be completed. Most "rapid" evaluation methods, based on building type and basic building characteristics, do not require structural drawings. Responsible evaluators will insist on a site visit. However, evaluations can be refined by more detailed data when available for building age, physical condition, and configuration.

When warranted, site-specific studies can be performed to obtain timely and locally derived data. However, other seismic site hazards, such as liquefaction, landslide, and surface fault rupture, are less well mapped and may require a site-specific study if there is reason to suspect their potential. Formal evaluation techniques, such as ASCE 31, have addressed this issue.

11.4 MODEL BUILDINGS AND MODEL RETROFIT

If occupancy is significantly increased (say from a transformation of a warehouse to office space), building codes require updating of life-safety features in number or level of hazards. As early as the mid-1960s, this concept was beginning to be applied to seismic systems.

11.4.1 The FEMA Standard Model Building

In 1985 FEMA created a set of model building types that are defined by a combination of gravity- and lateral-load—carrying systems. A local

inventory is divided into these types. The majority of buildings, new or old, can now be assigned a type.

11.4.2 Conceptual Design of a Retrofit Scheme for an Individual Building

The selection of a specific retrofit scheme depends on local cost, availability, and suitability for the structure in question. Any system that resists lateral loads in new buildings can also be used for retrofit. The five basic issues of concern to building owners or users are seismic performance, construction cost, disruption during construction, long-term effects, and aesthetics. Objectives that include a very limited amount of damage or "continued occupancy" severely limit the retrofit methods that can be used and may control the other four issues.

Adjacent Buildings

If contact is expected, short adjacent buildings can cause a soft-story effect on the levels immediately above them. If floors do not align, load-bearing columns or walls can be damaged by pounding, potentially causing collapse. If walls share a structural ("common") wall, interaction may be extreme, and individual analysis is required. Taller buildings, particularly Unreinforced masonries (URMs) or buildings with URM exterior walls or parapets, may drop debris on shorter buildings.

These conditions are difficult to mitigate without cooperation from both property owners. Potential contact areas can be strengthened, but this may cause additional damage to neighboring buildings. Supplemental vertical load systems can be installed to prevent collapse caused by local damage. The potential for falling debris from taller buildings can be minimized by adding supports and ties on adjacent buildings, and failure of the roof can be minimized with roof reinforcing.

Configuration of Soft/Weak Stories

Disproportionate drift is concentrated on the soft story, potentially causing collapse. A weak story may not be initially soft, but after yielding as a story first, it becomes soft and displacements concentrate in those elements that have already yielded, potentially causing collapse. The most straightforward retrofit is to add elements to the soft or weak story to force displacement to be more evenly distributed throughout the building height. In some cases, it is possible to soften other stories to be more evenly matched.

Discontinuous Wall/Brace

Shear walls or braces that do not continue to the foundation create forces at the level of discontinuity that must be designed for, including shear forces that must be transferred into the diaphragm and overturning tension and compression that must be resisted from below. New walls or braces can be added. The transfer forces can be made acceptable by reinforcement of the diaphragm and/or strengthening of columns below the ends of the wall/brace.

Setback

A setback often creates a dynamic discontinuity because the story below is often much stiffer than the one above. This discontinuity can create larger than expected demands on the floor immediately above. The floor above the setback can be strengthened to accept and smooth out dynamic discontinuity.

Plan Irregularity

Plan irregularities such as L or T shapes often displace the center of mass from the center of lateral rigidity, causing torsion and resulting in high drifts on some elements. Re-entrant corners often present in these buildings create large demands on floor diaphragms, tending to pull them apart at these negative corner conditions. Lateral force—resisting elements can be added to balance mass and resistance. Chords and collectors can be added in diaphragms to resist re-entrant corner forces.

Unreinforced Masonry

Not all the buildings built before the introduction of seismic codes are hazardous, but most of them are likely to suffer more damage than modern seismic-resistant buildings. URM buildings (a popular building type of early twentieth century), for example, are now recognized as perhaps the worst seismic performers as a class. In California, URM was outlawed in zones of high seismicity in the 1933 code. Even some of the masonry buildings designed to "modern" seismic codes may be susceptible to high damage levels and even collapse.

Severely cracked or leaning walls are some of the most common earthquake damage. Also hazardous is the damage that may occur between the walls and roof or floor diaphragms. Separation between the framing and the walls can jeopardize the vertical support of roof and floor

systems. The following deficiencies were observed in several earthquakes and are known to be responsible for causing substantial damage to an unreinforced masonry building:

- Lack of reinforcement to resist bending tension
- Poor mortar
- Inadequate roof-to-wall ties
- Soil liquefaction

Another deficiency is the soft-story effect; soft-story collapse due to inadequate shear strength at ground level was observed during the Loma Prieta earthquake in 1989. The absence of adequate shear walls on the ground level caused damage to structures. Case studies showed that only the rigidity of the floor above combined with the support on the two hidden sides by continuous walls, not penetrated with large doors as on street sides, prevented full collapse.

Liquefaction exists where soil consists of loose granular materials with the tendency to develop excessive hydrostatic pore water pressure of sufficient magnitude and compact. It can cause nonuniform settlements and tilting of structures. This caused major damage to thousands of buildings in Niigata, Japan, during an earthquake there in 1964.

11.4.3 Damage to Unreinforced Masonry Buildings

Among structural types, unreinforced masonry is most vulnerable when located in seismic zones. The type of hazardous damage normally observed during earthquakes can be severely cracked or leaning walls, damage that may occur between the walls and roof or floor diaphragms. Separation between the framing and the walls can jeopardize the vertical support of roof and floor systems.

11.4.4 Drift

Drift may be minor or severe and varies between 0.5% to 5%. It can be permanent or transient. Differential settlement may also take place. Widths of concrete cracks may vary.

11.4.5 Traditional Brick, Block, and Adobe Construction

Traditional construction materials rely on the weight of the masonry and the bonding capacity of the mortar to provide structural stability. The criteria used for historic buildings are more stringent than for new building planning and design. They include the following

- Bracing needs to be installed without damaging decorative details or appearance of parapets, chimneys, or balconies.
- Diagonal frames, such as X, K, or struts should have minimal impact on the primary façade.
- Visible features of exterior buttresses, reinforcements, and anchor washers should be adequately designed to blend with the building's appearance.
- Shear walls should be located in utilitarian interior spaces to reduce the impact on finishes in the primary areas.
- Thinner applied fiber-reinforced coatings should adequately strengthen walls or supports without the need for heavier reinforced concrete.
- Adequate funds are required to retain, repair, or reinstall ornamental finishes once strengthening or retrofit has been installed.
- Base isolation and wall damping systems should protect significant materials and reduce the amount of intervention.
- Seismic treatments should be reversible in a way that allows the historic materials to be retained and allows future repair and restoration.

11.5 MEASURES TO REDUCE VIBRATIONS

Seismic Vibration Control Devices: These mitigate seismic impacts. They may be classified as in the following way:

- *Passive control devices* have no feedback capability between themselves, the structural elements, and the ground.
- *Active control devices* incorporate real-time recording instrumentation on the ground integrated with earthquake input processing equipment and actuators within the structure.
- *Hybrid control devices* have combined features of active and passive control systems and are referred to as tuned mass dampers (TMD) for tuned (*passive*), as active mass dampers (AMD) for *active*, and as hybrid mass dampers (HMD) for *hybrid mass.*

Damping Methods to Control Seismic Waves: After seismic waves enter a superstructure, there are a number of ways to control them in order to minimize drift, vibrations, and oscillations:

- Elevating the building foundation
- Dissipating the wave energy with dampers
- Dispersing the wave energy

- Absorbing the resonant portion′s mass dampers

Seismic dampers have been successfully installed in high-rise buildings, predominantly in Japan and the United States. Well-known types are TMD, AMD and HMD.

Base Isolation Method: Seismic or base isolation is partial suppression of the seismic energy flow from the ground into the superstructure. The objective is to substantially decouple the superstructure from its substructure. The earliest use of the principle of base isolation was in sixth-century B.C. Pasargadae, a city in ancient Persia.

11.6 FEMA REHABILITATION PROCEDURES

FEMA encourages hazard mitigation projects, including the restoration of historic buildings, by providing technical assistance and funding through the Hazard Mitigation Grant Program (HMPG), which can underwrite up to 50% of the cost of a project.

11.6.1 FEMA Program to Reduce Seismic Risk in Existing Buildings

In 1985, FEMA recognized that the principal seismic risk in this country is located in the existing building stock, the majority of which was designed without adequate seismic provisions. FEMA programs were significant in enabling communities to understand and mitigate their seismic risk, most notably in the development of the regional loss-estimating computer program, HAZUS.

FEMA's public-assistance program provides financial and other assistance to rebuild disaster-damaged facilities that serve a public purpose, such as schools, hospitals, government buildings, and public utilities. FEMA, in cooperation with the Building Seismic Safety Council, has produced comprehensive materials dealing with the seismic retrofit of existing buildings. A project with ATC-43 involves earthquake analysis procedures for unreinforced-masonry, reinforced-concrete, and historic buildings.

In the 1990s, FEMA sponsored the development of guidelines for the seismic evaluation and rehabilitation of existing buildings that introduced methods to inform the future conceptual basis of codes for new buildings. Nonlinear analysis, an analytical method that integrates the deformation of a structure into the analysis of a structural design, was identified as an

essential tool for some seismic design applications; the concept of performance goals was also introduced.

11.6.2 Required Steps in Design

The required steps are (1) dissipate seismic energy without failure; (2) create a low post-earthquake repair cost; (3) introduce stable cyclic behavior in seismic system; and (4) control lateral drift.

11.6.3 FEMA Publications for Seismic Retrofit Techniques

Post-earthquake repairs and retrofits are addressed in FEMA publication 172, *NEHRP Handbook of Techniques for Seismic Rehabilitation of Existing Buildings*, 1992, which outlines the basic methods of seismically strengthening a building, including conceptual details of typically added structural elements. Other publications of interest are FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, 1997, and FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*.

11.6.4 ATC

The ATC-20-3 report, *Case Studies in Rapid Post-earthquake Safety Evaluation of Buildings*, contains 53 case studies of specific buildings evaluated using the ATC-20 Rapid Evaluation procedure, including updates described in the ATC-20-2 Addendum. These case studies include 21 from the 1989 Loma Prieta earthquake and 12 from the 1994 Northridge event. Rapid evaluation is the first, and many times the only, safety evaluation performed. ATC's report is intended to be used as a training and reference manual for safety evaluations.

11.7 CATEGORIES OF REHABILITATION

According to FEMA recommendations, the primary focus of a viable retrofit scheme is vertically oriented components (columns, walls, braces, etc.) because of their significance in providing either lateral stability or gravity-load resistance.

11.7.1 Connections

Connections are the weakest points in the link. There may be large deformations due to their weakness especially when they are within the

load path. A building with a relatively weak or brittle load path needs retrofit by seismic isolation to keep the load below the brittle range.

11.7.2 Seismic Vibration Control

Seismic vibration control is a set of technical means aimed to mitigate seismic impacts in building and nonbuilding structures. As discussed earlier, all seismic vibration control devices may be classified as passive, active, or hybrid, where passive control devices have no feedback capability among themselves, the structural elements, and the ground. Their purpose is as follows:

- To dissipate wave energy inside a superstructure with properly engineered dampers.
- To disperse wave energy among a wider range of frequencies.
- To absorb the resonant portions of the whole wave frequencies band with the help of so-called mass dampers.

11.7.3 Configuration Irregularity

Configuration irregularity is largely responsible for two undesirable conditions: stress concentration and torsion. These conditions often occur concurrently.

Stress Concentration. Irregularities tend to create abrupt changes in strength or stiffness that may concentrate forces in an undesirable way.

Torsion. Configuration irregularities in plan may cause torsional forces to develop, which cause significant uncertainty in analysis and are perhaps the most frequent cause of failure.

Four serious configuration conditions (two vertical and two in plan) originate in the architectural design and have the potential to seriously impact seismic performance:

- Soft and weak stories
- Discontinuous shear walls
- Variations in perimeter strength and stiffness
- Re-entrant corners

Soft-First-Story Failure

The most prominent of the problems caused by severe stress concentration is the "soft" story, a term that has commonly been applied to buildings whose ground-level story is less stiff than those above. There is a difference between "soft" and "weak" stories. Soft stories are less

stiff, or more flexible, than the stories above; weak stories have less strength. A soft or weak story at any height creates a problem, but since the cumulative loads are greatest toward the base of the building, a discontinuity between the first and second floors tends to result in the most serious condition. A soft story may be created by an open first floor that supports heavy structural or nonstructural walls above. This situation is most serious when the walls above are shear walls acting as major lateral force—resisting elements.

The best solution to the soft- and weak-story problem is to avoid the discontinuity through architectural design. There may, however, be good programmatic reasons why the first floor should be more open or higher than the upper floors. In these cases, careful architectural/structural design must be employed to reduce the discontinuity.

Discontinuous Shear Walls

The discontinuous shear wall is a fundamental design contradiction: the purpose of a shear wall is to collect diaphragm loads at each floor and transmit them as directly and efficiently as possible to the foundation. To interrupt this load path is undesirable; to interrupt it at its base, where the shear forces are greatest, is a major error. The solution to the problem of the discontinuous shear wall is unequivocally to eliminate the condition.

11.7.4 Symmetry

A building's seismic behavior is strongly influenced by its perimeter design. If there is wide variation in strength and stiffness around the perimeter, the center of mass will not coincide with the center of resistance, and torsional forces will tend to cause the building to rotate around the center of resistance. A common instance of an unbalanced perimeter is an open-front design in buildings such as fire stations and garages in which large doors provide for passage of vehicles.

One solution to this problem is to reduce the possibility of torsion by balancing the resistance around the perimeter. A second solution is to increase the stiffness of the open façades by adding sufficient shear walls, at or near the open face, designed to approach the resistance provided by the other walls. A third solution is to use a strong moment-resisting or braced frame at the open front that approaches the solid wall in stiffness.

11.7.5 Re-Entrant Corners

The re-entrant corner is common in buildings that are in the shape of an L, T, H, and the like, or that combine these shapes. These shapes tend to produce differential motions between different wings of the building that result in local stress concentrations at the re-entrant corner. They also cause torsion because the center of mass and the center of rigidity cannot geometrically coincide for all possible earthquake directions. The result is rotation.

Solutions for the Re-Entrant Corner Condition

There are two basic solutions to the problem of re-entrant corners: structurally separate the building into simpler shapes or, in smaller buildings only, tie it together more strongly with elements positioned to provide more balanced resistance. Structurally separated entities of a building must be fully capable of resisting vertical and lateral forces on their own, and their individual configurations must be balanced horizontally and vertically.

11.7.6 Buildings May Fall Down, Not Overturn

Although building mass or weight is part of the $F = \mathrm{MA}$ equation for determining horizontal forces, there is another way in which building weight may act under earthquake forces to overload the building and cause damage or even collapse.

Vertical members such as columns or walls may fail by buckling when the mass of the building exerts its gravity force on a member distorted or moved out of plumb by lateral forces. This phenomenon is known as the *P-e* or *P*-delta effect, where *P* is the gravity force or weight, and *e* or "delta" is the eccentricity or the extent to which the force is offset. All objects that overturn do so as a result of this phenomenon.

Geometrical proportions may also influence whether the *P*-delta effect will pose a problem, since a tall, slender building is much more likely to be subject to overturning forces than a low, squat one. It should be noted, however, that if lateral resistance is provided by shear walls, the proportions of the shear walls, not those of the building as a whole, are significant.

Another undesirable condition exists when a shear wall is perforated by aligned openings for doors, windows, and the like, so that its integrity is compromised. Some types of perforated shear wall with unaligned openings have performed well.

11.8 EARTHQUAKE SIMULATIONS IN ANALYSIS AND DESIGN

Theoretical or experimental evaluation of anticipated seismic performance generally requires a structure simulation based on structural likeness or similarity. Similarity is some degree of analogy or resemblance between two or more objects. The notion of similarity rests either on exact or approximate repetitions of patterns in the compared items. The following structural groups must be classified into different categories for modeling:

- High-rise buildings, which have a tendency to overturn
- Buildings with girders supported on corbels and without moment connections, or with spread footings not tied in both directions
- Buildings without shear walls (if the number of girders is minimum, there is poor redundancy) and little resistance to collapse if lateral resistance is lost at an edge
- Unsymmetrical buildings with irregular frames or skew in plan.

Chapter 9 has a full discussion of simulations and modeling that applies to both buildings and bridges.

11.9 RETROFIT PRIORITIZING

Determination of retrofit priorities is based on structural vulnerability such as in girder connections and footings, weak soils, seismic and geotechnical hazards, and building importance. As discussed in Chapter 9, the seismic rating system is both quantitative, based on structural vulnerability and seismic hazard, and qualitative, based on an overall priority index consisting of importance, remaining useful life, nonseismic deficiencies, and redundancy. (Refer to Chapter 9 for a full discussion of retrofit prioritization using VRS as well as the liquefaction vulnerability scale (LVS). The focus of that chapter is on bridge retrofit, but prioritization evaluation for bridges and buildings is similar.)

11.9.1 Determining Vulnerability

The vulnerability rating scale (VRS) can be summarized in reverse order as

- Not applicable (no exposure to specific type of vulnerability)
- No action (likelihood of failure is remote)

- Inspection program action (low priority based on inspections and monitoring)
- Capital program action (priority for remedial work based on capital program)
- Safety program action (high priority for remedial work)
- Safety priority action (immediate priority for remedial work)

11.9.2 Ensuring Safety through Retrofitting

Based on ratings such as VRS and LVS, repairs and retrofit are recommended to achieve required safety factors. General considerations include planning, funding and cost, functional requirements, soil conditions, geometry, constructability, alternate design, aesthetic requirements, new technologies, and innovative methods. Methods include isolation bearings, dampers, restrainers, galvanized wire mesh wall reinforcement, and snubbers. Other methods now being employed include corebraces to strengthen existing frames (Figure 11.4).

11.10 NEW DEVELOPMENTS IN SEISMIC RETROFITTING

Electronic sensors that detect seismic shaking can tell the building how to react to avoid damage. Buildings with those sensors have been built in Japan. Some use accelerometers, which are also found in newer smart phones, to detect motion. If a sensor's input exceeds a certain level, a damper

Figure 11.4 As part of continuation education program between ASCE Philadelphia Section and Institution of Engineers, Lahore, Pakistan, the author as Director of ASCE Philadelphia awarding certificate of attendance to a young engineer, on completion of one day workshop on disaster management.

system goes into action and reduces the amount of shaking. A prototype sensor made of flexible skin-like fabric equipped with electrical properties adheres to areas of structures where cracks are likely to appear, such as the underside of a bridge.

Much less technical are solutions such as reinforcing concrete buildings with steel rods and bolting wooden buildings to their foundations. Reinforcement was used in the columns of the reinforced-concrete ductile moment–resistant space frames in a modern 30-story building completed in 1984. The Pacific Park Plaza Building is located in Emeryville, California. Reinforcement used in the columns provided ductile moment-resistant space frame and this modern 30-story building was completed in 1984.

11.11 NONSTRUCTURAL COMPONENTS

Like structural components, nonstructural components should be safe. Nonstructural systems and components are all of the parts of a building that do not lie in the primary load-bearing path; that are not part of the seismic-resisting system; and that are designed to support their own weight, which is then transferred to the primary structural system. Among them are ceilings, doors and windows, piping, chimneys, bookshelves, and furniture.

The dead weight of each component is an important fraction of a building's mass and needs to be kept to a minimum so that lateral forces are not excessive. Movable items such as book shelves and computer equipment should be securely tied. Exposed electric wiring and power cables should be insulated so that any short circuit during violent shaking does not start fires.

Rigid nonstructural walls spanning structural columns change the local stiffness of the structural system and alter its response, possibly creating a stress concentration. Partitions may suddenly be called upon to perform a supporting role.

The first standards for nonstructural components and systems, other than walls, parapets, and chimneys, were introduced in the mid-1970s. The provisions have grown to include a wide variety of nonstructural components and systems, but they have yet to recognize the need for qualification. The number and complexity of nonstructural systems and components far outweigh the structural components. Acceptance criteria

for rehabilitation of discrete nonstructural components are based on Operational, Immediate Occupancy, Life Safety, and Hazards Reduced performance levels.

Seismic forces increase as they travel upward in a building and are transmitted to nonstructural components at interface with the structure. Many nonstructural systems and components are often very flexible, in contrast to the relatively rigid building structure. This flexibility often leads to a much higher level of excitation.

Injuries are generally due to falling hazards, such as with large sections of plaster ceilings, HVAC registers, lights, filing cabinets, and the like. Indirect threats include the inability of occupants to safely exit a building because of blocked doorways or stairwells.

Hospitals have a need for both continued function and reduction of economic loss. The owner-supplied equipment and contents they hold are often significantly more valuable than the building itself and will be more costly and take more time to repair than will equipment in an office building or school.

Heavy parapets should be braced back to the roof structure. This is a typical problem with unreinforced-masonry buildings, in which large parapets are unsupported. Seismic codes specify the size of ducts and length of supports that require seismic bracing.

The IBC 2003 seismic code deals with the problem of nonstructural components in two ways. First it imposes limits on the horizontal drift or deflection of the main structure. This is to prevent nonstructural damage (such as glass breakage or fracturing of piping) resulting from too much flexibility, causing racking in wall panels, partitions, and glazing framing. The allows for considerably reduced forces to be used in design, but this solves the structural problem at the expense of the nonstructural components. The imposition of drift limits ensures that the flexibility of the structure will not be such that excessive nonstructural damage results.

Second, it assigns force values based on acceleration to the critical nonstructural components and their connections to ensure that they will be strong enough to resist seismic accelerations in their own right or as the result of attachment to the structure.

The analysis of nonstructural components is similar to that for determining equivalent lateral force on the main structure. The basic $F = Ma$ equation takes into consideration importance, component amplification, component response modification, and component height. All of these modifiers increase the design forces relative to the spectral accelerations

derived from hazard maps, so an exemption is allowed (ICC 2003) for all components in Seismic Design Category A structures because they are subject to lower seismic effects.

11.11.1 Practical Considerations with Nonstructural Components

Base isolation has limitations and is subject to special conditions, such as the need for utilities to accommodate large lateral movement where they enter a building above or below ground. In general, base isolation reduces, but does not eliminate, the effects of horizontal motions.

Historically, earthquake codes paid little attention to the vertical component of shaking generated by earthquakes. As a rule of thumb, the maximum vertical ground motion is generally 60% to 70% of the maximum horizontal ground motion. However, while it may generally be unnecessary to consider the vertical motions of the structure as a whole, this is often not the case with nonstructural design. The building, usually because of its architectural configuration, can act as an amplifier for both horizontal and vertical motions. Even though codes most often do not require vertical design resistance, the designer must be cognizant of the implications of vertical motions and their potential effects.

Tall shelves need longitudinal bracing and attachment to the floor. The top bracing should be attached to the building structure and strong enough to resist buckling when the shelves attempt to overturn.

11.11.2 Mechanical and Electrical Components

Mechanical and electrical components include plumbing, communication, and furniture and contents. They are treated differently in seismic codes depending on weight, mounting height, importance, and whether or not they have flexible connections to associated ductwork, piping, and conduits. Detailed guidelines for the design of seismic restraints for mechanical, electrical, and duct and pipe are to be found in FEMA publications 412, 413, and 414.

Piping Supports. Equipment mounted on spring vibration isolators needs to be fitted with "snubbers," especially in nuclear power plants. Snubbers are faced with resilient material that cushions impacts.

Laser Scanning (Johnson and MacQuarrie 2012). Unlike residential buildings, manufacturing plants have a maze of piping, wiring, ducts, valves, cable trays, and other equipment. After an earthquake, existing piping drawings may not be applicable and there are difficulties in

inspection caused by obstructions. Retrofit and restoration of piping is very difficult and expensive. Still, the addition of MEP seismic restraints can prevent much of the secondary damage in the aftermath of earthquake such as failure of sprinkler system coupling.

Use of standard bracing components for seismic restraint is one option. Another is to scan MEP systems with a 3D laser scanner and create 3D (point cloud) models. Scans are quick and give an accurate picture of damage; construction drawings can easily be prepared from a CAD library. Three-dimensional visualization programs such as AutoDesk's® NavisWorks® can be used by the contractor. Laser scanning can also be useful in mapping of existing architectural and structural systems.

11.11.3 Architectural Nonstructural Components

All architectural components (except parapets supported by bearing or shear walls) in Seismic Design Category B are exempt if they have an importance factor of 1.00, which indicates that they are not a life safety threat. (The importance factor is a 1.5 multiplier for components that are needed after the earthquake for life safety, that contain hazardous contents, or that are large storage racks open to the public.) The importance factor is selected by the engineer based on criteria in the code and consultation with the building official.

Of all elements in the building envelope, heavy precast concrete wall cladding panels attached to steel or reinforced-concrete-frame structures require the most design and construction attention to ensure seismic safety. These typically span from floor to floor. Horizontal drift or deformation of the building structural frame can create considerable racking forces in panels that are rigidly attached at top and bottom, resulting in damage or possible drop-off. The attachment of these panels must permit differential movement of floors without transmitting racking forces. This is achieved by special detailing of connections between panels and structure.

Seismic codes require that heavy panels accommodate movement either by sliding or ductile connections. In high seismic zones sliding connections are rarely used because of the possibility of incorrect adjustments when bolts are used and in cases of jamming or binding due to unwanted materials left after installation and jamming due to a geometrical change in the structural frame under horizontal forces. The need for disassociating the heavy panel from the frame has a major impact on connection detailing.

As a result, a connection commonly termed "push-pull" has been developed, primarily in California, which provides, if properly engineered and installed, a simple and reliable method of decoupling the panel from the structure. The generic connection method consists of supporting the panel by fixed bearing connections to a structural element at one floor to accommodate the gravity loads, and using ductile "tie-back" connections to a structural element at an adjoining floor.

Notable Earthquake Damage

Because of pounding against adjacent buildings during the 1989 Loma Prieta earthquake in California the five-story St. Joseph's Seminary in Los Altos collapsed, causing one fatality. Pounding depends on differences in the adjacent buildings' lateral displacements and period, which should be accurately estimated and accounted for.

Effects of Completely Shattered Joints in a Concrete Frame

During the 1994 Northridge earthquake the joints in the Kaiser Permanente building, (located in the neighborhood of Los Angeles, California) a concrete-frame structure, completely shattered, revealing inadequate confinement steel, which resulted in the collapse of the second floor. In the transverse direction, composite end shear walls, consisting of two wythes of brick and a layer of shotcrete that carried the lateral load, peeled apart because of inadequate through-ties and failed (Figure 11.5).

Figure 11.5 Damage to Kaiser Permanente Building after Northridge 1994 Earthquake.

The most spectacular major building collapses were typically of reinforced concrete-frame buildings designed and built prior to about 1975, although some post-1975 parking garages collapsed or were severely damaged. The near-total collapse of a department store in

Northridge, and the partial collapse of a five-story medical building in Granada Hills—adjacent to an undamaged hospital—are two such examples. In these cases, as well as in others, the older concrete frames had inadequate strength and reinforcing details. website: http://foreshock.wordpress.com/significant-la-area-earthquakes-1769-present/

Also in the Northridge quake, a seven-story, reinforced-concrete building on a steep slope collapsed for the following reasons:

- Improper construction site on a foothill
- Poor detailing of the reinforcement (lack of concrete confinement in the columns and at the beam–column joints, inadequate splice length)
- Seismically weak soft story at the first floor
- Long cantilevers with heavy dead load

Observations about earthquake damage can be listed as follows:

Concrete Frames: cracking and hinge formation; limited cracking; splice failure in nonductile columns; severe damage in short columns; spalling in frame; joint damage; buckling of reinforcement.

Steel Moment Frames: distortion of beams; distortion of column panels; fracture at moment connections

Braced Steel Frames: extensive yielding of braces; buckling of braces; connection failure

Concrete Walls: flexure cracks; shear cracks; sliding at joints; failure around openings; reinforcement buckling; boundary element damage; coupling beam damage; shattered panels

Reinforced- and Unreinforced-Masonry Walls: cracking, crushing; damage around openings and at corners; shattered panels

Wood Stud Walls: splitting of members and panels; loose connections; withdrawn nails; split and fractured framing

Concrete Diaphragms: cracking; crushing

Precast Diaphragms: crushing and spalling at joints; unit shifting; connection failure

11.12 REPAIR AND RETROFIT OF NONENGINEERED BUILDINGS

Generally it is not economically feasible to repair or strengthen damaged rural housing. Replacing it with simple, well-detailed reinforced-concrete or reinforced-masonry dwellings is preferable. However, some masonry structures are of great historical interest while

others may have to be strengthened because there is no place for their inhabitants. In any case, a systematic approach to the problem is necessary.

11.13 SEISMIC RETROFIT OF HISTORIC BUILDINGS

Engineers, architects, code officials, agency administrators, and building owners need to be actively involved in preparing, modifying, and applying various methods of seismic retrofit. Project personnel working together can ensure that the architectural, engineering, financial, cultural, and social values of historic buildings are preserved while rendering them safe for continued use.

11.13.1 Principles and Practice of Preservation

The following important preservation principles should be followed when undertaking seismic retrofit projects:

- Reinforcing a historic building to meet new construction requirements can be introduced sensitively. In such cases, its design, placement, patterning, and detailing must conform to the building's historic character.
- Historic materials need to be preserved, not replaced, in process of strengthening.
- New seismic retrofit systems should conform to the character and integrity of the historic building and be visually compatible with it in design.
- Seismic work should be "reversible" to allow removal for future use of improved systems.
- Federal tax incentives should be in place for the rehabilitation of income-producing historic buildings, including seismic strengthening.
- Both primary and secondary spaces, features, and finishes should be identified. For a historic building it is a great challenge to protect major features, such as domes, atriums, and vaulted spaces or highly decorative elements, such as mosaics, murals, and frescoes.

11.13.2 Case-by-Case Study

Rehabilitation is generally the most cost-effective time to make major upgrades that affect a building's structural performance. New elements

such as concrete shear walls or fiber-reinforcing systems can be added while the structure is exposed. The seismic retrofit is as follows:

Step 1. Inspect and improve all lateral tie connections and diaphragms.

Step 2. Reinforce walls and large openings to improve shear strength in locations of doors, windows, and storefront openings. Carefully locate "X" and "K" bracing to avoid visual intrusion, or use moment frames, which are hidden in large openings. From a preservation perspective, the use of a more hidden system in finished spaces is generally preferable.

Step 3. Strengthen masonry walls or columns with new concrete reinforcement or fiber-wrap systems. Avoid the use of heavy spray concrete or projecting reinforced walls that seriously alter the historic relationship of the wall to windows, trim, and other architectural moldings or details.

Step 4. Selectively locate new shear walls for continuous transfer of loads from the foundation to the roof. If these walls cannot be set behind historic finishes, they should be located in secondary spaces in conjunction with other types of reinforcement of the primary spaces or features.

Step 5. Consider the internal grouting of rubble masonry walls using an injected grout mixture that is compatible in composition with existing mortar. Ensure that exposed areas are repaired and that the mortar matches all visual qualities of the historic mortar joints in tooling, width, color, and texture.

Step 6. Evaluate odd-shaped buildings and consider the reinforcement of corners and connections instead of infilling openings with new construction. Altering the basic configuration and appearance of primary façades of buildings is damaging to the qualities that make the buildings architecturally significant.

It has been established that well-maintained buildings fare better than those in poor condition, during and after an earthquake. Thus, maintenance and seismic retrofit are two critical components for the protection of historic buildings in areas of seismic activity. Few codes consider historic buildings, but the California State Historical Code and the Uniform Code for Building Conservation provide excellent models for jurisdictions to adopt. Code officials should always be asked where alternative approaches can be taken to provide life safety if the specified requirements of a code would destroy significant historic materials and features. Professionals on the retrofit team should be prepared with alternatives

that allow for mitigating potential damage to such features while retaining them through reattachment or strengthening.

11.13.3 Specialized Technologies in Historic Building Preservation

ATC-43 outlines earthquake analysis procedures for unreinforced-masonry buildings and reinforced-concrete buildings. These entail nationally applicable technical criteria to ensure that buildings will withstand earthquakes better than before. There is a great deal of information that is applicable to historic buildings. Preparedness as a tool for Disaster Management will help preserve historic buildings. A one day workshop was offered by the author to practicing engineers in Lahore, Pakistan (Figure 11.6).

New technologies are being developed all the time and many may have applicability to historic preservation projects. These include vertical and center core drilling systems for unreinforced-masonry buildings, base isolation at foundations, superstructure damping systems, bonded resin coatings, and reproduction of lost elements in lighter materials. Using computer modeling of how historic buildings may act in an earthquake suggests options for seismic upgrade using a combination of traditional and modern technologies.

While most projects involving base isolation and other complex damping systems constitute only a small percentage of projects nationwide that are seismically reinforced, they may be appropriate for buildings with

Figure 11.6 As part of continuation education program between ASCE Philadelphia Section and Institution of Engineers, Lahore, Pakistan, the author as Director of ASCE Philadelphia awarding certificate of attendance to a young engineer, on completion of one day workshop on disaster management.

significant interior spaces that should not be disturbed or removed during the retrofit. Each building will need its own survey and evaluation to determine the most appropriate seismic reinforcement.

The retrofit team should be knowledgeable of historic preservation goals for historic buildings, building code provisions, and alternative solutions. Team members should also have experience with similar projects. Local and state building officials can identify regulatory requirements regarding historic preservation or building conservation code.

Damage to historic buildings *after* an earthquake can be great. In this case they must be quickly shored up and stabilized before the sensitive work of rehabilitation can begin. Without earthquake hazard reduction plans in place, communities put their historic buildings—as well as the safety and economic well-being of their residents—at risk. Historic and older buildings can be seismically upgraded in a cost-effective manner while important character-defining qualities are retained.

Questions to ask when considering and retrofit technology for historic buildings include

- Can unsightly exposed reinforcement, such as X braces, within the immediate viewing range of the public be avoided?
- Can shear walls be located in utilitarian interior spaces?
- Should base isolation, wall damping systems, or core drilling be considered?
- Are the seismic treatments being considered "reversible" in a way that allows future repair and restoration?
- Can moment frames or reinforced bracing be added around historic storefronts?

11.14 CONCLUSIONS ON PERFORMANCE BASED DESIGN AND RETROFIT METHODS FOR BUILDINGS

PBSD criteria are applicable to building design and retrofit and are the latest stage in the evolution of seismic code provisions. They are still in their development phase, but it has been established that they make it easier for designers to propose new building systems not covered by existing code provisions, and they extend the use of existing systems beyond code limitations. Also, they consider both elastic design and limit-state inelastic design performance levels. More broadly they provide a unified basis for comparison of design alternatives and give decision makers a consistent means of quantifying risk.

Post-earthquake building rehabilitation entails a choice of repair, retrofit, or replace. Seismic retrofit is the second design, or redesign, of a building because of changes in seismic criteria. Various retrofit measures are available, such as isolation bearings and dampers. Generally, the variety of retrofit methods and diverse options requires an action plan. The retrofit team must deal with replacement design, design of nonstructural components, retrofit of historic buildings using special criteria, retrofit of existing buildings based on new seismic criteria and priority, and retrofit of nonengineered and adobe construction.

Since there are no design codes specifically for retrofit, ATC, FEMA, and NEHRP guidelines and specifications for new construction can be useful for a variety of construction issues such as the use of steel, concrete, masonry, and timber. Research related to new construction such as shake tables and laboratory analyses on scale models to better understand failure modes will promote the development of safe retrofit designs. Currently, bracing parapets, tying buildings to foundations, and anchoring brick walls at the highest, or roof, level are a few simple retrofit measures, along with devices to reduce vibrations. Nonstructural components influence the response of buildings during earthquakes and also require retrofit.

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APPENDIX A

Example of Seismic Design of Buildings with QA/QC Check List

A.1 THE COST FACTOR FOR SEISMIC DESIGN

Historic buildings always receive special attention for a long-term design by allowing indirectly for a major earthquake. They therefore cost more than traditional buildings per square foot. Buildings like the Taj Mahal have survived many earthquakes over a long period due to long term safety in structural planning, symmetry and providing adequate seismic resistance of each member.

The construction manager on a number of reinforced concrete building projects (in which the author was involved for a number of years) was concerned about increases in reinforcement costs, ductile moment resisting frames detailing and costs of footing tie beams for upper Zone 2 seismicity. ACI, BOCA, and UBC codes were adopted for these projects. For a reinforced concrete building, ACI Code had made minor changes to some of the load combinations of IBC.

A.1.1 General Procedure for Design of a Building in Seismic Zone

Factors to be Considered in Detailed Design:

1. Seismic load combinations: Environmental effects including wind or thermal forces need to be considered.
2. Design method such as ASD or USD or both.
3. Ductility: Connections design and detailing allowing for ductility, which is in-built carrying capacity of the structural system, of distributing the peak loads and formation of plastic hinges, without causing sudden collapse.
4. Lateral force resisting systems such as frames and shear walls.

5. Irregular building configuration.
6. Damping effects from non-structural members.
7. Need for Retrofit: Evaluation of structural elements and for lower values, strengthening or retrofit required.
8. For a low rise building less than five stories in height, only fundamental mode rather than higher modes be considered.
9. Use of Response Modification Factor R: Due to composite action in building floors and frames, advantage can be taken of the energy absorbing capacity of the redundant load path system. As a result the actual base shear will be less than the computed value. Also the peak base shear is instantaneous and damping will reduce it to zero, in a matter of seconds.

   A response modification factor = Computed base shear/Reduced base shear can be used leading, to scale down moments and forces and enable achieving an economical design costwise.
10. Columns should be checked for P-Delta effects or lateral column deflection.

The author carried out comparative designs for buildings located in a seismic zone and in a zone with low seismicity. Single story building for mass scale low cost construction (but located in higher seismic zones) and four storied reinforced-concrete buildings were considered for increase in cost of building for higher zones. *STAAD-3* software was used. Moment and shear force analysis from the Handbook Method was compared with the computer stiffness matrix method. Refer to analytical and design methods, described in Figures 10.1 to 10.3 of Chapter 10.

The builder's concern was a valid one since 4-storied buildings do not have elevator shafts to act as shear walls. The dead weight of plastered concrete walls increases the mass and lateral forces. The thick perimeter walls are used for insulation against extreme weather. Solid walls extending the full height of the building are 9 in. thick with additional inside and outside plaster of 2 in. They are supported on deep plinth beams with moment connections to the columns generally located at ground level. With 0.18 g lateral force, the cumulative wall seismic moments on the spread footing are added to the frame moments. Also, the member sizes and reinforcement in beams, columns, and footings increase, and the soil-bearing pressure may be exceeded, requiring a larger footing size. Individual footings are tied together by buried beams. Unlike timber buildings in the United States, the weight of the concrete building is many times greater. However, they last longer than timber buildings. The load combinations for a typical

building with 2 rows of 4 symmetrically placed columns as per earlier local building code were as follows (Figure A.1):

A.1.2 Load Combinations

(Table 10.3 compared International Building Code 2009 load combinations with AISC, ASCE and ACI Code combinations).

In the example given, alternative UBC basic load combinations recommended by an earlier code were used.

For ASD unfactored loads:

Combination of *DL* (or D) plus

LL (or L) plus

WL (or W) or

EL (or E).

A reduced E value may be used such as E/1.4 for ASD.

Seismic Effects: Dynamic forces due to inertia of a building's vibrating elements are idealized as equivalent static forces, acting at the joints of the building frames.

Load combination with earthquake forces would normally govern for higher seismic zones compared to lateral wind forces. Location of building near a fault rupture of ground needs to be taken into consideration.

A geotechnical report is needed to evaluate soil behavior during earthquakes. Soil profile type will provide the range of shear wave velocity. Liquefaction effects need to be considered when water table is high and would influence foundation settlement.

Response spectra may be utilized for selecting acceleration coefficients.

Preparation of Site Data Prior to Analysis:

Seismic load in transverse or longitudinal direction Using IBC,

For USD:1.4D + 1.7L:

Equation 8.7 (based on ACI 3-8), Merritt et al. 1996. Standard Handbook for Civil Engineers, McGraw-Hill (conservative factors from earlier ACI code prevalent at the time of design, resulted in safe but slightly over seismic design).

Please note that the load combination which gave the highest moments, forces and deflections was used to design member size and reinforcement.

0.75(1.4D + 1.7L + 1.7W) (Equation 8.8a, Merritt's Handbook)

0.75(1.4D + 1.7L + 1.87E)

0.9D + 1.43E

0.9D + 1.3W (Equation 8.8b Merritt's Handbook)

Additional Data Compilation for Seismic Loads Design: More information is needed compared to wind lateral loads.

1. Occupancy category and Importance Factor
2. Acceleration Maps of Area
3. Response Spectral Coefficients
4. Site class and geotechnical soil type
5. Response Modification Factor R
6. Seismic Coefficient C_s used for computing Design Base Shear ($V = C_s \cdot W$ and its Vertical distribution).
7. Modal Analysis Method used.

Comparison of seismic and nonseismic design: Khan (1987)

- For seismic design, maximum beam and column moment show an increase of 100% over nonseismic moments.
- Plinth beam moments supporting the walls are much higher.
- Column width and beam depth increase by about 6 inches.
- Footing sizes increase by 30% depending on allowable bearing pressure.
- Increase in beam longitudinal and shear reinforcement is over 20%.
- Increase in column reinforcement is nearly 50%.
- The handbook method from the empirical formulae given in Code of Practice can be used for preliminary design only. A computer-based stiffness matrix should be used for final design.
- The structural cost of the building increased by 25 to 30% compared to a building in a nonseismic zone.
- The large variations shown are for symmetric buildings only. For non-symmetric frames,
- Differences with a nonseismic design would be even greater.

A.1.3 Frame Analysis and Design for the Solved Example (Also refer to: Alan Williams, 2003)

Equivalent Static Lateral Force

$$V = \frac{CvI}{RT} W$$

Equivalent Static Lateral Force $V = CsW$ where

W is the combination of dead and live load of the structure (10 kips in the example);

R is the response modification factor for a specific structural system. $R = 8.5$ and is for concrete special moment-resisting frame.

The formula just given is based on the assumption that the structure will undergo several cycles of inelastic deformation and energy dissipation without collapse.

Beam Design

$$\text{Assume } b = \frac{1}{2}d$$

$$\text{Wt} = \frac{(12)(h)}{144}(\textit{concrete wt})$$

Moment Due to Uniform Load

$$\frac{Mu}{\phi bd^2} = \rho f_r^{\gamma}\left(1 - \frac{1}{1.7}\frac{\rho f_r^{\gamma}}{f_r^{c}}\right)$$

$Pu = 1.2(Dead\ Load + beamload) + 1.6(Live\ Load) + 1(Earthquake\ Load)$

(conservative factors used based on earlier code compared to IBC, 2009).

$Pu = .80[.85f'c(Ag - Ast) + fyAst]$– ACI Equation 10.2

$$Mu = \frac{MuL^2}{8}$$

Computing the R value,

$$\rho = \frac{0.85(f_c^{'})}{fy}\left(1 - \sqrt{1 - \frac{2Rn}{0.85f_c^{'}}}\right)$$

$$Rn = \frac{Mu}{\phi bd^2}$$

(For source of equations, refer to Text book by McCormac and Nelson, 2005, Sec. 3.4, Derivation of Beam Expressions. Also, McCormac and Russell Brown, 2009)

Selecting reinforcing area,

$$As = \rho bd$$

Select beam size using data collected from previous formulas

$$\frac{Mu}{\phi bd^2} = \rho fy\left(1 - \frac{1}{1.7}\frac{\rho fy}{f_c^{'}}\right)$$

(Refer to McCormac and Nelson, 2005, Sec. 4-3 Beam Design Examples)

Solve for bd^2

(Refer to McCormac and Nelson, 2005, Sec. 4-3 Beam Design Examples)

Column Design

When designing for a column we took an additional $\rho\ fy$ load into account, which is now included in the dead load. That load is for the beam, which was calculated with the previous formulas.

First we had to use the required strength formula to find the cumulative load to be applied on the column, since the column will be holding a majority of the load. We had to be very careful in this calculation.

Required Strength

$$Pu = 1.2(DL) + 1.6(LL) + 1(EL)$$

(From ACI Equation 9-3 or Equation 9-5)

The following equation was used to find the dimensions of a square tied column:

$$Pn(\max) = .80[.85f'c(Ag - Ast) + fyAst]\ \text{(ACI 318 Code Equation 10-2)}$$

Column Loads

The plan encompassed the dimensions of the house and the spacing of the columns. Each column was given a letter and a number according

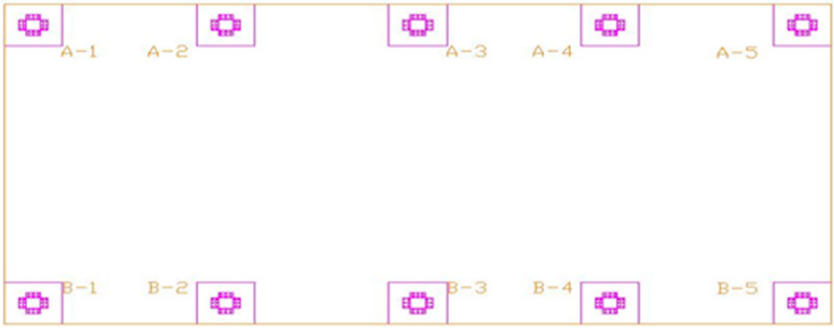

Figure A.1 Plan layout of 2 rows of columns to calculate tributary area.

to its arrangement in our structure (Figure A.1). The tributary area of each column was calculated. This was the area supported by each column. The dotted area in Figure A.1 illustrates the tributary area over a corner column in our structure. We utilized a common engineering method, which was to assume that the column would support a load area halfway to the next column. Columns with similar tributary areas were then grouped together. In total, there were three groups of column types:

A1, B1, A5, B5 (corner columns)
A3, B3 (center columns)
A2, B2, A4, B4 (last but one column grid)

ASD Load Combinations

$DL + LL + LL$-roof
$DL + LL +$ wind
$DL + LL +$ seismic/1.4
$DL + LL +$ wind + snow/2
$DL + LL +$ snow + wind/2
$DL + LL +$ snow + seismic/1.4

A table was made for each grouping of columns. An example can be seen in Table A.1., which has the sum of loads on the corner columns of A1, A5, B1, and B5.

The structure shown was one level, so the roof and the floor were the only levels supported by the columns. For each level, the tributary area (T.A.) was multiplied by the design dead load and live loads (ksf), which resulted in dead and live point loads (kips). Additional dead loads from wall, column, and beam weights were calculated using the following equations:

Dead Load Calculations

$$W_w(\text{lbs}) = t_w(\text{ft}) \times h_w(\text{ft}) \times W_c(\text{lb/ft}^3) \times \text{T.A.}$$
$$P_c(\text{lbs}) = A_{cs}(\text{ft}^2) \times h_c(\text{ft}) \times W_c(\text{lb/ft}^3)$$
$$\text{P}_\text{b}(\text{lbs}) = \text{W}_\text{b}(\text{lb/ft}) \times \text{L}_\text{b}/2\ (\text{ft})$$

All the additional dead load calculations were converted into kips and added to the point dead loads. Finally, the total dead and live loads were summed up in separate columns.

Table A.1 Computed Loads on 12 × 14 in. Columns

Columns A1, B1, A5, B5				Point Load (kips)		Additional D.L. (kips)			Total Column Loads		
Level	**T. A. (ft^2)**	**Design *LL* (ksf)**	**Design *DL* (ksf)**	***LL* kips**	***DL* kips**	**Wall Weight**	**Col Weight**	**Beam Weight**	***LL* kips**	***DL* kips**	
Roof	44.7	0.03	0.1	1.341	4.47	5.4	0.842	0.329	1.341	11.041	12.382
Floor	44.7	0.04	0.1	1.788	4.47	0	0	0.329	1.788	4.799	6.587
									3.129	15.84	

Connections Design

Beam to Column: The column size is 12 × 14 (in.) and the beam size is 12 × 24 × 12 (in.). This is a pin connection, and the two elements will be secured by ¼-in-thick steel plate with six holes that are in ¾ in. in diameter. The column elements are made from precast concrete.

Column to Foundation: The column size is 12 × 14 (in.) and made from precast concrete. The foundation size is 7 × 7 × 2 (ft) and made from cast-in-place concrete (Figure A.2)

Looking at the figure, it can be seen that this is partially fixed connection. The steel plate placed in the center of the foundation is ½-in. thick with four holes that are 1 in. in diameter. The column will sit on top of the plate and will be secured by four steel brackets that are ½-in. thick. With each bracket, it will consist of six holes that are ¾ in. in diameter.

Roof to Column: The elements will be secured by four ¼-in-thick steel plates with six holes that are ¾ in. in diameter (Figure A.3).

Plinth Beam to Column: The plinth beam is connected to the column with two steel plates. Looking at the bottom left of the figure (the corner

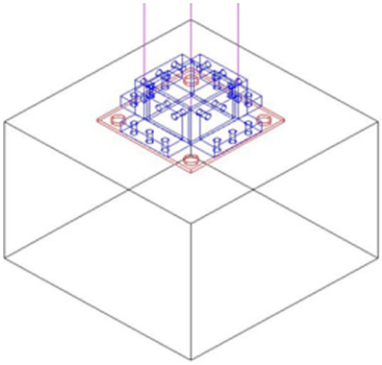

Figure A.2 Typical Column-to-foundation connection with steel angles and anchor bolts.

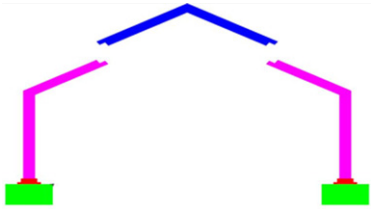

Figure A.3 Roof-to-column connections with beam joints (3 precast members used for ease of transportation).

connection), we see that each plate has six holes that are ½ in. in diameter. The plinth beam is 24 in. high from the foundation and will sit the floor and the wall.

A.1.4 Results

After analyzing six different structural frames, it was found that the sloped roof is a better frame to use because of the arch-effect type of minimum deflection as opposed to the flat roof's tendency to dip down. Furthermore, the sloped roof is best for rain and snowfall since it enables this precipitation to run off, resulting in less loading on the roof. The two-pinned connections at roof beam level are favorable because they facilitate construction; the frame could come in three pieces and be constructed like a puzzle. Moreover, the analysis on the two pinned connection frames resulted in lower frame moments and deflections.

Using lightweight aggregate concrete in the design reduced dead and seismic loads from structural members, which allowed design of smaller members. A lighter structure with smaller members dissipates energy induced by earthquakes far better than a heavier structure can. Lighter structures are also more economical to construct. With this Zone 3 design, the structure should not fail suddenly and will undergo minimal damage.

A.1.5 Notations Used in the Equations

A_{cs} = cross-sectional area of column
A_g = gross cross-sectional area of a concrete member
A_s = reinforcing-steel area
A_{st} = total area of longitudinal reinforcing
B = effective width of beam flange
b_o = perimeter of critical section for shear for slabs or footings
c = distance from extreme compression fiber to neutral axis
C_a = ground response coefficient
C_v = velocity-based coefficient
C_t = 0.30 for reinforced-concrete moment-resisting frames
C_s = seismic response coefficient
d = effective depth of section measured from extreme compression fiber to centroid of tensile reinforcement
d_b = bar diameter
DL = dead load

E = earthquake load or seismic load
f'_c = specified compression strength of concrete
f_y = specified yield strength of concrete
h = total thickness of member
h_n = roof height
h_w = wall height
h_c = height of column
I = important factor
K_{tr} = transverse reinforcement index
L = length of beam
L_b = beam length
l_d = development length of straight bar embedded in confined concrete
M_u = moment due to uniform load
P_b = column load from beam weight
P_c = column load due to self weight
q_e = effective soil pressure
q_u = bearing pressure
R = response modification factor
R_n = term used in required percentage of steel expression for flexural member
T.A. = tributary area for load distribution on column
T = fundamental period of vibration
t_w = wall thickness
W = seismic dead load
W_b = weight of beam
W_c = unit weight of concrete
W_w = wall weight
V = Seismic base shear
V_u = total shear

Greek Symbols:
β = Stress reduction factor
Φ = capacity reduction factor
λ = lightweight aggregate concrete factor
ρ = ratio of prestressed reinforcement in section
α = reinforcement location factor

Interior and Exterior Walls

Interior walls will be designed with typical studs and drywall, and the frame of the wall will be nailed to the floor slab. Since exterior walls will be subjected to seismic loads, they will be designed with fiber-reinforced-polymer concrete with lightweight aggregates.

Quality Control Checks for Code Provisions in Calculations Document and Construction Drawings:

An example of Inventory checklist of design calculations is presented below.

NAME OF FACILITY: SUBMITTED BY: DATE: CHECKED BY:				
	SUBMISSION REQUIREMENTS	**YES**	**NO**	**REMARKS**
	1. SOIL INVESTIGATION REPORT/ RECOMMENDATIONS FOR FOUNDATION TYPE/ SOIL TREATMENT (LIQUEFACTION EFFECTS / NO WETLANDS)			
	2. DESIGN REPORT INCLUDING			
a.	Codes & Standards (IBC/UBC/ASCE 7/ STATE CODE)			
b.	Material strength & properties (Laboratory Test Results)			
c.	Approved hazard maps used for acceleration factors			
c.	Backup calculations and checks provided for input data used			
STRUCTURAL DRAWINGS LIST WITH TITLES (Signed and sealed by a Licensed Engineer)				
COMPREHENSIVE GENERAL NOTES				
	3. SPREAD OR CONTINUOUS FOOTINGS/PILES AND PILE CAPS			
a.	Foundation plan			
b.	Minimum thickness of footing slab provided is 18″			
c.	Depth of footing from NSL provided corresponds to the requirement laid down in soil investigation report			
d.	Negative Reinforcement provided in case of combined footings			
e.	Minimum Reinforcement in footing as per ACI 7.12.2.1			
f.	Column transverse Reinforcement terminating 18″ into footings are per ACI 21.4.4.5			
g.	90″ hooks of column bars into footings with bars oriented towards center of column for fixed end condition ACI 21.10.2.2			
h.	Area of footing provided is adequate for soil bearing pressure			
	1. REINFORCED CONCRETE SHORT COLUMNS			
a.	Shortest cross sectional dimension >12″ ACI 21.4.1			
b.	Minimum & maximum Reinforcement ration satisfy ACI 21.4.3.1			
c.	Lap splices within centre half of column ACI 21.4.3.2			
d.	Transverse Reinforcement enclosing lap slice confirms to ACI 21.4.4.2			
e.	Lap length provided comply with ACI 21.2.7			
f.	Transverse Reinforcement throughout length of column satisfying ACI 21.4.4.4			
g.	Cross ties provided to unattended bars in column as per ACI 21.4.4.3 & 7.10.5.3			
h.	Ties shall have a 135° hook at ends with 6db or 3″ extension whichever is greater as per ACI 7.10.5.3			
i.	Reinforcement provided is as required in design			
	2. REINFORCED CONCRETE ROOF AND FLOOR BEAMS			
a.	Width to depth ration >0.3, width >10″ as per ACI 21.3.1			
b.	Minimum Reinforcement at every section is not less that given in ACI 21.3.2.1			
c.	Lap splices are not used within joints & within 2h from column face ACI 21.3.2.3			
d.	Hoop spacing in lap splices is not exceeding d/4 or 4″ as per ACI 21.3.2.3			
e.	Maximum spacing of the hoop at every section is as per ACI 21.3.2.3			
f.	Reinforcement provided is as required in design			
	7. BEAM-COLUMN JOINT CONNECTION DETAILS			
a.	Column dimension parallel to beam Reinforcement. >20db and vise versa ACI 21.5.1.4			
b.	Beam Reinforcement is anchored in tension & compression into column far face as per ACI 21.5.1.3			
	8. BRICK/CEMENT BLOCK HOLLOW MASONRY WALLS			
a.	1#4 vertical bar is provided at both sides of openings			
b.	2#4 bars are provided after every 8th course			
c.	If blocks are used size is not less than 8″ for main walls & 6″ for partition walls			
d.	Wall footing is provided for partition walls			

Figure A.4 Sample inventory of quality control checklists for reconstructed buildings on a mass scale.

	1. WALL COLUMN JOINTS			
a.	2#4 bars with a 2′ lap is provided at both faces of the wall column joint.			
	2. ROOF AND FLOOR SLABS			
a.	Minimum thickness of one way & two way slabs as per ACI 9.5.2.1 & 9.5.3			
b.	Minimum reinforcement in slabs as per ACI 7.12.2.1			
	3. STEEL TRUSSES			
a.	Structural steel confirming to AISC Grade 50			
b.	All high strength connection bolts confirming to A325 & all others confirm to A307			
c.	Welding is done in accordance with AWS structural welding code D1-11			
d.	Elevations of all trusses as shown in truss layout plan/sections is given			
e.	Sizes of gusset plates for all the connections is given			
	4. PREFABRICATED STRUCTURES			
	LIST OF PROPRIETARY DRAWINGS WITH DETAILS			
	5. LITERATURE FROM APPROVED SUPPLIER COVERING:			
a.	Technical and commercial aspects			
b.	Performance characteristics of joints and connections			
c.	Period since this structural system has been in use			
d.	Precedents of the use of this structural system in other seismic areas of the World			
e.	Details of attachments of non structural elements with structural system			
	6. AN APPROVED SOFTWARE USED: SAP, STAAD, ETABS	**Yes**	**No**	**Remarks**
	3. 3-D COMPUTER MODEL SKETCHES PROVIDED			
	4. SEISMIC ANALYSIS & CORRECT SEISMIC PARAMETERS			
LOAD APPLIED ON THE STRUCTURE ARE CORRECT				
a.	Seismic loads			
b.	Dead loads			
c.	Live loads			
d.	Snow loads			
e.	Wind loads			
	5. LOAD COMBINATIONS DEFINED			
	6. SECTION PROPERTIES AND DATA OF MEMBERS ARE SAME IN COMPUTER MODEL AND WORKING DRAWINGS			
	7. ALL MEMBERS PASSING UNDER GIVEN LOAD			
a.	Columns			
b.	Beams			
c.	Truss members			
	8. REACTIONS FROM TRUSSES ARE APPLIED ON MAIN MODEL			
	9. DESIGN OF FOOTINGS GIVEN/ LIQUEFACTION CHECKS PROVIDED			

Figure A.5 Optional quality control checklist for structural information.

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APPENDIX B

Computer Software for Seismic Analysis and Design of Superstructure and Substructure

B.1 APPROVED SOFTWARE FOR SUPERSTRUCTURE

The following software has been approved by many Departments of Transportation and Highway Agencies in USA:

STAAD-Pro, Bentley Software, 2013 is the structural engineering software for steel, concrete, timber, including culverts, tunnels, bridges, piles, design

GT STRUDL, 2013 Structural Design Language information, Georgia Tech - CASE Center, Atlanta, Georgia

PSLRFD - Prestress Concrete Girder Design and Rating Program LRFD, PennDOT Software for concrete bridges, PA DOT Bridge Design and Technology Division, PenndotBisEngineer@pa.gov

STLRFD, PennDOT Software for steel bridges, PA DOT Bridge Design and Technology Division, elsavage@pa.gov

SPLRFD, Steel Girder Splice Design and Analysis Program, www.portal.state.pa.us/portal/

FBLRFD - LRFD FLOORBEAM ANALYSIS AND RATING www.portal.state.pa.us/portal/

CON/SPAN 2013 is a patented modular precast system for total set-in-place construction of bridges, culverts and under-ground structures, West Chester, Ohio, csinfo@con-span.com

CSI, SAP2000, CSi Bridge, ETABS, Contact CSI Sales, Berkeley, CA

OPENSEES, Open system for earthquake engineering software, University of California, Berkeley, opensees.berkeley.edu

ADINA, Automatic Dynamic Incremental Nonlinear Analysis, Finite Element Software, info@adina.com, Watertown, MA

ANSYS, Finite Element Analysis, ozeninc.com, Sunnyvale, CA
ABAQUS FEA software package, which includes high level non-linear Finite Element Analysis capability along with explicit dynamics, SIMULIA Advantage Support, Providence, RI, USA

B.2 APPROVED SOFTWARE FOR SUBSTRUCTURE

STAAD-Pro, Bentley Software, 2013
CSI, SAP2000, CSi Bridge, ETABS, Contact CSI Sales, Berkeley, CA
ABLRFD, PennDOT LRFD Abutment and Retaining Wall Analysis and Design Program, www.portal.state.pa.us/portal/
PAPIER – PENNSYLVANIA PIER ANALYSIS, www.portal.state.pa.us/portal/
WinSEISAB, Window Seismic Analysis of Bridges. It analyzes the response of a bridge subjected to earthquake excitation, Roy Imbsen, NISEE, Rancho Cordova, CA, ytthao@trcsolutions.com
COM624P: Laterally Loaded Pile Analysis Program for the Microcomputer. Version 2.0, User's Manual Publication Number: FHWA-SA-91-048, Ensoft, Inc., Austin, TX
L-PILE, analyzes a pile and pile group under lateral loading using the p-y method, ENSOFT INC., lcra@ensoftinc.com, Austin, Texas
spMats, (formerly pcaMats) Structure Point MATS is a program for analysis and design of concrete foundation mats, combined footings, and slabs on grade, http://www.structurepoint.org, PCA Skokie, IL
A number of bridge piers and abutments, located in Northeast USA, were analyzed and compared by the author using PennDOT and other approved software. The results are reported in the following publications:
Khan, Mohiuddin Ali, 2002, Seismic analysis of Bridge Substructures Using AASHTO and AREMA Code Methods, ASCE-SEI, Proceedings of International Conference, Denver, CO, 2002
Khan, Mohiuddin, 2009, Modeling and Seismic Analysis of Integral Abutments, ASCE-SEI Structures Congress, Nashville

Khan, M. A., and R. W. Dunne, 2008, Recent Developments in the Design of New Jersey Bridges Using Accelerated Bridge Construction Concepts, FHWA National Seismic Conference on Bridges and Highways, Baltimore

APPENDIX C

Red Cross and Recommended Survival Kits

The Red Cross has a variety of packages at a reasonable cost on their website. It is better to be safe than sorry. A small investment can be a secured life insurance if you live in a seismic zone.

C.1 EMERGENCY SUPPLIES

Have on hand:

- Emergency supplies of food and water (at least one gallon per day per person)
- Prescribed medicines for injuries in quantities sufficient for three days, rotated with normal stocks to keep supplies fresh
- Water replaced every six months.

C.2 PERSONAL PREMIUM DISASTER SURVIVAL KIT

This comprehensive kit contains essential products to insure your family's survival during any type of natural or man-made disaster designed with recommendations and guidelines set by experts in the disaster preparedness industry. All items are packed securely by *Survival Kits Online* in a deluxe duffel bag. The kit includes the following:

1 Large First Aid Kit—contains the essentials needed for any type of fist aid emergency

4 Mylar Sleeping Bags—Recommended over common "space blankets"

48 Pouches of Filtered Water from leading suppliers and each box having a 5-year shelf life

48 200-Calorie Food Bars—5-year shelf life

4 Personal Hygiene Kits includes Soap, Toothbrush, Toothpaste,
9 Handy Wipes
1 Hand Crank AM/FM/Band Radio never needs batteries
2 Hand Squeeze Rechargeable Flashlights
1 5-in-1 Survival Whistle—compass, signal mirror, flint starter, water-proof container, and shrill whistle
1 Box 50 Water-proof Matches
1 Fire Starter—Provides sufficient shavings to start hundreds of fires
4 Emergency Ponchos with Hoods
2 2-Person Tube Tent with Rope
1 16-Function Knife
1 Pair Leather Palm Gloves
1 50-Foot Rope
4 N95 Masks
8 Tissue Packs all-purpose
1 On-Duty/Gas Shutoff Wrench Designed by a firefighter. Use to shut off gas and water. Pry open doors and windows. Will not spark if struck against metal. Safe to use.
1 Can Opener
2 Waste Bags
1 2.5-Gallon Water Bag
4 Body Warmers—last for 16—20 hours
4 12-Hour Light Sticks
1 Portable Stove with Fuel Tablets
1 Survival Candle
40 Water Purification Tablets. Each tablet purifies 1 liter of water.

C.3 OTHER EMERGENCY ITEMS

Have ready and at hand:

- Fire extinguishers
- First aid supplies and manual
- Battery-powered radio
- Flashlights with extra batteries
- Nonelectric can opener.

C.4 UNSTABLE FURNITURE AND OTHER ITEMS

The following may fall from shaking:

- Bookcases, files, shelving, televisions, computers, heavy vases (and large objects on lower shelves)
- Glass, china, and bottles in low closed cabinets with latches
- Heavy pictures and mirrors hung away from beds and couches
- Overhead light fixtures to be tightly fastened
- Water heaters and movable furnaces to be strapped to walls
- Inflammable items and pesticides (placed on bottom shelves of latched cabinets).

APPENDIX D

Sample Problem: LRFD Method to Solve Reinforced Concrete Bridge Beam Seismic Problems

Adjust DL, spans, or E values to match sample solved examples OR REDO Similar New Problems by Changing Data Systematically. Also see Appendix A.

D.1 FLEXURAL AND SHEAR RESISTANCE FOR CONCRETE MEMBERS

Procedure: The structural strength of reinforced-concrete beams in bending and shear is computed using Standard AASHTO equations.

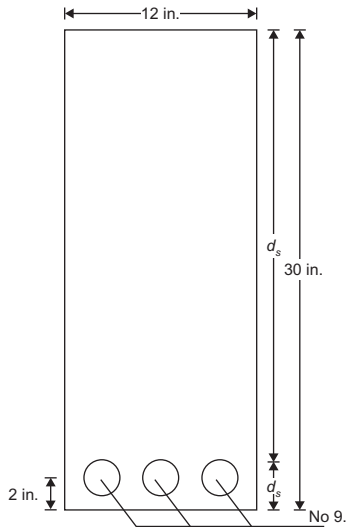

Figure D.1 Beam cross-section.

Objective: To compute the strength of a member of a given dimension in bending and shear using the LRFD approach for bridges. The required reinforcement for flexural resistance, shear resistance, crack control, shrinkage, and temperature and minimum reinforcement will be evaluated. The data will be assumed as a representative of bridge beams.

Reference: National Highway Institute. 2005. LRFD for Highway Bridge Structures and Earth Retaining Structures, FHWA-NHI-05-095.

Data: Beam size is 12 in. × 30 in. Figure D.1 shows size of beam and assumed reinforcement.

D.2 FLEXURAL RESISTANCE

Given material strength and beam reinforcement area:

$f'c = 3.50$ ksi

$fy = 60.00$ ksi

$ds = 27.44$ in.

Calculated Gross Area: $Ag = 360.00$ in.^{2}

Calculated Reinforce Area: $As = 3.00$ in.^{2}

For equilibrium: $C = T$

$0.85\ f'ca\ b = As\ fy$

$a = 5.04$ in.

$Mr = \varphi Mn = \varphi As\ fy\ (ds - a/2) = 4035.23$ kip/in.

D.3 CRACK CONTROL

$F_{sa} = z/(dc\ A)^{1/3} < 0.6fy$ given: z = 170.00 k/in.

$d_c = 2.56$

$A = b\ (2d_c)$/no. of bars

$A = 20.51$ in.^{2}

$F_{sa} = 45.37$ ksi

D.4 FACTORED CONCRETE SHEAR RESISTANCE

$d_v = (d_s - a/2) = 25.91$ (governs)
$d_v = 0.9d_s = 25.119$
$d_v = 0.72h = 21.60$
$V_r = \varphi V_n = \varphi\, 0.25 f_c' b_v d_v = 235.45$ K
$\beta = 2.00$
$\varphi V_c = \varphi\, 0.0316\, \beta \sqrt{f_c'} b_v d_v = 31.82$ K

D.5 SKRINKAGE AND TEMPERATURE

$A_g = 12*36$
$A_s = 0.11 A_g / f_y = 0.66$ in.^{2}
$A_s/2 = 0.33$ in.^{2} per face
$s = 3h = 35.00$ in. and $s = 18.00$ in.

APPENDIX E

Sample Calculations: To Compute Equivalent Static Lateral Forces and to Determine Vertical Force Distribution

Adjust DL, spans, or E values to match solved examples, OR REDO SIMILAR NEW PROBLEMS BY CHANGING DATA.

Also see Appendix A.

E.1 COMPUTE EQUIVALENT STATIC LATERAL FORCES

Accurate determination of lateral seismic forces is the most important step in seismic analysis and design. ASCE 7 simplified approach neglects vertical acceleration of building and neglects liquefaction effects.

Determine the design lateral forces due to earthquake on a 6-story concrete-frame hospital using the equivalent lateral force procedure.

Given: S_s 2.0, $S_1 = 0.9$, site class B, floor height = 12 ft.:

$W_{floor} = 450.00$ K

$W_{roof} = 200.00$ K

General procedure is as follows:

1. Determine F_a and F_v:

   From Table 21.1(a): $S_s > 1.25$, $F_a = 1.0$

   From Table 21.1(b): $S_1 > 0.5$, $F_v = 1.3$

   Also, refer to Chapter 21, McCormac, J. C. and Brown, R. H., 2009. Design of Reinforced Concrete, John Wiley and Sons.

2. Determine S_{ms} and S_{m1}:

   $S_{ms} = F_a S_s$ (ASCE/SEI Equation 11.4-1)

   $S_{m1} = F_v S_1$ (ASCE/SEI Equation 11.4-2)

3. Determine S_{DS} and S_{D1}:

   $S_{DS} = 2_{SMS}/3$ (ASCE/SEI Equation 11.4-3)

$S_{D1} = 2_{SM1}/4$ (ASCE/SEI Equation 11.4-4)

4. Hospital occupancy category = IV; importance factor $I = 1.5$.
5. Seismic design category requires SDC D.
6. Response modification coefficient $R = 8$.
7. Determine fundamental period, $T_a = 0.1N = 0.6$.
8. Determine T_s and T_L from Design Response Spectrum shown in Figure 21.1, ASCE/SEI 7-05):
   $T_s = S_{D1}/S_{DS} = 0.59$
   $T_L = 14.00$
9. Determine total design lateral seismic force on structure:
   $C_s = S_{d1}/(R/I)T = 0.25$ for $T_s < T_L$
   Since $T_a < T_L$, C_s need not exceed
   $C_s = S_{D1}/(T^*R/I) = 0.24$
   $V = C_sW = 597.1$kips. (5 floors,1 roof)

Spreadsheets may be developed for the formulae shown.

Alternate method is to use STAAD-Pro software. Examples of data files from STAAD Manual are listed in Appendix G for ready reference,

Force at top level (roof): Combining ASCE 7, Equations 12.8-11 and 12.8-12, lateral force can be expressed as,

$$F_R = [((W_r h_R)^{1.05})/((\sum W_i H_i)^{1.05})]V$$

E.2 VERTICAL FORCE DISTRIBUTION

Determine the vertical force distribution for a 2-floor building with dimensions 26 ft × 60 ft × 80 ft (Figure E.1)

Given: 1st-floor height = 14 ft; 2nd-floor height = 12 ft.:

Roof = 20 lb/ft^2

2nd floor = 20 lb/ft^2

Walls = 100 lb/ft^2

$T = C_t(h_n)^{3/4}$, From ASCE 7-05, Equation 12.8-7 $C_t = 0.02$ for bearing-wall system

$C_t = 0.23 < 0.7$ sec

Building located on Zone 3 site.

Given: $Ca = 0.36$ for soil type O: $I = 1$ for standard-occupancy structure

$R = 4.5$ for masonry bearing-wall system

Level 1, $W_1 = 408$ kips

Level 2, $W_2 = 240$ kips

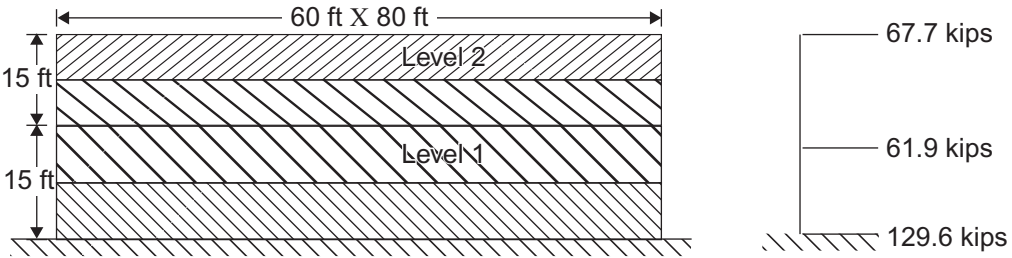

Figure E.1 Elevation of a 2-floor building. Ref. Alan Williams "Seismic Design of Buildings and Bridges", Oxford University Press, 2003. *(Note:* Example illustrates applications of 1997 UBC Equations (30-8) and (30-15).

$W_{total} = 648$ kips
$V = C_s W = (2.5 C_a{}^* \mathrm{I/R}) W_{total} = 129.6$ kips
$F_x = V(W_x h_x / \sum wh)$
$F_1 = 61.94$ K
$F_2 = 67.66$ K

Refer to Figure E.2: Seminar on ASCE 7 latest changes to loads given by Jim Rosberg, Director of SEI, Reston, VA.

Figure E.2 Left to Right: Treasurer, Vice Chair and author Ali Khan, (Philadelphia SEI Chairman) presenting a Check to Jim Rossberg (Director ASCE 07 Building Code Standards) after his SEI Endowment Lecture on Changes to Wind and Seismic Loads.

APPENDIX F

STAAD-Pro Space Frequencies of Vibration of a Skewed Bridge

Input data arranged as follows:

1. Select joint locations in plan and elevation and number each joint
2. Define line member numbers and Set up each element using cartesian coordinates X, Y and Z
3. Complete geometry of skew bridge
4. Allocate element properties E, poisson ratio and density of concrete
5. Define dead and live loads from vehicles
6. Compute mode shape and frequency

```
STAAD SPACE

UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 4 0 0; 3 6.5 0 0; 4 9 0 0; 5 11.5 0 0; 6 15.5 0 0;
11 -1 10 0 25 16.5 10 0
REPEAT ALL 3 4 0 14

MEMBER INCI
1 1 13 ; 2 2 15 ; 3 3 17 ; 4 4 19 ; 5 5 21 ; 6 6 23
26 26 34 ; 27 27 36 ; 28 28 38 ; 29 29 40 ; 30 30 42 ; 31 31 44
47 47 55 ; 48 48 57 ; 49 49 59 ; 50 50 61 ; 51 51 63 ; 52 52 65
68 68 76 ; 69 69 78 ; 70 70 80 ; 71 71 82 ; 72 72 84 ; 73 73 86
101 11 12 114
202 32 33 215
303 53 54 316
404 74 75 417

DEFINE MESH
A JOINT 11
B JOINT 25
C JOINT 46
D JOINT 32
```

```
E JOINT 67
F JOINT 53
G JOINT 88
H JOINT 74

GENERATE ELEMENT
MESH ABCD 14 12
MESH DCEF 14 12
MESH FEGH 14 12

START GROUP DEFINITION
MEMBER
_GIRDERS 101 TO 114 202 TO 215 303 TO 316 404 TO 417
_PIERS 1 TO 6 26 TO 31 47 TO 52 68 TO 73

ELEMENT
_P1 447 TO 450 454 TO 457 461 TO 464 468 TO 471
_P2 531 TO 534 538 TO 541 545 TO 548 552 TO 555
_P3 615 TO 618 622 TO 625 629 TO 632 636 TO 639
_P4 713 TO 716 720 TO 723 727 TO 730 734 TO 737
_P5 783 TO 786 790 TO 793 797 TO 800 804 TO 807
_P6 881 TO 884 888 TO 891 895 TO 898 902 TO 905

END GROUP DEFINITION

MEMBER PROPERTY
_GIRDERS PRIS YD 0.6 ZD 0.6
_PIERS PRIS YD 1.0

ELEMENT PROPERTY
YRA 9 11 TH 0.375

UNIT NEWTON MMS
CONSTANTS
E 22000 ALL
POISSON CONCRETE ALL

UNIT KNS METER
CONSTANTS
DENSITY 24 ALL
SUPPORTS
1 TO 6 26 TO 31 47 TO 52 68 TO 73 PINNED
CUT OFF MODE SHAPE 65
```

```
UNIT KGS METER
LOAD 1 FREQUENCY CALCULATION
SELFWEIGHT X 1.0
SELFWEIGHT Y 1.0
SELFWEIGHT Z 1.0

*PERMANENT WEIGHTS ON DECK
ELEMENT LOAD
YRA 9 11 PR GX 100
YRA 9 11 PR GY 100
YRA 9 11 PR GZ 100
*VEHICLES ON SPANS — ONLY Y & Z EFFECT CONSIDERED
ELEMENT LOAD
_P1 PR GY 700
_P2 PR GY 700
_P3 PR GY 700
_P4 PR GY 700
_P5 PR GY 700
_P6 PR GY 700
_P1 PR GZ 700
_P2 PR GZ 700
_P3 PR GZ 700
_P4 PR GZ 700
_P5 PR GZ 700
_P6 PR GZ 700
MODAL CALCULATION REQUESTED
PERFORM ANALYSIS
FINISH
```

Source: Bentley Corporation STAAD-Pro Users Manual.

APPENDIX G

Seismic Analysis Problems

1. Static finite element analysis of building frame with shear walls.
2. Plane response spectrum analysis for frames.
3. Plane frame example for time-history analysis.
4. Space frame example for harmonic-loading generator.
5. Space frame example for UBC accidental load.

G.1 INTRODUCTION

The provisions contained in the AASHTO LRFD bridge design specifications are based on provisions and approaches carried over from Division I-A, "Seismic Design" of AASHTO's *Standard Specifications for Highway Bridges* (AASHTO 1996). Any differences are minor, such as for live loads, braking forces, returns periods of earthquakes, and load resistance and response modification factors. The current LRFD provisions are based on seismic hazard, design criteria, and detailing.

Detailed solutions are presented here based on LRFD-factored moments, shear forces, and reactions resulting from extreme load combination of dead, live, and seismic loads. The solved examples were generated with STAAD-Pro software. Typical quasi-static and dynamic analysis problems follow. Data files from STAAD-III originated by Research Engineers/Bentley software are included as practice examples to compare new solutions for a different geometry and load data. The format of data files in Appendices G.2 to G.7 is in keeping with the standard methods of analysis, which is summarized in each case.

In Appendix G.7, output file format is included to show the various steps given in Appendix G.6 input data file.

Source: STAAD-3 Getting Started and Example Manual released in 1997.

Adjust DL, spans, or E values to match solved examples OR REDO.

G.2 STATIC FINITE ELEMENT ANALYSIS OF BUILDING FRAME WITH SHEAR WALLS DATA FILE ONLY FOR DEAD LOAD AND WIND ANALYSIS

SUMMARY OF DATA FILE SET-UP

Input data is arranged as follows:

1. Select joint locations in plan and elevation and number each joint.
2. Define line member numbers and Set up each element using cartesian coordinates X, Y and Z.
3. Complete geometry of shear wall with frame.
4. Allocate element properties E, poisson ratio and density of concrete.
5. Apply floor dead load and wind loads. For seismic analysis replace wind loads with equivalent lateral forces.
6. Perform Analysis and compute reactions, forces and moments.
7. Perform ACI CODE Design of concrete members.

The following is a STAAD SPACE* example problem with frame members and finite elements.

```
UNIT FEET KIP
JOINT COORD
1 0 0 0 ; 2 0 0 20
REP ALL 2 20 0 0
7 0 15 0 11 0 15 20
12 5 15 0 14 15 15 0
15 5 15 20 17 15 15 20
18 20 15 0 22 20 15 20
23 25 15 0 25 35 15 0
26 25 15 20 28 35 15 20
29 40 15 0 33 40 15 20
34 20 3.75 0 36 20 11.25 0
37 20 3.75 20 39 20 11.25 20
MEMBER INCI
*COLUMNS
1 1 7 ; 2 2 11
3 3 34 ; 4 34 35 ; 5 35 36 ; 6 36 18
7 4 37 ; 8 37 38 ; 9 38 39 ; 10 39 22
11 5 29 ; 12 6 33
```

```
*BEAMS IN Z DIRECTION AT X = 0
13 7 8 16
*BEAMS IN Z DIRECTION AT X = 20
17 18 19 20
*BEAMS IN Z DIRECTION AT X = 40
21 29 30 24
*BEAMS IN X DIRECTION AT Z = 0
25 7 12 ; 26 12 13 ; 27 13 14 ; 28 14 18
29 18 23 ; 30 23 24 ; 31 24 25 ; 32 25 29
*BEAMS IN X DIRECTION AT Z = 20
33 11 15 ; 34 15 16 ; 35 16 17 ; 36 17 22
37 22 26 ; 38 26 27 ; 39 27 28 ; 40 28 33
*
DEFINE MESH
A JOINT 7
B JOINT 11
C JOINT 22
D JOINT 18
E JOINT 33
F JOINT 29
G JOINT 3
H JOINT 4
GENERATE ELEMENT
MESH ABCD 4 4
MESH DCEF 4 4
MESH DCHG 4 4
MEMB PROP
1 TO 40 PRIS YD 1 ZD 1
ELEM PROP
41 TO 88 TH 0.75
UNIT INCH
CONSTANT
E 3500 ALL
POISSON CONCRETE ALL
SUPPORT
1 TO 6 PINNED
UNIT FEET
```

```
*
LOAD 1 DEAD LOAD FROM FLOOR
ELEMENT LOAD
41 TO 72 PRESSURE -0.75
*
LOAD 2 WIND LOAD
JOINT LOAD
11 33 FZ -20.
22 FZ -100.
LOAD COMB 3
1 0.9 2 1.3
PERFORM ANALYSIS
*
LOAD LIST 1 3
PRINT SUPP REAC
PRINT MEMBER FORCES LIST 27
PRINT ELEMENT STRESSES LIST 47
START CONCRETE DESIGN
CODE ACI
DESIGN ELEMENT 47
END CONCRETE DESIGN
FINISH
```

G.3 PLANE RESPONSE SPECTRUM ANALYSIS FOR FRAMES (STAAD DATA FILE)

SUMMARY OF DATA FILE SET-UP

Input data is arranged as follows:

1. Select joint locations in plan and elevation and number each joint.
2. Define line member numbers and set up each element using cartesian coordinates X, Y and Z.
3. Complete geometry of shear wall with frame.
4. Allocate element properties E, poisson ratio and density of concrete.
5. Apply dead load.
6. Apply response spectrum load with masses provided in terms of seismic loading selfweight X 1.0, selfweight Y 1.0.
7. Perform analysis and print mode shapes.

```
UNIT FEET KIPS
JOINT COORDINATES
1 0 0 0 ; 2 20 0 0
3 0 10 0 ; 4 20 10 0
5 0 20 0 ; 6 20 20 0
MEMBER INCIDENCES
1 1 3 ; 2 2 4 ; 3 3 5 ; 4 4 6
5 3 4 ; 6 5 6
MEMBER PROPERTIES AMERICAN
1 TO 4 TA ST W10X33
5 TA ST W12X40
6 TA ST W8X40
SUPPORTS
1 2 PINNED
UNIT INCH
CONSTANTS
E 29000. ALL
POISSON STEEL ALL
DEN 0.000283 ALL
CUT OFF MODE SHAPE 2
*LOAD 1 WILL BE STATIC LOAD
UNIT FEET
LOAD 1 DEAD AND LIVE LOADS
SELFWEIGHT Y -1.0
MEMBER LOADS
5 CON GY -5.0 6.0
5 CON GY -7.5 10.0
5 CON GY -5.0 14.0
5 6 UNI Y -0.5
NEXT LOAD WILL BE RESPONSE SPECTRUM LOAD
WITH MASSES PROVIDED IN TERMS OF LOAD.
LOAD 2 SEISMIC LOADING
SELFWEIGHT X 1.0
SELFWEIGHT Y 1.0
MEMBER LOADS
5 CON GX 5.0 6.0
5 CON GY 5.0 6.0
5 CON GX 7.5 10.0
5 CON GY 7.5 10.0
```

```
5 CON GX 5.0 14.0
5 CON GY 5.0 14.0
SPECTRUM CQC X 1.0 ACC DAMP 0.05 SCALE 32.2
0.03 1.00 ; 0.05 1.35
0.1.95 ; 0.2 2.80
0.5 2.80 ; 1.0 1.60
LOAD COMBINATION 3
1 0.75 2 0.75
LOAD COMBINATION 4
1 0.75 2 −0.75
PERFORM ANALYSIS PRINT MODE SHAPES
PRINT ANALYSIS RESULTS
LOAD LIST 1 3 4
PARAMETER
CODE AISC
SELECT ALL
FINISH
```

G.4 STAAD PLANE FRAME DATA FILE EXAMPLE FOR TIME-HISTORY ANALYSIS

SUMMARY OF DATA FILE SET-UP

Input data is arranged as follows:

1. Select joint locations in plan and elevation and number each joint.
2. Define line member numbers and Set up each element using cartesian coordinates X, Y and Z.
3. Complete geometry with support conditions of frame.
4. Allocate element properties E, poisson ratio and density of concrete.
5. Define type 1 forces.
6. Define type 2 accelerations.
7. Define arrival times.
8. Define damping.
9. Apply static load.
10. Apply time history load.
    Selfweight X 1.0.
    Selfweight Y 1.0.
11. Apply lateral joint load.

12. Apply time load.
13. Apply ground motions.
14. Perform analysis.
15. Print joint displacements.
16. Print memberforces.
17. Print support reactions.

```
UNITS FEET KIP
JOINT COORDINATES
1 0.0 0.0 0.0
2 0.0 3.5 0.0
3 0.0 7.0 0.0
4 0.0 10.5 0.0
MEMBER INCIDENCES
1 1 2 3
UNIT INCH
MEMBER PROPERTIES
1 2 3 PRIS AX 3.0 IZ 240.0
SUPPORTS
1 4 FIXED
CONSTANTS
E 15000 ALL
DENSITY 0.0868E-3 ALL
POISSON CONCRETE ALL
DEFINE TIME HISTORY
TYPE 1 FORCE
0.0 -0.0001 0.5 0.0449 1.0 0.2244 1.5 0.2244 2.0 0.6731 2.5 -0.6731
TYPE 2 ACCELERATION
0.0 0.001 0.5 -7.721 1.0 -38.61 1.5 -38.61 2.0 -115.82 2.5 115.82
ARRIVAL TIMES
0.0
DAMPING 0.075
UNIT FEET
LOAD 1 STATIC LOAD
MEMBER LOAD
1 2 3 UNI GX 0.3
LOAD 2 TIME HISTORY LOAD
SELFWEIGHT X 1.0
SELFWEIGHT Y 1.0
```

```
JOINT LOAD
2 3 FX 1.0
TIME LOAD
2 3 FX 1 1
GROUND MOTION X 2 1
PERFORM ANALYSIS
UNIT INCH
PRINT JOINT DISPLACEMENTS
UNIT FEET
PRINT MEMBER FORCES
PRINT SUPPORT REACTION
FINISH
```

G.5 STAAD SPACE EXAMPLE FOR HARMONIC-LOADING GENERATOR

SUMMARY OF DATA FILE SET-UP

Input data is arranged as follows:

1. Select joint locations in plan and elevation and number each joint.
2. Define line member numbers and set up each element using cartesian coordinates X, Y and Z.
3. Complete geometry with support conditions of frame.
4. Allocate element properties E, poisson ratio and density of concrete.
5. Define timehistory.
6. Type 1 force.
7. Harmonic loading generator.
8. Function sine
   Amplitude
   Frequency
   Cycles
   Arrival times
   Damping.
9. Load 1 static load case member load uniform GY.
10. Load 2 dynamic load case.
    Selfweight X 1.0.
    Selfweight Y 1.0.
    Selfweight Z 1.0.

11. Joint load FX FY FZ.
12. Time load FX.
13. Perform analysis.
14. Print analysis results.

```
UNIT KIP FEET
JOINT COORDINATES
1 0 0 0 ; 2 15 0 0 ; 3 15 0 15 ; 4 0 0 15
5 0 20 0 ; 6 7.5 20 0 ; 7 15 20 0 ; 8 15 20 7.5
9 15 20 15 ; 10 7.5 20 15 ; 11 0 20 15
12 0 20 7.5
MEMBER INCIDENCES
1 1 5 ; 2 2 7 ; 3 3 9 ; 4 4 11 ; 5 5 6 ; 6 6 7
7 7 8 ; 8 8 9 ; 9 9 10 ; 10 10 11 ; 11 11 12 ; 12 12 5
UNIT INCH
MEMBER PROPERTIES
1 TO 12 PRIS YD 12 ZD 12
SUPPORTS
1 TO 4 FIXED
CONSTANTS
E 3500 ALL
DENSITY 0.0868E-3 ALL
POISSON CONCRETE ALL
DEFINE TIME HISTORY
TYPE 1 FORCE
* FOLLOWING LINES FOR HARMONIC LOADING GENERATOR
FUNCTION SINE
AMPLITUDE 6.2831 FREQUENCY 60 CYCLES 100
*
ARRIVAL TIMES
0.0
DAMPING 0.075
LOAD 1 STATIC LOAD CASE
MEMBER LOAD
5 6 7 8 9 10 11 12 UNI GY -1.0
LOAD 2 DYNAMIC LOAD CASE
SELFWEIGHT X 1.0
SELFWEIGHT Y 1.0
SELFWEIGHT Z 1.0
```

```
JOINT LOAD
8 12 FX 4.0
8 12 FY 4.0
8 12 FZ 4.0
TIME LOAD
8 12 FX 1 1
PERFORM ANALYSIS
PRINT ANALYSIS RESULTS
FINISH
```

G.6 STAAD SPACE EXAMPLE FOR UBC ACCIDENTAL LOAD DATA FILE ONLY

Input data arranged as follows:

1. Select joint locations in plan and elevation and number each joint.
2. Define line member numbers and set up each element using cartesian coordinates X, Y and Z.
3. Complete geometry and support conditions.
4. Allocate element properties E, poisson ratio and density of concrete.
5. Define ubc accidental load.
6. Zone RWX RWZ STYP 4 NA 1 NV 1.
7. Selfweight.
8. Floor weight
   Y range F load
   Y range F load.
9. Load 1
   UBC load X.
10. Perform analysis print load data.
11. Change
    Load 2
    UBC load Z.
12. Perform analysis print load data.
13. Change
    Load 3.
14. Self Y -1.0.
15. Floor load

Y range F Load
Y range F Load.

16. Perform Analysis print statics check.
17. Change
    Load list.
18. Print joint displacements.
19. Print support reactions.
20. Print member forces.

```
STAAD SPACE
SET NL 5
unit kip feet
joint coord
1 0 0 0 ; 2 0 10 0 ; 3 13 10 0 ; 4 27 10 0 ; 5 40 10 0 ; 6 40 0 0
7 0 20.5 0 ; 8 20 20.5 0 ; 9 40 20.5 0
repeat all 1 0 0 11
member inci
1 1 2 5 ; 6 1 3 ; 7 4 6 ; 8 2 7 ; 9 7 8 10 ; 11 9 5 ; 12 2 8 ; 13 5 8
21 10 11 25 ; 26 10 12 ; 27 13 15 ; 28 11 16 ; 29 16 17 30 ; 31 18 14
32 11 17 ; 33 14 17
41 2 11 44
45 7 16 47
51 1 11
52 10 2
53 2 16
54 11 7
55 6 14
56 15 5
57 5 18
58 14 9
member properties
1 5 8 11 21 25 28 31 ta st W14X90
2 3 4 22 23 24 ta st w18x35
9 10 29 30 ta st w21x50
41 to 44 ta d c12x30
45 to 47 ta d c15x40
6 7 26 27 ta st hsst20x12x0.5
51 to 58 ta ld l50308
12 13 32 33 ta st tub2001205
constants
```

```
e steel all
poisson steel all
density steel all
support
1 6 10 15 pinned
Member tension
51 to 58
unit pound
DEFINE UBC ACCIDENTAL LOAD
ZONE 0.3 I 1 RWX 2.9 RWZ 2.9 STYP 4 NA 1 NV 1
SELFWEIGHT
FLOOR WEIGHT
YRANGE 9 11 FLOAD 0.4
YRANGE 20 21 FLOAD 0.3
LOAD 1
UBC LOAD X
PERFORM ANALYSIS PRINT LOAD DATA
CHANGE
LOAD 2
UBC LOAD Z
PERFORM ANALYSIS PRINT LOAD DATA
CHANGE
LOAD 3
SELF Y -1.0
FLOOR LOAD
YRANGE 9 11 FLOAD -0.25
YRANGE 20 21 FLOAD -0.2
PERFORM ANALYSIS
CHANGE
LOAD 4
REPEAT LOAD
1 1.0 3 1.0
PERFORM ANALYSIS PRINT STATICS CHECK
CHANGE
LOAD 5
REPEAT LOAD
2 1.0 3 1.0
PERFORM ANALYSIS PRINT STATICS CHECK
CHANGE
```

```
LOAD LIST 4 5
PRINT JOINT DISPLACEMENTS
PRINT SUPPORT REACTIONS
PRINT MEMBER FORCES LIST 51 to 58
FINISH
```

GLOSSARY

Basic definitions and concepts can be obtained from well-known seismic websites such as USFS, FEMA, the Lehigh Earth Observatory, MCEER, NISEE UC Berkeley, and Wikipedia's Earthquake entry. Additional definitions and explanations may be found in the text.

Seismology

Aftershocks are earthquakes that follow the largest shock of an earthquake sequence, which is the *mainshock*. They are formed as the crust around the displaced *fault* plane adjusts to the effects of the mainshock. Aftershocks are smaller than the mainshock and occur within one to two fault lengths from the mainshock fault. They are in the same region of the mainshock. Unlike the mainshock, aftershocks can continue over a period of weeks, months, or years. If an aftershock happens to be larger than the mainshock, it may be redesignated as the mainshock and the original mainshock redesignated as the foreshock.

Body waves are the primary (P) and secondary (S) seismic waves, which generate from tectonic plate movements and propagate from the earthquake *focus* to the surface.

Foreshocks are earthquakes that precede *mainshocks* in the same location. They are relatively smaller than mainshocks. All mainshocks have aftershocks, but not all may have foreshocks.

Earthquake clusters are a phenomenon in which, according to one seismology theory, earthquakes recur in a regular pattern. Most earthquake clusters consist of small tremors that cause little or no damage.

Earthquake swarms are sequences of earthquakes striking in a specific area within a short period of time. An example of an earthquake swarm is the 2004 activity at Yellowstone National Park.

The elastic rebound model reflects the sudden release of stored strain in rocks that results from movement along a *fault*.

The epicenter is the location on the earth's surface that lies directly above the *focus* of an earthquake

***A* fault** is a break in the earth's crust, caused by movement of tectonic plates, on which earthquakes occur. Normal earthquakes occur on normal faults, thrust earthquakes occur on thrust, or reverse, faults, and strike-slip earthquakes occur on strike-slip faults. The fault surface can be vertical or horizontal, or at some angle to the surface of the earth.

The dip of a fault is the angle the fault surface makes with a horizontal plane. The *strike* is the direction of the fault line exposed or projected at the ground surface relative to the north. Strike-slip, or transform, faults involve displacements of rock laterally, parallel to the strike. Strike-slip events include the 1857 and 1906 San Francisco earthquakes.

A dip-slip fault is one in which motion is largely parallel to the fault dip. It has vertical components of displacement. There are two types of dip-slip faults: normal and reverse. When structures such as dams and embankments must be built across active

faults, their design usually incorporates joints or flexible sections in the fault zone. The maximum horizontal offset in the 1906 San Francisco earthquake was about 18 feet.

The focus of an earthquake is the zone within the earth in which rock displacement occurs to produce the earthquake. Earthquakes occurring at a depth of less than 70 km are classified as *shallow-focus*; those with a focal depth between 70 and 300 km: are commonly termed *mid-focus* or *intermediate-depth*; and those occurring from 300 to 700 kilometers in depth are classified as *deep-focus*.

Liquefaction is a process in an earthquake by which loosely packed, water-logged soils temporarily lose strength and stiffness and behave like liquids, causing the ground to sink or slide.

The lithosphere is the rigid outer layer of the earth; it includes the crust and the upper mantle

Modified Mercalli intensity is the measured felt intensity of an earthquake. It is rated on a *Modified Mercalli intensity scale* from I to IX.

The Richter scale is a logarithmic instrument for measuring the magnitude of an earthquake.

A seismograph is an instrument that records earthquake *waves* to locate the epicenter and focus of an earthquake.

Secondary effects are nontectonic surface processes directly related to primary earthquake shaking.

Seismology is the scientific discipline that deals with earthquakes. Seismologists try to predict when and where earthquakes will occur.

A subduction zone is a long, narrow underground region where one tectonic plate descends beneath another.

Surface waves are seismic waves that travel along the outer layer of the earth. They are classified as *Love* and *Rayleigh* waves.

The theory of plate tectonics states that the lithosphere is divided into a number of relatively rigid plates that collide with, separate from, and translate past one another at their boundaries. This disruption commonly results in earthquakes.

Seismic Engineering

Anchor ties or anchor bolts are threaded rods or bolts that connect walls to framing. Washers or plates anchor the bolts in place. *Grouted bolts* are anchor bolts set generally on an angle in a concrete grout mixture to avoid the problem of using an exposed washer. These bolts requires a greater diameter hole than anchor bolts with washers.

Base Isolation is a technology that uses bearing supports to isolate structures and release seismic forces by providing a flexible layer between foundations and supporting members.

Core drilling creates a continuous vertical core in *masonry* or concrete that is filled with steel reinforcing rods and grouting to resist in-plane shear and out-of-plane bending.

A damper is a shock-absorbing material or device placed under and around the foundation of building to progressively diminish the frequency of vibrations or oscillations.

A cripple wall is a frame wall between a building's first floor and its foundation.

Diagonal braces or other types of X or K vertical bracings provide lateral resistance to adjacent walls.

A diaphragm is a floor, roof, or continuous membrane that distributes earthquake loads to a structure's exterior or interior *shear walls*.

Fiber-wrap reinforcement is a synthetic compound of filaments that increase shear capacity when wrapped around structural members.

Lateral forces are the horizontal forces transferred to a structure from the dynamic effects of wind or seismic forces.

Life safety is the ultimate goal of seismic engineering and seismic design codes. The risk of loss of life is kept to minimal levels by strengthening structures to avoid collapse. Threats to life safety during an earthquake and its aftermath include structural collapse, falling debris, blocked exits and/or emergency routes, and consequential fires.

A moment-resisting frame is a steel frame with extra strong joints between beam and columns. It provides in-plane resistance to lateral loads particularly by reinforcing joint connections without the need for *diagonal braces*. A frame of this type is often used as a perimeter frame around storefronts or large door and window openings.

Seismic Retrofit encompasses structural measures that improve the earthquake performance of an existing structure by improving its stability so as to reduce the potential for structural damage or collapse in an earthquake.

Shear stress is a concept in physics whereby forces act on a body in opposite directions but not in the same line. Horizontal forces applied to a wall that is insufficient to move with them will crack, often in a diagonal or X pattern. Connections at beams and walls will also crack from shear stress.

A shear wall resists lateral forces with columns designed to minimize sway and transfer loads from a building's roof and floors to its foundation as a means to protect a building from collapse due to wind or earthquake forces.

Unreinforced masonry includes traditional brick, block, and adobe in buildings that rely on the weight of the masonry and the bonding capacity of the mortar to provide lateral strength and structural stability.

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