Designing for Earthquakes

Proceedings from the 1978 Summer Seismic Institutes for Architectural Faculty

AIA Research Corporation
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AIA Research Corporation
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INTRODUCTION

The AIA Research Corporation, supported by a grant from the National Science Foundation, Divison of Problem Focused Research convened the 1978 Summer Seismic Institutes For Architectural Faculty in two sessions. The first session was convened at the University of Illinois, Champaign-Urbana June 18 to 23, 1978. The second session was held at Stanford University, August 20 to 25, 1978.

The purpose of the Institutes has been to bring concerns for earthquake safety more broadly into the architectural community. One important way of accomplishing that goal is to foster a greater awareness and understanding of earthquakes and earthquake resistant design among architectural faculty, who can, in turn, pass their knowledge along to their students. Architectural curricula are a natural vehicle for the dissemination of earthquake hazard information and design response methodologies to the profession. The Summer Seismic Institute participants were faculty members from schools of architecture throughout the United States, and Puerto Rico.

This document is a compendium of papers which were condensed and edited from the Institute Lectures. The lectures were presented by experienced professionals from various disciplines within the earthquake field, and cover the broad range of concerns for seismic design.

The 1978 Institutes focused on the broad and complex issues which affect the architectural response to earthquake risk. The program provided a background on the nature of earthquakes and earthquake hazards, and identified the public policy issues which affect planning and land use in seismic risk areas. It also serves as a vehicle for the interaction of faculty members from a diverse range of architectural schools with various curriculum strategies. This intermingling of educators has aided the development of viable strategies for the application of seismic design elements to their respect college curricula.

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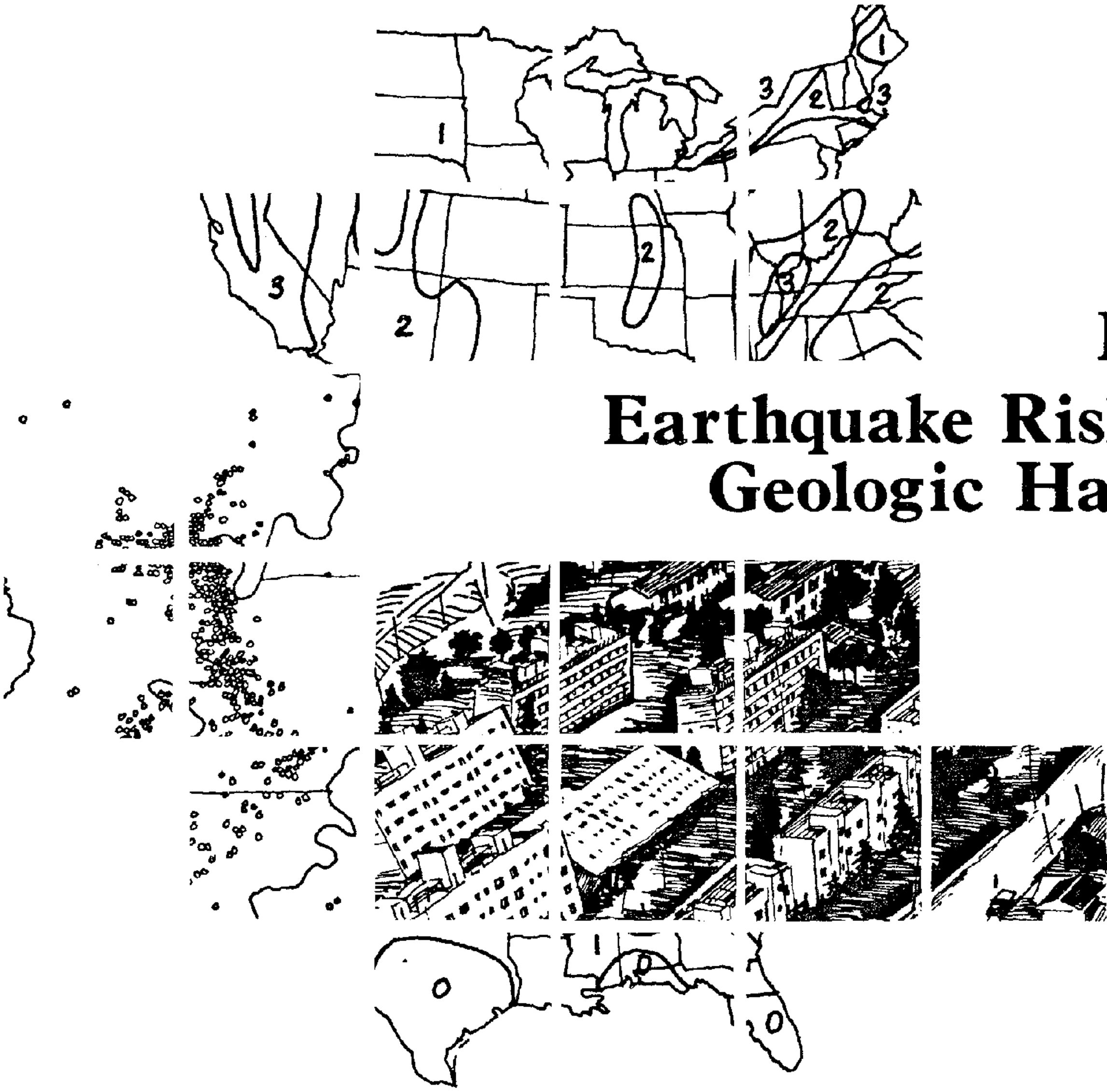

Part I
Earthquake Risk and Geologic Hazards

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Chapter 1. EARTHQUAKE RISK REDUCTION

O. Clark Mann, P.E.

INTRODUCTION

As a profession almost wholly dedicated to building for human needs, it is appropriate that architects seriously consider the seismic environment. The architect, whether he knows it or not, makes a significant contribution to the safety or dangers of that environment by the structures he designs. The best earthquake engineering imaginable cannot make up for an inadequately conceived building.

Seismic effects on structures are vastly different from those of gravity. As such, they pose new design problems as well as opportunities for the well-informed architect. If we hold a glass of water above a table and then let go, everyone knows what will happen. We have been conditioned since birth to understand the force of gravity. But, if we ask what will happen to the glass of water sitting on the table during an earthquake, we must deduce the answer. We have not been conditioned to a horizontal force field. The complexity of seismic loading and intercomponent behavior demands a greater-than-usual science-engineering input into all facets of building design from the concept through the construction.

This paper will be a discussion of seismic risks and when and where these risks can be important. Reference materials by a number of risk analysts have been used in order to outline as broad a perspective as possible and to illustrate each point. The analytical process and criteria that our firm use for risk analysis are often unique since we are usually searching for the most economical means of achieving a given level of seismic safety. This objective no doubt permeates these comments; thus you are encouraged to read the work of analysts with different views. The factors which are discussed have been selected with the hope that they will enrich the architect's intuition and judgment and suggest when and how expert help should be used. Above all, I hope these remarks will, in some positive way, lead to a safe and economically affordable seismic environment.

EARTHQUAKE HISTORY AND SEISMOLOGY

There is nothing new about earthquakes, nothing new except the growing risks they pose to modern society. Earthquakes have occurred across most of the earth's surface and throughout its discernable history. Earthquakes are a part of the geologic process. This being true, one may reason with confidence that the process will not change very much and earthquakes will continue to occur throughout the foreseeable future.

Ancient peoples conjured, half from fact and half from fantasy, many mythical explanations for earthquakes. Some myths told of the earth borne by an animal whose stumbling shook the earth. Others blamed earthquakes on the wrath of gods. A few myths were methaphysical, such as Aristotle's theory of winds blowing through subterranean caverns. Surpisingly, these myths flourished until the mid-19th century when Robert Mallet presented to the scientific world a series of scientifically conceived and logically developed papers addressing the dynamics of earthquakes. Today, we know that much of his theory was correct. For example, his conception of a sudden release of energy within the earth's crust as the causative mechanism is now known to be responsible for earthquakes.

Following the pioneering of Mallet, many investigators devoted energies to observing the damage caused by earthquakes throughout the world. It was natural that these investigators should invent numerical scales in order to measure and classify the earthquake characteristics they observed to facilitate their recording and communication. In 1883 DeRossi and Forel of Switzerland developed a scale based on human response and damages to man-made structures. Their scale was widely used until replaced in 1931 by the currently used Modified Mercalli scale. Both scales are shown in Figure 1.1. The Mercalli is a twelve-point scale that embraces the full range of known earthquake intensities. A very light earthquake, scarely felt by people, and causing no damage is called intensity 1. As the effects increase, the identification number increases up to intensity 12, or "catastrophic" events. When an earthquake occurs, physical observations and dollar damage descriptions are currently collected by the U. S. GeologicalSurvey (USGS), using a standard questionaire as is shown in Figure 1.2. The results from each city in the affected area are interpreted by seismologists, assigned a Modified Mercalli intensity, and recorded in the form of isoseismal maps. This mode of communication allows the investigator to succinctly record and convey the collected information on the intensity range of the earthquake and their spacial distribution around the earthquake epicenter. The most severe recent earthquake to strike the central United States occurred November 9, 1968, in southern Illinois: the isoseismal record is shown in Figure 1.3.

1. Not felt. Marginal effects from distant long-period large earthquakes. (Rossi-Forel (RF-I)
2. Felt by persons at rest, on upper floors, or favorably placed. (RF-I to II)
3. Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake. (RF-III)
4. Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt of a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of 4, wooden walls and frames crack. (RF-IV to V)
5. Felt outdoors; directions estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start change rate. (RF-V to VI)
6. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, and so on, off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring. Trees, bushes shaken visibly, or heard to rustle. (RF-VI to VII)
7. Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, unbraced parapets, and architectural ornaments. Some cracks in masonry C. Waves on ponds; water tubid with mud. Small slides and caving in along sand of gravel banks. Large bells ring. Concrete irrigation ditches damaged. (RF-VII)
8. Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes. (RF-VII to IX)
9. General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames racked. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake foundations, sand craters. (RF-IX)
10. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly. (RF-X)
11. Rails bent greatly. Underground pipelines completely out of service.
12. Damage nearly total. Large rock masses displaced. Lines of slight and level distorted. Objects thrown into the air.

MODIFIED MERCALLI INTENSITY SCALE
Figure 1.1

U.S. DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
EARTHQUAKE REPORT

Form Approved
OMB No. 42-R1700

Please answer this questionnaire carefully and return as soon as possible.

1. Was an earthquake felt by anyone in your town or zip code area recently?
   ☐ Not felt: Please refold and tape for return mail.
   ☐ Felt: Date __________ Time ________ ☐ AM ☐ PM ☐ Standard time ☐ Daylight time

Name of person filling out form ____________________

Address ____________________

City ____________ County ____________

State ____________ Zip code ____________

If you felt the earthquake, complete the following section. If others felt the earthquake but you did not, skip the personal report and complete the community report.

PERSONAL REPORT

2a. Did you personally feel the earthquake?	1☐ Yes	☐ No
b. Were you awakened by the earthquake?	2☐ Yes	☐ No
c. Were you frightened by the earthquake?	3☐ Yes	☐ No

d. Were you at 4☐ Home 5☐ Work 6☐ Other?

e. Town and zip code of your location at time of earthquake ____________

f. Check your activity when the earthquake occurred:

7☐ Walking	8☐ Sleeping	9☐ Lying down	10☐ Standing
11☐ Driving (car in motion)	12☐ Sitting	13☐ Other	

g. Were you 14☐ Inside or 15☐ Outside?

h. If inside, on what floor were you? 16☐☐

Continue on to next section which should include personal as well as reported observations.

COMMUNITY REPORT

Check one box for each question that is applicable.

3a. The earthquake was felt by	☐ No one	17☐ Few	13☐ Several	19☐ Many	20☐ All?
b. This earthquake awakened	☐ No one	21☐ Few	22☐ Several	23☐ Many	24☐ All?
c. This earthquake frightened	☐ No one	25☐ Few	26☐ Several	27☐ Many	28☐ All?

4. What outdoor physical effects were noted in your community?

Parapets or cornices fallen	29☐ Yes	☐ No	
Trees and bushes shaken	30☐ Slightly	31☐ Moderately	32☐ Strongly
Standing vehicles rocked	33☐ Slightly	34☐ Moderately	35☐ Strongly
Moving vehicles rocked	36☐ Slightly	37☐ Moderately	38☐ Strongly
Ground cracks	39☐ Wet ground	40☐ Steep slopes	41☐ Dry and level ground
Landslides	42☐ Small	43☐ Large	
Underground pipes	44☐ Broken	45☐ Out of service	
Water splashed onto sides of lakes, ponds, swimming pools	46☐ Yes	☐ No	
Elevated water tanks	47☐ Cracked	48☐ Twisted	49☐ Fallen (thrown down)
Air coolers	50☐ Displaced	51☐ Rotated	52☐ Fallen
Railroad tracks bent	53☐ Slightly	54☐ Greatly	
Stone or brick fences	55☐ Cracked	56☐ Fallen	57☐ Destroyed
Tombstones	58☐ Displaced	59☐ Cracked	60☐ Rotated
	61☐ Fallen		
Chimneys	62☐ Cracked	63☐ Twisted	64☐ Fallen
	65☐ Broken at roof line		66☐ Bricks fallen
Highways or streets	63☐ Cracked slightly	68☐ Large cracks	63☐ Displaced
Sidewalks	70☐ Cracked slightly	71☐ Large cracks	72☐ Displaced

Continued on the reverse side

USGS EARTHQUAKE SURVEY
Figure 1.2

5. What indoor physical effects were noted in your community?

Windows, doors, dishes rattled	73☐ Yes	☐ No	
Buildings creaked	74☐ Yes	☐ No	
Building trembled (shook)	75☐ Yes	☐ No	
Hanging pictures	76☐ Swung	77☐ Out of place	78☐ Fallen
Water in small containers	79☐ Spilled	80☐ Slightly disturbed	
Windows	81☐ Few cracked	82☐ Some broken	83☐ Many broken

6a. Did hanging objects, doors swing? ☐ No 84☐ Slightly 85☐ Moderately 86☐ Violently

b. Can you estimate direction? ☐ No 87☐ North/South 88☐ East/West 89☐ Other

7a. Were small objects (dishes, knick-knacks, pictures) ☐ Unmoved 90☐ Shifted 91☐ Overturned 92☐ Fallen, not broken 93☐ Broken?

b. Was light furniture ☐ Unmoved 94☐ Shifted 95☐ Overturned 96☐ Fallen, not broken 97☐ Broken?

c. Were heavy furniture or appliances ☐ Unmoved 98☐ Overturned 99☐ Shifted 100☐ Broken?

8. Indicate effects of the following types to interior walls if any:

Plaster	101☐ Cracked	102☐ Fell
Dry wall	103☐ Cracked	104☐ Fell
Ceiling tiles	105☐ Cracked	106☐ Fell

9a. Check below any damage to buildings or structures.

Foundation	107☐ Cracked	108☐ Destroyed	
Interior walls	109☐ Split 110☐ Fallen	111☐ Separated from ceiling or floor	
Exterior walls	112☐ Hairline cracks	113☐ Large cracks	114☐ Bulged outward
	115☐ Partial collapse	116☐ Total collapse	
Building	117☐ Moved on foundation	118☐ Shifted off foundation	

b. What type of construction was the building that showed this damage?

119☐ Wood	120☐ Stone	121☐ Brick veneer	122☐ Other
123☐ Brick	124☐ Cinderblock	125☐ Reinforced concrete	

c. What was the type of ground under the building?

126☐ Don't know	127☐ Sandy soil	128☐ Marshy	129☐ Fill
130☐ Hard rock	131☐ Clay soil	132☐ Sandstone, limestone, shale	

d. Was the ground: 133☐ Level 134☐ Sloping 135☐ Steep?

e. Check the approximate age of the building: 136☐ Built before 1935 137☐ Built 1935-65 138☐ Built after 1965

10a. What percentage of buildings were damaged?
Within 2 city blocks of your location ☐ None 139☐ Few (about 5%) 140☐ Many (about 50%) 141☐ Most (about 75%)

b. In area covered by your zip code ☐ None 142☐ Few (about 5%) 143☐ Many (about 50%) 144☐ Most (about 75%)

11a. Were springs or well water disturbed? 145☐ Level changed 146☐ Flow disturbed 147☐ Muddied ☐ Don't know

b. Were rivers or lakes changed? 145☐ Yes ☐ No ☐ Don't know

12a. Was there earth noise? ☐ No 149☐ Faint 150☐ Moderate 151☐ Loud

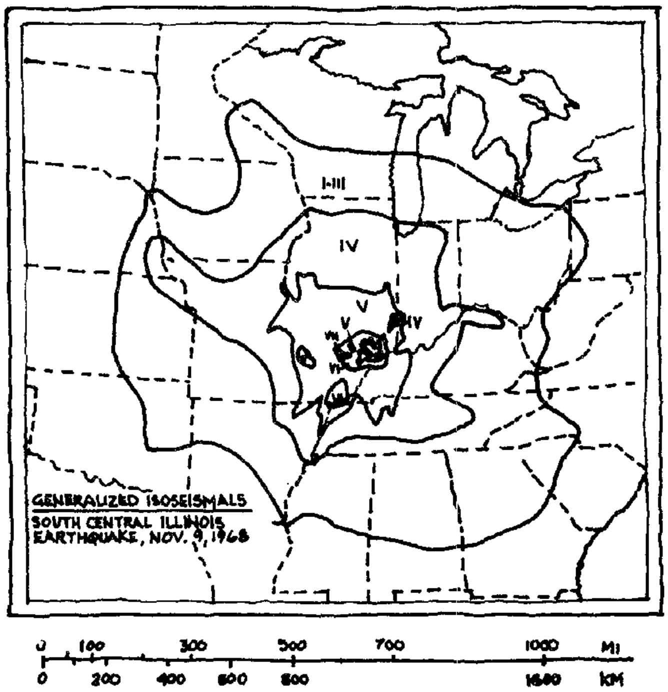
b. Direction of noise 152☐ North 153☐ South 154☐ East 155☐ West

c. Estimated duration of shaking 156☐ Sudden, sharp (less than 10 secs) 157☐ Long (30-60 secs) 158☐ Short (10-30 secs) 159☐ Other

13. What is the approximate population of your city/town? 160☐ Less than 1,000 161☐ 10,000 to 100,000 162☐ 1,000 to 10,000 163☐ Over 100,000
Or are you in a 164☐ Rural area?

This community report is associated with what town or zip code? ____________

Thank you for your time and information. Refold this card and tape for return mail.

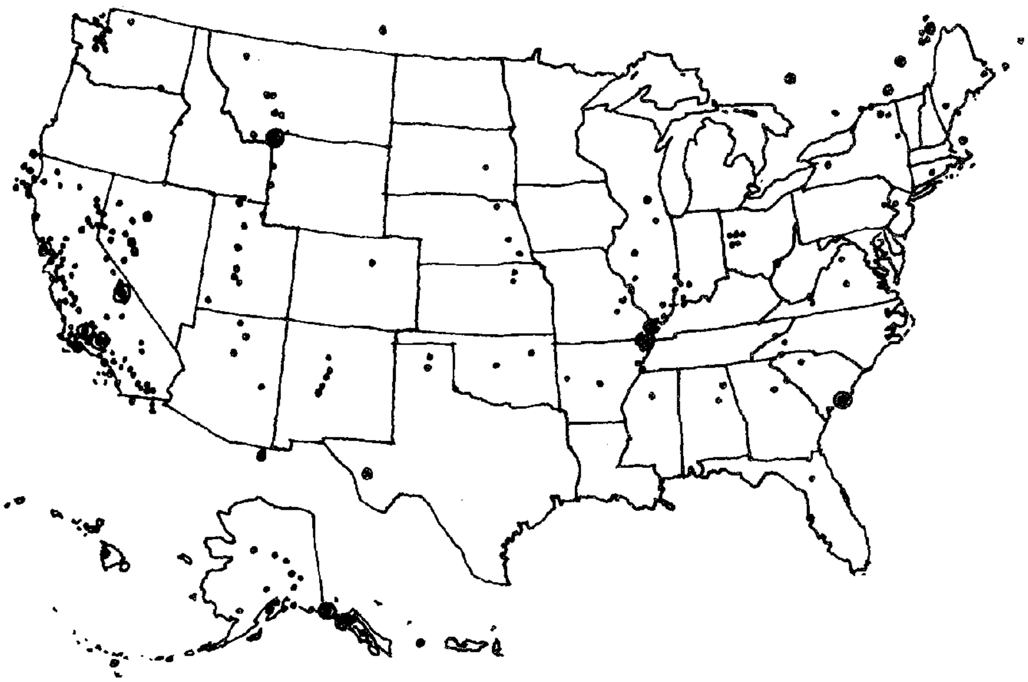


TYPICAL ISOSEISMAL - INTENSITY 7
after D. W. Gordon et al
Figure 1.3

Another system of measurement is used by the seismologist for more precise, scientific work. It is known as the Richter magnitude scale after the famous American seismologist Charles Richter. The scientific scale measures the motion characteristics and energy levels of the earthquake; but for the purpose of a discussion of risks, the simplier Modified Mercalli scale is used. Unless otherwise noted, all further references to earthquakes will be made in intensity units.

EARTHQUAKE HAZARDS INDENTIFICATION

In discussing the effects of earthquakes, one of the first tasks is to differentiate between earthquake hazards and risks. It is helpful to look upon "hazards and risks" as "causes and effects." Seismic hazards exist anywhere on the surface of the earth where the crust can become stressed beyond its strength. When overstressing occurs, the bedrock ruptures and violent energy-releasing movements result. These hazards are natural phenomena related to reasonably well-defined geographic areas. On the United States map, Figure 1.4, one can see where earthquakes have occurred within recorded history. Where the events are clustered, the most conspicuous seismic hazards exist.

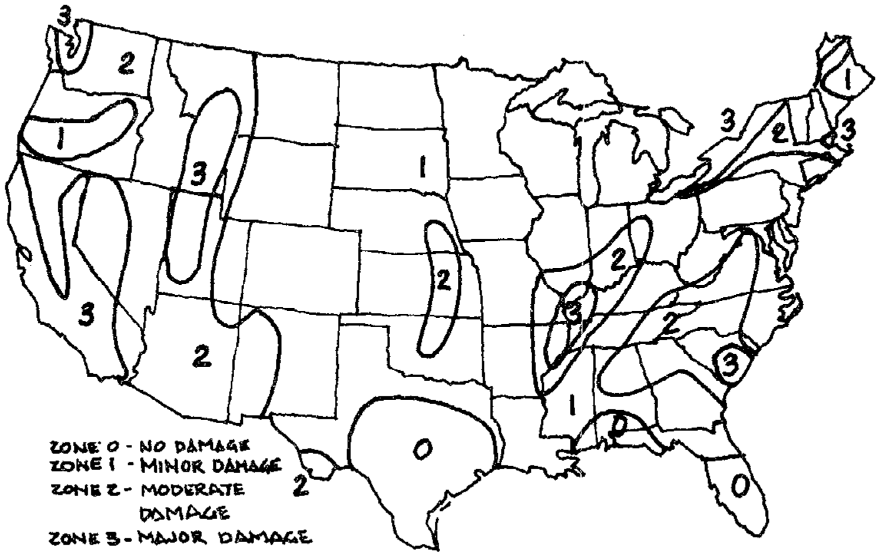


DAMAGING EARTHQUAKES OF THE U. S. THROUGH 1968
Figure 1.4

In contrast to hazards, seismic risks are human phenomena that exist only when something of human value is exposed to a hazardous event. One might say there is no risk from an earthquake in an unihabited area. Mathematically, risk is defined as the product of seismic hazard times the human value at risk, or: R p H x V.

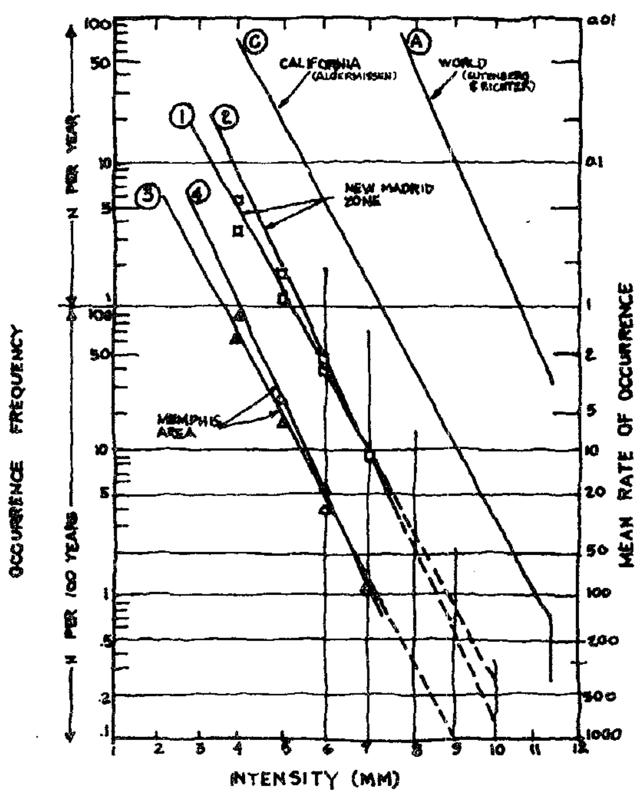
The hazard level of geographic areas is classified according to their geologic character and their seismic history. If an area is visibly faulted and exhibits recent movement (the last 100,000 years), it is judged to be a seismic hazard. Along the San Andreas Fault, in California, movement is large enough to be measured annually, so no one questions its being active. Other faults exhibiting no present movement have been judged by geologists using indirect methods. If an area has a history of experiencing earthquakes, even though faults are not visible, a seismic hazard is considered to exist. Most often these two indices, faulting and seismic history, are found in the same region, as for example, along the San Andreas fault in California.

Since 1949 seismologists have compiled a number of maps to reflect earthquake hazards throughout the United States. These are immensely useful as references for all seismic risk work. One hazard map in current use is shown in Figure 1.5 (by Algermissen) and is based on both regional geology and the most intense earthquake events known to have occurred in each region. If the earthquake occurrence map in Figure 1.4 is compared to the hazard map, Figure 1.5, one can see that where an abundance of earthquakes has occurred, the area is given a rating of 3. Notice the California area where faulting is visible and severe earthquakes have occurred; the area is zoned 3.



HAZARD MAP
(After Algermissen)
Figure 1.5

In addition to severity, engineers and planners are interested in knowing how often earthquakes have occurred in an area. This combination of intensity and frequency can best be shown graphically. Figure 1.6 shows the range of intensity of earthquakes and the frequency with which they have occurred in the central United States, California, and the world. Notice the average occurrence rate is inversely proportional to intensity; that is, the greater the intensity of an earthquake, the less frequently they occur. As an example, in the New Madrid, Missouri zone the greatest intensity known to have occurred is 10; it has occurred only once in 500 years.



RECURRENCE MODEL
Figure 1.6

In summary I wish to emphasize that seismic hazards are a continuing natural phenomenon that can affect many people in the United States. Earthquakes are related to fractures within the earth's crust; these faults may be visible (as in California) or buried beneath hundreds of feet of sediments (as in the central United States). Those geographic areas where earthquakes have occurred have been identified and mapped by seismologists. These hazard maps are readily available and are immensely useful in estimating seismic hazards of an area. In short, geologists and seismologists have made it possible for planners, design professionals, and building officials to take that first step toward seismic safety -- recognizing the hazard.

EARTHQUAKE RISKS

Recognizing a risk is man's first step toward creating a safe environment. Within the scope of this discussion, earthquake risks are measured in terms of property and life losses. One can approximate the risk level in a given seismic region in almost direct proportion to the population density and the investment in buildings. This approximation focuses attention immediately on buildings and what happens to them during a severe earthquake.

Buildings respond to a given earthquake in a wide variety of ways, depending primarily on the building design, construction, and the quality of maintenance. In order to better describe building behavior, engineers who have investigated damages following an earthquake have developed four general categories of construction based on the main structural system:

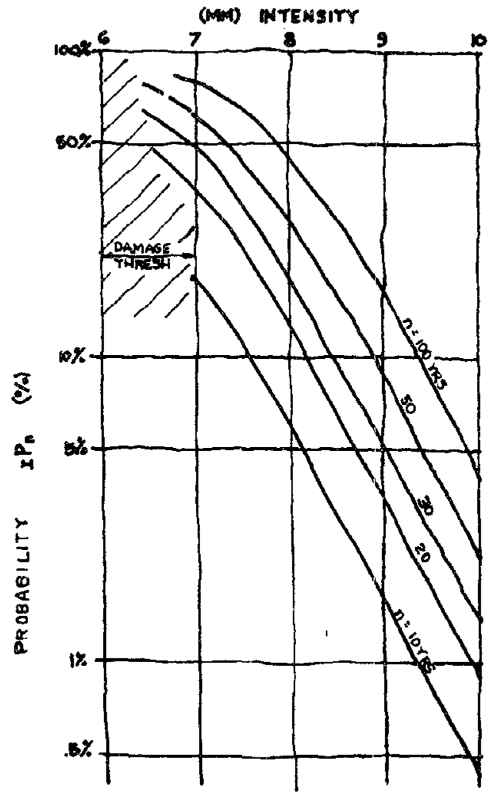
- Steel frame - A;
- Concrete frame - B;
- Ordinary masonry unreinforced and discontinuious concrete - C; and
- Wood frame - D.

In general, categories A and D have been found to experience the least damage while category C, expecially old masonry and unreinforced concrete, has experienced greatest damages. Another way of expressing a structure's seismic performance is in terms of ductility, the property opposite of brittleness. If a structure can yield without breaking, it possesses ductility -- it can absorb energy, it can withstand an earthquake. If one applies the test of ductility to conventional construction, it can be seen why investigators have found the least damages in types A and D. Thus property risks are to a large degree determined by the type of structural system used.

It is a recognized fact that almost all lives lost due to earthquakes can be traced to the failure of a building or part of a building. For this reason, the risk to lives in a given area is dependent on the type of structure occupied and the population densities. Population density patterns are a primary consideration when studying life losses. During the night-time the population distribution is very diffused since most people currently reside in single-family dwellings and a low density per structure results. During the day-time, our lives are dominated by group activities, so factories, offices, schools, and commercial centers house most of our population. The diurnal cycle is probably unique to each city, but a review of a number of distributions leads us to believe that typical variations are shown in Figure 1.8 as found in Memphis, Tennessee, in 1973. These cyclic variations in shelter and population densities result in a vastly different day-time community life risk as compared to night-time. Furthermore, diurnal population density can be very high in special types of structures such as schools, intense commercial areas, and large office buildings.

Life losses in a given structure are roughly propotional to property damages, especially structural damages. Most single-family dwellings are wood frame type D construction, which seldom collapses even in severe earthquakes. This and the low population density normally found in homes combine to produce minimal life risk during the night-time. On the other hand, many people spend the day in factories, schools, offices, and stores which are often constructed with materials and methods that provide little or no ductility. This, combined with high population densities poses a maximum life loss and injury risk.

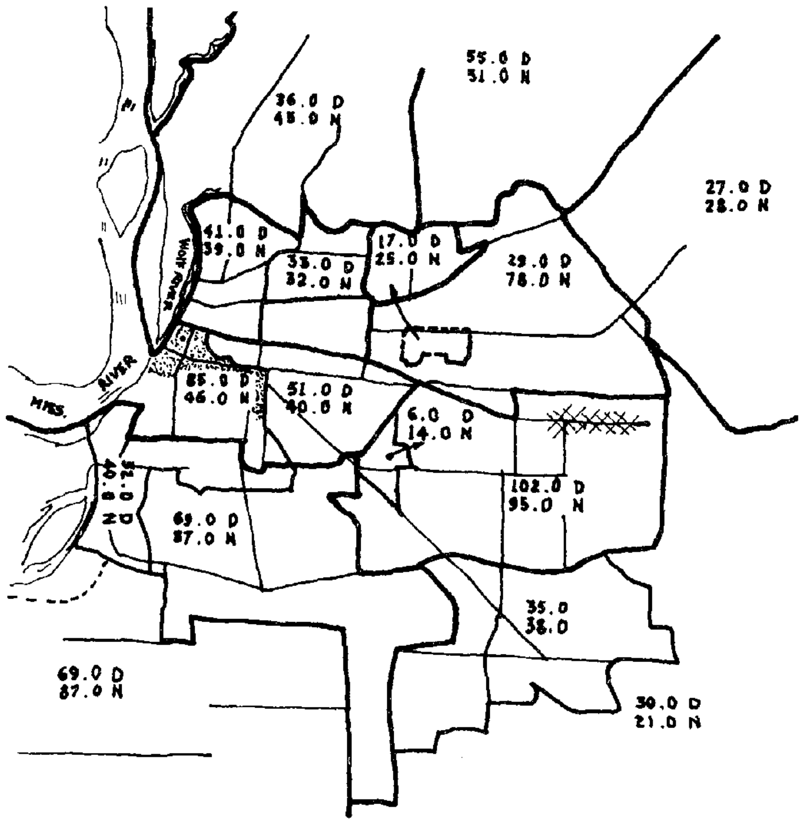
This graphical mode of expressing area seismicity was introduced by Gutenberg and Richter and is a widely used method to express both world and regional seismicity. In addition to giving one a sense of an area's seismic hazard, historical data in this form can be used to develop probabilistic predicitions for any time period, such as a building life span. In Figure 1.7, periods from 10 to 100 years have been used to express the probability of a damaging earthquake in Memphis, Tennessee.



PROBABILITY OF EARTHQUAKE FOR YEARS
OF EXPOSURE IN THE MEMPHIS AREA
Figure 1.7

Local geology or soil conditions have a significant influence on earthquake hazards. In general, one can say that the better a soil is for static foundation loads, the better it carries structures during an earthquake. In addition to high shear strength, firm soil transmits less violent vibrations than do weak soils; that is to say, firm soils amplify ground vibrations less than soft soils. From this generalization, one can rank soils seismically, descending from rock, to firm soil, and finally to weak soil. In order to quantify the influence of local geology at a given site, this generalization must be modified, preferably in consultation with a geotechnical engineer.

A special hazard arising from local geology is liquefaction. Liquefaction is a type of ground failure that results when vibrations cause the ground -- usually fine sands or silts -- to loose temporarily all of its shear strength. If the soil mass has a free boundary (as a river or hillside) it can flow like an avalanche, carrying everything with it. Liquefaction was responsible for the massive ground movements in the Turnagain Heights section of Anchorage, Alaska. Liquefaction can also be a hazard in level areas removed from river banks or hillsides. Under vibration, the ground, especially fine-grained waterbearing sands, can become soft to depths of 50 feet. If the crust is not strong enough to spread or bridge, the structues tip over as their foundations sink. The hazard of liquefaction was responsible for the wide-spread failure of well-built apartment houses in Niigata, Japan in 1964.



D-DAY POPULATION (1000)
N-NIGHT POPULATION (1000)

DIURNAL/NOCTURNAL POPULATION
DISTRIBUTION FOR MEMPHIS AREA
Figure 1.8

Dams are similar to buildings in that they are property at-risk; but unlike buildings, they pose a unique or double risk. Not only can the dam be destroyed, but it also poses an extreme danger to downstream inhabitants and their properties should it fail. Nevertheless, the universal attraction that a body of water holds for people has resulted in the development of residential or commercial areas around most dams. Throughout the United States one can find many dams, especially earthen dams, that were not designed for earthquakes. Thus, when an earthquake occurs near the dam, it places a dynamic load on the structure which may be large enough to cause a failure. During the recent San Fernando earthquake, the Van Norman Dam experienced a partial failure. Had the earthquake lasted a few more seconds, only a miracle could have saved many of the 75,000 people living downstream. The life loss would no doubt have exceeded the total life loss due to all other earthquakes in this country, even though the San Fernando event was of medium intensity.

When the engineer is faced with evaluating the seismic hazard of a dam, the engineering process is basically the same as for a dam failure from any other weakness. Most dam failures start by a crack opening through which water begins to flow. The process becomes progressively worse as time passes until a failure has occurred. The progressive failure may go on over a few days or a week. In either case, there is ample time to warn and evacuate those in danger. But in the case of seismic failure, there are additional considerations: the seismic failure of the dam structure is likely to be a massive, possibly total, failure of the entire dam, and the failure can be almost instantaneous. Both possibilities serve to amplify the risks by creating a great surge of water that allows no warning time for evacuation. The area that may be inundated, the depth of flooding, and the velocity of the flood waters are factors that must be considered in estimating the risks. In some states maps are being developed that define the land area that would be flooded to different depths in the event of a dam failure. Such maps are excellent tools for quantifying seismic risks.

Let me summarize. Risks to both property and lives are almost completely dependent on the structures man builds. Property losses are measured in terms of building losses. The life loss risk is dependent on population densities as well as the type of structures men erect. It is important that we realize that populations and structures are subject to man's control, and therefore, that there are the means by which human safety can be effectively provided.

MEASURING THE RISKS

The measurement of a risk is man's second step toward the creation of a safe environment. Measurements of seismic risks are made in both property and human terms, and they are made both before and after an earthquake occurs. After an earthquake, physical risks are measured through systematic field observations and records research by teams of experienced professionals, seismologists, engineers, economists, and sociologists. It is a somewhat laborious and expensive, yet fruitful, endeavor. The results are reasonably free of conflicting opinions and are publicly credible. On the other hand measuring the risks before an earthquake is dependent on an analytical process, which for a large area is very laborious and expensive and often arouses conflicts of opinion because of numerous uncertainties in the data. Both measurements are complementary and make unique contributions to human knowledge? Both serve society's decision-makers in many ways. Measurements after an earthquake are primarily useful to the seismologists and engineers as educational material, while estimates of future events are useful to all land planners, architects, governmental bodies, and emergency relief groups. In the course of time both processes will lead to substantial reductions of seismic risks.

The most conspicious losses that people suffer due to earthquakes are the direct property and life losses. The President's advisory council recently stated "Damaged or collapsing structures are the source of most life loss and injury during an earthquake; therefore, nearly all impacts of an earthquake ultimately revolve around damage to the structures."

Following this lead, let us first examine property losses as a means of measuring risks. Figure 1.9 shows the recorded property losses from major earthquakes occurring between 1865 and 1975 in the United States. These records were compiled by seismologists after the earthquake and offer a wealth of information from which a great deal has been learned.

In studying historic property loss records, one of the first things that we do is to account for significant changes that have occurred since the event, such as changes in money value. In our risk analysis work, we change the recorded value to a common point in time by using basic value indices such as those published by Standard & Poor. The revaluing process has the effect of vastly increasing the losses associated with older earthquakes. For example, the San Francisco earthquake of 1906 has a recorded value loss due to earthquake and fire of $524 million. The value index for 1974 is 5.4 which revalues the San Francisco loss to $2.8 billion. Translating all losses to 1974 dollars, the total increases to $5 billion.

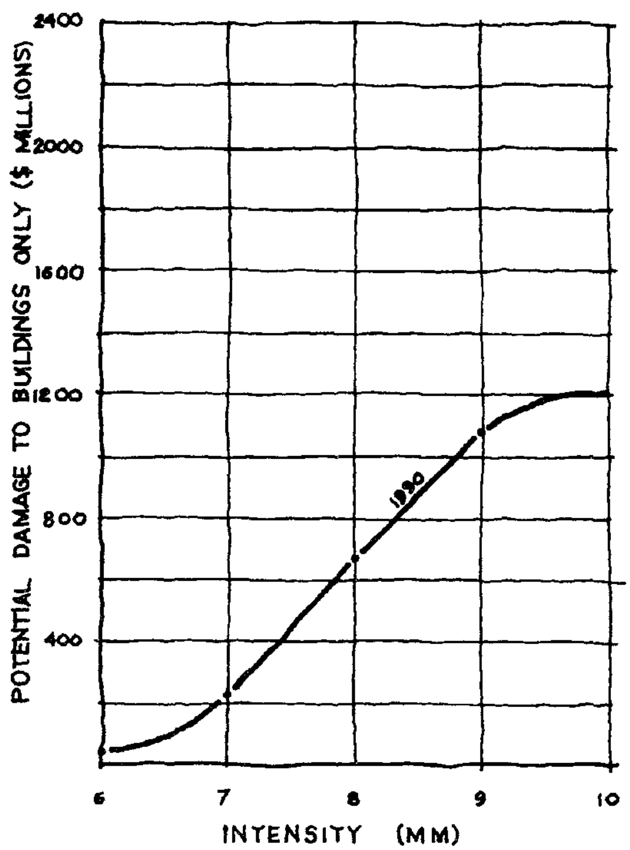
YEAR	LOCALITY	ACTUAL DAMAGE [1]	1974 DAMAGE [2]
1865	San Francisco, Calif	.5	1.6
1868	San Francisco, Calif	.4	1.5
1872	Owens Valley, Calif	.3	1.2
1886	Charleston, S.C.	23.0	125.1
1892	Vacaville, Calif	.2	1.1
1898	Mare Island, Calif	1.4	7.6
1906	San Francisco, Calif	524.0	2851.2
1915	Imperial Valley, Calif	.9	4.4
1918	Puerto Rico (tsunami damage)	4.0	13.1
1918	San Jacinto and Hemet, Calif	.2	.65
1925	Santa Barbara, Calif	8.0	22.5
1933	Long Beach, Calif	40.0	151.8
1935	Helena, Mont	4.0	14.3
1940	Imperial Valley, Calif	6.0	21.0
1941	Santa Barbara, Calif	.1	.33
1941	Torrance-Gardena, Calif	1.0	3.3
1944	Cornwall, Canada-Massena,N.Y.	2.0	5.6
1946	Hawaii (tsunami damage)	25.0	62.8
1949	Puget Sound, Wash	25.0	51.5
1949	Terminal Island, Calif (oil wells)	9.0	18.5
1951	Terminal Island, Calif (oil wells)	3.0	5.7
1952	Kern County, Calif	60.0	111.2
1954	Eureka-Arcata, Calif	2.1	3.8
1954	Wilkes-Barre, Pa	1.0	1.8
1955	Terminal Island, Calif (oil wells)	3.0	5.5
1955	Oakland-Walnut Creek, Calif	1.0	1.8
1957	Hawaii (tsunami damage)	3.0	5.3
1957	San Francisco, Calif	1.0	1.8
1959	Hebgen Lake, Mont	11.0	18.6
1960	Hawaii and U.S. West Coast (tsunami damage)	25.5	42.4
1961	Terminal Island, Calif (oil wells)	4.5	7.4
1964	Alaska and U.S. West Coast	500.0	794.1
1965	Puget Sound, Wash	12.5	19.5
1966	Dulce, N. Mex	.2	.3
1969	Santa Rosa, Calif	6.3	8.4
1971	San Fernando, Calif	553.0	675.0
1973	Hawaii	5.7	6.2
1975	Aleutian Islands	3.5	3.2
1975	Idaho/Utah (Pocatello Valley)	1.0	.9
1975	Hawaii	3.0	2.7
1975	Humboldt, Calif	.3	.27
1975	Oroville, Calif	2.5	2.3
	TOTAL	1878.0	5077.25

[1]Damage in millions of dollars at the time of the earthquake.
[2]Damage in millions of dollars corrected to 1974 dollar values.

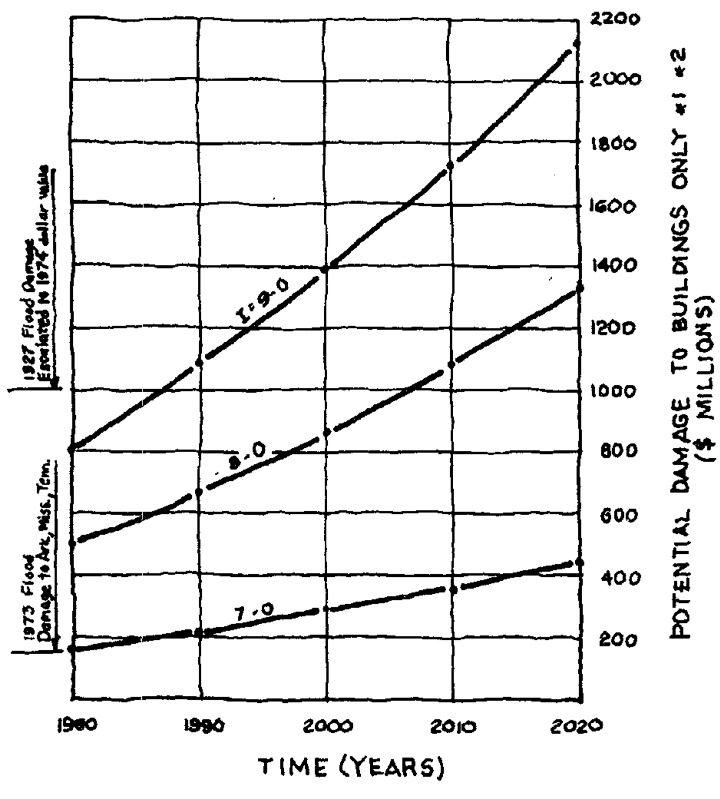
PROPERTY DAMAGES IN MAJOR U.S. EARTHQUAKES 1865-1975
Figure 1.9

The second means of measuring property risks is analytical. There is a natural human tendency to regard the analysis as a "crystal ball" operation, especially seismic analysis. However the process is, in fac, based on a unique combination of structural engineering and actuarial methods. Using the ground motion for different intensity earthquakes and damage records from past earthquakes, structural engineers have developed a matrix of nondimensional parameters that reflect the damage one can expect in a given category of structure for each intensity event. These parameters are known asmean damage ratios or MDRs. The construction in an area to be analyzed is inventories by grographical units into structural, soil, and maintenance categories and given present-day monetary values. The area is then hypothetically exposed to credible earthquakes that one can expect based on area seismicity. The method employed is a simulation based on a credible event and a credible human environment. The losses calculated for each type of inventoried construction and condition are then aggregated to determine the total property risk as shown mathematically in the following equation: Loss L = (MDR)V.

Analyses by simulation have been conducted for four large cities -- Memphis, Tennessee, San Francisco and Los Angeles, California, and Seattle, Washington, as well as numerous individual property developments. Figures 1.10 and 1.11 are taken from an analysis made by our firm for the city of Memphis, Tennessee. In Figure 1.10 potential losses are shown for the range of expected intensities 6 through 10 through the year 1990. Here it can be seen that property losses increase very rapidly as the intensity increases from 7 through 9. A full spectrum of expected losses is shown in Figure 1.11 for intensities 7, 8, and 9 for the years 1980 through 2020. Of especial interest is the top curve marked 9-0, for it illustrates a repeat of the New Madrid event of 1811-12. If the event occurred in 1990, the losses in building construction alone would amount to over a billion dollars; the losses in Memphis would be second only to those of San Francisco in 1906. Compare this potential loss with the loss occasioned by the great flood of 1927 along the Mississippi River. The total flood losses along the river, given in 1974 dollars, were $1 billion. A repeat of the 1811-12 earthquake would do more property damage in Memphis alone than the greatest flood did all along the Mississippi and Ohio valleys.



DAMAGE VS INTENSITY FOR MEMPHIS
Figure 1.10



*1 INCREASE DAMAGE VALUES BY 50% TO APPROXIMATE BUILDING PLUS CONTENTS DAMAGES.
*2 NO CORRECTION FOR INFLATION HAS BEEN MADE.

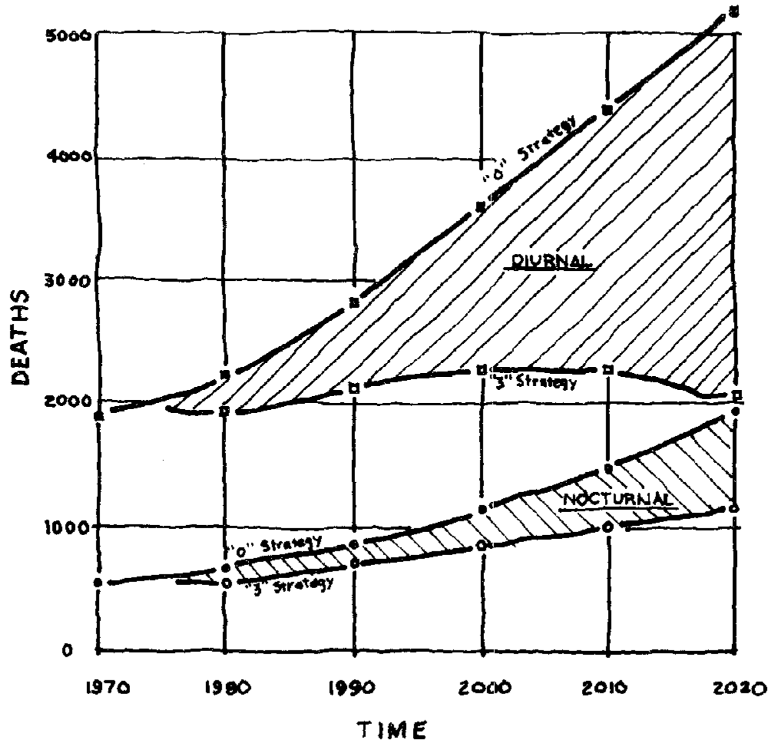
POTENTIAL MEMPHIS EARTHQUAKE DAMAGES TO BUILDINGS
Figure 1.11

Next, let us measure the earthquake risks in terms of life losses. From the President's Advisory Council, we have excerpted a table, Figure 1.12, showing lives lost in the United States since 1811 due to earthquakes. The over-riding message that comes from this list of life losses is shown in the greater danger posed to city populations. The New Madrid and San Francisco events were major earthquakes; yet the life losses varied from "several" in the sparsely populated frontier settlement of New Madrid to 700 in a city with a population of 400,000 persons. This difference in life losses is easily traceable to the construction and the population density. New Madrid was an area of villages built of low flexible wood structures in contrast to San Francisco with its higher buildings and closely built residences.

YEAR	LOCALITY	LIVES LOST
1811	New Madrid, Mo	Several
1812	New Madrid, Mo	Several
1812	San Juan Capistrano, Calif	40
1868	Hayward, Calif	30
1872	Owens Valley, Calif	27
1886	Charleston, S.C.	60
1899	San Jacinto, Calif	6
1906	San Francisco, Calif	700
1915	Imperial Valley, Calif	6
1918	Puerto Rico (tsunami from earthquake in Mona Passage)	116
1925	Santa Barbara, Calif	13
1926	Santa Barbara, Calif	1
1932	Humboldt County, Calif	1
1933	Long Beach, Calif	115
1934	Kosmo, Utah	2
1935	Helena, Mont	4
1940	Imperial Valley, Calif	9
1946	Hawaii (tsunami from earthquake in Aleutians)	173
1949	Puget Sound, Wash	8
1952	Kern County, Calif	14
1954	Eureka-Arcata, Calif	1
1955	Oakland, Calif	1
1958	Khantaak Island and Lituya Bay, Alaska	5
1959	Hebgen Lake, Mont	28
1960	Hilo, Hawaii (tsunami from earthquake off Chile coast)	61
1964	Prince William Sound, Alaska	131
1965	Puget Sound, Wash	7
1971	San Fernando, Calif	65
1975	Hawaii	2

LIVES LOST IN MAJOR U.S. EARTHQUAKES 1811-1975
Figure 1.12

In the 1970 risk analysis of San Francisco, Steinbrugge found that if an earthquake equal to the 1906 intensity should occur during a normal week day, about 10,000 would be killed. Our risk analysis of Memphis, Figure 1.13, indicates that if the New Madrid event had recurred in the year 1970, 1900 deaths would have been expected. Both these analytical analyses show that the earthquake risk is ten times greater in 1970 than in 1906. Hidden within these numbers is a terrifying fact; this risk is spread unevenly, terribly unevenly, across our population. Our analysis has shown that 60 percent of the deaths will be among 20 percent of the population, the school-age children.



POTENTIAL LIFE LOSSES FOR MEMPHIS - INTENSITY 9
Figure 1.13

Equipment and stock will also be lost throughout the manufacturing and commercial communities. We have estimated these losses for a number of geographic areas to be an average 50 percent of gross real property loss. Some industries will suffer a great deal more than 50 percent especially those using brittle materials such as glass and delicate equipment, especially electronic equipment.

In addition to the property and lives lost, there is the loss of economic productivity resulting from disruption of physical facilities and personnel. This translates into extended unemployment for many wage earners and failure for many smaller businesses. Various estimates place this loss in the same monetary range as the property losses.

In short in seismically active areas, both property and human risks are enormous. The records of past earthquakes indicate that when a major earthquake strikes a city, the property losses reach into the millions and life losses into the hundreds. Analyses of our large and modern cities show that property losses from future earthquakes will reach into the billions and life losses into the thousands. A risk with such measurement demands the attention of every profession that plays a role in the safety of our environment.

REDUCING THE RISKS

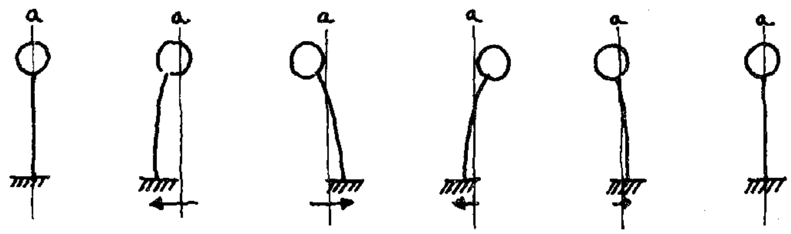
While recognizing and measuring the risks are the first and second steps toward a safe environment, we must recognize that nothing is accomplished until society takes those steps necessary to reduce these risks.

If we accept the twin premises that earthquakes will recur and that large property and life losses will occur if our cities are struck, it becomes meaningful to ask what can we do about reducing the risks. It appears that three areas of action are effective:

1. retrofit or build stronger and more ductile structures;
2. use land planning to avoid local geological hazards; and
3. spread the risks through insurance.

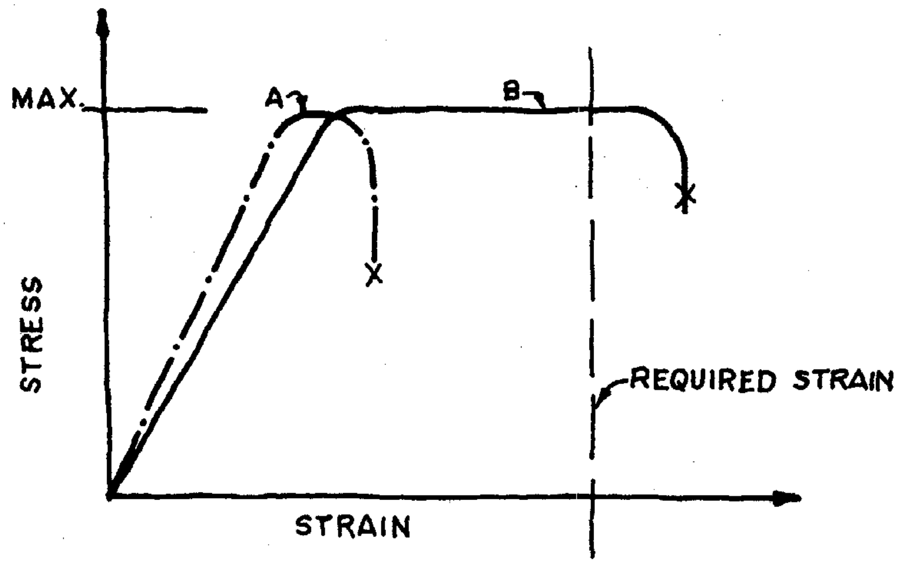
I quoted the President's Advisory Committee to the effect that damaged or collapsing buildings were the cause of most earthquake losses. Since this observation is unquestionably correct, it follows that progress toward stronger and more ductile structures, new or old, is the way that leads toward reductions in seismic risks.

Stronger structures begin with engineering design procedures that are uniquely addressed to seismic motion. The loads are horizontal and inertia-generated reactions that are proportional to the mass or weight of the structure and the ground acceleration. An inverted pendelum, as shown in the following sketch, is convenient to use as an illustration of the motion-forced interaction. The ground moves first one direction and then the other as the earthquake vibrations pass. The body at the top of the column is forced to follow the ground motion by the horizontal force generated in the deflected column. The motion or response of the body is different from the ground and depends on the exciting motion, the mass of the body, the column stiffness, and internal damping. If the column is very stiff, the motion of the body will approach that of the ground. If the column is very flexible, the body will move little except, for very long period ground motion. But a medium stiff column may be harmonic with the ground motion and cause the body to move much more than the ground. The question of a structure's dynamic response is central to any sound design procedure; fortunately, it is within the design capability of many structural engineers.



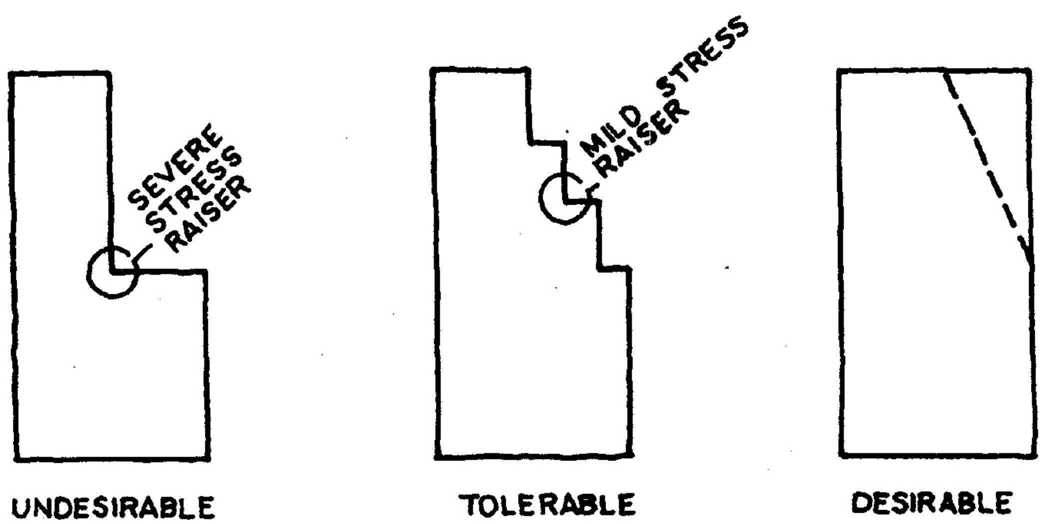
MOTION-FORCE-INTERACTION
Figure 1.14

While the responsibility for structural dynamics rests largely on the engineer, the architect has a vital and unique role to play in the dynamic design procedure. That role, when played well, reflects site influences, building shapes, material selections, component interfacing, and cost-benefit relationships. At the risk of over-simplifying structural behavior, it can be said that if a structure can sustain, in a satisfactory way, the deflections induced by an earthquake, it is a satisfactory structure. I prefer this criterion to a stress criterion because it (1) leads the architectural designer to develop a better structural intuition, (2) leads naturally to the recognition of the superior performance of ductile materials and assemblies, (3) recognizes the interaction of structural and nonstructural components, and (4) leads to the most inexpensive acceptable structure. Figure 1.15 shows a simple stress-strain diagram for two different materials. Both materials, A and B, sustained the same maximum stress, but material B is able to sustain the high stress over a much greater range of strain. If two structures built from materials A and B are exposed to a severe earthquake, structure A will collapse while structure B may sustain some permanent deflections, but will not collapse. The life loss associated with structure B will be minimal, while the loss associated with structure A can be almost everyone in and near the structure. Futhermore, if the non-structural components -- walls, windows, partitions, heating and ventilating -- are compatible with the deflections or are expendable, the property losses in structure B will be small, Structure B, which is ductile, is a satisfactory structure, while A, which is nonductile, is a human death-trap.



MATERIAL DUCTILITY
Figure 1.15

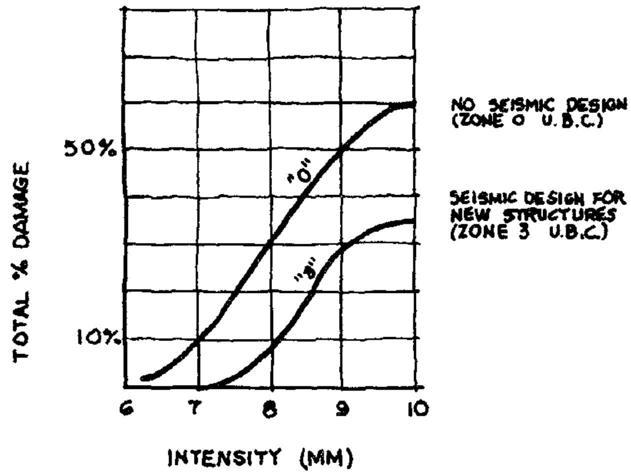
The shape of a structure influences its ability to withstand earthquake shaking. Asymmetrical plans and elevational offsets are two most frequently encountered shape features that reduce a structure's ability to sustain vibrations. These are illustrated in Figure 1.16.



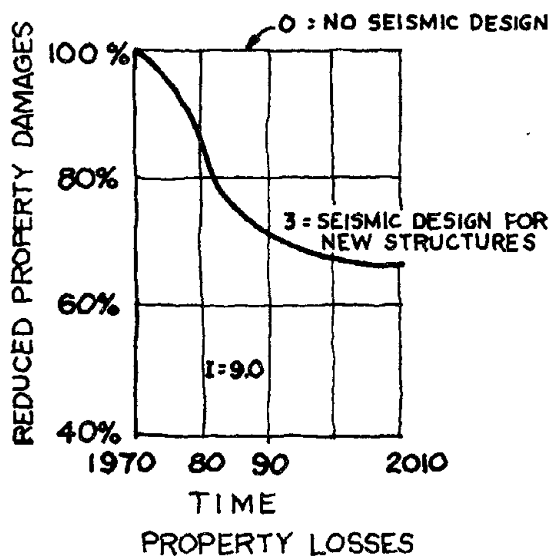
BUILDING SHAPE INFLUENCES
Figure 1.16

The effects of ductility and shape have been translated into design code requirements as can be found in the Uniform Building Code. Their influences in reducing building damage have been demonstrated over many years in many earthquakes. The code requirement grade is from 0 for zones of lowest seismicity to 4 for areas of highest seismicity. Figure 1.17 shows the effectiveness of Zone 3 seismic loading and other design requirements by comparing actual measured damages. Curve 3 represents average damages to type A structures designed to Zone 3 requirements, and

Curve 0 represents similar structures given no specific earthquake design consideration. Figure 1.18 shows the influence of introducing a Zone 3 requirement in Memphis Building Code in 1970. If all new construction, except residences, had been designed to Zone 3 requirements, the expected loss to all types of construction from a repeat of the 1811-12 earthquake would have progressively decreased to about 67 percent of that expected had no specific seismic design requirement been used. This reduction can now be made much greater since recent research in reinforced masonry has shown that residential loss can be greatly reduced at a very modest cost increase.

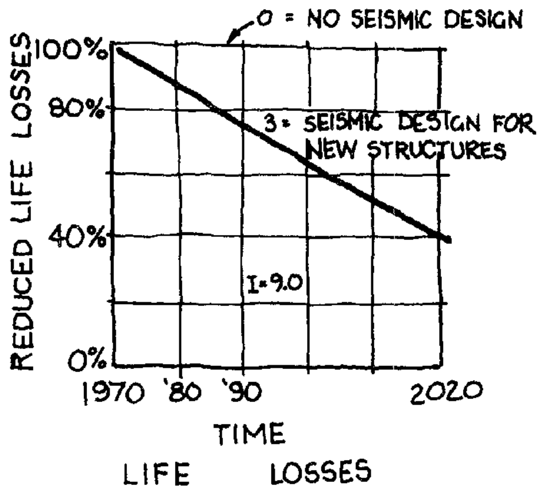


EFFECTIVENESS OF SEISMIC DESIGN
STEEL STRUCTURES
Figure 1.17



EFFECTIVENESS OF SEISMIC DESIGN
ALL STRUCTURES
Figure 1.18

In a manner similar to that described for property in the preceeding paragraph the effectiveness of the Uniform Building Code requirements can be measured in terms of a reduction of lives lost. Figure 1.19 shows the reduction in expected life losses in Memphis had all new construction after 1970 been designed for Uniform Building Code Zone 3 requirements. The effect is a drastic reduction in life risk, reaching a residual of 40 percent.



EFFECTIVENESS OF SEISMIC DESIGN
ALL STRUCTURES
Figure 1.19

One of the most difficult problems facing society today is how to utilize existing structures. In the urban environment many of the existing structures are sturdy buildings of ordinary masonry construction and obviously too valuable to demolish. Yet they have a notorious reputation for poor seismic performance. We are not prepared today to say how this problem is to be answered. Currently a multi-million dollar research program is underway that may provide a widely accepted basis for seismic evaluation of existing buildings. In addition to the technical difficulties associated with retrofit work, there is the professional liability that is totally unexplored. At present all that can be said is that the potential value of existing structures is so enormous that their use is inevitable, but the technical problems are complex and the professional liability can be very serious. Hopefully in the near future these adversities will be conquerable.

Consideration of risks of any kind is incomplete if insurance is not considered. This statement is probably more true of earthquake risks than most other serious risks that society faces. Insurance is a "risk spreading" instrument and obviously most useful when the losses are localized. The loss may be spacially and annually systematic but affect only a few persons at a time, such as auto and fire risks. In which case the losses are spread across the total population on an annual basis. Earthquake risk is somewhat different in that it is not annually or spacially systematic. This allows the risk to be spread both spacially and across time. If one refers back to Figure 1.9, it can be seen that only three earthquakes causing losses in the $500 million range have occurred in 100 years. Speaking approximately, this allows a spread of 33 years for major United States earthquake losses.

Earthquake insurance is available throughout the United States and at present the rates are very reasonable. For example, if you choose to insure a steel frame, fireproof office building valued at $1 million in Memphis, Tennessee, the annual premium would, at current rates, be about $1,300.

The availability of insurance raises, for the designer, the question of how much strength to build into a structure and how much insurance to carry. When we are advising a client building a new structure or rehabilitating an old one, we explore the different possibilities. Stated a bit more accurately, we look for that combination of structural resistance, human safety, expected damages, and insurance that offers the best "pay-off" on an investment.

In summary it can be said with confidence that working together, property owners and the design professionals, architects, engineers and planners, can materially reduce the seismic risks to both property and lives. This can be accomplished most effectively by designing structures with strength, ductility, and shape that respond best to earthquake ground motion. Many existing structures can be safely used, but great care must be exercised by the architects and engineers to find a safe and economical means to "rehabilitate" them seismically. If a structure is found to carry too great a risk; it should be reinforced or phased out of high risk service into a lower risk service rather than demolished. Local areas with significant soil liquefaction potential and the areas downstream from dams should be recognized as extraordinarily dangerous; the efforts of land planners and governmental bodies should be directed to encouraging low risk use of these areas. Seismic insurance is a responsible means of dealing with risks, and its use should be explored when a project is in its formative stages.

Seismic effects are present, to some degree, throughout the United Sates. As time passes and our societal life patterns become more urbanized, the potential for seismic catastrophe increases. In fact, the most recent research suggests that between 1906 and 1970 the seismic risk has increased over ten times. When one looks at the great risk that society faces by ignoring seismic exposure, there can be no doubt that the potential rewards are worth our maximum efforts to provide a safer seismic environment.

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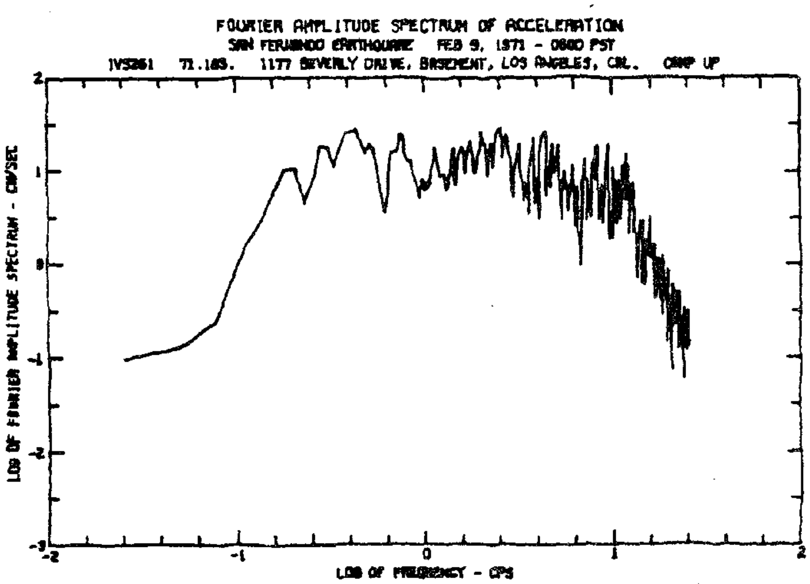
Chapter 2. NATURE OF EARTHQUAKES

Otto W. Nuttli

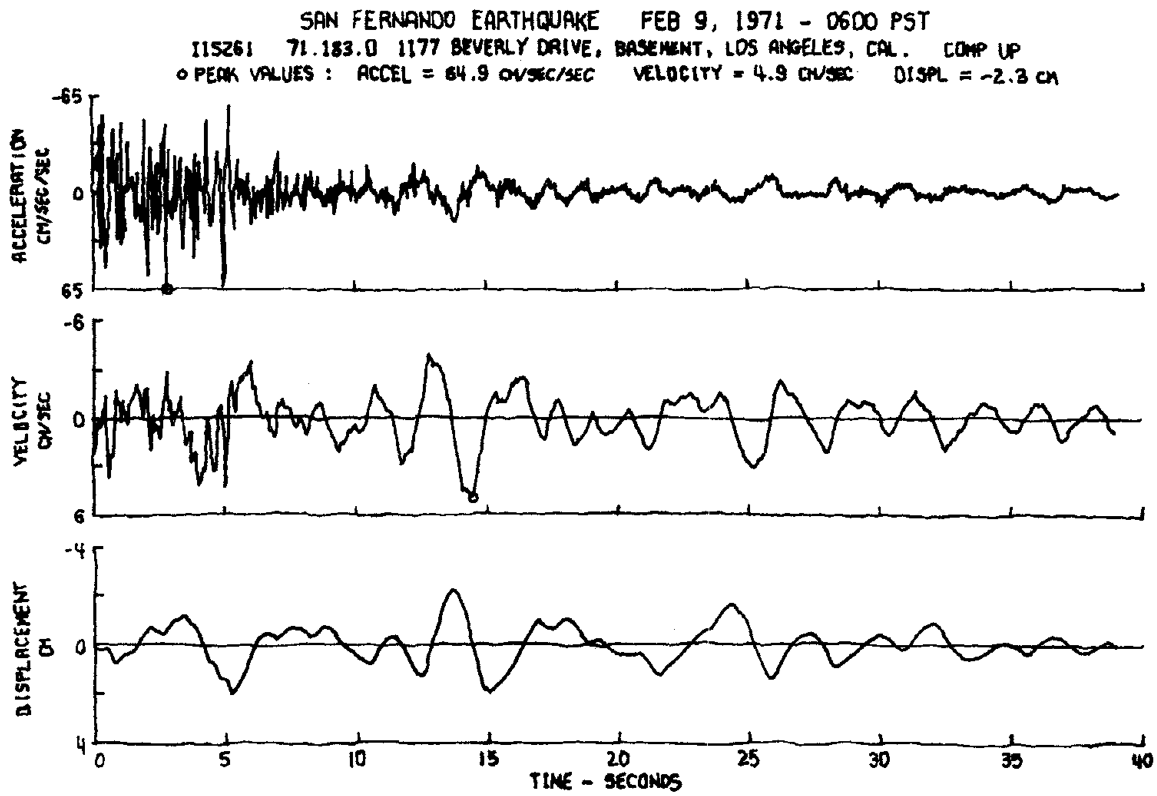
MECHANICS OF AN EARTHQUAKE

An earthquake is a sudden dislocation or displacement of rock mass along a more-or-less plane surface called the fault surface. It results from the localized build-up of stress and strain until the point where the cohesive and frictional forces holding the rock mass together are exceeded. Usually, this motion takes place along a pre-existing fault surface, where earthquakes previously had occurred.

The ground motion resulting from the earthquake can be characterized by its spectrum, i.e., by a curve that shows the displacement of the gtound as a function of the wave frequency of the ground shaking. Alternatively, the ground motion can be represented by its time history, which shows the displacement (or velocity or acceleration) of the ground as a function of time at a particular place. Figure 2.1 shows the Fourier amplitude spectrum of acceleration for the vertical component of ground motion in the basement of a building 40 kilometers from the epicenter of the 1971 San Fernando, California, earthquake. Note that the spectral acceleration is practically constant for frequencies of 0.15 to 10 Hz, and that on the log-log plot it has a slope of approximately 2 at frequencies less than 0.15 Hz and a slope of about -3 at frequencies greater than 10 Hz. For longer earthquakes, the spectrum would be flat for lower frequencies. The fall-off at the high frequencies is due to absorption of wave energy, which increases as the wave frequency increases. The corresponding time histories are shown in Figure 2.2. The acceleration record is measured by an instrument, and the velocity and displacement records are obtained by numerical integration of the acceleration record. The amplitude of the wave motion, its frequency content, and its duration will vary from earthquake to earthquake, and also will depend upon distance from the earthquake epicenter to the point where the time history is recorded.



FOURIER AMPLITUDE SPECTRUM OF GROUND ACCELERATION
Figure 2.1



TIME HISTORIES OF GROUND MOTION
Figure 2.2

There are a number of factors that determine the time history of the ground motion or the level and shape of its spectrum. The principal factors are

- The amount of displacement along the fault. This may vary from as much as 10 meters, for a very large earthquake, to less than a centimeter for a small earthquake. The 1971 San Fernando earthquake had a fault displacement of about one meter;

- The area of the fault surface across which displacement occurs. In extreme cases the fault length can be as great as 1000 kilometers and the width as large as 50 to 100 kilometers. The 1960 Chilean earthquake represents such an upper limit. More typical values for major earthquakes are fault lengths of about 100 kilometers and widths of 20 to 50 kilometers. The smallest earthquakes of damage potential have fault areas of about 1 to 10 kilometers^2;

- The depth of the earthquake. Earthquakes of very shallow depth – less than a few kilometers – can cause much greater damage than earthquakes of the same strength at somewhat greater depth. The greater damage potential of the shallow earthquakes results both from the breaking of the ground surface and the strong excitation of certain kinds of earthquake waves;

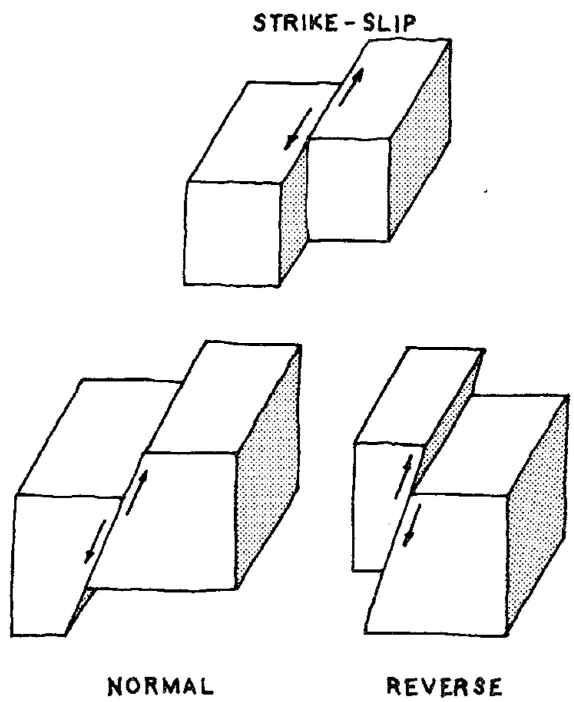
- The abruptness of the fault displacement, as well as the total time of rupture. The latter will vary with the fault length from as much as 100 seconds for a very large earthquake to a fraction of a second for a small one;

- The stress released by the earthquake, i.e., the difference between the initial stress before rupture and the stress after rupture. Typically this is about 10 to 100 bars or 10 to 10 dynes/cm (150 to 1500 lb/in^2).

From geologic field studies and from an analysis of seismograms (the records of earthquake ground motion made by instruments), we are able to distinguish three basic kinds of faulting shown in Figure 2.3. These are

- Strike-slip faulting, in which the slip motion is horizontal and the fault plane ususally is near-vertical. The San Andreas fault of California is a calssic example of a strike-slip fault.
- Normal faulting, in which the slip motion is in the direction in which the fault plane dips, with the motion as if gravity were the dominant force. This kind of faulting could occur if there were tension perpendicular to the long direction of the fault, to pull the fault blocks apart and let the one slide down due to the action of gravity. The Wasatch fault in Salt Lake City is an example of a normal fault.
- Reverse faulting, in which the slip motion is exactly opposite to that which occurs in normal faulting. Reverse faulting would occur if there were compression perpendicular to the long direction of the fault, forcing one fault block to slide upward over the other. Reverse faulting was responsible for the 1971 San Fernando, California, earthquake and the 1964 Alaskan earthquake.

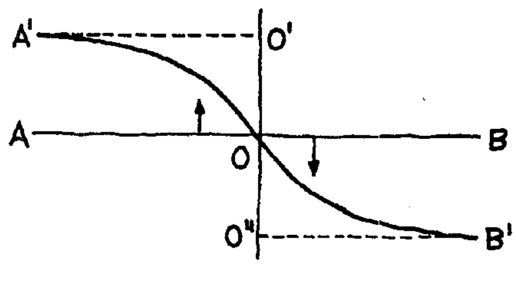
Earthquakes may have a combination of strike-slip and normal faulting, or of strike-slip and reverse faulting. From the point of view of an architect, it is important to distinguish among the three types of faulting because of the character of ground shaking produced (e.g., strike-slip is relatively rich in horizontal ground shaking, whereas normal and reverse are relatively rich in vertical ground shaking).



PRINCIPAL TYPES OF FAULTS
Figure 2.3

CAUSES OF EARTHQUAKES

The build-up of stress and strain in a localized volume of the Earth is the immediate cause of earthquakes. Measurements made by strainmeters and by accurate surveying demonstrate proof that the rock layers near the Earth's surface are strained or deformed before an earthquake. Figure 2.4 illustrates in schematic form the deformation that preceded the 1906 San Francisco earthquake. In the figure the line AOB corresponds to the unstrained state of the Earth. As strain accumulated, the line AOB was deformed to the line A'OB' just prior to the earthquake. After the earthquake, the rock masses slipped across the San Andreas fault so that the original line AOB became the lines A'O' and O"B', leaving the Earth once more in an unstrained state.



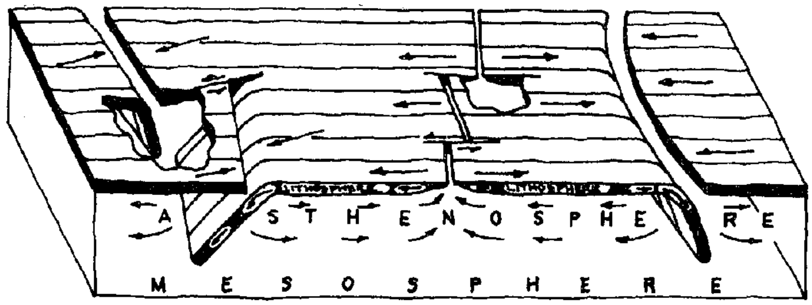
DEFORMATION ASSOCIATED WITH STRESS ACCUMULATION ALONG A FAULT (0'–00")
Figure 2.4

If we seek to find the ultimate cause of earthquakes, we must search for what is responsible for the localized accumulation of stress and strain. A related matter of interest is the prediction of where stress accumulation will occur, so that we can identify the regions of potentially large earthquakes and take adequate steps to design structures to withstand these future earthquakes.

The theory of plate tectonics helps us to better understand both why earthquakes occur and why they occur where they do. The theory postulates that the outer part of the Earth consists of a fairly rigid layer 50 to 100 kilometers thick, called the lithosphere, beneath which there is a less rigid (and perhaps partially melted) layer about 100 to 200 kilometers thick called the asthenosphere. Below the asthenosphere, there is more rigid rock matter called the mesosphere. The lithosphere is decoupled from the mesosphere and can be though of as sliding on top of the asthenosphere.

Figure 2.5 shows in schematic form the principal movements associated with plate tectonics. Hot rock ascends from the asthenosphere and breaks through the lithosphere to displace the lithosphere in either direction away from the spreading line. Such spreading activity is presently taking place along the mid-Atlantic ridge, the east Pacific rise, the mid-Indian ocean ridge, the Red Sea and east African rift valley, and the Gulf of California. In each case new oceanic lithosphere is being forced apart, resulting in horizontal tensional stresses perpendicular to the spreading ridge. Thus we would expect that earthquakes associated with such activity would have normal faulting. Seismographic observations confirm that this is indeed the case.

There is much geologic evidence to indicate that the surface area of the Earth, and thus its radius, has not increased significantly in geologic time. Therefore, if new lithosphere is being created at the spreading centers at the rate of 1 to 10 centimeters per year, there must be a consumption of lithosphere at a comparable rate taking place at some location on the Earth. The places where this occurs are called subduction zones. There the oceanic lithospheric crust, which is denser and thinner than the continental lithospheric crust, bends and plunges into the earth. This subduction can take place to depths as great as 600 kilometers, which marks the greatest depth of earthquake occurrence. Subduction is taking place along the west coast of Central and South America, along the coast of Alaska and the Aleutian Islands, and along the coast of the west Pacific. Earthquakes associated with subduction zones usually involve reverse faulting, which indicates a compressional stress regime.

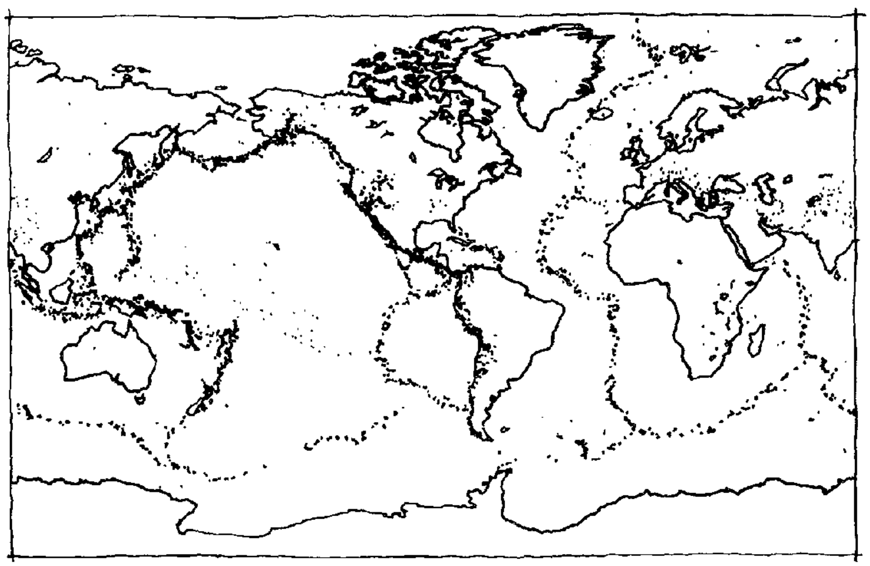


THE NATURE OF PLATE MOVEMENTS
Figure 2.5

When the spreading centers and the subduction zones are mapped, it is found that they are not long continuous features; rather, they consist of a series of broken line segments offset from each other. Along the line on which they are offset, the two plates slide horizontally against each other. This gives rise to a particular kind of strike-slip faulting known as transform faulting. The San Andreas fault in California and the Anatolia fault in Turkey are examples of transform faults.

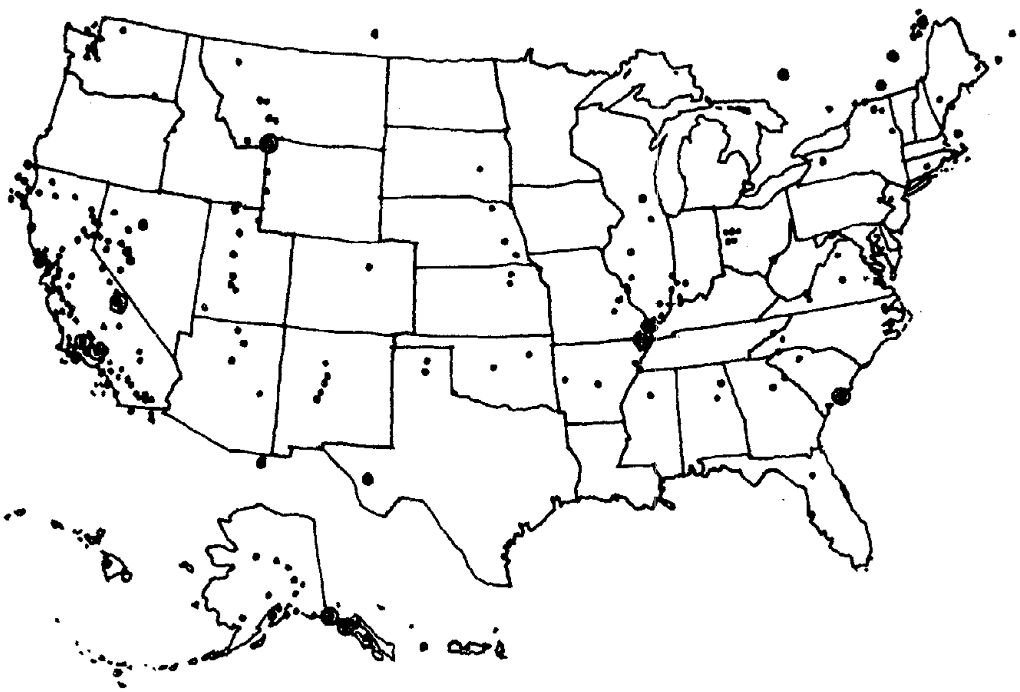
When two continental lithospheric plates come together from opposite directions, there is no subduction, as in the case of a continental and oceanic plate, but rather a collision. This causes large compressive stresses and a buckling up of the lithosphere to create a mountain range. The best example of this is the Himalayas, produced by the collision of the Indian continent which is moving north relative to the Asian continent.

Most of the world's earthquake activity is confined to spreading ridges, subduction zones, transform faults, or collision zones. Figure 2.6 shows the world seismicity. On it the subduction zones and ridges are clearly defined. The epicenters of the deep earthquakes are to the continental side of the shallow ones, indicating that the subduction zone is plunging under the continent.



WORLD SEISMICITY
Figure 2.6

This same figure also reveals that there are a few earthquakes in the interior of lithospheric plates, where no earthquakes are expected by the theory of plate tectonics. Figure 2.7 shows this more clearly. Here are plotted the epicenters of all known damaging earthquakes in the United States from historic times through 1970. Most of the western earthquakes can be explained by plate tectonics. But those east of the Rocky Mountains remain an enigma. Some were truly major earthquakes, such as the one that occurred in the St. Lawrence valley in 1663 and the three that occurred in the Mississippi valley in the years 1811-1812. All the earthquakes plotted in this figure were severe enough to cause structural damage. Many of them also happen to have been located in the most densely populated section of the United States, where they might cause widespread damage, injury, and loss of life when they occur again in the future.



DAMAGING EARTHQUAKES OF THE U.S. THROUGH 1963
Figure 2.7

Yet the fairly sizable number east of the Rocky Mountains are not predictable on the basis of plate tectonic theory. Our understanding of the cause of these eastern earthquakes is inadequate; but it appears that the continental interior is subjected to east-west compressional stress due to the westward pushing of the American plate from the mid-Atlantic ridge, and that this compressional field is locally modified by geologic structures such as arches and basins. These local modifications of the compressive stress field may result in concentrations of stress which could cause the earthquakes.

IDENTIFICATION OF ACTIVE FAULTS

A fault is a surface across which there has been relative movement of the rock masses, either by slow gradual movement, called creep, or by sudden movement associated with earthquakes. Faults are identified and maped by geologists and geophysicists, either by direct visual observation of offset of rock layers or by indirect physical measurements.

Faults on which there has been no movement in recent geologic time are called inactive or dead. More or less arbitrarily a fault is called inactive if there has been no movement in 100,000 years and if there is no creep or earthquake activity presently associated with it.

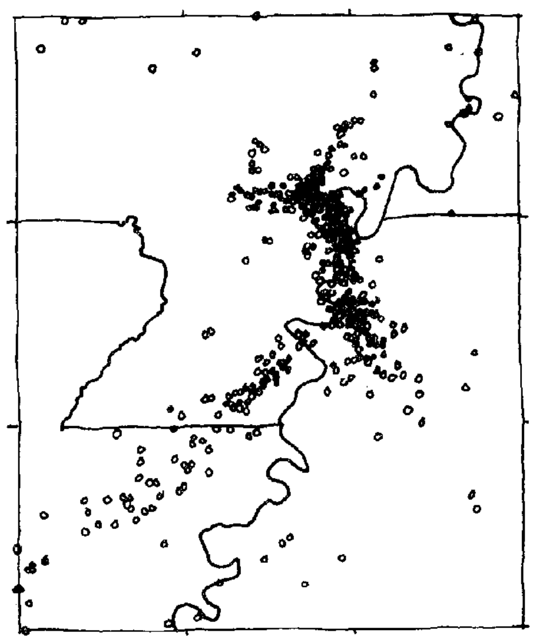
There are a few places in the world where earthquakes are accompained by visible surface movement of the rock masses. Notable among them is California, in particular the San Andreas fault. More generally, though, the slip associated with earthquakes occurs deep within the Earth and does not reach the surface.

Seismic history provides one means of identifying active faults. If the fault is visible at the Earth's surface, and if earthquake epicenters lie on this surface trace, we can conclude the fault is active. This certainly is the case for the San Andreas fault. But there are some problems. If the earthquakes occurred before seismographs were in operation, their location had to be estimated on the basis of their effects on structures and people. Particularly if the density of population was low or not uniform, errors in epicenter location of 50 kilometers can easily occur; sometimes they can be as much as 100 kilometers. Even at the present time, errors in epicenter location of five to ten kilometers can readily occur. If the epicenter is in a remote location, distant from the nearest seismographs, the error can be several times that much. For these reasons it sometimes is difficult to decide which faults of neighboring ones are active.

Geologic mapping of displacements of recent sedimentary rock layers, such as in lake beds,can provide not only evidence of recent fault movement but also an indication of the total amount of fault displacement in a certain period of time. From this, information can be deduced about the rate of earthquake activity along the fault and it can be classified as to its degree of capability to produce future large earthquakes. Sometimes the mapping is done by aerial photography or by observation from earth satellites, where fault lineations and offsets become more apparent when viewed from a distance. Other times a trench is dug across a fault to see the displacement of rock layers beneath the Earth's surface.

In many parts of the world where earthquakes occur, there is no surface evidence of faulting. The presence of earthquakes, however, indicates that there must be a fault which is active. There are two techniques for locating active faults in such a case. One is called the microearthquake technique; the other, the focal mechanism technique. Microearthquakes are very small earthquakes, too small to be felt, which occur frequently along active faults. They can only be detected by sensitive seismographs located close to the microearthquakes themselves. In general they tend to occur all along the fault; thus a map of their epicenters tends to outline the fault.

Figure 2.8 is an example of micorearthquake map of the New Madrid fault zone of the central Mississippi valley. The dots indicate the location of microearthquakes for a 24-month time interval



MICROEARTHQUAKES IN THE NEW MADRID SEISMIC REGION
Figure 2.8

as recorded by seismograph stations of the Saint Louis University network. Even though all evidence of the fault is covered by thousands to ten of thousands of feet of recent river sediment, the location and extent of the fault are clearly outlined by micorearthquakes. Notice for example, the lineation from east central Arkansas to western Tennessee, the offset to the north, and then a suggestion of another northeastward-trending lineation. Before the network of seismograph stations was installed, the New Madrid earthquakes were thought to occur in a diffuse zone about 100 kilometers wide and 200 kilometers long.

There are a number of microearthquake networks in the United States. At present they include ones in western Washington, central California, southern California, Utah, Nevada, Oklahoma and Kansas, Minnesota, western Ohio, the central Mississippi Valley, eastern Illinois and western Indiana, South Carolina, western New York and Pennsylvania, and New England. Some have just recently gone into operation. These have not recorded a sufficient number of earthquakes to delineate fault zones. The maping of the central Mississippi valley network has been one of the more successful operations to the present time.

The focal mechanism technique for identifying active faults makes use of the fact that the radiation of earthquake wave energy from a fault varies with direction. If we can observe this variation of wave energy with direction from observations at seismograph stations distributed at all azimuths around the source region, we can deduce the orientation of the fault surface and the direction of slip motion on the fault surface. In this way we can decide if epicenters of neighboring earthquakes have a common fault surface and, if so, determine the orientation of the surface. As an example, there were two fairly large earthquakes with epicenters on the southwest lineation in Figure 2.8. One occurred at the extreme southwest end of the trend, the other near the northeast end. Focal mechanism studies of these earthquakes indicated that in both cases the fault plane trended NE-SW, along the line of epicenters, and that the motion on it was strike-slip with the western side moving to the south relative to the eastern side.

There are potions of the United States where no surface fault motions are observed and where micorearthquake and focal mechanism studies have either not been carried out or have been unable to delineate active faults. Earthquakes have been observed to occur in these regions, but usually rather infrequently and usually only of minor to intermediate size. Yet they do present some potential threat for injury to people and damage to buildings, so they must be taken into account when structures are designed. The procedure normally employed is to identify the boundaries of a zone, on the basis of seismic history and structural geology, in which the earthquakes are likely to occur and to assume that the maximum magnitude earthquakes are associated with that seismotectonic region can occur anywhere within it. In general the poorer our understanding of the earthquakes of a region, the broader will be the area of the seismotectonic zone.

GEOLOGIC HAZARDS

Geologic hazards due to earthquakes fall into four broad categories: surface faulting, ground failure, tsunamis and seiches, and ground shaking. Each one will be discussed in turn.

When a fault breaks at the surface during the occurrence of an earthquake, there may be vertical or horizontal displacements of as much as one to ten meters over a distance of ten to a thousand kilometers. At any given place this displacement will be sudden, occurring within a second or less, thus giving rise to large accelerations and forces. Structures near the fault can experience heavy damage. You probably have seen photographs of road offset, railroad tracks bent, and pipelines damaged. However, in some cases small structures near the fault remain intact and unscathed. Although damage due to surface faulting can be spectacular, surface faulting effects are insignificant in terms of total earthquake hazard compared to those of ground shaking.

Ground failure can produce several types of damage, due either to landslides or liquefaction. (Liquefaction is a condition where by the soil losses its cohesiveness and behaves like a liquid.) The effects of landslides can be severe, particularly in mountainous regions or in river valleys. The 1970 Peru earthquake triggered landslides which caused mud to cover villages to the rooftops, resulting in the deaths of 50,000 to 70,000 people. The New Madrid, Missouri, earthquakes of the winter of 1811-1812 caused caving in of the banks of the Mississippi river and its tributaries over distances of hundreds of miles.

Some sandy and clay soils, when subjected to several or more cycles of sufficiently large shaking, lose their rigidity and behave like a fluid. Structures resting on these soils can, as a consequence, be severely damaged. In the 1964 Niigata, Japan, earthquake some tall buildings remained intact but completely tipped over, so that the occupants of the upper stories could practically step out onto the gournd. Often liquefaction is accompanied by the eruption of sand, water, gasses, and other material. Sand craters were common in the 1811-1812 New Madrid earthquakes over an area of thousands of square kilometers. Sand craters and earthquake fissures made previously fertile farm land so undesirable that the U.S. Congress passed the first disaster relief act in 1815 to give the farmers of southeastern Missouri and the surrounding area new farm land to the north. Earthern dams are particularly vulnerable to liquefaction, and careful attention must be given to their design to ensure that they will not fail by flowing.

A tsunami, sometimes called an earthquake tidal wave, is a water wave in the ocean that is generated by an earthquake or by a sudden displacement of the ocean floor. On the open ocean the amplitude of the water wave is only a few centimeters; but as the wave approaches shallow seas, the waves pile up and can attain heights of tens of meters. Thus islands and coastal areas can experience severe damage from a wall of water generated by an earthquake thousands of kilometers away. For example, the 1960 Chile earthquake produced tsunamis that were destructive in Hawaii and Japan.

Seiches are the natural or free oscillations of enclosed bodies of water, such as lakes, produced by earthquake-generated waves. They can be several meters in height and can result in loss of life along the shoreline. The 1755 Portugal earthquake produced seiches as far away as Scandinavia and Findland, and the 1964 Alaska earthquake produced seiches two meters high in the coastal areas of Texas and Louisiana.

Although surface faulting, ground failure, and tsunamis and seiches can sometimes result in catastrophic damage or loss of life, most of the damage caused by earthquakes is a result of ground shaking. When the earth suddenly moves in an earthquake, there are a number of waves set up that carry the disturbances great distances. Seismologists classify these waves into two principal types, and subdivide each of these. The basic division is into body and surface waves. The former are waves that travel through the interior of the Earth. sometimes as deep as to the very center, although at the distances involved in earthquake damage their depth of penetration probably is no more than 50 kilometers. Surface waves, by contrast, travel in paths parallel to the surface of the Earth and their amplitude dies off rapidly with depth. Except possibily in the very near source region, surface waves are responsible for most of the damage to structures.

Body waves are of two varieties: compressional (also longitudinal or P) and shear (also transverse or S) waves. The compressional waves cause small elements of volume of the Earth to contract and expand as the waves travel from one point to another. They are called P waves because they have the highest velocity (which depends on the incompressibility, the rigidity, and the density of the rock) and thus are the first, or primary waves to arrive. The compressional waves are called longitudinal waves because the earth particles move back-and-forth in the direction of the ray as the wave advances. The shear wave is called an S wave because it is the second to arrive. There velocity depends on the rigidity and density of the rock mass. It is also called a transverse wave because the earth particle moves perpendicular to the ray as the wave advances.

The surface waves are named after the two men who first predicted their existence by mathematical analysis, Rayleigh and Love. Both waves share a common property: their velocity depends on their wavelength. Thus a bundle of surface waves near the source spreads out into a long dispersed train of waves some distance away. Rayleigh waves have elliptical motion in a vertical plane, whereas Love waves have a horizontal, transverse motion. Surface waves have lower speeds than body waves, so they always arrive later than the body waves. The amplitude or surface waves die out rapidly with distance perpendicular to the Earth's surface.

There are two ways in which we can analyze the ground motion at a given point: time, in which we get a history of the ground displacement, velocity, or acceleration; and frequency, where we get a Fourier amplitude spectrum of the ground motion. Of particular interest to the architect or structural engineer are the amplitude of the ground shaking, its duration, its frequency content, and its division into vertical and horizontal components of motion. If we could describe exactly the source motion, and if we knew exactly the effect of wave transmission through the Earth, we could describe the ground motion at any selected point. These represent long-range goals of the seismologist and earthquake engineer. At present we must frequently resort to empirical methods to estimate the ground shaking.

Before there were seismographs, the only way to measure the severity of ground shaking was to observe its effects on people, the Earth, and structures. Such a measure of ground shaking is called earthquake intensity. In the United States we use the Modified Mercalli (M. M.) intensity scale. Figure 2.9 shows the divisions of the scale, from the lowest intensity I to the highest XII. Figure 2.10 is an example of the intensity distribution for the New Madrid earthquake of December 16, 1811, the first of three major earthquakes to occur during the winter of 1811-1812. Note the large distances in which the damaging intensities (VII and greater)occurred. Earthquake intensity maps continue to be constructed for recent earthquakes because they provide in a simple way a unique picture of the damage caused by the earthquake.

1. Not felt. Marginal effects from distant long-period large earthquakes. (Rossi-Forel (RF-I)

2. Felt by persons at rest, on upper floors, or favorably placed. (RF-I to II)

3. Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake. (RF-III)

4. Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt of a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In *the upper range of 4, wooden walls and frames crack.* (RF-IV to V)

5. Felt outdoors; directions estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start change rate. (RF-V to VI)

6. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, and so on, off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring. Trees, bushes shaken visibly, or heard to rustle. (RF-VI to VII)

7. Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, unbraced parapets, and architectural ornaments. Some cracks in masonry C. Waves on ponds; water tubid with mud. Small slides and caving in along sand of gravel banks. Large bells ring. Concrete irrigation ditches damaged. (RF-VII)

8. Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes. (RF-VII to IX)

9. General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames racked. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake foundations, sand craters. (RF-IX)

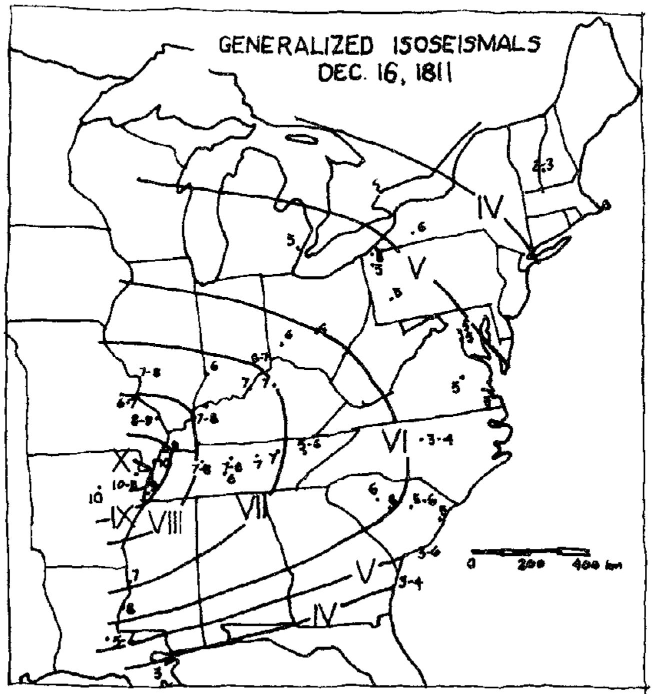
10. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly. (RF-X)

11. Rails bent greatly. Underground pipelines completely out of service.

12. Damage nearly total. Large rock masses displaced. Lines of slight and level distorted. Objects thrown into the air.

MODIFIED MERCALLI INTENSITY SCALE

Figure 2.9

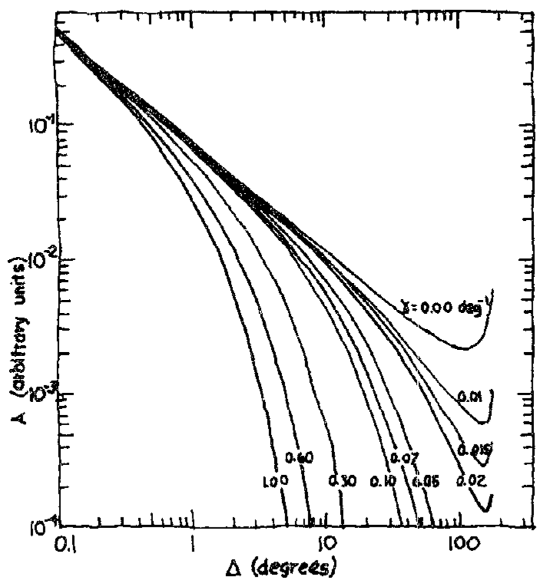


INTENSITY MAP FOR DECEMBER 16, 1811
NEW MADRID EARTHQUAKE
Figure 2.10

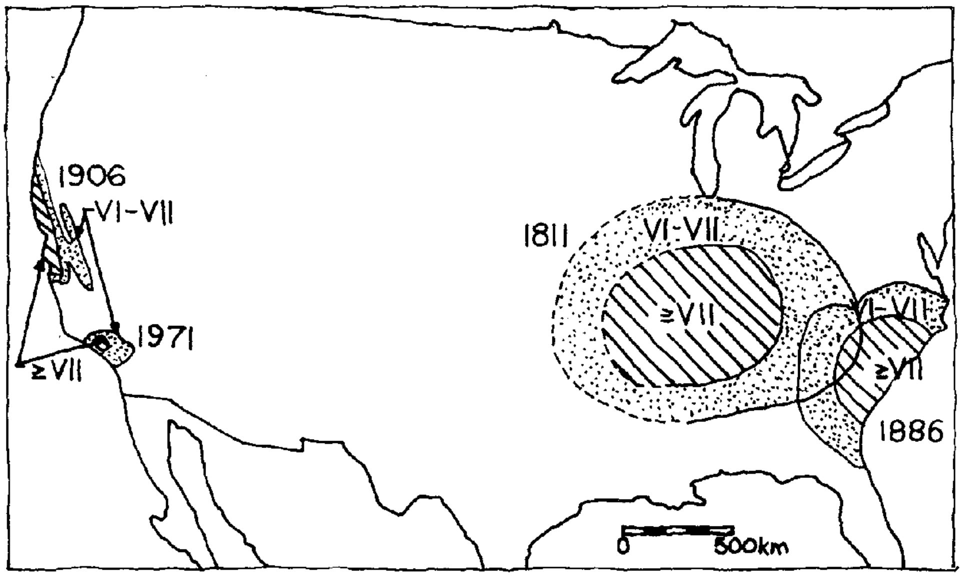
Another way in which we can estimate the ground motion is to specify its magnitude (which is related to its strength). There are several magnitude scales in use, each of which is a measure of the severity of ground shaking at a particular wave frequency. Unfortunately, many people who use earthquake magnitudes do not differentiate between the various scales but treat them all as the same, which they are not. For example, the body-wave magnitude measures the severity of ground shaking at a wave frequency of 1 Hz; whereas the surface-wave magnitude scale measures the severity of shaking at a frequency of 0.05 Hz. There are empirical curves and equations that give the maximum amplitude of the ground motion (usually ground acceleration) as a function of earthquake magnitude and epicentral distance. These curves differ for different geological regions because of the variability of the attenuative properties of the Earth's crust. There are other empirical or semi-empirical formulas that give amplitude of the ground motion of a given wave frequency and of a given wave type as a function of magnitude and epicentral distance. All of these can be used by the engineer to estimate ground motion at a particular site, if the maximum magnitude earthquake can be estimated and if the attenuation of the earthquake waves is known.

Duration of earthquake ground shaking can have a pronounced effect on the amount of damage produced. A structure, whether it be a building, dam, or bridge, might be capable of withstanding a few cycles of large amplitude ground motion but will fail if subjected to a larger number of cycles. Unfortunately, there are a number of definitions of duration of ground shaking in use. One that is fairly commonly employed is the bracketed duration, defined as the time between when the ground acceleration first and finally attains a value of 0.05 g, as measured from the seismogram or time history. There are a number of empirical formulas that relate duration to earthquake magnitude and epicentral distance.

Attenuation of wave energy depends upon wave type, wave frequency, and earth structure. There is a mathematical theory of attenuation that explains the observational data on the attenuation of high frequency surface waves. These waves, as noted earlier, are responsible for most of the damage done by ground shaking. Attenuation is the result of two processes, namely a spreading out of the wave evergy over a large surface and an absorptional or non-elastic loss. The former type, which controls the attenuation at short distances, is essentially independent of wave frequency. The latter is more important at large distances; its effect increases as the frequency increases. It is also dependent on the local geologic structure. Figure 2.11 shows some theoretical curves for surface waves. In this figure the distance is given in degrees of arc on the Earth's surface, with one degree equal to 111 kilometers. A typical curve for eastern and central North America for 1-Hz waves would be 0.07 deg^{-1}; whereas for western North America it would be 0.6 deg^{1}. At 1 degree (111 kilometers) this would mean the western amplitude would be about 2/3 of the eastern amplitude, if they started out the same. At 5 degrees (550 kilometers) it would only be one-tenth as large. For 10-Hz waves the eastern and central North American curves would correspond to 0.6 deg^{-6}. The latter is not shown in the figure, but it would fall-off rapidly. Even at distances as small as one degree (111 kilometer). We can illustrate the effects of differences in attenuation between eastern and western North America in another way. This is shown in Figure 1.12, which compares damage areas for two pairs of earthquakes. The first pair is the San Francisco earthquake of April 1906 and the Mississippi valley earthquake of December 1811, each of about the same magnitude. Notice how much larger the major and minor damage areas are for the eastern earthquake. The same is true for the 1971 San Fernando earthquake and the 1886 Charleston, South Carolina, earthquake. The dividing line between eastern and western-type attenuation is the Rocky Mountain front.



THEORETICAL ATTENUATION CURVES
OF EARTHQUAKE SURFACE WAVES
Figure 2.11

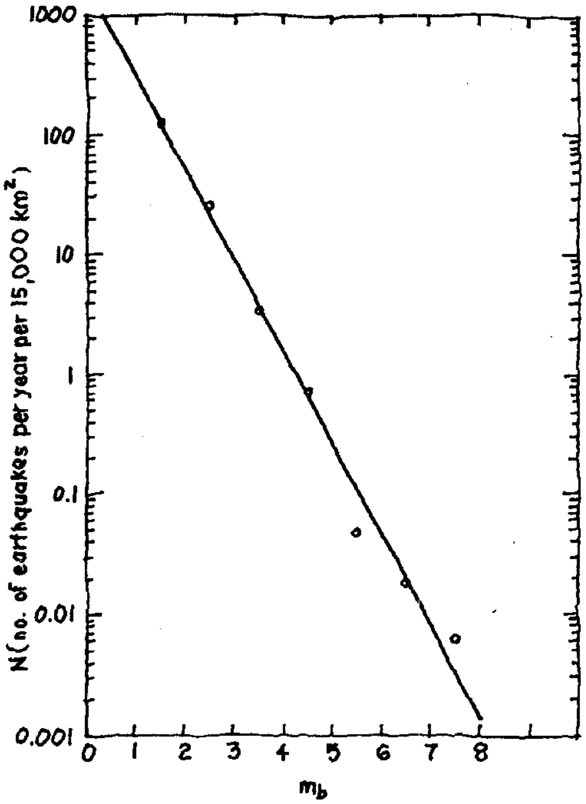


COMPARISON OF DAMAGE AREAS FOR EASTERN AND WESTERN U.S. EARTHQUAKES
Figure 2.12

In general earthquake damage is greater to structures on loose soils than on hard rock. Even though the acceleration may be less on the soils, the velocity and displacement of ground shaking usually are greater. This suggests that ground velocity may be a better indicator of damage than ground acceleration, although the latter ordinarily is used as the design parameter for structures.

EARTHQUAKE RISK

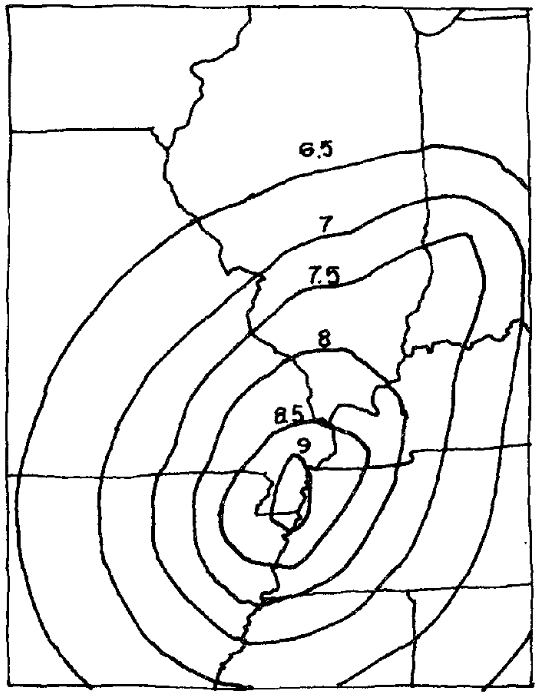
There are two broad ways of estimating earthquake risk. The one is deterministic; the other, probabilistic. In the first case we use the seismic history of the area and a knowledge of the recent geology to estimate the largest magnitude earthquake that will occur in the area, as well as its frequency of recurrence. We also can estimate the magnitude of an earthquake of a specified recurrence period. Figure 2.13 gives an example of such a curve for the New Madrid seismic region. From the curve we can expect on the average, over a long enough period of time, one earthquake of m = 7½ to occur every 300 years. This is a major earthquake , of the size of the 1811-1812 events. Somewhat smaller but nontheless damaging earthquakes of m = 6½ would occur on the average about every 100 years. There are about three barely felt earthquakes of m = 3 per year. Curves similar to this can be constructed for different earthquake source regions. If we know the recurrence curve and the attenuation of wave energy for a given region, we can predict the maximum ground shaking to be expected at a given site in a specified number of years, say the lifetime of the structure.



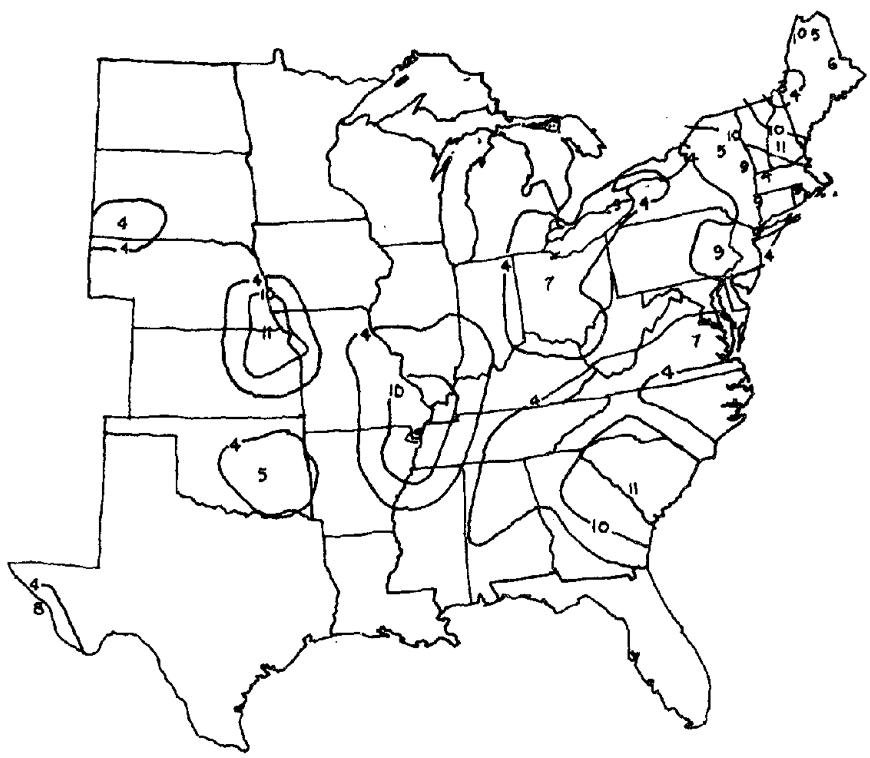
MAGNITUDE RECURRENCE RELATION FOR EARTHQUAKES IN CENTRAL MISSISSIPPI VALLEY
Figure 2.13

The probabilistic approach also requires a knowledge of the seismic source regions, the recurrence curve, and the maximum-magnitude earthquake for each region, as well as the attentuation. The end product is a map that gives the M.M. intensity (or the maximum ground acceleration or ground velocity) for a given area which has a certain probability of being equaled in a specified number of years. Figure 2.14 gives the intensities that have a ten percent probability of being exceeded in a

50-year interval for the central Mississippi Valley. Observe for Memphis it is M.M. intensity VIII, and for St. Louis it is M.M. intensity VII. Figure 2.15 shows the maximum horizontal acceleration that has a ten percent probability of being equaled in a 50-year period for the eastern United States. Note that the largest value of 0.19 g, which occurs in the New Madrid zone.



INTENSITY RISK MAP FOR CENTRAL MISSISSIPPI VALLEY
Figure 2.14



SEISMIC RISK MAP OF EASTERN UNITED STATES - HORIZONTAL ACCELERATION EXPRESSED AS PERCENT OF GRAVITY
Figure 2.15

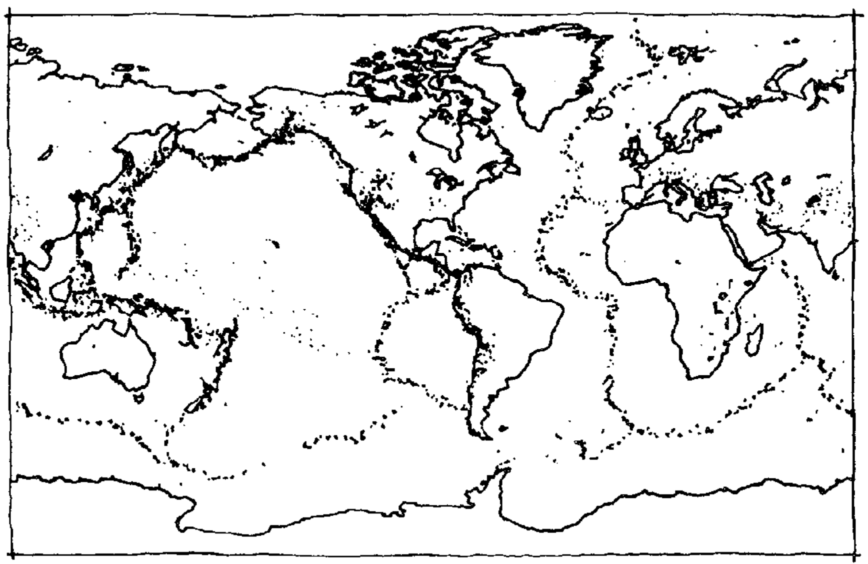
Risk maps such as these are expected to replace the old maps with their seismic zones 1 through 3, or 4, which are included in many of the existing building codes.

Chapter 3. GEOLOGIC HAZARDS OF EARTHQUAKES

Lloyd S. Cluff

CAUSES OF EARTHQUAKES

Figure 3.1 is a plot of earthquake epicenters throughout the world. Looking at the patterns from a global view, it becomes clear that earthquakes tend to be concentrated in certain zones. For example, note the pattern or zone that follows the west coast of South America along Central America up through the west coast of North America around Alaska through the Allutians onto Japan and the Philippines to north of Australia and then, finally, through New Zealand. So the question becomes not only what causes earthquakes, but why they occur in these patterns. The answers to both are related.



WORLD SEISMICITY
Figure 3.1

About 15 years ago, intensive geological and geophysical studies, particularly in the offshore areas and one specific area off the coast of Northern California, revealed what looked like stripes in the bottom of the ocean floor. What we found was that new crustal material was being generated in what we now call spreading centers. This material comes up from the molten part of the earth and moves outward in two opposite directions. When it solidifies, it records or "freezes" the direction of the magnetic field in the earth. Studying these spreading centers, we have found that once every few hundred thousand years there is a reversal of the earth's magnetic field from north to south. Thus it is possible to look at these stripes and find the orientation of the earth's magnetic field and tie that to time. This "clock" tells us that the ocean's crust is being pushed in two directions, which in turn causes the rigid plates in the crust of the earth to move with respect to one another. This interaction between the plates, along fault zones, or along the plate boundaries produces earthquakes.

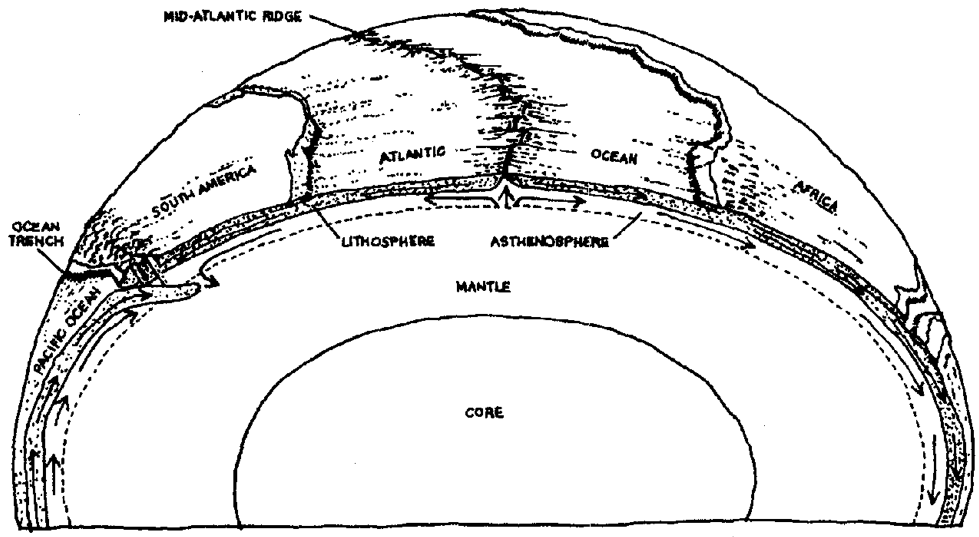


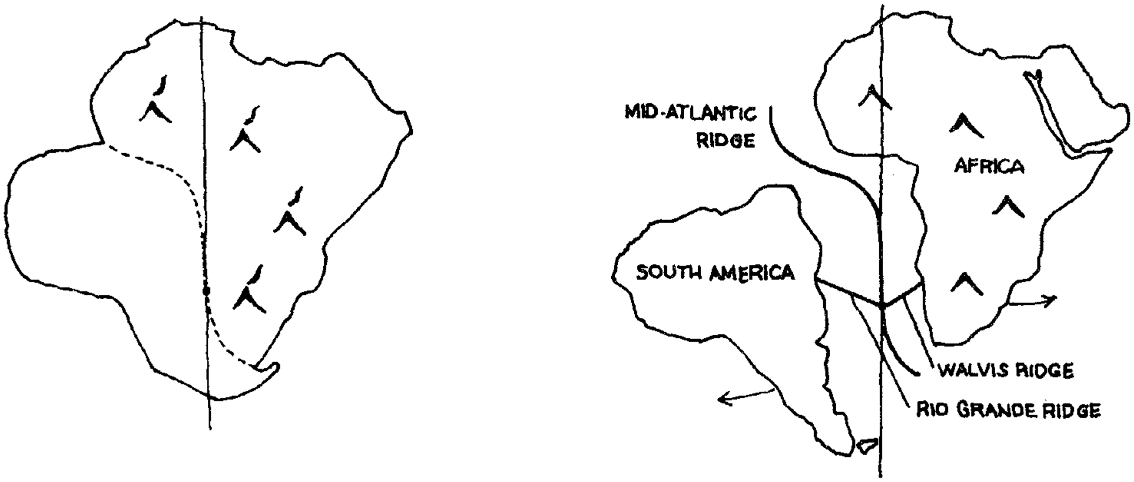
PLATE TECTONICS
Figure 3.2

Along the west coast of South America, for example, an oceanic plate is colliding with the continental plate (see Figure 3.2). Where the oceanic plate is being subducted, that is, where it is going underneath the continental plate, the result is some very dramatic topographic effects. The highest mountain ranges in the world are produced in those environments. Plate collision is the explanation for the high Andes and the high mountains in Alaska.

Spreading centers can be studied in the one place where they are exposed on the surface—Iceland. Features that are otherwise on the bottom of the ocean and are only able to be studied through indirect means are present on the surface of this island, which straddles the mid-Atlantic ridge. Iceland is actually being torn apart. The consequences of the conflicting movement of the plates are big fault zones that are very spectacular. Speaking from a geological point of view, Iceland is made up of very new crustal material.

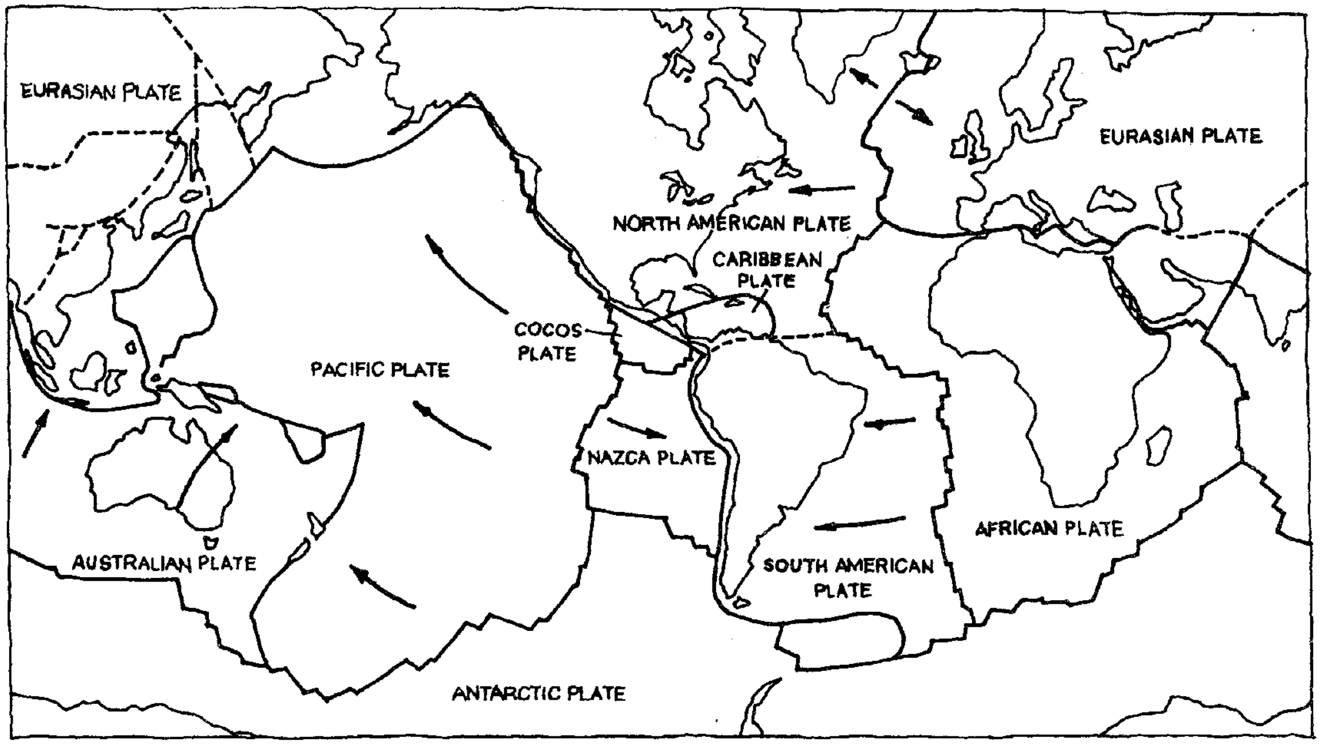
Where the crust is thin, molten material comes up along the fault zones and leaks to the surface. In Iceland this means some spectacular volcanic effects, particularly where this leakage occurs on the nearby ocean floors and reacts with the water. All around Iceland new oceanic material is being formed, which provides the driving mechanism to move the plates along the mid-Atlantic ridge.

Let us step back in time several millions of years. South America and Africa once fit exactly. As the spreading center developed, these two continents were split. Figure 3.3 represents the situation as it was then and as it exists today along the mid-Atlantic ridge spreading center.



MID ATLANTIC RIDGE SPREADING CENTER
Figure 3.3

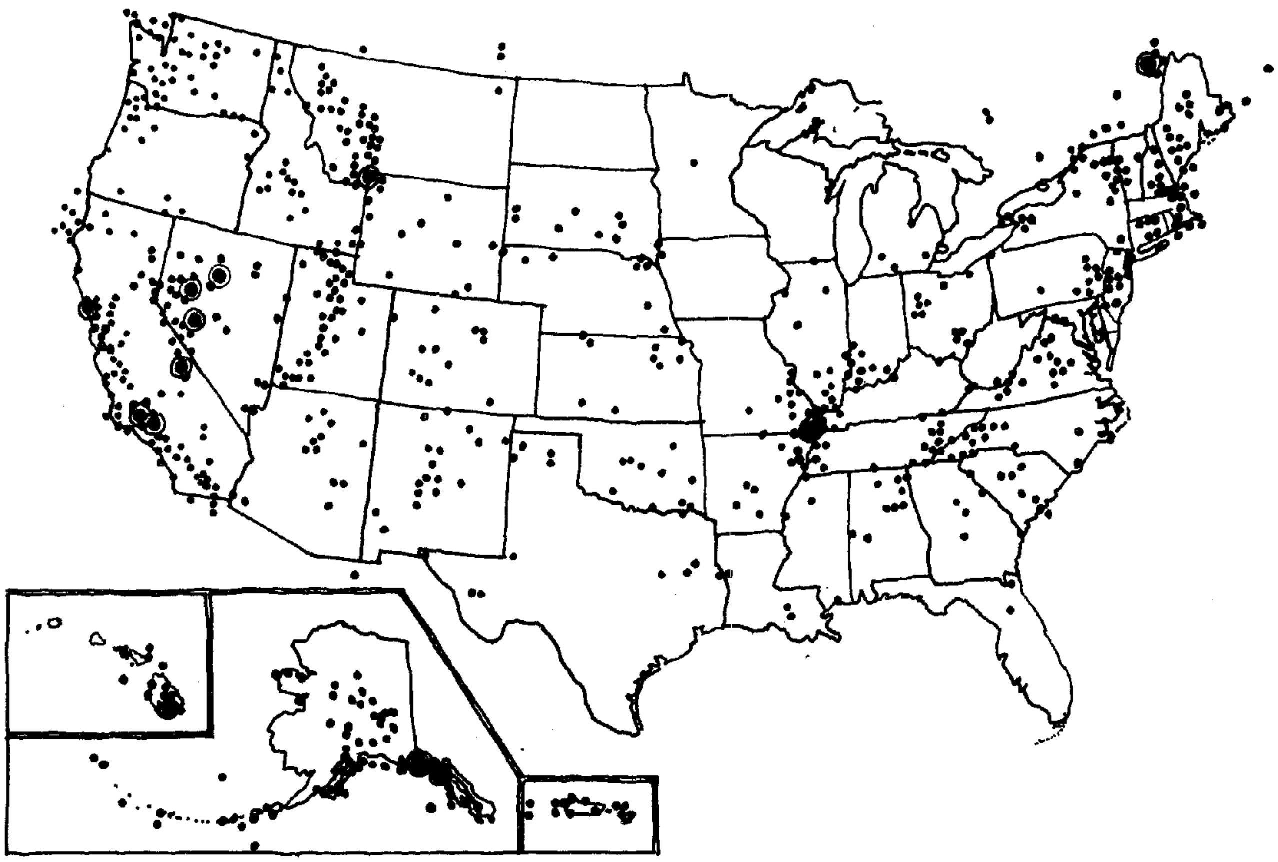
The vectors on Figure 3.4 show the relative motions of these moving plates. Most of the arrows are on the oceanic plates. Again, the world-wide distribution of earthquakes very clearly corresponds to and follows the large fault systems that are at the plate boundaries. The particular boundary that follows the California coast is called the Circum-Pacific Belt. The Pacific plate is the largest single plate. It interreacts with North American plate to form the San Andreas fault, the source of most earthquakes in California.



RELATIVE MOTION OF PLATES
Figure 3.4

Another interesting plate collision occurs in Asia. At one time, India was an independent continental mass; through time it has been drifting northward. The result? One continental mass colliding with another. That collision takes place at the boundry of India, Nepal, and Pakistan. The topographic expression of that is the Himalayan Mountains, the highest range in the world. Here the Indian plate is actually underthrusting the Asian plate.

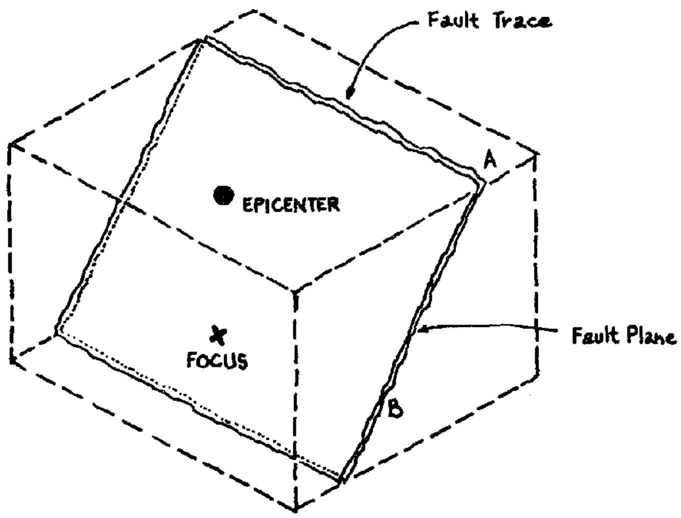
Up to this point, we have been considering most plate boundry fault zones and earthquakes. But to look at a seismic map of the continent of North America, particularly the United States (see Figure 3.5), is to discover that not only are there earthquakes along the plate boundry, but there is also a scattering of earthquakes in the mid-western part of the United States and some earthquakes in the east as well. What triggers these so-called intra-plate earthquakes in the continental masses? They are caused by the same geologic process, a process described by the Elastic Rebound Theory. That is, slip along a fault releases the energy that is stored in the crust of the earth.



EARTHQUAKES IN THE UNITED STATES
Figure 3.5

What happens, according to the Elastic Rebound Theory, is that for a period of time, energy is stored in the crust of the earth. The result is deformation across the rocks near the fault zone without shear. When the elastic limit of the rock is reached in the fault zone, then that stored energy in the crustal rock is released. The elastic snap of those rocks back to their original position is what releases the energy that forms the seismic waves of an earthquake. This is the way all earthquakes are generated. Let us follow the process further.

Figure 3.6 shows what happens along a fault zone. We have a distance from A down to B, maybe 10 or 15 kilometers in depth, and we get an inclined fault surface there. The initial energy that is released starts at a point X called the focus. Then the fault starts breaking randomly in all directions along the plane. The epicenter that is plotted on the map locating the earthquake is the point directly above the focus. However, in long fault zones this fault may rip for four or five hundred kilometers. So although the epicenter is a point on a map, the energy is released along the ruptured length of the fault.



FAULT ZONE
Figure 3.6

That brings us back to the troublesome phenomenon of intra-plate earthquakes. One can see from plotting historical epicenters that there have been earthquakes in the eastern part of the United States. The problem is that not enough study has been done to identify the sources of these earthquakes. In other words, we know that at major plate boundaries, particularly the collision boundaries, those fault zones have the potential of the largest earthquakes. Nevertheless, although the potential for earthquake activity is not so great in the more stable continental areas (such as the eastern part of the United States), there have been some important exceptions. Back in 1811 and 12, some of the world's largest earthquakes occurred in Missouri; actually, a whole series of them. They were about magnitude 7 to 8, or almost as large as the 1906 California earthquake.

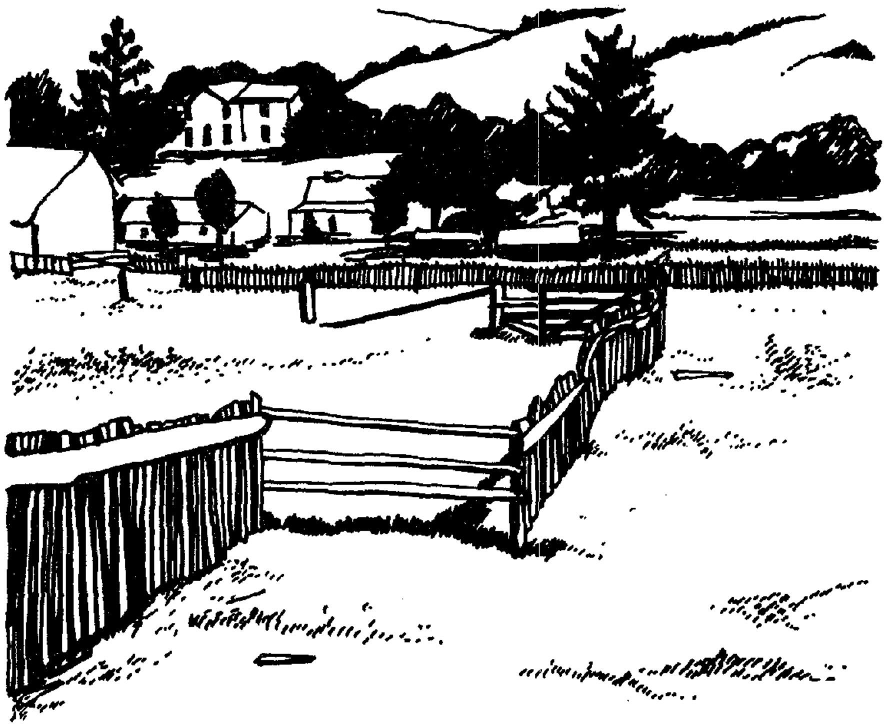
It is easy to find the faults along the plate boundaries; it is much more difficult to identify the faults that exist in the intra-plate regions. But clearly it is important that we do so. For if we want to predict where an earthquake is going to occur, we will have to know where the faults are that have the potential for causing the phenomenon.

EARTHQUAKE EFFECTS

It is convenient to classify earthquake effects into four distinct and separate categories of damage:

- damage from surface fault displacement;
- damage from shaking;
- damage from ground failure (such as landslides); and
- damage from tsunamis or seismic sea waves.

Beginning with damage from fault displacement, one would want to be careful about locating any facility astride a fault. Along the San Andreas fault in 1906, the highway at Point Reyes Station was displaced about 21 feet. The road was completely severed because of the lateral motion. Figure 3.7 shows a Marin County farmer's fence which was displaced. The fence is still standing and, although the buildings very close to the fault clearly must have experienced extreme shaking, they, too, are nevertheless still standing. This suggests that proximity to the fault is not so important as the design of the structures and the land the buildings are built on.



FENCE OFFSET - 1906 SAN FRANCISCO EARTHQUAKE
Figure 3.7

This is a very important consideration to deal with when we discuss shaking. Clearly, a structure can be close to the fault yet survive the fault displacement—as long as it is not astride the fault.

Salt Lake City is located near one of the world's most spectacular normal faults, the Wasatch fault. It is produced by part of the earth's crust being shoved up with respect to which part is going down. There are indications that the fault slips periodically. If we look at a cut that was made for an apartment building a few years ago in Salt Lake City (see Figure 3.8), we do not need to be geologists to see there is something funny happening. The foundation straddles the location of the Wasatch fault. There could be as much as 15 to 20 feet of slip along that fault in the vertical direction during an earthquake. One does not need much of an imagination to speculate what would happen to that building should that fault slip, not 10 or 20 feet, but even six inches. It is a masonry structure that is not designed to resist the shaking let alone the displacement of the ground.

APARTMENT BUILDING - SALT LAKE CITY
Figure 3.8

This raises a question: from the standpoint of surface fauling, who has the authority to make the decision on whether to build on the fault? Do the people that live in that apartment house? Should they have some say in what their risk is? Are the people who made the decision to locate that building on top of that fault legally liable? They knew that the fault was there. When the excavation was opened, geology classes from the University of Utah used to take their students to show them the fault before the building was built. Of course the degree of risks depends upon what can happen, how often it will happen, and what the consequences will be.

To make the case even worse, the same fault cuts right through the most densely populated part of Salt Lake City. There is not just one apartment building on top of that fault; there are three more. So from the standpoint of acceptable risk, we have a very high risk situation along this fault trace.

To that we have to add the intense effects that will occur due to the shaking. Remember, not only are we going to have slip along this fault right through Salt Lake City, the energy is going to be released directly beneath the city as well. That fault is dipping toward the city. It is inclined about 70 degrees and the focus of the earthquake that will first slip will be right beneath the city; it will be what we call a direct hit. Some of the taller buildings in Salt Lake City have taken earthquakes into account in recent years, but most are built of non-reinforced masonry. However, it is clear that if there is going to be construction near an active fault, the structures built will have to be designed to resist or accommodate displacement.

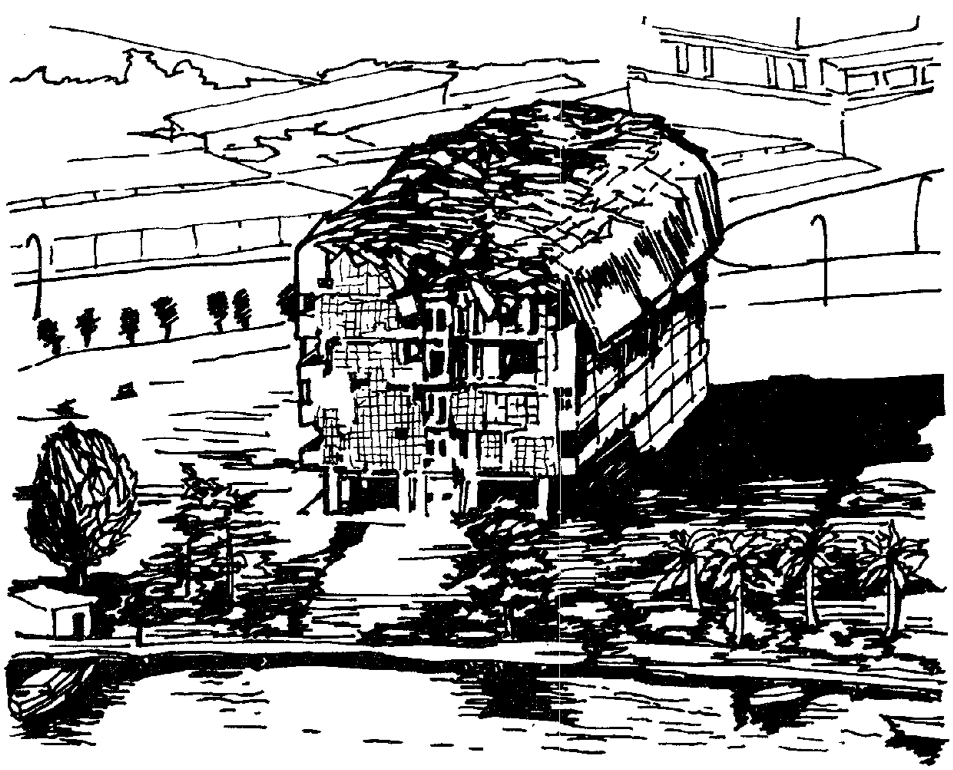
A discussion of displacement leads into the second category of earthquake damage, the effects of shaking.

The shaking effects are influenced by the size of the earthquake. And size is generally expressed in Richter magnitude. There are a number of other magnitude scales, but they are all based on the same idea—the amount of energy released. Two other factors are duration and the response characteristics of that shaking. Some earthquakes may last only a few seconds. The Alaska earthquake of 1964 was as long as two or three minutes in some places. Duration is also important because some buildings that can stand a few cycles of shaking start to deteriorate after several hundred cycles.

The response of soil conditions to the ground motion is dramatically illustrated by reconsidering an example discussed earlier. Fifteen miles away from the farmhouse that withstood the 1906 slippage of the San Andreas fault is the city of Santa Rosa. Most people do not realize that Santa Rosa was about 80 to 90 percent destroyed in that earthquake. What we find is that the character of the ground under that city controlled the intensity and the characteristics of the strong ground motion that the buildings received. And we find a great deal of difference from one place to another. It is important to know what the difference is in designing for ground motion.

In 1967, a magnitude 6.5 earthquake occurred about 30 miles out to sea from Caracas, Venezuela. Caracas, has more high-rise buildings than those Los Angeles and San Francisco combined. All these buildings were built within the previous ten years according to zone 2 of the California earthquake code. Since most of them were reinforced concrete, lift slab strucures, it is instructive to look at the response of these buildings in this earthquake. Only four buildings totally collapsed killing 264 people.

In the earthquake, the top four stories of an apartment building totally collapsed. Looking into the records of this building, we found that one contractor built the first four floors; another built from there. Same design, same materials, but different contractors. The importance of the quality of construction is very clear from this example.

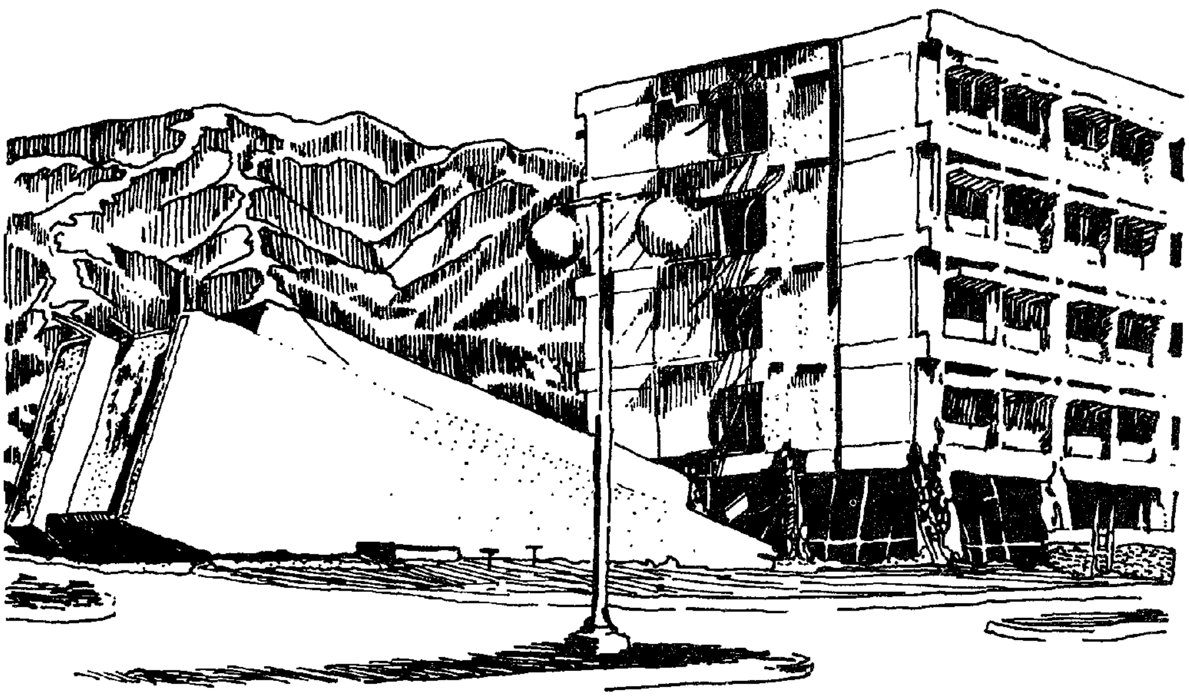


PANCAKE OF UPPER FLOORS
MANSION CHARAIMA - 1967 CARACAS EARTHQUAKE
Figure 3.9

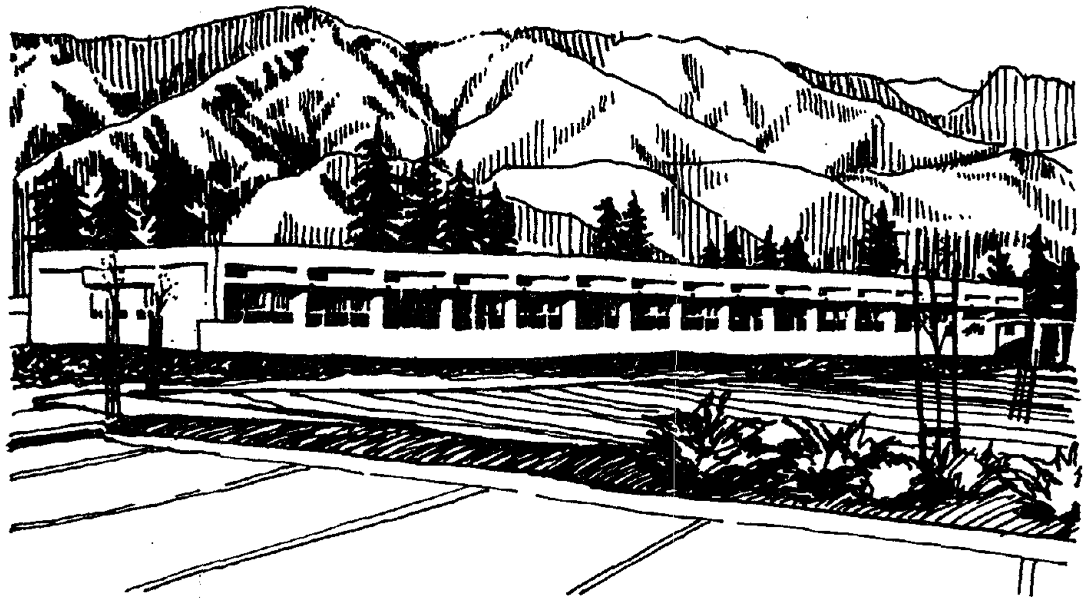
In other parts of Caracas, smaller structures were completely destroyed, yet the high-rise buildings were not damaged. So a pattern started to emerge.

What we found was that when the dominant period of ground motion matched the natural period of vibration of the building (in this case about 1 second) the building had a serious problem. There was a range of about 0.8 to 1.2 seconds, which was the strongest component of the ground motion. All the buildings that were damaged had approximately the same natural period of vibration. So out of this earthquake we learned the importance of understanding the local as well as regional geologic conditions beneath the site, which to a large extent controlled the character of the motion. The point is to make sure that buildings that will be close to the natural period of the possible earthquakes are strengthened to resist this motion. This might be achieved by making the buildings higher or lower depending on the natural period of the site.

In 1971, the San Fernando earthquake occurred. Like Caracas, it, too, was a relatively small earthquake; magnitude 6.4. It was not a direct hit. It did strike a part of the San Fernando Valley, but not directly. Nevertheless, a modern structure, the recently dedicated Olive View Hospital, which had been designed to the most stringent earthquake codes in California, was seriously damaged. The stair towers tipped over (see Figure 3.10), and the psychiatric building ended up one story shorter than it was when it was built (see Figure 3.11).



FAILURE OF STAIR TOWERS
OLIVE VIEW HOSPITAL - 1971 SAN FERNANDO EARTHQUAKE
Figure 3.10



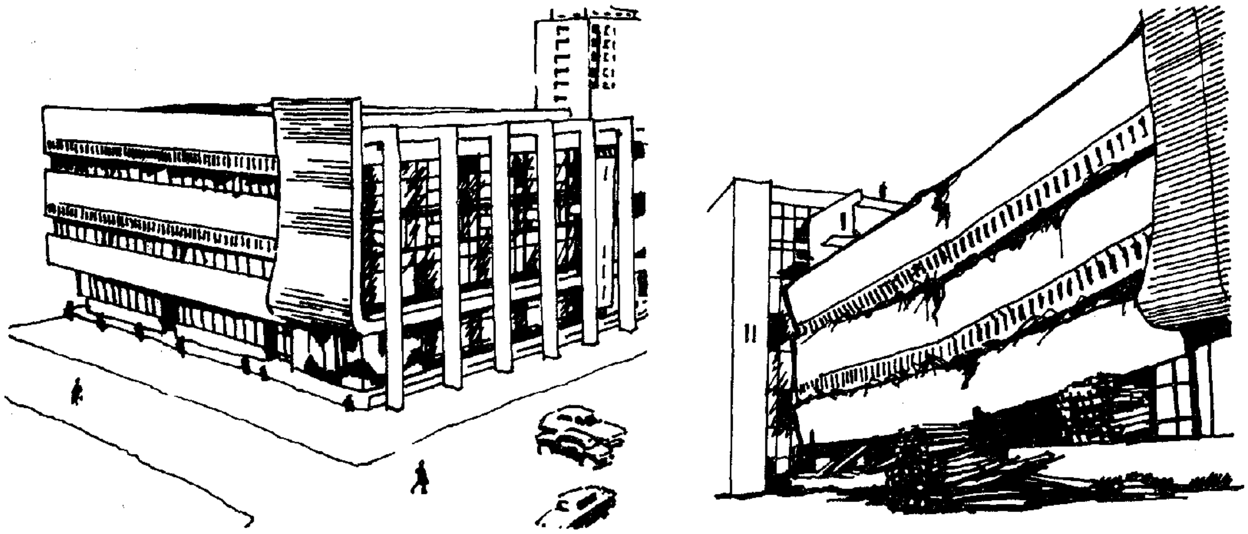
COLLAPSE OF LOWER FLOOR
PSYCHIATRIC CENTER - 1971 SAN FERNANDO EARTHQUAKE
Figure 3.11

What happened? The stair towers had not been tied into the main structure. As for the psychiatric center, it was fortunate that the earthquake occurred at 6:30 in the morning. On a normal working day 800 people occupied that first floor. In addition, someone forgot to take into consideration that the canopy protecting all the ambulances was not really designed to resist earthquakes. It fell and trapped all the ambulances which were needed during the disaster.

Another important factor that we learned recently came from the Bucharest, Romania, earthquake that occurred in March 1977. We knew that it was an important and different tectonic environment. The earthquake occurred some 150 kilometers from Bucharest, but the main damage was in the capitol city. There was damage even 250 kilometers away on the flood plains of the Danube.

It is my hypothesis that the earthquake energy was released at a much deeper focus than what is common to earthquakes in California, which are released in the upper 10 kilometers of the earth's crust. The earthquake in Romania was released at a depth of about 110 kilometers. Because of that depth and the focusing effect of the geologic structure, the energy was focused in the path of Bucharest.

Bucharest was not wiped out, as the press might have led the rest of the world to believe. There was selective destruction based on that focusing effect. The buildings that were weakest corresponded to the dominant period of ground motion. One modern building that had just been completed (their new national computer center Figure 3.12) suffered a total pancake collapse of all four stories. What happened was that the long period waves, probably on the order of 1.0 to 2.0 seconds, happened to correspond to the period of the new computer center.



PANCAKE COLLAPSE
COMPUTER CENTER - 1977 ROMANIA EARTHQUAKE
Figure 3.12

The next, or third, category of earthquake effects is ground failure. In 1968, Anchorage, Alaska, was subjected to landslides and liquefaction as a result of the Good Friday earthquake. There were clear indications that the Turnagain Heights area was susceptible to both landslides and liquefaction (see Figure 3.13), although this was not recognized prior to the earthquake. In such a situation, responsible design is a matter of knowing the condition and either correcting it or avoiding the site. For no matter how well the architect or engineer designs a building to resist the strong shaking that will be associated with an earthquake, if he does not know that he is building on a slip plane or a landslide, he subjects his design to the same disruption that fault displacement would cause.

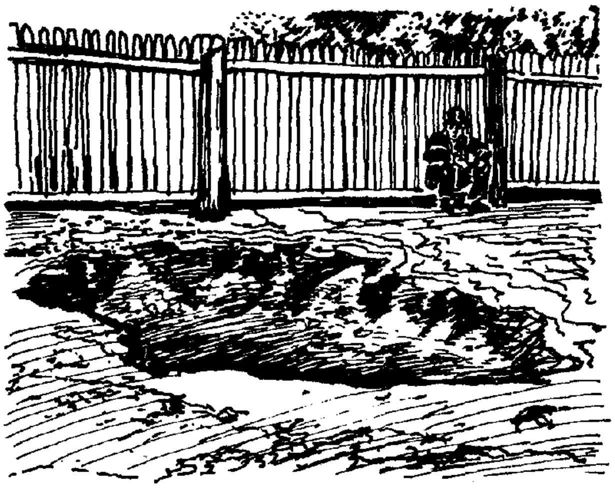
TURNAGAIN HEIGHTS - 1964 ALASKA EARTHQUAKE
Figure 3.13

The Niigata, Japan earthquake of 1964 offers a spectacular example of liquefaction (see Figure 3.14). Apartment buildings were built on material that liquefied. Materials that will liquefy are grandular materials like sand and silts that derive their strength from grain-to-grain contact. Between the sand grains there is water. When the earthquake occurs, the materials are shaken and the strength is transferred to the water. Water has no shear strength, so the soil turns to a liquid. Those buildings that did not tilt had deep footings that went below the liquefiable sand. It is that simple. If you know that the material exists, you can remove it, compact it, drain it, or design for it.

LIQUIFACTION - 1964 NIIGATA, JAPAN EARTHQUAKE
Figure 3.14

In other words, where the potential for liquefaction exists, the designer has several options: avoid the site, correct the site conditions, or cause the building design to overcome it by putting the footings through the sensitive material. The buildings in the figure above were designed structurally to resist the shaking. Yet they were like unbalanced battleships capsizing in water. They tipped over because the weight of those multistory buildings was suddenly bearing upon a liquid and therefore unsupportive ground. It is a simple lesson of knowing the ground conditions, not only for shaking but for ground failure.

One of the more dramatic effects of liquefaction is called sand boils. What happens is that when the material liquifies during an earthquake, there is often a volume change at depth. The result is small volcano type craters from which sand boils out onto the surface. Figure 3.15 shows a spectacularly large sand-boil from the 1886 earthquake in Charleston, South Carolina. These same features occurred over a widespread area in Missouri and Tennessee in the New Madrid earthquakes in 1811 and 1812. The important thing is how often to expect these to occur in areas where the soil is subject to liquefaction. There is no reliable answer yet to that question.



SAND BOIL
1886 CHARLESTON, S. C. EARTHQUAKE
Figure 3.15

The fourth and final category of seismic phenomena to be considered is the tsunami, which is a Japanese word for seimic seawave. Tsunami is a problem that any facility on the coastline along the Pacific Ocean needs to take into consideration. This means accepting that they are going to occur and, therefore, building facilities high enough to resist the waves. The highest waves, by the way, are on the order of 50 feet. In most cases, however, they are about 30 feet high. Keep in mind that tsunami waves can come from any source around the Pacific Ocean. In other words, tsunamis are generated at the source area and can then travel across the open ocean, causing destruction where they come in contact with land. There has been, for instance, a lot of damage in Hawaii from Chilean earthquakes as well as earthquakes from New Zealand, Japan, and Alaska. So Hawaii is sitting in an especially vulnerable spot.

Fortunately, there is a warning system set up so that when the waves are generated, an alert is sent out. In California in 1964, all the coastal regions were evacuated in anticipation of a wave. It was tracked across the ocean and accurate predictions were made as to when it would strike different parts around the Pacific.

The only time deaths occur is when people ignore the warning system. This is what happened in Crescent City, California, in 1964. An alert for an earthquake tsunami was sent out and the people evacuated from Crescent City, an area that was thought to be most vulnerable. Two hours after the first wave came in, which was about eight feet high, people had gone back into their houses. At that point a 20-foot wave hit. So it is a matter of being able to understand how the effects might be amplified along the coast. Usually just studying the coastline can generate a pretty good idea of potential wave height. The tidal fluctuation has to be taken into account also. It is not a complex problem, but for very critical facilities it needs serious consideration beyond the present warning systems.

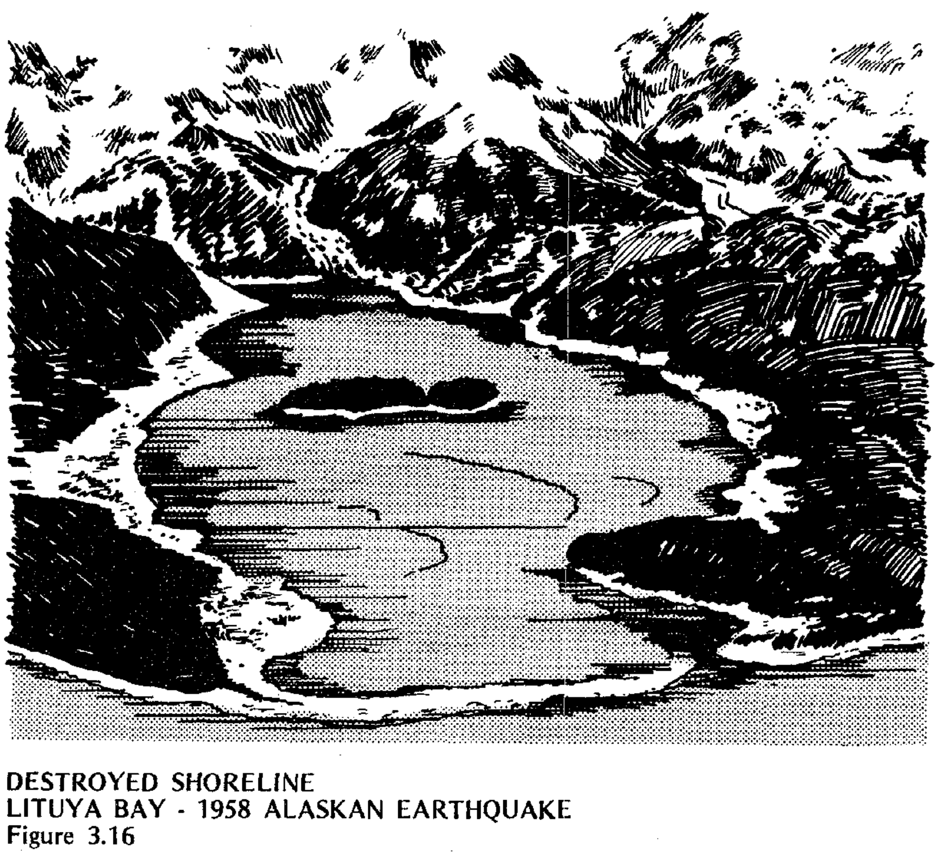
Up to this point, I have been considering effects of earthquakes independently. But this is not to imply that seismic effects occur independently of each other. In 1958, a magnitude 8.0 earthquake was generated along the Fair weather fault in Alaska. All the effects I have been discussing occurred at one location—faulting, ground motion, ground failure, and tsunami. Because of the intense shearing in the earth's crust, the zone of rock at the site was weak and shattered. This was aggravated by the glacial action there. The result was an exceptionally unstable condition.

The strong shaking unleashed by this earthquake caused the materials in this area, shown in Figure 3.16, to be dislodged. Forty million cubic yards of materials came down in a debris avalanche that displaced the water of Lituya Bay. The displacement caused by the large volume of material dumped into the water generated a huge wave that stripped the trees from an adjacent rock ridge. The vegetation is growing back, but even today the trimline can still be seen. And what it shows us is that the height of that wave was 1786 feet, the biggest splash in history.

It is important to remember that the National Park Service had planned to develop this picturesque bay as a tourist attraction. Plans called for building hotel facilities around the edges of the bay. Ships would come in and dock there. They were going to build an air strip . . . and so forth. Fortunately, the earthquake hit before the plan was approved.

But suppose the earthquake had not occurred? Could such a potential catastrophe have been predicted? A study of the tree ring growth in this area reveals that this same occurrence had taken place five times in the last 200 years.

My experience in the Himalayas, in the Andes, the Alps, the Rocky Mountains, and Alaska has shown me that where there is the potential for earthquakes and a high differential topography, there tends to be a concentration of seismic effects. So you have to take into consideration something that might happen miles away that could affect and perhaps even totally devastate a site.



DESTROYED SHORELINE
LITUYA BAY - 1958 ALASKAN EARTHQUAKE
Figure 3.16

EARTHQUAKE RISKS

The question then becomes where or when are earthquakes going to strike and how often? In the future we may come to the point where it may be possible to predict when they will occur. That point cannot come too soon considering the risks we are already aware of.

Let us go back to the case of Salt Lake City and the Wasatch fault. It is not only Salt Lake City that is threatened; this fault traverses every major urban development of Utah. Eighty percent of Utah's population lives within ten miles of the Wasatch fault. Ogden, Salt Lake City, Provo—they all could be destroyed in a major earthquake.

In the San Francisco Bay region of California, San Francisco, Berkeley, and Stanford all lie close to major active faults. The San Andreas fault does not cut through San Francisco; it runs offshore to the west. However, the Hayward fault cuts right through the East Bay area. There is no question that a repeat of 1906 will cause a lot of damage. The more severe damage will come from rupture of the Hayward fault because this fault possesses the threat of a direct hit.

What risks, then, are acceptable? It is a problem architects have to face because they are in positions to make these decisions. Should we allow non-conforming facilities to continue to exist? Should these facilities be replaced or should the property be vacated. Should we build new structures in threatened areas or leave the land vacant? These are very sticky questions that we are faced with in California.

As an example, I would like to discuss a problem that is creating some heated political debate right now—the proposed Auburn Dam in California's San Joaquin Valley. The completed dam will be a double curvature, thin arch structure that will be 146 feet thick at the base and 40 feet thick at the top. It will be 4,150 feet long—the longest thin arch structure in the world—and 700 feet high. The completed structure is designed to hold a reservoir containing 2,300,000 acre-feet of water having a maximum depth of 665 feet.

The problem is that the site was picked some ten years ago in an area which, according to U.S. Bureau of Reclamation studies, was thought to be free from earthquakes. However, an earthquake occurred in the vicinity in Oroville on August 1, 1975. That occurrence caused a reinvestigation of the potential for earthquakes in an area where very few had occurred before.

As we started this investigation, I asked the Bureau what they wanted to consider as criteria for an active fault. They decided that an "active" fault was any fault that had experienced displacement in the last 100,000 years. To be considered "inactive" it had to bé demonstrated that a fault had been inactive for that time. Those faults for which no definitive evidence was available was labeled "indeterminate."

We found a trend of faults along the Sierran foothills that traverses the Auburn site and is connected to the focus of the Oroville earthquake. Surface faulting occurred at that location during that earthquake. So the obvious question was, if the fault that caused the moderate, 5.7. magnitude, Oroville earthquake is active, then was it possible that the faults in the Auburn area could also be active and have the potential for causing even bigger earthquakes? If so, what would be the effects of the dam? Fortunately, the dam had not be built yet. Everything came to a halt while we spent 18 months working on this problem.

We found the answer to one of our questions. Some faults that are in the bedrock showed geologic eivdence of being active by the Bureau's criteria. Could these faults have been identified prior to the Oroville earthquake? Earlier, we had taken aerial photographs for other studies that we were doing in the region at the time. From these photographs, we had identified topographic features that indicated fault activity. So the answer was a clear yes. That fault could have been, in fact had been identified prior to the earthquake.

In short, our conclusion on the Auburn site was that the faults are active. They have the potential not only of generating significant earthquakes right at the site, but also of slip right through the structure. If the site does suffer much slip, what will happen to the dam? Everyone has a different answer.

This raises a philosophical problem about fault activity criteria and acceptable risk. The Nuclear Regulatory Commission, the U.S. Bureau of Reclamation, the Corps of Engineers, and the State of California all say that any fault that shows activity within a given period of years should be considered active. Although the length of time differs somewhat, depending upon the point of view, it is fundamentally a deterministic method of assessing fault activity.

Let me explain the fallacy of using a deterministic method of assessment. If we take a San Andreas type fault, we find that the slip rate is about five centimeters a year. If we look at how much cumulative slip will occur, the San Andreas fault will have slipped, by the most conservative criteria, some 25,000 meters in 500,000 years. Now, if we take the fault that slipped in the Sierran foothills, we find the slip rate is 0.006 centimeters a year. In 500,000 years it will have slipped three meters. Clearly there is a difference in degree of activity between three meters and 25,000. Yet, under the deterministic criteria that we presently use, all these faults are equal. There is no consideration given to the degree of activity. Once an active fault is identified, it is active, period.

I think it is about time that we consider the degree of activity, as well as the rate of slip. I am proposing that we change the criteria to take into account

- the degree of fault activity, which includes the rate of slip;
- the amount of slip, both in displacement and length of rupture in terms of cumulative slip and slip per event; and
- the frequency of occurrence.

If we move from the deterministic to a probablistic approach, the degree of fault activity will be taken into consideration. Comparing the San Andreas fault to the situation at the Auburn Dam site reveals a tremendous difference in degree of activity, yet at present they are equated. As things now stand, it is necessary either to spend an enormous amount of money to design the structure to resist the faulting, or move that facility.

Our study was to look at the scientific aspects; our contract was not to make the value judgment that still has to be made on the Auburn Dam. What I see happening are a lot more of these important value judgment/acceptable risk problems coming up. If we adopt what I think is the logical approach—the degree of activity concept—it will shift that decision to the designers or society, or the politicians. There is nothing that can be guaranteed to be absolutely safe, and society has to know that nothing can be absolutely safe. We have to set up a reasonable system of assessing what risks are acceptable and then proceed. Right now, we are going through a very serious and agonizing period of trying to decide which way to go. In my view the only way to go is to determine the degree of activity, and then, once we have assessed what the consequences are, make a decision and move on, whether it be to abandon the facility or assume the risk as acceptable because of the tradeoffs and the benefits derived from various projects, whether they be dams, nuclear reactors, high-rise buildings, or whatever.

One final comment by way of a summary. I have identified the basic four categories of earthquake hazards—faulting, shaking, ground failure and tsunami effects. Each of these can be handled provided the designer knows about it; and he should know about it in the early planning stages rather than after he starts to build the structure. So the time for geologic study, the time for seismological evaluations is in the early feasibility study or site selection state. Unfortunately, geologists often called in to assess a problem after major commitments of money have been made and the structure is either in the design phase or under construction.

We can take all the factors into account and either relocate sites or shift buildings on a given site to avoid the particular hazard. It is theoretically possible if we all work together—the geologist, the seismologist, the structural engineer, the architect, the planner. If we do our job right and we are all conscientious, rather than have a disaster in an earthquake, we will have nothing more than an exciting experience.

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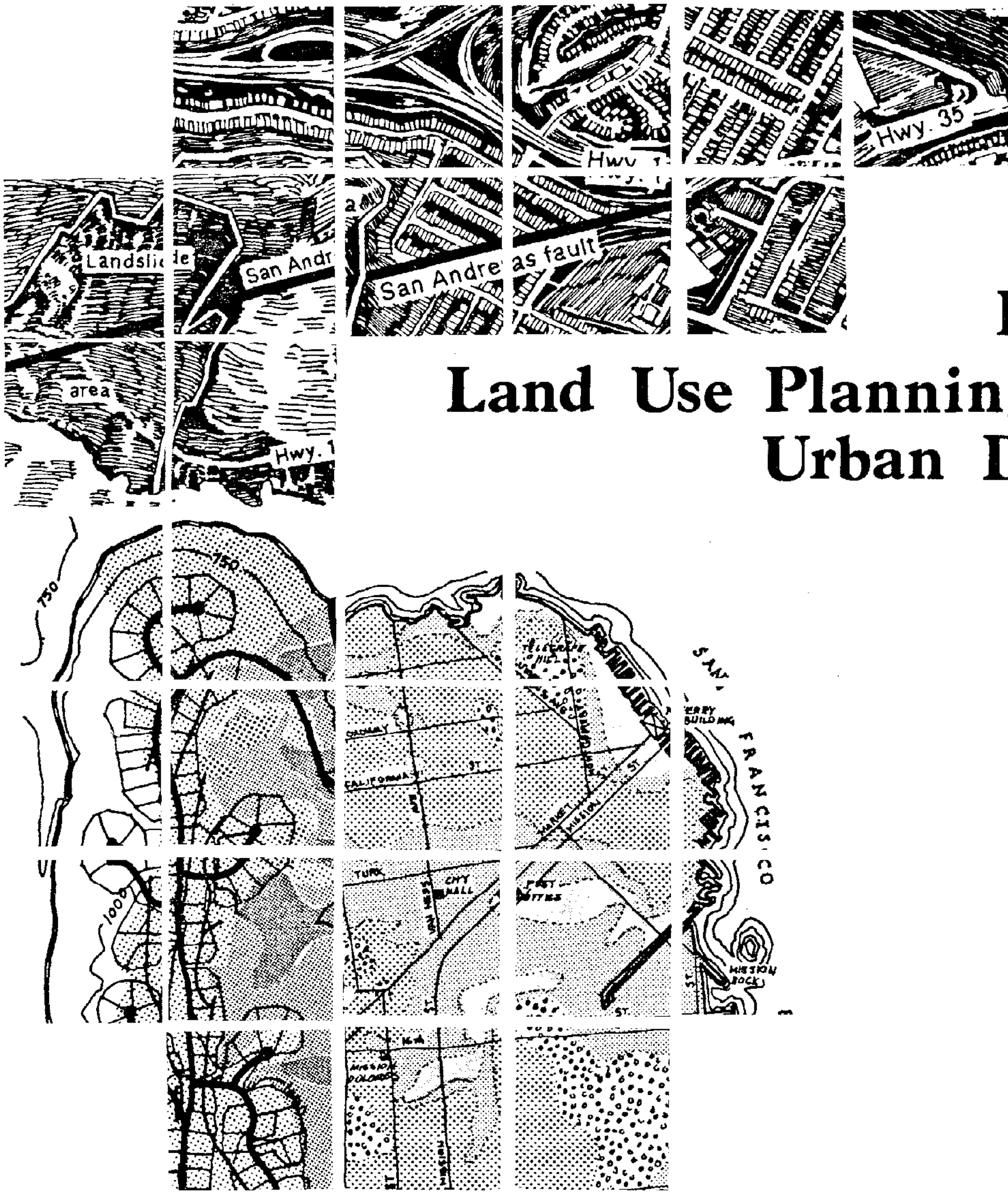

Part 2
Land Use Planning and Urban Design

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Chapter 4. LAND USE PLANNING FOR SEISMIC SAFETY

George G. Mader, A.I.P.

IMPORTANCE OF SEISMIC SAFETY IN LAND USE PLANNING

Land use planning can have a major impact in increasing seismic safety. Basically, planners are involved in the distribution of land uses in urban areas. This distribution is affected by many factors—social, economic, political, and physical. Geology and seismic safety fall into the physical factor category. In areas where there are no significant seismic problems, we certainly do not deal with the problems. However, where we have major seismic concerns, seismicity can be an overriding factor in determining appropriate land uses.

While land use planning is an important method for reducing risk from seismic hazards, planners have to realize that there are many other professions involved in trying to improve seismic safety. Structural engineering is certainly one. We know that structures can be designed to withstand a great deal of shaking and still come through an earthquake in good shape. There are cases, however, where the cost of design to withstand shaking may be prohibitive; therefore, an alternate land use might be appropriate. Also, in the case of land failure—that is, where it is not just a problem of building shaking, but where land actually fails, such as in a landslide—the structural engineering solution may not be appropriate and one may be compelled to look at alternate uses of land.

The planner is in between the earth scientists on one hand and the engineers and designers on the other. The engineering geologist and the soils engineer provide information on the method of land movement and possible land failure. The building designer will tell the planner how designs can be arrived at which will help mitigate against these hazards. The planner attempts to prescribe appropriate land uses within this context.

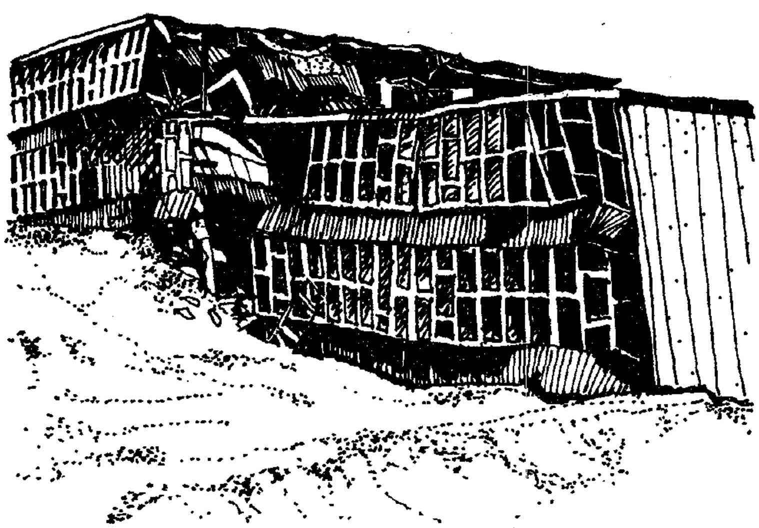
TYPES OF EARTHQUAKE HAZARDS

Earthquake hazards generally fall into three broad categories: surface-faulting, ground-shaking, and tsunamis/seiches. Each of these effects generate particular hazards, and planners and designers have to be aware of each.

Surface-faulting results in the actual disturbance of the ground along an active fault. Only a small percentage of damage in most earthquakes is caused by surface-faulting. Thus, it is not a major concern in terms of the amount of damage; however, where structures are over a fault, they are often destroyed. Therefore, the hazard is very high in the vicinity of an active fault.

FAULTING THROUGH BUILDING
JUVENILE FACILITY - 1971 SAN FERNANDO EARTHQUAKE
Figure 4.1

The second category of hazard is ground-shaking. Ground-shaking produces different effects. One effect is to simply shake a building. The building needs to have an adequate structural design so that it can withstand the shaking. Ground-shaking also induces various forms of land failure. Land failure is of importance to the planner because it can affect the land use pattern. One form of failure is liquefaction, where the soil actually turns to a liquid state and will not support structures. Another form is landslides. Other effects are ground-cracking or lurching of the ground, where it actually moves as if it were thrown about like waves. Finally, there is differential settlement, where the ground settles in different amounts within an area.



GROUND FAILURE
GOVERNMENT HILL SCHOOL - 1964 ALASKAN EARTHQUAKE
Figure 4.2

The third and last category is that of tsunamis, which are sea-waves, and seiches, which are the sloshing effect in enclosed bodies of water. Tsunamis can result in considerable inundation, depending upon the size of the body of water, the depth, and other criteria. Tsunamis and seiches can be tens of feet in height and inundate areas surrounding the body of water.

TSUNAMI DAMAGE
KODIAK, ALASKA - 1964 ALASKAN EARTHQUAKE
Figure 4.3

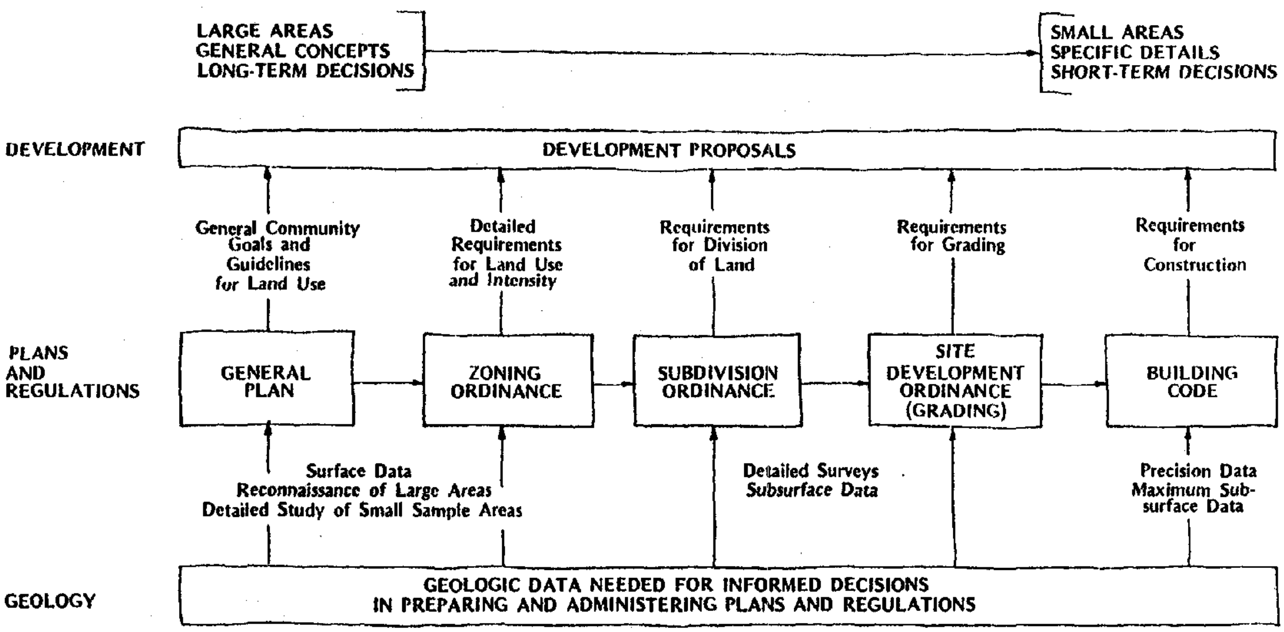
PLANNING–REGULATION–DEVELOPMENT PROCESS

The general plan is the starting point in land use planning. The general plan prescribes land uses in an urban area for a 20-to 30-year period. It indicates the desires of a community and should be based on generalized geologic and seismic data as well as other data basic to plan preparation.

Based on the plan, the next step is to develop a zoning ordinance. The zoning ordinance is a law as opposed to the general plan which is a policy document. Zoning ordinances need to take into account seismic and geologic problems.

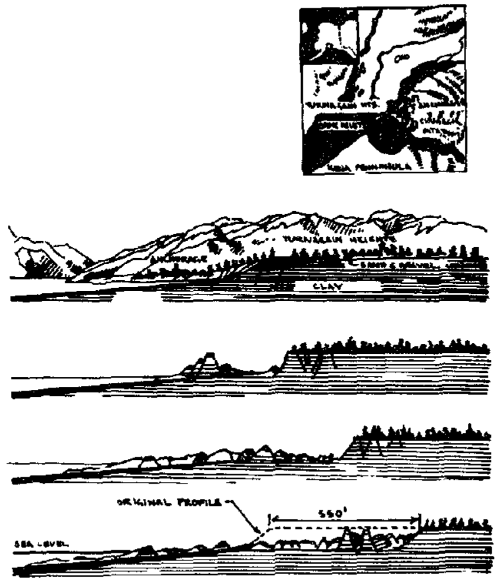
Next is the subdivision ordinance. The subdivision ordinance is crucial in that it controls the way land is divided into parcels and sold. Once land has been subdivided, it is very unlikely that it will ever be reassembled. So, when parcel lines or property lines are established through the subdivision process, we want to make certain that the parcels are useably safe. Therefore, geology plays, or should play, a major role in the subdivision process.

The final two steps in the process address the matter of site development or grading, and building regulations. Planners are somewhat less involved in these latter stages; while geologists, building officials, structural engineers, and architects are deeply involved. Yet here, too, concern must be given to geology and to seismology to make certain that grading is safe and will stand up in the event of an earthquake and to make certain that the structures that are built are safe.



THE PLANNING-REGULATION-DEVELOPMENT PROCESS
Figure 4.4

In the context of the procedure I have outlined, I would now like to give some brief examples of the potential use of the general plan, zoning and subdivision regulations and post-earthquake planning techniques in reducing geologic and seismic risk. The general plan, as I have indicated, should set aside appropriate land uses in various portions of the community and should take geology into account. For example, Turnagain Heights in Alaska was virtually destroyed by a massive landslide in the 1964 earthquake. The landslide reached inland some 500 feet from the bluffs, was almost a mile in length, and destroyed 75 houses. The potential of this landslide was known to geologists prior to the earthquake and had been reported upon. However, this information had not found its way to the planners and decision-makers. A potential major failure could have been dealt with in the general plan had the data been adequately utilized.



TURNAGAIN HEIGHTS LANDSLIDE -
1964 ALASKAN EARTHQUAKE
Figure 4.5

With respect to zoning, we can refer to the San Andreas Fault. It runs through the northern part of San Mateo County in the San Francisco Bay Area. The San Andreas Fault, as we know, is an active fault and is therefore likely to move again. It last moved in the San Francisco Earthquake of 1906 but has been covered by major development in recent years. Homes have been built across the fault with no respect to this major hazard. Zoning could have prevented this; however, the available information was not utilized.

DEVELOPMENT ALONG THE SAN ANDREAS FAULT
Figure 4.6

Subdivision ordinances constitute a third category of regulation. A subdivision in the City of San Jose in the San Francisco Bay Area known as the San Jose Highlands suffered major landslide damage. This large landslide in the late 1960's resulted in property loss valued at about $500,000 and cost the City during the period of 1968 to 1971 about $750,000. Proper subdivision regulations for this area with the information on geology utilized at an earlier stage could have allowed the subdivision to have been designed around the landslide area, or perhaps the potential landslide areas in some areas could have been stabilized.

Finally, there is the matter of post-earthquake planning. The basic post-earthquake problem is how to deal with an urban area after an earthquake. How do we take into account the failures that occurred? Consider the Van Norman Dam complex following the San Fernando earthquake. The lower dam failed partially. Had it failed totally, it could have resulted in flooding of an area of some 80,000 population. That flooding could have been catastrophic had it been sudden. The dam was rebuilt, but the questions that were raised by this near disaster—building a dam above a populated area and the safety of rebuilding the dam—certainly had to be considered. The point is that after an earthquake, such considerations need to be carefully evaluated in the rebuilding of an urban area.

With the foregoing overview of the planning—regulation—development process in mind, lets look at each step in the process in greater detail.

GENERAL PLANS

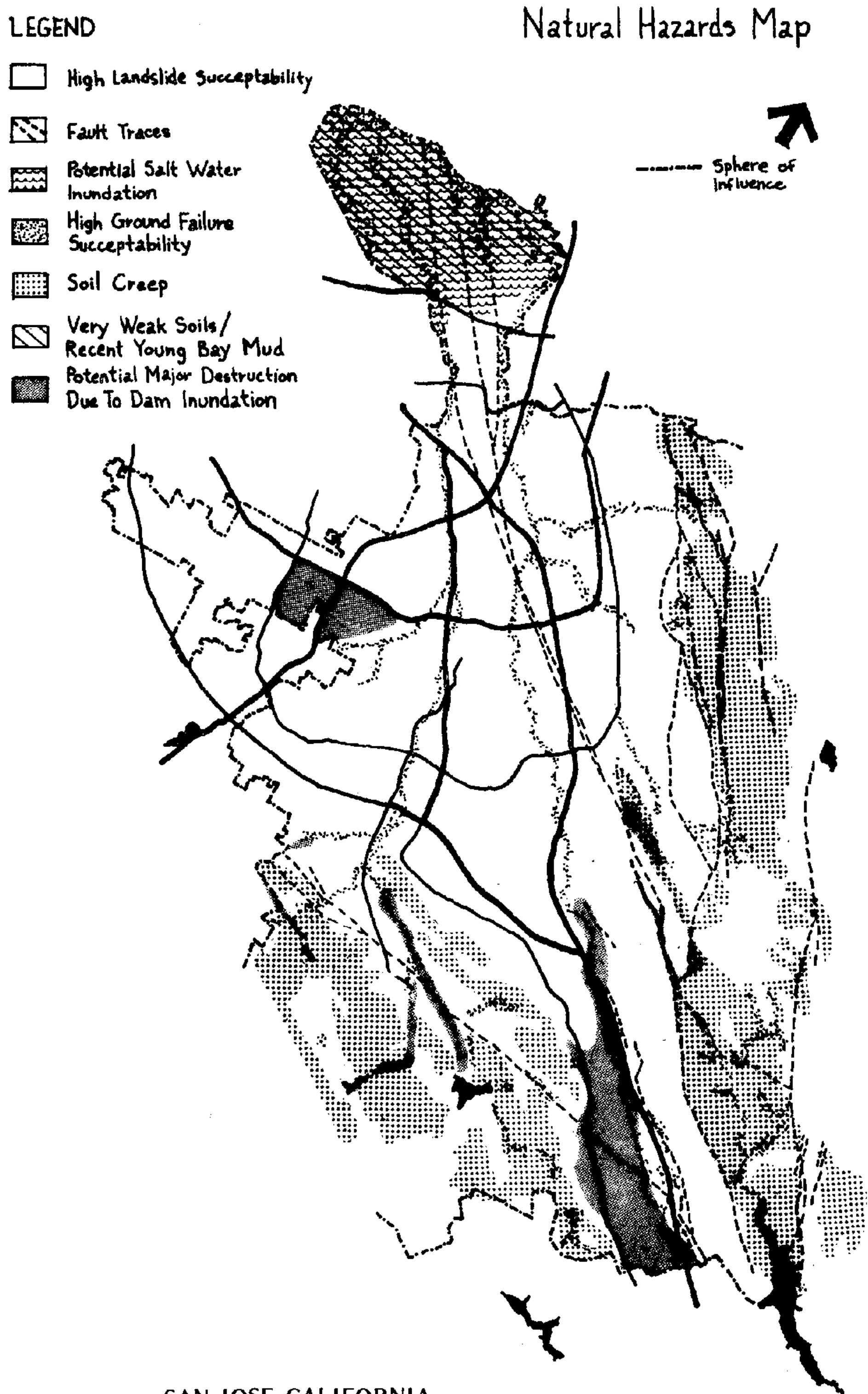
General plans, are of major importance to the planner. General plans usually deal with a wide range of topics such as land use, circulation, housing, open space, noise, geology and seismic safety. California law requires that zoning be consistent with the general plan. Thus, if seismic safety is taken into account in the general plan, zoning must help carry the provisions. The California requirement for a seismic safety element was enacted in 1971 soon after the San Fernando earthquake in accordance with a recommendation of the Joint Committee on Seismic Safety. Section 65302 (f) of the Government Code requires that general plans include:

> A seismic safety element consisting of an identification and appraisal of seismic hazards such as susceptibility to surface ruptures from faulting, to ground shaking, to ground failures, or to the effects of seismically-induced waves such as tsunamis and seiches.

In 1971 the California Seismic Safety Commission conducted an evaluation of seismic safety elements in the State to determine what effect they were having. The survey found that the requirements for seismic safety elements had greatly heightened the awareness throughout the State of seismic concerns. City councils, planning commissions, and their staffs were much more aware of seismic issues; thus, the State law had been salutary in bringing these concerns to the attention of officials and the public.

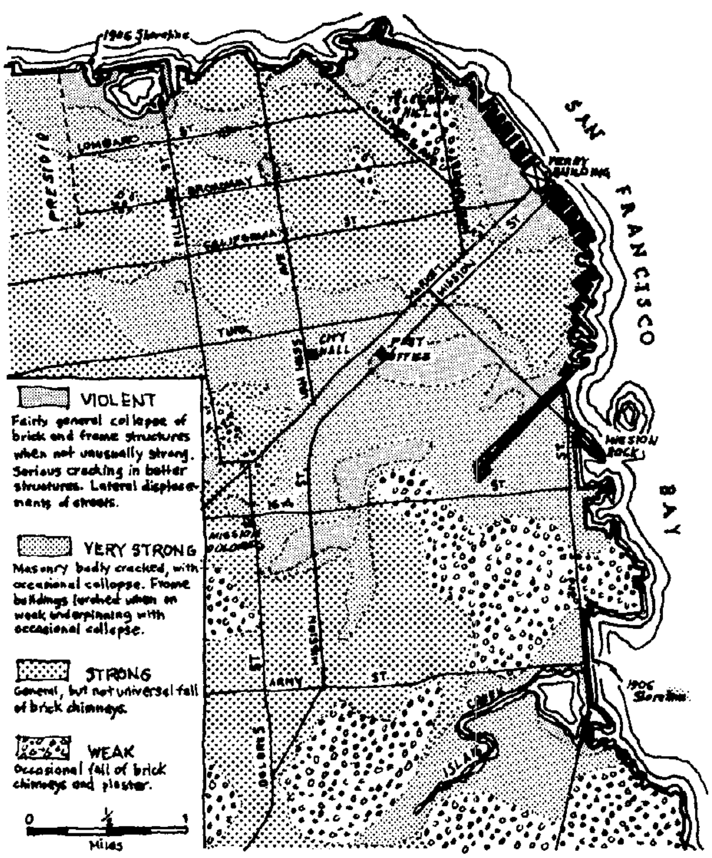
The recently adopted (1975) general plan of the City of San Jose is one of the first efforts in California to include seismic concerns in making major land use proposals. San Jose has been a rapidly growing city. From 1950 to 1975, it grew under 100,000 to 547,000. In developing the seismic safety element, the City staff were greatly concerned that the new growth not get into hazardous areas. A part of that study is shown in the Natural Hazards Map in Figure 4.7. This map shows that in some parts of the Bay there is unstable Bay mud, which would be subject to severe seismic shaking. Also, there are areas in the hillsides that would be subject to landslides in the event of a major earthquake.

In the San Jose plan, there are certain Bay margins that are slated for low intensity uses, perhaps some light industry, or open space and recreation. In response to the possibility of major landslides, extensive hill areas were designated for very low intensity uses and for open spaces.



SAN JOSE CALIFORNIA
Figure 4.7

An additional example of a seismic safety element is the one for the City of San Francisco. It is a significantly different situation than San Jose. San Francisco, of course, experienced severe damage in the 1906 earthquake and has been rebuilt since then, but there are many buildings within the city that cause great concern because they are quite old. The concern is with what are called pre-code Type C buildings, buildings built before 1948, which was the date when comprehensive lateral force requirements, specifically considering seismic forces, were incorporated into the San Francisco Building Code. Type C buildings have masonry or concrete exterior bearing walls with wood floors and roofs. Inventories indicate that in the event of a major earthquake, there are over 14,000 residential buildings with nearly 35,000 living units and 22,800 nonresidential buildings (all of which are identified as pre-code Type C construction) that could be affected. Not all of these would be destroyed during a major earthquake, but it is likely that a high percentage would. Replacing these buildings at 1974 construction costs would run over one billion dollars. Thus, the City has a major investment in these structures but also knows that they would not be structurally sound in the event of a major earthquake.



SAN FRANCISCO CALIFORNIA
Figure 4.8

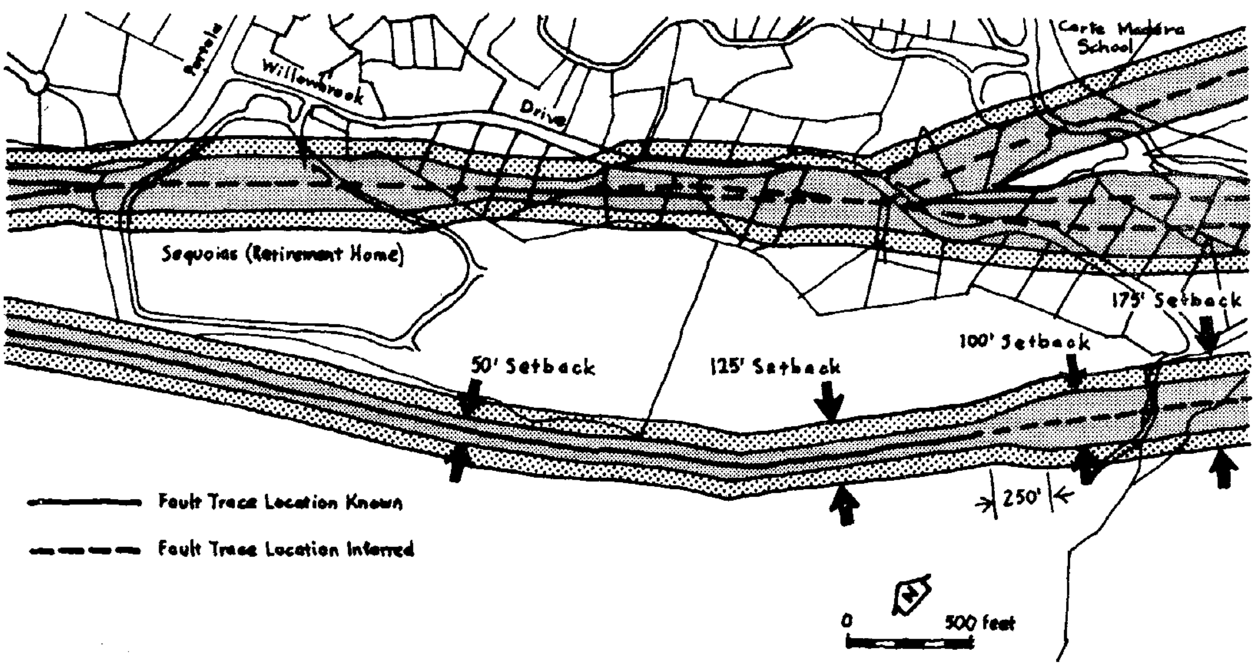
This poses a dilemma for San Francisco because of the amount of value in these structures and because of their importance as historical buildings and architectural significance. The City has adopted an ordinance requiring the removal of parapets and appendages that are hazardous, but has not strictly enforced it because it would require removal of ornamentation from many structures.

The community safety plan attempts to deal with this problem and suggests that these older buildings be gradually strengthened, while the most hazardous portions be removed and reconstructed. San Francisco realizes it has a problem. It is a problem typical of many cities in this country that have large inventories of old structures. The answers are not at all obvious at this time.

ZONING ORDINANCES

Zoning ordinances can be very effective in promoting seismic safety. Portola Valley, a small community lying in the Southern part of the San Francisco Bay Area, astride the San Andreas Fault provides an example. The community recognized the fault hazard and commissioned a study by a geologist to determine the exact location of the fault.

Figure 4.9 shows the study. The dark heavy lines indicate the mapped traces of the fault. The dashed lines indicate inferred locations; that is, locations the geologist could not ascertain directly by looking in the field. He thought the fault must connect between the known traces and so indicated in dashed lines.



PORTOLA VALLEY CALIFORNIA
Figure 4.9

The fault passes, in part, under a school in the community. It then passes on either side of a large rest home. Then, the traces go through the southern portion of the town where they pass through residential lots of one acre in size, passing directly under some residences.

With this mapping completed, the town council appointed a committee which included four geologists, a civil engineer, a planner, and a building official to advise them and make recommendations. The committee made recommendations to the town council that resulted in fault setbacks being prescribed in the zoning ordinance. The ordinance establishes a zone of 100 feet in width centered on the mapped traces. In this zone there can be no building for human occupancy because of the likelihood of ground failure.

Beyond that 100 foot wide band, there are zones 75 feet in width on either side in which the most intensive use permitted is a single-family, one storey, woodframe residence. Although these structures can be damaged, they are relatively safe and, of course, the hazard to population is lower at that density. Beyond the outer band, any major structures require geologic studies regarding the fault hazard.

This regulation is tailored to a particular fault. Geologists believe ground breakage would be confined to a very narrow zone should it move again. Other faults might move over a much broader area, disrupting the ground for a considerable width. In such cases, this particular regulation would not be appropriate. It must be kept in mind that it addresses only the question of fault offset; it does not address other matters—such as landsliding—that are also of concern in that particular community.

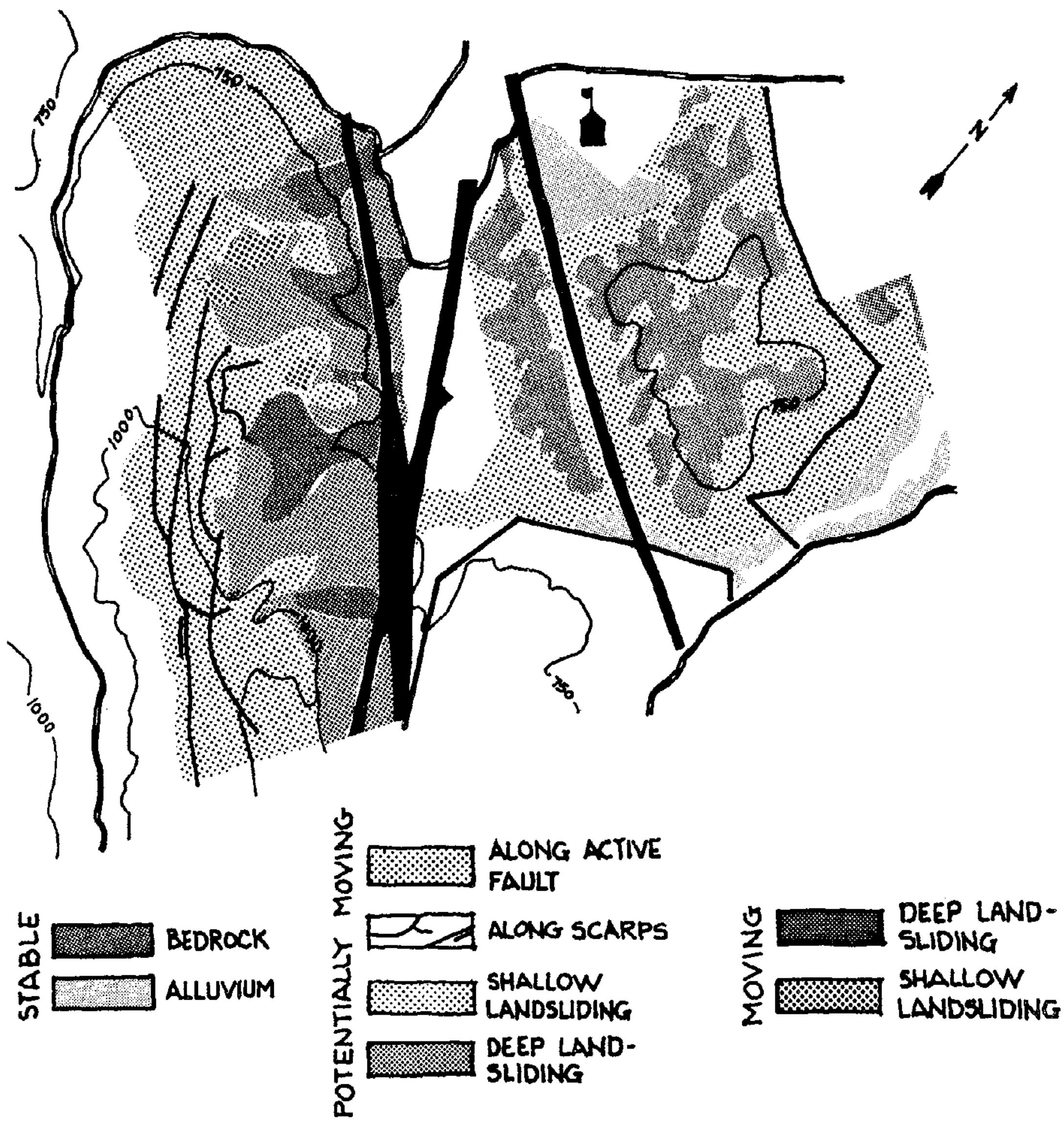
The State of California has also established fault zone regulations, but of a different fashion. In 1972, the State of California enacted what is known as the Alquist-Priolo Special Studies Zones Act. This act prescribes that the State Geologist map the fault zones of the major active faults throughout the State of California. After these zones are mapped, the law then indicates that structures cannot be built astride the fault and that studies must be made for any new major construction within the fault zone. This statewide regulation provides that local jurisdictions require developers to make the studies within the fault zones. The local jurisdictions must then have their own geologists or their consultants review the studies.

SUBDIVISION REGULATIONS

If they have not done it earlier, local jurisdictions should look at geology and seismic safety at the time of subdivision. In California, state law requires the preparation of soils reports for subdivisions, but does not require the preparation of geologic reports. However, a number of cities and counties in the State have elected on their own to require geologic reports. The City of Los Angeles has been a major leader in this field. I would like to point out an example of the use of geology in a subdivision in our case study area, the town of Portola Valley. For many years a 450-acre parcel of land had been used as a ranch. By town ordinance, a developer has to prepare a geologic study as part of a proposed subdivision. Thus, one of the first things the developer did was to have a map prepared showing various geologic formations on the property. Since it was not easy for non-geologists to understand, it was translated to a land stability map (Figure 4.10) that categorized the geologic formations into three major groups:

- land that is actually moving, that is, landslides;
- land that has a potential for moving, that is, could landslide; and
- stable land.

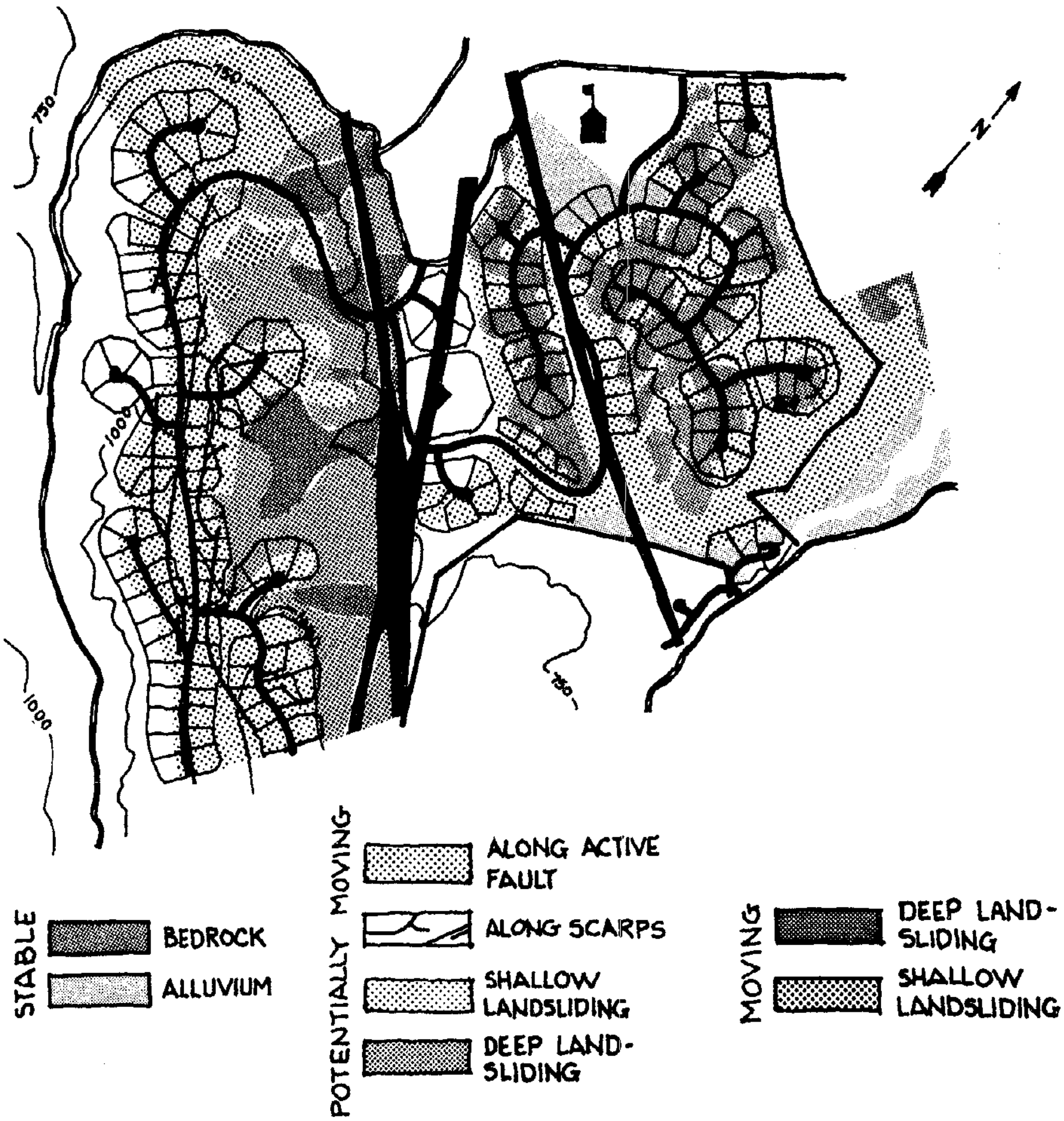
In addition, the San Andreas Fault was located on the property.



LAND STABILITY MAP
Figure 4.10

The general plan for the town also imposed constraints on the developer; it required that the valley in the center part of the property be kept largely natural and also that a road to run up a particular ridge. This road had been a part of the general plan for many years. However, with the geologic data that was developed with the subdivision, the road was no longer deemed desirable and was therefore deleted.

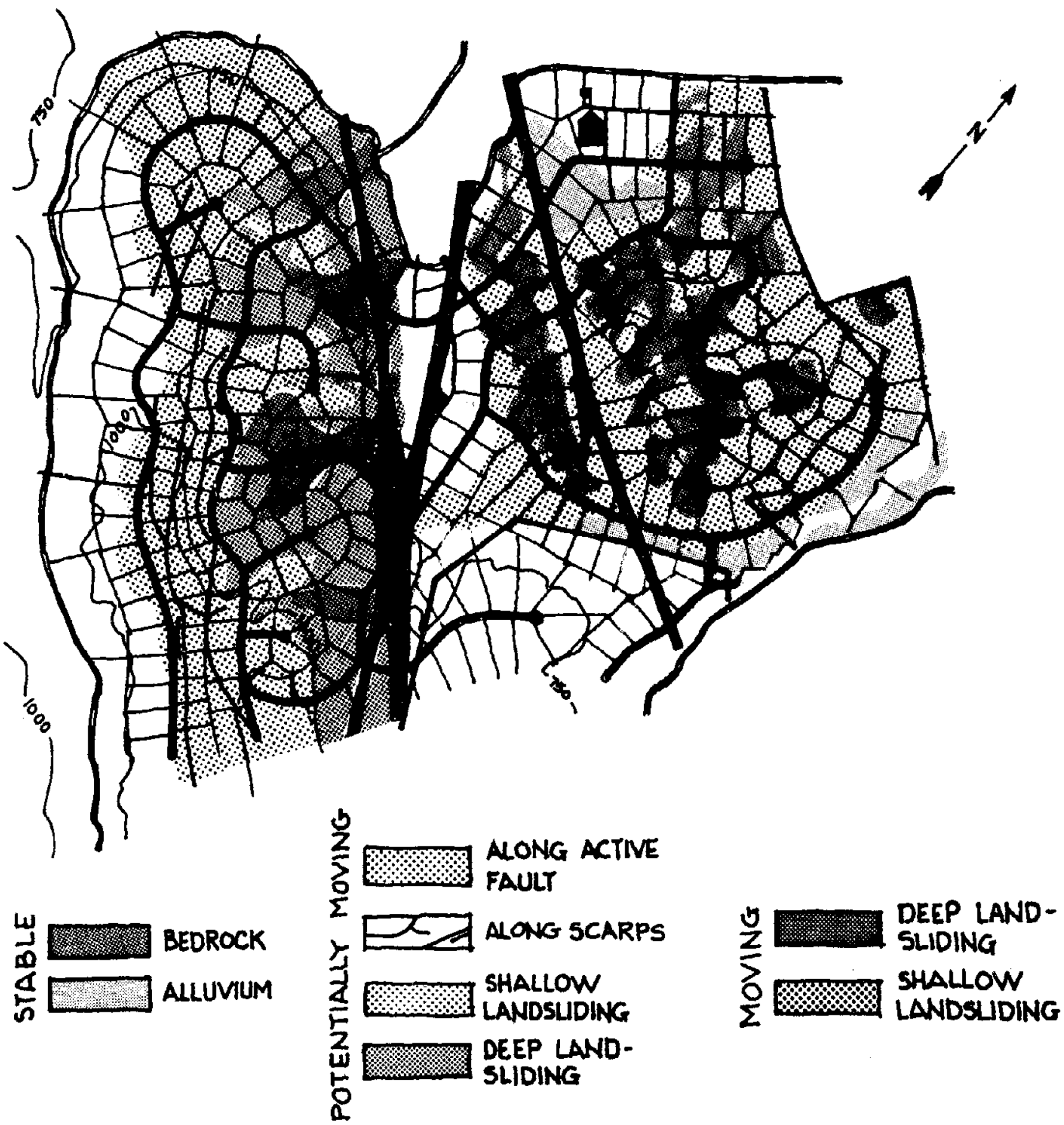
Figure 4.11 indicates the road network and the location of houses in this 1969 proposal. The houses were designed to miss the fault traces and stay off the large landslides area. They were clustered high on the ridge where the land is stable. The plan was for a cluster subdivision, with houses clustered in such a way as to remove development from the vicinity of landslides and faults. The open space was to be land held in common by landowners within the subdivision. The distribution of clusters reflects and respects the geology of the property.



DEVELOPMENT PROPOSAL - 1969
Figure 4.11

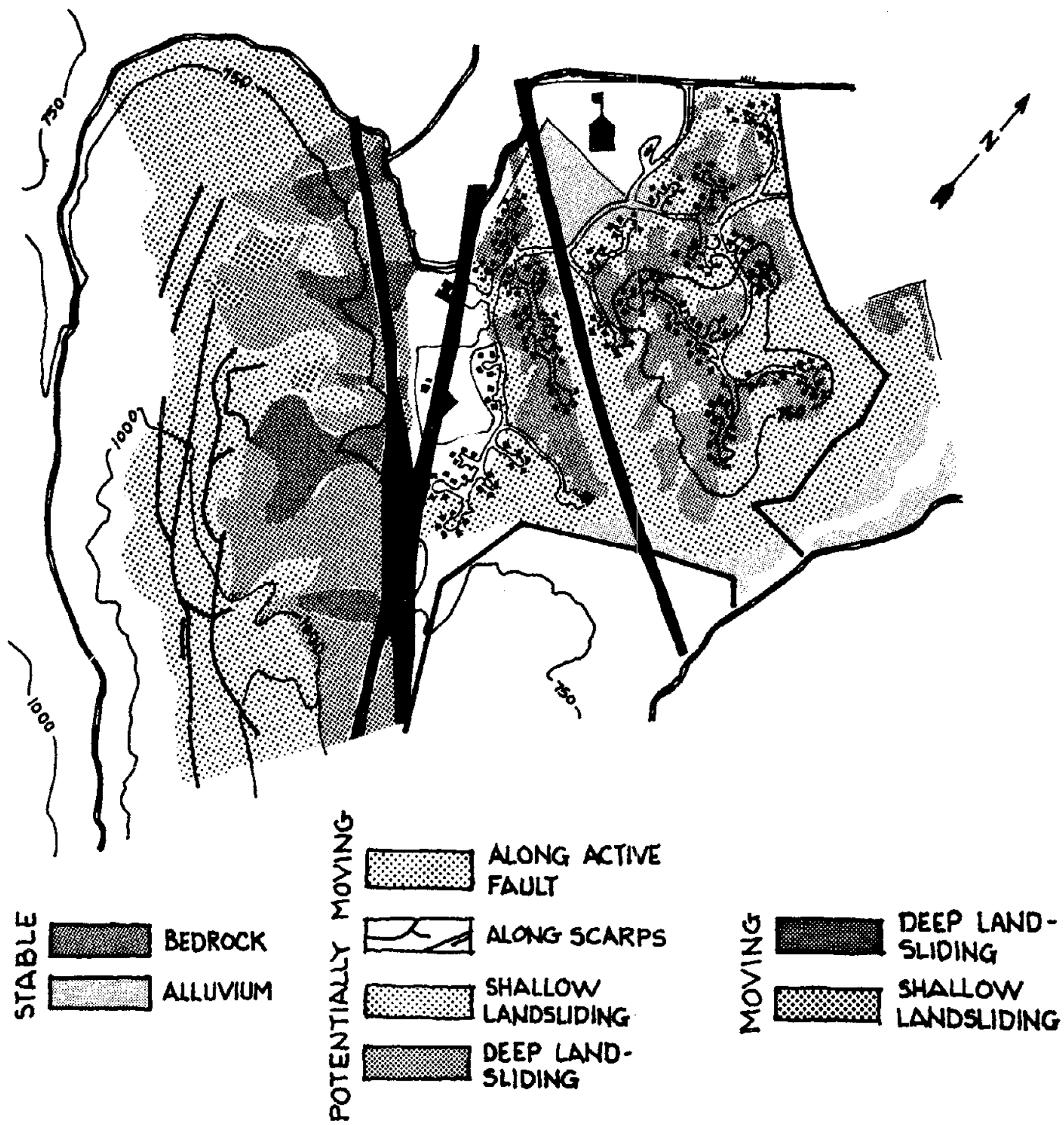
For comparison, one should look at Figure 4.12, which is a subdivision design prepared for this property in the 1950's by a civil engineer for the property owner when the property owner was trying to evaluate the development potential of the property. This was long before the property was sold for development. The property owner reviewed what he thought was a suitable design and counted the lots, something in excess of 200.

By superimposing that development proposal over the land stability map, which was developed much later, it is quite obvious that many of the lots, had they been developed at that earlier stage, would have been directly astride the fault or on landslide terrain. In the 1950's when this proposal was proposed, it is quite likely that this design would have been approved because the state of the art in the use of geologic data in subdivisions had not been developed nearly to the extent that is has been to date.



DEVELOPMENT PROPOSAL - 1956
Figure 4.12

There was a third subdivision proposal made when another developer finally bought the property. Considering the geological studies that had been prepared at an earlier date, this developer decided not even to attempt to develop the more difficult areas, but to cluster all development on the more stable portion of the land as shown in Figure 4.13. Basically, the developer proposed a cluster pattern of some 200 homes on lots as small as about 20,000 square feet or a half acre. The lots are clustered around open spaces and all the lot owners have an undivided interest in the balance of the property, which serves as open space.



DEVELOPMENT PROPOSAL - PRESENT
Figure 4.13

The subdivision is progressing today. Geology continues to play a role as more detailed studies are made at each step of the grading process and as home sites are prepared. The house sites themselves are carefully sited with respect to the terrain. The foundation designs are based on adequate borings and surface information. Adequate structured bracing against shaking from earthquakes is required.

Thus, we have in this subdivision an example of the use of geology throughout the design and development process. The subdivision could have been hazardous had the seismic and geologic hazards not been recognized and dealt with.

POST-EARTHQUAKE PLANNING

After an earthquake, there is a strong tendency to put things back the way they were. People like to rebuild communities, forget the earthquake, and go about their lives. This is understandable. There is great pressure for that. There is also an economic need for the city to function again; investments in property are large and one does not easily erase those investments. Furthermore, it is difficult to make changes. Thus, the easiest thing to do is to patch the city up and go on with business as usual.

We are beginning to see, however, that there is a need to give consideration to planning differently after an earthquake to help avoid a repeat of a disaster. Several examples illustrate this point.

The Fourth Avenue landslide in Anchorage, Alaska, caused considerable damage to the downtown area. After the earthquake, studies were made to determine whether this area should be rebuilt. The studies indicated that it would be possible to put in a large buttress fill that would stabilize the landslide area. The project went ahead. There are new buildings and the buttress fill was installed.

Another example in Alaska is the port city of Valdez. Valdez is a small city that lies on the coast. At the time of 1964 earthquake, there was a massive underwater landslide that took out part of the harbor and triggered waves that inundated and destroyed a major part of that city.

Valdez is important because it is an ice-free port and a potential outlet for the development of the natural resources of the interior. These concerns led the Federal government to take a major interest in Valdez. The result was that the community was located three-and-one-half miles north at Federal expense. A completely new town was built in a safe location. This action is, of course, very drastic, but in some cases such actions may be the only appropriate course.

Other countries have had similar problems. One example of a major earthquake that affected a foreign country is that of Skopje, Yugoslavia. Within five seconds the 1963 earthquake, killed 1,000 people, injured 3,000, and left 150,000 people homeless.

While Skopje allowed immediate emergency reconstruction, they also started a major replanning effort, particularly in the central portion of the city. With the aid of the United Nations, officials came up with a dramatic plan to rebuild the city to recognize the major seismic problems. The plans developed included temporary as well as permanent housing and included major changes to the downtown areas.

But whatever the strategy, whether circumstances call upon the planner to develop a proposal for safe land use after an earthquake, as in the case of Valdez and Skopje, or to investigate the risk to existing or proposed development before an earthquake occurs, the lesson that has been learned through experience is that seismic safety needs to be incorporated throughout the planning, regulation, and development process. For all the evidence points to a single, hopeful conclusion: land use planning can have a major impact on increasing seismic safety.

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Chapter 5. URBAN DESIGN AND SEISMIC SAFETY

Henry J. Lagorio, A.I.A.

INTRODUCTION

In the United States, as well as in other highly-developed countries which were early beneficiaries of the industrial revolution, large scale urbanization began in the latter half of the 19th Century. Since then, there has been a steady migration from rural areas to urban centers -- from the countryside to cities and metropolitan areas. In the past few decades the number of urban places has still been increasing, although the growth rate has leveled somewhat, with most of the population growth taking place in existing, well-established urban concentrations. It has been predicted that by the year 1990, more than half the world's people will be living in cities with a population of 100,000 or more. It is clearly evident that the risk to populations exposed to earthquakes is most critical for those in highly congested urban centers in contrast to those living in the suburbs. The San Fernando earthquake of 1971, as a case study, is noteworthy in this respect because of what might have happened had it occurred in the center instead of at the edge of the Los Angeles metropolitan area, or if the Van Norman dam had collapsed completely without prior evacuation of the 70,000 downstream population.

North America is situated in a seismically active region of the globe. The United States is laced with earthquake faults that spread over much of the nation and lace many important urban areas. Accordingly, it is the responsibility of the Urban Designer in concert with the other design professions to take into consideration all practicable measures that will reduce the high levels of risk to earthquake hazards currently faced by major cities. Thoughtful Urban Design concepts that may be used to address the problem may dramatically mitigate the country's urban exposure to earthquake hazards if comprehensively developed and diligently implemented.

Records of testimony given by expert witnesses during U.S. Senate hearings on "Governmental Response to the California Earthquake Disaster of 1971" reveal that the increasing population density of our cities is creating problems whereby a very localized earthquake in an urban setting can cause a major catastrophe such as was not possible some years ago. The pressures of population growth that have resulted in high levels of urbanization throughout the nation have also caused expansion into areas that are more difficult to develop safely than those of the past decade as society becomes more complex and interdependent.

SPECIFIC ISSUES

Despite many technological advances in earthquake engineering and earthquake prediction, our major cities are still known to be extremely vulnerable to a major earthquake event. Since the 1906 San Francisco earthquake and fire, we have been most fortunate, or very "lucky" from the probabilistic point of view, that at least not one of our major metropolitan centers has been hit in a "bull's-eye" fashion by a severe earthquake or even a moderate one. Yet such an occurrence is long overdue and inevitable.

The problem is further compounded by the fact that most urban concentrations are totally unprepared to absorb such an event. Professor Karl Steinbrugge, former Chairman of the Seismic Safety Commission of the State of California, has openly stated that some of our major cities are "catastrophes waiting to occur." Taking the area of San Francisco, for example, the last major earthquake occurred in 1906 with an 8.2 magnitude on the Richter scale. Reputable seismologists indicate that, for California, the recurrence interval for such an event is related to a 60-100 year cycle which signifies that that area entered into a critical period of time starting with 1966, now already eleven years past.

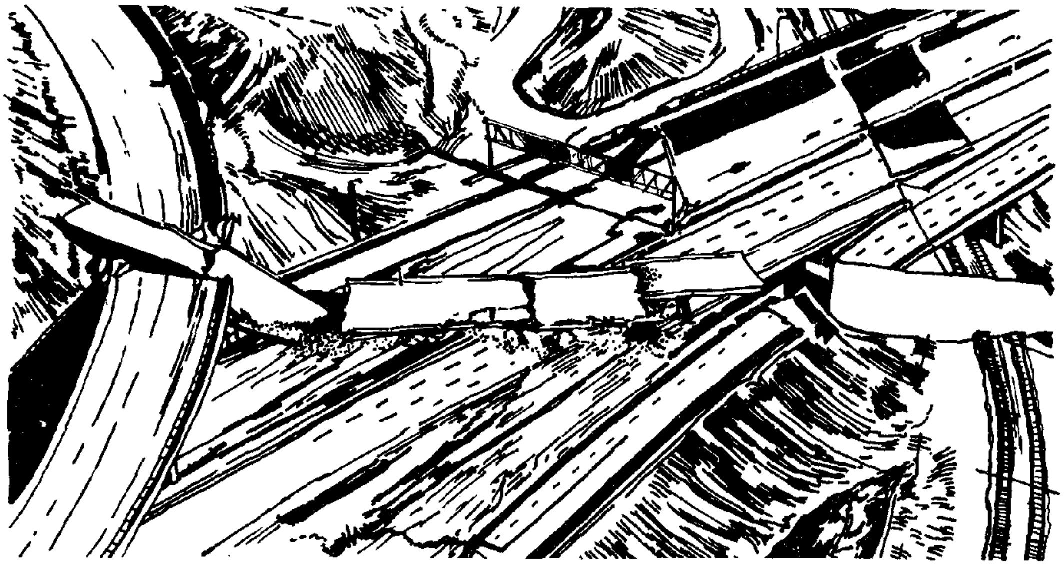
INTERRELATIONSHIP OF URBAN SYSTEMS

The urban environment is a complex and closely knit fabric composed of many interdependent activities, services, and facilities. Given its complexity the failure of one of its single components can severely affect the functioning of others. The recent 1977 "brown out" experienced in New York City in which the inability of a public utility company to supply power after only one of its secondary feeder lines was struck by lightning is but one example of how an isolated problem may become compounded into a full-scale crisis of many consequences including 'collapse" of the function and integrity of an entire urban system.

Similar parallels exist on the occasions when a major metropolitan center is subjected to a severe earthquake. Two recent examples on record, among many others, may be cited as follows:

Example 1

The dependence of a major city on the continued functioning of its transportation system, freeways and highways, as a component of life line considerations came into sharp focus as a result of the 1971 San Fernando earthquake. A total of 58 State highway bridges were damaged and, of these, seven either collapsed or were demolished, according to a report prepared by the State Division of Highways. Large sections of Interstate 5, the San Diego Golden State Freeway, were closed to traffic which had to be rerouted through local streets. Accordingly, entire sections of the north-south transportation system of the Los Angeles metropolitan area were severely crippled and removed from significant participation in post-earthquake recovery. If the situation had become further complicated by the collapse of the Van Norman dam, complete failure of the entire urban system to respond to the disaster would seem to have been likely.



DISRUPTION OF TRANSPORTATION SYSTEM
GOLDEN STATE - FOOTHILL FREEWAY - 1971 SAN FERNANDO EARTHQUAKE
Figure 5.1

Example 2

Conflagration due to fire following an earthquake is one of the most dreaded hazards facing our urban centers. In the 1972 Managua, Nicaragua, earthquake large areas damaged by the earthquake were later completely razed by the fire that followed. That the fire remained unchecked and resulted in conflagaration was, in major part, because failures occurred in the water supply system when two major 24-inch lines were severed by ground breaks and pumping stations were blocked by landslides; and because of the inability of the Fire Department to respond to the emergency when two major fire stations, including the headquarters building, suffered major damage and most of the fire fighting equipment was trapped under the rubble. These two isolated events had drastic consequences for the entire urban fabric of Managua. Fires broke out in four or five places shortly after the earthquake and within two days fires were burning in all parts of the city.

DISRUPTION OF CRITICAL SERVICES
FIRE STATION - 1972 NICARAGUA EARTHQUAKE
Figure 5.2

Analysis of these examples, and others not cited, clearly indicates that the combination of two or more isolated but closely interrelated events is the source of the problem. It is not enough to attempt to mitigate the problem by strengthening buildings or legislating public policy; rather, it is necessary that the entire range of Urban Design skills be considered as an integrating factor in reducing risks assumed by our metropolitan centers. Urban design requires a holistic, systems approach to the problem in order to develop and implement a coordinated effort in earthquake hazards reduction programs. It requires a multidisciplinary team composed of the following to address the problem: Design Professions, Government Officials and Community Representatiives.

While much research has focused on the individual elements of the urban environments, such as independent studies on life line systems, building and structural design, land use, and hazards vulnerability, little or no work has been done to date in approaching the problem from a synthetical, rather than an analytical, point of view. Urban Design, by its very nature in synthesizing the problem, emphasizes the functional relation between parts and wholes. Just as the earthquake event indiscriminately affects everything in a metropolitan area through overall ground shaking, Urban Design treats the complexity of the problem as being irreducible to the sum of its parts. Urban Design theory starts with the assumption that it is impossible to simplify or make the problem easier by analysis due to the inherent and dynamic complexity of the physical, urban setting and its intricate interrelationships.

VULNERABILITY OF URBAN AREAS

The magnitude of the problem facing major urban areas, in light of the threat posed by potentially destructive earthquakes, is staggering. Figure 5.3 is a partial, representative listing of some of the major metropolitan areas in the United States found in zones subject to moderate or severe earthquake activity, that is, located in Seismic Zones 2, 3, or 4 as designated by the Uniform Building Code (UBC), 1976 Edition. Not listed in the table are other major cities like Cleveland, Chicago, or Detroit which, although not located in critical seismic zones, will nonetheless be affected by "long period" ground motions resulting from the occurrence of major earthquake events in other highly active seismic areas miles distant. In the Los Angeles metropolitan area alone, it can be seen that the population at risk is close to 9 million people.

Recent earthquake studies completed by the United States Geological Survey (USGS) for the Federal Disaster Assistance Administration (FDAA), and prior studies by the National Oceanic & Atmospheric Administration (NOAA) for the Office of Emergency Preparedness (OEP), reveal that the recurrence of a maximum credible earthquake in selected high density urban areas around the country would result in major disasters with staggering costs in casualties and life loss. Figure 5.4 lists the life safety and homeless implications of the USGS and NOAA studies in the selected urban areas.

It is significant to note that a recurrence of the 1906 San Francisco earthquake today would result in over 10,000 deaths and 40,000 hospitalized injuries (not counting the possibility of dam failure in the study area, which has a high probability). Another recent study on the recurrence of the 1906 San Francisco earthquake today indicates that the dollar loss in terms of repair costs to single family wood frame dwellings alone would approach $1,240,000,000. This is in contrast to well-documented statistics on the actual San Francisco earthquake in 1906 that show that there were only 700 deaths and a total dollar loss of $524,000,000 at that time caused by the original event and fire. Even taking inflation dollars into account, the projected losses for the recurrence of

Name of Area:	Seismic Zone 1976 - UBC	Population at Risk
Anchorage, Alaska	4	166,000
Los Angeles, California	4	8,960,000
San Francisco, California	4	4,450,000
Boston, Massachusetts	3	3,795,000
Buffalo, New York	3	505,000
Charleston, S. Carolina	3	296,000
Memphis, Tennessee	3	555,000
Salt Lake City, Utah	3	561,000
Seattle-Tacoma, Washington	3	1,788,000
San Diego, California	3	1,355,000
Atlanta, Georgia	2	1,780,000
Cincinnati, Ohio	2	1,162,000
St. Louis, Missouri	2	1,747,000

SOURCE: (a) 1977 Commercial Atlas & Marketing Guide
Rand McNally & Company

(b) UBC, 1976 Edition

SEISMIC ZONE LOCATION OF SELECTED U.S. METROPOLITAN STATISTICAL AREAS
Figure 5.3

Name of Area	Injuries	Deaths*	Homeless
San Francisco, California	40,000	10,000	58,000
Los Angeles, California	75,200	18,800	140,000
Salt Lake City, Utah	9,200	2,300	29,600
Seattle, Washington	28,000	7,000	45,000

*Note: Excluding dam failure

EARTHQUAKE VULNERABILITY ANALYSIS OF SELECTED U.S. URBAN AREAS
Figure 5.4

the 1906 earthquake in the currently expanded and highly congested urban concentrations of the San Francisco Bay Area are still overwhelming and present an unacceptable risk to the public.

Most city planning provisions and building ordinances throughout the nation usually do not take into account the possibility of surface faulting due to earthquake. In recent times, responsible design professionals and practitioners have been known to sometimes persuade clients to consider alternative sites only to have the faulty site developed by others who may have been uninformed of the hazard. Currently, only the State of Califonia has a fault-zone hazards ordinance in effect that mandates cities to take fault-line hazards into account before site development begins. In other major cities outside of California, it is entirely conceivable that major structures will continue to be constructed in fault zones. Even in California some major facilities, such as general hospitals and the San Francisco Bay Area Rapid Transit System (BART), were located across known fault zones, prior to enactment of the Alquist-Priolo Fault-Zoning Hazards Act of 1974. In all probability the pressures of urban growth and increased population needs were responsible for such design decisions. Certainly there is far less chance that such questionable urban design procedures are found in sparsely settled rural areas. Accordingly, due to the public risk involved, the emphasis can be on Urban Design to form a balanced program in addressing the problem.

An Urban Design principle, cited as but one example among many others to illustrate its potential use in earthquake hazard mitigation, relates to zoning ordinances that affect the height, volume, and setback of buildings in relation to public thoroughfares. This same relationship of buildings to the street is most important from the earthquake hazards mitigation point of view when dealing with "debris removal" in immediate post-earthquake recovery. The height of the building, the width of the street, and the distance of the building from the street are important variables to consider when dealing with the debris problem. It is critical to urban recovery goals that fire and rescue teams have clear access to damaged areas as soon as possible following a severe earthquake.

STREET SCENE - 1906 SAN FRANCISCO EARTHQUAKE
Figure 5.5

SPECIFIC TOPICS

The following represents a partial preliminary listing of potential representative topics that might be considered as they relate to urban design and seismic safety. These will require additional review to verify their importance and significance to seismic safety and urban design concerns:

- Urban District Training Programs
- Existing Hazardous Buildings
- Emergency Critical Use Facilities
- Fire Following Earthquake
- Debris Removal in Urban Areas
- Homeless and Temporary Urban Housing
- Urban Redevelopment Options
- Life Lines (Utility and Communications Systems)
- Transportation and Urban Mass Transit Systems
- Earthquake Hazards Reduction and Implementation Plans
- Urban Seismic Safety Elements
- Urban Open Space Zoning
- Earthquake Hazards Vulnerability Analysis Studies for Metropolitan Areas
- Typology of Urban Disasters
- Urban Disaster Prevention Plans
- Urban Supply and Disposal Systems
- Urban Microzonation Techniques

In terms of earthquake disaster mitigation, Urban Design represents a new, emerging field that has been under-utilized as a profession. Its role and usefulness in earthquake hazards reduction programs should be reviewed and assessed as a potential tool in devising an integrated design approach to the problem. As a discipline, it involves a multidisciplinary, team approach to problem solving relative to the physical three-dimensional design of major urban centers. Proper use of urban design principles could have significant potential for application to seismic safety concerns and goals.

Microzonation techniques based on urban topologies should enable Urban Designers, representing a specific sector of the design professions, to improve the manner in which they give consideration to earthquake hazards reduction measures. This should help both governmental and private sector agencies to plan and implement physical design principles in urban redevelopment efforts as well as allow public officials an additional method of responding to earthquake hazards. An urban design approach should be of long-term use to design professionals and public officials as they plan for new communities and the redevelopment of existing city centers.

A multidisciplinary approach composed of design professions, government officials, and community representatives is necessary for a balanced program in disaster mitigation.

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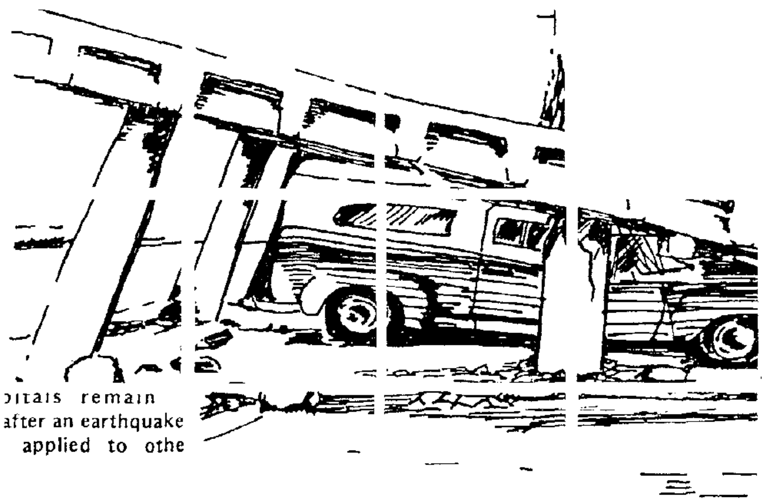
POLICY-POST 1971

Part 3
Public Policy and Seismic Codes



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Chapter 6. PUBLIC POLICY FOR SEISMIC SAFETY - EASTERN U.S.

Norton S. Remmer, P.E.

HISTORY OF PUBLIC POLICY DEVELOPMENT IN SEISMIC SAFETY

The development of public policy concerning seismic safety is a relatively recent phenomenon. The Federal government first became involved after the 1964 Alaska earthquake. The 1971 San Fernando earthquake significantly accelerated this involvement. Why the concern? In part because the Federal government has the responsibility of providing the disaster relief. However, most local and state authorities still are not aware of the immediacy of seismic safety problems, except on the West Coast, especially California.

Certain of the problems that are involved with developing policy have to do with the cost of enforcement and the ability to enforce. Much of the policy for the existing seismic codes is related to controlling the design and construction of buildings through the code. The end building product is the result of the regulatory body's ability to control conformance with the code and to ensure that the end product represents a building with a certain level of safety. This means inspection. That process is perhaps most difficult in the East because building officials have not been faced with the degree of technology in their administration that seismic provisions require; thus the problem of either training building officials or bidding higher for more qualified building officials is beginning to become a critical element in the enforcement of seismic code provisions.

Then there is the question of economics—the economics of incentives and the problems of costs. Simply creating codes does not accomplish the desired end result. Inevitably, it will mean additional cost in building inspection, building inspectors, etc. To convince state and local governments to incorporate expensive provisions in codes or to provide the ability to enforce them means convincing them that the cost is worthwhile.

Finally, one of the last practical questions is the matter of existing buildings. The greatest number of buildings that we have to worry about right now are the existing buildings which have not been designed for seismic resistance. In the east, they represent the greatest hazard. The West Coast has had a range of seismic provisions for a number of years, but the East Coast has had very few seismic provisions. In the west the need for seismic provisions is not in question. In the East, on the other hand, the need is not nearly so apparent. The Massachusetts Seismic Commission, for example, has stated that they are not going to get involved to any great degree in existing buildings. In many respects it would be a policy decision for the legislature or local officials.

PUBLIC POLICY RESPONSE IN CALIFORNIA

It is important to get a base reference in discussing public policy in seismic safety. California is useful because it is subject to earthquakes—it has had earthquakes and it has had them very recently. California represents one aspect of public response on a state-wide basis. Its citizens have been compelled to face the imminence of disaster due to earthquakes. Their response represents a guideline for comparison of what may be expected for policy development throughout the United States.

After the San Francisco 1906 earthquake, the city was rebuilt based on a code providing 30-pounds-per-square-foot loading for both earthquake and wind resistance. The city simply established a requirement for a lateral force to respond to both earthquakes and wind.

In 1927, the Uniform Building Code incorporated a provision for lateral earthquake forces proportional to the mass of a building. This was the first code recognition of the validity of some of Newton's principles. It represented the first step toward developing a concept that reflected dynamic forces.

In 1928, the California State Chamber of Commerce recognized a need for a building code dedicated to the safeguarding of buildings against earthquake disaster. This was the first real policy statement relative to earthquake safety for California. It generated studies by leading structural engineers of the state, studies that formed the basis for the codes that followed.

Once the Long Beach earthquake occurred in 1933, there was the realization that there were problems of imminent hazard. The State Legislature adopted the Field Act which set guidelines for the design and the construction of public school buildings. Many schools had been destroyed in the Long Beach earthquake and the state realized that except for the good fortune of the time of occurrence of the earthquake, much of the school population in Long Beach would have been affected.

Up to this point, although they recognized the earthquake hazard, California was to a great degree reacting to events on an individual basis. Furthermore, up to that point, the nature of the problem had been defined by the San Francisco earthquake which, with the fire that followed, destroyed an extensive amount of San Francisco. Here, in other words, was a specific response to a general problem. One of the reasons that I bring this up is that this reflects what constitutes a good deal of the public policy still affecting seismic codes and seismic code policy—specific and limited reactions to specific occurrences.

Also, in 1933, the California Legislature adopted the Riley Act. It required that all buildings, except dwellings and farm buildings, be designed to resist a certain lateral force. It did not require detailed provisions for seismic design; it was simply a recognition of a minimal level necessary to provide for earthquakes.

In 1933, a Los Angeles building ordinance required a lateral force of eight percent of the dead load plus one-half the live load. In 1935, the Uniform Building Code adopted provisions that were similar to those that Los Angeles adopted; that is, eight percent and one-half the live load. In 1943, Los Angeles incorporated a coefficient recognizing the influence of flexibility.

Finally, in 1947, San Francisco adopted a table of varying coefficients applied to vertical design loads for buildings of different heights with variations for soil conditions.

It was not until 1947 that codes began to reflect modern seismic design provisions. From 1906 (that is, the San Francisco earthquake) until 1947 (the initiation of what we can call modern seismic design provisions), we had a very limited response to recognizable actual disasters and to imminent disasters.

The Structural Engineers Association of California (SEAOC) "Bluebook" is a code for lateral force provisions for the design of structures. In 1960 SEAOC published a seismic code which was the first time that fairly comprehensive seismic design provisions were incorporated in one code. This marked the beginning of modern concepts of seismic design in codes.

In 1963, the code was republished with one minor revision. In 1966, it was republished with several changes—again, these were minor. In 1967, it was republished with an updated commentary. In 1973, it was republished with additional commentaries. In 1974, it was republished with a great many significant changes. Thus, there was an interval of 14 years, which might be called a developmental period, before there was an extensive rewrite and update of the seismic code provisions in California.

San Francisco	After the earthquake. Rebuilt under a code providing 30 pounds per square foot loading for both wind and earthquake resistance.
1927	Uniform Building Code included provision for lateral earthquake forces proportional to masses.
1928	California State Chamber of Commerce recognized need for a building code "dedicated to the safeguarding of buildings against earthquake disaster."
March 1933	Long Beach earthquake destroyed many public school buildings.
1933	State legislature adopted "Field Act" controlling design and construction of public school buildings.
1933	Riley Act adopted required all buildings except dwellings and farm buildings to be designed to resist a certain lateral force.
1933	Los Angeles Building Ordinance required a lateral force of 8 percent of dead load plus half live load.
1935	Uniform Building Code adopted similar provisions.
1943	Los Angeles incorporated a coefficient recognizing the influence of flexibility.
1947	San Francisco adopted table of varying coefficients applied to vertical design loads for buildings of different heights with variations for soil conditions.

EARTHQUAKE CODES IN CALIFORNIA
Figure 6.1

It is interesting to look at what has occurred in California since 1971, the date of the San Fernando earthquake. In 1971, the Legislature required all cities and counties to adopt a general plan that included a seismic safety element consisting of an identification and appraisal of seismic hazards, such as susceptibility to surface ruptures and faulting, to groundshaking, to ground failures, or to the effects of seismically-induced waves such as tsunamis and seiches. Again, it was a reaction to a specific event. I emphasize this because I think if one sees how California's code evolved, we can then understand the inertia or possibly the reluctance in terms of developing seismic policy in other parts of the country.

In 1973, the California Legislature passed legislation requiring that hospitals remain completely functional during and after an earthquake. Again, as a result of experience in the San Fernando earthquake, they found that although they could design buildings that did not collapse there could nevertheless be a problem with critical facilities that were totally inoperable during a disaster period.

1960	• First Publication of Seismic Code by SEAOC
1963	• Republished with one revision
1966	• Republished with several changes
1967	• Republished with updated commentary
1973	• Republished with additional commentaries
1974	• Republished with significant changes

SEAOC CODE - "BLUE BOOK"
Figure 6.2

Consequently, the state decided that there was a need to provide hospitals that could remain not only standing but totally functional and that the critical equipment inside the building would remain functional for the worst possible earthquake. Again, this was a direct response to the 1971 earthquake, although it was not until 1973 that they responded to that event.

1971	San Fernando Earthquake
1971	Seismic Safety Element As part of legislation requiring all cities and countries to adopt a general plan is included in the following: "A Seismic Safety element consisting of an identification and appraisal of seismic hazards such as susceptibility to surface ruptures from faulting, to ground shaking, to ground failures, or to the effects of seismically induced waves such as tsunamis and seiches."
1973	Hospital Safety Act Requires that hospitals remain "completely functional" during and after an earthquake. This concept may be applied to other "critical" buildings also.

EARTHQUAKE POLICY-POST 1971
Figure 6.3

PUBLIC POLICY RESPONSE IN MASSACHUSETTS

The history in Massachusetts reflects to a great extent the eastern approach to seismic codes. In 1947, the first state-wide building code was issued. It was not a mandatory code; it was simply a uniform code based on the then current version of the BOCA code. It was applied primarily to state-owned buildings and to certain assembly uses. At that time there was nothing in the Building Officials & Code Administrators International (BOCA) code about earthquakes, nor was there any state concern about earthquakes.

In 1962, the building code of the City of Boston contained no mention of earthquake consideration. There was the knowledge in academic circles that Boston had had two earthquakes in the 1700's and that there was the theoretical potential for a significant disaster. But people simply do not consider earthquakes a hazard in Massachusetts, much less Boston.

In 1969, the U.S. Geological Survey permitted the publication of a revised seismic risk map of the United States which placed Boston and the North Shore in Zone 3 (highest risk), most of the State in Zone 2, and the Southwest corner in Zone 1. This map in essence microzoned the State into three zones. Suddenly, part of Massachusetts had been catapulted into a Zone 3; in fact, probably half of the population of the State lives in the area that is bounded by Zone 3.

In 1970, the Uniform Building Code (UBC) incorporated the seismic risk map as published with slight variations. This map incorporated not only Boston in Zone 3, but also Memphis and Charleston. To adopt the UBC as written with that map meant that designers in Boston, Memphis and Charleston, would be designing buildings in accordance with the same provisions and requirements as Los Angeles or San Francisco. This is understandable if you reflect that the maximum potential earthquakes possible in Boston, Memphis, and Charleston may be at a level that would justify the increase. But the interval between earthquakes is much longer; thus one had to look further into public policy and ask if a high risk designation is justified considering the long interval between each event.

In 1970, BOCA (which traditionally Massachusetts has basically adopted and followed) sited Massachusetts in Zone 2 with relatively minor earthquake provisions. However, there was a good possibility that BOCA would adopt the new UBC map. Because BOCA was generally the basis for the Massachusetts code, this posed a problem. To adopt BOCA in 1970 as a matter of course meant adopting the BOCA risk map and simply following those earthquake provisions. Therefore, either Massachusetts would have to implement rigorous requirements or they would have to develop their own code.

On July 1, 1970, the revised building code for the City of Boston was published by the City's Building Department. At that time, Massachusetts allowed each city or municipality to adopt its own code. They accepted Zone 1 for Boston and used the 1967 Uniform Building Code (UBC) earthquake regulations that required only that buildings be capable of withstanding lateral forces. The code remained vague on the question of required ductility. Nevertheless, it was the first introduction of seismic requirements into the Boston building code.

In essence, Boston was forced to look at the seismic problem and follow the 1967 UBC. Both Boston and Massachusetts were reacting to the problem because they had no choice; and they were referring back to a 1967 UBC which at that time had not yet incorporated extensive provisions for ductility.

The year 1971 marked the first incorporation of seismic provisions in the code of an eastern state. Massachusetts developed a separate school code that was independent and overrode the state building codes. Every school in the state had to be designed according to the state standard and had to be inspected by state inspectors. The school code required that earthquakes be considered and based its requirements on a slightly amended 1970 BOCA code. This meant the code incorporated virtually nothing except provisions for considering a force factor that varied with dead load and was distributed over the height of the buildings, as well as a few details for tieing the building together.

As a result of all of these developments and as a result of the fact that the Massachusetts Institute of Technology had received a National Science Foundation (NSF) grant to do a study for seismic design decision-analysis for Eastern metropolitan areas, the Boston Society of Civil Engineers formed a seismic committee in 1973. The program reflected the academic understanding of the problem now brought into the area of public policy. There was the realization that Boston had a history of earthquakes and was, presumably, in an active zone. Therefore, Boston could be subjected to a high level of shaking from earthquakes.

This awareness at MIT led to a proposal for a grant from NSF to reflect the fact that eastern metropolitan areas—Memphis, Boston, and Charleston—needed to consider public policy that reflected the very real potential of earthquakes in these areas. This proposal prompted other members of the Boston Society of Structural Engineers to start working on a seismic code, primarily for Boston, but a code that also responded to the entire state problem.

The Legislature had in the meantime passed a law creating a building code commission that provided for a mandatory state building code. So the Boston seismic committee was formed at a good time, for what they were developing ultimately came to represent mandatory state-wide requirements. Moreover, here was an eastern state that represented three seismic zones, ranging from the most severe to virtually no seismic requirements, that had embarked on developing a uniform code that contained seismic provisions. What finally came out in Massachusetts in 1975 was approximately equal to the UBC 1.5 zonation. The Massachusetts code, however, incorporated extensive ductility requirements without any specific masonry requirements.

1947	State Building Code promulgated by State Board of Standards (non mandatory, based on current version of BOCA).
1962	1962 Edition of the Building Code of the City of Boston. (no mention of earthquake consideration.)
1969	U.S. Geological Survey permits publication of revised seismic risk map of the United States prepared by S.T. Algermissen. The Algermissen map places Boston and the Northshore in Zone 3, most of the state in Zone 2, and the southwest corner in Zone 1.
1970	UBC incorporates Algermissen map. Buildings in Boston, Memphis, Charleston should meet same earthquake design requirements as buildings in Los Angeles and San Francisco.
1970	BOCA-Massachusetts Zone 2 with relatively minor earthquake provisions. However, possibility that BOCA could adopt new map. BOCA generally basis for Massachusetts codes.
July 1, 1970	Revised Building Code, City of Boston, published by Building Department of Boston. Accepts Zone 2 for Boston and by reference UBC 1967, Volume 1, Section 2314, "Earthquake Regulations." Requires only "capable of withstanding lateral forces," remains vague on the question of required ductility. First introduction of seismic requirements in Boston Building Code.
1971	First incorporation of seismic provisions in a state code. Schoolhouse Code required earthquakes to be considered. Based on slightly amended 1970 BOCA Code.
1973	Seismic Committee formed to advise the State Building Code Commission.
1975	Seismic provisions adopted in new mandatory statewide code. Approximately equal to UBC 1.5 zonation. Incorporated extensive ductility requirements. No specific masonry requirements.
1978	Revision process for code. Incorporation of masonry provisions. Question of several zones in state. Question of existing buildings.

MASSACHUSETTS HISTORY
Figure 6.4

DEVELOPMENT OF THE SEISMIC CODE IN MASSACHUSETTS

It is interesting to review the background behind the policy that occurred in the seismic code development from 1970 to 1978. The State Code Commission established as its first priority the protection of life safety. In the case of code requirements to resist seismic effects, it was necessary to establish a fundamental understanding of risk to apply to the concern of life safety. To start with, it seemed apparent that Boston should be distinguished somehow from Los Angeles in terms of implied risk and threat to life safety. Throughout the rest of the state, which fell into Zone 2, it was also necessary to obtain an understanding of what was implied by design requirements.

The question that naturally arose was whether the basis of design requirements and the zone designation, and consequently the total lateral force applied, was biased more towards economics rather than life safety. The economic issue represented a delicate problem. It was impossible to ignore the potential impact on construction costs that the provisions would have at a time when the construction industry in Massachusetts was severely depressed. It was necessary to ensure a defensible balance between what appeared mandatory for life safety and what was reasonable as an increased construction cost. It is important to note that the primary concern relative to economics was not what the threat of an earthquake represented in terms of regional impact or financial burden on the owner or the community subsequent to the event; it was the financial penalty imposed on construction and its effect on developing the economy.

In the area of seismic provisions, the BOCA code generally lags well behind the UBC code issued by the International Conference of Building Officials (ICBO), a California-based code organization. The UBC generally follows the recommendations of the Seismology Committee of the Structural Engineers Association of California (SEAOC). Thus, the 1970 BOCA earthquake provisions were not in keeping with the most recent SEAOC Seismology Committee recommendations.

In an early meeting of the Boston Seismic Committee, Professor Whitman of M.I.T. presented a review of the Seismic Design Decision Analysis study of seismic design considerations for high-rise buildings on firm ground in Boston. He observed that loss estimates developed in the Seismic Design Decision Analysis study suggested that expected dollar values of damage were so low that additional code requirements could probably not be justified on solely economic grounds. Reference was made to the SEAOC objectives of providing reasonable protection. These objectives are translated into three possible criteria as stated below:

- For the maximum likely earthquake: no damage
- For the maximum probable earthquake: no structural damage
- For the maximum possible earthquake: no building collapse

In the third meeting of the Committee in November 1973, the Commission stated that it did not feel that the 1970 BOCA seismic provisions were adequate but that it did not believe it appropriate to adopt the same provisions as California. It was felt the Commission would probably adopt what the Committee recommended, but it did not wish to adopt requirements that would seriously escalate the cost of construction in the state.

Professor Holley of M.I.T. provided a paper that suggested that the Commission might benefit from a statement on how to deal with the low level of seismic risk which typified Massachusetts. Professor Holley's approach focused on avoiding hazard to life rather than risk of damage to buildings. He suggested that the code should consider the following variables:

- geographical location within Massachusetts, as related to seismic risk;
- soil conditions at the site;
- function of the building; and
- density of building occupancy.

Discussion centered on the need for an independent approach to the problems presented in areas of relatively lower seismic activity. It was recognized that there was a need for an approach that could balance costs and risks more effectively.

A strategy had to be developed that would reduce risk at minimum cost. For it was evident from the loss estimates that because of the relatively long return periods of potentially damaging earthquakes in Massachusetts, dollar loss due to building damage would not be a major consideration of the Code.

Therefore, the directive to the Committee from the Code Commission was to develop measures to protect public health, safety, and welfare. Given the low level of expected average annual loss, the priority of the life safety consideration was established.

Eventually, the Committee as a whole came to two important conclusions. First, it was generally accepted that the return period for an event giving an intensity of MMI VII+ on firm ground in Boston would be on the order of 10,000 years. This level of shaking in Boston might correspond to a hypothetical recurrence of the 1755 Cape Ann earthquake with its epicenter in Boston Harbor. This order of event was accepted as a design basis earthquake for the purpose of code formulation.

The second consensus arrived at was on the question of acceptable risk. It was decided that the public probably would not accept collapse of more than one to three percent of the structures during this design earthquake. The understanding here was that collapse was associated with loss of life. The intention was not to guarantee against any possible loss of life; rather it was that a one to three percent degree of collapse would be acceptable. This, of course, applied only to new structures subject to the codes.

The Committee decided not to consider special structures in the initial version of the Code. It was decided that the Committee would limit itself to design criteria for general purposes structures because decisions on selection of particular structures, i.e., hospitals, places of public assembly, etc., would have to be the responsibility of the Commission and the Legislature.

The Committee decided to adopt provisions along the lines of the Uniform Building Code ductility provisions for Zone 2. The design static force coefficient was set at 3/8. This was later changed by the Commission to 1/3. These values corresponded to a design acceleration of about 0.12g. The impact of these added requirements on initial costs of construction was estimated to be at most on the order of one percent of total building cost.

On the basis of the seismic risk studies, it was possible to identify UBC 1.5 as the appropriate zonation for Massachusetts. The issue of variations in seismic risk across the state was subordinated to the interest of developing a uniform statewide code.

It was also decided by the Committee to include a soil factor as a multiplicative factor in the equation for base shear. (The 1973 UBC on which the Committee work was partially based did not include a soil factor.) The Committee decided that for the sake of simplicity it was reasonable to consider only two categories of soil—soft and firm. For some parts of Boston, with 120 feet of clay, the soil factor is 1.5, which means that the seismic coefficient approaches 1/2 rather than 1/3 as in the case of firm soil.

How to classify the state as a whole proved to be a policy problem. Meeting in 1978, the Seismic Committee at first considered the possibility of dividing the state into two zones, east and west, with a lower zone factor for the western part of the state instead of the three zones previously discussed. But in reviewing recent design requirements by the Nuclear Regulatory Commission for the area, the Committee established no significant difference between the western and eastern parts of the state.

There appeared, however, to be "ample reason to argue for a reduction in seismic requirements" for the western part of the State. Should this occur, a boundary would have to be established between the west and east along county lines. One logical possibility was Worcester County, although it would be necessary to decide whether to use the eastern or western boundary.

It was finally decided to leave the State as one seismic zone for several reasons:

- Uniformity of enforcement;
- The error inherent in settling an east-west boundary is so great that it is not meaningful; and
- The division into two zones may be a future necessity as a trade-off for incorporating requirements for reinforcing masonry through the State.

Except for high occupancy buildings in Boston, it was also decided that existing buildings should not be reviewed or evaluated. There is a need for a set of minimum standards to be used for buildings being renovated, although the standards may require lower resistance than that required for new construction. There is also the possibility of requiring the evaluation of buildings of high occupancy.

PUBLIC POLICY RESPONSE IN THE FEDERAL GOVERNMENT

The general problems pertaining to the potential threat from natural disasters were recognized in the disaster preparedness study published in January 1972 by the U.S. Office of Emergency Preparedness as provided in Public Law 91-606. A cooperative federal program was initiated in 1972 by the National Science Foundation (NSF) and the National Bureau of Standards (NBS). A National workshop on building practices for disaster mitigation to consider the effects of earthquakes, extreme winds, and other dynamic hazards was held by the National Bureau of Standards in Boulder, Colorado, in August 1972. The workshop was co-sponsored by the NSF's Research Applied to National Needs Program and the National Bureau of Standards. Recommendations were developed at this workshop for the implementation of improved practices in the design and construction of buildings at the professional and policy-making levels. The objectives of recommendations included minimizing human suffering, reducing property losses, and maintaining vital functions in buildings after a disaster. The recommendations proposed at the workshop were the first major result of the cooperative Federal program.

Among the significant recommendations was the development of seismic design provisions for buildings. In November 1974, the Applied Technology Council received a contract with NBS (funded by NSF's Research Applied to National Needs) to develop seismic design provisions applicable throughout the state-of-the-art in one place, and publishing it. The resulting document "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC-3) has been completed and is now being disseminated.

In June 1976, the President of the United States requested his science advisor to convene an advisory committee on earthquake prediction and hazard mitigation and to propose a program of priorities and of funding for NSF and the U.S. Geologic Survey. This step represented direct Federal involvement in the development of a federal earthquake program. This committee, the Newark Committee, developed a set of programs which were translated into funding for a three-year period. This reflects what, in essence, is the best thinking for developing provisions, controls, and a system for implementing seismic hazard mitigation through the Federal Government.

January 1972	Disaster Preparedness Study published by: U.S. Office of Emergency Preparedness as provided in Public Law 91-606.
1972	"Cooperative Federal Program" initiated by the National Science Foundation and National Bureau of Standards.
August 1972	"National Workshop on Building Practices for Disaster Mitigation" held at National Bureau of Standards, Boulder, Colorado sponsored by the National Science Foundation Research Applied to National Needs, and the National Bureau of Standards.
November 1974	Applied Technology Council Contract with the National Bureau of Standards, funded by the National Science Foundation to develop up-to-date comprehensive seismic design provisions applicable throughout U.S.
June 1976	"Newmark" committee meets. Advisory Group on Earthquake Prediction and Hazard Mitigation. Proposed program of priorities and funding for National Science Foundation and U.S.G.S. Congress approves increased funding for Earthquake Program.
April 1978	ATC-3 Seismic Document complete and in preparation for dissemination.

FEDERAL GOVERNMENT INFLUENCES
Figure 6.5

PROFESSIONAL PRACTICE IMPLICATIONS

There is no doubt that as seismic design requirements as well as a more comprehensive regulatory system are developed, the legal profession will start looking towards who is responsible for ensuring that those standards are met. Also, once a policy-making body, such as the code commissions, establishes risks as public policy and issues regulations for mitigating the risks, there are responsibilities implied for regulatory officials and for design professionals. At present, no one has come up with an analysis or evaluation of the implied liability of design professionals or what the potential is.

One of the problems faced by a building official is that many codes say that the official shall be held immune from liability, which really only says that the building code official can not be held liable by the city or by the state. But, of course, anyone can bring him into court and sue him personally for implied liability or for implied malfeasance. There is nothing that can protect him from that, nor any design professional either. In other words, it will be the courts that finally decide the issue of professional liability.

Chapter 7. PUBLIC POLICY FOR SEISMIC SAFETY - WESTERN U.S.

Robert A. Olsen, P.E.

INTRODUCTION

Public policy is the body of formal and informal laws, rules, regulations, and procedures that govern the approaches taken by government to serve the public interest. This is a very broad definition, difficult to put into operational terms. For instance, what is the "public interest" or is there a "public interest?" Since the discipline was founded, debates rage in political science about that issue alone.

Further, you do not have to have a formal program to call it public policy. Policy can include action or inaction, informal pressure, decisions that are made or not made, that reflect an attitude or an approach that has an important bearing on what happens. There are these dimensions of public policy. I want to emphasize that when we discuss public policy, we are discussing formal and informal systems.

The formal is visible legislation, agencies, programs, etc. The informal system includes decisions made by administrators, friendly comments made to congressmen and legislators about what ought to be and is not, attitudes taken by those people responsible for administering the program when they communicate whether or not something will be done or not done, supported or not supported.

Both formal and informal systems have a single focus—power, political power. Thus, public policy is, to a large degree, the arena of politics. Here, the matter under review is the politics of earthquake hazard reduction.

It needs to be emphasized at the outset that the policy-making process is very complex. It consists of administrative hierarchies, interest groups, lobbyists, legislators, committees, staff, public interest groups, and advocates of all sorts.

In this paper I want to

- give a brief historical overview of earthquake safety or seismic policy;
- discuss principles of earthquake hazard reduction programs;
- summarize forces that seem to facilitate or impede the development of public policy and comment about some model organizations, or at least offer some things to think about in terms of organizing for earthquake hazard reduction; and
- close with a summary of what I think are some current policy issues and some future trends that are going to affect earthquake hazards reduction in the foreseeable future.

HISTORICAL OVERVIEW OF SEISMIC SAFETY POLICY

Traditionally, articles on the history of seismic safety policy have been historical in the sense that they generally followed a chronological sequence of what has happened. For example, 1925 was the year of the Santa Barbara earthquake. In the State of California, that event launched the beginning of the local codes for earthquake resistant design. In 1933, the Field Act was passed after the Long Beach earthquake in which a number of public schools were damaged. This act has since governed the construction of public elementary and secondary schools in the State.

In 1971 there was the San Fernando earthquake following which there was a cluster of policy developments; among a number of other measures, a law was passed regulating hospital construction, a state-wide strong motion instrumentation program was created, and an act was passed that called geological studies in some of the major fault zones.

However, some earthquakes have not been followed by major public policy changes or developments. There was no significant change in terms of public policy in this State after the 1906 earthquake in San Francisco nor the 1952 earthquake in Bakersfield. Nor did the Santa Rosa earthquake in 1969 really lead to any basic changes in policy for earthquake hazard reduction.

Therefore, I think we cannot always associate earthquakes with cause and effect in policy, but must rather try to identify those other things that were present at the time or in the process that were influential either in promoting policy or stopping proposals.

FORCES THAT FACILITATE OR IMPEDE POLICY CHANGES

What, then, facilitates the development of public policy for earthquake hazard reduction? Obviously, earthquakes help. They should be treated as opportunities to innovate, to catch attention, and to present proposals. Earthquakes certainly do play a role in reminding decision-makers, policy-makers, and advocates that there is a hazard, there is a problem. For a short time at least you have their attention. For those of us interested in improving earthquake hazard reduction, the earthquake is an opportunity to do something.

Secondly, there is the presence of advocate organizations and opinion leaders. There are certain interested and informed people in the legislatures who are looked-on as the in-house experts. Advocate organizations and opinion leaders have to be present to speak out on the issues, take positions, and represent themselves in the game of power.

In addition, there has been an increasing concern about seismic and geologic hazards that has come through the whole environmental improvement and environmental quality movement. These subjects have to be dealt with now in official impact statements and in all kinds of other processes.

Also, we cannot ignore the increased financial and human resources that have been made available in the area of earthquake research and earthquake hazard reduction over the last ten or fifteen years. We have witnessed increases at the Federal level and certainly there is new legislation; but even more than that, there has been a general growth in terms of talent and investment. We have seen curricula and seminars and institutes put on to help professionals and to train students to become aware of the earthquake problem as they go into their professional practice.

Another factor that facilitates earthquake hazard reduction policy has been a generally increased awareness of the hazard and the publicity surrounding it. This involves a presentation of information, the acquisition of information, through newspaper stories, publicity, and television. When you talk to people, they may have a lot of things mixed up about earthquakes, but they are far more aware than they ever have been.

One of the big factors in the increased awareness has been the rapidity with which we can receive information today about damaging earthquakes in other countries. Devastating earthquakes in Guatemala and Nicaragua are on our TV news within a matter of hours, and that makes these places seem very close. There has also been a lot of interest in the earthquake prediction research. The hope of developing an ability to successfully predict earthquakes has loomed large in the public's mind. Whether or not research in this area leads anywhere, the issue and the desire to be able to accurately predict earthquakes is another factor in the development of earthquake hazard reduction programs.

What, then, impedes the development of public policy? Other than long periods when there are no significant earthquakes and the attention of policy-makers drops off; there is the question of conflicting priorities. Local government is faced with lots of issues—finances, parks, libraries, fire and police services, tax problems. Consequently, the problems of earthquake safety along with a lot of other long-range issues do not receive the attention that we would like them to. They are not salient in the decision-making environment that those groups have to operate in.

Another problem we have is an inability to define the threat in very precise terms. We know that California has earthquakes. We know that generally the seismicity is more in the coastal area, less in the inland valley and in the Sierra foothills. We know that we can expect more in the future. But can we say to a city council exactly what they might face, when the next earthquake will be, and just exactly what the scope of damage will be? Often our inability to give them precise information that they can deal with in their jurisdiction and in an immediate sense militates against initiating constructive action. You have to give policy makers something more precise, definitive, and acceptable to the decision-making process.

There is yet another problem. It is difficult to show legislatures how effective some of the hazard reduction programs are. We can go out and do assessments, evaluate them, and talk about the processes; but it takes decades to show whether a given program is really effective or not. It has only been in the last couple of decades that the Field Act governing school construction has really demonstrated its wisdom. We have had enough damaging earthquakes and enough ability to compare schools built under the Field Act to the buildings that were built beforehand to show that they do perform significantly different and that the Field Act was a good investment. But that has taken decades. So we really have a young program and a young process.

POLICY PRINCIPLES AND EARTHQUAKE PROGRAMS

When we talk about basic principles of public policy, we are talking about the fundamental question of "why?" Why was something done? What was the belief or the value that was represented in the various proposals? Subsequent to enactment or implementation of given programs, there is always some controversy, depending upon the complexity of the program. Yet most of these difficulties that come afterward involve important, but primarily operational questions, such as what standards will be used, what kind of administrative process is going to be followed, what are the rules and regulations, and how is the game played? The philosophical question of what ought to be done or what ought to be required slips into the background.

The Field Act of 1933 is a kind of a landmark in the State of California because it represents the first State level enactment of public policy to cope with earthquake hazards. The Field Act was enacted because numerous elementary and secondary public schools failed in Long Beach during the earthquake. The Act has since regulated the construction of all elementary and secondary schools in the State. Plans developed for a given school have to be reviewed and approved by the Office of the State Architect.

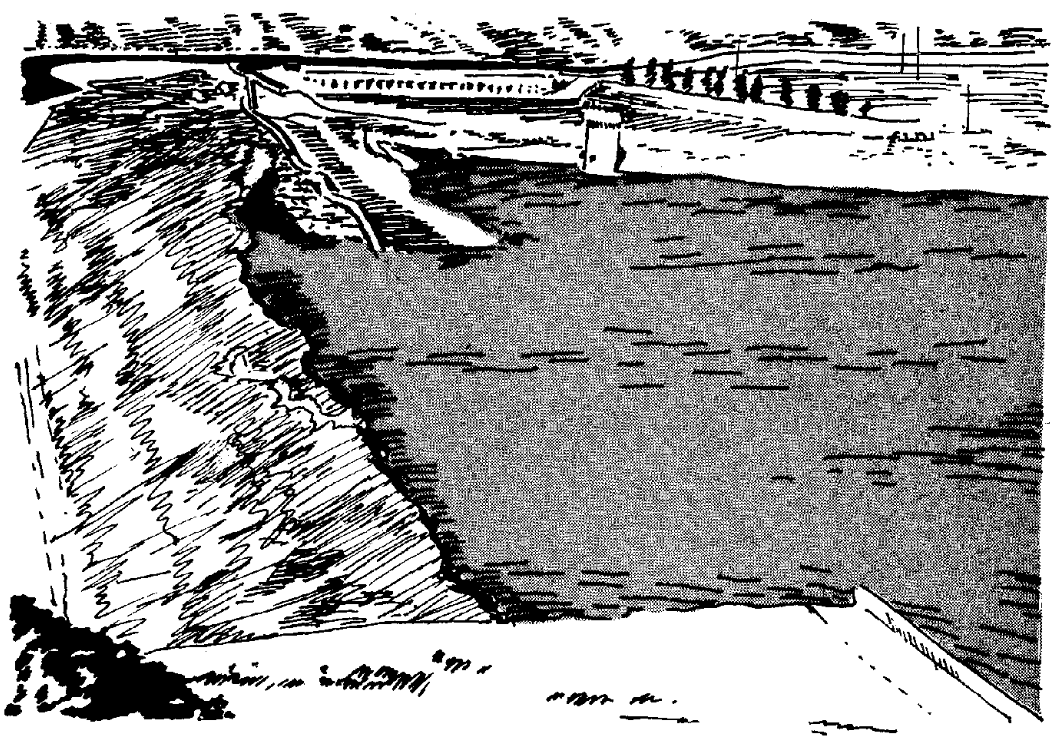
COLLAPSED SCHOOL BUILDING
HAMILTON SCHOOL - 1933 LONG BEACH EARTHQUAKE
Figure 7.1

Why were some of these things done? Of course there was the damage; but there was the greater concern about a special population. Children are special. We did not want to expose them to this kind of failure. The Act preempted local authority for the first time; it took away from local government the authority to review plans for schools. The local jurisdiction, the city, or the county in which that school happens to be located no longer has a real role to play in that process.

A similar pattern was followed in 1972 following the San Fernando earthquake. The Hospital Seismic Safety Act of 1972 was enacted using the Field Act as its model. The Hospital Act required that standards be established by the State; plans for these facilities had to be reviewed and approved by the State. This Act was a little different from the earlier Field Act. It applied only to new hospitals, not to existing ones. It also included a principle of functionality or damage control. The legislation said that the hospital shall also remain fully functional after an earthquake. Now that does not just mean the structural system: the law read that it shall be fully functional, i.e., capable of performing an emergency medical service for the population. So we had a principle here of hospital regulation, State pre-emption, and the principle of damage control to help guarantee the survival of that facility. The underlying theme is that hospitals are a critical community resource. Since they probably are more important after a disaster, they should have a higher degree or a higher probability of survival.

LOSS OF CRITICAL EQUIPMENT
OLIVE VIEW HOSPITAL - 1971 SAN FERNANDO EARTHQUAKE
Figure 7.2

Another program enacted after 1972 was a requirement to develop inundation maps for areas below dams. The City of Los Angeles police department had maps showing the areas that might be inundated and the areas that would have to be sealed off. When the Van Norman Dam was threatened during the 1971 San Fernando earthquake, public officials were able to move quickly in an orderly way to evacuate that area. Tens of thousands of people were able to be safely moved out of the way. This led to a proposal that we ought to require something similar for all jurisdictions in the State and that maps ought to be prepared by the dam owners who were given certain criteria by the State government.



DAM FAILURE
VAN NORMAN DAM - 1971 SAN FERNANDO EARTHQUAKE
Figure 7.3

Although we cannot assume 100 percent safety for any kind of facility, we can plan for a potential failure, so that the local jurisdictions, and particularly emergency service functions within those jurisdictions, will have some idea of the potential effects of an earthquake. The principle is very simple: fail safe does not exist; we cannot be earthquake-proof.

Another program that relates to land use is a requirement in the State of California that every local jurisdiction have a seismic safety element in its general plan or city/county plan. The State does not say that the jurisdiction has to do anything or that they cannot issue a permit. But it does say that earthquake geologic hazards are just as significant as transportation and housing; it does say that in the planning process information ought to be developed, so, when the local decision-making bodies (the planning commission, the city councils) adopt a general plan, they have evaluated the hazard problem along with the other concerns that they must deal with. The principle in that program was that we hope that the land-use decision-making process could be improved by making information available.

One of the more controversial programs in California is the Special Study Zones program. That program requires geologic reports to be done for buildings built in some of the major fault zones in the State of California, and then it allows the State Geologist to add others as deemed necessary. It does not say that you cannot build in a fault zone. All it does is tell the local jurisdiction that they need to have a geologic report if a proposal is made to build in a fault zone, and that the findings of that report have to be considered along with everything else in issuing the permit. The burden is still on the local jurisdiction.

There are problems with this program. We found, for example, that local jurisdictions have gone beyond the basic law in their own ordinances; they apply it differently throughout the State. This has caused some real problems. There have been cases, for example, where the local jurisdiction has denied permits for single-family dwellings or even rooms to be added on.

At this point, I will take the same set of examples and turn them around to put them in a different context, to look at the intergovernmental relationship and the administration of the programs. There are programs related to earthquake hazard reduction in this State that are locally developed and locally administered. Some local jurisdictions have adopted their own hillside grading and siting ordinances. Others have ordinances dealing with the anchoring, or strengthening, or removal of parapets and other ornamentation on buildings. These are locally administered programs.

There are, in addition, a group of programs that are mandated by the State but which are administered by local government. The seismic safety element in general plans is one of them. The local jurisdictions are the ones that do the planning, adopt the plans, and ultimately make the land use decisions. The Special Study Zones is another example where there is a little more interaction, but it still is primarily a local responsibility. The State says, "We have major fault zones. For these types of construction, you will have to have geologic reports;" but it does not say that you cannot issue that permit. Local jurisdictions have to have that geologic information available and evaluate it, but then they can do as they see fit, although there is a State appeals process.

Besides these and the group of programs that I have talked about that are mandated and administered by the State (the school construction program under the Field Act, the hospital construction program, and professional licensing), there are other programs that we do not have any control over here in California, that for all practical purposes are the province of the National Government, either through pre-emption or through ownership. The Federal Government has largely pre-empted nuclear powerplant siting and design. Certainly, Federal dams, Federal office buildings, and other Federally-owned facilities are exempt from any state and local control.

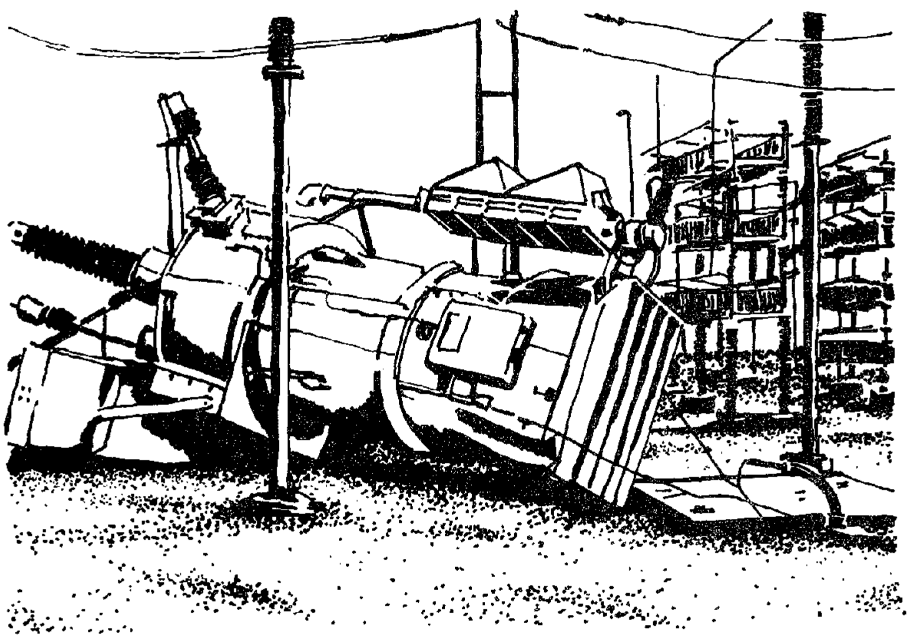
We have earthquake hazard reduction problems associated with National Government activities that are beyond state and local control. Thus, we can look at programs from several viewpoints: locally developed and administered, state mandated and locally administered, mandated by the state and administered by the state, and mandated and administered by the Federal government.

Another way of looking at the question of public policy is what kinds of decisions are going to be affected. We can take the same group of programs again and say that certain of the programs are going to affect land use decisions—that includes the seismic safety elements, the special studies zones, local siting and grading ordinances, permits by the coastal commission for construction in the coastal zone, or around San Francisco Bay by the Bay Conservation and Development Commission which issues permits for construction. We can talk about the kinds of decisions that affect building systems. There we can look at the Uniform Building Code, or the Field Act for schools, or the Hospital Act, elevator regulations by the State Department of Industrial Safety, and others.

The area of lifeline facilities, or lifelines and critical facilities—such as nuclear plants, dams, liquefied natural gas terminals—and the broad area of lifelines—sewage, water, electricity, telephone, etc.—is a little more ambiguous. Decisions affecting the construction of those seem to be a little less organized. Federal or state dams are pretty effectively controlled. However, there is no real check on the gas and the water utilities in regard to earthquake-resistant design and construction—which does not mean to say that the utilities do not do a good job. However, there is no independent review and there is no consistency.

On the other hand, some of our biggest utilities are governmentally-owned. Cities and counties, for instance, own water districts. They supply a tremendous amount of the service yet they are under no review at all. The Public Utilities Commission regulates private industry, but it does not regulate the same service supplied by a governmental agency.

In short, the area of lifeline and critical facilities is one subject area that requires some looking at. I am not too comfortable with what I have seen in that area. It varies tremendously.



DISRUPTION OF LIFELINES
SYLMAR CONVERTER STATION - 1971 SAN FERNANDO EARTHQUAKE
Figure 7.4

PUBLIC POLICY BODIES IN SEISMIC SAFETY

We have witnessed recently in the States of California, Utah, Montana, and Nevada, the creation of some kind of institution or group to do something about earthquake hazard reduction. These groups vary tremendously. In California, we have a Commission established by statute. In Utah, we have a similar group, created again by legislation. In Montana, there has been a Governor's committee formed by the current administration. Nevada has appointed a committee that will meet to take a look at what is going on in the state and local programs to see what Nevada should be doing. The approaches vary, but, generally, the existence of those groups indicates a growing recognition that earthquake hazard reduction is a legitimate area of concern and that there is need to institutionalize some way to give attention to this. There is also work currently being done by The Council of State Governments for states and other kinds of grant programs that are going to policy-oriented organizations. At the national level we have a proposal that the new National Earthquake Hazards Reduction Program include a lead agency to provide coordination, oversight, stimulus, and review at the national level of a comprehensive earthquake hazard reduction program.

Like California's Commission, the existing state groups are policy-oriented. We do not go out in the field and crack rocks or build buildings or review plans or anything like that; we try, instead, to identify problems. We try to assess the general scope and direction of programs. We try, also, to assess the effectiveness of existing programs and provide a means for oversight and coordination. I see these groups—our commission, the council in Utah, the committee in Nevada, and the committee in Montana—as starting to fulfill some of these functions.

There will be a new focus at the Federal level for the same kind of thing. It is too early to tell just exactly where it is going to be, but the implementation plan that was required by Congress in the Hazards Reduction Act recommends some lead agency or some focus to provide the breadth-of-view and coordination, because a lot of the activity—especially in the State of California—is divided up among several agencies, i.e., the Division of Mines and Geology, the State Architect's Office, the Departments of Health or Transportation to name a few. In the State of California, these agencies and their earthquake programs consume nearly $17 million a year in funds, but their individual programs are very specialized. That does not mean, however, that every problem is being dealt with. So some of these oversight functions will be concerned with what ought to be done, what problems are not being addressed, and what we can do about them.

In California, the Commission is a broad mix of professions and backgrounds. We have architects, engineers (structural, mechanical, soils), geologists, planners, persons from local government, social services, disaster services, fire-suppression, and seismology, as well as others. The idea was to get a broad multidisciplinary group together whose focus was the earthquake problem, knowing that we have to take a broad look since there would be a lot of different problems involved. This Commission is advisory, we do not write regulations, we do not issue permits, we do not have any operating programs. We do review existing programs and go to the Legislature with what we think should be done to change the laws, or we go to the agencies to try to get them to change their regulations if we think the programs need to be improved. In addition, we provide some means of interagency coordination on particular projects and programs.

Generally speaking, we use committees on some projects and we bring in a variety of people who are not on the Commission. These task-oriented committees sometimes meet twice and then go out of existence. But sometimes, as in the case of our Hazardous Building Committee, the group has been in place since we started, and I have a feeling it may become one of our two permanent committees. However, all projects are designed to have a product—a report, a recommendation, a piece of legislation.

What kinds of policy concerns are we currently involved in? One is the independent review of major hazardous or critical facilities. We are trying, for example, to develop a philosophy that will eventually lead to requiring independent review of dams so that there is some outside check or review of the siting, design and construction.

We are interested in the problem of hazardous buildings. We consider the greatest earthquake hazard in California to be the unreinforced old masonry buildings. They are not very glamorous, but there sure are a lot of them, and they do not perform very well during earthquakes. We are not going to solve that problem today, this week, or even in years; but maybe we can do a number of things as opportunities present themselves. We have supported an ordinance in Los Angeles, for example, that would have started dealing with the problem there. We are sponsoring legislation next year that will give local government the authority to adopt less-than-current standards; so that if the renovation threshold is crossed, the owner will not have to bring the building all the way up to current code, but will just have to meet a basic life-safety code that prevailed in California until about 1965. Since 1965, the code has become more and more complicated. As a result, it has become more difficult economically and technically to do something with these old buildings. In the meantime, we are involved in a review of earthquake disaster preparedness in the state. This means talking to local government people and collecting data about what local decision-making board supervisors, city managers, and others would like to see in an emergency preparedness program.

We have also been involved in looking at disaster preparedness education. We have gone to the schools, talked to teachers and administrators and asked what they need to build into their curricula to teach young children and high school kids something about earthquakes and self-protection. Partly as a result of our findings, we may try to find some money to fund some curriculum materials for elementary and secondary school teachers for use in the classroom. Related earthquake and other safety issues can be incorporated into the existing curriculum instead of creating a separate course of taking the teachers out two or three days and giving them a crash course on earthquakes.

We are doing an interesting study on the earthquake hazard of state-owned buildings. A couple of years ago, the University of California requested almost $150 million to strengthen buildings on all their campuses in the state. The Department of Finance and Legislative Analyst came to us and asked how they should decide the priority. If they had $100 million, which buildings should get the money?

There was another related problem. If the University got $150 million, what would the State colleges want, and then what is the Department of Health going to want for hospitals, and what is General Services going to want for office buildings, and the prison system? Where will it stop? They could see billions out here, so they asked us to develop a system to come up with a way of assigning priorities. We are almost done with it, and we are coming up with a relatively simple formula that is designed to be a survey. It is not designed to tell whether this building will perform well or not; rather, it takes a building and all the others like it in a class or all the classes of buildings and evaluates their performance in past earthquakes. This survey then takes a look at what they are being used for, their occupancy on a 24-hour basis, the square footage, the age, the type of construction, the general soil conditions in the area, and the location within the state. Working with a couple of structural engineers, we go through a calculation that gives a number or coefficient. Those buildings with a high coefficient indicate that, generally, there is a high hazard. Then you can rank the buildings, and you can put these vastly different buildings into a common framework.

CURRENT POLICY ISSUES AND FUTURE TRENDS

I am going to finish with a quote from an article in the July, 1976 Smithsonian Magazine. It captures what I think is going to happen.

> During the next quarter of a century, research will thrive in terms of expenditures, scientists and technicians at work, ambitious projects, and costly facilities. But it will do so as a tightly reined captive of the surrounding society, and in an atmosphere of caution and thrift. Most important, opportunities for scientific research will no longer be judged in the traditional terms of knowledge being a desirable goal in itself regardless of the purposes to which it might be applied. Scientists and research administrators will find that whole regions of research and technological application have been declared off limits, shunned as economically unworthy of pursuit or legally prohibited as too dangerous to be entrusted to frail humans and their unpredictable social and political institutions. Research in many, if not most fields will be actively encouraged, because the thickening problems of mankind on a crowded globe will demand scientific and technological solutions. But the choice of what to research will not be left entirely to the people who perform the research. Rather, it will receive the scrutiny and require the approval of a wide-ranging assortment of non-scientists who are concerned with cost, priorities, social impact, public safety, and political significance.

This characterizes what is going on today and the environment that will govern research. One of the issues that I see coming up is the nature of federal, state and local roles in earthquake hazard reduction and research. There is increasing participation by the Federal Government. Simultaneously, there is increasing interest within local governments, and some states are increasing their activities as well. There will have to be negotiation, if you will, among the federal, state, and local roles in this business.

There is another policy problem—the implementation of research that has been done and that which is currently being done. Other specific subjects that are going to be around awhile are critical and hazardous facilities, old buildings, nonstructural damage (the architectural, mechanical, electrical engineering problems), problems of independent review, earthquake prediction research, fire following an earthquake (especially in tall buildings), earthquake and disaster insurance, and the problems of economics and risk.

Looking back over the last few years, I have observed that earthquake hazard reduction has become a legitimate area of concern as reflected in additional funds, more research, and the institutionalization of programs and approaches. We have seen a growth in multidisciplinary involvement. It used to be purely the province of seismology, geology, and a few engineers. Now we have more engineers, architects, and others involved. There has been an encouraging growth and awareness of specific hazard reduction measures, and there has been a continuing institutional commitment at all levels of government.

Chapter 8. SEISMIC DESIGN CODES AND PROVISIONS

Roland L. Sharp

INTRODUCTION TO SEISMIC CODES

Seismic codes in the United States were not written until the late 1920's, although the San Francisco earthquake of 1906 did considerable damage. San Francisco was rebuilt on the basis of a 30 pound per square foot wind loading. We did not know much about earthquakes and the way structures respond to them. Little was done to respond to the danger until—the Santa Barabara earthquake of 1925. From that came the Uniform Building Code in 1927 which spelled out a code coefficient $C = 0.075 \times$ the weight of the structure. Other codes followed that requirement closely. For example, after the 1933 Long Beach earthquake, Los Angeles came up with a C factor of 0.08, roughly the same as the Uniform Building Code.

But further work was being done by engineers in Los Angeles and San Francisco. In 1943, Los Angeles came out with the first code that started to deal indirectly with the period of vibration by using a formula $C = 60/N+4.5$, where N was the story height. It was realized that the response of a structure to earthquake motions was a function of its dynamic properties or period. A limit of 13 stories was also put on buildings.

Later the American Society of Civil Engineers (ASCE) and the Structural Engineers Association of Northern California (SEAONC) formed a joint working group and in 1952 published the first seismic recommendation where the period of the building was explicitly introduced; and K was a constant that varied depending on the type of building.

However, they realized that since earthquakes are dynamic phenomena and period is just one measure of the dynamic characteristics of the structure, more was needed. Therefore, in 1957 the Structural Engineers Association of California (SEAOC) appointed a Lateral Forces Committee. The name was later changed to the Seismology Committee. After two years work, the committee came up with the Recommended Lateral Force Requirements for Buildings, the so-called "Blue Book."

The Blue Book has since become the basis for most building codes in many countries of the world. It has been extensively worked on, studied, and modified since then, the latest edition appearing in 1974, which ended up with a rather long formula — $V=ZIKCSW$. Z is the zoning factor, I the importance factor, K the factor dealing with the type of material and structural framing, and S the soil factor which is related to the period of the structure divided by the period of the site. W is the weight of the structure, that is, the gravity or dead load of the structure (plus for warehouses about 25 percent of the live load). The 1976 Uniform Building Code adopted almost verbatim the 1974 SEAOC recommendations.

In 1970, SEAOC organized a committee to look at the "Blue Book" and earthquake codes in general. This committee recommended that a group be put together to make an extensive survey of existing design practices, research data, and codes. This was in recognition of the fact the "Blue Book" is a limited document that deals with the structure only. The report, published in the Proceedings of the

American Society of Civil Engineers, provided impetus for the ATC-3 Project. In 1972, a number of engineers consulted with the National Science Foundation and others, trying to see if federal support could be obtained for an extensive study of seismic design provisions.

In 1973 the National Science Foundation granted initial planning money. A group was formed of some 20 people from throughout the United States. This group developed a program, a budget, and an optimistic prediction—that it could be done in two-and-a-half years. It took a little over three. We ended up with 85 participants representing structural engineers, mechanical and electrical engineers, architects, code officials, representatives from governmental agencies, and a number of professors and researchers from various universities throughout the United States. The resulting document is the present "Tentative Provisions For the Development of Seismic Regulations For Buildings" (ATC-3-06).

POST 1906 San Francisco rebuilt to 30 PSF wind

1927 Uniform building code ($C = 0.075$ to 0.10)

1933 Los Angeles City code ($C = 0.08$)

1943 Los Angles City code ($C = \frac{60}{N + 4.5}$, $N \leqslant 13$ stories)

1952 ASCE-SEAOC ($C = \frac{K}{T}$, $K = 0.015 - 0.025$)

1959 SEAOC $V = KCW$, $C = \frac{0.05}{3\sqrt{T}}$

1974 SEAOC, 1976 UBC: $V = ZIKCSW$, $C = \frac{1}{15\sqrt{T}}$

1978 ATC-3 Tentative recommendations

SEISMIC DESIGN CODES IN THE UNITED STATES
Figure 8.1

INTRODUCTION TO ATC-3

There was early concern that the UBC and SEAOC provisions were being misused. They are intended for buildings, but for want of an appropriate code they have been used for off-shore structures, towers, piers, and other non-building structures. Having raised this caveat, I would like to pursue the matter of seismic codes and a general discussion of ATC-3. The reader should refer to ATC-3-06 "Tentative Provisions for Development of Seismic Regulations for Buildings" for detailed provisions and commentary.

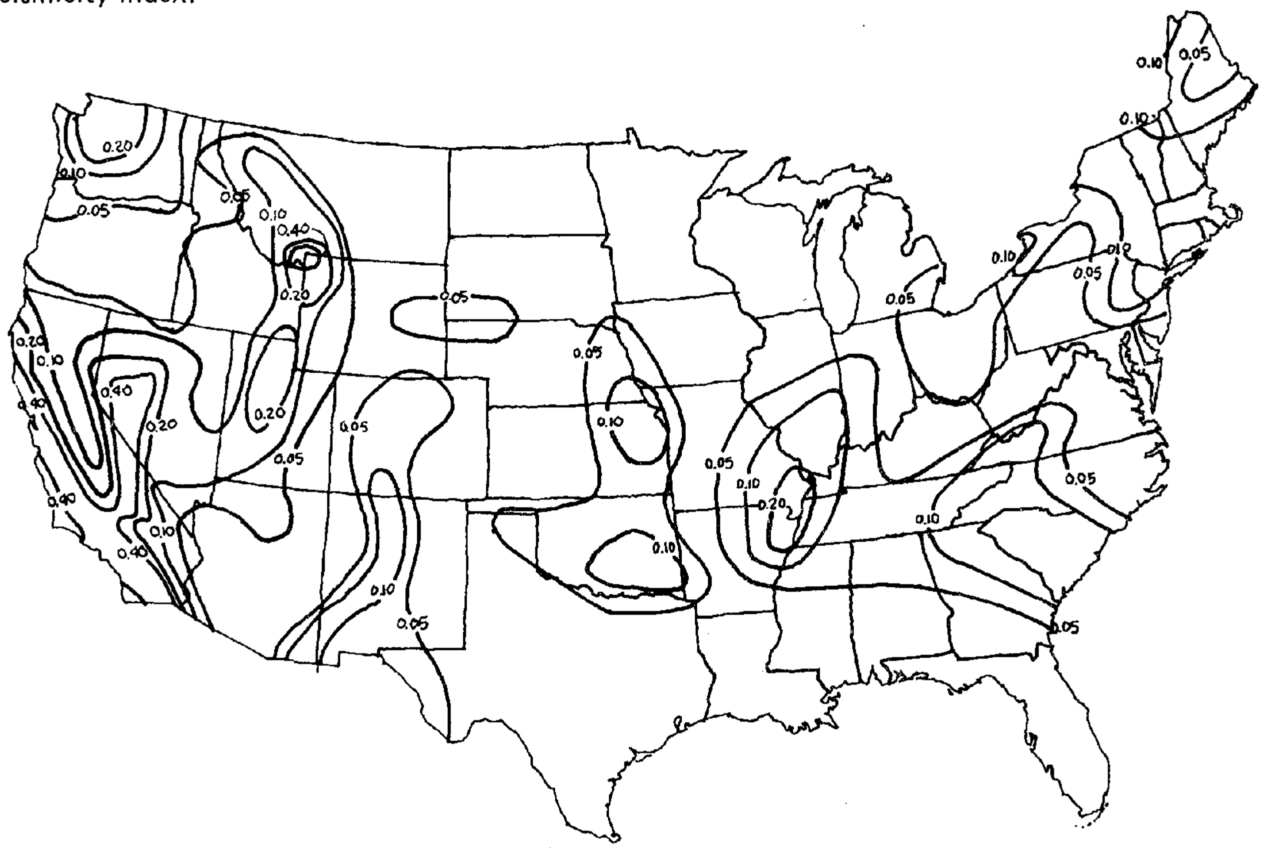
Present codes are based on certain seismic design values that are not always realistic. We have found, for example, that the ground motion intensities during earthquakes induce responses in structures which are often much greater than the C factors currently used in design. Another factor to consider especially with tall buildings, is the effect of distant earthquakes. If one starts with realistic ground motions and the values are, two, three, or four times present code values, there should be some kind of response modification coefficient to account for energy absorption in the building, inelastic yielding and other factors.

The response modification coefficients described in ATC-3 are somewhat similar to the K factor used in the UBC code. Except, rather than as multipliers, they are used to divide the ground motion. The interest is to provide a more realistic concept of what happens in an earthquake. By taking into account the type of structure, the materials that are used, and the design detailing, the seismic design force can be reduced because the structure, including nonstructural components, will be able to resist seismic forces in excess of that for which it was designed. It might sustain damage, but it will not collapse. In order to make this applicable to the entire United States, we decided in ATC-3 that the analysis and design should be dependent on the seismicity index, seismic hazard exposure group and seismic performance categories. The term seismicity index was used rather than zone in order to minimize confusion between the UBC, ATC-3.

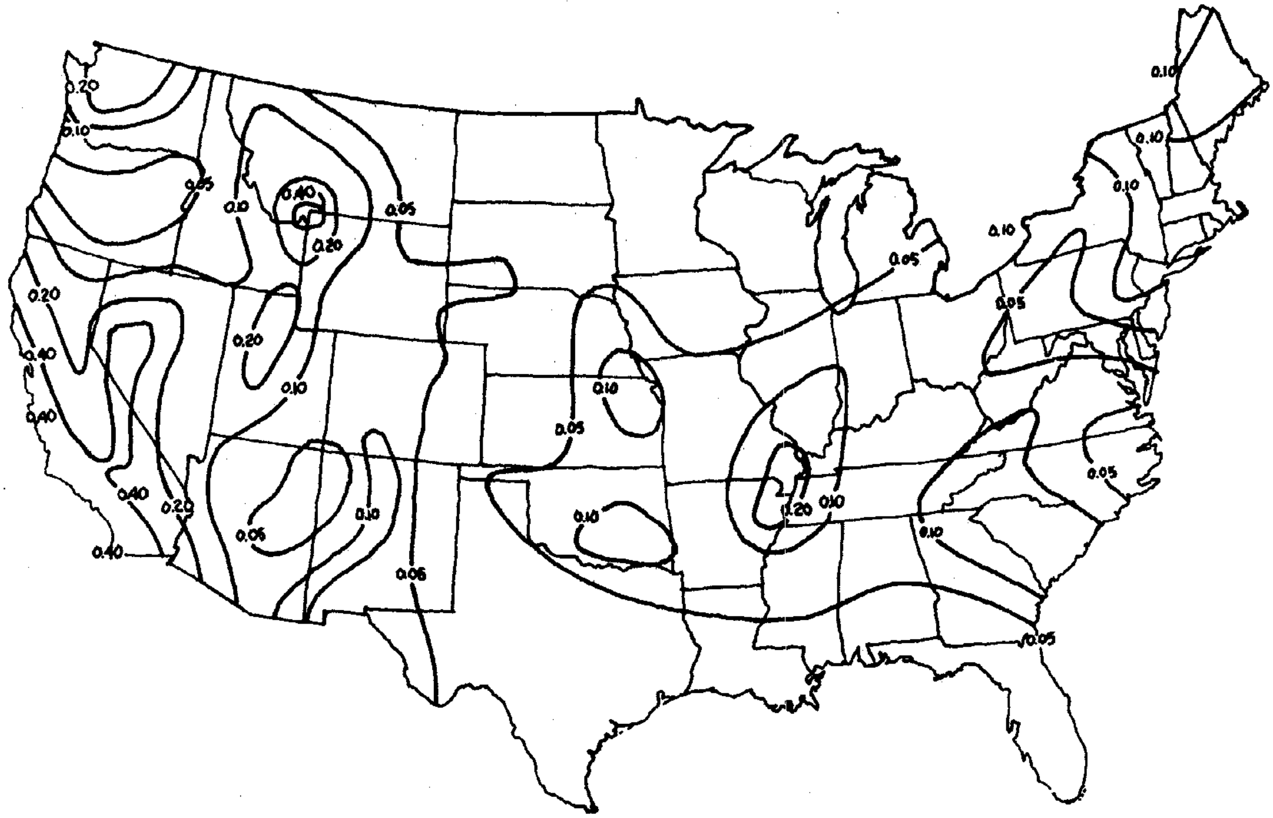
Another important factor relates to the use of the building, or the seismic hazard exposure group. Based on the seismicity index and the use of the building, seismic performance categories were developed for buildings. We decided not to go along with the UBC code because we concluded that increasing the force factor does not necessarily result in better building. For more important buildings, therefore, we require better detailing, in order to get better performance in an earthquake rather than increasing the force factor by 25 to 50 percent. Non-structural systems and components are also covered fairly extensively in the document.

SEISMIC COEFFICIENTS

The design steps in ATC-3 are fairly simple. First, locate the site on the County-by-County Map (Figures 8.2 and 8.3 show the contour maps from which the County-by-County maps were developed. Next, find the area number on the map and go to Table 1-B, shown in Figure 8.4. If Effective Peak Acceleration (A_a) is to be used, the coefficient is selected from the table and varies from 0.40 to 0.05. Effective Peak Velocity-Related Acceleration (A_v) which is used for most cases is similar to A_a, except that the areas are different on the maps. The seismicity index is selected from Table 1-B. Table 1-A is then used to determine the seismic performance category based on the hazard exposure group and the seismicity index.



CONTOUR MAP FOR EFFECTIVE PEAK ACCELERATION (Aa) COEFFICIENT SHOWN AS PERCENT OF GRAVITY
Figure 8.2



CONTOUR MAP FOR EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION COEFFICIENT (A_v) SHOWN AS PERCENT OF GRAVITY
Figure 8.3

TABLE 1-A

SEISMIC PERFORMANCE CATEGORY

Seismicity Index	SEISMIC HAZARD EXPOSURE GROUP III	II	I
4	D	C	C
3	C	C	B
2	B	B	B
1	A	A	A

TABLE 1-B

COEFFICIENTS A_a AND A_v AND SEISMICITY INDEX

Coeff. A_a Figure 1	Map Area Number	Coeff. A_v Figure 2	Seismicity Index
0.40	7	0.40	4
0.30	6	0.30	4
0.20	5	0.20	4
0.15	4	0.15	3
0.10	3	0.10	2
0.05	2	0.05	2
0.05	1	0.05	1

Figure 8.4

SEISMIC HAZARD EXPOSURE GROUP

Seismic hazard exposure groups are shown in Figure 8.5. Group III exposures are essential facilities, Group II are those with high density or where the occupants are limited in their movement and Group I includes all other buildings.

Group III

Essential facilities necessary for post-earthquake recovery with capacity to function during and after earthquake

Examples:

Fire suppression facilities
Police facilities
Medical facilities having surgery and emergency treatment areas
Power stations or other utilities required as emergency back-up facilities

Group II

Large number of occupants or occupant movements restricted or mobility impaired

Examples:

Public assembly for 100 or more
Open air stands for 2,000 or more
Day care centers, schools, colleges
Retail stores - 5000 sq ft/floor or over 35 ft in height
Shopping centers with covered malls, over 30,000 sq ft area excluding parking
Offices over 4 stories or over 10,000 sq ft/floor
Hotels or apartments over 4 stories
Wholesale stores, printing plants, or factories over 4 stories
Hazardous occupa~~ni~~esc - flammable or toxic liquids

Group I

Buildings not classified in group III or II

Multiple Use

Multiple use buildings shall be assigned to highest S.H.E. Group which occupies 15% or more of total area

SEISMIC HAZARD EXPOSURE GROUPS
Figure 8.5

The A, B, C, and D in Figure 8.4 denote a building's seismic performance category. A, in essence, is no design except for a few minor details or connections. B is characterized by collector elements and some seismic design, but not a full design. C is equivalent to the present UBC for zone 3, and D is for essential facilities in high intensity zones. D is roughly equivalent to hospital regulations in California. What this means is that for seismicity index 1—which would cover a large part of the United States—most buildings would need only minor seismic design. An essential facility, however, should have some seismic design. In the next higher intensity area or index 2, a minimum amount of design is specified but not the kind of detailed design called for by performance categories C or D. The requirements for the seismic performance categories are discussed in more detail in a later section.

SITE EFFECTS

The seismic ground motions at a site and their effects on buildings are a function of the soil or rock profile at a site. The UBC code gives a factor based on the ratio of the building's fundamental period over the period of the site. ATC-3 outlines a more simple approach. Rock or soil that has a shear wave velocity greater than 2500 feet per second, or stiff soil that is less than 200 feet deep is identified as S1; a deep cohesionless or stiff clay or soil depth over 200 feet is S2; S3 is soft soil. If it is rock or a firm site, the soil profile coefficient is 1.0, in between, it is 1.2, while the soft site is 1.5.

- Soil Profile Types

  S_1 - Rock or over 2500 FPS shear wave velocity or stiff soil under 200 ft deep

  S_2 - Deep cohesionless or stiff clay or soil depth over 200 ft

  S_3 - Soft- to-medium clays and sands over 30 ft deep

- Site profile coefficient S

	S_1	S_2	S_3
S	1.0	1.2	1.5

SITE EFFECTS
Figure 8.6

The designer can also consider soil-structure interaction for some facilities. ATC-3 has a chapter covering soil-structure interaction. There are also 25 to 30 pages of excellent commentary explaining the intricacies of this procedure.

FRAMING SYSTEMS

ATC-3 considers four general types of framing systems. These systems can be combined, as for example, shear walls and frame. But the Response Modification Coefficient (R) at any level should not exceed the lowest R derived from the table (Figure 8.7) for the framing system above that level. In other words, if there is a shear wall in the upper stories with a frame below, the building cannot be designed as if it were a frame with a higher R value and hence a lower coefficient for design.

What about the case of common components such as columns or systems having different R values? This could happen in a building if it were designed in one direction as a frame, and in the other direction as a shear wall structure. ATC-3 requires for this case that the columns be designed as ductile in both directions.

For seismic performance categories A and B, ATC-3 provides that any type framing system permitted in the provisions may be used. Category C buildings over 160 feet must have a special moment-resisting frame, or be a dual system (moment resisting frames with shear walls or braced frames). Braced frames or shear walls may be used for buildings not over 240 feet in height.

RESPONSE MODIFICATION COEFFICIENTS[1]

Type of Structural System	Vertical Seismic Resisting System	Coefficients R^7	C_d^8
BEARING WALL SYSTEM: A structural system with bearing walls providing support for all, or major portions of, the vertical loads. Seismic force resistance is provided by shear wall or braced frames.	Light framed walls with shear panels	6½	4
	Shear walls		
	Reinforced concrete	4½	4
	Reinforced masonry	3½	3
	Braced frames	4	3½
	Unreinforced and partially reinforced masonry shear walls	1¼	1¼
BUILDING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with Shear panels	7	4½
	Shear walls		
	Reinforced concrete	5½	5
	Reinforced masonry	4½	4
	Braced frames	5	4½
	Unreinforced and partially reinforced masonry shear walls[6]	1½	1½
MOMENT RESISTING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by Ordinary or Special Moment Frames capable of resisting the total prescribed forces.	Special moment frames		
	Steel[3]	8	5½
	Reinforced concrete[4]	7	6
	Ordinary moment frames		
	Steel[2]	4½	4
	Reinforced concrete[5]	2	2
DUAL SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. A Special Moment Frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities.	Shear walls		
	Reinforced concrete	8	6½
	Reinforced masonry	6½	5½
	Wood sheathed shear panels	8	5
	Braced Frames	6	5
INVERTED PENDULUM STRUCTURES: Structure where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical loads.	Special moment frames		
	Structural steel[3]	2½	2½
	Reinforced concrete[4]	2½	2½
	Ordinary Moment Frames		
	Structural steel[2]	1¼	1¼

[1]These values are based on best judgment and data available at time of writing and need to be reviewed periodically.
[2]As defined in Section 10.4.1.
[3]As defined in Section 10.6.
[4]As defined in Section 11.7.
[5]As defined in Section 11.4.1.
[6]Unreinforced masonry is not permitted for portions of buildings assigned to Category B.
Unreinforced or partially reinforced masonry is not permitted for buildings assigned to Categories C and D; see Chapter 12.
[7]Coefficient for use in Formula 4-2, 4-3,a and 5-3.
[8]Coefficient for use in Formula 4-9.

Figure 8.7

How do the R values in Table 3-B work in practice? The higher the factor the lower the design shear force will be. Let us consider a frame system. This kind of building can deform quite a bit and absorb energy; thus it can be designed for a lower shear force, using a factor of 7. But if the building has shear walls, the factor decreases to 5½ or 4½; for braced frames it is 5. Why is the value for braced frames a little lower than for reinforced concrete shear walls? Because the failure of bracing can be very abrupt; whereas for a shear wall there is some energy absorbing capacity after it has fractured. In unreinforced masonry, the R value decreases to 1¼ or 1½ because there is minimal reserve capacity after failure. For a moment-resisting frame system of steel, use a factor of 8; if it is reinforced concrete use 7.

The last type of structure listed is the inverted pendulum structure. Typically, this type is characterized by a single column supporting a heavy precast or cast-in-place roof. Such structures are common to walkways around shopping centers and schools. These structures are particularly vulnerable to earthquake motions. If there is a special moment frame, either structural steel or reinforced concrete, a low R factor of 2½ is specified; if it is a steel ordinary moment frame, the reduction factor is only 1¼.

What about in-filled walls in the framing system? In seismic performance category C, consideration must be given to the interaction of the frames and adjoining elements. In the Caracas, Venezuela, earthquake of 1967, frame buildings designed according to the SEAOC requirements failed. What happened is that they had filler walls of tile that did not allow the frames to act as frames. With different force distribution, more force was transmitted into some components of the structure. The result was column failure either from overturning or overstress in shear. The lesson is that when designing a building, beware of putting in a lot of stiff partitions that fit tightly to the frame. If partitions or other filler walls between frames are desired, either allow some space or make provisions so that the frame can move without being limited by the filler walls. At least consider these in the analysis because they will make the building respond differently with a corresponding higher shear force. Deformational compatability works the same way. If in designing the building there are elements that are very brittle and cannot take much deformation, isolate them or provide a separation so that they are not subject to these forces. Finally, ATC-3 incudes the requirement that for category C special moment frames should be used when the building is over 160 feet.

Conformance with category D follows the pattern spelled out for C, except in the area of height limitations for the frame. For braced frame or shear wall buildings, the height limit of 240 feet is reduced to 160 feet; the 160 foot limit of C is reduced to 100 feet for category D buildings. Therefore, for buildings in category D over a 100 feet, there must be a special moment-resisting space frame or a dual system.

- Seisimic performance categories A and B

  Any type of framing system permitted in provisions may be used

- Seismic performance category C

  For buildings over 160 ft:

  1. Special moment resisting frame
  2. Dual system
  3. Braced frames or shear walls

  Interaction of frames and adjoining elements
  Deformational compatability
  Special moment frames

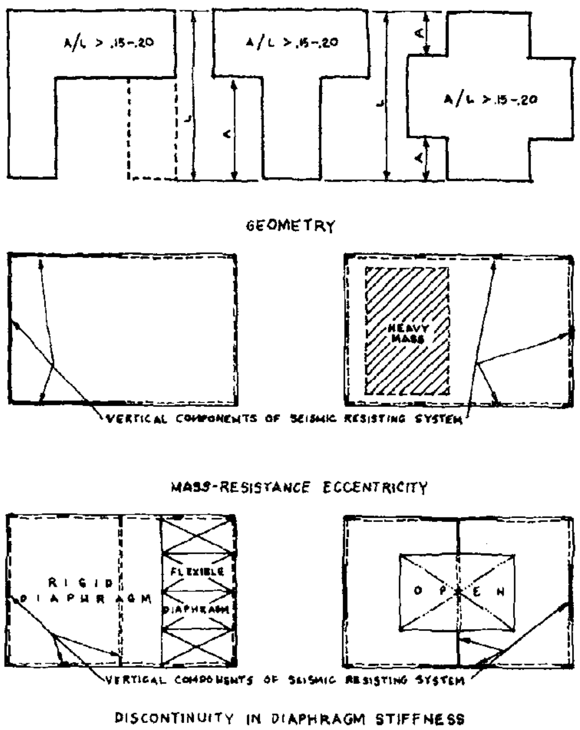
- Seismic performance category D

  Conform to category C

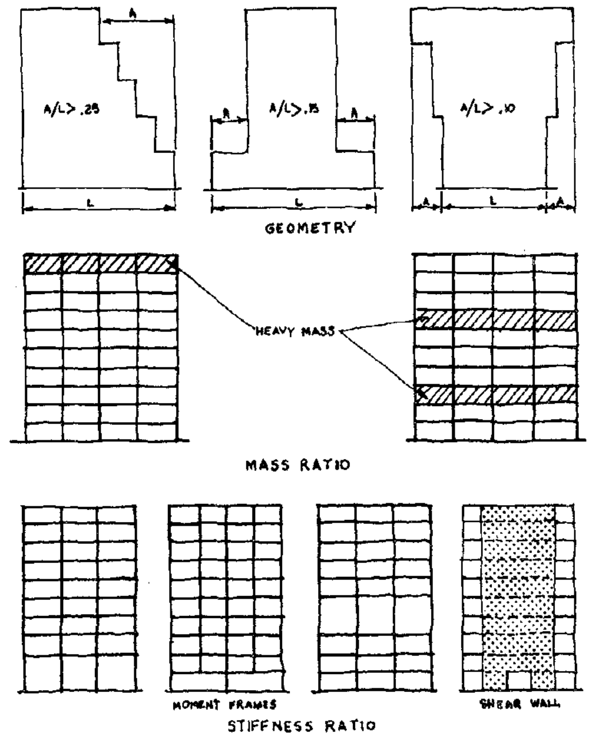
  Reduce height limitations for frames from 160 ft to 100 ft and for braced frame or shear wall systems from 240 ft to 160 ft

SEISMIC PERFORMANCE CATEGORIES
Figure 8.8

ATC-3 also considers building configuration. The design for a structure that is irregular—i.e., significant eccentricity between the seismic-resisting system and mass at any level, or the shear walls have been put off to one side, or the diaphragm at any level has significant change of strength or stiffness—extra requirements must be considered in the analysis. Figure 8.9 illustrates plan situations; Figure 8.10 illustrates vertical irregularities.



PLAN IRREGULARITIES
Figure 8.9



VERTICAL IRREGULARITIES
Figure 8.10

For analyzing regular buildings, the Equivalent Lateral Force procedure can be used as a minimum for categories C and D. But for irregular buildings, special considerations have to be given to the dynamic characteristics. That usually means a dynamic analysis. Figure 8.11 summarizes the analytical procedures.

- Seismic performance category A

    No overall seismic analysis

- Seismic performance category B

    Equivalent lateral force (elf) procedure

- Seismic performance categories C and D

    Regular buildings-elf procedure as minimum

    Irregular building-special consideration of dynamic characteristics (for vertical irregularities only-can use modal analysis given)

ANALYSIS PROCEDURES
Figure 8.11

SEISMIC PERFORMANCE CATEGORIES

ATC-3 specifies different design requirements for the four seismic performance categories A, B, C, D. Seismic performance category A requires ties and continuity; all parts of the building must be tied together. Walls must be tied to floors and roofs and minimum factors are given depending on the seismic area for the structure. It is recommended that all nonstructural elements be at least nominally anchored.

Category B includes all the above as well as provisions for combined load effects; earthquake effects must be considered together with snow load, gravity load, and live load effects. Wind load, however, does not have to be combined with earthquake loads.

Another consideration in category B is the effects of orthogonal directions of motion. The earthquake motion does not come only in one direction, but for simplicity of analysis the motion is considered in one direction and then the other. Because these motions occur simultaneously (although the maximum motions may not occur at the same time) corner columns are often under-designed. The ATC-3 method for handling this is to take the design forces on the column in one direction plus 30 percent of the force from the other direction. The major effect occurs on the corner columns.

ATC-3 includes a general statement about strength discontinuities in a building structure; the first time that this has been specified in seismic design provisions. For instance, in designing a building the engineer asks for 20 feet of shear wall on a given floor. The architect is agreeable but points out that planning requirements necessitate a 30 to 40 foot long wall on the floor but only 20 feet is permissable on the floor above. The structure will be stiffened and the earthquake response of the building is changed, often adversely. Thus, the provisions remind the designer to consider the potential adverse effects. The commentary outlines a procedure for handling such situations.

Another important factor arises in buildings over 240 feet tall. The provisions do not allow a structure with only a single shear wall. There must be four shear resisting planes in each direction so as to provide redundancy in the building structure. ATC-3 offers a discussion on redundancy in the commentary.

What about diaphragms? The provisions require the designer to consider the deflection of the diaphragm as it deforms as a horizontal beam. If it deforms too much, the walls that are connected to it may be overstressed in a direction normal to their plane. ATC-3 specifies requirements for the design of the diaphragm including openings which have to be considered and reinforced. It is necessary to provide drag-bar or collector elements to pick up the shear in the diaphragm and carry it to the shear walls. If the diaphragm is wood, the chords should be continuously tied together; if the diaphragm is steel, the chord would be a steel member.

Structures in category C should conform to all the above requirements for A and B. In addition, consideration should be given to vertical seismic forces. Post-stressed or pre-stressed members have to be looked at carefully to make sure that the vertical acceleration does not change the stress distribution enough to cause a problem. For category C, ATC-3 spells out certain requirements for foundations. For instance, spread footings and pile caps should be tied together in certain types of ground.

The chapters covering material requirements (wood, concrete, steel and masonry) specify additional requirements for category C buildings.

Category D requirements are the same as for C, except that post-stressed and pre-stressed piles cannot be used in bending. The remaining provisions for category D are pretty much the same, though the material limitations are somewhat more stringent for D than for C.

- Seismic performance category (SPC) A

  Ties and continuity
  Concrete or masonry wall anchorage
  Anchorage of nonstructural systems

- Seismic performance category B

  Conforms to SPC A
  Combination of load effects
  Orthogonal effects
  Strength discontinuities of vertical resisting system
  Consider redundancy
  Collector elements
  Diaphragms
    Deflection
    Design
  Bearing walls
  Materials limitations
  Openings

- Seismic performance category C

  Conform to SPC B
  Consider vertical seismic motions
  Foundation requirements
  Materials limitations

- Seismic performance category D

  Conforms to SPC C
  Special pile limitations
  Materials limitations

DESIGN AND DETAILING REQUIREMENTS
Figure 8.12

Now a few words about deflection and drift limits. To begin with, all parts of the building should be designed to act as a unit, unless separation is provided between units. Such separation must be adequate to take care of the drift or the deformation of each portion of the building. ATC-3 specifies limitations for story drift in Table 3-C as shown in Figure 8.13. There is a smaller drift limit for essential facilities than for seismic hazard exposure Group I buildings. The limits are increased by one-third for buildings three stories or less which do not have brittle finishes.

ALLOWABLE STORY DRIFT Δ_a

Seismic Hazard Exposure Group

	III	II	I
Δ_a	$0.010h_{sx}$	$0.015h_{sx}$	$0.015h_{sx}$

[1]Where there are no brittle-type finishes in buildings three stories or less in height, these limits may be increased one-third.

h_{sx} = the story height below level x

Figure 8.13

EQUIVALENT LATERAL FORCE ANALYSIS

The Equivalent Lateral Force analysis procedure uses a typical formula: $V = C_sW$ as shown in Figure 8.14. W is the weight of the building, and C_s, is the seismic design coefficient. C_s is determined by using the factors that have been discussed, A_v, or A_a, from the map areas and Figure 8.4. R is the Response Modification Factor; and T is the period of the building to the 2/3 power. For lower buildings, C_s does not have to exceed 2.5 A_a, divided by the R value. Thus for three-and four-story buildings, there usually is no need to calculate the period. Where there is very soft soil and A_a is greater than .3g, C_s need not exceed 2 A_a/R.

- Seismic Base Shear

$$V = C_sW$$

where

C_s = seismic design coefficient

W = total gravity load including partitions and permanent equipment, 25% of storage and warehouse floor loads, and effective snow load.

- Seismic Design Coefficient

$$C_s = \frac{1.2A_vS}{RT^{2/3}} \leqslant 2.5\ A_a/R^*$$

where

A_v or A_a = coefficient from Yable 1-B

R = factor from Table 3-B

T = period of teh building

* For soil profile S_3 where $A_a \geqslant 0.30$

$$C_s = 2A_a/R$$

Figure 8.14

The formula for the building period in ATC-3 is not much simpler than in other codes, see Figure 8.15. For moment-resisting structures where the frames are not enclosed or adjoined by more rigid components, the formula $T_a = C_t H_n^{3/4}$ can be used with the factor C_t, equal to 0.035 for steel frames and 0.025 for concrete frames, and H is the height in feet above the base to the highest level of the building. For all other buildings, the formula is the same as the present code, with L being the overall length in feet at the base of the building in the direction being considered.

- <u>Period Determination</u>

Moment-resisting structures where frames not enclosed or adjoined by more rigid components

$$T_a = C_t h_n^{3/4}$$

where

C_t = 0.035 for steel frames

C_t = 0.025 for concrete frames

h_n = height in feet above base to highest level of building

For all other buildings

$$T_a = 0.05 h_n / \sqrt{L}$$

where

L = overall length in feet at base of building in direction under consideration

Figure 8.15

In the case of vertical distribution of seismic forces in buildings, the formula has been slightly changed to a parabolic distribution. The concentrated force at the top which is in the present UBC and SEAOC requirements has been deleted. The horizontal shear and torsional forces have to be distributed and designed for at each level. They are distributed to the vertical seismic-resisting components in the story below by considering the relative stiffness of the components and diaphragm. This is different from some other codes. In some countries, the codes specify requirements for flexible or rigid diaphragms, but the ATC-3 group felt that it is almost impossible for code purposes, to define such ranges in diaphragms. It is important to consider the relationship of the diaphragm's stiffness to the stiffness of the vertical resisting components such as the columns or shear walls.

- **Vertical Distribution of Seismic Forces**

Lateral seismic shear force at any level

$F_x = C_{vx}V$

where

$$C_{vx} = \frac{w_x h_x{}^k}{\sum_{i=1}^{n} w_i h_i{}^k}$$

K is related to building period T:

$T \leqslant 0.5$ second, $k = 1$

$T \geqslant 2.5$ second, $k = 2$

In between interpolate or use $k = 2$

w_i, w_x = portion of W at level i or x

h_i, h_x = height above base to level i or x

Figure 8.16

ATC-3 requires that the effects of torsional moments due to eccentricities between the building mass and the seismic resisting system be considered in the design. In addition, a 5 percent accidental torsional moment equal to the mass being moved a distance equal to 5 percent of the building dimension (normal to the direction under consideration) each way from the calculated center of mass, must be considered in the design. The accidental torsion provision is recognition of the effects of the random multi-directional nature of earthquake motions.

- **Horizontal Shear Distribution and Torsion**

Seismic shear force at any level

$$V_x = \sum_{i=x}^{n} F_i$$

V_x and torsional forces - distribute to vertical seismic resisting components tn story below level x considering the relative stiffness of components and diaphragm

Provide for actual torsional moment plus 2% accidental torsion

Figure 8.17

ATC-3 partially solves another problem—that of overturning due to seismic forces. To design for overturning at all levels, calculate the overturning moment by the formula shown in Figure 8.18, then distribute the increment of the overturning moment at that particular floor in the same proportion as the horizontal shear is distributed for the walls or frames. There is a factor of 1.0 for the top ten stories. (The result is no reduction for buildings up to ten stories.) It is 0.8 from the 20th-story from the top and below; interpolate for stories in between.

In the case of foundations, ATC-3 permits a reduction to 75 percent of the moment at the foundation/soil interface providing the resultant of the forces does not fall outside the middle half of the components that are resisting shear. This is a substantial reduction from other codes, however, we are starting with higher forces. A strong recommendation is made that increased research be done in this area.

- Overturning

Design for seismic overturning forces

At any level, distribute increment of overturning moment to walls or frames in same proportion as distribution of horizontal shears to those walls or frames

Calculate overturning moment

$$M_x = k \sum_{i=x}^{n} F_i (h_i - h_x)$$

where

κ = 1.0 for top ten stories

κ = 0.8 for twentieth story from top and below

Interpolate between

Building foundations

Foundation overturning design moment, M_f, at foundation-soil interface, $M_f = 0.75M_x$

Resultant of seismic forces and vertical loads shall not fall outside of middle half of the base.

Figure 8.18

This brings us to drift determination and P-Delta effects in Figure 8.19. The design story drift, Δ, equals the difference of the deflections at the top and bottom of the story. The Cd factor is taken from Table 3-B, and is an amplification factor that depends on the type of material and type of framing.

- Drift Determination and P-Delta Effects

Design story drift, Δ, equals difference of deflection, δ_x,

$$\delta_x = C_d \delta_{xe}$$

where

C_d = Factor from Table 3-B

δ_{xe} = deflections from elastic analysis

- P-Delta Effects

Need not be considered if stability coefficient, $\theta \leqslant 0.10$

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

where

$P_x = \sum_{i=x}^{n} w_i$, total gravity load at and above level x

V_x = Formula in Figure 8.17

h_x = story height below level x

Figure 8.19

Another novel factor (for codes) in ATC-3 covers the P-Delta effect. When a structure deflects in an earthquake, the force from the gravity load is still vertical and bending moments are equal to the vertical force times the deflection. These secondary moments in the columns and griders are called the P-Delta effect. A big question used to be when to consider this effect. ATC-3 takes a different approach by saying if the stability coefficient does not exceed 0.10, forget it. If it exceeds 0.10, methods for calculating the effects are given in the commentary.

MODAL ANALYSIS

For those buildings that have vertical irregularities, a modal analysis can be used. In order to keep the procedure fairly simple, so it could be put into code format, a lumped mass model was used. Established methods of mechanics can be applied for a fixed base condition to calculate the period and the modal base shear. The formula for the base shear is similar to that for the equivalent lateral force procedure, $V_m = C_{sm}W_m$. However, the factors are derived from a dynamic analysis.

- **Modeling**

  Lumped mass model

- **Period**

  Established methods of mechanics for fixed base condition

- **Modal Base Shear**

  $V_m = C_{sm}\overline{W}_M$

  where

  $\overline{W}_m$ = effective modal gravity load

  $$= \frac{\left[\sum_{i=1}^{n} w_i\phi_{im}\right]^2}{\sum^{n} w_i\phi^2_{im}}$$

  ϕ_{im} = displacement amplitude at i th level when vibrating in m th mode

  $$C_{sm} = \frac{1.2A_vS}{RT_m^{9/3}}$$

  *For soil profile S_3 where $A_a \geqslant 0.30$

  $C_{sm} = 2\ A_a/R$

Exceptions

1. For soil profile S_3, C_{sm} for modes other than fundamental and which have periods less than 0.30 second.

   $$C_{sm} = \frac{A_a}{R}\ (0.8 + 4.0T_m)$$

2. Where $T_m > 4.0$ seconds

   $$C_{sm} = \frac{3\ A_vS}{RT_m^{4/3}}$$

Figure 8.20

For very tall buildings, that is buildings with a fundamental period greater than four seconds, ATC-3 states that a second formula can be used, see Figure 8.20. The modal forces and deflections are then calculated at each level. The modal story shear and moments are computed by linear elastic methods. to get the design values, combine the modal values using the square root of the sum of the squares.

Then the design base shear derived from the modal analysis is compared with a shear that is calculated using a T value of 1.4 T_a. If the base shear calculated by using modal analysis, is less than 80 percent of this second value, then it is necessary to go back and raise the values. Why an 80 percent factor? There are many ways of calculating period. There have been some instances where designers have calculated periods of 1.5 seconds for buildings only six stories high. If the building is very flexible with a low C factor, chances are the building will exhibit inadequate seismic resistance. The 80 percent limit was developed to prevent this type of interpretation and the resulting poor building.

- Modal Forces, Deflections and Drifts

  Modal Force, F_{xm}, at each level

  $$F_{xm} = C_{vxm} V_m$$

  $$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}}$$

  Modal deflection, δ_{xm}, at each level

  $$\delta_{xm} = C_d \delta_{xem}$$

  $$\delta_{xem} = \frac{g}{\substack{4\ \pi \\ 4}} T_m^{2} F_{xm}$$

- Modal Story Shears and Moments

  Compute by lienar static methods

- Design Values

  Combine modal values by square foot of sum of squares

  Compare design base shear with shear icalcuated using
  $T = 1.4\ T_a$

Figure 8.21

NONSTRUCTURAL PROVISIONS

For nonstructural systems and components, the formula $F_p = A_v C_c P W$, is a great deal similar to the one that has been used in the Uniform Building Code. A_v, is derived from the map, C_c is determined from Table 8-B as shown in Figure 8.22. P is the performance factor, and W is the weight. For mechanical and electrical components two amplification factors, a_x and a_c, are added; a_x reflects the effects of height of the building while a_c reflects the type of anchorage.

TABLE 8-B

SEISMIC COEFFICIENT (C_c) AND PERFORMANCE CHARACTERISTIC LEVELS REQUIRED FOR ARCHITECTURAL SYSTEMS OR COMPONENTS

(See Table 8-A for S, G and L Designations)

Architectural Components	C_c Factor	Required Performance Characteristic Levels Seismic Hazard Exposure Group		
		III	II	I
Appendages				
Exterior Nonbearing Walls	.9	S	G[2]	L[4]
Wall Attachments	3.0	S	G[2]	L[4]
Veneers	3.0	G	G[1]	L
Roofing Units	.6	G	G[2]	NR
Containers and Miscellaneous Components (free standing)	1.5	G	G	NR
Partitions				
Stairs and Shafts	1.5	S	G[3]	G
Elevators and Shafts	1.5	S	L[3]	L[5]
Vertical Shafts	.9	S	L[3]	L[6]
Horizontal Exits including Ceilings	.9	S	S	G
Public Corridors	.9	S	G	L
Private Corridors	.6	S	L	NR
Full-height Area Separation Partitions	.9	S	G	G
Full-height Other Partitions	.6	S	L	L
Partial-height Partitions	.6	G	L	NR
Structural Fireproofing	.9	S	G[3]	L[6]
Ceilings - Fire-rated Membrane	.9	S	G[3]	G
Nonfire-rated Membrane	.6	G	G	L
Architectural Equipment - Ceiling, Wall, or Floor Mounted	.9	S	G	L

NR = Not Required.

[1]May be reduced one performance level if the area facing the exterior wall is nominally inaccessible for a distance of 10 feet plus one foot for each floor of height.
[2]May be reduced one performance level if the area facing the exterior wall is nominally inaccessible for a distance of 10 feet and building is only one story.
[3]Shall be raised one performance level if building is more than four stories or 40 feet in height.
[4]Shall be raised one performance level if building is in an urban area.
[5]May be reduced to NR if building is less than 40 feet in height.
[6]Shall be raised one performance level for an occupancy containing flammable gases, liquids, or dust.

Figure 8.22

How are these factors applied? A_V is derived from the map. The next coefficient is the performance factor. What value should the components have? Should it have superior performance, good performance, or low performance? This depends on the seismic performance category and the seismicity index. The required performance gives the P value from Table 8-A (Figure 8.23). Having the A_V and P, the C factor is found in Table 8-B (Figure 8.22).

TABLE 8-A

PERFORMANCE CRITERIA

Designation	Performance Characteristic Level	P
S	Superior	1.5
G	Good	1.0
L	Low	0.5

[1]See Tables 8-D and 8-C.

Figure 8.23

SEISMIC COEFFICIENT (C_c) AND PERFORMANCE CHARACTERISTIC LEVELS REQUIRED FOR MECHANICAL/ELECTRICAL COMPONENTS

(See Table 8-A for S, G and L Designations)

Mechanical/Electrical Components[1]	C_c Factor[2]	Required Performance Characteristic Levels Seismic Hazard Exposure Group III	II	I
Emergency Electrical Systems (code required) Fire and Smoke Detection System (code required) Fire Suppression Systems (code required) Life Safety System Components	2.00	S	S	S
Boilers, Furnaces, Incinerators, Water Heaters, and Other Equipment Using Combustible Energy Sources or High Temperature Energy Sources, Chimneys, Flues, Smokestacks, and Vents Communication Systems Electrical Bus Ducts and Primary Cable Systems Electrical Motor Control Centers, Motor Control Devices, Switchgear, Transformers, and Unit Substations Reciprocating or Rotating Equipment Tanks, Heat Exchangers, and Pressure Vessels Utility and Service Interfaces	2.00	S	G	L
Machinery (Manufacturing Process)	.67	S	G	L
Lighting Fixtures	.67[3]	S	G	L
Ducts and Piping Distribution Systems				
● Resiliently Supported	2.00	S	G	NR
● Rigidly Supported	.67[4]	S	G	NR
Electrical Panelboards and Dimmers	.67	S	G	NR
Conveyor Systems (non-personnel)	.67	S	NR	NR

NR = Not Required.

[1]Where mechanical or electrical components are not specifically listed in Table 8-C, the designer shall select a similarly listed component, subject to the approval of the authority having jurisdiction, and shall base the design on the performance and C_c values for the similar component.

[2]C_c values listed are for horizontal forces. C_c values for vertical forces shall be taken as 1/3 of the horizontal values.

[3]*Hanging- or swinging-type fixtures shall use a C_c value of 1.5 and shall have a safety* cable attached to the structure and the fixture at each support point capable of supporting 4 times the vertical load.

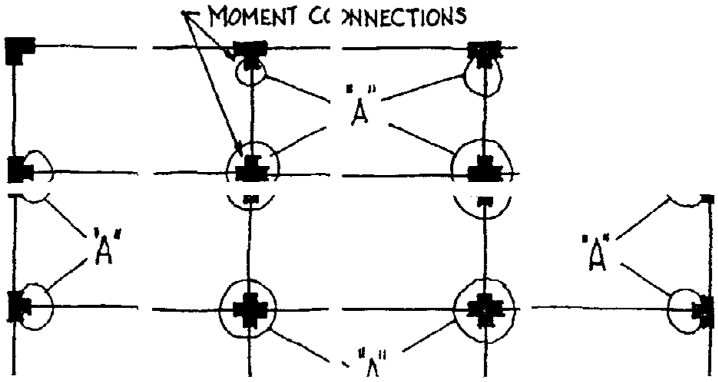
[4]Seismic restraints may be omitted from the following installations:

a. Gas piping less than 1-inch inside diameter.
b. Piping in boiler and mechanical rooms less than 1-¼ inches inside diameter.
c. All other piping less than 2-½ inches inside diameter.
d. All rectangular air-handling ducts less than 6 square feet in cross-sectional area.
e. All round air-handling ducts less than 28 inches in diameter.
f. All piping suspended by individual hangers 12 inches or less in length from the top of the pipe to the bottom of the support for the hanger.
g. All ducts suspended by hangers 12 inches or less in length from the top of the duct to the bottom of the support for the hanger.

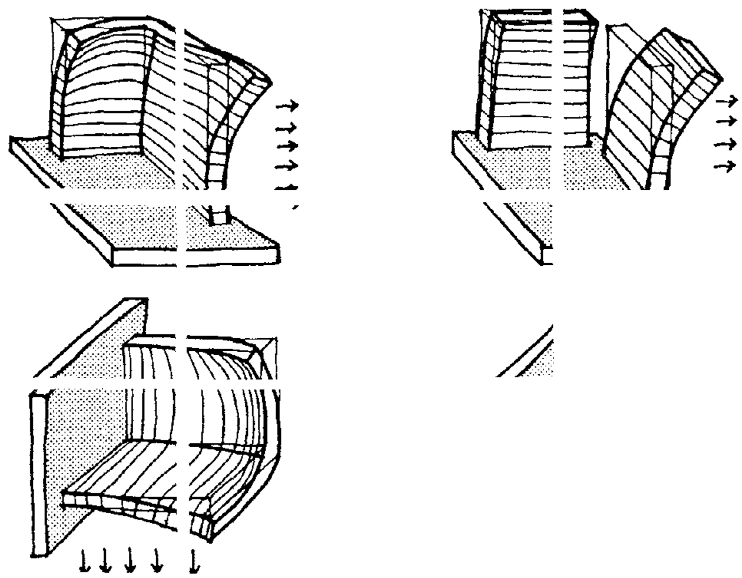
Figure 8.24

A similar procedure is followed for mechanical and electrical components. The C factors are quite a bit higher, particularly for fire and smoke detection systems, than they are for other equipment. An S factor (superior performance), is required for emergency electrical fire and smoke detection, and fire suppression systems as well as life safety system components. The C factors are determined from Table 8-C (Figure 8.24).

The above text has presented a summary of "Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC-3-06). The document represents the cooperative effort of design professionals, building code officials, and the research community. The results of their efforts reflect the state of the art of earthquake design in 1978. Of course, every table, every word of commentary will be subject to lengthy and intensive review throughout the country. Whatever shape the document finally takes, we have come a long way since those first seismic codes in the 20's. However, it is just as clear that we still have a distance to go in developing data and procedures that will give designers the answers to all questions that have been and are being raised in the field.



Part 4
Building Form and Structure



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Chapter 9. BUILDING FORM AND CONFIGURATION

Christopher Arnold and Eric Elsesser

CONFIGURATION AND THE CODES

This paper discusses the relationships between building configuration and seismic design; that is, the influences that the size and shape of a building, in combination with other factors, have on the way in which a building withstands earthquakes. Since the architect is the design professional with the most control over building configuration, this area of seismic design is of most relevance to the architect. To the extent that configuration is an important element in seismic design, it then becomes important that as the architect evaluates alternative configurations at the early stages of the design process he has the knowledge to assess their seismic design implications as one of the design variables.

Configuration is indeed an important influence as inspection of recent seismic codes, or textbooks on seismic design, will stress. The way in which U. S. seismic codes deal with configuration reveals, however, the uncertainty engendered by this topic which engineers in the seismic field have long recognized as a key issue. For many years, until the 1973 edition of the Uniform Building Code (UBC), the code essentially did not deal with configuration at all. And at present it treats the issue only with a general caveat. If the subject is important, why is this so?

The UBC now deals with the configuration issue in three places, each of which points out by implication that irregular structural configurations make seismic problem more complicated and difficult, but none of which prescribes specific rules by which irregular configurations are defined or assessed: in other words, warnings are provided, but no guidance.

Configuration has been found too difficult to codify, that is, to reduce the simple sets of rules of thumb that make up our typical code format. This difficulty is explained in the commentary portion of the Structural Engineers Association of California's Recommended Lateral Force Requirements and Commentary (1975):

> Due to the infinite variation of irregularities [in configuration] that can exist, the impracticality of establishing definite parameters and rational rules for the application of this Section are readily apparent. These minimum standards have, in general, been written for uniform buildings and conditions. The subsequent application of these minimum standards to unusual buildings or conditions has, in many instances, led to an unrealistic evaluation.
>
> (pp. 33-C to 34-C)

The state law governing school construction in California has a specific statement about configuration:

> When the design of a structure or parts of a structure result in unusual configuration (sic) or irregular distribution of lateral stiffness, evidence shall be presented to show that equivalent safety to that established by these regulations is provided or the office shall withhold its approval.

This language is currently being incorporated into the California hospital code. In other words, configuration is taken so seriously here that use of unusual configurations -- a condition that is not precisely defined -- can result in the design being rejected, which would be a very unfortunate setback to any project.

CONFIGURATION DEFINITION AND DERIVATION

Clearly, there is a need for information. Our office is currently engaged in a research program for the National Science Foundation which is looking in more detail at the questions of seismic design and building configuration, and it is hoped to provide some information and guidance to designers to start filling in the gap that exists after the general truisms have been stated. In so doing, we will point the way to a positive approach to design in which proper understanding of seismic design may enhance, rather than restrict, the architect's formal vocabulary.

We need to define configuration. In our study we are using two related definitions: one is the conventional concept representing the size and shape of the building; the other is the nature and disposition of seismic resistant elements within it. These two sets of characteristics are closely related and must be viewed as a conceptual entity.

The configuration and the formal elements that create it originate in the building program, which can be summarized as a description of the activities that are housed in the building, the services, furniture and equipment they need, and the space that they require. Activities produce a demand for certain settings and specific kinds of space division, which result in patterns of circulation; the combinations of activity spaces and circulation lead to certain dimensions and finally into a building form. But there are other determinants of configuration, such things as the site characteristics, its geology, its size and its geometry; urban design requirements; and architectural stylistic concerns. Sometimes these may dominate. The final configuration choice is the result of a decision-process which balances these varying requirements and influences, and resolves conflicts into a single result.

THE IMPORTANCE OF CONFIGURATION

The importance of configuration in seismic design has two basic aspects. One is that configuration influences or even determines the kinds of resistance systems that can be used and the extent to which they will, in the broadest sense, be effective. The second aspect is our hypothesis that many failures of engineering detail, which result in severe earthquake damage or collapse, are in fact failures of configuration.

In other words, the configuration of the building is such that seismic forces place intolerable stress on some engineering detail and it fails. This is not to suggest that configuration is primary, and engineering detail secondary or of no consequence; the issues are obviously related. But it does mean that the designer's first ideas on configuration are very important because at an early conceptual stage he is making decisions of great significance to later engineering analysis and design.

Configuration choice affects the design cost because if the configuration is seismically unsuitable it is going to be more difficult to design; it will need more time, it will need more computation, and probably a dynamic analysis. Second, it will affect construction cost, because an unsuitable configuration will result in more resistant elements, larger resistance elements, more reinforcing or more costly connections. Third, the configuration choice may affect the use of a building, because of the relationship between the nature, size, and location of the resistant elements which are a function of configuration. For example, the imposition of large shear walls and extremely restricted openings may have a large influence on the planning of a complex building such as a hospital. Fourth, in an extreme case, configuration may affect safety; the configuration may be such that safe or reasonably predictable design is impossible, except at unaffordable cost or inconvenience in use. So the importance of configuration is considerable.

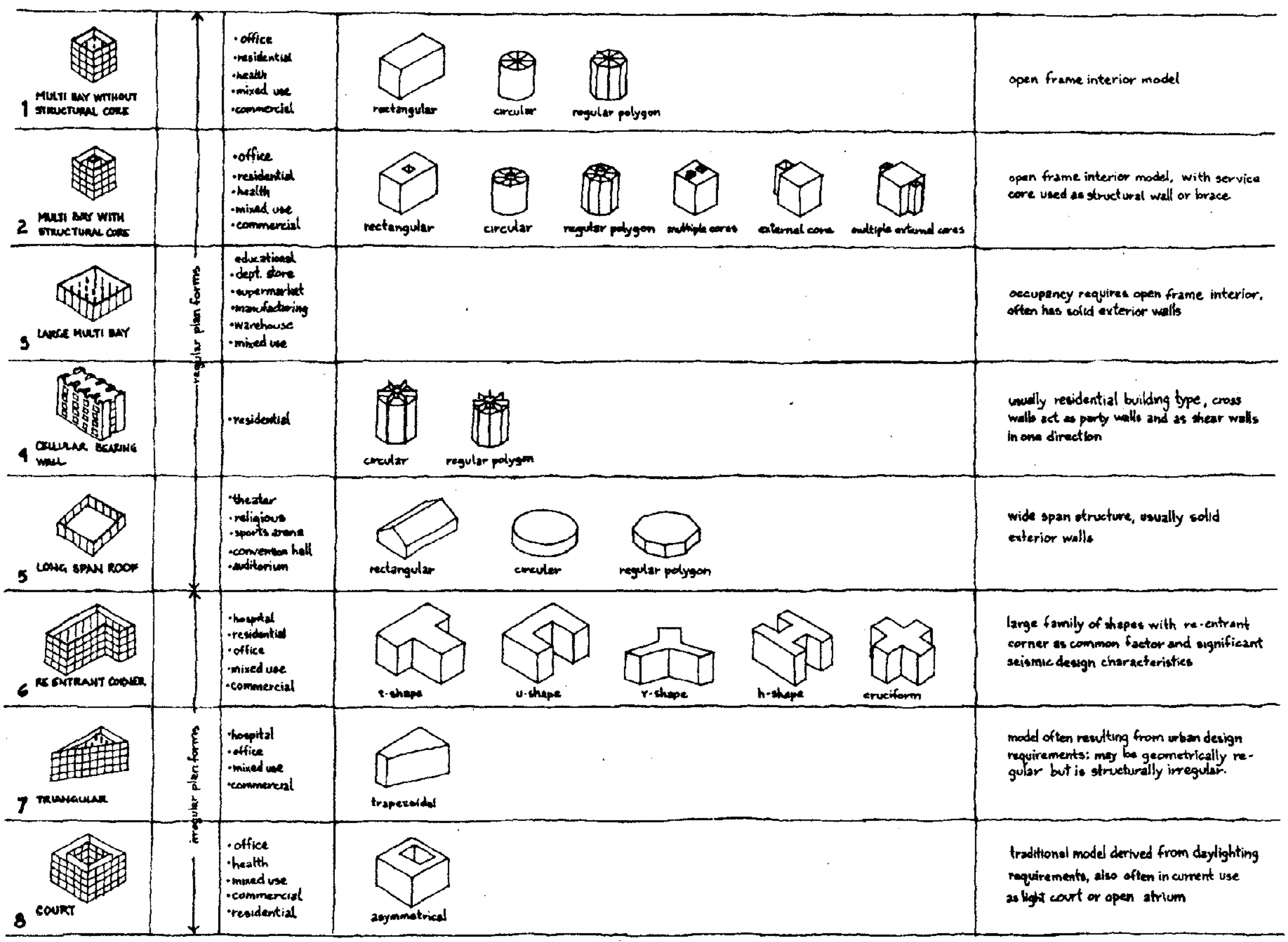
CONFIGURATION TYPES

What is configuration? What are the sizes and shapes in which buildings are created? One's immediate reaction is that the variety is infinite; but if one attempts to classify configuration in terms that are important to our particular way of thinking, the majority of building configurations can be reduced to eight basic types as shown in Figure 9.1. These cover essentially all common occupancies and building types; excluded might be special occupancies such as linear acceleration or special structures such as air-supported roofs. This classification is based on three issues. The first is geometry and dimension; this is truly infinite, but the other two issues quickly reduce the quantity to manageable proportions. The second issue is building use, for certain activities dictate certain kinds of shapes. For example, the need for perimeter exposure in residential use demands rather narrow building shapes; while the rise of fluorescent lighting and air-conditioning, combined with urban real estate considerations, has resulted in the deep floor plan of many of our office buildings. The third issue is that of structural type, defined in ways which, from experience, we know are significant to seismic design. The eight models are combinations of these three factors; this serves very quickly to reduce infinite variety to something much more useful.

The importance of this classification is that as configurations start to be conceived in schematic form, they can be related to the model. As more is learned about the significance of these models from the seismic design viewpoint, this can help to assure design development that reflects understanding of seismic design problems and opportunities.

The first two configuration models shown are called a multi-bay without a structural core and a multi-bay with a structural core. The distinction between them is structural that is, whether the circulation and service core is part of the building structure or not. This distinction makes a tremendous difference in the way in which the resistant elements of the building will be designed. You cannot tell by looking at the outside of a building which one of these two it is. The third model is a large multi-bay configuration that is a familiar form for the shopping center, department store, warehouse, or large school. Its large size raises some characteristic seismic problems, particularly if, as is usually the case with these occupancies, a minimum of interior walls is used. The fourth model, the cellular bearing wall, is a very characteristic residential form; the significance of this lies in its use of closely spaced cross walls that can be used structurally. These are often party walls that work well acoustically as heavy structural walls. This gives a good structural form for a shear wall design in one direction, but the other direction must also be considered. The fifth model is the long span roof, a common form for auditoria and stadia. The sixth model is the re-entrant corner family which is of real interest to us because it is inherently a problem shape, and it will show up again when we consider problems. Many urban buildings have re-entrant corners, designed before the advent of air-conditioning and efficient artificial lighting to obtain light and air.

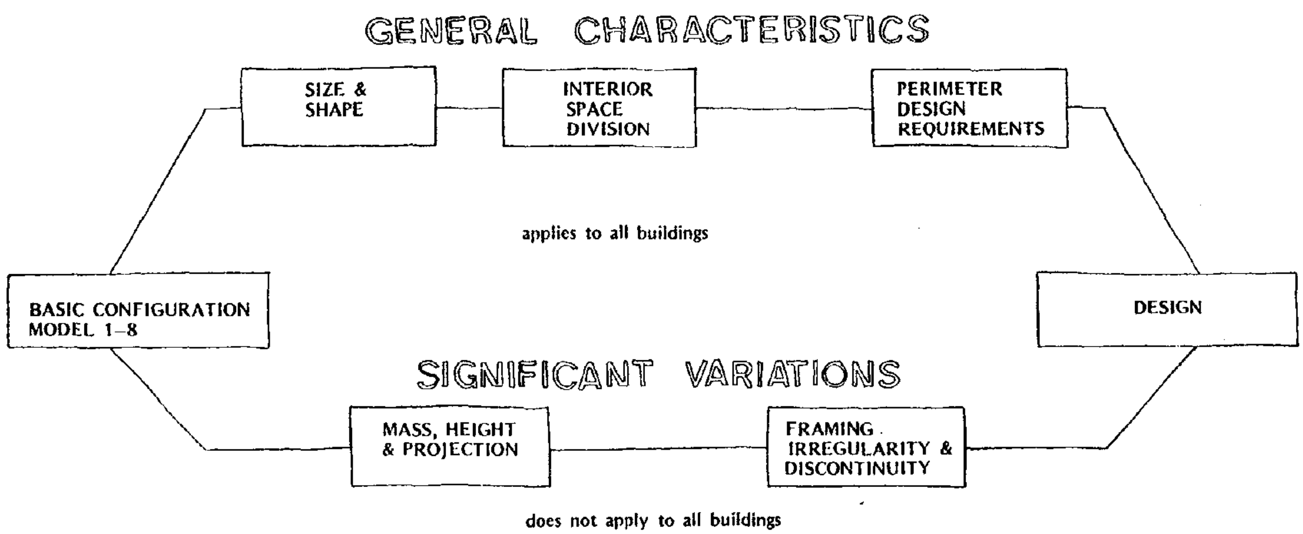
The final two models are triangular forms and court forms. These are both of geometrical significance and also problem types. Triangular forms occur very often for urban design and city planning reasons. For example, when a street cuts through a grid pattern on a diagonal, many awkwardly shaped triangular buildings are the result. But sometimes these shapes are also created voluntarily; currently, triangular and trapezoid forms are very popular. Triangular forms are geometrically regular but dynamically unbalanced; seismic forces will produce torsion, which is discussed later.



BASIC CONFIGURATION MODELS
Figure 9.1

The courtyard building is a very characteristic form in the inner city, because of the need to provide light and air in large buildings, but the shape is very common elsewhere as well. This form is a special case of re-entrant corner condition.

We often find these eight models in their pure form, but we also need to modify them so that we can develop the full range of building forms that arise. Space does not permit a full discussion of the methodology of definition, but Figure 9.2 shows the way in which modifications are applied to the Basic Models in order to define a specific design. Examples of two of these modifications are shown in Figure 9.3 and 9.4, and Figure 9.5 shows a design fully defined with reference to a Basic Model.



- some designs can be identifed as basic configuration models in pure form, and defined by the upper row of characteristics only
- *other designs can be identified as basic configuration models defined by the upper row of characteristics, but modified by the significant variations in the lower row*

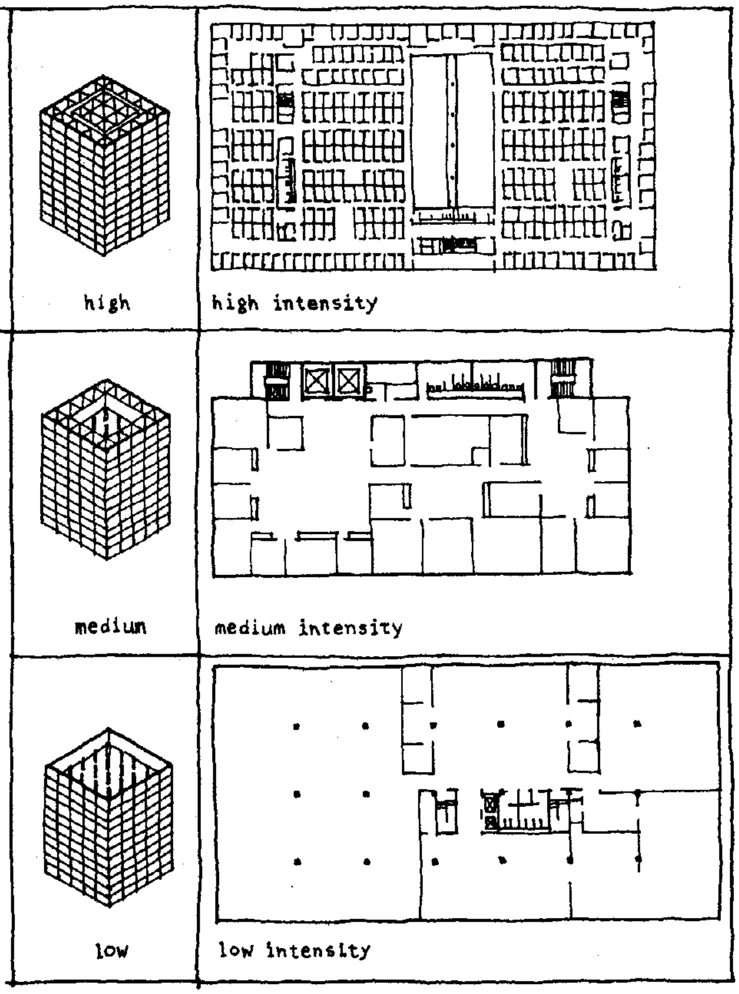
SPECIFIC DESIGN
Figure 9.2

intensity

-defines the quantity of space division - the lineal footage of interior walls relative to floor area. Weight of partition material is also significant.

-affects: weight, hence magnitude of inertial forces and damping.

INTERIOR SPACE DIVISION

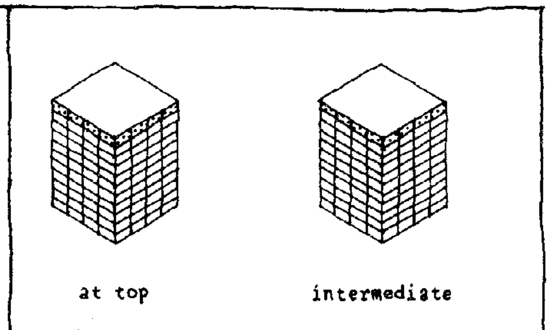


GENERAL CHARACTERISTICS
Figure 9.3

variation in mass

-identifies large variations in building mass between different building portions

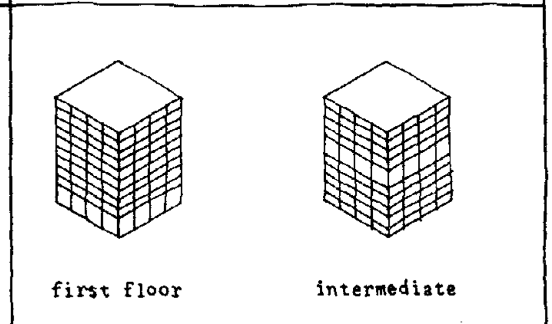
-affects: characteristics of building response



variations in floor height

-identifies localized variations in floor to floor height

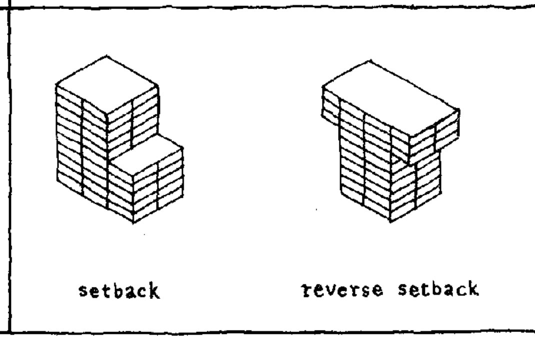
-affects: structural stiffness in localized positions



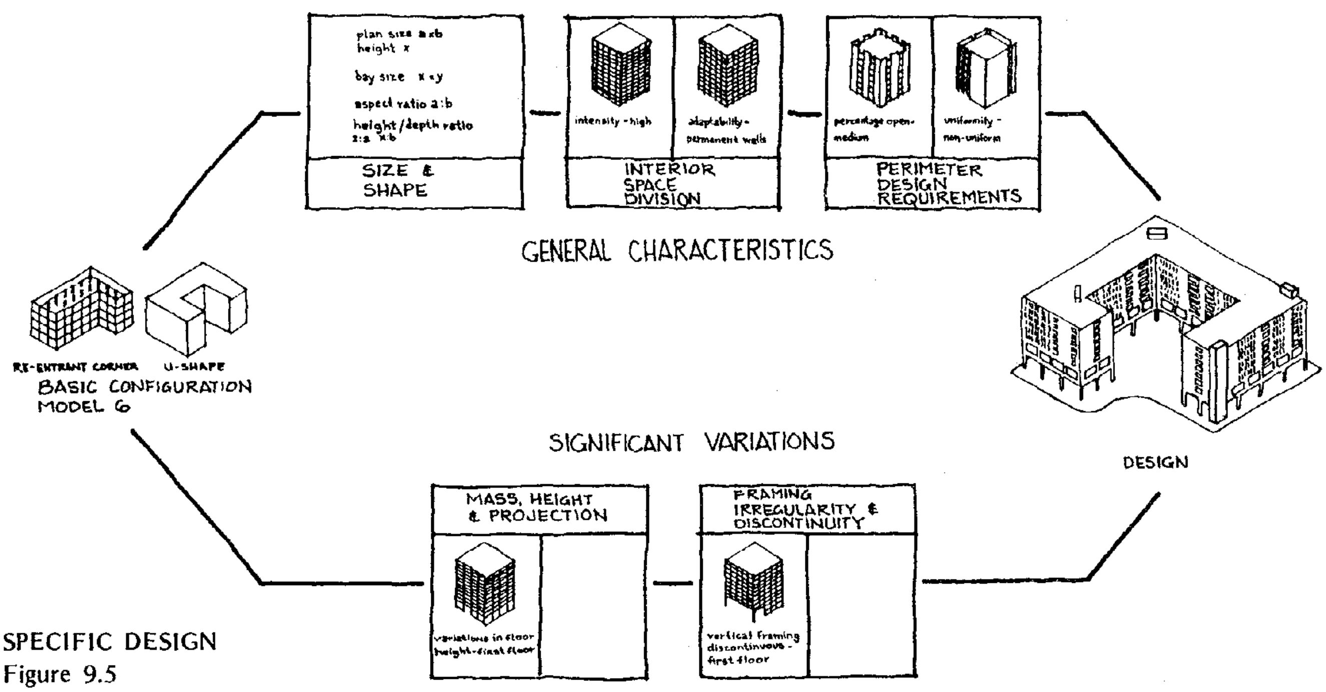
setback

-identifies changes in vertical configuration of structure

-affects: characteristics of building response, possibilities of pounding damage



SIGNIFICANT VARIATIONS
Figure 9.4



SPECIFIC DESIGN
Figure 9.5

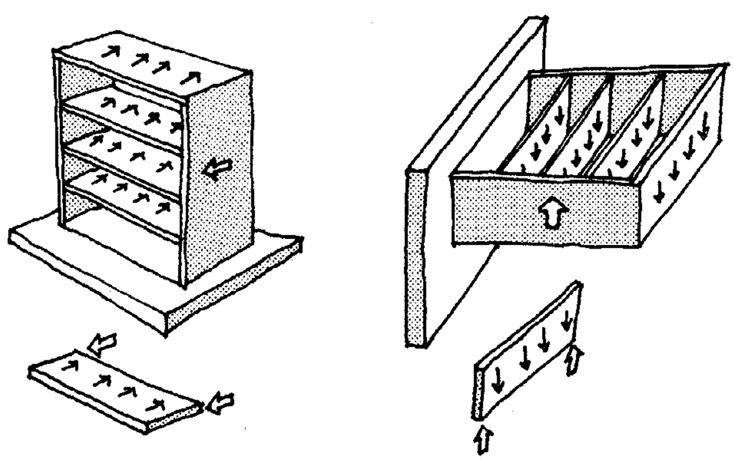
CONCEPTS OF SEISMIC DESIGN

In designing to resist seismic forces, the structural engineer uses a quite small vocabulary of components and systems. Rigid frames, shear walls, and bracing, for example, represent the three basic ways in which we ensure that structures can resist lateral forces. In addition, the configuration of the building and the way in which its lateral resistance systems are disposed have a large effect on the nature of the forces that the lateral structural system must resist. It is useful for the designer to acquire an intuitive sense of some of these effects; the detailed calculations can be left to the structural engineer. We use the term intuitive to suggest knowledge that is based on theoretical understanding so well absorbed that knowledge becomes feeling. This generally occurs as the result of experience. The architect may not be in a position, or may not wish to acquire, the depth of theoretical understanding and experience that is necessary for the engineer. But it is worth attempting to transfer the feeling for structural forces because once acquired, this feel can act as an almost unconscious guide to the designer.

Most architects have acquired a good sense of vertical forces. One way of attempting to transfer a feeling for the way in which lateral forces work is to imagine them as vertical forces, rotated 90°. The accompanying sketches represent an elementary course in this approach. However, the reader should note that seismic forces are more complex than vertical forces, and more complex than the other major lateral force, that of wind.

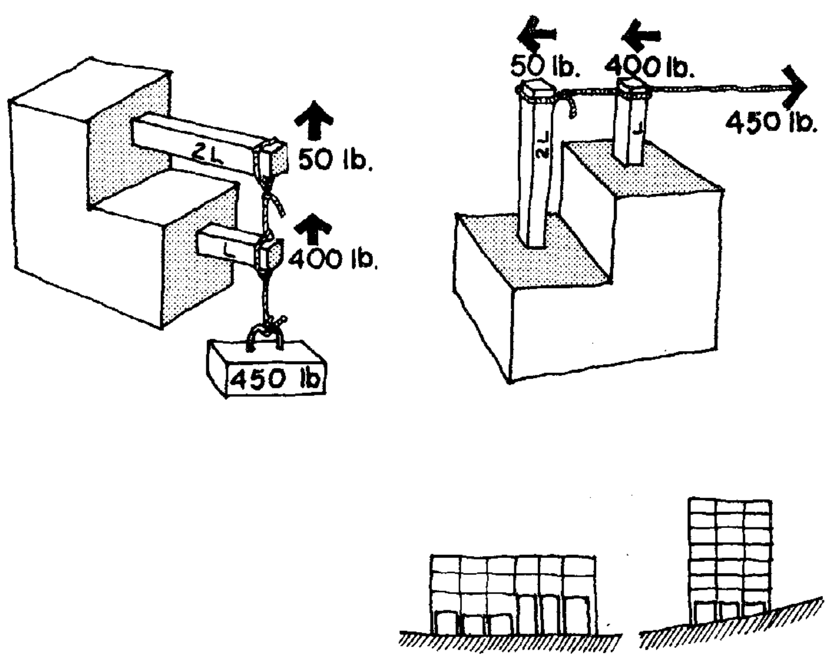
Seismic forces have two very important characteristics not shared by wind or gravity. One is that the seismic force may enter the building from any direction, so the building must be prepared to resist accordingly. Vertical forces act as one directional (down) with occasional provision for uplift. Wind forces generally have a prevailing direction, although one would generally design a building for a wind from any direction. The second difference is that seismic forces are dynamic or moving in a random but more or less cyclical manner, and their effect is to set the building vibrating. The combination of these two characteristics greatly complicates the computation of seismic forces and the design for their resistance.

- Shear Walls. Figure 9.6 shows a building with shear walls at its end. Inertial forces caused by ground motion are created in the building, causing the floor diaphragms to move towards the right. This is resisted by the shear walls. The dynamic nature of the quake will cause a reversal of these forces, and a vibration sequence for a number of seconds depending on the nature and size of the quake. If the building is rotated 90°, it is clear that the shear walls are acting as cantilever girders supporting beams represented by the floor diaphragms.



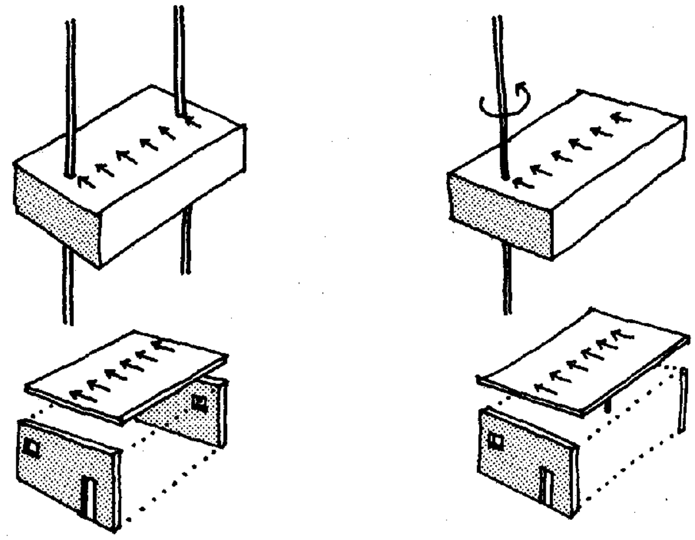
SHEAR WALLS
Figure 9.6

• Stiffness variation. Figure 9.7 shows a short and a long cantilever beam, between them supporting a load of 450 lbs. Mechanics tells us that this load will be distributed in such a way that the short, stiff beam will carry eight times the load of the longer flexible beam. This is entirely a characteristic of the geometry of the beam. If this is rotated, we have a common condition in first floor columns of buildings, as shown in the illustration. Architectural or site considerations, as shown in the small sketches, may create this condition. When the earthquake occurs, the short stiff columns will bear the brunt of the forces and will fail if not designed in recognition of this. Sometimes the forces may be so great that it is not possible to make the structural detail resist the forces. This condition also often occurs when a flexible column is stiffened by a non-structural but stiff element (such as a concrete block wall) which runs part way up the column leaving a short piece at the top to resist its disproportionate share of the load.

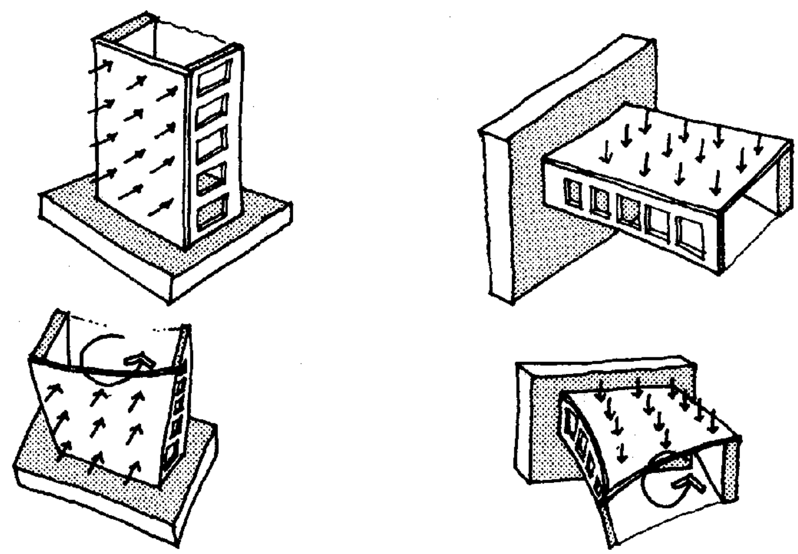


STIFFNESS VARIATION
Figure 9.7

Torsion. Torsional, or twisting, forces are created when the building rotates about its center of rigidity. This occurs when the center of rigidity of the building is eccentric to the center of gravity. In the illustration in Figure 9.8 the block has a symmetrical resistance system against the lateral force; in a building this might be on a diaphragm connected to two equally resistant shear walls. No torsion. However, if the center of gravity of the block is eccentric to the one support, then the lateral force causes rotation. The building equivalent might be a building with a strong shear wall at one end, and a weak frame at the other. Figure 9.9 shows a portion of a building with this condition. As the lateral force enters it, the building tends to twist. Rotating it 90°, and imagining that this structure cantilevers from a wall, it is easy to sense what is going to happen. These torsional forces are different to calculate, and may be very great.

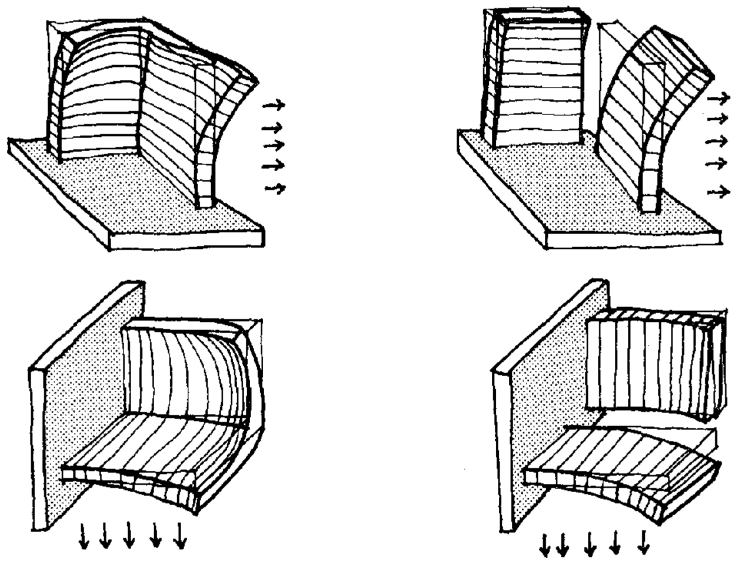


TORSION
Figure 9.8



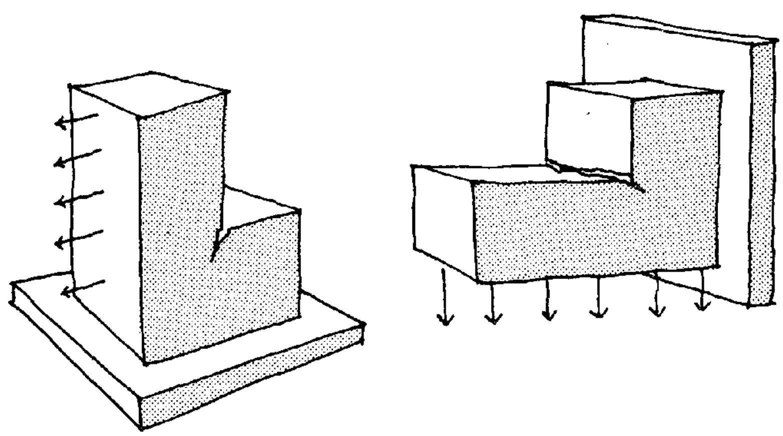
TORSION
Figure 9.9

L-Shaped buildings. L-shaped buildings are very liable to incur torsional forces. Figure 9.10 shows an L-shaped building in which the hinge of the L acts like a stiff shear wall, so that the lateral forces tend to make the weak ends rotate and create torsion. Rotate the building 90° and it is clear what is happening. One solution is to disconnect the two wings so that each building will act as a simple rectangle; the forces then are much easier to predict and compute. The rotated illustration begins to suggest the kind of resistance system necessary. Of course, if the earthquake strikes from another direction, then the roles of the two sections of the building will reverse.



L-SHAPED BUILDINGS
Figure 9.10

The Notch. L-shaped buildings have a notch, at the re-entrant corner; an identical situation also occurs in buildings with a setback, which are vertical L-shapped buildings (Figure 9.11). The notch is a weak point because there is heavy force concentration at this location. This is easy to see if the building is rotated to become a cantilever structure. If it is made out of a homogeneous material, it will fail at the notch as an overload is applied.



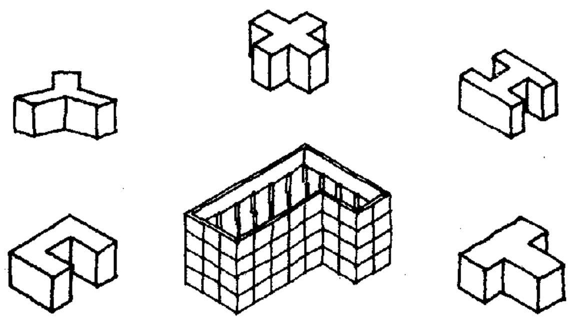
THE NOTCH
Figure 9.11

This discussion of seismic design concepts is given in order to indicate an approach to the understanding of seismic design. The selection of items shown represents issues that are particularly relevent to the study of configuration and seismic design, and these themes will re-occur in the next section that discusses seismic problems.

CONFIGURATION PROBLEMS

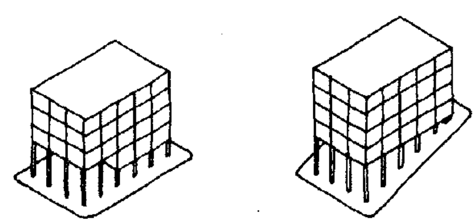
We have identified seven seismic problem areas in which configuration is a major issue. These may be grouped under three types: problems instrinsic to the geometry of the configuration as an aspect of the building as a whole; problems that relate to the disposition of resistant elements within the geometry of the building as a whole; and problems in which external or programatic requirements force a configuration that is intrinsically problematical.

Of those problems that relate to the configuration as a whole, the first type is the re-entrant corner family, as shown in Figure 9.12. This type is a problem because there is a configuration of forces at the corner, or notch. In addition, the wings of the building are restrained at the corner but free to move at their outer ends. Depending on their size and shape, this differential stiffness results in the probability of torsion. These torsional forces may be very great, and the performance of the building, therefore, unpredictable. The best solution to a large scale re-entrant corner building is to divide the building by seismic joints into two or more simple rectangular buildings whose performance will be much better and easier to analyze. But the seismic joint must be large enough to be effective or pounding will result. A seismic joint must completely separate the parts of a building, and must run through the building's interior systems, allowing for movement of mechanical systems. This can become a difficult and costly piece of architectural detailing.



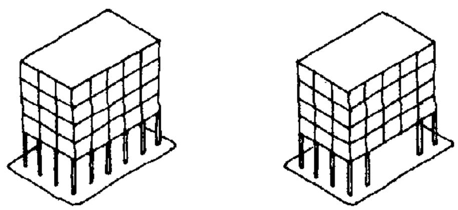
RE-ENTRANT CORNERS
Figure 9.12

The second problem is the general issue of variations of stiffness. Figures 9.13 show discontinuity of stiffness in columns that vary in height due to ground topography or other design considerations. The short columns are going to take much more of the load than the long flexible columns and must be designed accordingly. This problem, as shown in the diagrams, is a direct result of architectural considerations. In addition, the problem can be created by an interaction between architectural and structural elements. For example, when structural columns are partially restrained by a non-structural infill wall which nevertheless has the capability of stiffening the column, the result is that the short columns will recieve the major forces. The best solution to these problems is to avoid them.



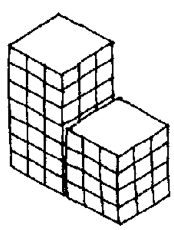
VARIATIONS IN STIFFNESS
Figure 9.13

The third problem is a specific case of discontinuity, the high or "soft" first floor. Figure 9.14 shows this kind of design which, for various reasons, is extremely popular and significant; however, it is intrinsically difficult in seismic terms, because of the major discontinuity created where the long first floor columns change to shorter, stiffer columns above. Here, too, the best solution to these problems is to avoid them or, by design subterfuge, obtain the desired architectural effect without incurring the structural problem. If these solutions are not possible, then the recourse is to use careful design and to ensure that extreme conditions are avoided. This means, for example keeping the height of the first floors as equivalent as possible to those above. Keeping the spans short (more columns) is also helpful.



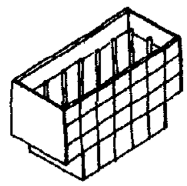
HIGH FIRST FLOOR
Figure 9.14

The fourth problem is pounding. Figure 9.15 shows a building with a setback situation, which is comparable to a vertical L-shape, or the common urban situation in which buildings are designed alongside one another. The buildings may sway at different periods, so that they pound one another and create structural damage. The solution is to allow sufficient space between the buildings; but this is often not done because of the loss of usable space within the building. The problem is similar to that dealt with in the design of a seismic joint.



POUNDING
Figure 9.15

These problems all relate to the basic configuration choices of the building as a whole. The fifth problem is related to our choice of the disposition of seismic elements within the configuration. In doing this we may convert an intrinsically simple or "good" configuration into a problem. The problem is that of the discontinuous shear wall (Figure 9.16). This is a serious problem with a known history of bad performance.

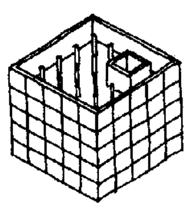


DISCONTINOUS SHEAR WALLS
Figure 9.16

The shear wall must transmit forces from the roof down to the foundation; if it is not continuous, then eccentricities and discontinuities are introduced. A major contributing cause of the serious damage to Olive View Hospital in the San Fernando earthquake was that its shear walls ended at the third floor, creating a major discontinuity at that level; the result was a concentration of forces that structural detailing could not deal with.

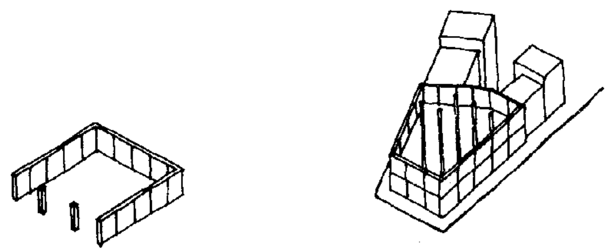
The actual failure mechanism is the major stresses introduced by discontinuity. In effect this is a special case and the most serious form of the discontinuity problem. This is because we have a discontinuity in a major resistant element which, by its nature, is generally a heavy component, which further increases the size of the forces. We can characterize a "worst case" situation by postulating a tall building with a shear wall that stops at the second floor to allow a wide span open first floor with a very high ceiling.

The sixth problem we call false symmetry. Figure 9.17 shows a building that looks symmetrical until additional investigation reveals that resistant elements are placed in such a way as to introduce torsion. For example, it has a structural core, or some other aspect, which is non-symmetrical. From the investigations which we have done, rather small variations of this sort can be very significant. Conversely, it is possible to have a configuration that is symmetrical but, through design, is dynamically symmetrical. That is, its resistant elements are placed and their stiffnesses are controlled in such a way that the centers of mass rigidity concide dynamically, although they do not do so geometrically. Not an easy thing to design, but this is, in fact, what one tries to do with an asymmetric design.



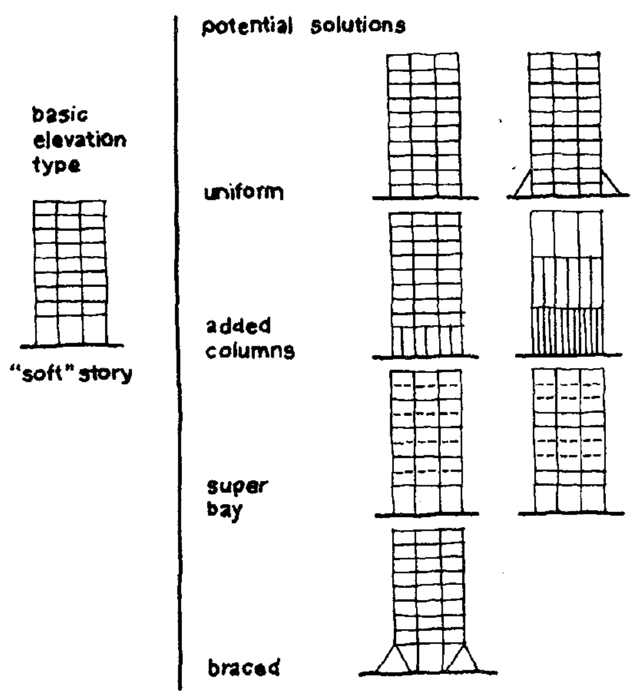
FALSE SYMMETRY
Figure 9.17

The seventh, and final problem, we call forced asymmetry (Figure 9.18), in the disposition of the resistant elements. Asymmetry is intrinsically bad because it introduces torsion, but there are occasions in which it cannot be avoided. For example, a fire station must have large doors, while the rest of the walls may be solid. Another situation is a street pattern that forces an asymmetric building with a solid party wall whether it is structurally useful or not. So this is really a kind of problem in which some external requirements or programmatic issues require us to design something in a seismically undesirable way. The main solution here is awareness, so that one can do the best design job possible within the imposed conditions. For example, one may choose to design the fire station as an open frame with braced walls and a lightweight infill in order to reduce the forces that must be resisted by the frame around the wide opening and to try to equalize the stiffnesses of the different elements of the structure.



FORCED ASYMMETRY
Figure 9.18

We are currently looking systematically at these problems; we are trying to quantify their effects and from this develop sets of solutions and alternatives. For example, in designing a high first floor, you may provide extra bracing, add some columns, change the bay size, or design a super bay, so that instead of having a stiffness discontinuity, you have a continuous stiffness throughout the building as shown in Figure 9.19. Alternatively, the high first floor may be braced. The drawing shows an example of a vertical structure that goes right down to the ground, but the architectural effect of a very high and wide first floor is achieved by placing props outside the main structure. In order to know which of these steps to take, the designer must relate formal requests to those of economy and safety, and thus needs information about these factors, information that is presently lacking. So we will try to develop guidelines that will express, in economic terms, the penalties and benefits of these alternative solutions.



NON-UNIFORM STRUCTURAL SYSTEM
Figure 9.19

SEISMIC DESIGN AS AN OPPORTUNITY

One final direction. As seismic design and configuration begin to be taken seriously, there are some architectural consequences that far from being restrictive (don't do L-shaped buildings, don't do "soft" first floors, etc.) may in fact suggest the possibility of an interesting, positive approach to design. This might be called a "new regionalism" in design. It seems right to us that when a building is designed in San Francisco, it might look rather different from a building elsewhere that does not have the same seismic requirements.

In fact, there is a long history of the expression of lateral forces elements in buildings -- not seismic forces, for the ideas of design for seismic forces had to await modern analytical methods. But the expression of lateral force resistance in a Gothic cathedral is a very important part of the imagery of that building type.

In fact, traditional designers substituted for analytical methods a very sound intuitive sense of building behavior. One of the consequences of the split between engineers and architects is that the form giver relies on the engineer's analysis and losses an intuitive sense of the forces acting on the building. It is also true that with traditional buildings in earthquake country, configuration was the first line of defense, for the materials and construction methods did not exist which make today's homogeneous and ductile structures possible. This is the major explanation for why some historic structures, which fall far short of meeting seismic codes, have survived earthquakes, often repeatedly. It reinforces the notion that, for the buildings we design, the seismic influence of configuration is worth our while to try to understand.

Chapter 10. STRUCTURAL DYNAMIC CONCEPTS

Based on material from a lecture by
John A. Blume

INTRODUCTION

Earthquakes have been occurring for millions of years, and there is no reason not to expect them to continue for many more millions of years. When they strike where man has built his structures, damage occurs, and this damage causes loss of life and tremendous other losses. In addition to the loss of life from damaged buildings, there is some loss of life from earthquake-caused natural events, such as tsunamis (waves generated in the ocean by earthquakes), mudslides or landslides. Usually the casualties are worst around villages or towns. It is almost unbelievable, when you really look back on it, that essentially nothing has been done in a technical sense about the destruction of buildings until the last few decades. The typical procedure has been to bury the dead, to rebuild the buildings in the same old way, and at the same old vulnerable places. This is still true in most parts of the world, except in this country in the last few decades. Structures can be built today to withstand very severe earthquakes with little or no loss of life and minimal damage. However, the principles involved are complex; they are not generally accepted or applied, as yet; and in places they are believed not to be necessary. When I lose patience with lack of progress in the earthquake field, I look at progress in such things as cancer, heart disease, crime, poverty, inflation, and a few other things, and then I feel that maybe we have not done that badly after all, comparatively speaking.

In this respect, architects, as designers of buildings in which people work and play and sometimes die, have a vital role in the earthquake problem. Over the decades that I have been in this field, there have been very few architects who have taken a deep interest in the problem. They have, instead, relegated to acts of God or to the building code or to structural engineers or to the philosophy, perhaps, that it cannot happen to *my* buildings. The typical earthquake philosophy seems to be, "It is going to happen to the other guy, not to me."

EVOLUTION IN THE BUILDING PROCESS

First, I would like to discuss the evolution in buildings, the types of buildings, and the types of design processes that have been going on. Back in the early 1900's, the buildings of those days consisted of massive walls, usually of masonry or stone or combinations. They may have had steel framing depending on whether they were high enough to generate a wind problem. The steel frame, if it was used at all, was designed only for wind, not for earthquakes, even in California. With these traditional-type buildings, we had the odd situation that the steel frame designed to take the wind, which could also help out in case of an earthquake, actually could not come into play until the massive architectural filler walls were either cracked or demolished in some way to allow the steel to move. The record of the buildings in the 1906 earthquake in San Francisco is of great interest, and it has had great influence on the building codes. I have made a historical study of all of the 52 major buildings that were in San Francisco at that time. Each was either tall or big or in some way an engineered, architect-type building. None of these 52 buildings had concrete walls, they all had massive brick walls on the outside, some faced with stone; they generally had lots of the partitions on the inside. Not one of these buildings collapsed, despite the fact that there was tremendous loss of life and, later on, fire damage. Some of the buildings wound up out-of-plumb. Some walls cracked, and the steel bracing, if they had it, bent a little or tore at the joints. But, by and large, the record was pretty darn good because of all 52 of these buildings, not one had come down. However, three were gutted by fire so badly that they were torn down and replaced. The 1906 earthquake had a great influence on the design coefficients that were put into early building codes for earthquake problems; people pointed to the record of 1906 and concluded that an 8.5 magnitude earthquake of close proximity in which people survived and the buildings stood, though badly wrecked, was not that bad. The problem is today the buildings are entirely different but the code coefficients are not so very different; they are a little higher and they are slowly being increased.

Today, instead of having massive noncalculated (for earthquake forces) walls, buildings, generally speaking, have very light walls. They may be of precast elements, they may be sheet metal and fiberglass, they may be mostly glass. Moreover, the fireproofing that goes around the steel columns and girders is no longer massive concrete or masonry, it is often some sort of spray-on, lightweight material. Generally, buildings today—unless they have shear walls deliberately put into them—are flexible; they are stronger than their older brothers, but they are of an entirely different character. We therefore have to be very cautious in applying design coefficients generated to a large extent upon history when we are turning around and building an entirely different type of building.

Another factor was that, in the old days, the structural engineer was almost unheard of and there was no such thing as a specialty of structural engineering and, later on, seismic engineering such as developed in California and in Japan, two places which have led the world in the development of anti-earthquake procedures.

The field today of earthquake engineering is quite complex. There is now also a lot of money for earthquake research in this country. Japan has also had tremendous backing of research money, both from industry and from government, ever since 1923—their big disaster. The Japanese came to San Francisco in 1906, learned a tremendous amount about what happened by looking at our damage, went back to Japan and started to instigate some what-we-call static building codes long before we ever did in this country. The first earthquake building code to be passed in California—which was the first in the country—was in 1933. That was a state act, a very nominal requirement of 2% of lateral force, which is in today's parlance almost nothing.

TYPES OF EARTHQUAKE HAZARDS

What are the types of earthquake hazards? I like to divide them into basic types. The first which is obvious is the rupture of a fault. A fault is a crack in the earth's bedrock, and this crack can slip. If you are building astride such a crack, no matter how strong a building is, you are in for trouble. There are ways to mitigate that trouble, but the better way is to get off the fault and to one side of it.

A second hazard is creep; where there is no sudden lurch of the ground or the bedrock, but a gradual movement. Even though it may be only a quarter of an inch or a half an inch per year, after several years, this gets to be a nuisance. In Watsonville, California, the walls of a winery are gradually shearing apart from creep activity.

A third major cause of damage is failure of soil for one reason or another due to the shaking of the ground. This may be in the form of slides (landslides or mudslides), it may be liquefaction, in which sandy or silty material temporarily turns liquid—this has caused a lot of damage—or the ground may simply settle and the building may go down evenly or unevenly based upon its mass distribution and foundation design. The greatest hazard in the United States in my opinion is generally the shaking due to the earthquake, very heavy shaking which will last from 30 to 60 seconds in a big earthquake, depending upon where you are relative to the earthquake source. This shaking is what does most of the damage.

Not to be overlooked, however, are all the secondary effects, which can be to some extent controlled in the design process. These effects include fire caused by the earthquake. For example, a ruptured gas line which is exposed to a spark or to fire may burn the whole building down or may cause an explosion. There may be a tsunami which may wash ashore and cause flooding. There may be flood from the breaking of either a natural or an artificial dam. In 1971 in San Fernando, an earthfilled dam failed, but it managed to hold the water behind it because it had been partially emptied before the earthquake thanks to sheer luck. The result was that 90,000 people downstream were not endangered. However, the dam did fail.

Another secondary effect is disease. We have modern disaster methods that are supposed to cope with this, and in most cases they do pretty well all around the country. But, with the rupture of waterlines and sewage lines and many other problems, disease can spread easily after an earthquake.

During this discussion I will address the reasons for damage. The architect must have respect for the power of the earthquake. I will also discuss some elementary dynamics and talk about building configurations and some other factors.

Earthquakes are happening all the time around the world in some place or another (see Figure 10.1). One in Japan, a 7.5 magnitude offshore event, only killed about 25 people. We had a 6.4 in Greece, near Salonika about the same time; it killed 50 people, 40 of whom were in one eight-story apartment building.

SOME MAJOR EARTHQUAKES CAUSING EXTENSIVE
DAMAGE AND LOSS OF LIFE
(NOT A COMPLETE LIST)

YEAR	LOCATION	MAGNITUDE	ESTIMATED NO. KILLED
July 1978	Tawian	7.5	---
June 1978	Japan	7.5	25
June 1978	Greece	6.4	50
1977	China	8.±	660,000
1976	Guatemala	7.5	22,868
1975	Turkey	6.7	2,385
1974	Pakistan	6.7	4,000
1972	Managua, Nicaragua	6.2	5,000
1970	Iran	6.3	5,374
1970	Peru	7.8	66,000
1968	Iran	6.3	11,000
1967	Caracas	6.5	2,666
1962	Iran	7.5	12,000
1960	Agadir	5.7	12,000
1923	Japan	8.3	145,000
1920	Kansu, China	----	180,000
1755	Lisbon	8.6	60,000
1737	Calcutta	----	300,000
1556	Shensi, China	----	830,000

Figure 10.1

It is not the size of the earthquake that counts; it is mainly the type of buildings and how hard the earthquake hits. In 1977 in China, we had a large earthquake that killed 660,000 people. That is a tremendous number; it has only been exceeded in our known history by another event in China back in 1556 in which 830,000 people died. Other very great disasters are happening all the time: Guatemala, Turkey, Pakistan, Nicaragua. In China, the reason for such tremendous loss of life is twofold: (1) they experience big earthquakes and they have great density of population, and above all (2) they have very vulnerable structures in general.

In the United States, the loss of life is small compared to that in other countries. Our buildings are generally a little better but we have been just plain lucky (see Figure 10.2). For example, in San Fernando with 90,000 people downstream, the dam that I mentioned held water even though it had failed. The time of day in most U.S. earthquakes has been a deciding factor. For example, the 1906 San Francisco quake happened at 5 a.m. when people were home in their wooden residences—which, by the way, are generally far safer than the commercial or other buildings. For most U.S. earthquakes a reason can be found for the nominal loss of life. For those who think that earthquakes only happen in California, I would like to point out the Charleston, South Carolina, and New Madrid, Missouri, quakes, which were so large that they were felt from the Gulf of Mexico up into Canada. In any country, as the damage increases, the loss of life increases. There are exceptions such as Peru in 1970, where mudslides accounted for tremendous loss of life. In general, however, the loss of life goes along with the damage.

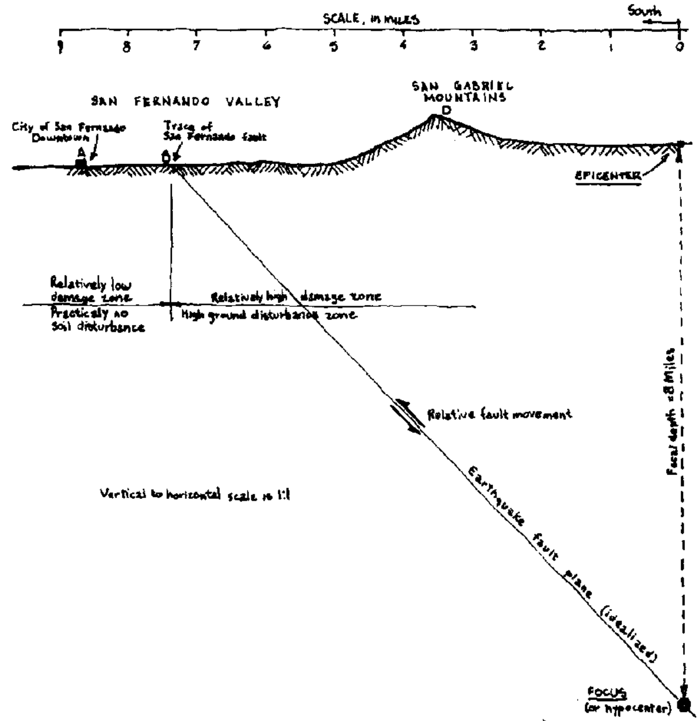
SOME MAJOR EARTHQUAKES IN THE UNITED STATES
(NOT A COMPLETE LIST)

YEAR	LOCATION	MAGNITUDE	ESTIMATED NO. KILLED
1971	San Francisco, Ca.	6.5	65
1969	Santa Rosa	5.7	---
1965	Seattle, Wa.	5.8	7
1964	Alaska	8.4	125
1957	Dale City, Ca.	5.5	---
1952	Kern Co., Ca.	7.7	14
1949	Seattle Area, Wa.	7.1	8
1940	El Centro, Ca.	7.1	9
1935	Helena, Mt.	6	6
1933	Long Beach, Ca.	6.3	115
1925	Santa Barbara	6.3	13
1906	San Francisco	8.3	700
1886	Charleston, S.C.	----	60
1811	New Madrid, Mo.	7 to 8	several

Figure 10.2

EARTHQUAKE EFFECTS ON BUILDINGS

The epicenter of an earthquake is the point on the ground above the focus as Figure 10.3 shows, or hypocenter—which is where the earthquake first happens underground. Along a fault, the vertical distance between the epicenter and the hypocenter is called the focal depth. The magnitude, which is a measure of the force of an earthquake, has very little significance from the standpoint of design. You could have a magnitude 8 earthquake happening 1,000 miles away and not even feel it, or you could have a magnitude 5 right underneath your building and it might knock you right off your chair. Therefore, magnitude taken alone without consideration of distance is not a very good index.



EARTHQUAKE DEFINITIONS
Figure 10.3

Let us start with human perception of motion in the building or on the ground. Long before you have felt an earthquake, instruments are able to record it. They automatically trigger and record it at a level much less than you feel. As an earthquake becomes stronger, the initial damage is to buildings that are in poor condition; if it shakes much harder there is initial damage to what we might call a "good" building. If it keeps shaking harder, of course the ultimate reaction is collapse of the structure. Many people think that because they feel motion they are in a major earthquake. In general, that is not the case at all. However, if the mistake of building over a moving fault has been made, and the fault only moved about one foot, the damage can be tremendous.

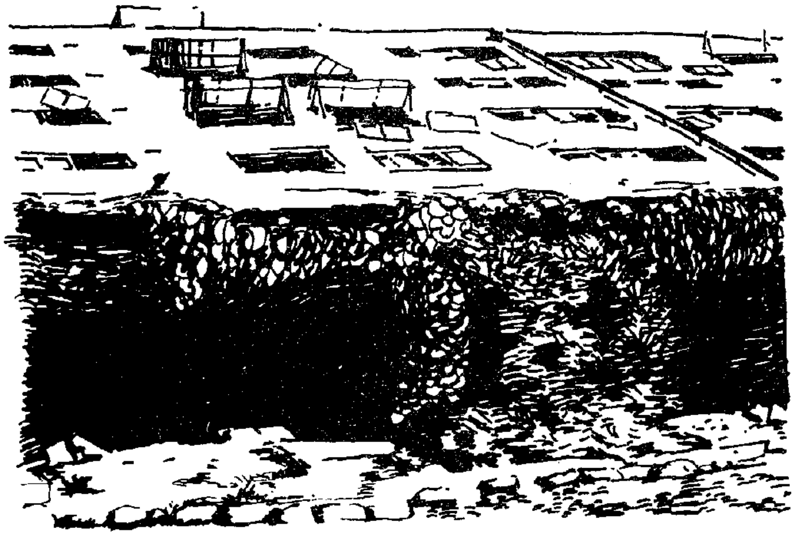
With older buildings as in Figure 10.4, even small ones, the walls sometimes fall out. Adjacent buildings may tend to hold each other up sideways, but the fronts often fall out. Chimneys usually break off at the roof line, but attached chimneys can pull off altogether away from the building. Chimneys should be well anchored at the roof line and maybe a couple of other points. They are extremely vulnerable; sometimes they fail inside the building. That can be very dangerous. Masonry fireplaces and chimneys can come apart and fall into rooms.

MASONRY WALL FAILURE -
1971 SAN FERNANDO EARTHQUAKE
Figure 10.4

In the San Francisco earthquake of 1906, some of the old wooden structures burned afterwards in the fire. The ground underneath many buildings failed, liquifying and otherwise causing settlement. But, normally speaking, if the ground holds up and if you have a well-constructed woodframe residence with a lot of partitions, it is a pretty good building in an earthquake. The reason is that wood has the great capacity to take sudden stress; it may double or triple its strength under sudden impact. Also, wood is flexible, and lightweight–but not even wood buildings can resist the effects of ground failure under their foundations.

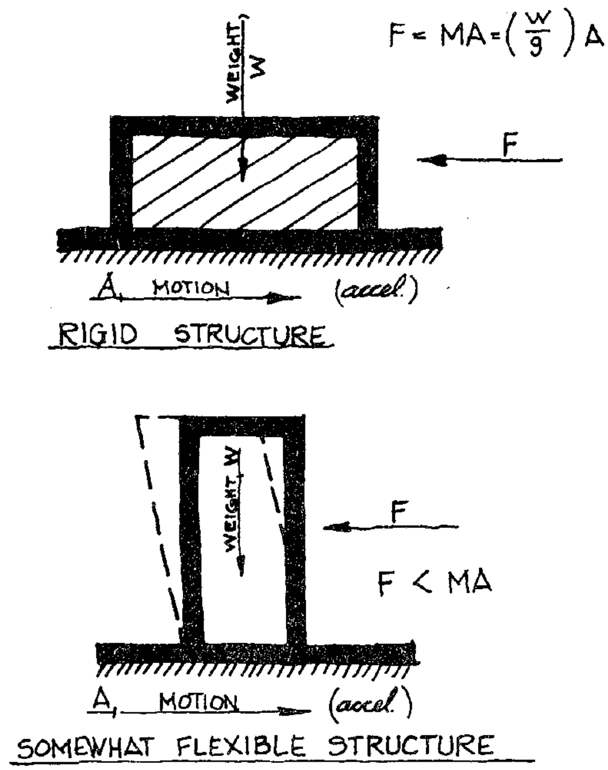
Liquefaction can be a major problem. In Niigata, Japan, in about 1970, a whole series of relatively new, code-designed, concrete apartment buildings suffered from liquefaction effects as Figure 10.5 shows. The soil underneath those buildings liquefied temporarily due to the ground shaking, and the buildings simply eased down into the liquid sand or even turned over. The damage to their superstructures was nominal, and not many lives were lost, if any, but the economic loss was unbelievable. However, there are ways of designing for liquefaction and for other soil problems, but it takes a lot more thought than has gone into it in the past.

Imagine a structure that is infinitely rigid. If the ground is subjected to some motion, the force "f" (the lateral force) according to Newton's Law would be mass times acceleration, or the weight divided by gravity and multiplied times the acceleration. It is a perfectly valid law that Newton derived. The only thing wrong with it is that when you get into most structures, they are not infinitely rigid even though Newton's Law still applies, the complexities of dynamics must be considered.



LIQUEFACTION
APARTMENT BUILDING - 1964 NIIGATA, JAPAN EARTHQUAKE
Figure 10.5

Consider the structure shown in Figure 10.6 (bottom). It is somewhat flexible, but not a completely flexible structure. It should yield as shown by the dash line—the force generally would be a little less than mass time acceleration. In a resonant situation it may be more, but, generally, it is a little less. The real answer to the problem requires complex dynamic analysis.



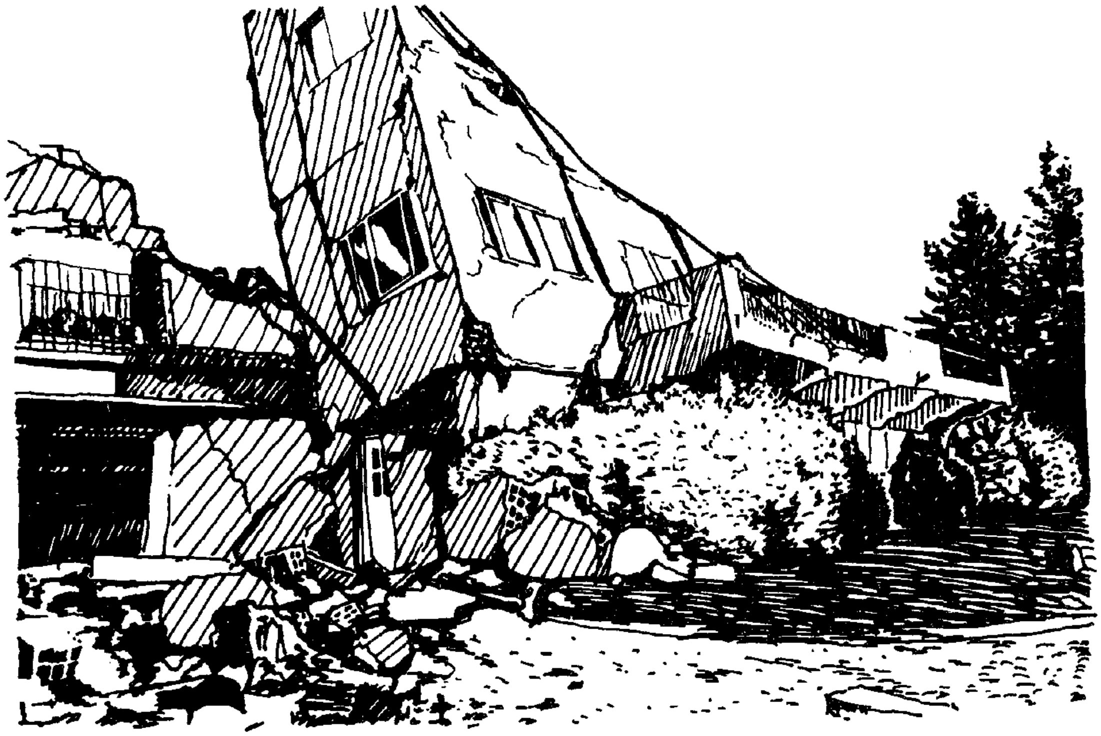
IMPULSE MOTION
Figure 10.6

So-called "Class C" buildings have a horrible history in earthquakes. "Class C" is not a designation used in the modern codes, but an old Class C building is usually a brick wall, non-reinforced. It may be a concrete wall with inadequate ties of the floors to the walls and of the roof to the walls--but it is usually brick. These buildings behave very badly in earthquakes, especially when the front is all cut up with openings for doorways or for business use. Figure 10.2 is an example of Class C buildings from the 1952 Arvin-Techachapi (Kern County, Ca.) earthquake. The entire second story had collapsed and the front of the building had fallen out.

Ancient construction in the old countries with massive masonry and rubble walls is also subject to damage. Usually, these structures are built with arches, and what happens is that the springline of the arches is pulled out a little by differential movement which opens a crack in the heavy arch and it starts to come down. The effect is cumulative. When traditional buildings with heavy massive walls with ornamentation come down, if you happen to be running out the front door as the building falls, it is very unfortunate.

DAMAGED "CLASS C" BUILDING
TEHACHAPI LODGE HALL - 1952 KERN COUNTY EARTHQUAKE
Figure 10.7

The San Fernando Olive View Hospital which failed in 1971 is famous because it was a so-called modern, code-designed building. It had been opened a few weeks before with a big ceremony. It was badly wrecked, with the worst damage in the first story columns which failed because they were not braced. While the entire upper portion of the building was seismically protected with shear walls, the walls were not continued below the second floor. The columns in the open first story had to carry the load of the rest of the building, which displaced laterally as a monolithic mass. Since the first floor had no shear protection, the displacement at those columns was destructive. The building was finally taken down, and it has been replaced at an enormous cost. In one of the older hospital-type buildings at the same place, 55 people lost their lives. (There was a total loss of life of about 65 in this earthquake.) These failures have led to a major review by the Veterans Administration of all of its hospitals in the United States.

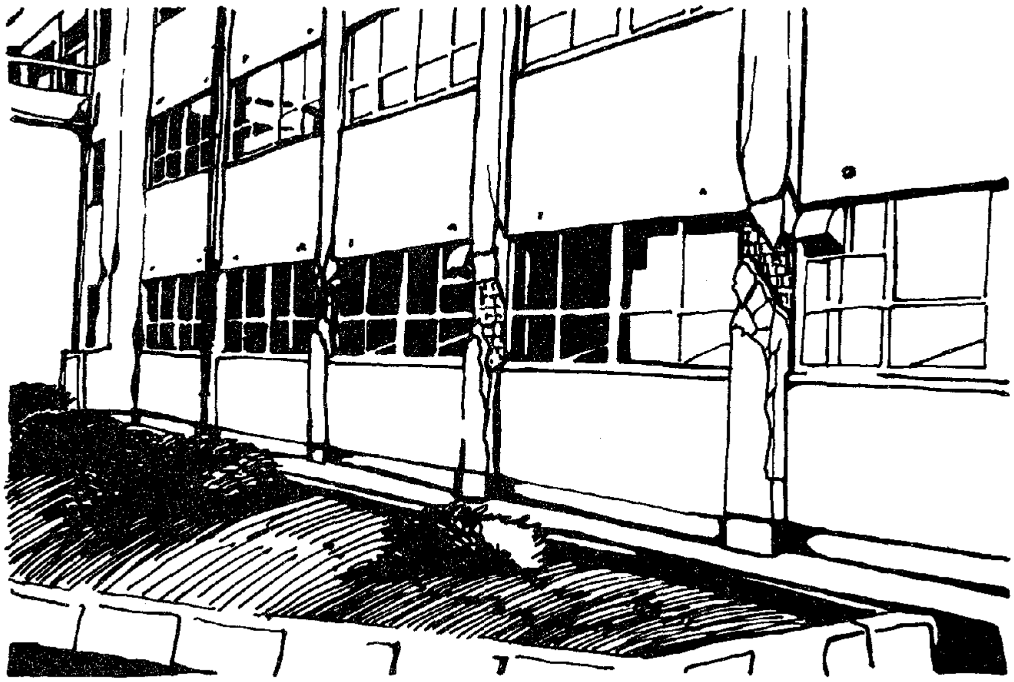


VETERANS HOSPITAL – 1971 SAN FERNANDO EARTHQUAKE
Figure 10.8

In Santa Rosa, old-type wooden buildings were often situated on what we call "cripple studs." In other words, they build a foundation and then they put in short studs to come up to the first floor level. Those cripple studs have a performance record in the earthquake field that is almost as bad as a Class C building. The most bracing that they ever put in, unless they are forced to, is a few little diagonal braces or "x" cut between two-by-fours. Nine times out of ten, the cripple stud simply collapses in a severe earthquake, and the whole house sets down a foot or two, breaking all the plumbing and all the electrical connections below the floor.

The Japan earthquake of June 1978 was only about a 7.5 magnitude earthquake situated about 100 kilometers offshore. But the ground in the region was quite soft, and the city of Sendai was heavily populated.

On the basis of column damage observed as a result of that earthquake, I wish to warn against the use of partial-height filler walls, because they thoroughly change the effects of column action. In the building in Figure 10.9, the frame was undoubtedly designed for the concrete columns to function all the way from the floor (the foundation) to the second floor connection. However, filler walls were injected part way up the building and the situation changed entirely. It is similar to the case of a traditional building, where you design a light steel frame and then jam in massive solid filler walls, except that here they jam in filler walls that are neither solid nor massive, and they do more harm than good. Therefore, nonstructural elements can be more than passive, they can become structural and alter the analyses whether or not one wants this.



DAMAGE FROM FILLER WALLS
1978 SENDAI, JAPAN EARTHQUAKE
Figure 10.9

In other classic examples of what I was just talking about, columns undoubtedly designed to be columns for a given height have partial filler walls inserted to create windows. With older buildings, an absolutely typical result is that you get "x" cracks. Sometimes the "x" only goes one way, but usually it crosses the walls. In some cases, the "x" cracks form in the spandrel or in the pier between windows, but if you have a strong earthquake and are not designed for it, it is bound to happen one place or another. Very short piers caused by very low windows tend to concentrate all of the shear at this zone, and the results can be quite devastating.

In the San Andreas Fault zone in south San Francisco, there is a landslide area near the ocean and the highway. Fifteen years ago the area was sparsely populated, but today the San Andreas Fault is still there, and so is the landslide area and the ocean, and a great deal of construction has also been added right astride the Fault. The fault moved permanently 22 feet in 1906 and we do not know how far it will move next time, but you can imagine what is bound to happen to all of the construction along that fault. Unfortunately, there are many similar situations. First of all, there should be information as to where the faults are. There should be planning and regulations so the homes and buildings are not built astride faults, but some, such areas should be turned into green belts such as golf courses or tennis courts, which would not be as vulnerable. This kind of planning should be of great interest to architects.

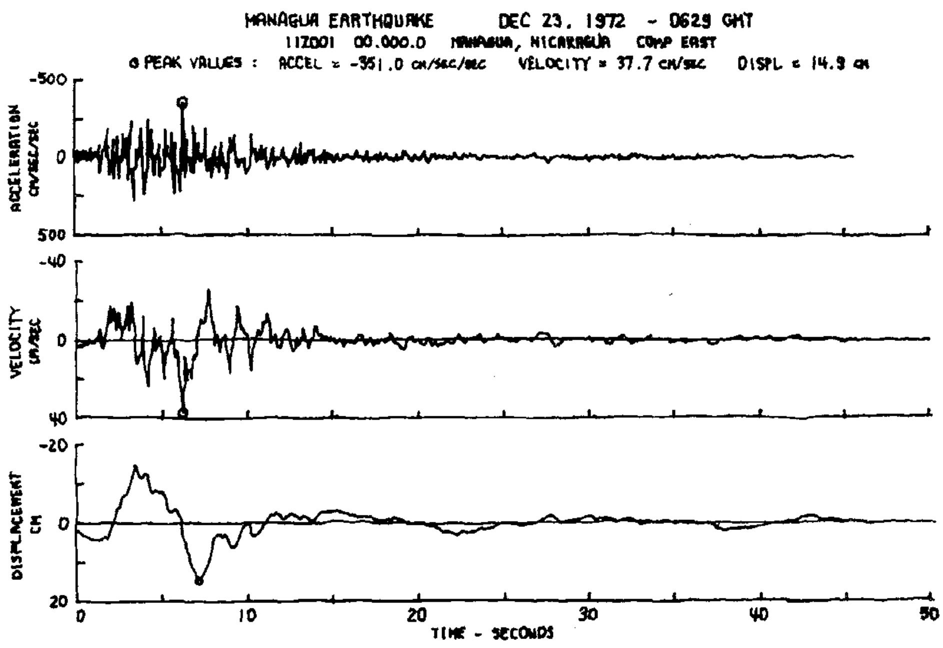
If you are going to build on top of a hill for a good view, you must give it a lot of thought. You can design the supports for a vertical load easily enough, but if you apply a horizontal force of one-fourth to one-half of the weight of the building, it is essential that the structure and foundation be capable of taking the horizontal action. Designing a dramatic house with a view on the side of a hill is possible, but it should be given very, very special attention. As they say, you should handle it very carefully—something like milking a polar bear.

There are thousands and thousands of cases where interior things fail. Ceiling work and light fixtures can come down, and such damage is not only very costly, it can be very dangerous. Most of the old codes did not contain mitigating provisions for this; however, the new codes are starting to deal with the problem.

GROUND MOTION CHARACTERISTICS

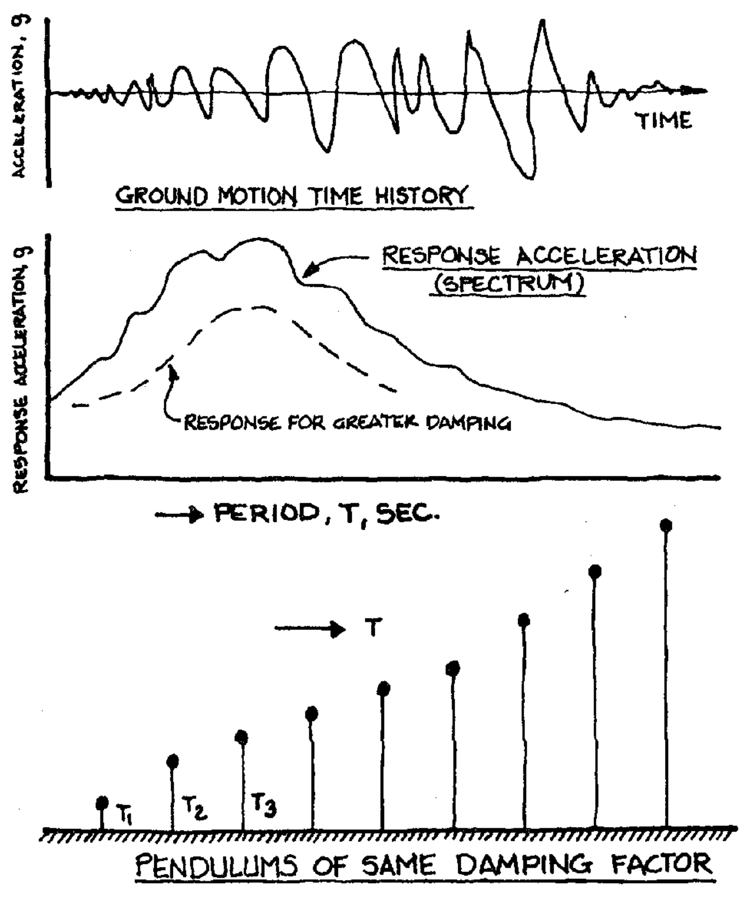
In a study of ground motion, we can make time-history records. The one in Figure 10.10 shows acceleration, velocity and displacement. This example shows acceleration the way it is recorded on most of the strong motion instruments (upper trace of Figure 10.10). By integrating with respect to time, velocity is obtained (middle trace). By one more integration, displacement is obtained (lowest trace). Notice that the zero crossings of acceleration are very numerous; zero, of course, is the horizontal line at the center. Velocity has less zero crossing, and displacement has only a few zero crossings. This is typical of most of the earthquake records.

Without getting into too much detail, I want to explain what a response spectrum is. Figure 10.9 shows response acceleration plotted against period. Period is the natural period of vibration of the structure in which one is interested. This figure is a way of plotting response spectra on log paper; one can read response, acceleration, velocity and displacement. There are different curves, of course, for different damping ratios. Damping has to do with friction and energy absorption.



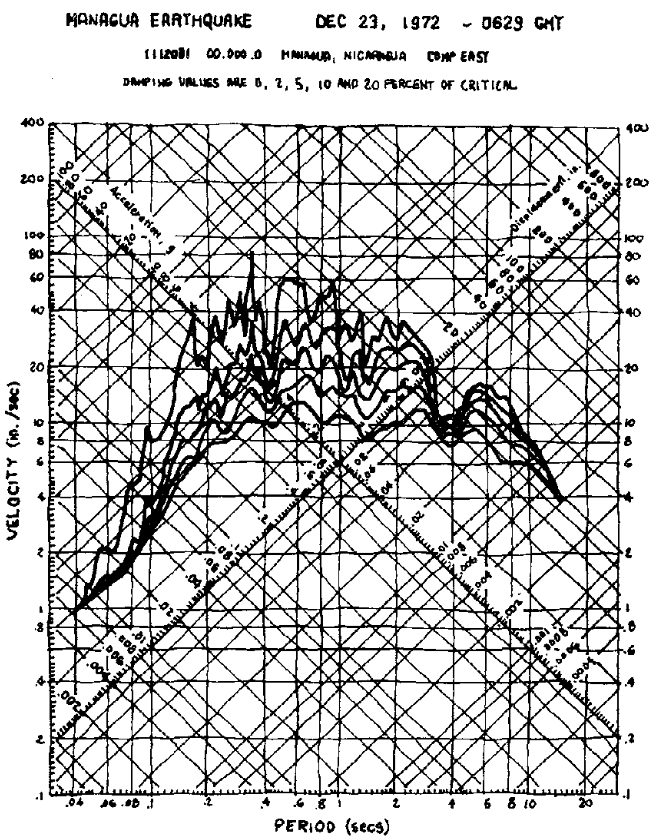
EARTHQUAKE RECORD
Figure 10.10

Figure 10.11 is a hypothetical sketch that illustrates an important principle. In the upper diagram, we are just drawing in a hypothetical acceleration time-history. Of course, this is not a real one because real ones do not ever go backwards like the one in the figure. Imagine that in the next step we take a series of lollipops (or a one mass system such is shown at the bottom of Figure 10.11), each one of which has its own period of vibration (such as the one with a period of T_2). We then subject one lollipop (which has a certain arbitrarily assumed damping value) to the entire disturbance of that time-history, and plot the maximum result from that analysis as one point. We will do the same thing for all these lollipops. By doing that for a whole band of assumed simple one-mass systems, such as these lollipops, we connect the points and draw what we call a response spectrum, which is one of the most useful tools in structural dynamics. It is very simple and yet it is somehow elusive.



ACCELERATION TIME-HISTORY
Figure 10.11

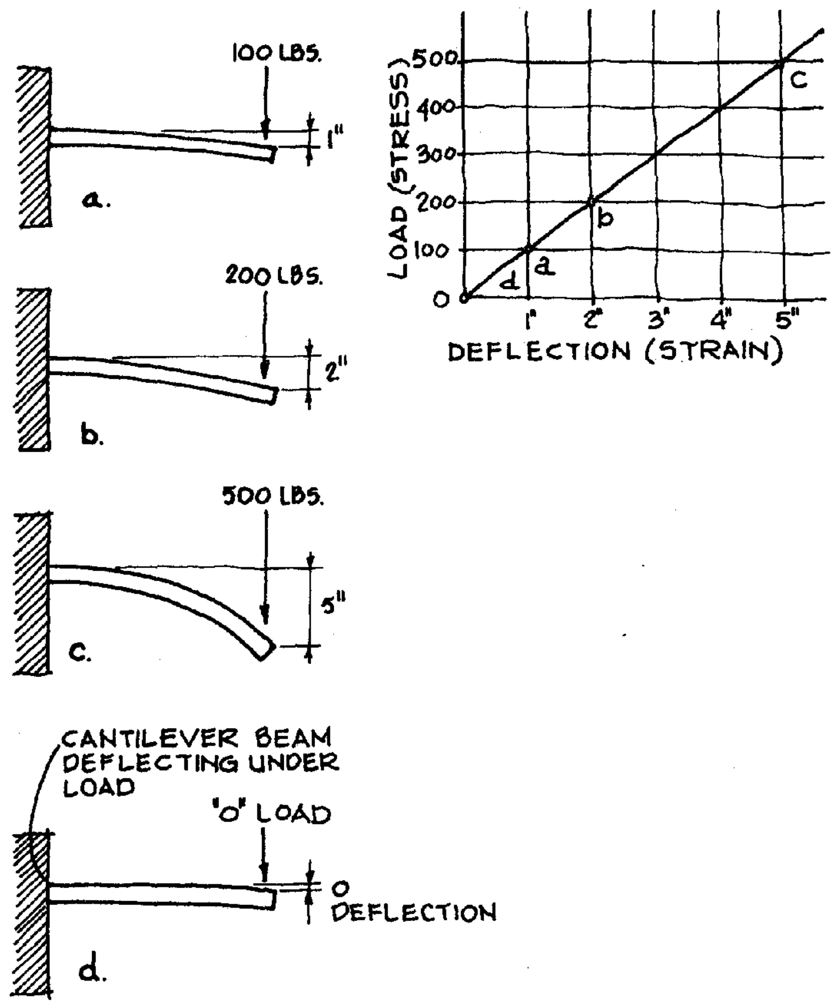
If we then look at the response spectrum diagram we can compare the diagram with a known period and desired damping value of, perhaps, a fundamental node of any structure in which we are interested and simply read the amount of acceleration that that system would be subjected to by that charted time-history earthquake. Now if the damping would be assumed greater, the curve would come down as I have shown by the dash line; if the damping were less, the curve would be higher. Damping, as I said, is a function of energy absorption. Without damping, structures would vibrate forever. Figure 10.12 is another way of plotting response spectra. In this log-paper figure, one can read response, acceleration velocity and displacement.



RESPONSE SPECTRUM
Figure 10.12

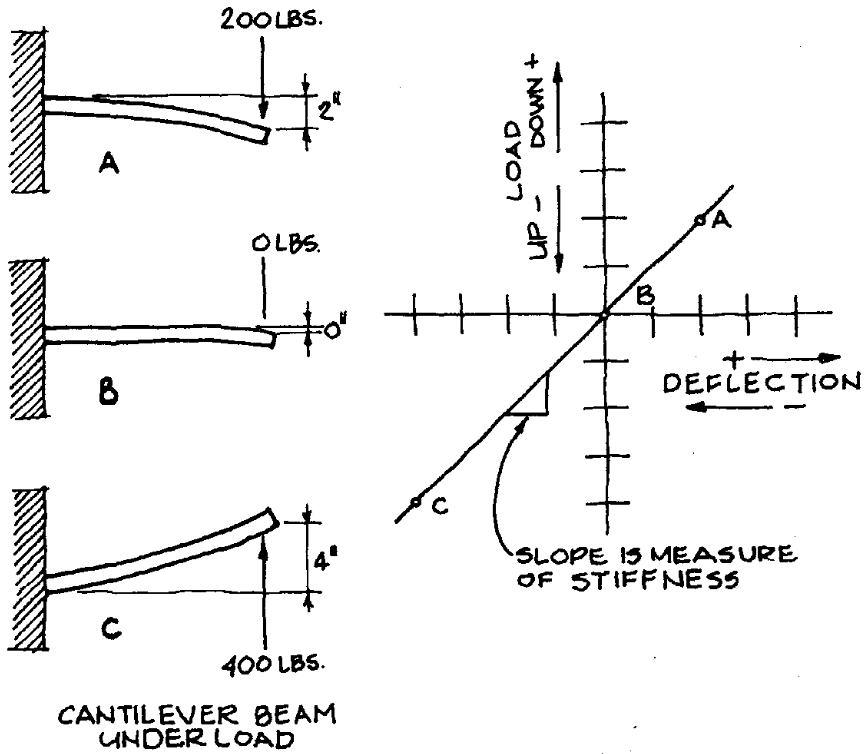
RESPONSE OF BUILDINGS TO GROUND MOTIONS

A point that may be "old hat" to most of you has to do with the principle of "Hookes Law" or the linearity of strain and load. In other words, if a cantilever (Figure 10.13) is loaded with 100 pounds, it may move down an inch; loaded with 200 pounds, it will move down 2 inches; loaded with 500 pounds; it moves down 5 inches, and so on. The assumption here is that everything is linear, and that the yield point has not been exceeded. The results can be plotted on a straight line, the load or the stress versus the deflection or the strain. That is called linear response; we assume that everything is proportional. It is a very, very convenient assumption.



LOAD DEFLECTION CURVE
Figure 10.13

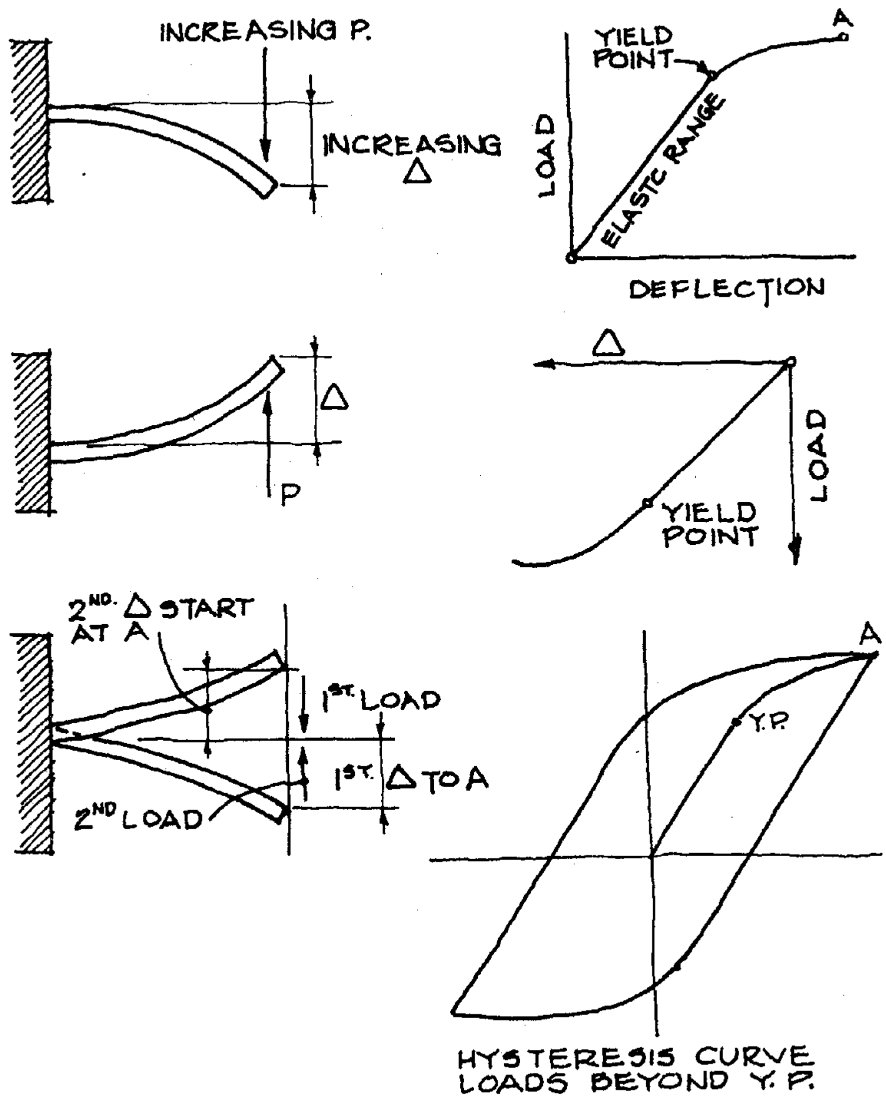
The reversal can also be entertained. Using Figure 10.14, if we not only go down 2 inches to, let us say, point "A" but, we reverse and push up to point "C;" the response is still linear. The plot of the movements can be connected with a straight line. This is what happens in an earthquake; a back-and-forth movement from the zero point.



LOAD DEFLECTION CURVE
Figure 10.14

Next, we advance into the inelastic range, which is the way that things really happen in nature. I would venture to say that there is not a single multistory building in any major city (including San Francisco and Los Angeles) that can survive a major earthquake without venturing far into the inelastic range, that is exceeding Hookes Law. Figure 10.15 shows inelastic as well as elastic behavior and cyclical loading.

What happens when you go too far with this loading is that you go beyond the yield point. You go beyond where the curve is straight, rather it starts to bend; and, when loaded in the reverse direction, it does the same thing. Moreover, with cyclical loading–the building moves back-and-forth, starting from zero, bending up to the yield point, then, past it to point "A." As the load is applied in the other direction the building motion reverses and it moves back cycling around. This is known as a hysteresis loop–the area under this curve is tremendously important because it is another measure of absorbing energy and taking away the energy of the earthquake motion.

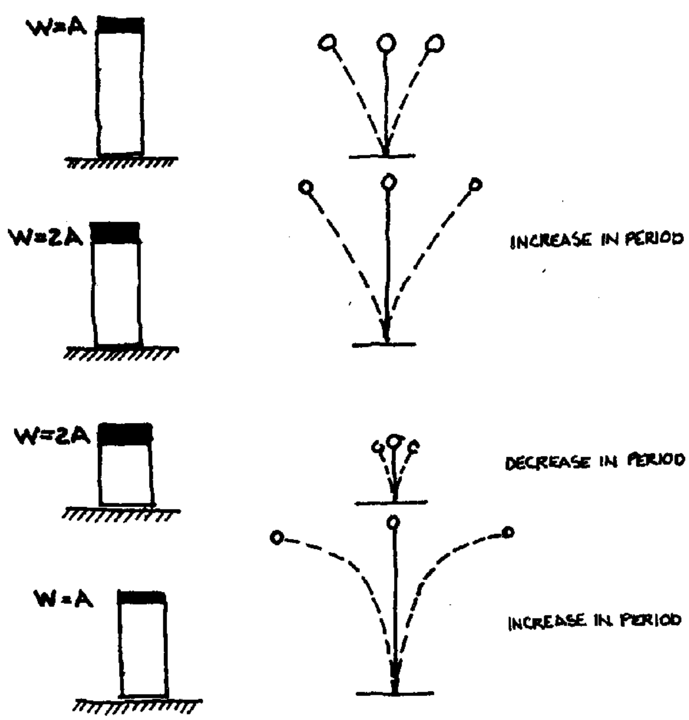


LOAD DEFLECTION CURVE
Figure 10.15

To review, we have dealt with two aspects of absorbing energy. The first is damping or hysteresis in which the energy of motion is converted to heat. (You do not feel the heat, but the affected structural member gets a little bit hotter, from the energy dissipated within it; but it is not damaged.) In the second aspect, or in the second stage, you do work because you are stretching something or cracking something, so that you are really absorbing energy. The two together, the damping and the work done, in my opinion, are the key items in earthquake resistance of 99% of the structures. Control of this inelastic situation is important so that it does not go too far and either destroy your building or kill somebody. On the other hand, if you design to eliminate inelastic deformation entirely the structure of tall buildings is going to be prohibitively expensive.

The natural period of vibration of an ordinary metal structional system is very fast, and it keeps going for a long time. The reason that it keeps going is that the natural damping of that system is very, very low. By the way, pure welded steel has very low damping; bolted or riveted steel has much higher damping due to friction. One phenomenon that occurs in earthquakes is that a tall building may respond to a distant earthquake, and the guys in a short building will not even feel it. The exact opposite response can occur where short buildings are wrecked and tall buildings hardly feel the motion at all.

The fundamental mode on a tall building changes when weight is added on top (see Figure 10.16). The fundamental mode is much slower, which is the effect of adding mass to a structure without increasing its stiffness. If I add the mass to the short one, its period not only becomes longer, but it gives it a different mode shape. (Mode shape is the shape of the vibration pattern.) If I put weight up on top—I may have a catastrophe, because weight at the top of a building, such as a water tank full of water, is much more effective than in the lower portions. Natural period becomes slow and sloppy as it approaches the inelastic range where it may actually go to failure. Mathematically, failure occurs when the period of vibration becomes infinite. That is, it moves to one side and never comes back.



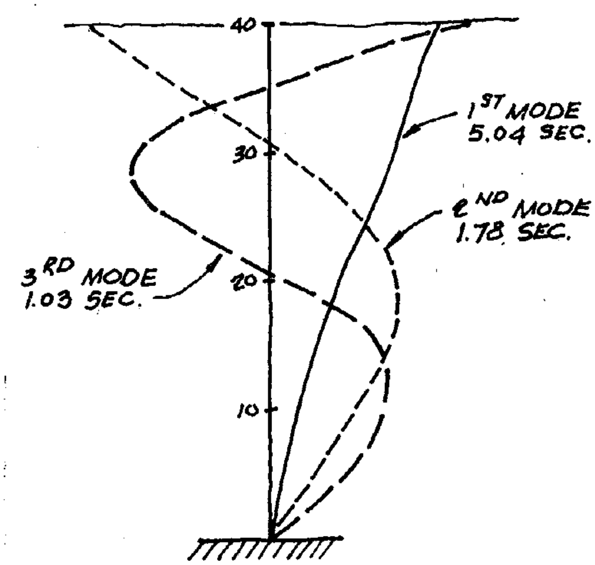
MASS/STIFFNESS EFFECTS
Figure 10.16

One other principle that I would like to discuss is the action of friction. The reason for the friction is it absorbs energy, does work, converting motion to heat. In other words, the damping is greatly increased by friction and the motion of a building under earthquake loading is tremendously reduced.

Therefore, a building with friction built in, by use of bolted connections, and other such elements, has much more damping than a structure that is all welded or just a solid piece of metal or something else. The joints, the connections, are very, very important, not only to have integrity under severe motion, but also to yield enough in a severe crisis to create friction and damping and energy absorption. Moreover, partitions and stairways add friction and increase the energy absorption capability.

An example of the effects of elements which have been added to a basic building frame upon its natural period of vibration is a building constructed in San Francisco years ago. Construction took three years between 1958 and 1960. The period of construction was broken down month-by-month. The natural period of vibration of the building was obtained by measuring periodically with instruments. This monitoring showed how the periods changed as the work progressed. For example, during early months, there was nothing but a steel frame and the fundamental mode was in the order of .06 to 1.0 seconds. Then, as the slab concrete and the spandrels and the column concrete was added increasing the mass and the stiffness the period increased considerably. When the building was completed the natural period was about 1.5 or 1.55 seconds. What had happened was that initially the steel frame had provided all of the stiffness, but as we added all of the other elements, we added mass, which made the period longer.

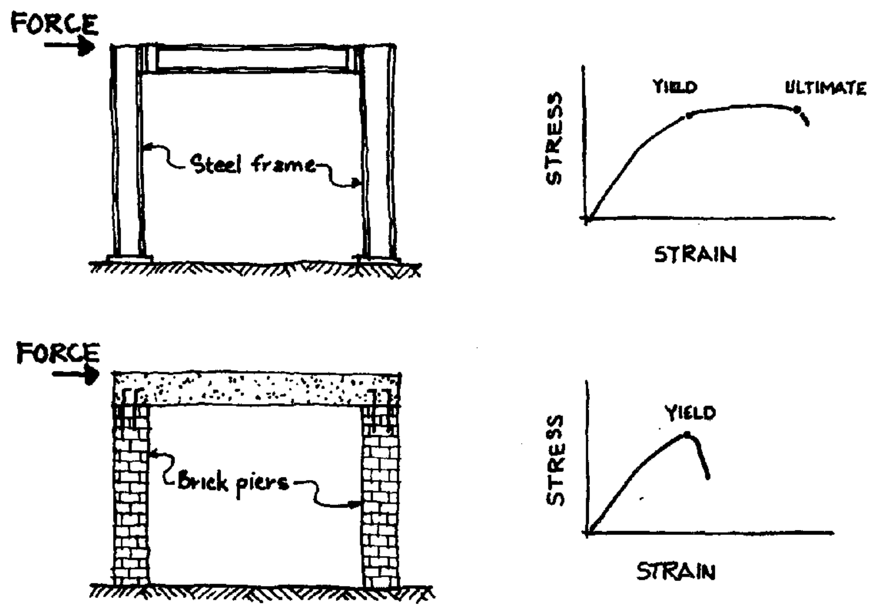
There are also second and third modes of vibration, as shown in Figure 10.17. This figure depicts the mathematical mode shapes of a 40-story building. The fundamental mode is almost a straight line though there is a little curvature. You can tell the number of the mode by the number of loops it has. The fundamental mode has really only one loop; the second mode has two, etc..



MODE SHAPE - 40 STORY BUILDING
Figure 10.17

In an earthquake, any of the modes can tune into the earthquake spectrum; it does not have to be the fundamental or the second or the third, it might be the fourth or a combination of both. Generally, the higher the mode, such as the fourth or the fifth, the less chance of it really getting started in a big way. There are very sound mathematical reasons for this.

A very important principle is ductility. Assume two structures as depicted in Figure 10.18: one is reinforced concrete or steel with a rigid floor system and flexible columns; the other is a hypothetical, brittle, strong masonry or concrete structure. The ultimate load is designed to be exactly the same for both structures. The difference is that as the flexible steel or special reinforced concrete structure is deformed up to the yield point, it then bends over and it keeps going and going and going until finally it reaches ultimate. From there on down the thing would have failed. Whereas, the brittle structure resists bending to its yield strength point, cracks wide open with a destructive deformation. We call this a brittle system, we call the former a ductile system. The ductile system has work capacity or ability to absorb energy from the earthquake, as does the brittle one, but the ductile system has a much greater work capacity. The moral is that if you are relying upon a brittle, strong structure—and you can do that—you had better allow for the extra earthquake forces that might exceed the codes values and you had better provide a great deal more strength because you do not have, to any significant degree, the tremendous work capacity of a ductile system.



DUCTILITY
Figure 10.18

Two of the most important structural aspects of tall buildings of any type are ductility and redundancy. Redundancy means that there is more than one way for the stress to be carried. There may be a whole multitude of ways. For example, aircraft are designed with redundancy; if one element fails, it spreads the load to other elements. A good seismic building should have both redundancy and ductility. It is not always specified in the codes, but it is something to know about.

I will briefly discuss the so-called shear building. The term "shear building" constantly appears in the literature. A shear building has massive floor systems and stiff floor systems; they are so stiff that you can assume that the motion of that structure deforming laterally will be entirely horizontal. There will be no bending, except local bending in the columns. There will be no rotation of the joints.

GROUND MOTION

Buildings react in an interesting way to ground movement. The inertial force that Newton discovered, which is the tendency for objects at rest to remain at rest means that the building lags behind the directional ground motion as though a force has been created that is pushing the building backwards. Then, of course, you get the reverse motion which makes the building move back and forth, though always tending to lag. Tall buildings sway because of this lag much more so than short buildings because so much of the mass of tall buildings is "cantilevered" above the ground.

Imagine a hypocenter right at that point where either an explosion or an earthquake centers. Theoretically at a distance, a tall building feels the motion and is moving back and forth; whereas, a short building does not feel it due to the periods of vibration—and the fact that the longer periods tend to show up at a greater distance and the shorter periods at short distances. The reverse would be true closer to the hypocenter where a tall building would probably not feel much motion and a short one would be shaking very vigorously.

We have been monitoring all the tall Las Vegas buildings for years for all the underground shocks, predicting in writing what will happen and what will not happen, measuring during the motion, and coming back afterwards and writing another report to find out any errors in our ways. It has provided a vast amount of good information for the earthquake field.

When we plot building motion as measured under different disturbances—under natural earthquakes, under windstorms, under force vibration with machines, placed either up in the building or down in the basement—we find that regardless of the source of the disturbance, the fundamental mode is just about the same. There is some variation, but it is about the same, regardless of amplitude. In other words, the disturbance itself does not upset the laws that I have been discussing.

If we took plan views looking down at the top of actual buildings during an earthquake, we would find—contrary to popular belief—that the motion is not back and forth in one direction. It usually is sort of an orbital path. Looking back to Figure 10.15 we see the mathematical mode shapes of a 40-story building. The fundamental (first) mode is almost a straight line although there is a little curvature. The second and third modes are also shown. You can tell the number of the mode by the number of loops it has, the fundamental mode has really only one loop, and the second mode has two, and so on. As I stated earlier, any one of these modes can tune into any earthquake motion and any combination or they can do it simultaneously.

A code design coefficient is slightly different than an earthquake acceleration, depending upon the dynamic characteristics of the structure. That, however, accounts for only a small part of the difference. The main difference—and the reason that structures may or may not survive, and most will—is what I have been talking about before; ductility and redundancy. If they have these qualities, the actual shears which are drawn for the theoretical elastic condition, will be significantly reduced. There will still be some damage. No code intends to prevent all damage; it would be an economic disaster if they did because of the tremendous cost of "total" protection. The intent of most codes is to prevent loss of life and hopefully injury. The economics of the situation, if an owner is really concerned should be a special study—which is beyond the score of the code.

We are able to reproduce recorded earthquake motion using models of the affected buildings. In the case of a Holiday Inn that went through the San Fernando earthquake, we modeled the recorded motion in one direction, horizontally. In another model we changed the damping from 5% damping to 10% damping. For the horizontal test there was quite good correlation between the response of dynamic model and the actual earthquake. Such tests provide sort of a feedback situation. In the other direction, the correlation was not so good, although it was not so bad either, but there were some differences that would have to be reconciled. NSF has funded research in this area.

EARTHQUAKE RESISTANT DESIGN

I would like to make a couple of very important points. In the older Uniform Building Code, the base shear "V" is equal to "ZKCW," where "Z" is the earthquake zone, indicated on a scale of increasing risk, 0, 1, 2, 3, or 4; "K" is a coefficient that is supposed to represent ductility and some degree of redundance; "C" is the coefficient which is a function of the fundamental period of vibration; and "W" is the weight of the structure. So "V" is applied to the base of the building, and then up and down in an inverted triangle shape. Now, if you have a moment-resisting frame alone, the Code allowed a "K" of only 0.67 in the old days, which is a reduction from the typical situation. If you have a shear wall or a box system, it is 1.33, exactly double—sort of a penalty that is given in the Code for the fact that you do not have a moment-resisting frame. If you have a combination of moment-resisting frame and shear walls, the Code allows a "K" equal to 0.8, which is in between these two. There are many cases where the building may be better than this; but in both cases, 1 and 2, it is necessary to design the frame for the lateral forces, even when we inject shear walls that may be more rigid than the frame, and that may try to take on the work, you still have to design the frame to carry at least 25% of the lateral forces. Many people think that it is very silly to duplicate. The rationale is to get redundancy and reserve back-up so that when the shear walls crack, the building is in no danger of collapse.

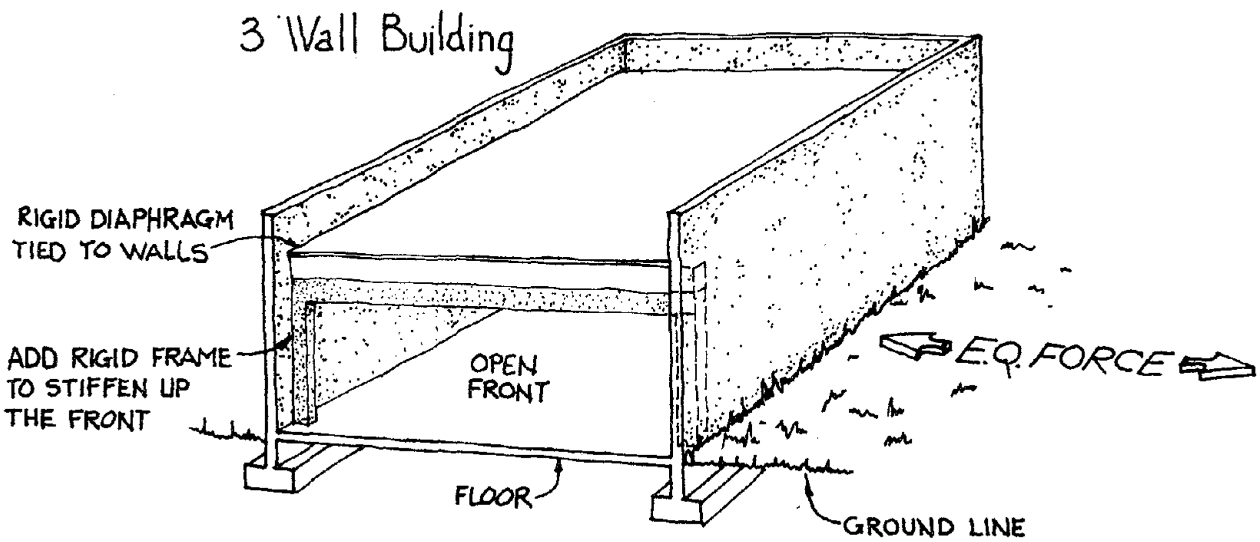
By the way, the new codes will have other factors: "S" and "I:" "S" for soil conditions and "I" for importance factor.

When you plan to put a building on a situation that is geologically very poor, such as on a hill that is subject to landslides, remember that earthquakes often trigger landslides. There are various possible ways that this might be mitigated. One would be to put in a massive retaining wall system, and also drain it. Putting in a complete subsoil drain system, which makes the friction go up in the sand or soil particles, is very effective. A combination of a wall with tie backs or rock bolts that go back into something solid can be used. The whole point is that siting of a building is very, very important. If you have that condition without realizing it, you will like what has happened in Managua, Nicaragua, a couple of years ago; the most expensive residences were all built on the edge of cliffs and bluffs to get the beautiful view and a great many of them went down those hills.

Liquefaction is another potential hazard of geologically poor sites. Sandy or silty soils that are both loose and saturated with water are subject to turning fluid under earthquake motion. There are several ways to get around that problem. One way is to drive piles through the materials down to a solid bearing. Another way is to deepen the basements. Another system is to inject chemical grout to stabilize the soil under the building. And yet another system is vibroflotation wherein you can pack the sand to such a density that it cannot liquify.

Symmetry is extremely important in earthquake resistance, because even with a perfectly symmetrical rectangle, you will still get torsion. We know that now; we used to guess at it. You will get torsion for two reasons: (1) the arriving waves in the soil come at such directions and angles that they induce torsion in a large foundation even though the building is symmetrical; so that in addition to the building moving back and forth, it is going to be twisting a little bit. The code allows a 5% accidental torsion, which may or may not be enough in some cases. When you build a T-shaped building, stress concentrations can be very, very high—the reason being that the stem of the "T" will be vibrating in its own mode back and forth, while the cross of the "T" is going in another mode, and the joints tend to part company. Any abrupt change in plan is a point of concentration of stresses. It does not mean that one cannot design buildings that way, it means that one has to be careful.

One of the world's most common building is a commercial-type with an all glass storefront, an example of which is shown in Figure 10.19. That type of building can be found in every city in the United States; it is one of the worst performing building types during an earthquake. One way to increase their resistance is to provide a very solid diaphragm for the roof and the floors, well-connected all around, and to provide framing across the front that is as strong and as rigid as possible without making it difficult to install the glass. In effect, you have created a bent across the front that will make up, in part at least, for the lack of wall (See Figure 10.19).

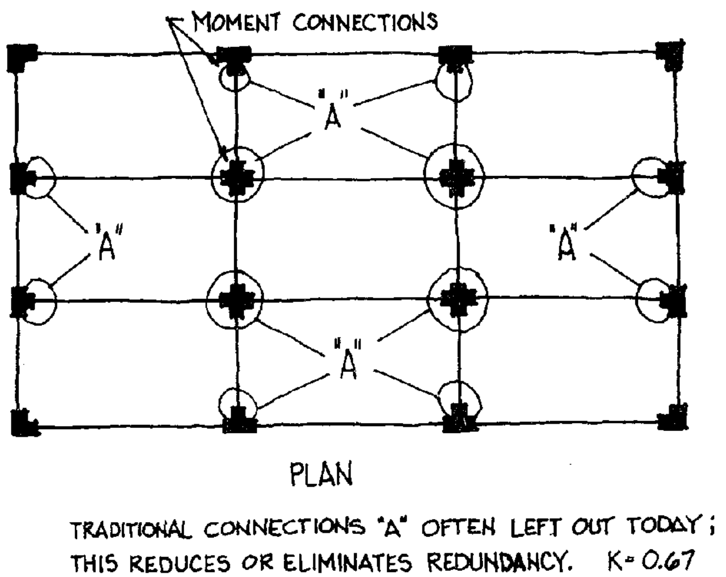


SEISMIC RESISTANCE OF FAMILIAR BUILDING TYPE
Figure 10.19

Let us look at buildings from their elevations. There are the conventional symmetrical buildings, low, rigid ones and tall, slender ones. We have many buildings with setbacks in earthquake risk areas. These are entirely possible, but in the setback situation, you have stress concentrations that have to be watched very carefully. You can have multi-setbacks like the Rockefeller Center buildings in New York or pointed buildings like the Trans-America Tower in San Francisco. Almost any shapes are possible, but when one gets away from the conventional block shapes, it sometimes is desirable to do a dynamic analysis in addition to ordinary code analysis. In fact, even if you are dealing with a very high, slender building, some cities such as Los Angeles now require a dynamic analysis in addition to the code requirements.

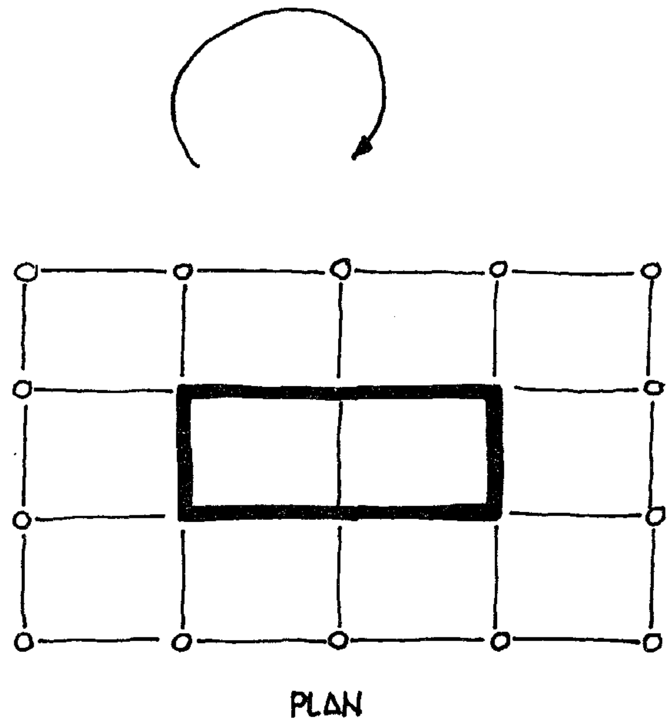
There is also what I call vagrant architecture, which means that it has no visible means of support. I have seen some, especially in foreign countries. Usually a massive block, with or without windows, is supported by skinny little columns. In order to get stairways and elevators in there may be some sort of a core in the middle, but that core is supposed to do all of the work. These buildings can be hot potatoes in an earthquake unless they are very specially designed.

Figure 10.20 is a framing plan of a steel building. When the codes were first written, they were based on the type of buildings that went through the San Francisco earthquake of 1906, and a few other earthquakes. Then steel frames had moment connections all around the outside and also on the inside joints—both ways, the theory being that the wind would be carried by those joints, if nothing else. Not all buildings were that way in the old days, but a great many of them were. The code today says that if you have a steel frame, you can use "K" equals 0.67 and get a big reduction in your design forces. The designer to delete every moment connection marked with an "A" in the figure and all the circled ones. This limits the moment connections to the outside, without the interior joints. The interior joints would be merely shear carrying or vertical load carrying, without any moment capacity. It is entirely possible for this design to pass a code in which the coefficient "K" equals 0.67. What you have done, even though you have passed a code, however, is to reduce your redundancy. The building may be sufficiently protected but in a severe earthquake I would like to be in a building that had more redundancy.



STEEL FRAMING
Figure 10.20

In a typical core building (see Figure 10.21) the core usually encase the utility shaft, the elevators, and the stairways and have only simple connections to the outside columns. Many buildings are built this way and they can pass a code. The code requires them to put in an artificial torsion of an assumed 5% eccentricity. In some cases that 5% will not be enough, especially if the core is small relative to the overall plan and if there is absolutely none or very little moment capacity in these outside connections. There is going to be torsion in such a building whether it is symmetrical or not (1) because of the ground waves that I spoke of earlier and (2) because when the building goes into inelastic response—which it has to survive—it will not do so symmetrically. One side may go a little more than the other, and so on, which creates a torsional condition sooner or later. Moreover, the polar moments of inertia—of a building like this is much smaller than for a building that has resistance around its periphery.



CORE BUILDING
Figure 10.21

It is possible to do a risk analysis of any building problems, if the owner is really interested, starting with the frequency of the earthquakes in a given locality and working on up to the probability of damage and what it would cost to reduce those probabilities. We do this all the time for nuclear power plants. Nuclear power plants are designed for ten, twenty, thirty, and even forty times the forces of ordinary buildings.

The procedure is to take the magnitude and to plot the number of those magnitudes that occurred and when they happened. For example, one would plot all of the magnitude 5's, magnitude 4's, etc., on a chart which would show the frequency of those magnitudes. An equation will give the frequency of those magnitudes. An equation will give the frequency of recurrence of any magnitude earthquake.

We can look at data of actual damage to small and primarily residential buildings. The damage ratio is shown in Figure 10.21 on the vertical scale. The horizontal scale "E" represents the envelope of the spectral acceleration. Because there may be several horizontal components of spectral acceleration, the greater one is plotted here. Foundations have very minor damage, windows have more, and so on as indicated. A very severe ground motion of 0.6 g produces only about 4% damage to windows, about 8% damage to exterior walls and about 12% damage to other things. Interior walls receive about 22% damage. Chimneys top the list. The overall damage at 0.6 g is about 40% to 50% of a building. Now, 40% to 50% of the building is a tremendous amount, but that is for a 0.6 gravity earthquake, which is a tremendous earthquake. This type of information which is very, very beneficial, not only to insurance companies, but also to owners and architects and others to decide what the risk is and what to do about it is being developed.

There are various types of walls on buildings; their shape, their stiffness, and how they are attached to the structural frame is very important. Walls can actually control both the dynamic and the structural properties of a building until such time as they might fail. The ratio of the horizontal stiffness of all the vertical members—walls, columns, shear walls and partitions is an important design factor. The results of a very comprehensive study made some years ago determined that if this factor (called RHO) is a high number, you have what is called a shear building. The structure of many buildings consists of nothing but a thin slab and vertical elements. These act as cantilever buildings; after walls have cracked they are no longer shear wall buildings at all; they are cantilevers, vertical cantilevers. At the other extreme you have a cantilevered building, where that RHO factor is extremely small.

All buildings have different characteristics in the way they respond dynamically and in the way they respond structurally. The important thing to note is the great importance of the wall configuration which really controls the dynamic frame response during an earthquake situation.

There are many details in construction which can help mitigate seismic hazards. The main thing is to tie everything together; anchor bolts to the foundation, use proper blocking, plywood should be well nailed. Masonry is usually reinforced now in earthquake country. The grouted brick wall is now very popular; it results in a good bond. Some people like to use ties in addition to bond, but if the wall is properly built, the bond is enormous. To reinforce a block wall you must fill the cells in the blocks. The object of the reinforcing bars, believe it or not, is not necessarily to act as a beam, but to hold the pieces together in case they crack. That is the philosophy behind it. Masonry work has to be inspected continuously, to insure that you get good work for "earthquake country."

The retrofitting of old buildings is a very, very important subject. Most cities have the greatest hazard from old buildings—such as buildings with high parapet walls of unreinforced masonry, or old buildings with fire-cut joists without any anchorage. Such details can be improved with ties added later, for example to try to tie roofs to walls. Parapets can be cut off and concrete copings put on to try to tie things together. The general objective is to provide integrity, tie everything together so it will remain as a unit.

If you must design heavy masonry ornamentations as the designers of the past often did, make sure that they are not only anchored to start with but they they will remain anchored over a long period of time and that the anchors do not corrode away. When such ornamentations fall (parapets, cornices, etc.) they can kill many people during major earthquakes.

According to rough figures from a study I once did on an occupancy factor for various buildings, theaters, churches and arenas are actually occupied only about 4% of the time; schools and colleges, maybe 20%, if you allow for weekend, holidays, vacations, sick leave, and other factors. College classes last only 50 minutes or 1 hour; churches are often occupied only Sunday mornings. Thus, there is a very small occupancy factor.

Warehouses generally house only the people working the warehouse. The occupancy factor for hotels and large apartment buildings is 36%; for hospitals it is 60%—it may be higher now; for family-type residences it is 62%. Jails seem to get the highest occupancy. You may wonder why it is not 100% for jails. The answer is that a lot of people work there during the day and go home at night, wardens and guards.

Why are these figures important? They are important probabilistically because the earthquake can happen any time of the day or night. So you are playing a difficult game. If, because of the low (4%) occupancy factor assigned to a theater, a designer uses lower design coefficients, he has overlooked the fact that that theater could collapse during a performance causing a tremendous loss of life. Thus, these things have to be balanced against the consequences.

CONCLUSIONS

My philosophy is that seismic-resistant buildings is not just the sum of its component parts. It has a character of its own from having those parts so tied together to make the building act as a unit and remain as a unit under forces and distortions much greater than code requirements imply. This means great attention to detail both in design and in construction. I think that is most important.

A brief note about the cost involved. With the people building configuration right from the start and early participation of engineers working closely with the architect, the cost to the owner does not have to be much greater to provide earthquake-resistance than to not provide it. It can be a very small percentage of the total cost, a very, very, very tiny percentage.

On the other hand, if the configuration is bad or if the engineering starts too late, the construction cost can be appreciable and also the costs of architectural and engineering services. With structural dynamic concepts and the other disciplines that constitute modern earthquake engineering, it is entirely possible today to design structures of any type to resist severe earthquakes with nominal damage and, hopefully, with very minor if any loss of life and injury.

Architects can play an increasing role in earthquake safety. With good planning, siting, and sound building configuration, we can get better seismic-resistance with less cost to all concerned. Moreover, the architect should choose his team to ensure that the architectural materials and the architectural elements, as well as the mechanical and electrical installations, are not hazards in themselves and are not going to be subject to very costly damage in an earthquake even though the structure may remain intact. It is a sad story when there is no structural damage and yet it is going to cost you 40% to 50% of its value to fix the building up after an earthquake. Seismic resistance can be cost effectively designed into buildings. In time, I think, it will be expected to be done all across the country.

Chapter 11. EARTHQUAKE RESISTANT DESIGN

Mete A. Sozen, P.E.

EXAMPLES OF STRUCTURAL DAMAGE CAUSED BY EARTHQUAKES

In this discussion I should like to lead to certain simple concepts for understanding the response of building structures to strong earthquake motions. However, before I get to those concepts, I should like to give you a few examples of recurring types of structural and architectural damage.

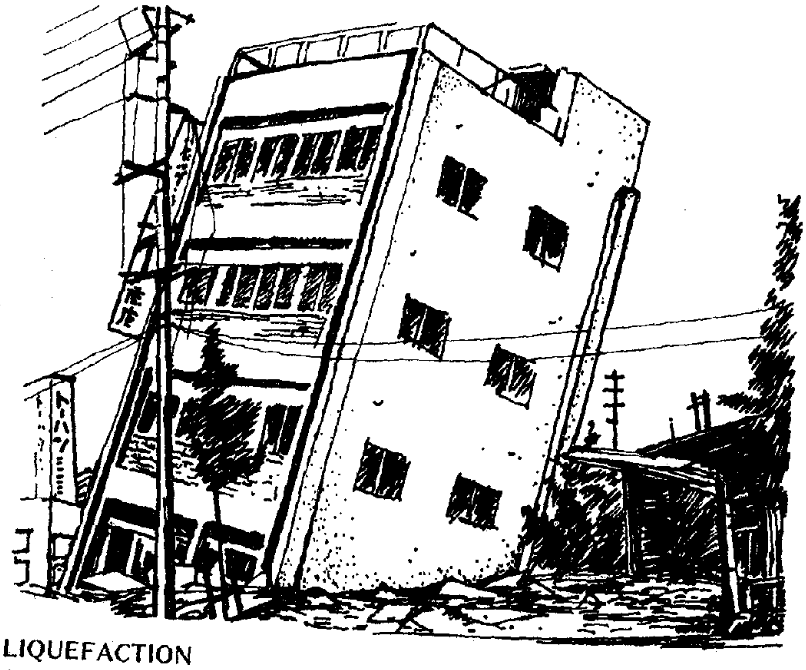
In considering structural damage, the first thing that comes to mind is what problems we encounter at the building foundation. Contrary to what has been popularly said in the press about the ground opening up and swallowing whole buildings or that entire states have disappeared under water there have been no written records of such holocausts. The problems we have with "ground" can be classified in two categories: failure of ground motion estimates and actual foundation failures.

The first problem can be a human failing and need not be discussed in the context of this paper other than by the comment that a conservative and inflated estimate of the ground motion is likely to cause more damage to a society than an under estimate. In the latter case the damage comes from the earthquake which is not a predictable certainty. In the former case the additional investment in safety (or the lack of it) is a certainty.

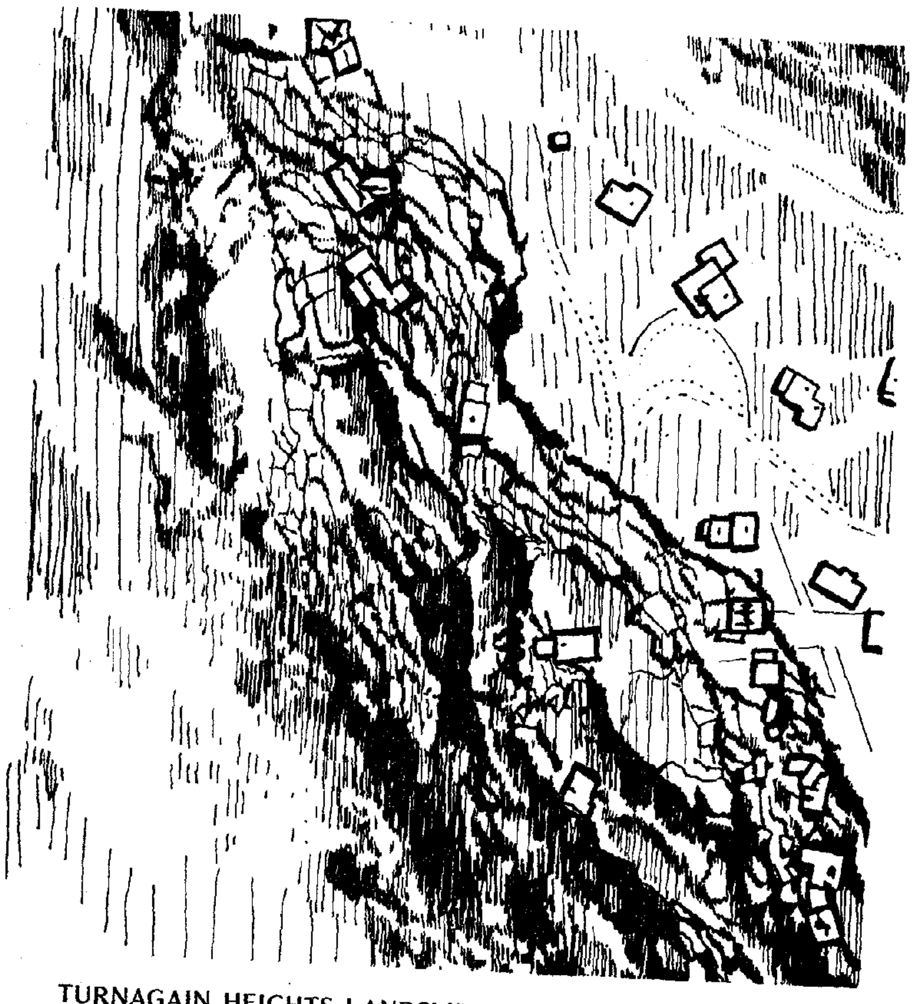
An impressive example of foundation failure occurred during the Niigata (Japan) earthquake of 1964. If you dump a handful of wet sand in a tray and shake the tray, the sand will behave like a viscous liquid. It will flow. In the case of Niigata, this phenomenon occurred over virtually the whole city. During the shaking caused by the earthquake, the city was like a lake with buildings behaving like floating vessels. Structures with reasonably uniform distribution of foundation pressure sank. Eccentrically loaded structures simply tipped over.

The silver lining to foundation failures is that, within the current state of the art, they can be anticipated in most cases, provided of course the building planners have been wise enough to seek geotechnical data. The Turnagain landslide which occurred during the 1964 Alaska earthquake (see Figure 11.2) had been anticipated by the U.S. Geological Survey, but this had not deterred land developers from locating a subdivision of Anchorage in that area.

Turning to structural damage caused directly by strong shaking, I should like to start with the experience we have had and we shall continue to have because of buildings which have been and, I am cynical enough to think, shall continue to be built without any consideration of earthquake effects. The salient example in this sad category is the unreinforced masonry building. One does not have to be an analyst to realize that something is going to go wrong with a building that carries a large elevated weight on weak brittle elements. I do not wish to generalize to the extent of implying that no masonry building is likely to survive an earthquake. In many cases unreinforced masonry can come through strong motion with relatively little damage if the story masses or weights are light and the walls do not contain many openings.



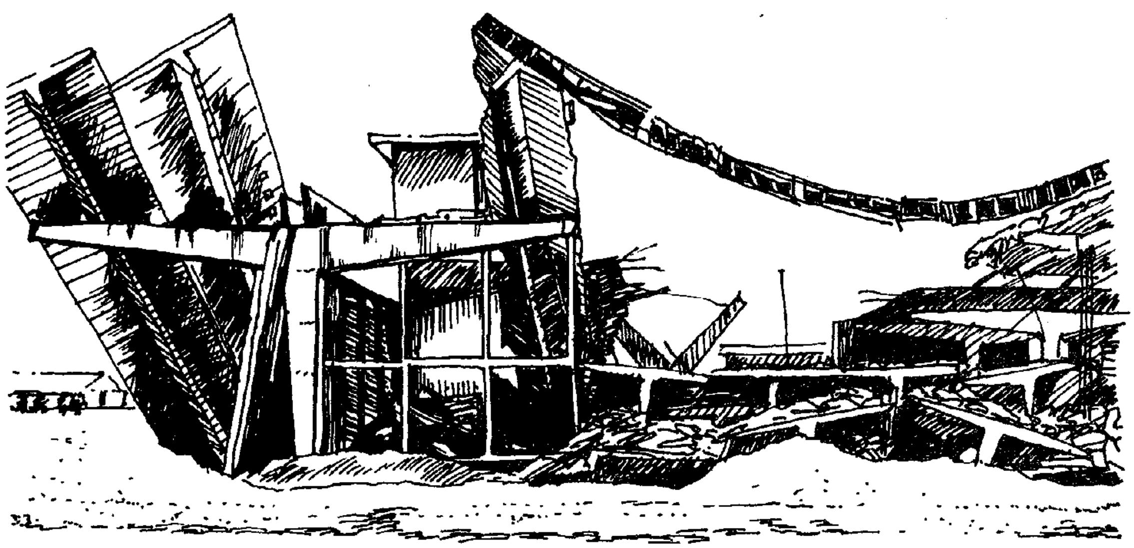
LIQUEFACTION
OFFICE BUILDING - 1964 NIIGATA, JAPAN EARTHQUAKE
Figure 11.1



TURNAGAIN HEIGHTS LANDSLIDE - 1964 ALASKA EARTHQUAKE
Figure 11.2

INADEQUATE MASONRY
CALIFORNIAN HOTEL - 1925 SANTA BARBARA EARTHQUAKE
Figure 11.3

One can say very few things absolutely in the earthquake trade. However, I would venture to say that if you have a building with heavy floor and roof weights, unreinforced masonry walls with many openings, it will go down if it is subjected to strong earthquake motion. One of the major problems confronting the building profession in seismic regions is whether to strengthen, demolish, or accept the risk and live with existing unreinforced masonry housing.



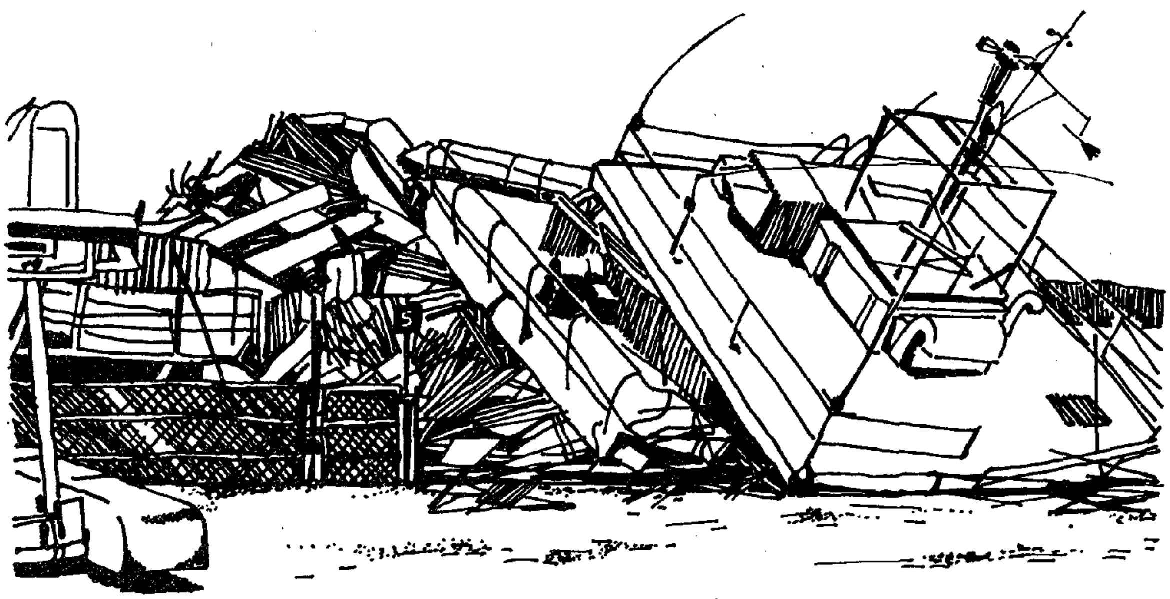
PRECAST CONCRETE FAILURE
ALASKA SALES & SERVICE BUILDING - 1964 ALASKA EARTHQUAKE
Figure 11.4

One of the more recent developments in the construction field, precast concrete, has problems similar to that of unreinforced masonry created by the fact that there is a strong temptation to assemble structures made up of precast elements by simply stacking these elements one on top of another. Such an assembly or nonassembly can perform very poorly during an earthquake. The reason is simple. As long as there are no lateral forces, one does not need connections resisting tensile forces between, say, beams and columns. In the event of lateral forces, however, the lack of a positive connection between precast elements leads to total collapse. I should point out that the earthquake problem is not with the type of construction but with the lack of proper joinery. Precast concrete may be used safely in earthquake zones provided the joints are adequate.

I should hasten to add that the lack of good joints is not the problem with precast construction alone. The tower of the Anchorage International Airport was an ordinary cast-in-place monolithic reinforced concrete frame. The column reinforcing bars were detailed to be spliced at the joints, as they often are. The failure of these splices to develop continuity of tensile force during the earthquake led to the collapse of the control tower.

A slightly different joinery problem is illustrated by the Four Seasons Apartment Building in Anchorage, Alaska. The floors were provided by prestressed concrete lift slabs. The building was designed to resist the lateral loading very much like a sailboat. During an earthquake, two reinforced concrete towers acting as masts would carry the lateral inertial forces, of the six floors as a ship's masts would carry the forces from sails. The building had been completely finished structurally before the earthquake but, fortunately, was unoccupied.

Because the anchorage of the reinforcement at the bases of the two reinforced concrete towers was not adequate, the two towers folded over during the earthquake (like the breaking of the masts of a sailboat) and the floors came down.



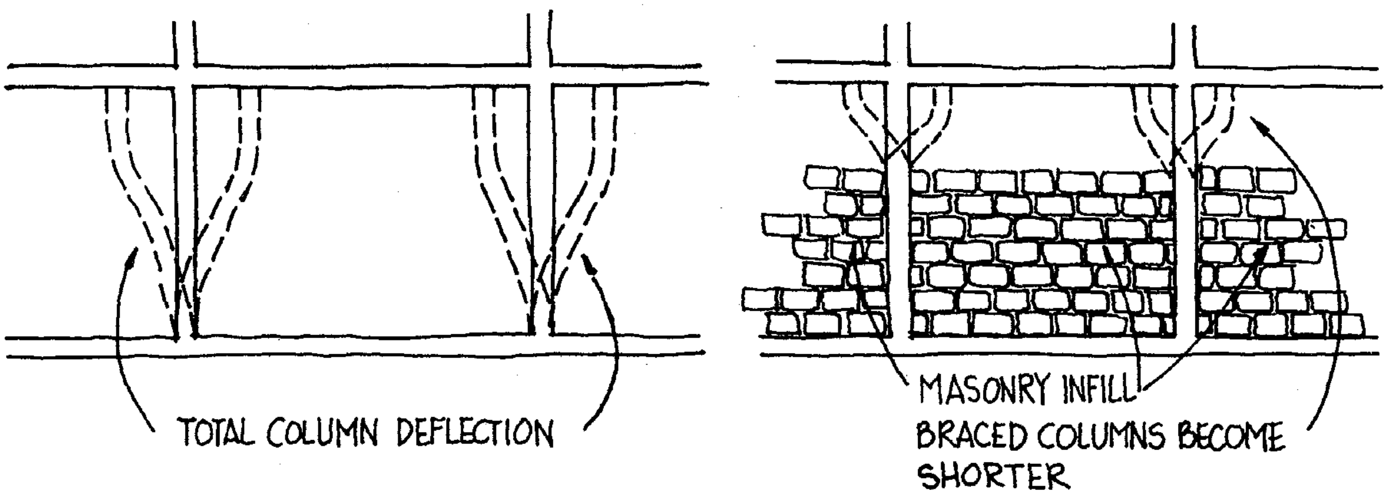
AIRPORT CONTROL TOWER - 1964 ALASKA EARTHQUAKE
Figure 11.5

FOUR SEASONS APARTMENTS - 1964 ALASKA EARTHQUAKE
Figure 11.6

The moral to these two cases is that one has to be very careful about splicing reinforcement in buildings which may be subjected to earthquake forces. Because reinforcement splices in columns are very seldom tested in ordinary construction subjected to gravity loads, and because we build primarily out of our experience, we have developed a certain amount of false comfort with respect to behavior of reinforcing bar splices. What appears to have been confirmed by everyday experience is actually misleading in this case because it is the wrong kind of experience. In fact, one of the difficult things about learning from experience with respect to earthquakes is that we accumulate a lot of mistakes in between earthquakes. Therefore, the "cut-and-dry" type of engineering is not quite suited to the earthquake problem.

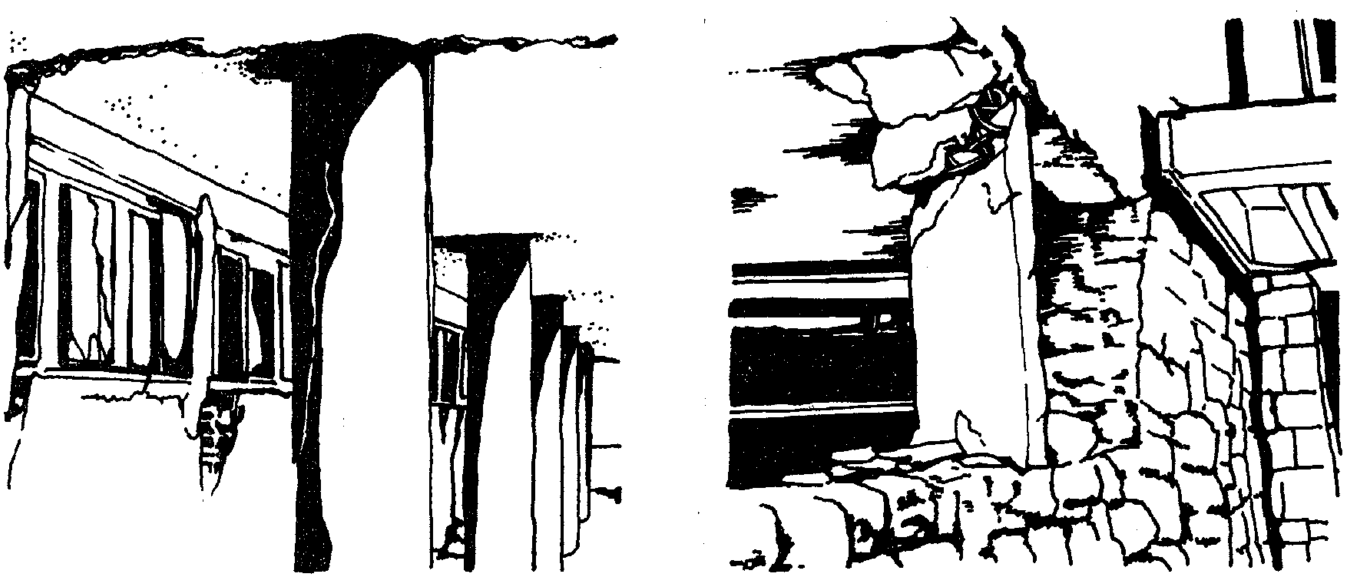
I should now like to talk about "architectural" damage and I do not limit that definition to damage to architectural elements but, I extend it also to damage to structural elements caused by architecture.

Insofar as many structural engineers are concerned, the structure consists of columns, beams and load bearing structural walls. Partition walls of brick or block masonry which are sometimes within the plane of structural frames are typically not considered to be part of the structure in the design of the system. This attitude typically leads to the captive column, a column bounded by masonry over part of its height, which is susceptible to shear failure during an earthquake. In the structural-design stage, the column is assumed to deflect over the height of the story with nothing impeding its motion. If a masonry infill is included, then the column becomes effectively much stiffer, develops larger shears, and may fail in shear as a result of forces which it was never expected to resist.



COLUMN DEFLECTION
Figure 11.7

An interesting example of the "captive column" was observed after the Skopje earthquake of 1963. The distribution of filler walls made one end of a two-story building much stiffer (for lateral forces) than the other end. In effect, the building was a horizontal pendulum with one end virtually fixed and the other able to move laterally, a condition which enhanced the lateral-drift requirement at that end. The columns would have been able to permit that movement without serious damage, but some of them were encased in a stone masonry (decorative) wall. Because of their additional and unplanned stiffness, those columns were sheared off by the movement.



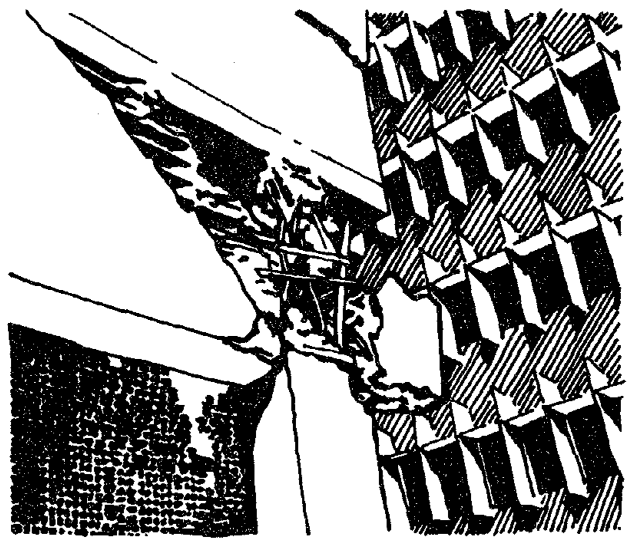
COLUMN FAILURES
STUDENT CLUB - 1963 SKOPJE EARTHQUAKE
Figure 11.8

A hotel in Guatemala (1973 earthquake) had essentially the same experience with more violent results. The tower portion of the hotel, four stories, had a kitchen on one end and a restaurant at the other. The kitchen was closed in by walls surrounding it. The restaurant portion at the other end was kept open, surrounded by glass. Because of those architectural decisions, this building was also very much like a horizontal pendulum, with the base of the pendulum at the kitchen end. The lateral motion sheared off the columns at the restaurant end leading to collapse on one side of the building.

Torsional stability, or the arrangement of masses and strengths in plan organized so as to strike a balance about a central point, is very important. This is one of the sore points that create conflict between architects and engineers. The conflict can be resolved provided it is understood that torsional imbalance will cost more, either at the time of construction or at the time of the earthquake.

COLLAPSE OF OPEN PORTION OF BUILDING
TERMINAL HOTEL - 1976 GUATAMALA EARTHQUAKE
Figure 11.9

Another case of interaction between architectural and structural elements was provided by the Mene Grande Building in Caracas which was damaged during the 1967 event. The building was supported by a structural frame built according to good engineering principles. It had nonstructural walls on each side of the building and these walls virtually wiped out any effect from the structural frame. Because the planar elements at either end of the building were much stiffer than the frames in between, the inertial forces were transmitted by the floor diaphragms to the end frames leading to direct axial compression failure in the columns.



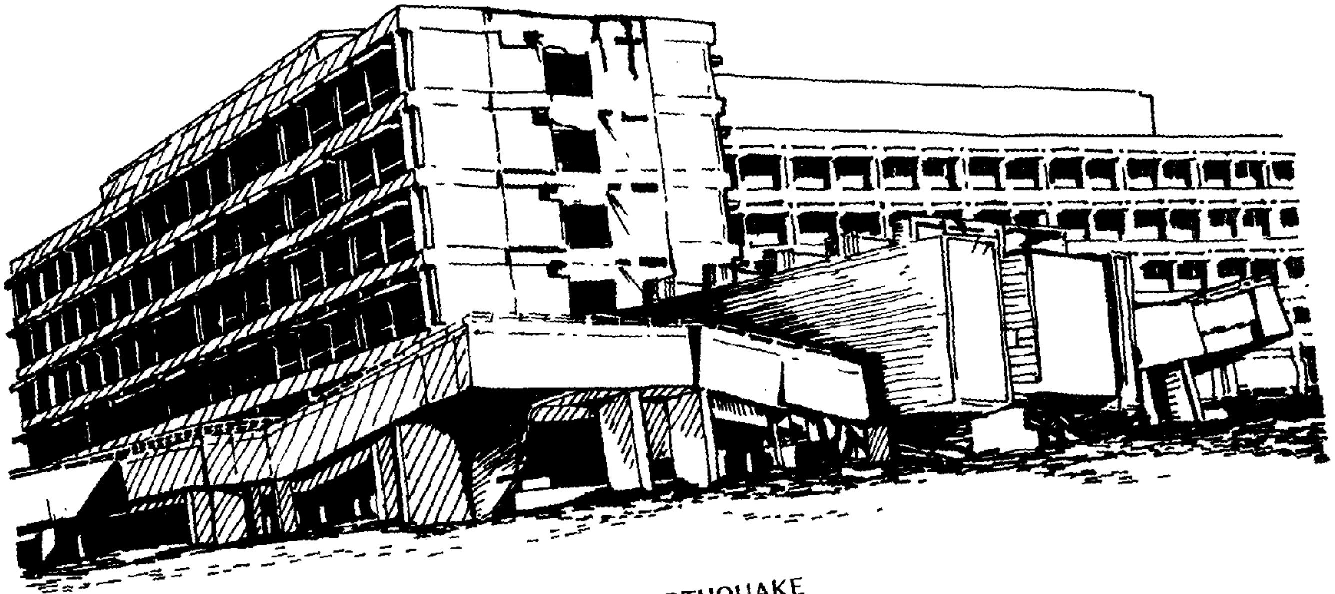
COLUMN FAILURE CAUSED BY ARCHITECTURAL WALL
MENE GRANDE BUILDING - 1967 CARACAS EARTHQUAKE
Figure 11.10

In resistance related to gravity loads, architectural and structural decisions may be made independently of each other. But in resistance related to earthquake effects, isolating the engineer from the architect is a formula for disaster.

Let me now talk about a problem that had to do primarily with the structural planning of the building.

The main building of the new Olive View Hospital in San Fernando, part of a reportedly $24 million complex of hospital buildings, was new at the time of the 1971 earthquake. The design of the hospital required open spaces at the ground level, as many buildings do. This condition prevented the structural engineer from continuing on with the structural walls of the upper floors through the ground level, which introduced a flexibility at that level. When the earthquake hit, the building was a total loss if not a total failure. (There were no casualties directly related to structural failure.)

In defense of the designer, I should point out that the building experienced an intensity of ground motion that most experts, at that time, did not consider possible. Furthermore, the total lateral strength of the building at the ground-floor level was approximately 50% of its weight above the level, assuming that all columns would develop their strength simultaneously. That is an impressive strength for a four-story building, but it was not enough.



FAILURE OF STAIR TOWER
OLIVE VIEW HOSPITAL - 1971 SAN FERNANDO EARTHQUAKE
Figure 11.11

SPIRAL COLUMNS VERSUS TIED COLUMNS
OLIVE VIEW HOSPITAL - 1971 SAN FERNANDO EARTHQUAKE
Figure 11.12

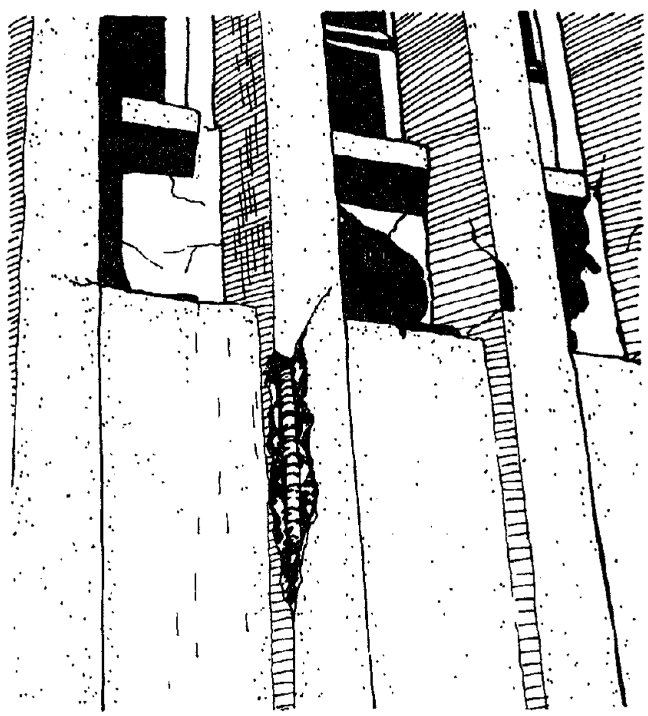
Another problem that came to the surface in connection with this building also deserves mention. The corner columns in this building were tied columns (the heavy longitudinal bars were tied with light reinforcement at a fairly wide spacing). The rest of the columns were spiral (the longitudinal reinforcement was encased by helical reinforcement at a short spacing). During the earthquake, the tied columns at the corners failed while the spiral columns held up even though they underwent lateral displacements on the order of 20 inches. The state of the building after the earthquake demonstrates, on the positive side, the inherent tenacity of columns having spiral reinforcement and, on the negative side, the susceptibility to brittle shear failure of the standard tied column.

STRUCTURAL WALL VS. THE STRUCTURAL FRAME

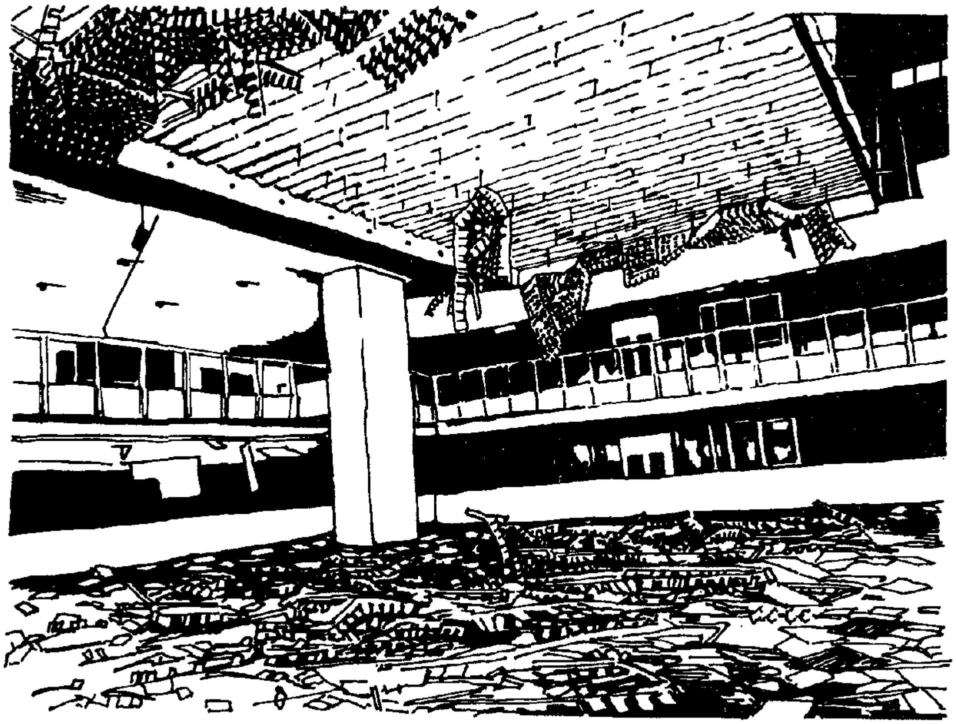
In order to take a specific look at the relative performance of structural frames and walls, I should like to use as an example, the observed behavior of two buildings during the Managua earthquake of 1972.

One is the Central Bank and the other is the Bank of America. The structural system for the Central Bank was a frame with some interference or stiffening from architectural elements. The structural system for the Bank of America which is slightly taller, was a fairly heavy structural core wall with the so-called "stressed skin" on the exterior. Both buildings came through the earthquake but both were damaged in different ways.

The Central Bank, the one with the frame, sustained visible damage almost all the way over the height of the building although, I would venture to say the building had relatively little critical structural damage. However, in terms of dollars, the architectural damage was enormous.

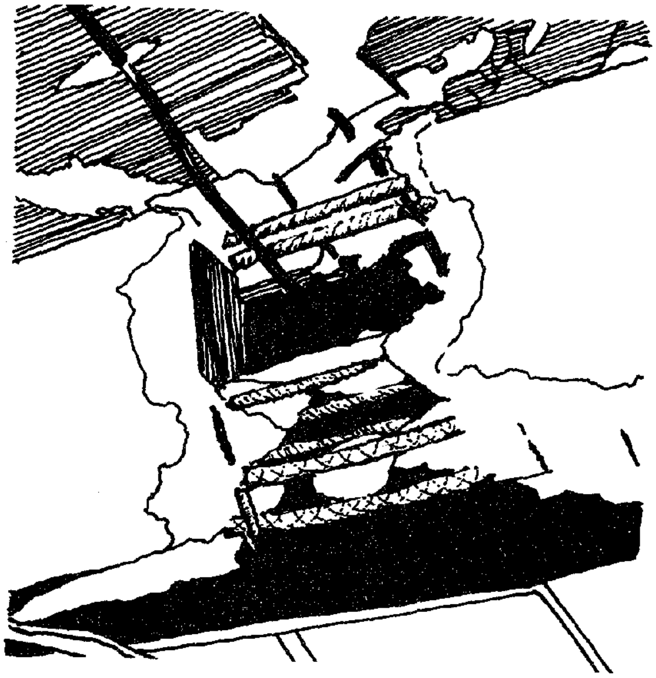


BANCO CENTRAL - 1973 NICARAGUA EARTHQUAKE
Figure 11.13

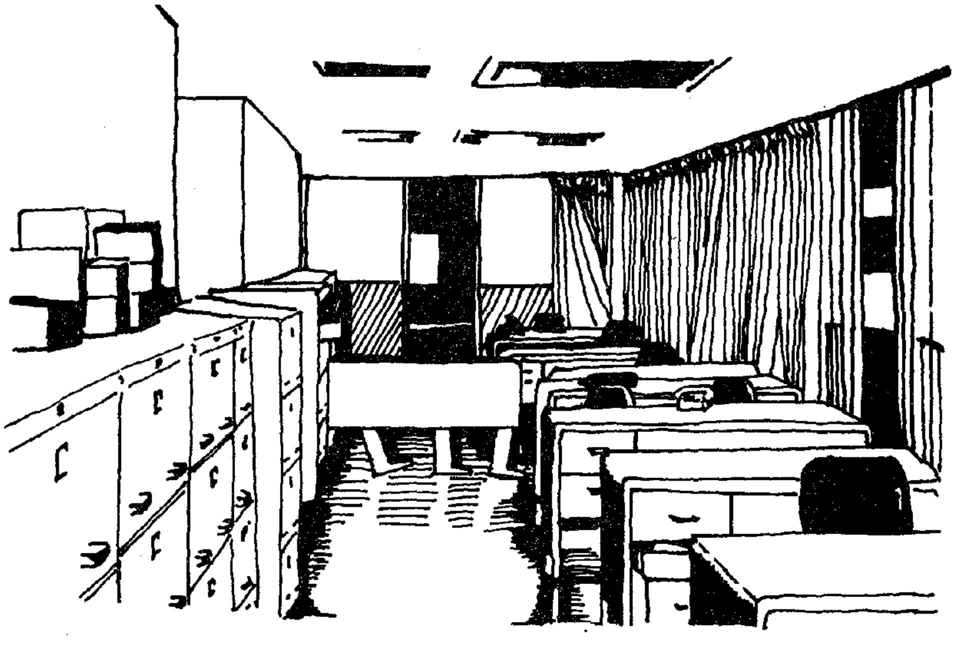


CEILING COLLAPSE
BANCO CENTRAL - 1973 NICARAGUA EARTHQUAKE
Figure 11.14

With the Bank of America, nonstructural damage was virtually nonexistent while there was some actual structural damage. Some of the connecting beams failed in shear, primarily because of the openings cut through the beams for air conditioning ducts. But the building itself including its nonstructural components was in very good shape. Although one should not draw general conclusions from a single observation, the tendency for almost all observers was to conclude that the structural wall was a better system. And I do not deny the experience. In this instance, the overall response of the structural-wall system was better than that of the frame system. Yet I must emphasize the fact that the structural frame for the Central Bank was indeed a very flexible frame. In general, I should like to contend that given the intensity of a design earthquake, one can design a frame which will not develop more than the permissible amount of drift. However, this result requires more thought for a frame than for a wall. The wall could be well proportioned by accident, but not the frame.



BANCO DE AMERICA - 1972 NICARAGUA EARTHQUAKE
Figure 11.15



LACK OF INTERIOR DAMAGE
BANCO DE AMERICA - 1972 NICARAGUA EARTHQUAKE
Figure 11.16

My observations in the field and in the laboratory do not make me come to a strong conclusion about whether one system is better than the other. I believe that either the wall or the frame can be used efficiently and successfully.

BUILDING RESPONSE TO EARTHQUAKE EXCITATION

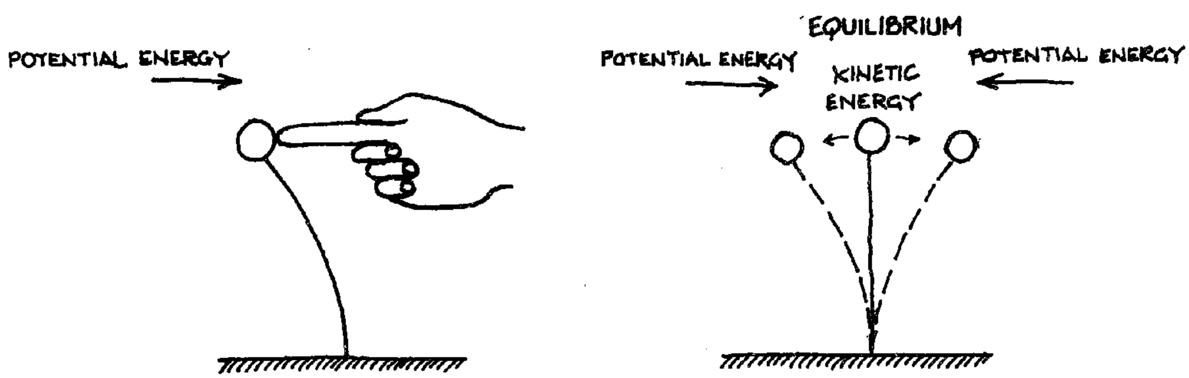
I can make few generalizations about structures without referring to the term "period" which is given in units of time. In general one could say that if a building is stiff, it tends to have a low period (or a high frequency). And if another building has the same mass but is flexible, then it is likely to have a longer period (or a lower frequency). In both cases I am referring to the lowest (natural) frequency or the highest period of the building.

One of the simplest ways to determine the period of a structure is to observe the relationship between kinetic and potential energies. If I take a mass on a spring (see Figure 11.16) and push it in a given direction and keep it there, it pushes against my finger (it has potential energy). If I let it go from that position, the system vibrates.

On the other hand, if I let the system come to rest at its position of neutral equilibrium, in the middle, then,the system stands at rest. In its position of neutral equilibrium, it has no potential energy.

When I let it go from a position where it has potential energy, the mass goes back and forth through the position of the neutral equilibrium. As I indicated earlier, at that point it possesses no potential energy but does have kinetic energy because of the fact it is moving (it possesses velocity) when it goes through that position.

The movement is from potential energy to kinetic energy and then back up to potential energy. Assuming the energy is conserved and assuming that the kinetic energy is equal to the potential energy, one can estimate the period of the system.



ENERGY RELATIONSHIPS
Figure 11.17

First let us look at the potential energy. If I refer to the lateral displacement of the mass as x, for a linearly elastic system one may represent the relationship between the force necessary to deflect the mass and the distance such that at a lateral displacement x and the force necessary is Kx. At that particular displacement x, then, the potential energy is the area under the curve which may be expressed as $(Kx^{2}/2)$.

The expression for the kinetic energy is a little bit less obvious and is expressed in terms of the mass (or weight divided by the acceleration of gravity) and the velocity $(MV^{2}/2)$.

To go a little further into the occult, I can express the velocity in terms of the circular velocity, in radian, of a particle moving around the circumference of a circle with its radius equal to the maximum deflection, from rest, of the oscillator we are dealing with. Let us call that distance X. Then, the kinetic energy term can be written as $Mw^2 X^2$.

Equating the expressions for potential and kinetic energy, I arrive at the familiar expression

$$KX^2 = Mw^2 X^2 \qquad (1)$$

$$w = \sqrt{\frac{M}{K}} \qquad (2)$$

The circular frequency, w, which is in radians per second is converted into cycles per second by observing that there are 2π radians in a cycle.

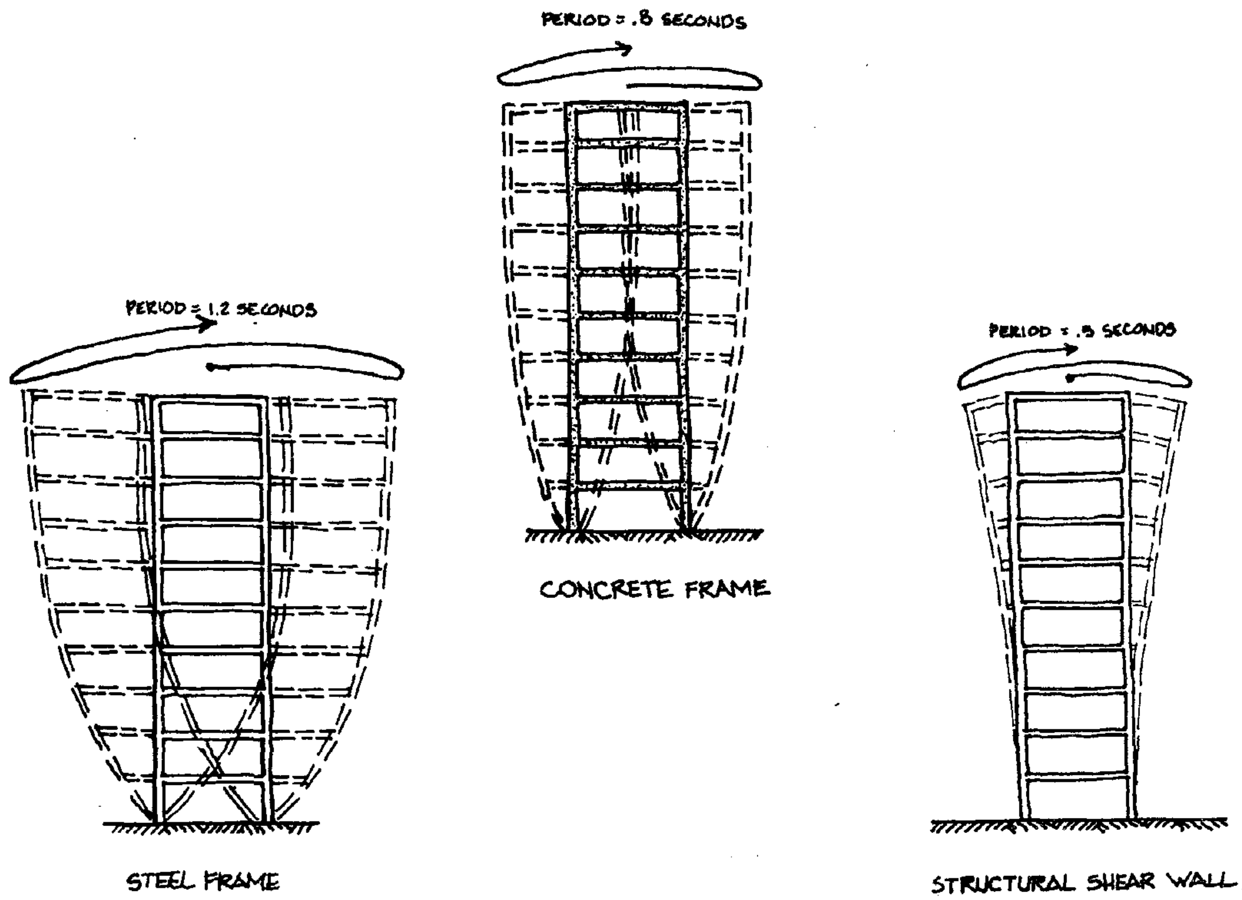
$$f = \frac{1}{2\pi} \sqrt{\frac{M}{K}}$$

The period is the inverse of the frequency

$$T = 2\pi \sqrt{\frac{M}{K}}$$

The derivation is not really that important for us. What is important is that the result appeals to reason, to one's consciousness of the physical world. Equation 3 suggests that the frequency of a system, or the rate at which a system will go back and forth, will be high if you have a stiff system (large K) and will be low if you have a flexible system (small k). Again as one would expect, an increase in mass will tend to decrease the frequency. One should note that the change in frequency with mass or stiffness varies as the square root: if stiffness is quadrupled, the frequency is doubled. Changes in stiffness or changes in mass are not directly related to the changes of the frequency of the system. The stiffness may change by a large amount but the frequency changes at a lower rate. In that respect, this simple equation does represent the overall changes in the frequency of a building, although the building is a much more complex system than the simple oscillator we have been looking at.

Certain rules of thumb give one an idea of the periods or frequencies of building construction. The period of a structural steel frame building is likely to be 0.12N where N is the number of stories. For example, for a ten-story building, the period should be approximately 1.2 sec. A comparable reinforced concrete building designed for earthquake is likely to have a period of about 0.08N. Similarly, if the lateral load resistance of the building is provided primarily by structural reinforced concrete walls, the period of the building is likely to be approximately 0.05N. These rules of thumb are quite useful in assessing the influence of a given earthquake motion on a building. However, one must always bear in mind that these relationships refer to buildings which have been designed to resist strong earthquake motions. They would certainly not apply to buildings built, say, in New York City where the seismic risk is not typically a design factor.



DEFLECTED FRAMES
Figure 11.18

RESPONSE SPECTRUM

The response spectrum is a convenient vehicle for evaluating or understanding the susceptibility of a given building defined by natural period, to different classes of earthquake motions. While the natural period provides an important characteristic of the building, the response spectrum describes the earthquake motion.

To understand the response spectrum, we shall have to approach it in a round about way.

Consider three different SDF (single-degree-of-freedom) oscillators having natural frequencies of 0.5, 1.0 and 2.0 Hz.

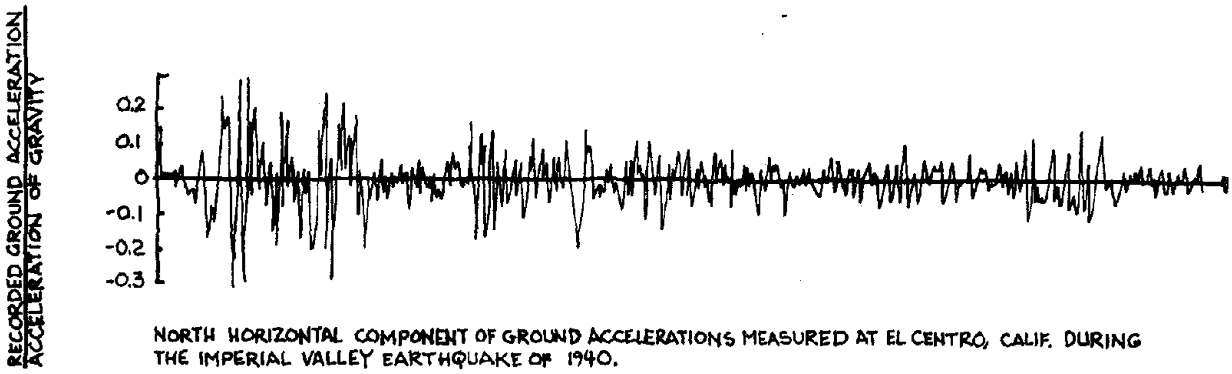
The response of each oscillator to a base motion representing one component of a measured earthquake acceleration—time record may be calculated. The velocity responses so calculated for the north component of the acceleration record obtained at El Centro in 1940 are shown in Figure 11.19.

Although these three response-history plots are shown primarily to explain the construction of response spectra, they provide an interesting demonstration which will be of use in discussing the response of complex structures. Each linearly elastic oscillator is excited by the earthquake motion, which can be considered as being made up of a collection of waves having different frequencies, to vibrate in its own frequency. Except for transient periods preceding the initial or major shocks, the motion of each oscillator is simple, it tends to vibrate in its own fundamental frequency, the main variant with time being the level of the response. The input is quite complex. The output is fairly simple.

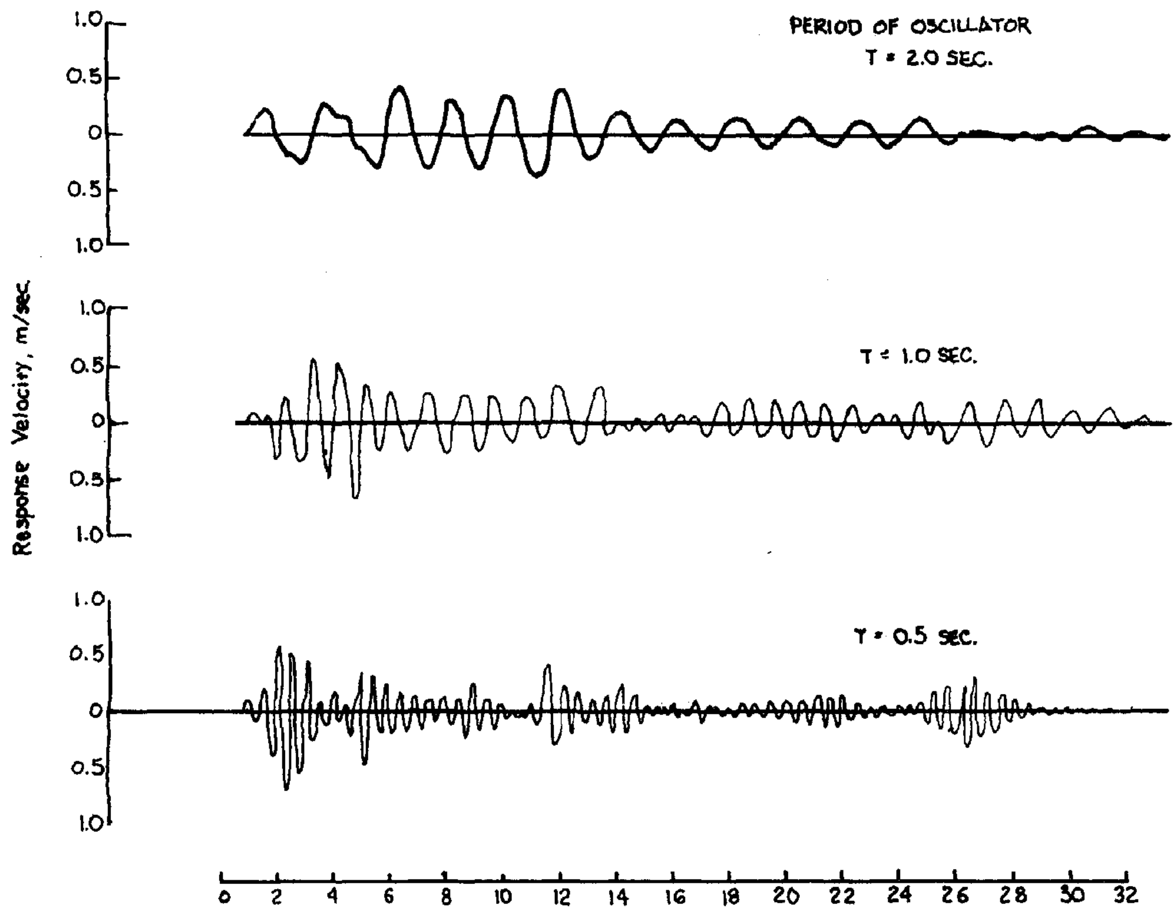
It will be noticed that each oscillator registers a different level of maximum response and the response waveforms are not alike. For the oscillator with a period of 2 sec., maximum response occurs after 6 sec. of strong motion but the amplitudes remain large to 10 sec. For the 1.0-sec. oscillator, the maximum occurs at an earlier time after which the response drops and remains at a low level. The 0.5-sec. osillator also develops its maximum at an early time but continues to have bursts of high excitation throughout the duration of the earthquake. These phenomena depend on the particular characteristics of the input motion. Other than the fact that each oscillator is likely to have a unique maximum, there are few useful generalizations to be made about the shape of the response waveform as it is likely to vary with the frequency content of a given strong motion.

The calculations related to the three oscillators provide three maxima which may be plotted against frequency as shown in Figure 11.20 resulting in a rudimentary response spectrum. Taken at face value, the broken line represents the anticipated maximum velocity response of simple oscillators subjected to the particular component of the ground motion considered.

The maximum response of a linearly elastic oscillator to a particular strong motion may vary rather drastically as its frequency changes. Therefore, the information shown in the three-value spectrum is crude to the point of being useless. To get an acceptable perspective of the range of *acceleration* response (or the acceleration-response spectrum) it is necessary to calculate and plot the acceleration response to a large number of frequencies. Calculated ranges (spectra) of acceleration and displacement response are shown in Figure 11.21., the horizontal axes in units of frequency to 0.5 Hz and in units of period beyond that value. This combination was arrived at by the necessity to expand the horizontal scale in ranges of engineering interest especially for the purpose of comparing calculated and idealized spectra. The particular combination is a matter of convenience. A logarithmic scale may



NORTH HORIZONTAL COMPONENT OF GROUND ACCELERATIONS MEASURED AT EL CENTRO, CALIF. DURING THE IMPERIAL VALLEY EARTHQUAKE OF 1940.



Velocity Response of Three Linear (Elastic) Oscillators to the Acceleration Record Measured at El Centro (1940 North)
(Damping Factor = 0.05)

VELOCITY RESPONSES: NORTH COMPONENT OF ACCELERATION RECORD
EL CENTRO EARTHQUAKE, 1940
Figure 11.19

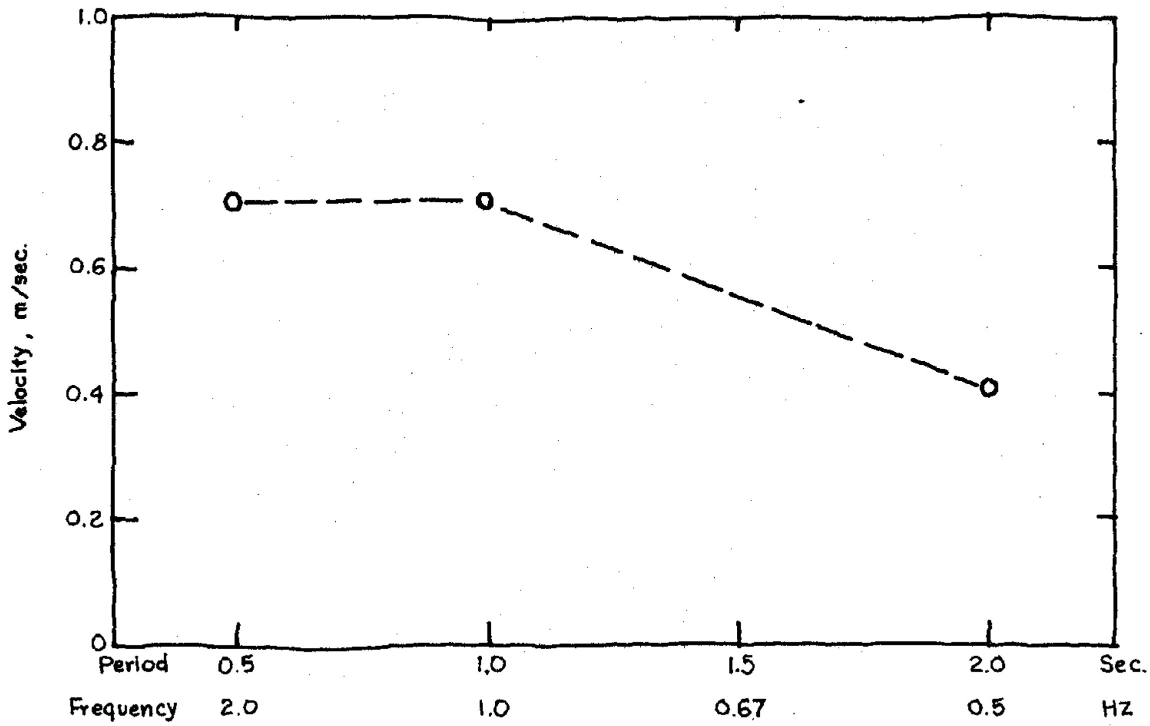


Figure 11.20 Maximum Velocity Responses for Three Linear (Elastic) Oscillators.
(Damping Factor = 0.05)

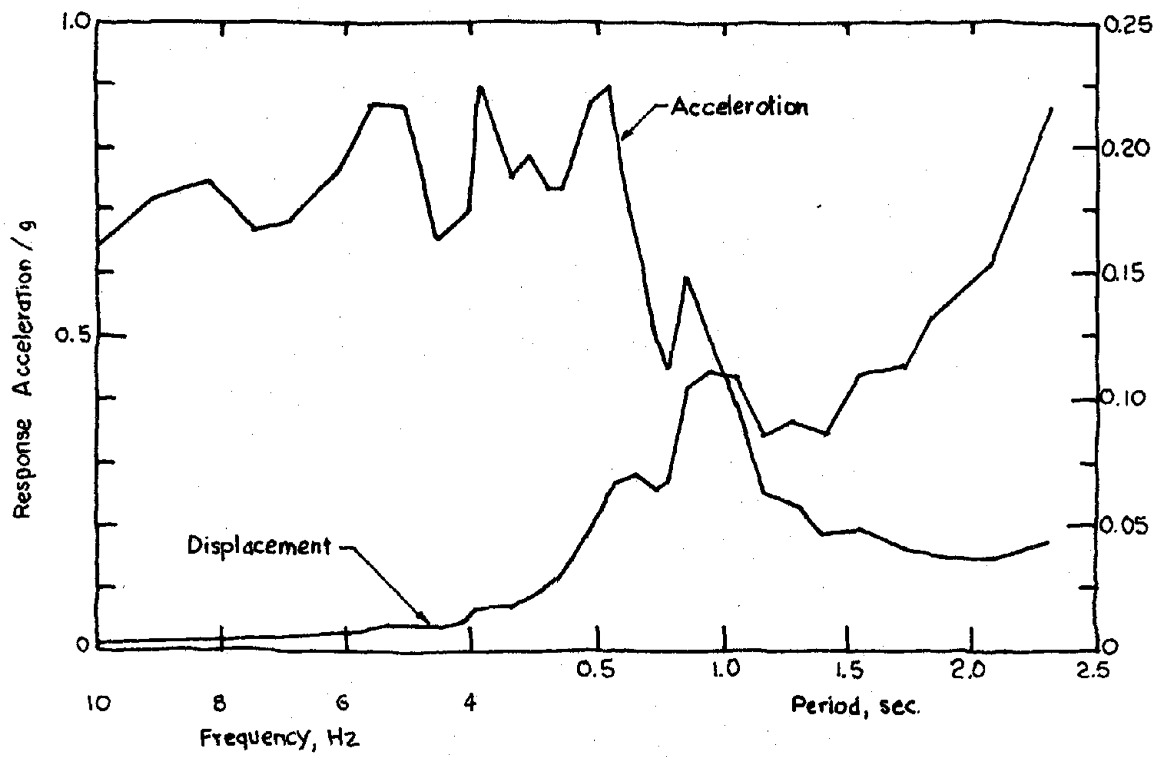


Figure 11.21 Linear Response Spectra for El Centro 1940 N.
(Damping Factor = 0.05)

also be used for the horizontal axis. However, a nonarithmetic scale for the vertical axis is not recommended for applications in design. Although the log-log (or tripartite) plot does offer the advantage of reading values of displacement, velocity and acceleration from the same plot, it has a tendency to distort the actual effect: large variations in response may appear to be of negligible magnitude though they actually are not. It takes constant vigilance not to be misled about relative quantities when using a log-log plot of response spectra. In design applications, the use of a linear scale for the vertical or response axis helps develop a better perspective.

Although the response spectrum does not provide a total perspective of the effects of ground motion, it does serve to emphasize certain important characteristics.

For example, the acceleration-response curve suggests that the response is reasonably constant up to a period of approximately 0.5 sec, after which it drops at a rapid rate. Recognizing the relationship between force and acceleration (force = mass x acceleration), we may conclude that the lateral force on a building expressed as a function of the weight of the buildings, is high for stiff (low) buildings and low for flexible (tall) buildings. This turns out to be a general characteristic of earthquake motion in firm ground near causative faults and makes it possible to determine design forces without the necessity of knowing the peculiarities of the particular earthquake motion to occur at a given site.

STRUCTURAL RESPONSE

How do we use the response spectrum in understanding structural behavior? I shall illustrate with a simple, if narrow, example. In the analytical plane we usually talk about linear response, meaning that the force displacement relationship for the system is linear or linearly elastic.

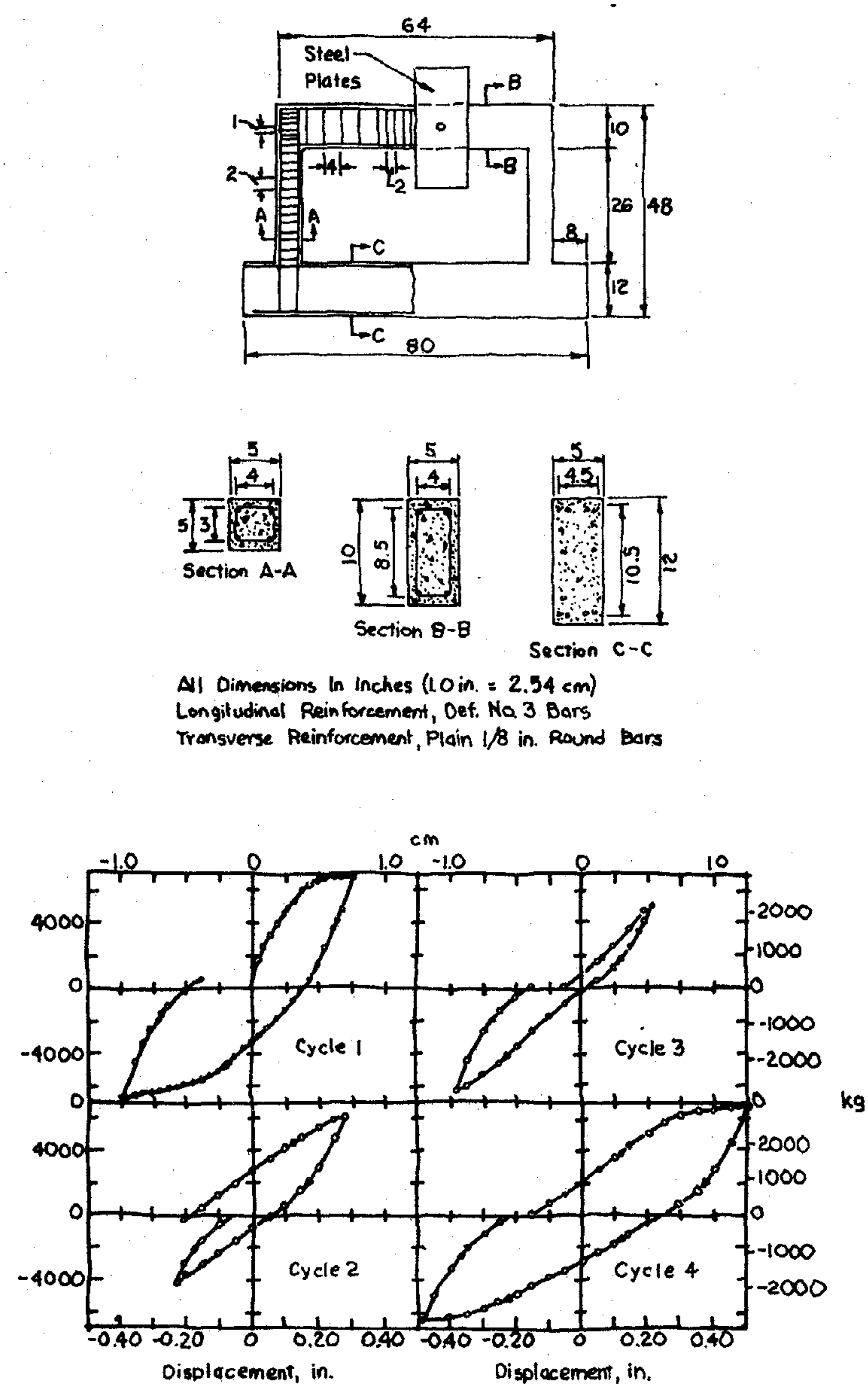
Another popular representation of force-displacement response is the elasto-plastic system, a system, which if loaded in one direction only, responds elastically initially to yield at a particular level and continue plastically. The representation is convenient for caculations but does not represent realistically any material that we use in structures.

I should like to use the typical hysteretic response of a well detailed reinforced concrete frame to provide a window into the response, in the nonlinear range, of buildings to earthquake motion.

To accomplish that I should first like to discuss the force displacement relationship for a particular frame under alternating loads (Figure 11.22).

In cycle one, the frame is loaded into the nonlinear range of response (the slope of the load displacement curve becomes nearly flat and the unloading curve does not follow the loading curve). On taking the load off, I see that the displacement does not return to zero. There is a "shortcoming" or hysteresis. As the frame is loaded in the opposite direction, there is a faint indication of yielding. In cycle two, there are no indications of yielding and the same is true in cycle three. In cycle four when the system is loaded to a larger displacement, in one direction see an indication of yielding but that is all there is to it.

The important characteristic to notice is that as the frame is loaded into the nonlinear range of response the average stiffness of the system decreases and as the displacement increases, the area of the loop tends to increase.



HYSTERETIC RESPONSE OF A REINFORCED CONCRETE FRAME
Figure 11.22

The area contained within the hysteresis loop indicates a measure of the energy dissipation capacity of the structure: the "fatter" this loop the smaller would be the response of the building to a given earthquake. In this respect, fat is good.

Given these characteristics and the spectral-response curve, we can develop a vehicle for understanding the nonlinear response of the structures in earthquakes. The earthquake starts with the building in a relatively stiff undamaged condition. As the building shakes back and forth during the strong motion, its stiffness decreases (its fundamental period lengthens) which, interpreted in terms of the acceleration response spectrum, corresponds to a reduction in the force capacity. At the same time, the "fatness" of the hysteresis loop increases, limiting the response of the structure even more.

The catch, of course, is that the structure must be able to undergo these motions into the nonlinear range of response without critical decrease in strength and certainly without collapse. This is roughly what we mean by ductile response. To attain that ability, the structure must leave special details.

Conceptually then, structural design for earthquake resistance becomes a negotiation between strength and tolerable drift. The more lateral drift one is willing to tolerate, the less strength one needs to put into the structure. And the more drift one permits, the greater is the cost of architectural damage likely to be.

Whatever the fine points of strength assignation or system selection, the designers of the building must realize that, come the earthquake, the building will distort and the main criterion of success will be the cost of the effects of that drift.

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Chapter 12. STRUCTURAL DESIGN CONSIDERATIONS

Thomas D. Wosser, P.E.

INTRODUCTION

During a recent meeting I had with an architect, we discussed a building that had experienced problems. It had been discovered that it did not conform to seismic code requirements. Beyond that, it was considered a hazard. In discussing the shortcomings of the engineering, we decided that the engineer should have known better than to come up with a building with that particular structural design, but we also decided that the architect should have been aware of at least parts of those problems because the planning of the project fell under his direction. In fact, the suit that is pending names the architect as well as the other designers involved in the project.

Given the twin threat of hazardous design and professional liability, the architect should certainly be aware of the effects of earthquake forces on the materials of construction, the configuration of the building, and how it is going to react to resist seismic forces. Equally important is the awareness of the need to work with the engineer early in the design so that the two can plan together the needs of the structure to resist earthquake forces.

Planning to design to resist earthquakes is a different game from the normal planning that goes on when earthquakes are not considered. For one, we are not dealing with an exact science when we talk about earthquake engineering. Instead, it is an art that depends to a great extent on judgment with the basic facts we do have. This poses obvious problems of its own. To point out what I mean, I'd like to quote from an article that was in *Consulting Engineer* magazine last year: "The possibility of being deceived by an assumption masquerading as a conclusion, is the designer's greatest enemy."

In designing to resist earthquakes, we must constantly remind ourselves that virtually everything that we do is based on an assumption. The hard facts are few and far between.

OBJECTIVES OF SEISMIC DESIGN

The function of seismic design is primarily to provide for life safety. We see evidence in all significant earthquakes that have occurred. Nor is the hazard restricted to those inside the building; there is also the threat posed to those on the outside by falling debris. So, again, the primary objective—and it is reflected by the codes—is to provide life safety, prevent collapse of the building, and remove exposure to falling hazards.

Another aspect that must receive considerable attention is to provide safe egress from the building. That means the stairs and the elevators should be usable; people should be able to find their way out of a building following an earthquake. Entrances must be free of debris.

Earthquake engineering and codes have historically dwelt on the life safety aspect. True, we have always been aware that property damage does occur. But the focus has been on structural damage. If structural damage occurred, it was supposed to lend itself to easy repair. It could not be damage that would in any way endanger the entire structure.

Lately, a new trend has begun to emerge. More consideration is being given to the architectural, mechanical, and electrical components within a building as a result of the great damage that has been observed in recent earthquakes. In some instances, the whole issue of non-structural damage is a matter of how much damage an owner is willing to absorb. Yet, obviously there are safety problems as well. With the recent adoption of the hospital code in California, hospitals are now required to be designed so that they remain functional following an earthquake. This means that certain mechanical equipment must continue to function, that life lines and services must continue to operate, and that the building itself must be useable. The new Uniform Building Code extends this to other important facilities such as fire stations and communication centers. For these, too, should remain functional following an earthquake; they are important to the rescue effort and work that goes on after a disaster.

The question of what is acceptable property damage is of great significance in the selection of a seismic-resisting system as well as the types of architectural details that are used. For example, if a building is designed using a stiff shear wall system, this may limit the deformations of the building to such a degree that there is no hazard to interior finishes. In that case, there might be very little nonstructural damage. On the other hand, if the system is a flexible moment frame without any other stiffening elements, it would deform a great deal in the earthquake. Unless the architectural components were designed with special detail, they would undergo the same deformation and would be damaged. In buildings of that sort, it is necessary to go into special details that allow, for example, relative movement between the structural and partitions.

STRUCTURAL CONSIDERATIONS

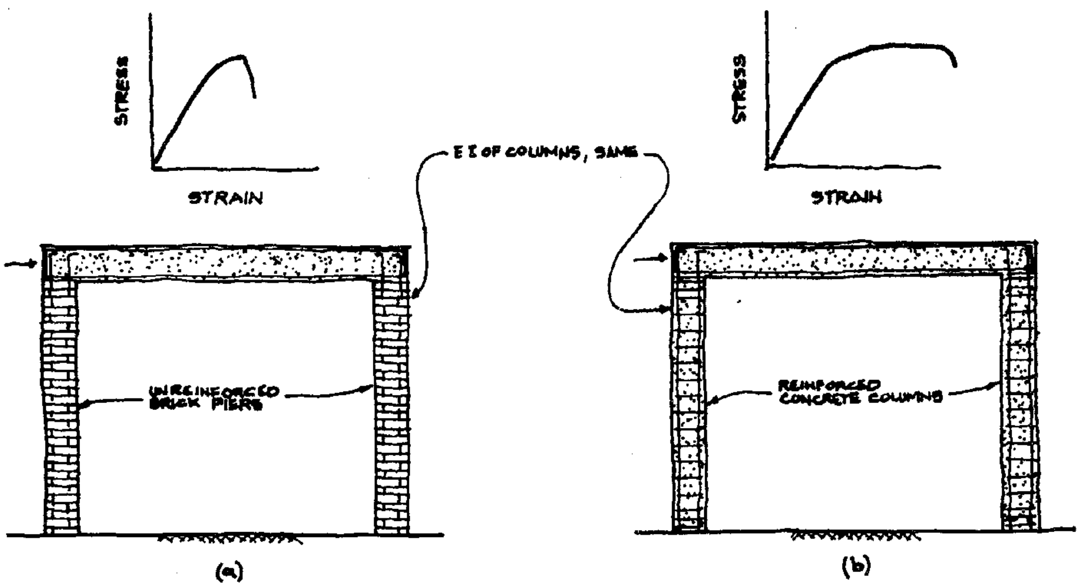
What are some of the significant structural considerations that the architects should be aware of? Of fundamental importance is the need to recognize the basic difference in designing to resist earthquakes as compared to normal vertical load design that is used in many parts of the country.

For normal vertical load design, we calculate dead loads within a very close percentage. We know our live loads. We know the performance of materials has been reasonably well documented for static conditions within a normal working stress range to which they are exposed under normal vertical load design. Furthermore, we design with a reasonable factor of safety so that except for some unusual conditions that might occur, we know that the material will resist the known loads well within the elastic properties of the material.

However, in designing to resist earthquakes, we really have only ranges of what the imposed forces may be. The information we have about the cyclic performance of materials in their ultimate range is rather meager. What we do know is the code design force for earthquakes is only a small fraction of those forces that will be imposed on a structure in a major earthquake. The structure will be overstressed by major amounts, yet it must respond beyond its yield strength. It must remain coherent, it must hang together, and it must be stable at deformations that are many times the yield deflection. Not only do we need to provide certain minimum strengths, but we must consider performance of the structure at great overloads and at large deformations.

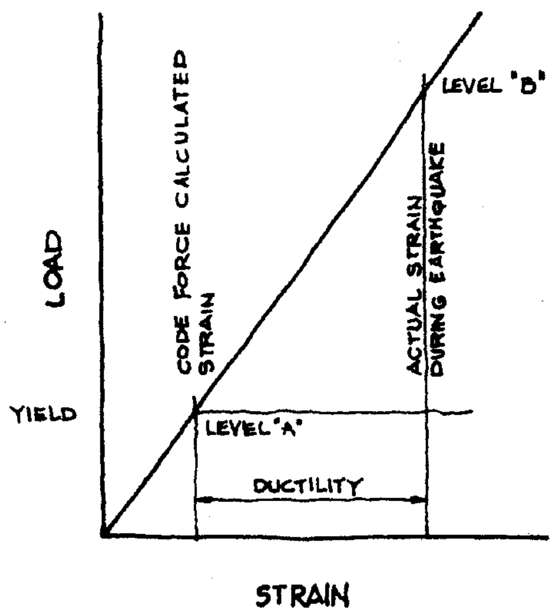
This underscores how imperative it is to rethink the usual design procedure to recognize that earthquake-resisting design requires a structure to respond well beyond its elastic range. We must consider its performance at ultimate strength levels and comply with this demand. The system must be stable and it must be capable of performing in a ductile manner; that is, in a manner that allows it to continue to carry a load at increasing strains beyond the elastic limit without significant reduction in that load.

Figure 12.1 shows two bents of comparable dimensions. The left in unreinforced brick piers; the right is of reinforced concrete. Under lateral loading, we would expect them to perform in much the same manner up to a certain point. In the case of the brick piers, stress increases with strain up to a maximum level; then, once it cannot achieve any more deformation, the bent collapses as represented by the stress strain diagram at the top. The concrete bent is represented by the stress strain curve at the bottom of the diagram. It shows the stress increasing up to its yield point; at which time, however, the bent continues to deform while still maintaining that same load. We know, therefore, that the concrete bent is going to resist forces from an earthquake much better than the unreinforced masonry.



STRESS-STRAIN DIAGRAMS
Figure 12.1

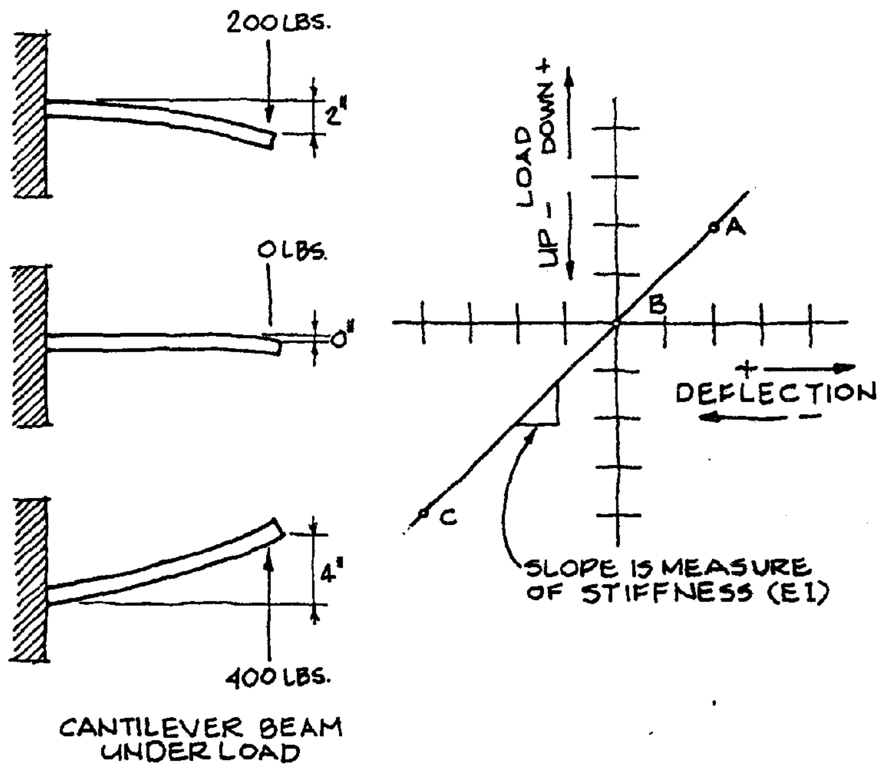
Figure 12.2 indicates ductility requirements. If the material is sufficiently strong and elastic, it can be loaded up to level B. This is a load strain curve. The slope of that line is the elasticity. However, another material which has a yield point at level A would deform plastically as a result of sustaining the load. In this case, it is assumed that the stress curve beyond that point would be a horizontal line. In other words you'd take off from point A and draw a horizontal line to where the strain would be the same amount as that indicated by level B.



CALCULATION OF DUCTILITY AND
DEFLECTION OR DRIFT
Figure 12.2

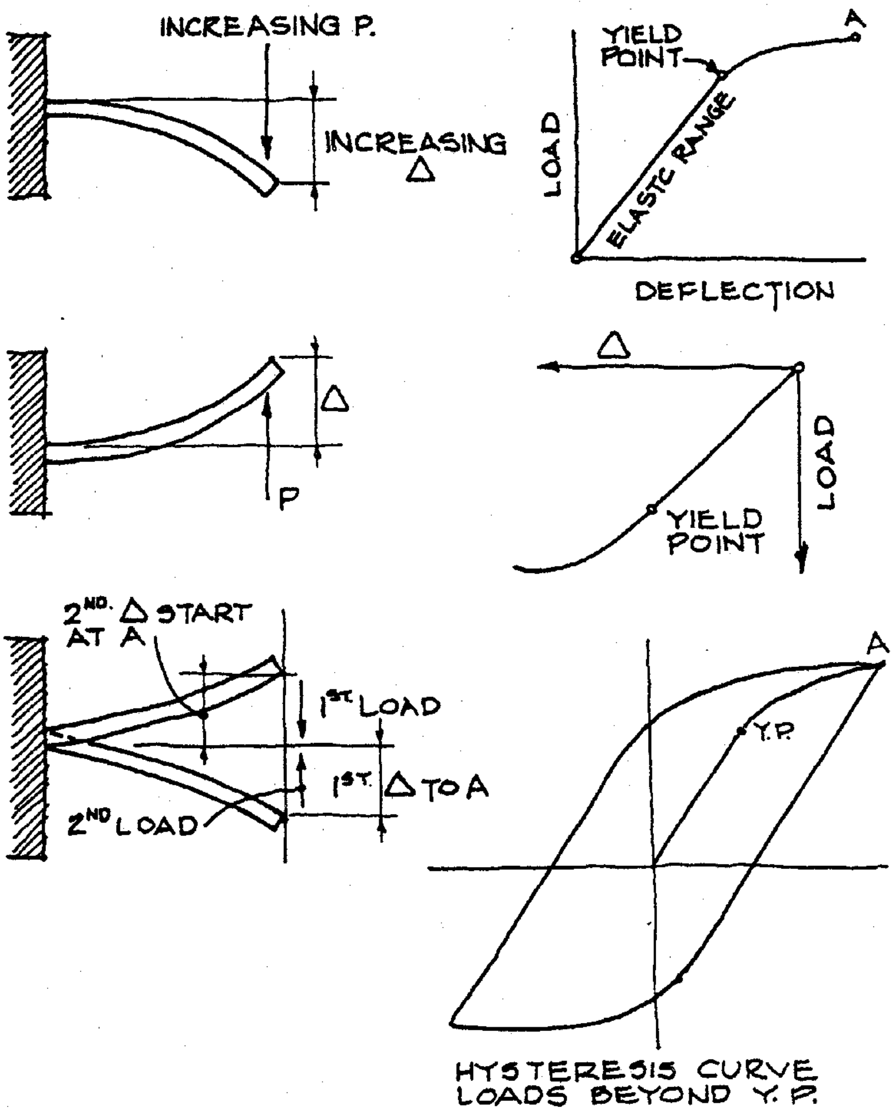
The distance between the yeild point of A and that which is achieved at level B is really an expression of a structure's ductility. It has been shown that for a single mass system this weaker but ductile structure deflects the same amount as the strong elastic structure under similar forces.

In Figure 12.3 we indicate first a load down, then release of the load, and the reversal showing a load pushing the beam back up. Again, the stress strain diagram shows a straight line that would rise to the upper right hand, come back down to 0, and then fall into the lower left hand quadrant. All that is in the elastic range is expressed as a straight line.



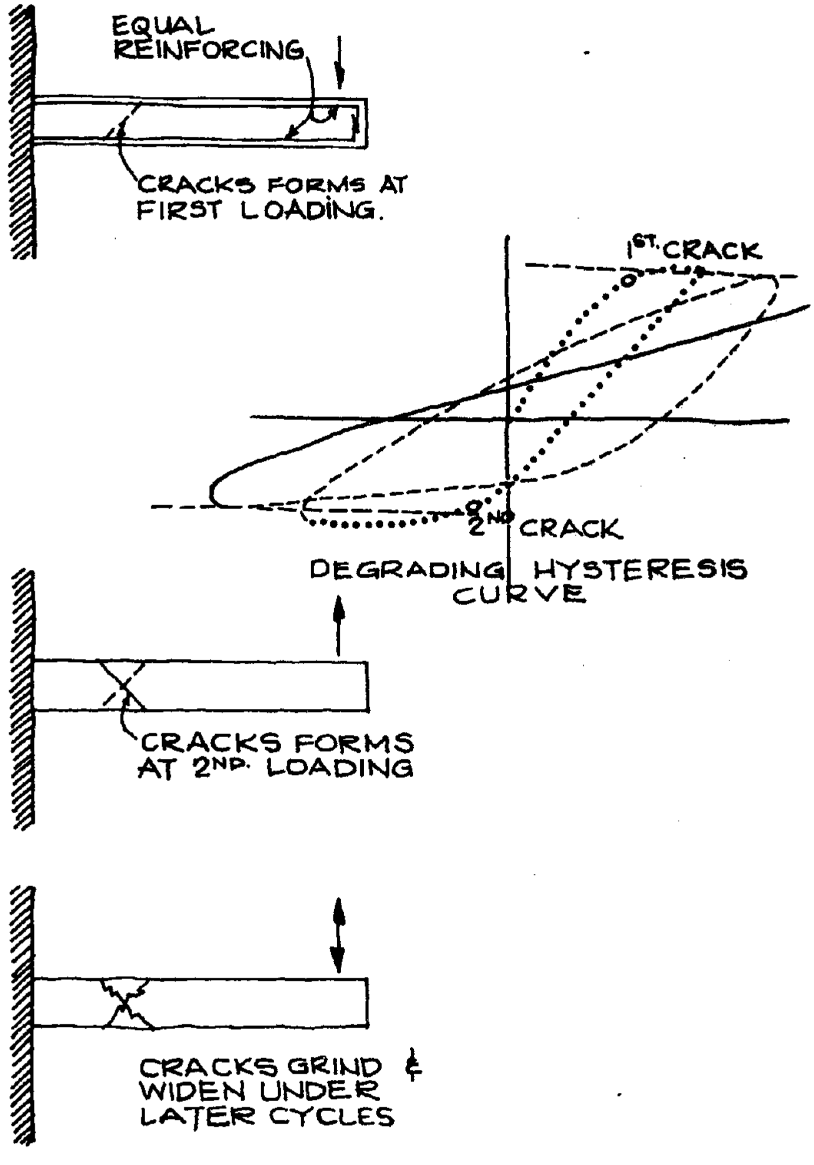
LOAD DEFLECTION CURVE
Figure 12.3

The stress strain diagram in Figure 12.4 shows what happens when the yield point is exceeded. We see a straight line that runs about three-quarters of the way up and then curves off to the right. That point of departure is the yield point of the material. It then moves far to the right with a greal deal of deformation and a small amount of increase in stress. Now, as the load is reversed and put into the other direction, it deforms elastically with the straight line until it reaches its yield point then it deforms plastically. Reverse the load again and the cycle is completed. This forms what is called a hysteresis loop. The area enclosed within that loop is a representation of the absorption of energy as the structure deforms in the plastic range.



LOAD DEFLECTION CURVE
Figure 12.4

Figure 12.5 shows a hysteresis loop with a material that degrades as it deforms. Observe that the curve becomes flatter and flatter as the cycles proceed. It is evident that the structure becomes less and less stiff and requires less and less load for given deformations, eventually becoming unstable. This can happen with large deformations of concrete beams that may be under-reinforced so that the beams crack. When the load is reversed in the other direction, the crack will have to close before the structure can reverse and perform the way it was designed.

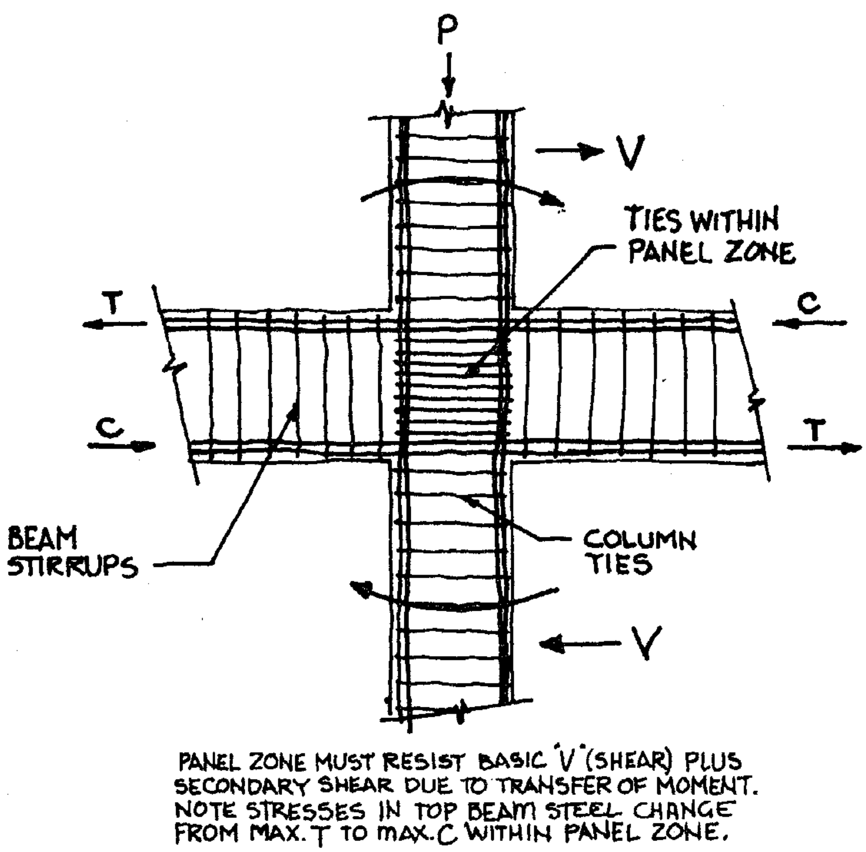


HYSTERESIS
Figure 12.5

There can also be a crawling effect if a structure is forced beyond the yield point in one direction and the return force does not bring it back to yield. It may undergo several such cycles, which means that it will continue to deform and to bend in one direction. As it goes beyond its yield point, it can become a permanent set. This has significance when considering the axial load on the building and what is called the P-Delta effect. Both can provide larger moments on the connections and effectively increase the lateral forces because of eccentricity. That is, as the structure deflects from one floor to the next, there is an eccentricity of the resistance of the axial load.

The usual methods of superposition of different loading conditions can no longer be used when designing in the inelastic range because the elastic limits will be reached. That affects the manner in which the structure will deform, so you cannot take the superposition of loads as we can when we assume the structure remains elastic.

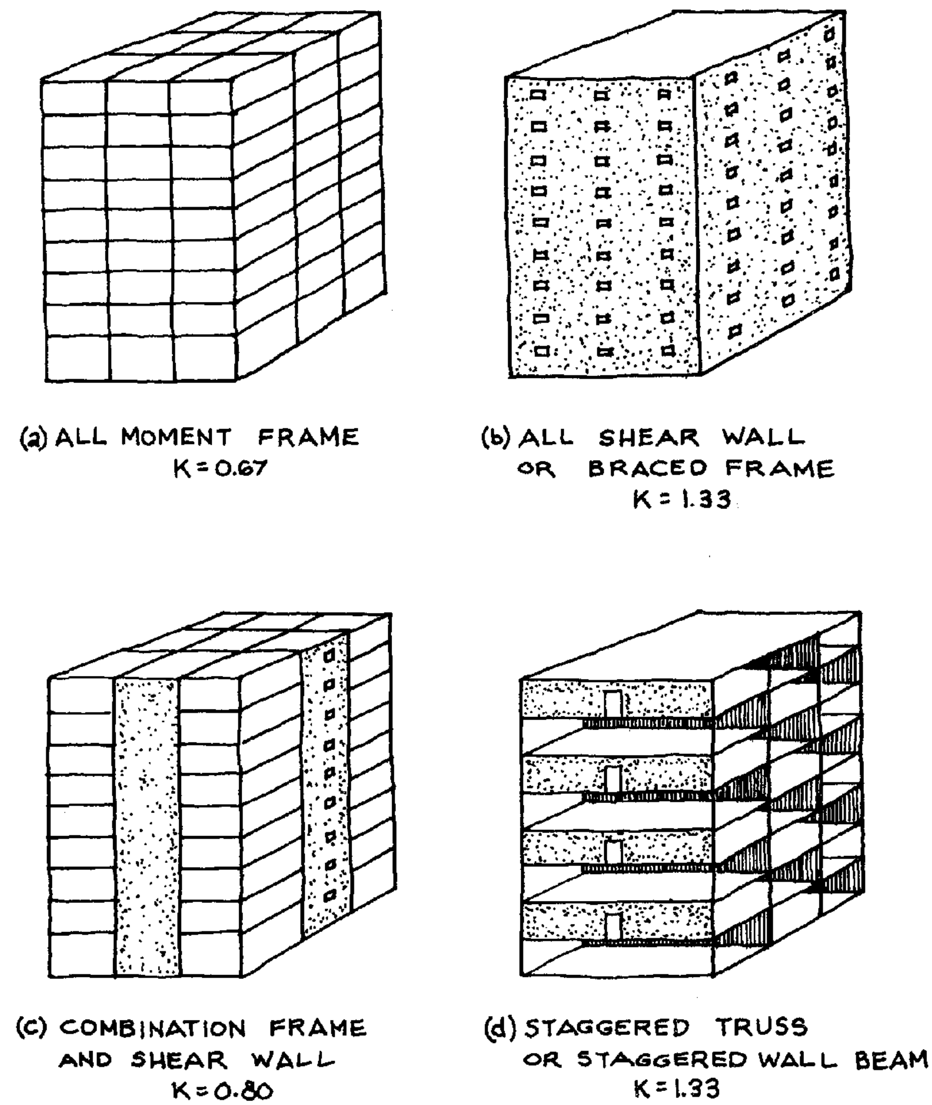
Most of the early thinking on ductile structure referred to the use of structural steel. In the early 1960's the concept of the ductile reinforced concrete was developed to gain better acceptance of reinforced concrete moment-resisting frames for earthquake-resistancy design. As a result of the poor performance of non-ductile concrete frames, a phenomenon observed in many earthquakes, it is now a code requirement that any concrete frame that is part of the earthquake-resisting system must be designed as a ductile concrete frame. The codes include a large number of specific requirements that are significantly different from those used in normal reinforced concrete design. They also require a consideration of the stresses within the panel zone; that is, the joint between the column and beam where the stresses must be transferred from bending of one element into bending of the other element. This creates very high shear forces within that panel zone. Prior to the use of ductile concrete, that zone was mostly ignored. Now, however, that area within the intersection of the column and the beam must have stirrups and ties to provide resistance for the shear stresses in the panel zone. In addition, the code also requires that a certain amount of the top and bottom steel in the beam be continuous in order to provide both positive and negative moment resistance at all points along the beam. Again, this corrects a defect that had been observed in several earthquakes; failures had occurred on horizontal members because of the cut-off points of the top steel. The beams were exposed to negative moments at points beyond which they contained any top steel. So they simply failed. The other main consideration in ductile concrete is that the material must be thoroughly tied together to provide enough stirrups and ties so that the bars, which may be in compression, will not buckle, thus, the concrete will not fail in shear. The concept is that failure must occur through yielding of tensile steel, without compression or shear failure. It is also necessary to confine the concrete to give it a high strength.



CONCRETE BEAM-COLUMN PANEL
Figure 12.6

BUILDING FRAME SYSTEMS

In considering larger buildings, I would like to discuss briefly several of the different types of framing systems that have been used over the years. Figure 12.7 shows diagramatically four basic framing systems; a moment-resisting frame; a shear wall system; a combination of the shear wall and the moment frame; and a staggered truss system. The latter is something of a special item which will not be considered here.

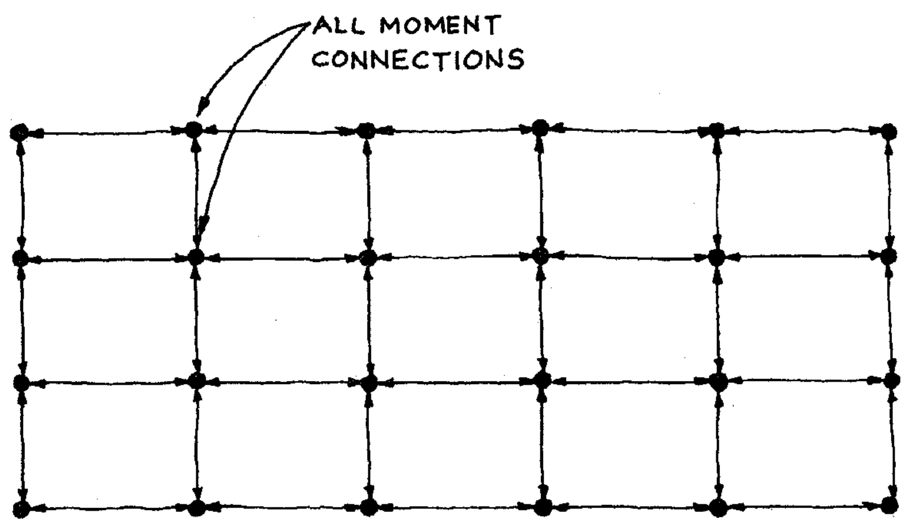


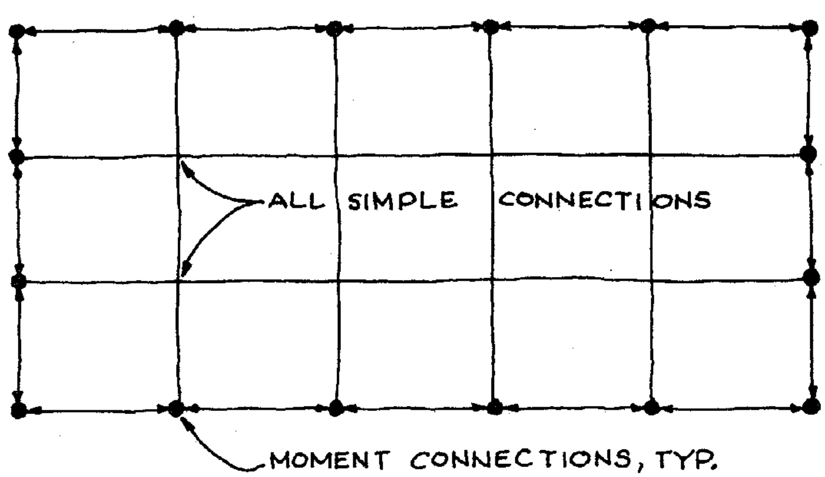
TYPES OF FRAMING
Figure 12.7

A shear wall is normally a wall of reinforced concrete, reinforced masonry or plywood. However, steel shear walls have been used. Further, braced frames with members evidenced as X-braces or K-braces are considered as a type of shear wall. Recently, there has been a good deal of research work done on the eccentric connections of a standard steel braced frame system that allows a structure to perform as a moment-resisting frame.

The moment frame was originally designed with all beams and columns connected for moment resistance for lateral load. Designing it that way provided a great deal of redundancy in the building. There could be a failure in one member, yet the other members would pick the remainder and allow the whole structure to continue to perform. This matter of redundancy is very important in seismic-resistant design. It needs to receive a great deal of consideration. For lack of redundancy can lead to the collapse of a building.

Figure 12.8 indicates some of the changes that have occurred in the concept of the moment frame. As just stated, the original intent was to achieve redundancy by having all beam column intersections tied in with moment-resisting connections. For that, the code gave moment frames a K-factor of 0.67 in recognition of their ductility and redundancy. However, many designs have not carried out that intent. This particular example shows that only the exterior columns have moment connections. Furthermore, recent buildings have been designed with larger and more widely spaced columns. The result? Less redundancy.

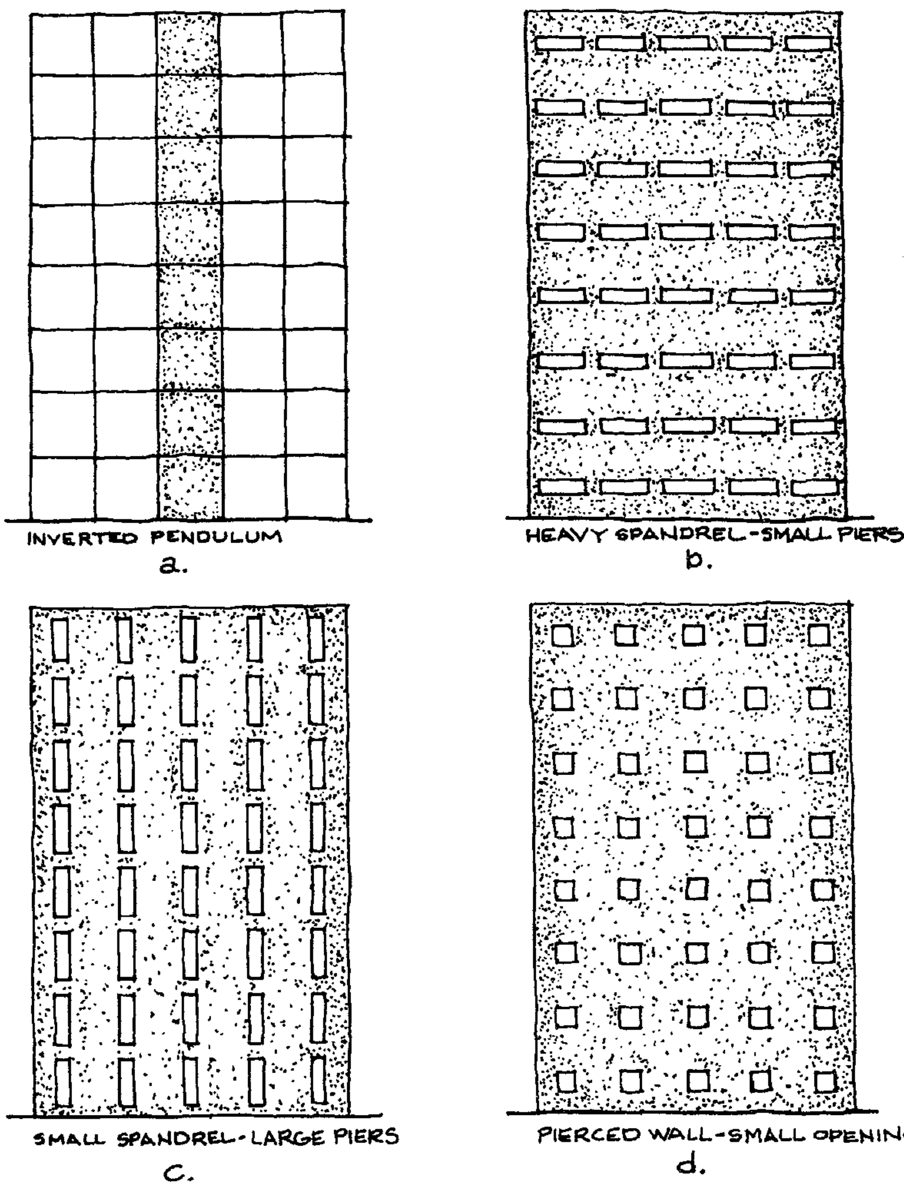




PLAN OF TYPICAL BUILDING (K = 0.67)
Figure 12.8

There has not been much exposure of this type of building to recent earthquakes, so we do not know how they are going to perform. But structures that have been designed in this manner are really only frame buildings without any shear wall stiffening elements and they lack redundancy. Clad in the new light-weight architectural curtain walls, they have little damping compared to some of the buildings that have been tested in earthquakes. This type of building will no doubt experience large deformations during an eathquake. Consequently, a great deal of nonstructural damage can be expected.

Figure 12.9 shows several different types of shear walls. All are designed for a K-factor of 1.33. This is twice the force for which a moment frame need be designed. The upper left hand corner indicates what would be referred to as an inverted pendulum. It is really more of a beam than a shear wall. Where there has been no reserve system behind this inverted pendulum shear wall, the performance has not been good.



SHEAR WALL BUILDINGS
Figure 12.9

The design on the upper right hand corner shows the type of shear wall that has very heavy spandrels and small piers. Performance of this kind of shear wall has shown problems because the short stubby columns have been weak. This was observed in the Japanese earthquake in 1968 in several of the schools in that area. The damage occurred largely in the columns.

The third type of shear wall (at the lower left) shows heavy piers and small spandrels. Because of the great relative stiffness of the piers compared to the spandrels, the performance of the building is changed. A traditional analysis would assume points of inflection at the pier mid-height and the spandrel mid-height. In this case, however, the spandrels are not stiff enough to provide the flexity to allow those points of inflection to occur. Figure 12.11 shows a part of an exterior wall for the full height. The figure also shows the original design moments and the piers assuming points of inflection at each mid-story height. However, a computer analysis considering the stiffness of the elements of the wall reveals that there is in fact no point of inflection below at least mid-height of the building and that the moments actually increase substantially out to the bottom.

FAILURE OF SMALL COLUMNS
SCHOOL BUILDING - 1968 JAPAN EARTHQUAKE
Figure 12.10

The design on the upper right hand corners shows the type of shear wall that has very heavy spandrels and small piers. Performance of this kind of shear wall has shown problems because the short stubby columns have been weak. This was observed in the Japanese earthquake in 1968 in several of the schools in that area. The damage occurred largely in the columns.

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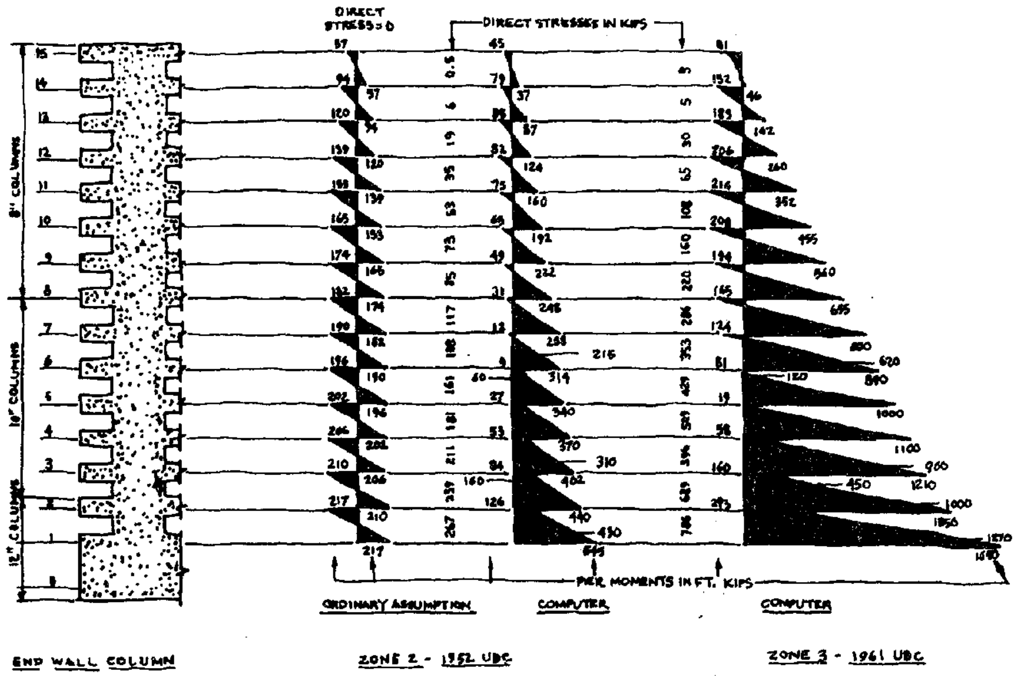
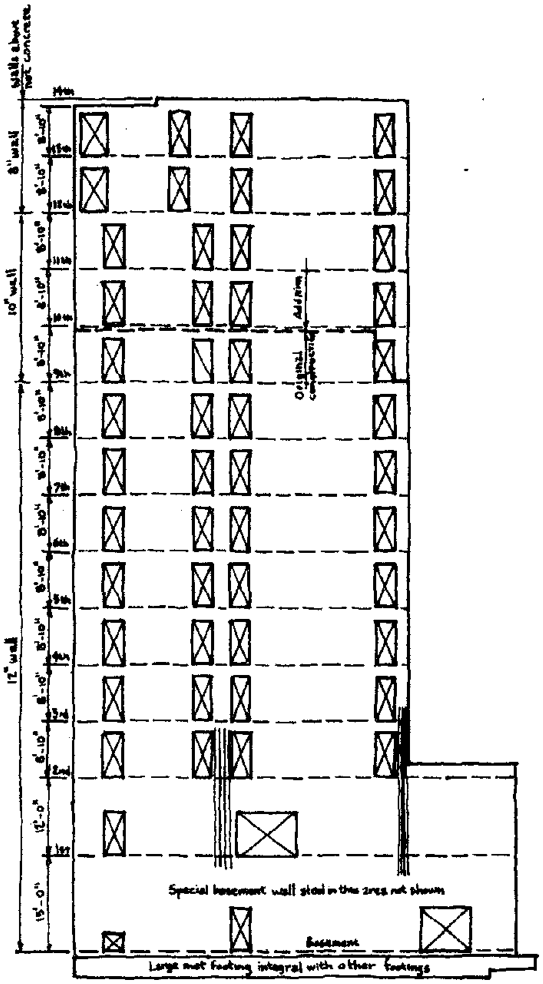
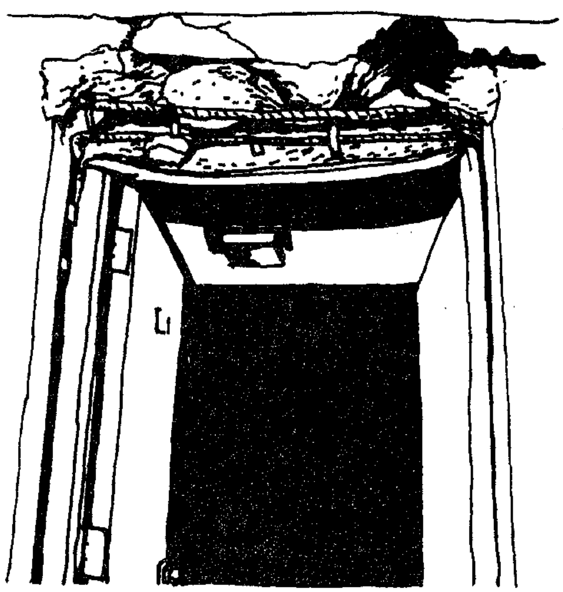


DIRECT STRESSES AND MOTION IN END WALL COLUMN UNDER THREE DIFFERENT ANALYSES
Figure 12.11

The fourth type is an essentially solid shear wall with relatively small openings. Normally such a wall has performed well. Yet shear wall problems have been observed. Consider the Anchorage Westward Hotel in Alaska, a light steel frame with shear walls. Figure 12.12 indicates an elevation of one of the shear walls and also indicates the problems resulting from the door openings being stacked one above the other in each story. The problem is in resisting the shears that result from overturning. As the building tends to overturn, for example, in a clockwise direction, tension is put on the left hand side and compression on the right. The stress must be transferred across the building and it must be delivered through the spandrels that occur over the door. But in a building of this sort with relatively low story height, the spandrels are shallow and weak.



SHEAR WALL
ANCHORAGE - WESTWARD HOTEL
Figure 12.12



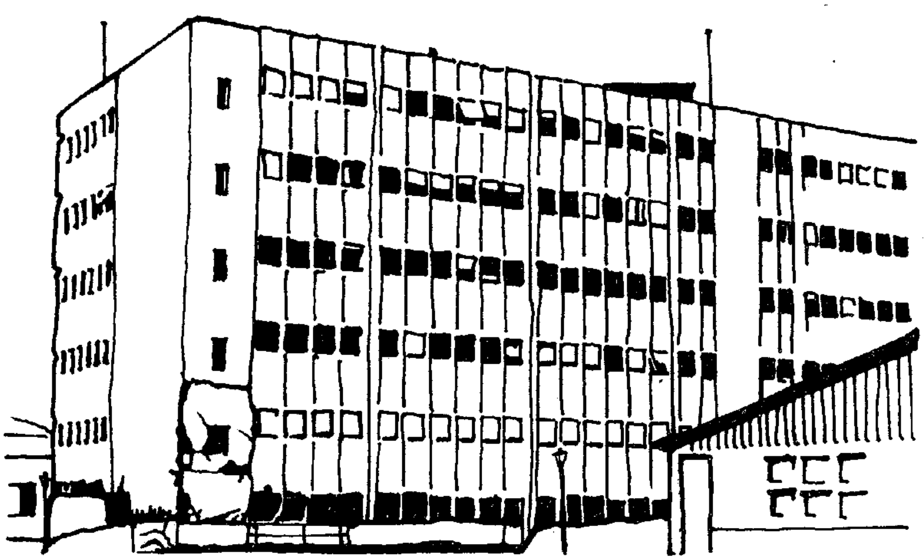
SPANDREL DAMAGE
ANCHORAGE - WESTWARD HOTEL - 1964 ALASKA EARTHQUAKE
Figure 12.13

The combination of a frame and a shear wall has been assigned a K-factor of 0.8. In other words here you can design for less load than you could for a strictly shear wall building. Why? Because there is the redundancy gained by having the moment-resisting frame as a back-up system to the shear wall. The system has several advantages: it provides a stiff structure for resistance to wind as well as small-to-moderate earthquakes; it limits the deformation that a building has to go through; and it also has the redundancy of the back-up moment steel or in some cases the ductile concrete frame.

INTERACTION OF STRUCTURE AND NONSTRUCTURE

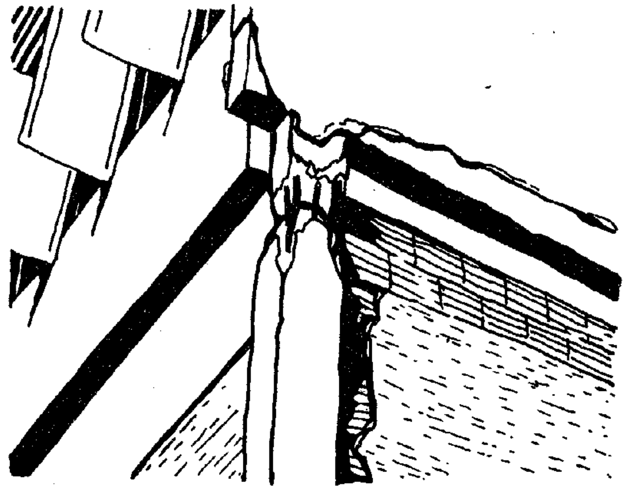
During an earthquake, action of many non-structural components may have a major effect on the structure itself. We have observed this kind of problem in most of the recent earthquakes. Generally, this is a consequence of not really being aware of the problem and a lack of team work between the architect and the engineer. There are any number of examples to choose from, but let us begin with a building in Anchorage, Alaska, during the 1964 earthquake.

The Cordova Building was a structural steel frame building that had been designed to resist seismic forces. Figure 12.14 shows one of the corners of the building. The corner column had been used as a support for steel stairs. The effect was to stiffen the column. This modified the performance of the building and caused the column to buckle with resultant damage to adjacent concrete panels.



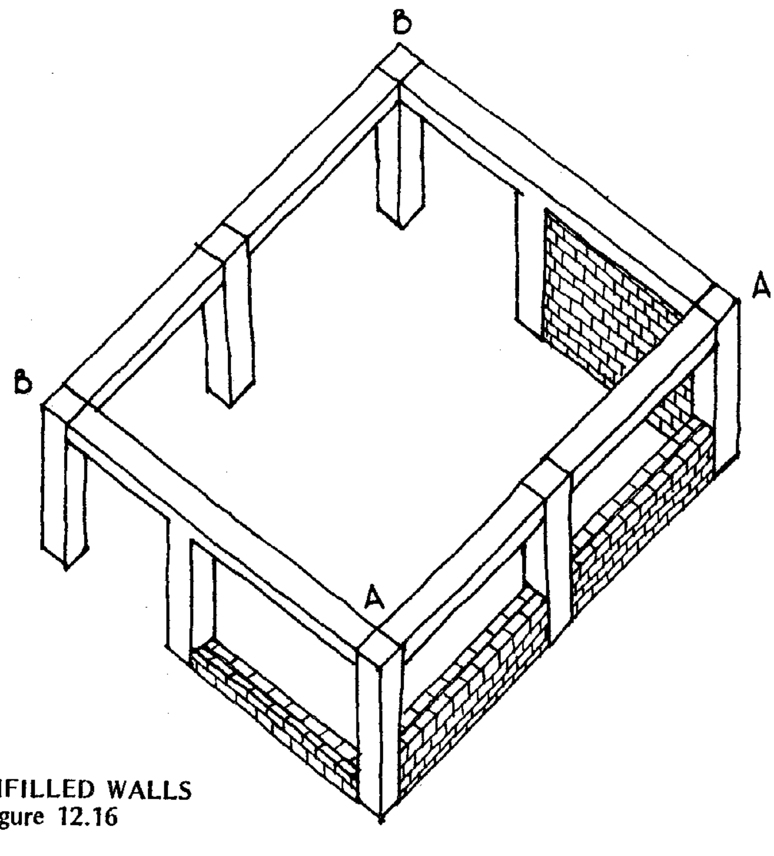
FAILURE OF STAIR TOWER
CORDOVA BUILDING - 1964 ALASKA BUILDING
Figure 12.14

The 1967 earthquake in Caracas revealed dramatically how infilled masonry walls can affect the performance of buildings. Figure 12.15 shows the Mene Grande Building, a structure approximately 15 stories high. It had been designed as a moment-resisting frame with all of the frames intended to share in resisting the lateral forces. However, the exterior walls of the buildings were infilled with clay tile walls between the columns. This so stiffened the exterior frame lines that they drew a much larger portion of the earthquake forces. The result? Column failure in the lower stories in all of the corner columns. True, these were not exactly ductile concrete columns, but they were designed at that time to meet the ACI requirements.



STIFFENING OF COLUMN
MENE GRANDE BUILDING - 1967 CARACAS EARTHQUAKE
Figure 12.15

I would like to pursue this matter of infilled walls by looking at Figure 12.16 which shows a one-story structure of reinforced concrete. In this case it is designed with columns and beams in both directions. Assuming that the columns are all the same height, they are the same rigidity; therefore, the earthquake load is divided equally among the columns on both sides. Now, if for some reason the design of the building had included infilled walls of different heights, those infilled walls would have stiffened the columns. Considering the longitudinal direction, the columns along line A would have been stiffened to a much greater degree because they are short and stubby, while on line B the columns remain much more flexible. Now this changes the distribution of the forces so that almost all the earthquake force has to be resisted by the columns on lines A. This means that those columns are exposed to about twice as much shear as they had originally been designed for, so they are subject to failure.



INFILLED WALLS
Figure 12.16

It is obvious that the action of the structure will have a significant effect on the architectural, mechanical, electrical components as well. Recently, such damage has caused increased concern because of exposure to falling hazards, the possibility of debris in exit ways, the desire for a building to continue functioning and because of the cost of putting buildings back into use following an earthquake. This type of damage is now getting more consideration in the design of buildings, particularly in buildings of important function, such as hospitals and communications centers.

In conclusion, if there is one critical general rule to talk about in terms of seismic design, it would be this: tie the building together. Particular attention must be given to collectors and chords in order to deliver the seismic forces to those elements that will resist the forces. This is especially important in odd-shaped structures, or with shear-resisting elements located unsymmetrically to the building configuration. It is also extremely important to tie together buildings that are composed of units. Pre-cast elements are normally just put together like a house of cards without much attention paid to the type of connection between elements.

Unfortunately, judging by the damage observed in past earthquakes, the connections that are used have been of a very brittle nature and they have failed. So that in addition to just tying a structure together, consider the kind of detail that is needed to connect and interconnect the various elements.

All elements of a building must also be tied together in a positive manner. Walls, for example, must have sufficient strength to resist out-of-plane bending. They must be sufficiently anchored into the structure itself so that they will not fall out. One should be able to follow the path of seismic forces all the way from the point of origin to the other end of the spectrum and provide a thoroughly tied together system to resist the forces.

All of which is to say that we must recognize that design to resist earthquakes is a completely different process from the normal planning that goes on when architects and engineers sit down to design a building.

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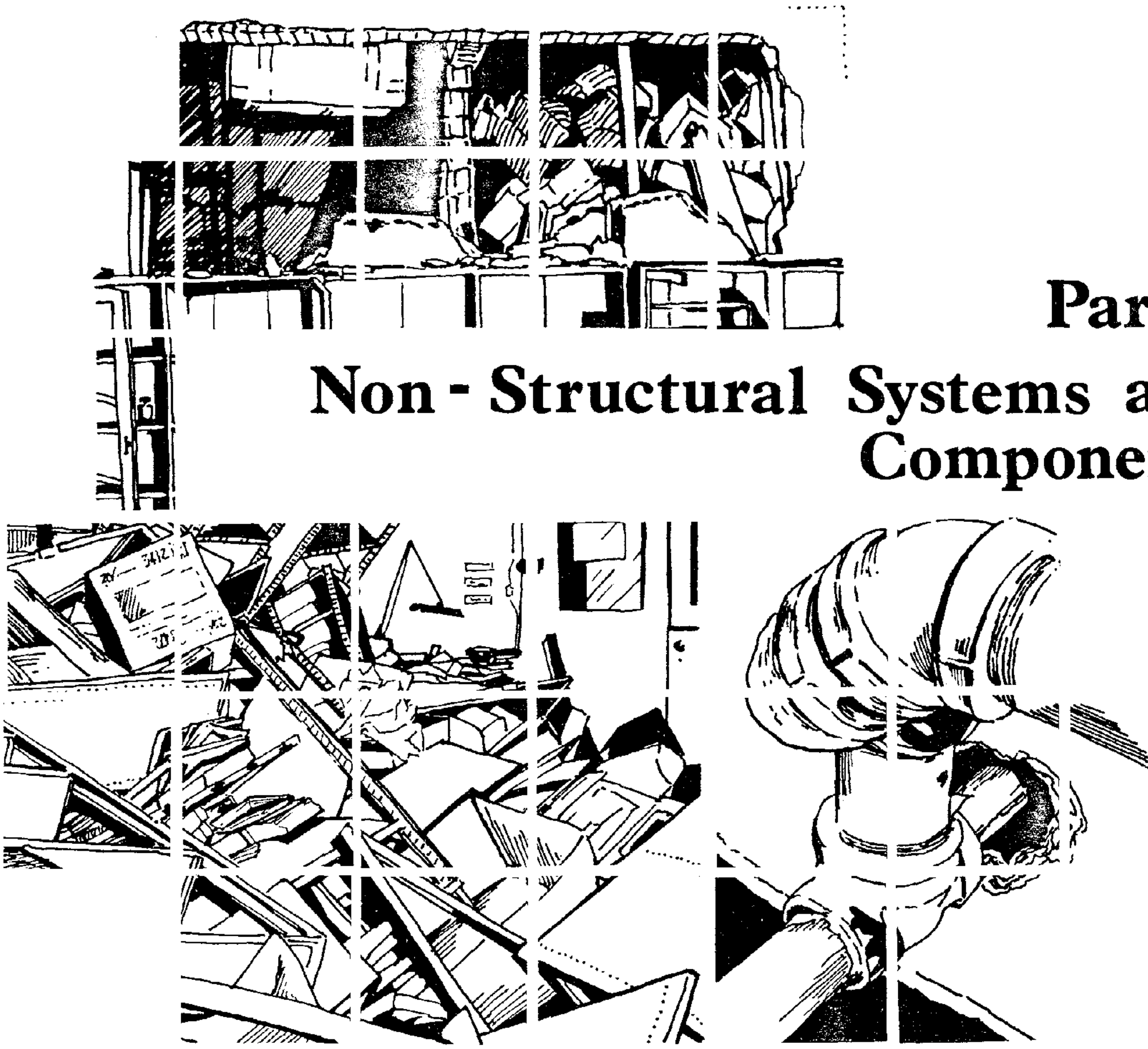

Part 5
Non - Structural Systems and Components

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Chapter 13. NON-STRUCTURAL HAZARDS

Arthur E. Mann, F.A.I.A.

OVERVIEW

Earthquakes are one of our world's most incredibly violent phenomena, and as the world is a geologically dynamic planet, it must follow that our development of the earth's surface will be subject to the liabilities of those dynamics. California is earthquake country and has recognized earthquake dynamics as a fact of life and possibly with Japan, leads the field in earthquake research.

Prior to the San Fernando, California earthquake of 1971, a most impressive amount of work had been done by engineers and scientists resulting in the availability of voluminous literature. It was the San Fernando event, a relatively minor event which again made the public and public officials aware of the inherent dangers of earthquakes and the lack of adequate disaster preparedness. The quake also gave support to the concerned design professions and scientists to accelerate their studies and to further disseminate information to the public for their education and safety. This is now done on a continuing basis as the attention span of the lay public is notoriously short.

There are few areas in the world where there is no record of earth movement and where the calculated chance of an earthquake is slight. Almost all areas in the United States have experienced earthquakes (the two largest events experienced were not in California) with enormous losses in lives and property.

It is now several years since the 1971 San Fernando earthquake, and although Southern California public officials wanted to mitigate the dangers and started immediately to do so, they have had to move with caution and make sure that their decisions are right. One mitigation step is to make an accurate count of the hazardous structures in the City and County of Los Angeles; it is estimated that the area contains between ten and twelve thousand such structures with some one-hundred thousand in the entire state. Almost all of these structures are constructed of unreinforced masonry. Another step is to develop ordinances on a phase by phase basis. The economic and social problems are vastly more difficult to solve than the engineering problems.

Earthquakes are only one of a number of types of events that can be classified as disasters; among them are fire, flood, wind, and insect deflorations. It is calculated that annually there are some 25,000 events in the United States which can be classed as disasters; this is about 60 per day. With this in mind we should lump such types of events together and try to solve or study them to aid the development of national policy. Nevertheless, there is no way to anticipate life, property, money losses, or develop safety measures which could provide a completely risk free situation.

Knowing the dangers faced, acceptable risk factors have been developed, and ordinances and laws can and have been passed that take them into account. Two laws in California which have been shown to be very successful, are the Field Act for schools from kindergarten through junior college, and the Riley Act for all other structures excluding some farm buildings.

The Santa Barbara City earthquake of 1925 was most damaging and caused the State Chamber of Commerce to commission the architectural and engineering societies to write an ordinance for earthquake mitigation. When the 1933 earthquake occurred the ordinance was ready and was adopted within a month by the State Legislature. Both the Field and Riley Acts are statutory law and do not in themselves cover detailed technical requirements. The schools were turned over to the then State Division of Architecture which would develop and enforce their detailed technical design requirements. The Riley Act mandated that communities pass ordinances to cover earthquake requirements. All earthquake ordinances are constantly updated as experience grows and new scientific data is developed.

When the Field Act passed in 1933, it did not have any retroactive provisions and applied only to new buildings. So school trustees ignored it even though it was well-publicized that they were housing children in very dangerous structures. In 1939, the Garrison Act was passed which said that "all buildings should be inspected." So they were inspected, and the inspection reports were filed away.

In 1965, the first of the two Green Acts were passed, at the same time the Attorney General in the State said that school board members were personally liable for any damage to buildings or occupants. In 1967, the second Green Act was passed which said that "all buildings shall be inspected and the building declared either "safe" or "unsafe." If it was unsafe, it had to be retrofitted or abandoned by 1975. As of 1977 all school buildings in California—elementary through junior college have either been structurally rehabilitated or abandoned.

It has been unfairly stated that the Field Act causes school construction costs to be disproportionally high. Changing teaching requirements and larger school sites also contribute to the total expense. The Field Act does not dictate size, shape or use of a school building; it merely assumes the safety of such structures.

For schools constructed in California a Total Project Cost figure is required. This figure must include all costs for land, buildings, fixed equipment, furniture, inspections, and fees.

The Construction Cost, (bid) equals approximately 80% of the Project Cost (that is, minus fees, furniture and equipment).

Of the Construction Cost some 20% goes for on and off site development so that the actual cost of the buildings is about 80% of the construction bid or about 63% of the Project Cost.

A breakdown of the elements that make up a building shows that the parts of a building affected by the Field Act requirements are the foundations, floors, walls and columns, structural roof (beams, joists, decking) ready for wall enclosures, finishes, and weatherproofing materials. This constitute from 22 to 25% of the complete structure. All other elements—doors, windows, hardware and mechanical—make up 75% of the finished cost.

It is agreed by many design professionals that Field Act requirements (added materials and labor) might add as much as 3 to 5% of the cost of the structural elements of a building. Other costs such as additional clerical help/paper work, inspection and fees might bring the cost increase to as high as 4 to 5% of the construction contract or the Project Cost.

As an example, take a typical elementary school with a project cost of one million dollars. The construction contract would be eight hundred thousand dollars. The on and off site improvements would be about two hundred thousand dollars. The buildings would then be six hundred thousand dollars and the structural elements 25% of that or one hundred fifty thousand dollars. The cost of labor and materials to meet Field Act requirements are, taken on the high side, 5% of one hundred fifty thousand dollars or about seventy five hundred dollars, rather cheap insurance for safety.

Another way of looking at costs is to estimate the increase in total project costs. I will use a high at the figure of 4%. On a million dollar project, 4% equals forty-thousand dollars. A California Commission in an official report states that a Field Act school structure should last a minimum of fifty years. This amounts to an additional cost of less than one thousand dollars per year. Insurance rates in Field Act structure have been 1/3 to 1/2 less than on Pre Field Act buildings.

The experience gained from almost fifty years of the Field Act can be used on any type of structure for cost benefits. The only purposes of the Field Act are to insure that schools are built properly to protect the lines of occupants through careful checking of plans and strict inspection of construction, and to protect the public investment.

Since 1933, the State of California has processed some 39,000 school projects totalling between eight and twelve billion dollars. Losses from earthquakes have been less than negligible. An average cost for seismic protection of 4% on ten billion dollars of school construction might be too high a price to pay from the public viewpoint but, that is a matter of judgement.

POTENTIAL EARTHQUAKE HAZARDS

Components to be considered other than the basic structure and outside wall enclosures, inside partitions, intregrated ceiling systems made up of combinations of lighting and ventilation systems, plaster acoustical materials and air and electrical distribution systems. These should be carefully tied into structure or wall systems to prevent swage and pounding, possibly some free floating to allow movement and action as a unit, with adequate space allowed at edges for such free movement.

The following elements are usually ignored in earthquake design and which, if not properly anchored, can and do cause considerable damage to the basic building structure, other nonstructural elements can possibly injure occupants.

CEILING FAILURE
1972 MANAGUA, NICARAGUA EARTHQUAKE
Figure 13.1

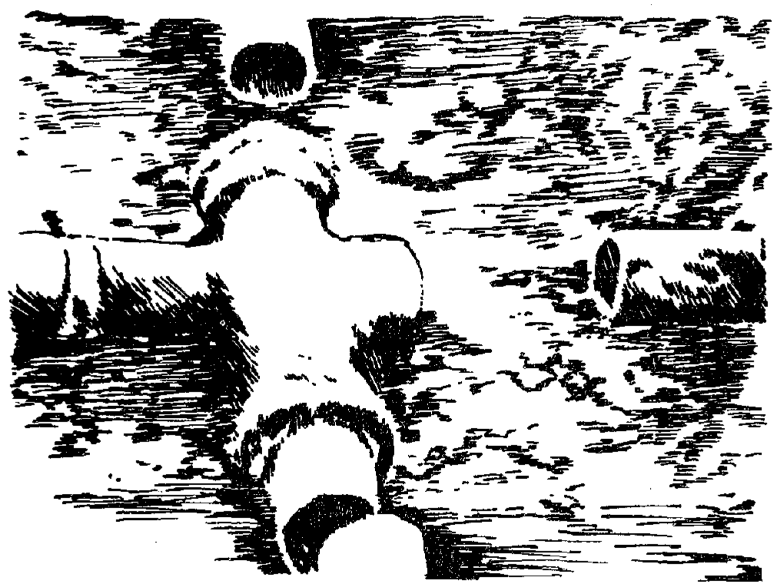
Experience in California has shown that ceilings should be allowed to float free from the surrounding walls or anchored firmly to at least two walls. Failures occur when pendant light fixtures are not braced and when plastic diffusion grids or lenses fall. All acoustic panels or tile should be mechanically anchored to the ceiling system, either winded or nailed.

GLASS BREAKAGE
OLIVE VIEW HOSPITAL - 1971 SAN FERNANDO EARTHQUAKE
Figure 13.2

Doors, windows and glass enclosures become jammed trapping occupants. Glazed elements are wracked stressing the glass which tends to explode throwing shards indiscriminately for considerable distances. The detailing of glass elements should allow for movement either by flexible frames or by providing soft material such as rubber or neoprene to hold the glass at its edges in its frame.

UNREINFORCED PARAPETS - 1969 SANTA ROSA EARTHQUAKE
Figure 13.3

It is appropriate to single out one element of existing buildings that is typically overlooked—parapets. In 1947, Los Angeles City passed an ordinance requiring parapets and cornices to be removed, replaced, or reinforced on buildings erected prior to the 1933 earthquake. As of this date 1979, there has been almost 100% compliance. Very few other communities have similar ordinances or have enforced them. All masonry materials, if used as an enclosing material must be reinforced and anchored to the basic building frame. If the masonry is used as the basic structure it must be reinforced and all floors and roofs be designed as diaphragms adequately anchored to the masonry in order to make the structure act as a unit and transfer the forces generated to the ground.

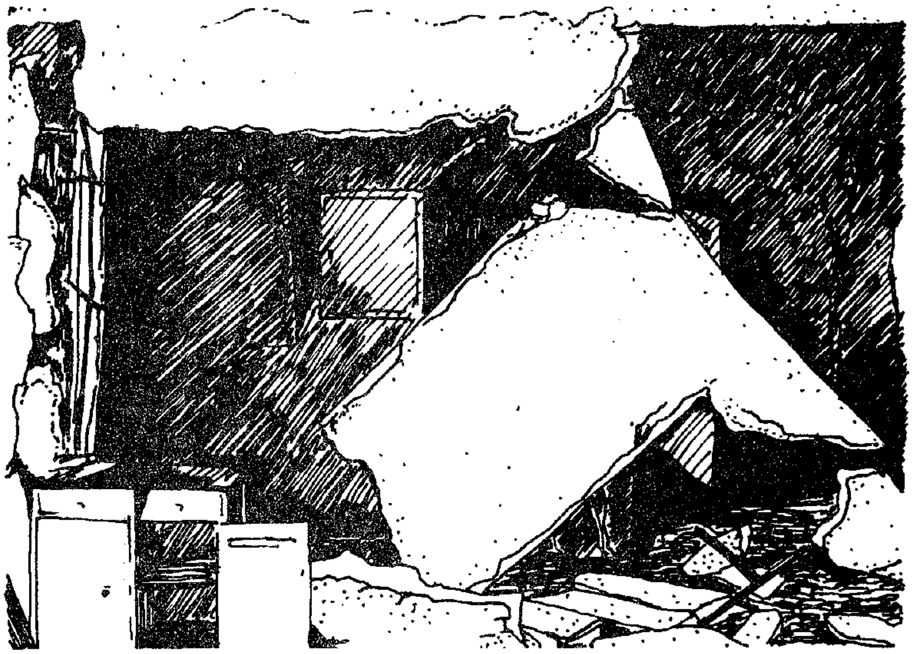


DISRUPTED LIFELINES - 1971 SAN FERNANDO EARTHQUAKE
Figure 13.4

Lifeline systems are areas of great concern in the earthquake design of strucutres. The primary systems; communications, people movement, water, gas, and electricity, if damaged, can disrupt all rescue attempts and negate safety measures. Concern should not be limited to lost structures, but to total area distribution primary systems. Perhaps not part of this, there should be awareness of other lifelines which can affect public safety and secondary structures. That is the vulnerability of highways, airports, bridges, railroads, and ferry systems. In California, the major water supply lines stretch hundreds of miles across faults,deserts and mountains.

All out efforts should be made to provide flexible ducting systems to prevent breakage even where firm anchorage is required.

Full-height partitions, as shown in Figure 13.5, can collapse and cut electrical conduit and water lines. Full-height partitions are also usually fairly rigid and can cause damage to the structure. If the plan does include partitions, they either should be braced or interlocked by angles or at floors so that they will stand up during an earthquake. There are many kinds of interior wall partitions and panels well detailed. Permanent partitions around stairs, toilet rooms and passageways should be regarded as basic structure and allowances made for them.

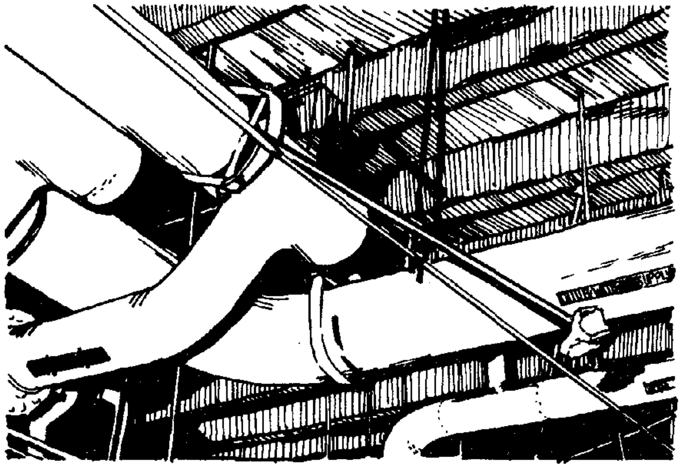


PARTITION FAILURE
SOCIAL SECURITY INSTITUTE HOSPITAL - 1972 NICARAGUA EARTHQUAKE
Figure 13.5

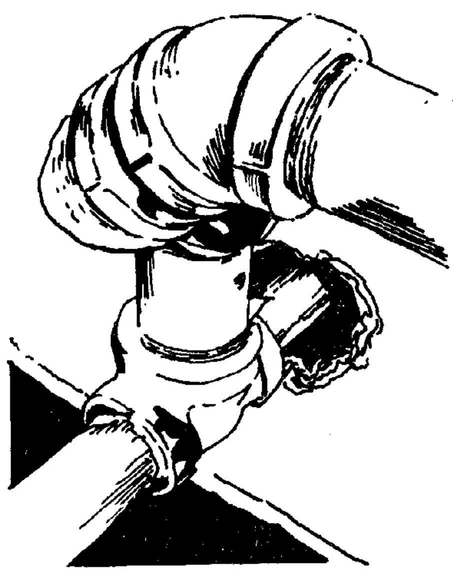
Furniture and portable equipment in buildings are usually unanchored. Desks, filing cabinets, containerized equipment, chairs, and small objects actually can become deadly flying missiles. In the 1964 quake in Alaska, typewriters were actually thrown through partitions in a military installation. All such should be anchored if possible. Tall bookshelves should be equipped with a wire stop or bar to hold books on their shelves. Grocery and liquor stores are particularly vulnerable to shelf damage.

BUILDING CONTENTS - 1971 SAN FERNANDO EARTHQUAKE
Figure 13.6

There are many other hazards as well that are not usually recognized as such. Art works, decorations, clocks, and shading devices can become very hazardous missiles. Refrigerators, which are usually on some kind of a wheel system, can careen across the room. Bookcases and tall cabinets should be anchored to studs in the wall. Storage racks can be quite hazardous as they usually are loaded with heavy items; they should be anchored, not just to the floor, but to the ceiling and anchored to each other. Hospital supplies—and this is really a critical matter, especially if they are drugs—can be destroyed when they are needed most.



PIPING FAILURE
JUVENILE FACILITIES - 1971 SAN FERNANDO EARTHQUAKE
Figure 13.7



STANDPIPE FAILURE
LOCKHEED BUILDING - 1971 SAN FERNANDO EARTHQUAKE
Figure 13.8

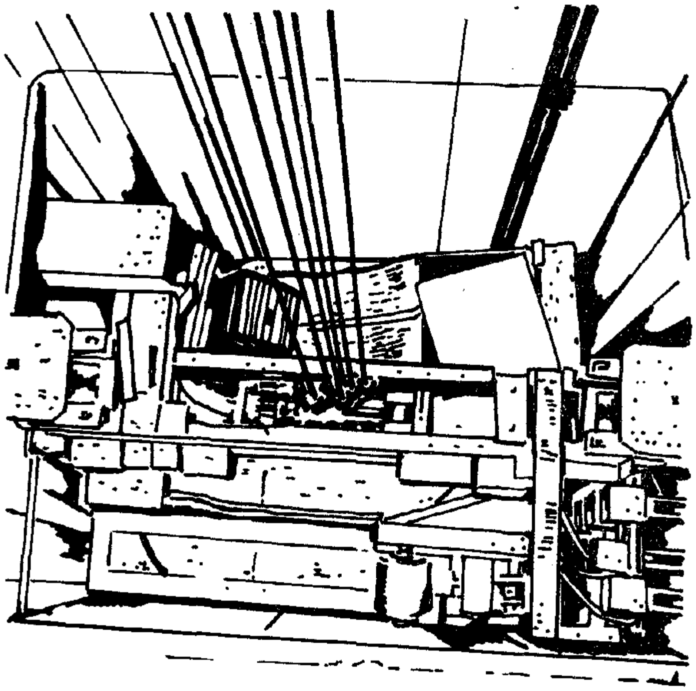
The piping and ducts of heating and ventilating systems can vibrate and break during an earthquake. Many vibration mounts have failed along pipes and ducts because of stress or pendulum action. Components of ceiling brackets often fail from overstress. All should be braced and properly designed for earthquake action.

Enclosures and mountain devices for electrical equipment and pumping systems fail because they are almost always not designed as a structural unit. Plumbing systems, water systems, and sprinkler standpipes are most critical. Pumps and mounts become loose; sprinkler standpipes can be broken, reducing the fire protection of the building. Usually rather simple bracing devices will take care most situations. Emergency generator systems, fuel power sources, and so on, can also be damaged very easily. Incomplete as it is, such a list of hazards makes one aware that earthquakes do not selectively damage the structural system, but everything in the building.

DISRUPTION OF POWER SOURCE
COMMUNICATIONS BUILDING - 1964 ALASKA EARTHQUAKE
Figure 13.9

What, then, can be done to reduce the hazards posed by the nonstructural components within the buildings? The recent actions in California to mitigate the potential threat posed by elevators is a good example. Los Angeles was the only city in the country that has its own elevator ordinances. In the 1971 San Fernando earthquake, the behavior of elevators caused these ordinances to be enacted. About 700 elevators were put out of commission out of 10,000 to 12,000 in operation.

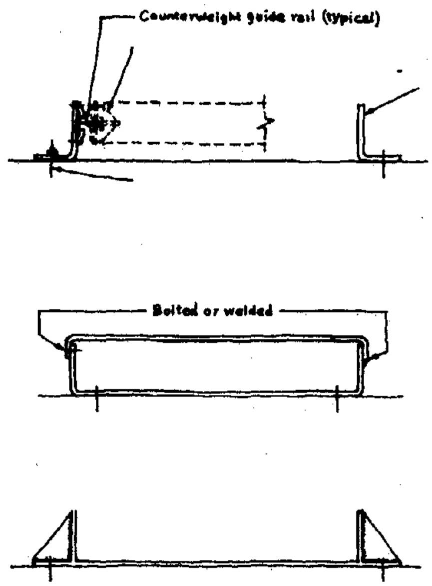
There are several reasons why counterweights broke and bent their guide rails so they swing free causing cable and brake shoes to fail, shearing electric cables and in some cases the counterweights smashed through the elevator cabs.



COLLAPSE OF COUNTERWEIGHT
UNION OIL BUILDING - 1971 SAN FERNANDO EARTHQUAKE
Figure 13.10

Further damage was caused in the elevator machine housings which are usually placed at the top of buildings. The mounts, the hoists, the motors were thrown off their bases, typically because they were not adequately anchored. These danced around and cut frequently both the support and the electrical cables. Fortunately, cabs are suspended by at least three cables, each one capable of holding the cab. But, when the equipment starts moving, it can injure people, start fires by breaking wires, smash electrical panels, and damage other equipment.

What steps can be taken to lessen such risks? The counterweights must be securely held in their guides and that the guide rails are securely anchored with extra brackets and bracing. We are required to install movement indicators to stop the cars when counterweights are shaken enough to pose a danger. In a fire, smoke detectors will automatically recall at least one elevator in each bank to the ground where it will be taken under the control of the fire department. This is so they can use the elevators in case of a general blackout or power failure powered from independent power sources. There are a number of variations, depending on the types of buildings and types of elevators and as the age and height of the building requires. All elevators have been brought up to ordinance. Costs are heavy to bring up to date the 10 to 12-story buildings that are maybe 20 to 40 years old. It will cost $3,000 to $4,000 per car for fire safety alone. To make them earthquake-resistant, it will cost another $6,000 to $8,000 per car, or an expected $9,000 to $12,000 per elevator. Only some 3,000 out of approximately 11,000 to 12,000 elevators will be affected in the city, however, because about two-thirds of the low-rise elevators are hydraulic.



COUNTERWEIGHT GUIDE RAIL BRACKETS
Figure 13.11

In modern structures, two thirds to three quarters of the cost of construction may not be part of the basic structural system. Structures must be regarded collectively, with all their components interacting, rather than as individual building elements. In other words, every component affects something else, making it very difficult to draw a line between the hazardous interplay of structural and nonstructural elements.

In analyzing the design of a structure, each basic element must be considered as an earthquake hazard. Architectural, structural, mechanical, and electrical designs must first be looked at as complete systems. Then they are analyzed down to the finest details of their individual components.

A building complex, connected, but made up of different sizes and shapes can have varying sway characteristics. In an earthquake, high narrow buildings next to squat structures could bang into each other causing considerable damage if not separated by spaces or structures. The spaces in connected structures are covered with flexible joints which if wracked or broken are easily explained.

Each major element of a structure, in earthquake design, should be considered from foundation to roof, and care taken to make sure that all are tied together; that the forces generated by an earthquake (both horizontal and vertical) are transferred to and absorbed by the ground.

CONCLUSIONS

All new buildings should be designed to counter the effects of both natural and man-made disasters: earthquake, wind, fire and flood. Present hazards can be mitigated by thoughtful application of resisting technical knowledge and experience. Careful inspection of potentially hazardous existing buildings can determine whether to rehabilitate or remove. A decision to phase out a structure when study might reveal a better and more profitable use which can be made of that property. A decision to rehabilitate can give the structure a higher and better use with a resulting profit which will, within a reasonable payback period, more than affect the expense of the rehabilitation.

Education of the public to the dangers of disasters is a major need. The general public traditionally has had a very short term interest in hazard mitigation even after experiencing a disaster. The attitude that "it won't happen again" or "it won't happen to me" is a basic human tendency making us reluctant to invest in safety measures which will protect our investments.

The experience in California with the Field & Riley Acts has proven without-a-doubt that good design and strict inspection of new construction costs, perhaps, a maximum of four percent of the overall construction cost and could insure a maximum useful life for buildings and safety of their occupants.

Architects as a professional entity have not assumed their professional obligations to the public in regards to the safety of the structure they design. We have left those matters mainly to building appointment officials and engineers. After the 1971 San Fernando earthquake, building officials, engineers, and construction people were in the field immediately inspecting thousands of buildings while the A.I.A. Chapters had no plans and stood on the sidelines just like the lay public did—wondering what was going on.

There is a clear area of responsibility that we as Architects say we assume in regard to the total area of disasters. Let us assume it in both areas of research and public responsibility.

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Chapter 14. NON-STRUCTURAL DESIGN CONCEPTS

John L. Fisher, A.I.A.

ARCHITECTURAL PRINCIPLES

Practically every part of a building is subject to earthquake damage. Therefore, it is important that design consideration be given to nonstructural systems, their components, and attachments to the structure.

I am reminded of the statement in Ayres and Hayakawa's report on nonstructural damage in the Anchorage, Alaska, Earthquake of 1964. It reads . . . "The final measure of a well constructed building is the safety and comfort it affords its occupants. If, during an earthquake, the occupants must exit through a shower of falling light fixtures and ceilings; maneuver through shifting and toppling furniture; stumble down dark corridors and stairs, and then be met at the street by falling glass, veneers or facade elements . . . then the building certainly cannot be described as a safe building."

The subject of earthquake design is thus not a simple, clear issue. It is not an issue that can be resolved by the engineer alone. The design and choice of materials in the nonstructural systems will have an impact on the structure, just as the structure will have an impact on the nonstructural systems.

But, first, what are nonstructural systems and their components? Nonstructural systems include stairs, public and private corridors, doors, interior and exterior walls, ceilings, roofs, mechanical/electrical systems, seismic joints, equipment, furniture and artwork. Nonstructural components are materials or elements used in systems, and include unit masonry, preformed siding, gypsum board, lath and plaster, glazing materials, acoustic ceilings, tile veneers, and precast concrete.

The Basic Building Code rationale is that in a moderate earthquake a building should continue in service even with the probability of light structural damage. In the event of a major earthquake, considerable damage will no doubt occur; however, the building should not collapse.

In designing the structure and the nonstructural systems, the architect has to ask himself these questions: How can nonstructural systems be designed to withstand earthquake forces? Where should the architect concentrate his attention?

The architect is accustomed to thinking of the structure as being subject to design forces that are static; however, because of the ground motion and the response of the structure during an earthquake, the forces will be dynamic and the stresses may not always be predictable.

Where, then, should an architect concentrate his attention? Certainly life safety is probably the most important consideration. Total survival of the building, on the other hand, may or may not be important; it may be a vital function or an economic consideration. This is a question the owner and his architect have to answer.

To lessen the possibility of injury or even death in an earthquake the architect should

- Improve the performance of the building's nonstructural systems by anticipating structural response; and
- Integrate the building's components with the basic parameters . . . with special attention to life safety.

To establish a basic design strategy for the nonstructural system, the performance criteria must

- Protect the occupant within the building during the earthquake;
- Keep disaster control and emergency communications operable during an earthquake;
- Allow rescue and emergency personnel to enter the structure;
- Secure the safety of the building and personnel property; and
- Return the building to useful service.

BASIC PRINCIPLES OF SERVICE DESIGN

Nearly all buildings are a composition of flexible and stiff components. Improperly combining such elements may create problems in the building's performance during an earthquake. The result may not only be designs whose behavior cannot be predicted, but structural systems that may be affected as well.

Non-structural components must, therefore, be properly integrated with or isolated from the basic structural frame. There are, in short, two basic design strategies:

- Deformation approach in which components are designed with ability to absorb stress from the structure through elastic response; and
- Detached approach in which components are free from structural movement and avoid direct stress.

There are also two basic relationships between nonstructural components and structural systems:

- Effect of nonstructural components on the structural system; and
- Effect of the structural system on the nonstructural components.

Figure 14.1 shows an example of the first—failure of a column at a stairwell caused when the stair construction stiffened the column so that it absorbed forces higher than other columns. Figure 14.2 shows an example of the second—the failure of infill walls caused by column deflection.

Story drift should also be considered in the design of the exterior wall system. For instance an exterior wall that is anchored at each floor in a simple span is seldom affected by cumulative action; whereas an exterior wall that is anchored at the floor slab and is cantilevered both up and down can be severely affected, unless properly designed.

Simple shearing or racking due to story drift can be imposed on floor-to-floor components by lateral movement in the floor system. In some cases bending failures may occur because the movement is perpendicular to the component. Racking failure may also occur when components are tightly fitted against columns due to deflection action of the column. Under severe seismic drift conditions, shortening may cause a change in the floor-to-floor height, thus crushing the top of the partition.

The design of elements, systems, or components that are in contact with or in close proximity to the other system must be given special attention to avoid damage or failure being induced when seismic motion occurs. Thus, if the ceiling supports the wall, the intersection must be detailed to account for the effect of the movement of one in relation to the other.

NONSTRUCTURE EFFECT ON STRUCTURE
CORDOVA BUILDING - 1964 ALASKA EARTHQUAKE
Figure 14.1

STRUCTURAL EFFECT ON NONSTRUCTURE
Figure 14.2

The energy-absorbing capacities of nonstructural components are a function of the type of construction and the materials used. Brittle materials will have little strength beyond their elastic limit; whereas ductile materials can withstand many cycles of deformation into their inelastic range and will have a considerable amount of energy absorbing capacity. A combination of materials will sometimes produce nonstructural components that have excellent dampening qualities.

SEISMIC DESIGN OF NONSTRUCTURAL SYSTEMS AND COMPONENTS

Let us begin by reviewing the previous statement about building code philosophy and place it in terms relative to the subject.

The seismic resistance concept for a moderate earthquake is that a building should provide adequate protection to life and property; it should continue to serve its function even with the probability that light nonstructural damage will occur. In a major earthquake, a considerable amount of nonstructural damage will occur with some structural damage.

A comparison between the 1973 Uniform Building Code and the 1976 edition reveals that many changes affect the performance of nonstructural systems and components. Probably the most important change is the recognition that essential facilities and occupancies used for more than 300 persons should remain operational during and after an earthquake. These changes were made to the Uniform Building Code in response to the efforts of the Structural Engineers Association of California and the Applied Technology Council.

In their program known as ATC-3, the Applied Technology Council has published a document entitled "Comprehensive Seismic Design Provisions for Buildings." This document will have an impact on the future development of building codes because it recognizes the importance of nonstructural systems and their components. It places building occupancies in three levels of performances and prescribes performance criteria for each as well as performance for the building systems.

- Group III—Consists of facilities that are essential to post disaster recovery. It requires that hospitals, emergency operating facilities, and fire and police facilities be operational;
- Group II—Represents buildings housing high occupancy densities or occupancy that restricts the movement of the occupant. Public assembly of more than 100, day care centers, schools, office buildings and detention centers would appear in this category; and
- Group I—Includes dwellings and occupancies less than 100.

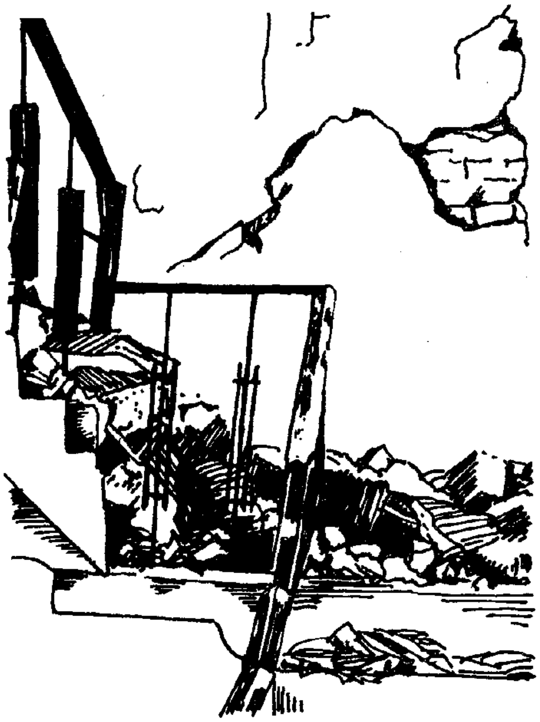
Store Fronts and Signs

The greatest hazard is falling objects within the glazed display area. Performance can be improved by using tempered or safety glass or designing the display area so that it will be protected. Special attention should be given where the display area and building exit occur. The means of egress should be free at all times.

Signs should be designed to provide seismic resistance in all directions. Many signs are supported on a pin connection and wires in three directions. Performance can be improved by providing a rigid brace.

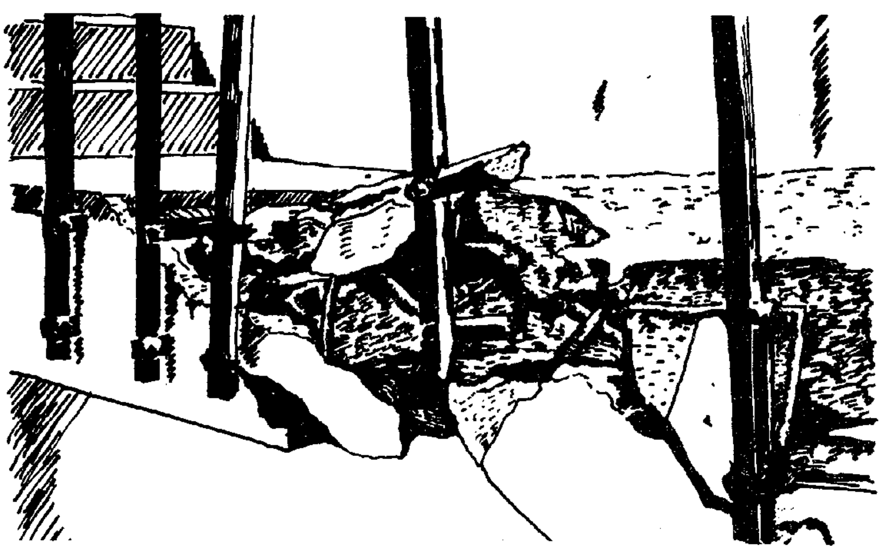
Exits and Corridors

Past experience indicates extensive damage has occurred in clay tile and unreinforced masonry walls. Damage also has occurred in reinforced masonry due to drift or shorting conditions as well as conditions where the concrete or reinforcing was not installed properly. Gypsum board and lath and plaster walls have performed better than masonry. However, when brittle finishes such as ceramic tile were installed, failures of the wall system did occur, see figure 14.3. There were numerous instances where building exits and corridors were blocked by fallen lockers, furniture, shelving units, and their contents.



BLOCKAGE OF STAIRWELL
BANCO CENTRAL - 1972 MANAGUA EARTHQUAKE
Figure 14.3

Stairways are often placed within the core of the building among the rigid elements. If they are placed where structural torsion may occur, the wall finishes may fail, see Figure 14.4. In the Anchorage, Alaska, earthquake of 1964, the Hill Building's lower floor stairs, which were fabricated in monolithic concrete, failed. The upper stories had metal stairs which were undamaged. The stairs in the Elmendorph AFB hospital, which were monolithic concrete and had construction joints at the floors, were also undamaged. Experience indicates that steel, concrete, and precast stairs can be designed for expected story drift.

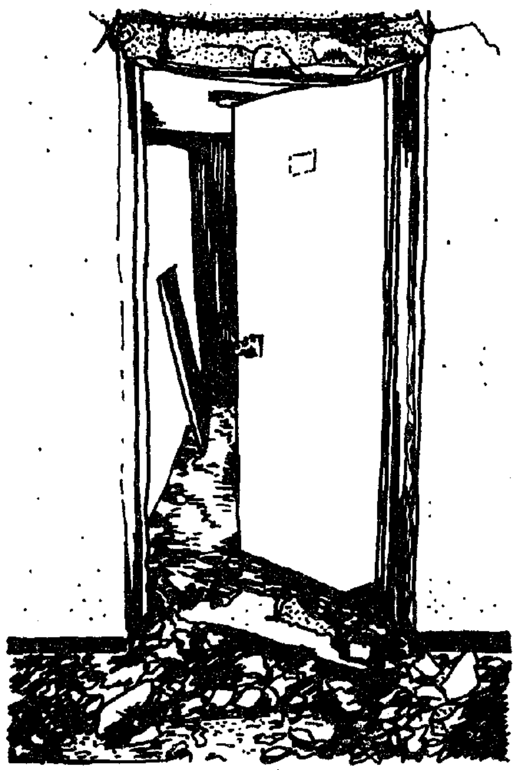


STAIRWAY LANDING FAILURE
Figure 14.4

Doors and Frames

Past experience indicates numerous earthquake failures in doors and frames. This is understandable considering that the door assembly must function with the normal clearances required for fire protection while the wall that contains the assembly must react to the structural movement.

It is possible to improve the performance of the door assembly by properly integrating it or effectively isolating the wall from the door assembly.



DOOR FRAME FAILURE
ANCHORAGE - WESTWARD HOTEL - 1964 ALASKA EARTHQUAKE
Figure 14.5

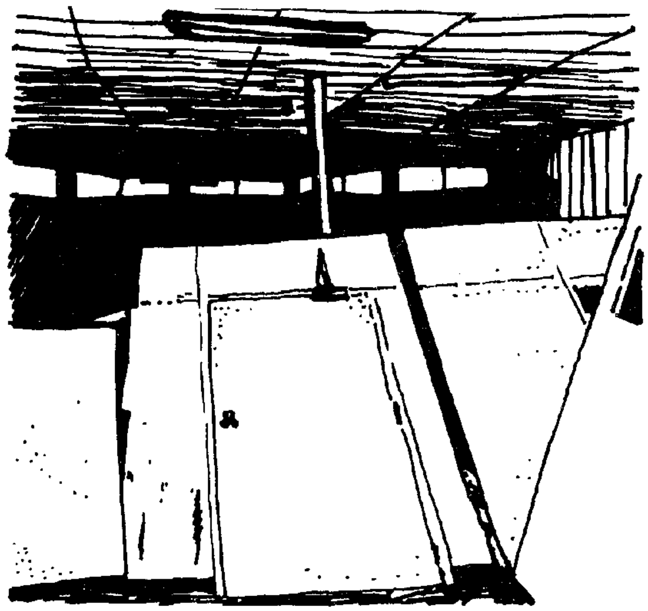
Partitions

Partition systems fabricated from many components that are mechanically fastened and are allowed to move under seismic conditions usually perform better than partitions of monolithic materials. The designer should not expect earthquake-imposed forces to run parallel to the partition. The actual movement may produce the combined effect of shear, bending, and crushing if the wall is restrained.

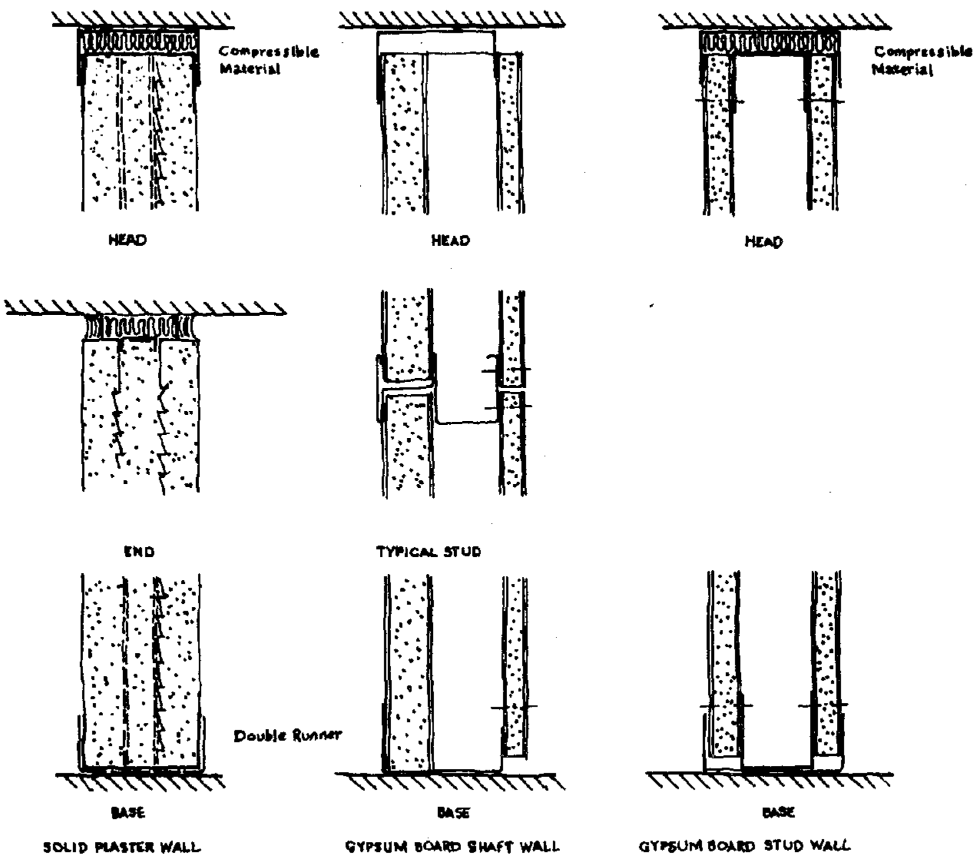
Partitions that do not extend to the structure should be braced or secured to the ceiling system and the ceiling properly engineered for the additional load, see Figure 14.6.

Full height partitions secured at the base and bottom side of the structure can be improved by providing a sliding shoe or double track with additional space at the top and sides to take care of anticipated deflection or racking of the structure, see Figure 14.7.

Partial height partitions should not be used at exit corridors unless designed and secured to resist seismic loads.



PARTITION FAILURE
TELEPHONE BUILDING - 1972 MANAGUA EARTHQUAKE
Figure 14.6



PERIMETER RELIEF TO NONBEARING PARTITIONS
Figure 14.7

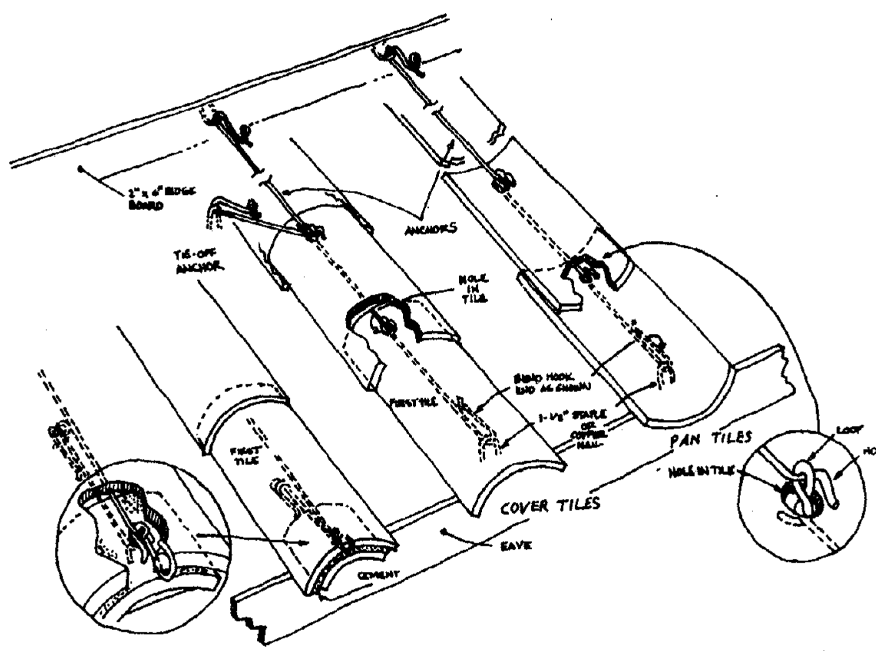
Canopy and Roof Systems

When properly secured, canopies usually perform satisfactorily. Sometimes consideration is not given to the fact that verticle loads can occur from all directions. An alternate approach would be to isolate the canopy from the structure.

As a general rule, roof coverings perform well when consideration is given to seismic movement between the roof and adjacent walls or parapet. However, heavy clay tile, cement tiles, or slate are often damaged because of the brittle nature of the material and because it is not usually restrained. Often the attachment is affected by corrosion or dry rot so that the attachment fails prematurely.

In recent years wire clips have been designed to provide additional support to roof tiles. If the designer feels these are visually objectionable, he may consider planting areas adjacent to roof overhangs and covered walkways at the entrances. These precautions will afford the occupant protection from falling broken tiles, see Figure 14.8.

Roof screens and ornamental parapets are susceptible to water damage. Therefore, protection from corrosion should also be considered when detailing their connections.



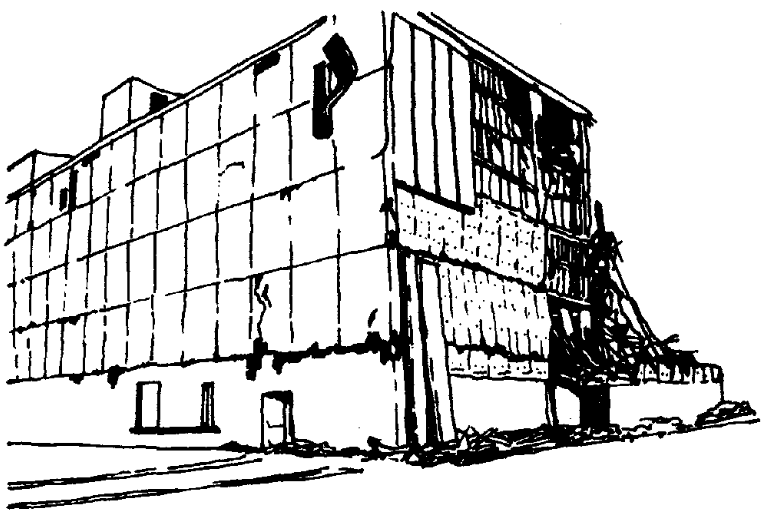
ROOF TILE CLIPS
Figure 14.8

Precast Concrete Panels

There are two paths for the designer to follow in designating supports for panels: design the support at a location on the structural frame where deflections in the structural frame will be minimized, or design the connections so they will sustain large deformation and rotations without failure.

While we have known these principles for years, building codes did not address this subject in detail until recently. The Uniform Building Code 1976 edition states that connections and panel joints shall allow for a relative movement between stories of not less than two times the story drift or 1/4 inch. Further, connections shall be sufficiently ductile and have rotation capacity to preclude fractures of the concrete or brittle failures in the welds. Also, inserts in concrete shall be attached to or hooked around the reinforcing steel.

Connection panels should be separated from the building structure to avoid contact under seismic movement. Internal stresses induced because of a statically indeterminate support should be checked; even in a statically determinate panel there may be built-in restraint at the panel. Figure 14.9 indicates in the precast panel connections and the structural frame. In this case, the interaction of the frame and nonstructural elements caused failures in both.



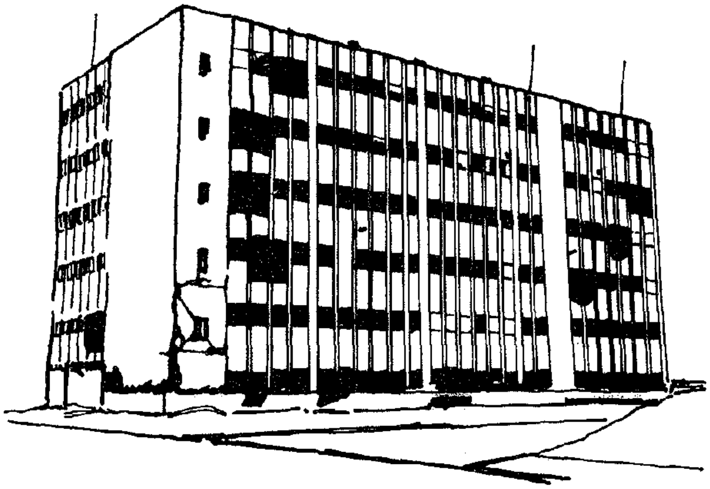
PRECAST CONCRETE PANELS
PENNEY BUILDING - 1964 ALASKA EARTHQUAKE
Figure 14.9

Curtain Walls

Curtain walls assemblies are ideal for light frame buildings where there will probably be seismic interaction between the structure and the exterior wall.

The principles of wall attachment are similar to those used in precast concrete panels; that is, design the support so that movement is minimized or design the connections so they sustain large deformations and rotations without failure. The principle cause for failure is the lack of design clearance in the frame or in the frame to glass clearance. Often, consideration is not given to tolerances for the entire assembly.

The two basic concepts–stick or unit frame. Construction is usually used in the design of a curtain wall. Sometimes a window wall assembly will use a combination of both types. Stick walls are usually assembled in the field. They consist of vertical and horizontal components with joints to take care of seismic movement as well as space between the structure and wall for interfloor displacement. The unit frame consists of individual window units which are either shop or field assembled. They are usually fixed at one point to the structure and are free to move. This type of window wall is best suited where story drift is anticipated. Often curtain wall assemblies contain rigid structural or nonstructural elements which may cause failures in one or the other or both elements, see Figure 14.10.



CURTAIN WALL FAILURE
CORDOVA BUILDING - 1964 ALASKA EARTHQUAKE
Figure 14.10

Glazing

Glass breakage is related to temper, fabrication, surface quality, support conditions, and the type of loading. Large windows usually break at somewhat lower stress levels than small windows. In small (or very thick) windows, deflections are small relative to the thickness. In large (or very thin) windows supported on all sides, glass behaves like a membrane or diaphragm. Glass edges are seldom intentionally fixed or damped; therefore, glass is allowed to move within the frame. With sufficient space for movement within the frame (and if the frame does not rack or glass loading is not increased by falling objects or blast effect), good performance can be expected. An example of glass failure is shown on Figure 14.11.

Heat-strengthened glass is about twice as strong as annealed glass. Also, its breaking pattern is granular. Where the designer desires to improve the performance of the wall, he may desire to use tempered, wire, or laminated glass.

Glass joint treatment is a factor in the overall performance of the curtain wall or window unit. If the edges are restrained, failure is likely. Sealants and gasket materials lose their resiliency with age and exposure. Therefore, the use of material that will remain resilient is important in the design of the wall. Structural glazing gaskets should not be used in locations where the gasket is subject to racking because of the glazing tolerance needed to seal the assembly.

Special attention should be given to glazing at atriums, especially where the atrium extends between two buildings or portions of buildings that may be subject to seismic motion. Skylights should not be installed in exit ways or where there is a possibility of falling objects penetrating the skylight.

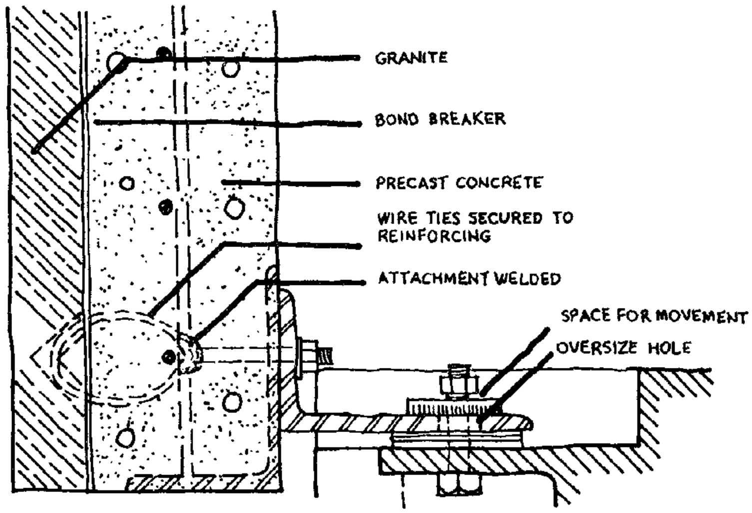
GLASS FAILURE
OFFICE BUILDING - 1974 LIMA, PERU EARTHQUAKE
Figure 14.11

Veneers

Model building codes require that veneers be either adhered or anchored to adequate backing. Adhered veneers are limited to 36 inches (914 mm) in the greatest dimension and 720 square inches (4645 cm) in total area. The weight can be no more than 15 pounds (6.80 kg). The adhesive must have a bond strength of 50 pounds per square inch (22.6 kg/6.45 cm). Anchored veneers have no limit in size. However, the anchorage must support two times the weight of the panel. An example of veneer failure due anchorage is shown in Figure 14.12.

VENEER FAILURE
FIRST PRESBYTERIAN CHURCH - 1971 SAN FERNANDO EARTHQUAKE
Figure 14.12

Past performance indicates that quite often not enough space is provided at the joints. Sometimes cement grout is used instead of a flexible material or used to space the stone from the backing. This grout may not bond to the stone or backing, thus, becoming loose and causing a wedging action to occur between the backing and stone. There are other methods of supporting veneers such as mechanical attachments or anchorage to precast elements, see Figure 14.13.



ATTACHMENT OF VENEERS
Figure 14.13

Ceilings

When combined with the seismic performance requirements, the model code's prescriptive standards for installing ceilings are difficult for the designer to understand. An important change occurred in the 1976 model code regarding ceiling systems. Prior to 1976, the code required the direction of the force be figured horizontally. The latest code requires the force to be figured in all directions.

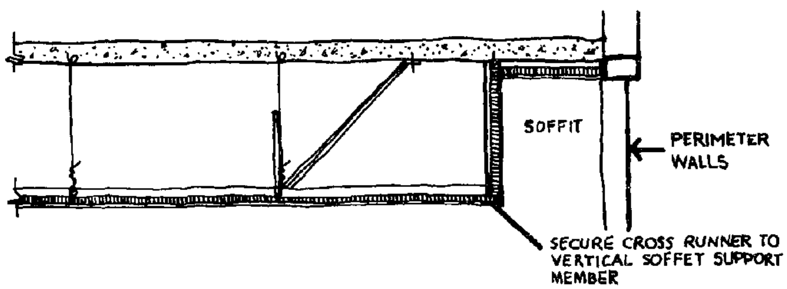
CEILING COLLAPSE
SUPREME COURT - 1972 MANAGUA EARTHQUAKE
Figure 14.14

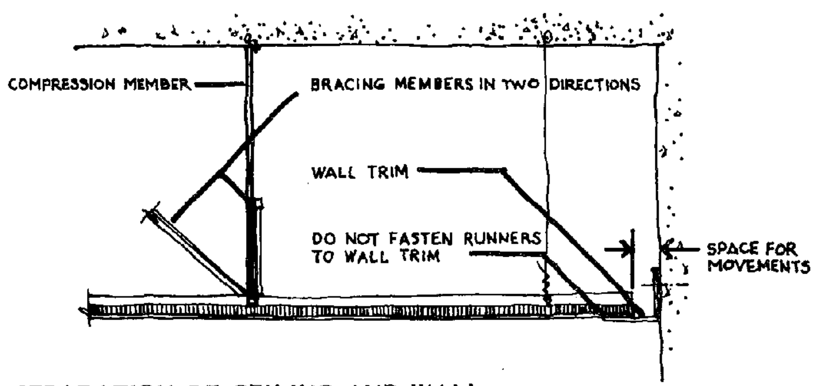
The two common basic types of suspended ceiling systems are acoustical tile and board, and gypsum plaster or gypsum board.

Acoustical tile and board are usually exposed tee or concealed spline. The exposed tee ceiling is easily damaged during an earthquake because the ceiling system usually lacks rigidity and the tile usually rests on the tee grid assembly. The concealed spline and tile ceiling usually sustains less damage because of the tiles tightly keyed together within the grid assembly.

The other type of ceiling is the gypsum board and lath and plaster. Lath and plaster ceilings weigh about five to ten times as much as acoustic tile. They are rigid and respond as a unit.

To avoid damage to both types, the ceiling should be isolated from the perimeter walls and braced to restrict lateral movement. When partitions are secured to metal ceiling runners, the ceilings should be designed to take the partition loads.





SEPARATION OF CEILING AND WALL
Figure 14.15

Seismic Joints

Seismic joints usually separate buildings or portions of buildings where seismic movement is anticipated, but one should not assume that movement will be in one direction. Joints are usually designed to function in a minor earthquake, but probably fail in a major earthquake without, however, causing a failure in the structure.

Fireproofing

It is important that the fireproofing integrity of the structure be retained during an earthquake. There were no large destructive fires after the Anchorage, Alaska, earthquake. And there were only a few fires after the San Fernando earthquake in 1971. However, San Francisco (1906) and Tokyo (1923) were partially destroyed as the result of the fire. In order that fireproofing be effective, 75% of the hourly rating should be maintained, that is four hours would become three hours, etc..

Contents

Building contents should be stabilized when possible. This can be done by bracing or anchoring the object. If this is not possible, then a barrier should be provided so that the means of egress is not blocked.

Vertical Transportation

During the Anchorage, Alaska, earthquake, several elevators sustained damage. In the San Fernando earthquake, extensive damage occurred to elevators in the Los Angeles area.

Common Failures were

- Derailment of the elevator cab and counterweights within the guide assembly, see Figure 14.17;
- Separation of the guide rail assembly from the structure;
- Entanglement of travel cables in the hoistway;
- Slapping of the counterweight cables against the elevator cab;
- Movement in the controller and motor generator sets; and
- Failure of door and frame assemblies, see Figure 14.16.

There are several design strategies that can be used to increase elevator performance during an earthquake. The elevator can be designed to the same seismic criteria as the building. Elevator performance can be improved by considering the following:

- Retainer plates at roller guides;
- Wire screens to protect traveling cables from contacting the elevator cab;
- Counterweight rail brackets; and
- Generators secured to the floor.

ELEVATOR DAMAGE - 1971 SAN FERNANDO EARTHQUAKE
Figure 14.16

WALL DAMAGE BY ELEVATOR COUNTERWEIGHT
BAUTISTA HOSPITAL - 1972 MANAGUA EARTHQUAKE
Figure 14.17

Mechanical and Electrical Systems

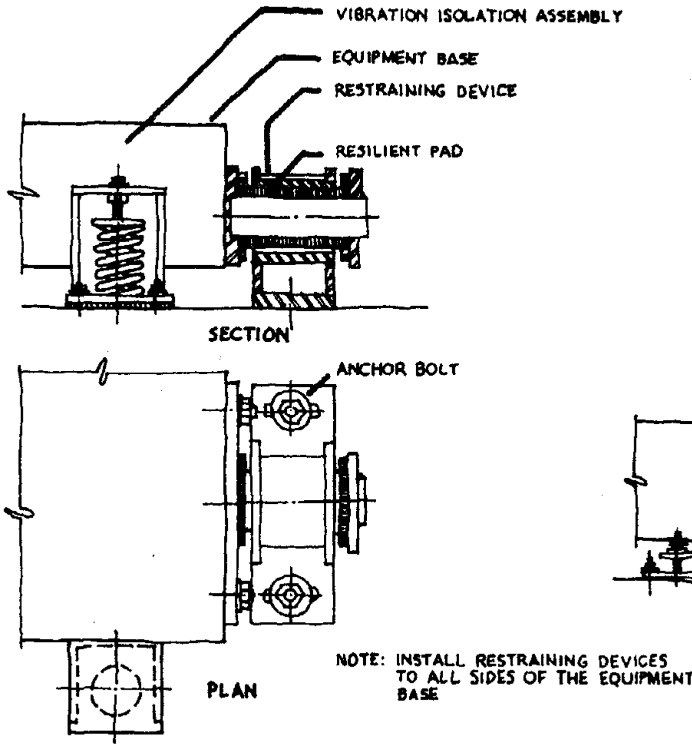
Mechanical, electrical, plumbing and fire protection systems sustained a considerable amount of damage in the 1964 Anchorage, Alaska, Earthquake and the 1976 San Fernando Earthquake. Past experience indicates that damage was extensive where there was excessive building movement, the equipment had a high center of gravity, and mountings were not adequate for dynamic loads.

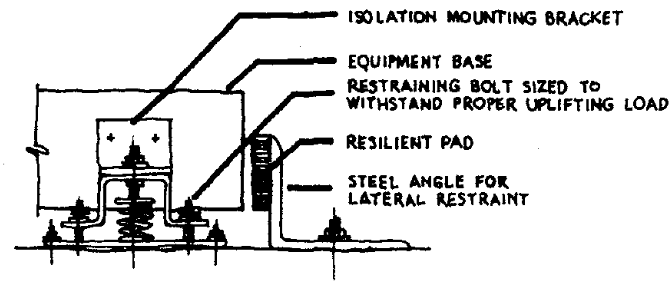
Earthquake damage in piping systems appeared to be minor in relation to damage to equipment mounting. A few pipes sheared, but most failures occured at the joints. There were numerous failures to light fixtures, ductwork, and pipes that were permitted to swing. Damage to sprinkler piping was minimal except for failures in riser standpipes. Electrical control centers and telephone equipment suffered extensive damage because they were not secured. Utility companies experienced failures in transformers and switching equipment which were not secured.

Mechanical and electrical equipment should be secured by fixed or direct attachment to the structure or by resilient earthquake mounts. Manufacturers offer numerous types of mounts that will limit seismic movement as well as function as a vibration isolater.

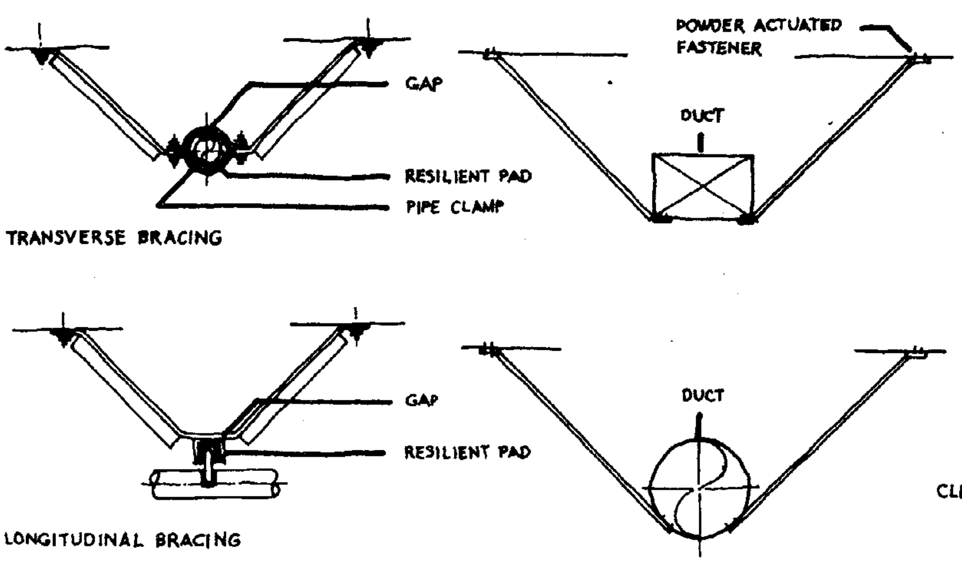
The 1976 Uniform Building Code contains many important revisions from the previous code. It recognizes the dynamic response of flexibly mounted equipment in relation to the dynamic response of the structure. The code also provides a coefficient factor for multi-story buildings where increased vertical acceleration occurs in upper stories.

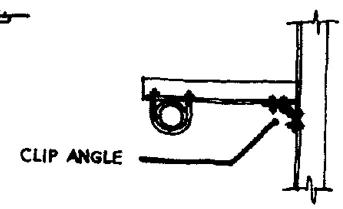
In addition, various industry groups have developed guidelines for supporting and bracing piping and ductwork. Figures 14.18 and 14.19 show methods of providing vibration restraints and braces.



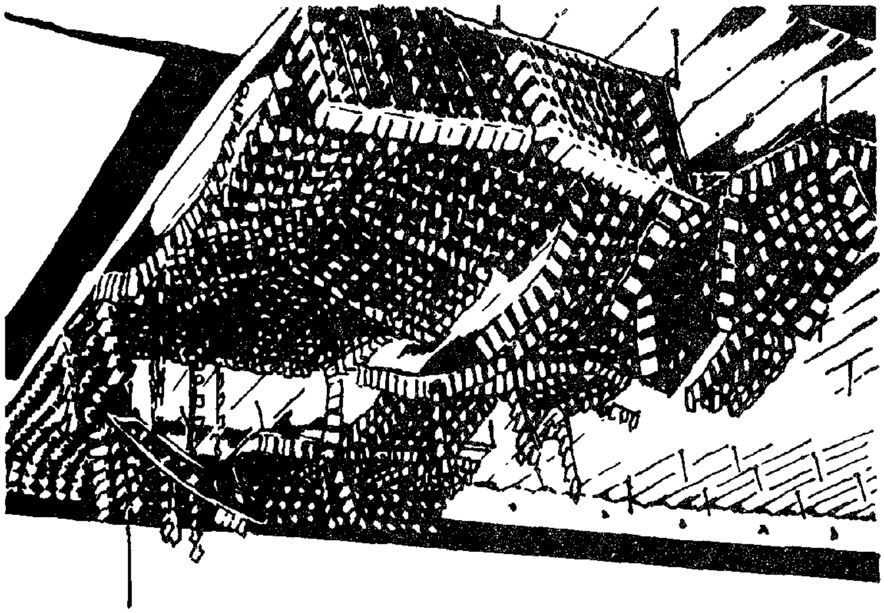


RESTRAINTS FOR VIBRATION ISOLATION MOUNTS
Figure 14.18





BRACING FOR PIPES AND DUCTS
Figure 14.19

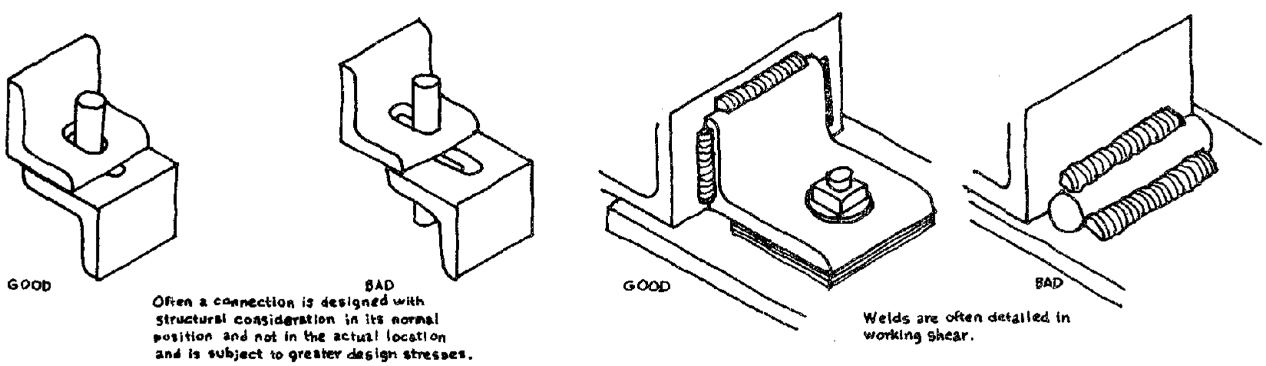


LIGHTING SYSTEM FAILURE
BANCO CENTRAL - 1973 NICARAGUA EARTHQUAKE
Figure 14.20

Connections and Fastenings

Stress is the primary cause of connection failures. Stress tends to concentrate or change direction and often exceeds the limits of the connection design. Several reasons for failure can be identified:

- Inadequate tolerances will often transmit loads to adjacent parts. Seismic tolerance is often confused with manufacturing or construction tolerance;
- Inadequate bearing may happen when an expansion anchor occurs in an oversize hole;
- Improper detailing of connections will frequently not allow proper movement. Welding heavy sections to light sections can be one problem, while welding galvanized metal may produce gas pockets which will reduce the strength of the weld. For examples, see Figure 14.21.



CONNECTION DETAILS
Figure 14.21

CONTRACT DOCUMENTS

The contract documents enable the architect to bring ideas into reality. The drawings are the graphic presentations of the design, and the specifications are the verbal description that complement the graphics.

Construction of equal quality is usually permitted. Shop drawings are prepared and equipment submittals are obtained based on the intent of the documents. When the contract documents are complete and accurate, their intent is more clearly discernible.

In administering the construction documents, the architect determines if the contractor's work conforms to the contract documents. Since many details of nonstructural systems are omitted from the documents because of "trade practice" or lack of knowledge, they are left to the contractor to provide.

Therefore, it is important that the specifications clearly indicate the seismic design concept and requirements for the systems. Contracts with the architect's consultants should also be clear in detailing their responsibilities.

There has been a growing tendency on the part of the building code enforcement agencies to require the architect of record or the design engineer to certify compliance with building code requirements. It is important that the owner, architect, or his consultants clearly understand their obligations. The alternative to the architect or engineer accepting this responsibility would be approval of the contractor's proposed design by a testing facility or agency.

CONCLUSION

From what we have seen in the past, and from what we know about the performance of buildings during earthquakes, several conclusions emerge relevant to proper seismic design:

- It is essential that the designer utilize the team approach for all aspects of the design;
- We do not profit from past earthquake experience. Experience gained from the Anchorage, Alaska, Earthquake (1964) had not been incorporated into buildings that went through the San Fernando Earthquake of 1971. Even today this experience is not being designed into new buildings;
- Building owners as well as many architects and engineers consider the building code minimum requirements as adequate protection against earthquake damage. After each earthquake, the public becomes interested and expresses its concern. But our memories are short; we forget the added costs of replacement or repair, or the loss of life, and, family;
- Past experience indicates that the design of nonstructural components must be based on both experience and theoretical knowledge. Thorough and competent inspection is necessary to ensure that the intent of the designer is executed.

Chapter 15. SEISMIC DESIGN FOR THE BUILDING AND NON-STRUCTURE

John F. Hartray, A.I.A.

SEISMIC DESIGN FOR THE BUILDING AND NON-STRUCTURE

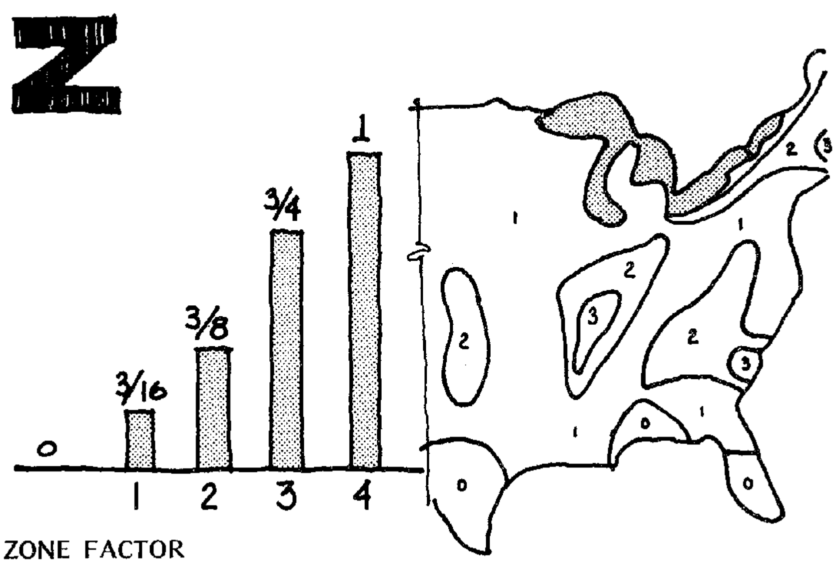
Code Definitions:

Section 2312(d) of the revised 1976 Uniform Building Code reads:

> Minimum Earthquake Forces for Structures. Except as provided in Section 2312 (g) and (i), every structure shall be designed and constructed to resist minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the structure in accordance with the following formula: V=ZIKCSW.

My eyes have always had a tendency to skip over formulas and I suspect that this ailment is shared with many other architects. The base shear formula, however, is a good place to start to develop a feeling for the way earthquakes effect buildings. When taken one factor at a time, the formula loses its mystery and emerges as a common sense description of the effects of shaking.

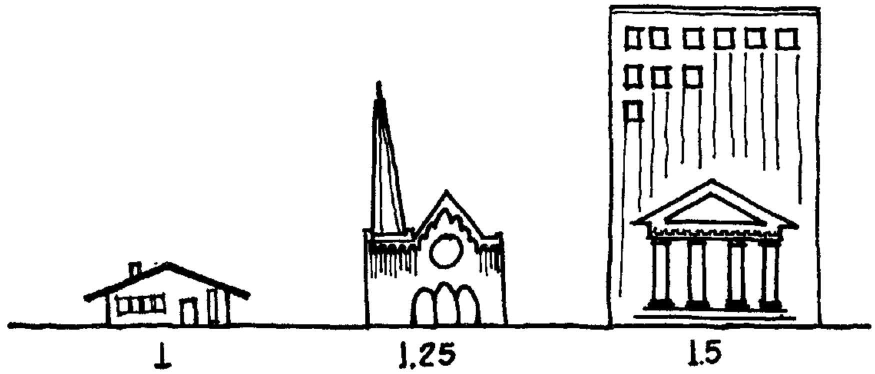
"V" of course is base shear or the sideways push of the earth at the bottom of the building.



ZONE FACTOR
Figure 15.1

"Z" merely refers to the value taken from the map of earthquake risk zones. A high value indicates a high probability of either frequent or severe earthquakes.

I

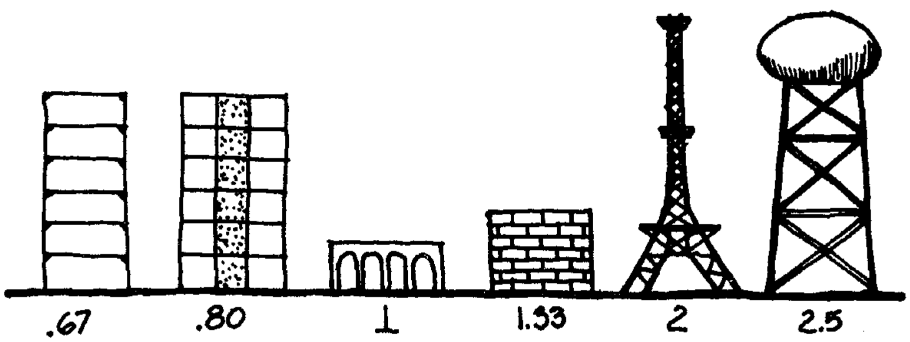


IMPORTANCE FACTOR
Figure 15.2

UBC TABLE 23-K

"I" has to do with the relative importance of a building in terms of public safety. High values are assigned to structures whose failure would cause a large loss of life or the interruption of critical public services. Police and fire stations, hospitals, and large assembly buildings are in this category.

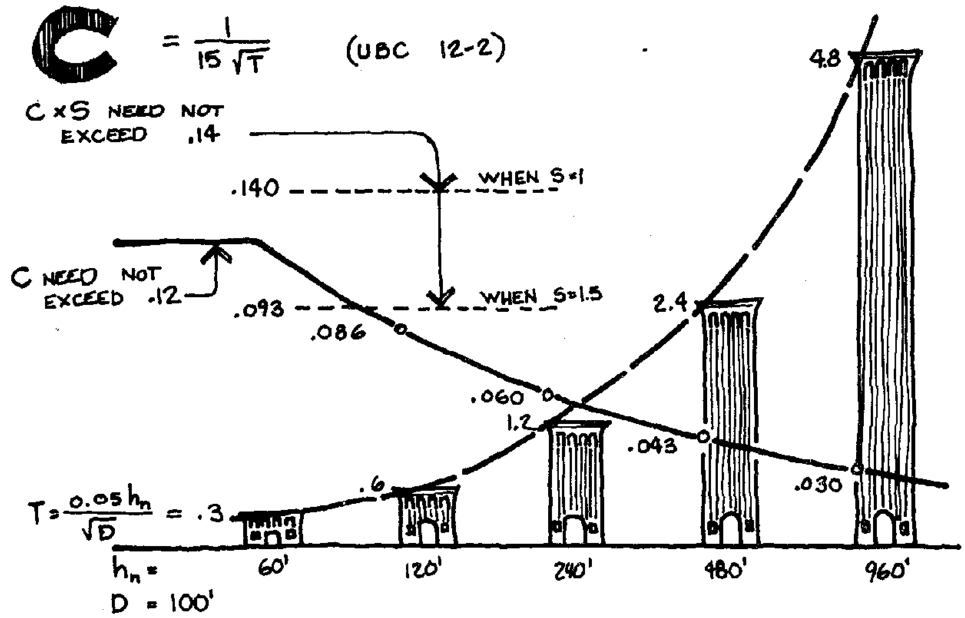
K



"K" FACTOR
Figure 15.3

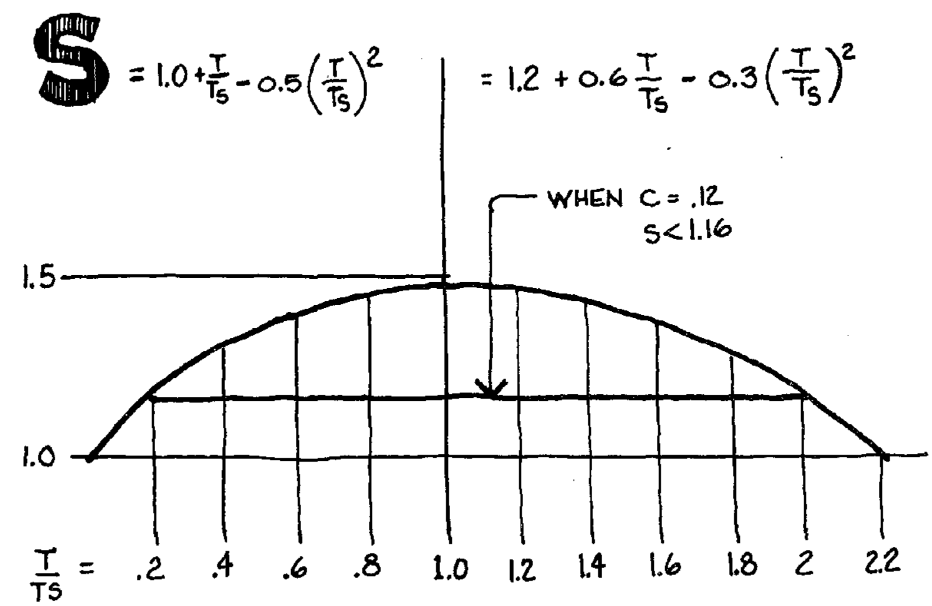
UBC TABLE 23-I

The "K" factor deals with the behavior of various types of structural systems when subjected to shaking. Lower values are generally assigned to more flexible systems which have greater ability to absorb kinetic energy.



"C" FACTOR
Figure 15.4

"C" is also concerned with the ability to absorb kinetic energy but relates to the building's overall proportions rather than to its method of construction. Tall, thin buildings with longer periods of vibration absorb energy more effectively and are assigned lower values of "C". When the ground moves horizontally a tall flexible building reacts like a palm tree or buggy whip in reducing the shear stresses at its base.

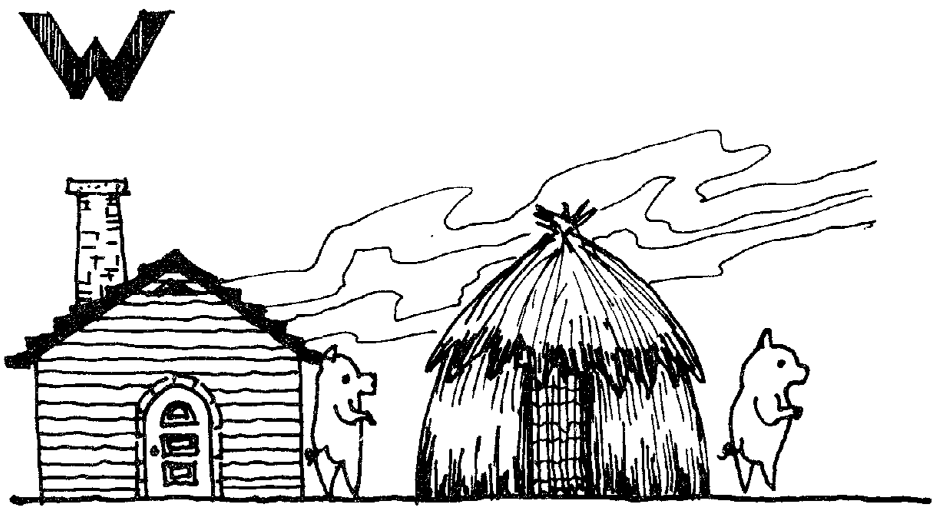


SOIL FACTOR
Figure 15.5

The "S" factor is a relative newcomer to the code. It takes into account the fact that buildings act as amplifiers for those earthquake motions that correspond to the buildings' natural period of vibration. Ground shaking that is in harmony with the building reinforces its natural movement so that each vibration adds to the motion caused by its predecessor. Rock vibrates at a relatively high frequency which will set up sympathetic vibrations in short, stiff buildings. Softer soils which vibrate at lower frequencies will create higher stresses in tall low frequency buildings.

Conversely short buildings on soft soil or tall buildings on rock may come through a severe quake with relatively little damage.

The formulas in the code which establish the value of "S" on the basis of the relationship between fundamental periods of buildings (T) and soils (Ts) are usually less intimidating to architects if they are expressed as graphs. In the age of the pocket calculator this is very easy to do.



WEIGHT
Figure 15.6

This leaves "W" which represents the weight of the building. And, here we confront a longstanding cultural prejudice. Westerners tend to equate mass with strength. Our monumental architecture, unlike that of the Orient, has usually been made of masonry. This tradition may well have grown out of our inability to manage our forests on a sustained yield basis but it is ingrained even in our children's literature.

The three little pigs and the president of the First National Bank both know that nothing beats mass for keeping the wolf away. Architects must understand that straw huts do better in earthquakes.

Once we have decoded the formula V=ZIKCSW we can gain a common sense feeling for the factors which contribute to earthquake damage. The upper limit of the base shear formula would occur in a highly active seismic zone like Anchorage, Alaska where a critically important building with a short stiff configuration was to be built on rock which had the same fundamental period as the building's structure.

Z = 1

I = 1.5

K = 1.33

C = .12 (its' maximum value)

S = 1.167 (because C x S need not exceed .14)

At its upper limit for buildings, therefore, "V" is equal to a little over a fourth of the weight of the building. A tower or tank structure would have a higher "K" value and might produce a base shear of over half its weight.

For architects who do not avoid the design of important buildings as a matter of aesthetic principal, the choice of the structural system (K) and the mass of the structure (W) offer the greatest possibilities for base shear reduction.

We can seldom convince our clients to move to a lower numbered zone and the decision to build a high rise structure is usually governed by land economics. Locational advantages also override subsoil problems. If this were not so there would be no Chicago.

A POLITICAL DIGRESSION

Before we proceed to non-structural hazards let us pause and return to the "K" factor as an illustration of the legal consequences of inadequate architectural education.

The base shear formula is not derived from nature like the laws of physics. It is, instead, a statement in mathematical notation of a civil law governing building design.

It was legislated by committees of experts to which society delegated the task. For the most part architects were not included on these committees. Earthquakes interested them less than art.

The engineers, who participated in writing the law, looked at buildings from the viewpoint of their own concerns and interests. They knew that the design of structures for seismic loads would be greatly simplified if ductile frames were encouraged. No architects were present to point out that high ductility and flexibility in the structure would compound the problems of detailing everything that was attached to it. And so, without protest or even notice, we allowed the engineers to pass the problem of ground motion on to the architects.

If we select a rigid structure the "K" factor requires us to assume higher loads. Shear walls must be designed to resist 100 percent of these loads while the remaining flexural connections must be designed to take an additional 25 percent. These rules add up to powerful economic encouragement for flexible, ductile frames.

As we shall see flexible structures do not guarantee a safe environment. There are trade offs to be made in detailing curtainwalls, exitways, interior finishes, and environmental systems. As these are evaluated we may find that stiffer structures are both safer and less expensive.

Hopefully, we are beginning to educate a generation of architects who will be equipped to contribute to the development of improved codes which will recognize these trade-offs.

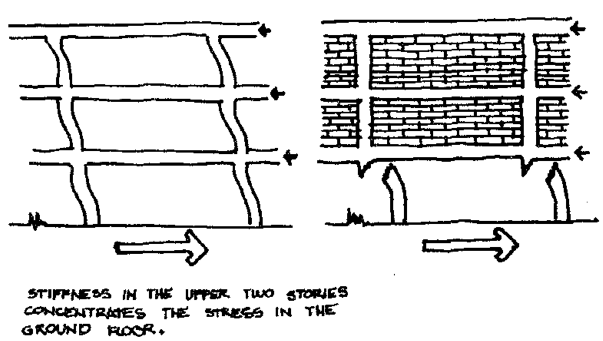
AN AESTHETIC DIGRESSION

It may be laziness but I have always admired the ability to eliminate problems more than the ability to solve them. This is particularly true of detailing problems which grow out of design concepts that defy nature.

I understand that we are now allowed to consider climate, geology, and even economics in designing buildings provided, of course, that we don't advertise our new practicality by putting north arrows on the drawings we submit to award juries.

Now that regionalism is socially acceptable we might want to exclude some of the more dangerous design cliches from earthquake country. The really bizarre stuff will fit in better in Miami anyway.

Let's begin with pilotis which may have come to us from the Doge's Palace by way of Corbu. Whether or not these are appropriate in seismic areas depends on the continuity of the frame and the weight and detailing of the walls above the ground floor. The Villa Savoye with its flexible steel frame and non-structural, light weight wall looks like a good bet. The Doge's Palace, on the other hand, looks pretty unstable. Frames with masonry infill on the upper floors which stiffen the upper part of the frame and concentrate drift at the ground floor seem to be casualties in almost every quake.



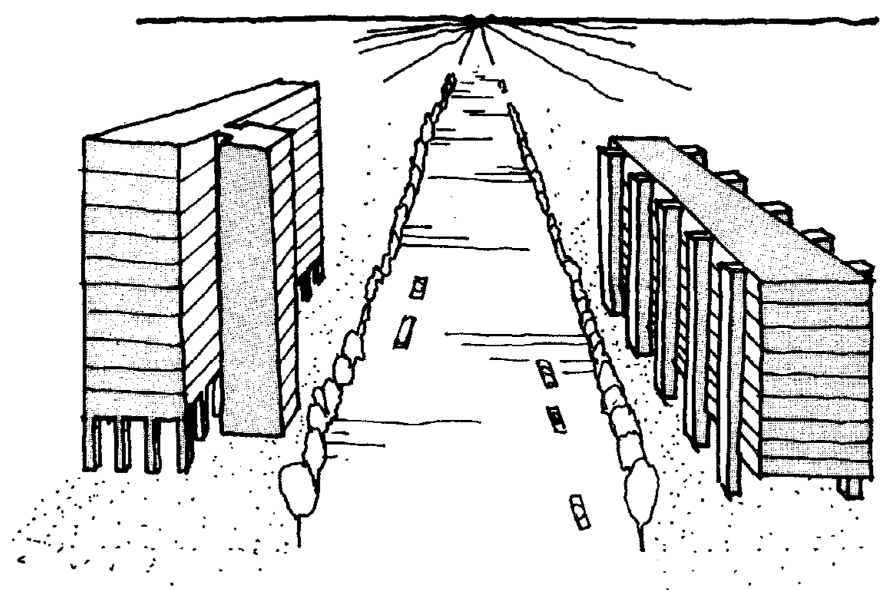
STIFFENED UPPER FLOORS
Figure 15.7

If the still masonry infill prevents a flexible frame from functioning as the structural engineers intended, glass infill provides an equally hazardous but different effect—it simply turns to shrapnel when the frame moves. For a while it seemed that tempered or heat strengthened glass was the answer. Experience has shown, however, that this often escapes from its frame unbroken. In this condition its presents a greater hazard than ordinary glass due to its greater strength.

Of course, there are solutions. A glazing system which allows the structural frame and the glass to move independently in separate planes can prevent excessive breakage. This kind of a glazing system, however, will probably not look like that of glass infill which we have become used to seeing.

During the fifties, when I was going to school, it became popular to pull the vertical service core out of the building and run it up the side as a separate element. This technique has saved a lot of boring facades but it has also created a monumental amount of torque from wind and seismic loading.

A similar method of creating shadows began with the Farnsworth House where the facia and floor spandrel beams passed inside the columns. What was an elegant detail on a one story building was soon carried over into high rise construction in such projects as the Inland Steel Building in Chicago and the Chase Manhattan in New York. At this larger scale the design idea prevents the spandrel beams from interacting with the columns to resist horizontal forces, in the longitudinal direction. This has not caused too much of a problem in terms of wind because the loads accumulated on the small ends of buildings are usually relatively light and the bents in the transverse direction can work effectively. Seismic loads, however, are equally strong in both directions. And so this formal design idea creates costly detailing problems when used in earthquake areas.

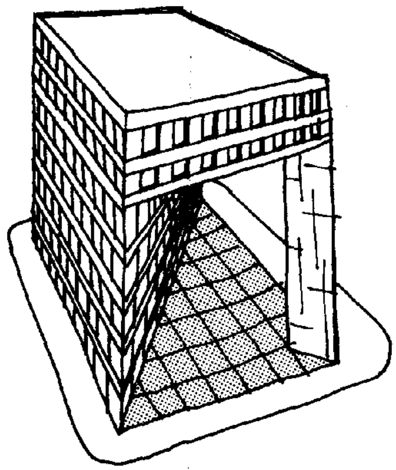


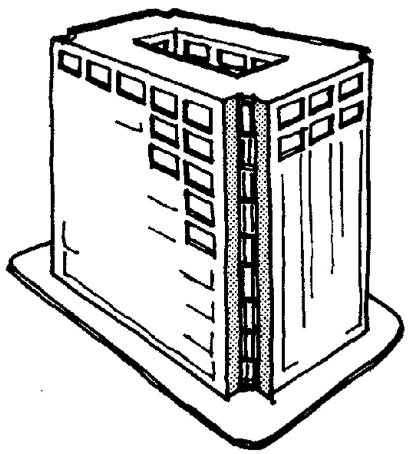
OFF-CENTER CORE SUPPRESSED SPANDRELS

Figure 15.8

Since we first learned of the design of the Ford Foundation we have been enthralled by asymetrical plans which provide gardens or covered outdoor plazas. As the center of gravity of these buildings is moved farther away from the center of resistance to horizontal forces, the structures become more subject to extreme rotational movements. These are difficult to predict and very expensive to overcome.

Another mischievous detail which is currently in vogue involves cutting away the corner of what would otherwise be a structural tube building to form a double column. This creates a great deal of flexibility right at the point where the shear stresses between the adjacent faces of the building are the highest. The detail is equally inappropriate for both wind and seismic loading and in tall buildings can create minor failures due simply to thermal movements.

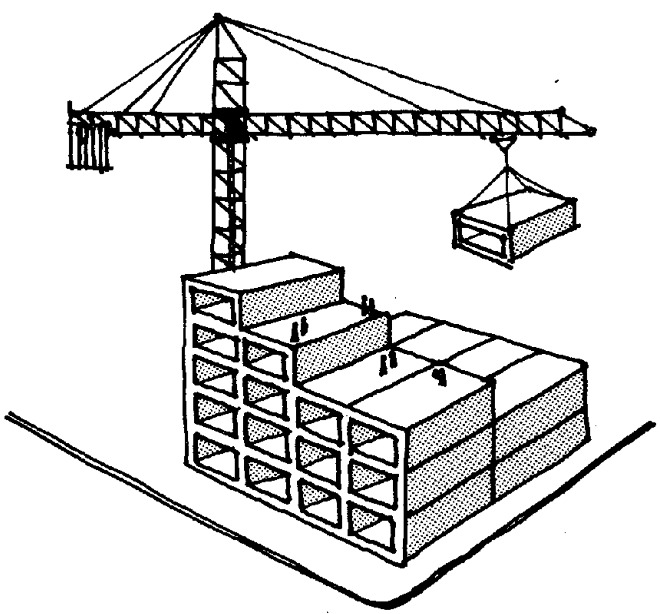




ASYMMETRICAL PLANS
Figure 15.9

OPEN CORNERS

We should probably also be wary of the machine esthetic as manifested in our continuing desire to construct buildings out of large prefabricated parts. It is very difficult to create structural continuity in these systems and their occasional collapse has been even more dramatic than the erection procedure.

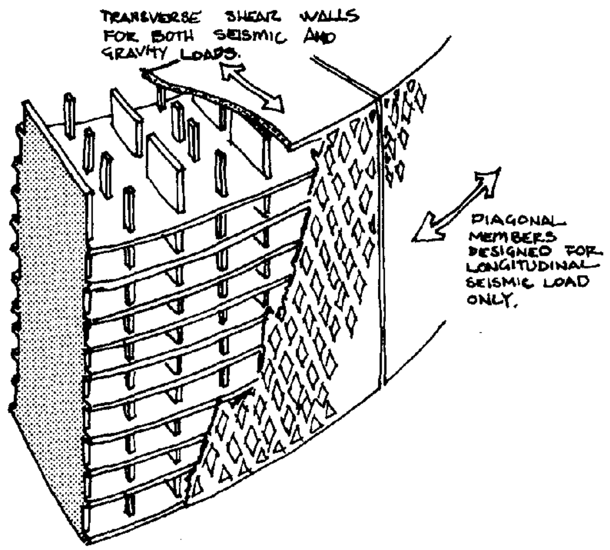


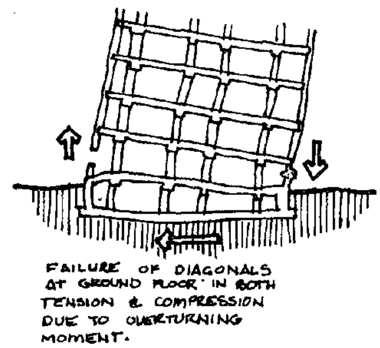
PREFABRICATION
Figure 15.10

The cardinal rules of seismic design are Symmetry, Continuity and Redundancy. The first two are obvious but the need for redundancy is sometimes more difficult to explain especially to students for whom theory is the only reality. An example from the 1976 Guatemala quake provides an interesting example of the need for redundancy.

A hotel was designed with shear walls to take the seismic loads in the transverse direction and a diagonal screen to resist longitudinal forces. Unfortunately, this screen was not strong enough to resist tensile and compressive forces which were introduced into it by the bending of the structure at the initial shock of the quake. Once failure had occurred at the ground floor, the screen became counter productive in that it concentrated longitudinal shear forces at the lower floor.

This building is also interesting in that although its structure survived the quake more or less intact, its interior was almost completely demolished even to the extent of elevator cabs being destroyed by their own counter-weights. So in addition to illustrating the need for structural redundancy it also points out the necessity for care in the design of nonstructural elements.





CAMINO REALE
Figure 15.11

Before leaving the subject of the esthetics it might be useful to think for a moment about the working relationship between architects and engineers. I have found in practice that many engineers are reticent to criticize architectural concepts. This may be due to a heartfelt belief in our genius, or perhaps to a contractual relationship under which the architect usually selects the engineer. There is probably also an element of professional pride in the engineer's ability to solve any problem we create for him.

The most effective relationship, however, between architect and engineer is one founded on candor, mutual respect, and empathy for each others problems. To foster this kind of collaboration in the area of structural design, architects must be aware at the schematic phase of the structural consequences of what they draw. Many of us, I'm sorry to say, don't even possess the vocabulary to discuss the issues.

THE FORMULA FOR NONSTRUCTURAL ELEMENTS

The UBC formula for horizontal seismic forces on nonstructural elements and building equipment is:

$$Fp = ZICpSWp$$

This is similar to and includes a number of the characters of the Base Shear Formula. Z, I, and F, are included to account for the zone, the building's importance, and the harmonic relationship between soil and structure. K, C, and W, factors which involve the structural behavior of the building, itself, have been replaced by Cp which represents the behavior under shaking of the various kinds of elements attached to the structure and Wp, their weight.

Let's work through a few examples which will illustrate the limits toward which this formula tends for various types of nonstructural elements and equipment. As in the example involving the base shear formula we will assume that the building is in Zone 4, that it is critically important to life safety, and that its harmonic period is equal to that of the soil beneath it.

We will begin by calculating the horizontal force for which a piece of essential equipment or machinery such as a fire pump or emergency elevator motor would be designed. Under this condition Fp would be equal to 75 percent of the weight of the equipment based on the following factors:

Z = 1
I = 1.5
Cp = 0.50 (Table 23-J-4d)
S = 1 (Table 23-J, footnote 5. SI need not exceed 1.5)

Forces on a precast concrete wall panel in the same building would be derived as follows:

Z = 1
Cp = 2.00 (Table 23-J-8)
S = 1 (Where Cp is more than 1.0 than value of I and S need not exceed 1.0-UBC 2312 g)
I = 1

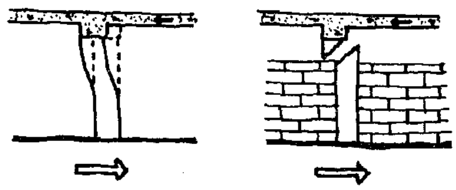
This gives us a value of two times the weight of the panel. Table 23-J also directs us to Section 2312 (j)3C which states the degree of freedom for differential movement between the frame and panel which we must allow for in detailing. This section of the code also specifies the technique of anchorage at these connections. The high value of the design loads on the panels is justified by their critical location in terms of life safety, and by the lack of redundancy in their attachment to the frame.

DESIGN GOALS

All seismic design techniques fall somewhere between the design earthquake approach, under which all structural and nonstructural elements are designed to resist a theoretical quake of a given magnitude, and the phased damage approach under which all elements are designed to resist the forces generated by frequent earthquakes, and the failure mechanism of the various elements is controlled to provide life safety under less frequent but more severe quakes.

Of the two approaches the phased damage technique seems to offer a greater degree of cost effectiveness and may in fact offer a higher level of life safety, particularly in situations where nonstructural elements might interfere with the proper functioning of the structural frame.

A classical case of this kind of interference is illustrated by masonry infill walls between columns with high windows or ventilation openings. Under seismic loading these nonstructural walls act to shorten the effective length of the columns and can convert a structure which was adequately designed to resist horizontal forces in bending into one which will fail in shear.



RESTRAINT OF GROUND FLOOR COLUMNS BY MASONRY WALLS CHANGED WHAT MIGHT HAVE BEEN AN ACCEPTABLE AMOUNT OF BENDING INTO A SHEAR FAILURE.

SHORTENED COLUMNS
Figure 15.12

WHAT IS VITAL

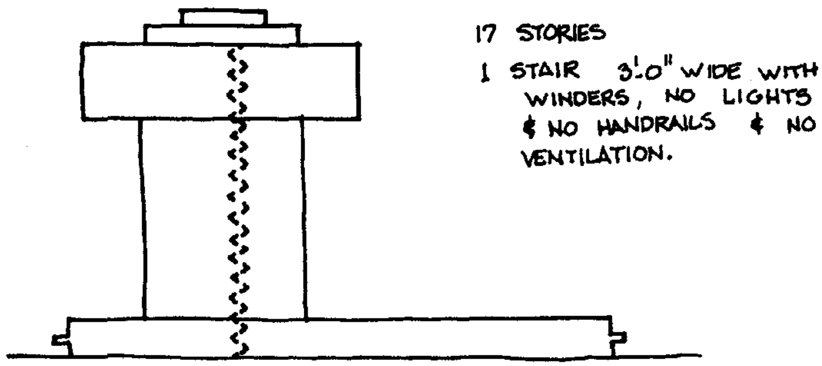
At present most building codes are quite definite about what must be done to protect vital building systems but vague about defining what these systems are. It seems to me that the design professions have an opportunity here to advance the state of the art by making the definition of vital systems a central issue in developing each building program. Our failure to do this will result in prescriptive codes, and rational seismic design will be replaced by semantic arguments between building officials and architects. To a large extent this is what has happened to those sections of building codes dealing with fire safety where a creative collaboration between building officials and designers is a rare exception.

In a typical building it would seem sensible to assume that any system supporting life safety would have to be kept operative following an earthquake. This is particularly true of fire safety systems because earthquake damage in a structure is so often followed by fire. Exit-ways for examples must be given special consideration in seismic zones.

Fire fighting systems must also receive the same kind of consideration. In high-rise buildings these will include those elevators which are required to operate on emergency power, sprinkler systems, and the ventilating systems required for the support of compartmented areas of refuge, and pressurized exit ways.

Overhead doors for garages housing emergency vehicles are the kind of building element that must not be overlooked in a seismic design analysis. The recently completed National Science Foundation book of seismic design for police and fire stations is an example of the kind of analysis of which the design professions, at their best, are capable of.

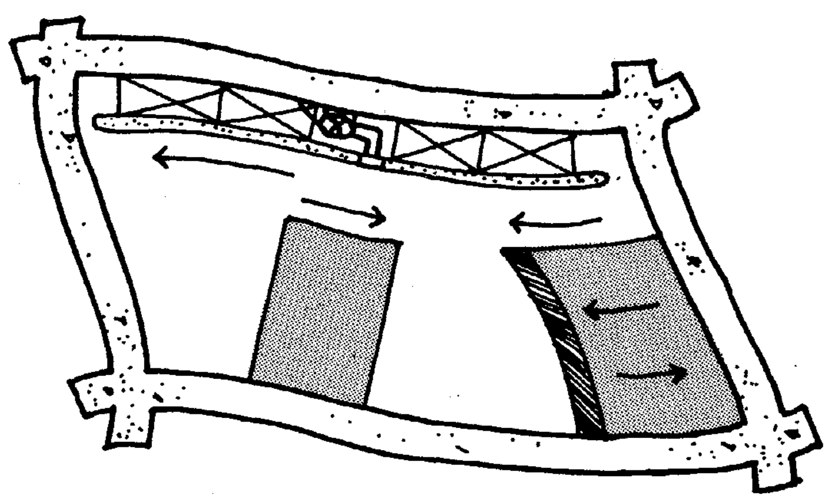
We shouldn't be smug, however. We are also capable of designing buildings such as the seventeen story Triangle Building in Guatamala City which had a single three-foot wide stair with winders, no lights, no hand rails, and no ventilation. During the earthquake residents in the upper portion of the building remained in their apartments rather than trying to escape on this stair. Fortunately, there was no fire.



LIFE SAFETY SYSTEMS
Figure 15.13

BUILDING SYSTEMS

There is virtually no part in a contemporary building that would not benefit from detailed seismic design analysis. In the following examples I will try to outline some of the issues involved for the more common elements.



DIFFERENTIAL MOVEMENT
Figure 15.14

Stairs

Almost every building has a stairway used either as the principle means of vertical circulation or as a back-up exit system. The stair by its very nature forms a diagonal brace which will behave like a shear wall but which may not have been included in the engineers analysis of the building frame. The location of such stairs is a critical issue for the architect and engineer. They should be located symmetrically within the structure to prevent torsional stresses.

If on the other hand it is possible to detail stairs in such a way that neither they nor their enclosure will resist shear and stiffen the structure, their location within the plan is not critical.

In detailing such a stair, we must first look at its connection to each floor and design details that will allow it to slip or to rotate so that the floors above and below may drift without restraint.

To maintain a fire separation around such a stair we need a soft joint that will allow slippage at the top of the partition. The new fiberglass insulations being marketed as fire stops might be investigated in such a joint. The stair enclosure must also be kept free from columns to prevent racking which would jam exit doors within their frames. Consideration of these criteria will create exit stairs that are quite a bit more elaborate than those we are used to seeing.

Wall Panels

Those of us who are used to designing curtainwall anchorages in buildings not subject to earthquakes have tended to feel comfortable with rigid anchors at the panel corners. Usually these connections have been made at points on the spandrel beams of the adjacent floors. We have sometimes found that such connections fail under thermal stresses and relatively light wind loading.

Recent developments in curtainwall design, however, have tended toward anchoring systems with only one or two rigid connections per panel. This kind of flexible anchorage was used in the Citicorp Building in New York to prevent usual distortion in its reflective metal skin. It points the way to detailing which would effectively eliminate shear stresses in wall panels under seismic loading.

Veneers

While architectural veneers are related to wall panels in terms of the kind of stress they receive under seismic loading, they are much more difficult to analyze because their anchorage systems have never been rationally designed and are usually impossible to inspect. There have been dramatic failures of conventional brick veneer with crimped sheet metal anchors which suggests that this technique is unsuitable in earthquake areas. We have also experienced failures of stone veneers under wind and earthquake loading where anchors have held the edges while the stone panels cracked and folded in the center. We are getting a better feeling for the kind of forces to which stone veneers are subjected and are approaching a time when these systems can be rationally designed.

In Chicago the terra cotta facades of many of our historic buildings are shearing away from steel frames. This is due to a combination of frame shrinkage and veneer growth from moisture absorbed over the years. A similar phenomenon is beginning to show in recent concrete frame buildings with brick skins. In these cases the masonry anchorage system may have failed before an earthquake occurs.

One of my partners recently brought back slides from Vienna showing buildings by Otto Wagner, a Viennese contemporary of Frank Lloyd Wright, who expressed masonry veneer by bolting through its surface. This is a decorative technique which seems to make a great deal of sense in seismic areas.

Partitions

The allowance for drift in partition systems is relatively easy to accommodate with caulked joints at the ceiling slab. Where fire or acoustic separations are required, however, we have little information on how well these soft joints work. This is an area where academic research would be quite valuable.

ARCHITECTURAL HISTORY

I am a part of a generation of architects which was not expected to learn architectural history. The theory being that an open mind would be more useful than a knowledge of past monuments in the building of a better society. In discarding the formal solutions of the past, we also threw away a great deal of technical information. I can now keep quite busy in our office enlightening recent college graduates about the techniques available for carrying away rain water.

As I belatedly learn more of history I am amazed at the technical virtuosity of our predecessors and I believe I see evidence of a consciousness of seismic design in the details and proportions of the historic architecture of seismic areas.

The styles of Pisa and Lucca for example allowed their builders to tie delicate arcaded facades back into the solid fabric of the building they covered through an extension of the impost blocks at the bottoms of the arches.

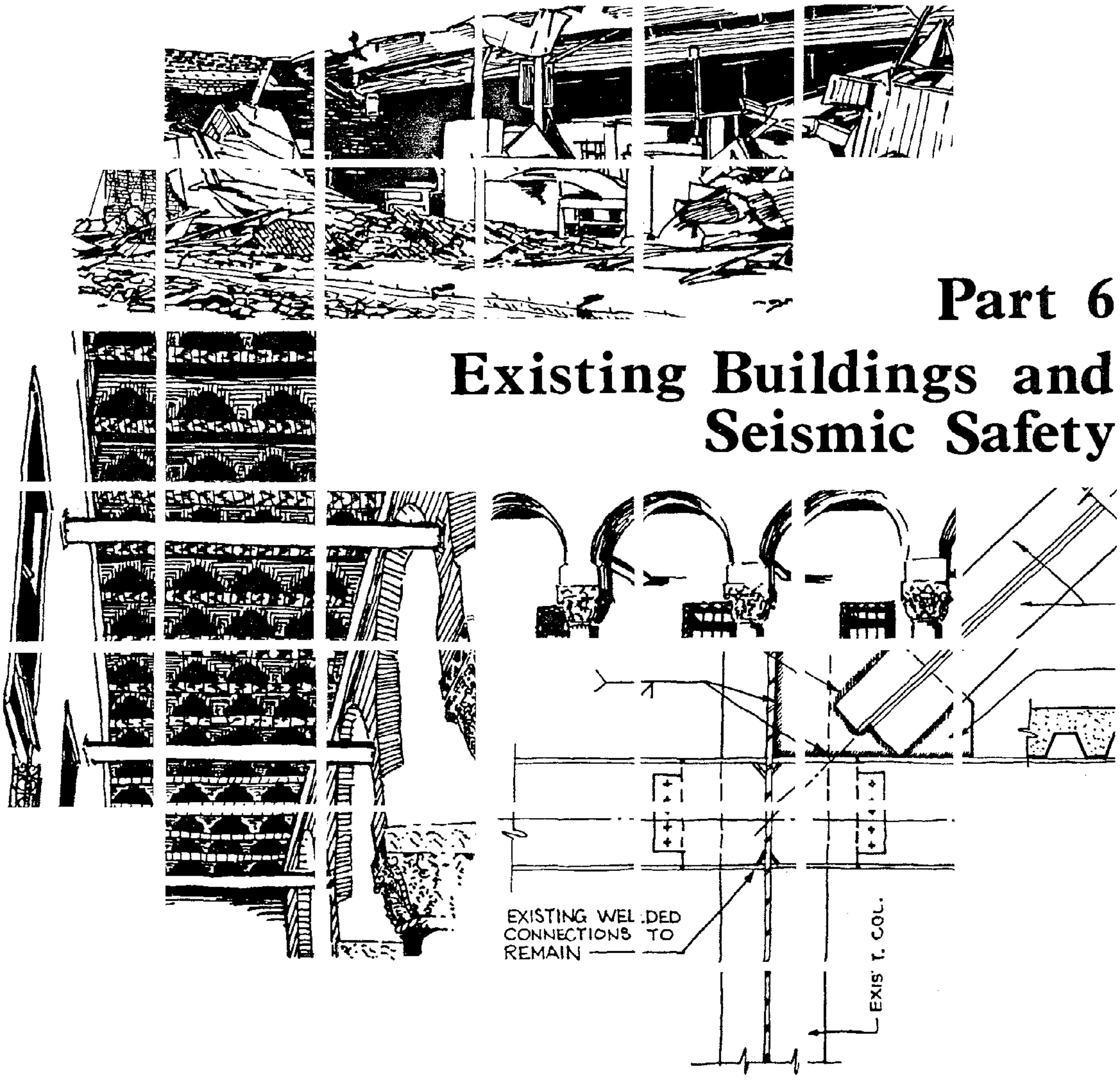
The arcades of Antigua are much heavier in proportion than those of their Florentine antecedants and generally terminate in shear walls. This appears to be evidence of at least a consciousness of the problems of earthquake design on the part of Renaissance architects.

There may also be an opportunity in the study of domed and vaulted structures which in spite of current theory seem to have stood up fairly well in highly active seismic areas over a long period of time. Buildings like Santa Sofia don't fit into the descriptions of construction types for which the code provides data for calculating the "K" factor and while we will probably never have another opportunity to build such buildings, I believe we could learn a great deal from an experimental understanding of their structural performance.

CONCLUSION

The least that can be expected of architectural education is that it provides an understanding in structural terms of the behavior of buildings under seismic loads. This, however, merely allows us to maintain the status quo. If the profession is to progress we must make students aware of seismic design as a form generator. Of course this does not mean it will be our sole concern. In addition to the functional and economic problems with which construction has always had to deal, we have recently added an awareness of accessability for the handicapped and a consciousness of the physical limits of our energy supply. The accommodation of these diverse criteria will result in serviceable buildings. Beyond this minimum level, if we work very hard, some of us may produce architecture.

Part 6
Existing Buildings and Seismic Safety



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Chapter 16. EXISTING HAZARDOUS BUILDINGS

Boris Bressler

EVALUATION OF SEISMIC HAZARD IN EXISTING BUILDINGS: TECHNICAL AND POLICY DECISIONS

In the following, I will address three aspects of the evaluation of the seismic hazard that existing buildings represent to life and safety: the assessment of the condition of buildings after an earthquake; the prediction of the behavior of buildings in the event of a major earthquake; and policy decisions based on such assessments of condition and probable response.

ASSESSMENT OF HAZARD FOLLOWING AN EARTHQUAKE

Communities located in seismically active regions must plan carefully for the social and economic disruption that certainly will follow an earthquake. Advance planning must consider fully the advantages afforded by existing facilities designed for use in an emergency. However, because the staffs and resources of such facilities are limited, communities must rely on the cooperation and expertise of volunteers from the engineering and design professions. A good emergency plan will thus include a means of mobilizing teams of professional engineers and architects trained in the process of evaluating buildings damaged in earthquakes. Such teams must be registered; members must know beforehand where they are to go if for some reason they cannot be contacted in the aftermath of an earthquake; and these teams must be provided equipment to facilitate their field work. It is desirable that the teams be so organized that those most qualified to inspect particular types of construction do so.

The first priority of inspection teams is field evaluation. The principal elements of buildings that serve to resist seismic forces must be quickly identified, their condition assessed, and a determination made of the ability of the structure to withstand possible aftershocks. Equally important to structural evaluation is an assessment of the condition of service facilities—gas, water, electricity, air. Professional teams must also evaluate the hazard, if any, represented by the condition of these and other utility connections.

Once the condition of a structure has been evaluated, a course of action must be decided on. Is the structure repairable? If so, what type of hazard abatement should be undertaken and how promptly can such repairs be made? Inspection teams should have the authority to declare buildings unsafe for occupancy, to prevent people from entering such buildings, and, if a structure is sufficiently damaged, to order demolition.

The decision whether to demolish or repair must be based on several considerations; a cost-benefit analysis conducted in the field may be essential. The judgment of an inspection team might, for instance, be that a less seriously damaged building could be repaired to a level of strength—say to 70 percent of original capacity—that would warrant occupancy, but that the original level of seismic resistance could not be restored. Is this level of strength, at an estimated cost, acceptable? In some cases, it may be possible to restore 100 percent of capacity, but at a substantial cost. In either case, it may be advisable to demolish the structure and reconstruct on the site. Inspection teams must thus be prepared not only to assess hazard, but also to determine a plan of action based on their assessments.

Repairs to earthquake-damaged structures fall into one of three categories: (1) cosmetic repair, in which cracks are covered with a coat of paint and such elements as moveable partitions or light fixtures are replaced; (2) repair of minor damage, in which cracks are properly grouted, epoxied, or covered with pneumatic concrete and steel is straightened, welded, or bolted; such repairs may restore a building to original strength; and (3) repair of major damage, in which either damaged members are replaced without any change in the overall structural system or the structural system is modified by the addition of structural walls or bracing members.

ASSESSMENT OF POTENTIAL EARTHQUAKE HAZARD

It is clear that a large number of buildings do not meet current seismic code requirements. As a matter of fact, a majority of existing buildings will never comply with current seismic requirements since code provisions are more or less continually revised as the understanding of and experience with earthquake response accumulate. Noncompliance with code requirements is thus inevitable.

While the noncompliance of many buildings does not necessarily mean that all such buildings are unsafe—many older buildings are well constructed and have withstood major earthquakes as well as any new construction—many are not as safe as similar new construction, designed in accordance with current code requirements, would be. The effect of the deterioration of older construction on seismic response must also be accounted for. Buildings may have undergone previous damage from earthquakes or other causes, but may not have been repaired to full equivalent strength. Buildings damaged in fires, for instance, may only have been repaired cosmetically. Such repairs may disguise serious reductions in strength and stiffness. Corrosion and other aging effects may be more easily observed, but the effects of such deterioration on the capacity of structures to resist seismic forces remains difficult to ascertain. Another major cause of reduction in the load-bearing capacity of structures may be alterations. As the utilization of space in a structure changes, walls, windows and doors may be removed or added, but these modifications are not always, or even frequently, planned to avoid altering the seismic response of the building's structure or, if necessary accompanied by structural modifications by which the strength and stiffness of the building are maintained.

The United States has been relatively fortunate with respect to loss of life in recent earthquakes. We should not, however, take a great deal of comfort from this fact since the most informed estimates project substantial losses in the event of a major earthquake. If a severe earthquake were to occur in the area of San Francisco, as many as 3,000 people might be killed and as many or more injured. This, of course, is a scenario that is impossible to document, but simply by looking around San Francisco, where there are many three-, four-, and five-story masonry buildings, it is not difficult to imagine that a severe earthquake would result in substantial loss of life and injury.

Although much earthquake damage in the United States is nonstructural, the cause of damage is usually the response of the primary structure. Damage occurs due to the interaction of the primary elements of a structure and secondary components. Evaluation of damage to nonstructural components is thus closely associated with a determination of the response of the primary building system to which such elements are connected. Only now are we beginning to realize the importance of secondary elements and to conduct studies the results of which will be used to enhance our understanding of structural response and its effect on the behavior of nonstructural elements and vice versa.

METHODS OF EVALUATING EARTHQUAKE HAZARD

We can begin to evaluate earthquake hazard by modeling the earthquake that we expect and the building whose response or condition we wish to assess. We must then determine the response of the model of the building to the model earthquake. Its response will depend on the demand that the earthquake places on the building (the forces, displacements, moments, accelerations induced in the building) and the capacity of individual elements and the building system as a whole to resist that demand. Although procedures for the evaluation of hazard vary, the basic process remains the same: (1) model the earthquake; (2) model the building; (3) determine response in terms of demand; (4) calculate capacity; and (5) compare demand to capacity.

A formal quantitative evaluation of a building can be performed at any one of several levels of sophistication. The simplest level may be that specified by the code. The code specifies a model of an earthquake and a simple method whereby the demand that that earthquake places on a building can be calculated. Because codes are written for the design of new buildings, however, older construction is implicitly neglected; the results of such evaluations are accordingly difficult to interpret for existing buildings.

Buildings can also be modeled in great detail. We now have relatively reliable methods of dynamic analysis by which we can analyze the dynamic behavior of a building and predict structural response in realistic terms. We cannot say precisely how a building will behave, of course, but out predictions are going to be much closer to the kind of structural behavior that will be observed than if we were simply to use procedures specified by the code.

There is inevitably uncertainty in such procedures. We can attempt to account for this uncertainty by the theory of probability, by risk analysis, and the use of probability distribution functions. We can also take the approach that deterministic methods are sufficiently accurate. Or we can eschew standards such as compliance/noncompliance and safe/unsafe and attempt to define the degree of vulnerability to damage, the damageability of a structure.

If we choose this latter course, we must consider the damageability of both structural and nonstructural components. Local, global, and cumulative damage must be assessed. Since local damage is not in itself ordinarily a hazard either to life or property, we will focus on global damage, damage over the building system as a whole. A meaningful estimate of damage must, however, include the effects of local damage; that is, such effects must be integrated into an assessment of global damage. This accomplished, we will have a good estimate of the response of a building in a major earthquake.

We must also consider the safety, functional, and socio-economic factors associated with particular types of damage. Safety is obvious. Functional considerations are related to the operation of elevators, communications systems, and other services within a structure. Socio-economic factors include the loss of community services from the closure of public buildings, such as schools, hospitals, police and fire stations, and the loss of employment through the closure of industrial facilities.

We must construct a global damageability index in order that these factors can be considered. A simple index might be developed as follows; other more sophisticated models have been developed. Let some number represent the local damageability of an element, say 30 percent. For every element in the building, damageability can be estimated. We must then determine the importance of this element with respect to safety, function, and socioeconomy. The importance factor of our hypothetical element might be 2. The importance of a girder collapsing may be only 1 or 1-1/2, while the importance of a light fixture falling from the ceiling may be 1/2. An importance factor can be associated with each element based on the consequences of the failure of that element, the effect of its failure on the overall damageability of the structure. An average overall global damageability for a structure, a damageability that considers the probable response and importance of each structural and nonstructural element in the structure, can thus be determined.

Several factors must be considered in order that the local damageability of an element can be determined:

- the force that an element can sustain without damage;
- the force that the element will resist before ultimate failure; and
- the point in time just prior to failure.

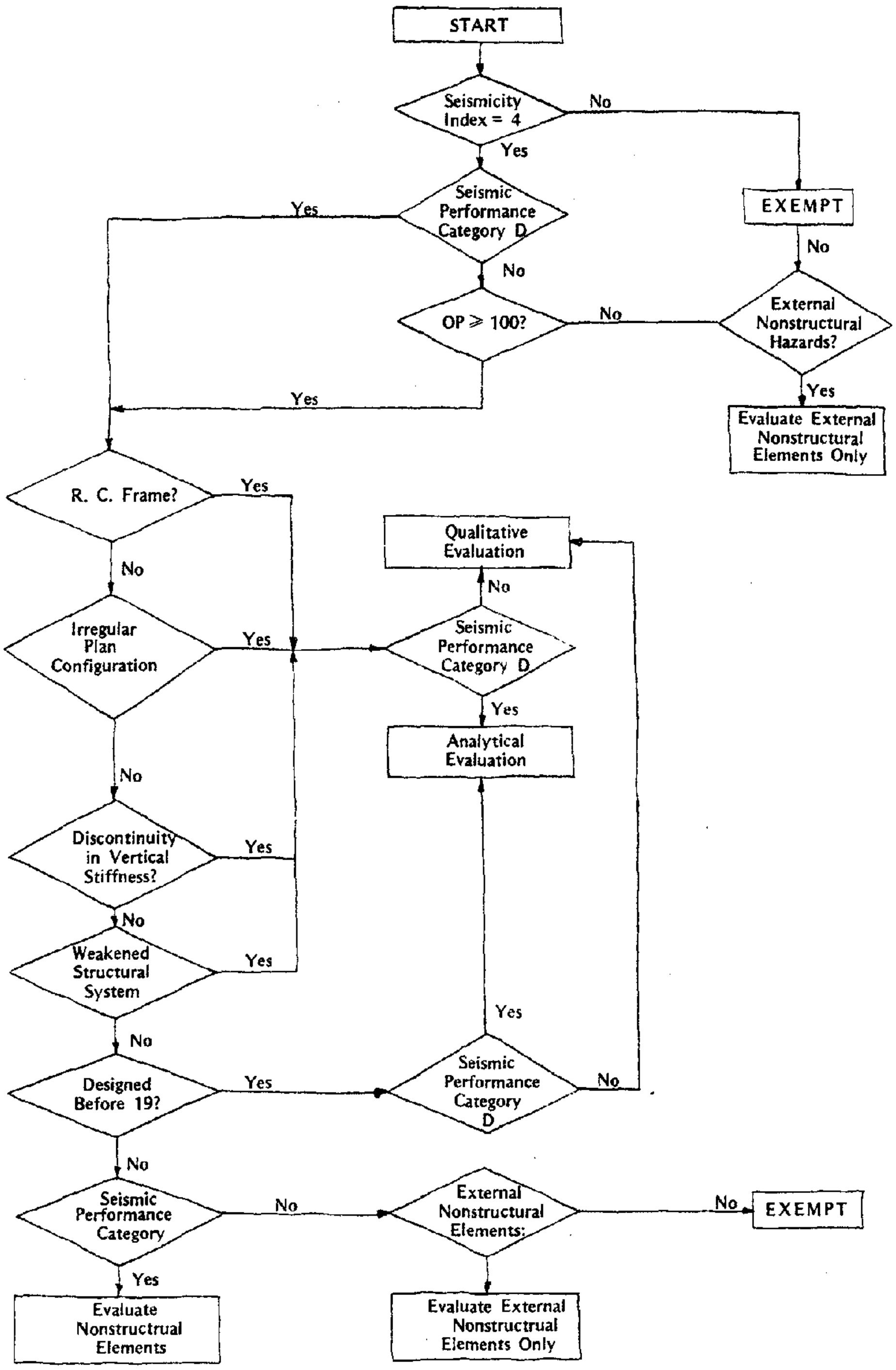
Code compliance as a measure of hazard is an effective measure for new construction, but codes cannot provide us much useful information about the potential for damage in existing buildings. Although most building officials and professionals will probably continue to rely on the code in order to safeguard themselves against legal action, I would like to suggest that an alternative method of evaluating the earthquake hazard represented by existing buildings could be developed under the equivalence section of the code.

POLICY DECISIONS

Many aspects of hazard abatement that we often consider technical concerns are in fact policy decisions. While we hear a great deal about hazard abatement, I believe that we should be discussing hazard reduction. Abatement suggests that the hazard will be done away with forever, a goal that I doubt can be realized. We can, however, reduce hazard to an acceptable level.

The decision to reduce hazard will be based on local and national economic priorities. While we may not find it desirable to invest large sums of money in hazard reduction, it is possible that such investment could result in long-term economic benefits. We cannot, however, be certain of the cost of hazard reduction until we have developed and accepted methods suitable for the evaluation of hazard and procedures by which decisions on acceptable levels of hazard can be predicated. We must also establish priorities. Let us assume that we have decided to evaluate all buildings in the United States. Even if all professionals were mobilized for the effort, we would not be able to do so within a limited period of time.

I have thus far addressed the problem of earthquake hazard alone, but it behooves the profession to consider an integrated approach to the reduction of hazard, all hazard. The present piecemeal approach, particularly with respect to code provisions on hazard, is in direct contrast to this concept. Again, the implementation of a plan for the reduction of overall hazard would require that a policy be formulated and a means whereby that policy could be translated into action found, either through codes or guidelines.



SELECTION OF BUILDINGS FOR EVALUATION
Figure 16.1

No requirement for hazard reduction will be enforceable without guidelines. People cannot do that which they have not been made to understand is necessary. What is essential are guidelines that serve as a type of code, but that are sufficiently broad to allow for general application. The ATC/NSF/NBS document, "Tentative Provisions for Development of Seismic Regulations for Buildings," especially the chapter on existing buildings, is an attempt to formulate such guidelines.

In that document, five intensities of ground motion were defined and the geographic area of the United States divided into zones, each of which was associated with one of these five intensities. Local effects within zones were considered. The demand on a building is thus determined not only by the zone in which the building lies, but also by the type of soil on which it was constructed.

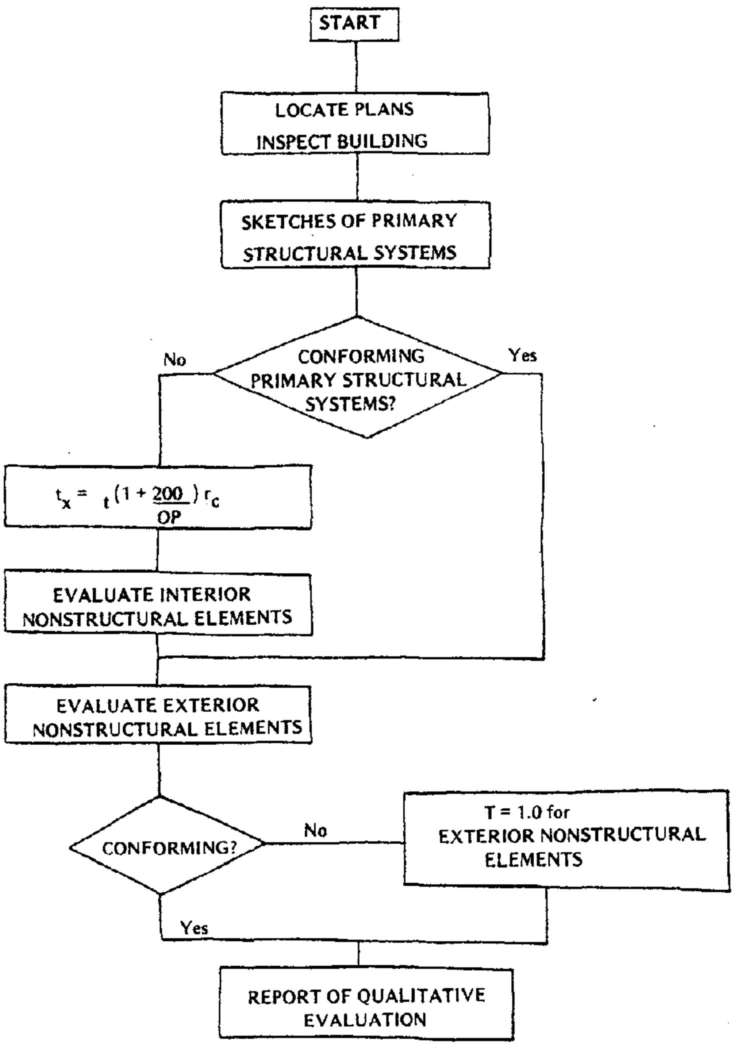
Buildings were categorized into one of three groups, which were established on the basis of the importance of certain types of buildings to the community. Hospitals, fire and police stations, and buildings in which other essential community services are housed form Group III; dwellings, theaters, and schools form Group II, the largest group; any building not classifiable in Groups II and III would be assigned to Group I. It was felt necessary to differentiate further, especially with respect to the large and varied number of buildings in Group II, and thus an index of importance based on the occupancy potential of buildings was developed. From tables in which the average number of square feet per occupant for types of buildings is listed, the potential number of occupants for a given building can be calculated.

SUGGESTED SQUARE FEET PER OCCUPANT PER FLOOR

USE	SQUARE FEET PER OCCUPANT
1. Aircraft Hangars (no repair)	500
2. Auction Rooms	7
3. Assembly Areas, Concentrated Use (without fixed seats)	7
Auditoriums	
Bowling Alleys (assembly areas)	
Churches and Chapels	
Dance Floors	
Lodge Rooms	
Reviewing Stands	
Stadiums	
4. Assembly Areas, Less- concentrated Use	15
Conference Rooms	
Dining Rooms	
Drinking Establishments	
Exhibit Rooms	
Gymnasiums	
Lounges	
Skating Rinks	
Stages	
5. Children's Homes and Homes for the Aged	80
6. Classrooms	20
7. Dormitories	50
8. Dwellings	300
9. Garage, Parking	200
10. Hospitals and Sanitariums-Nursing Homes	80
11. Hotels and Apartments	200
12. Kitchen-Commercial	200
13. Library Reading Room	50
14. Locker Room	50
15. Mechanical Equipment Room	300
16. Nurseries for Children (day care)	50
17. Offices	100
18. School Shops and Vocational Rooms	50
19. Stores- Retail Sales Rooms	
Basement	20
Ground Floor	30
Upper Floors	50
20. Warehouses	300
21. All Others	100

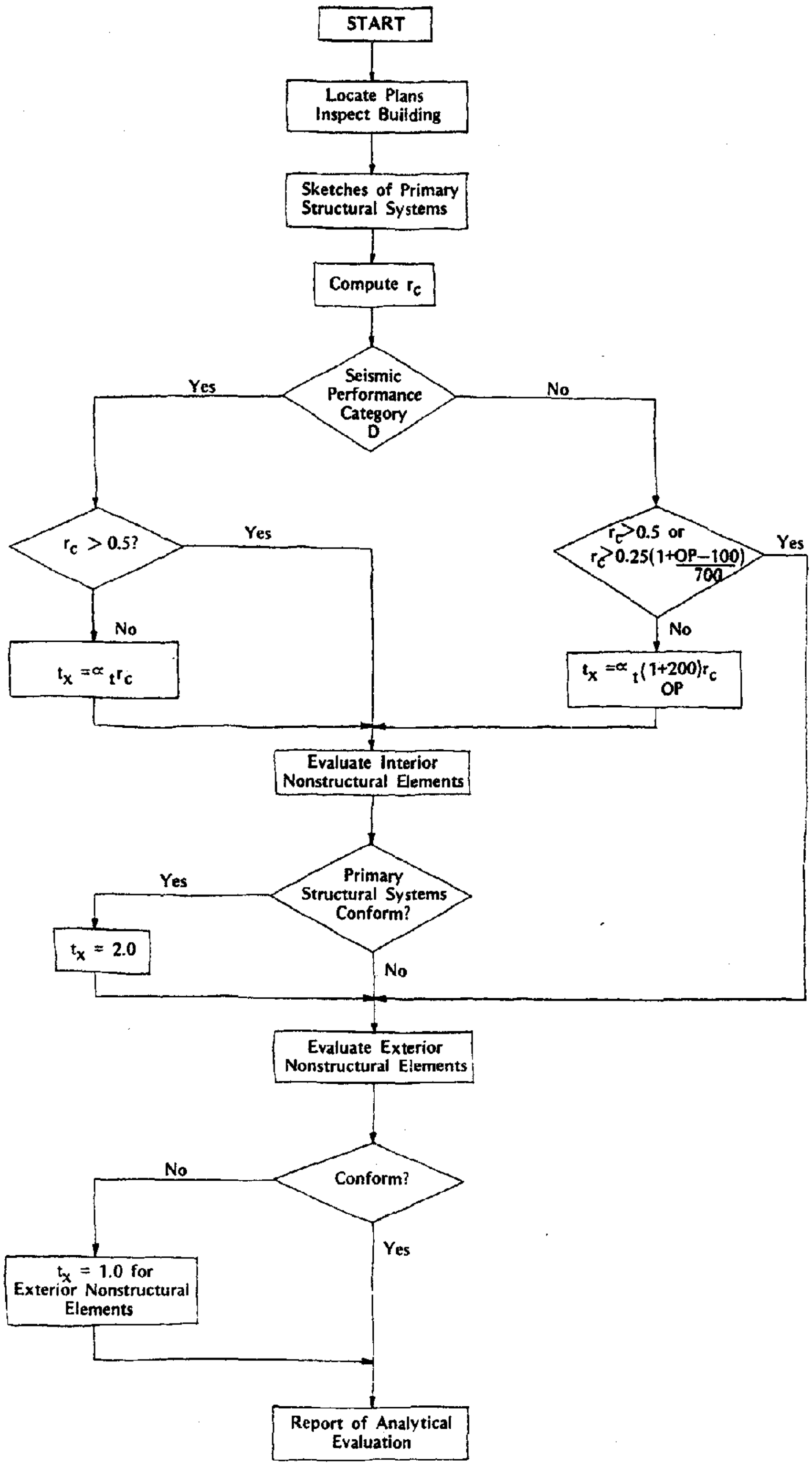
FLOOR SQUARE FOOTAGE PER OCCUPANT
Figure 16.2

For certain buildings a relatively simple, qualitative evaluation of hazard will suffice. These buildings, which are not categorized in Group III, can be evaluated in the field; the force-resisting elements should be identified and inspected. If there is no evidence of severe decay of the structural system, and if the structural system was originally sound, then a written report on the evaluation can be written and filed; no further action need be taken. If the design and construction documents—drawings and specifications—are available, they should be examined. If these documents are not available, specified measurements should be made and construction details noted during the field evaluation. Samples of material might be tested if from a visual inspection this was deemed necessary. The decision as to the adequacy or inadequacy of a building to resist earthquake forces would be based on the above observations, measurements, and, where necessary, material testing.



QUALITATIVE EVALUATION
Figure 16.3

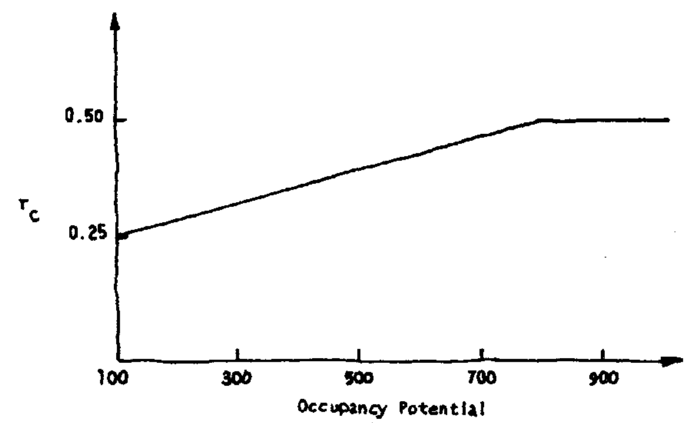
On the other hand, if the structure of a building is complex or for some other reason an inspector finds it difficult to assess the adequacy of a building, the condition of a structure may be termed uncertain and an analytical evaluation must be performed. The response capacity, rather than the damageability, of the building is assessed (which entails computing the base shear capacity for the building based on code provisions); response demand, i.e., the force that would be required for this building to resist the calculated demand were it designed as new construction under current code provisions, is accordingly calculated.



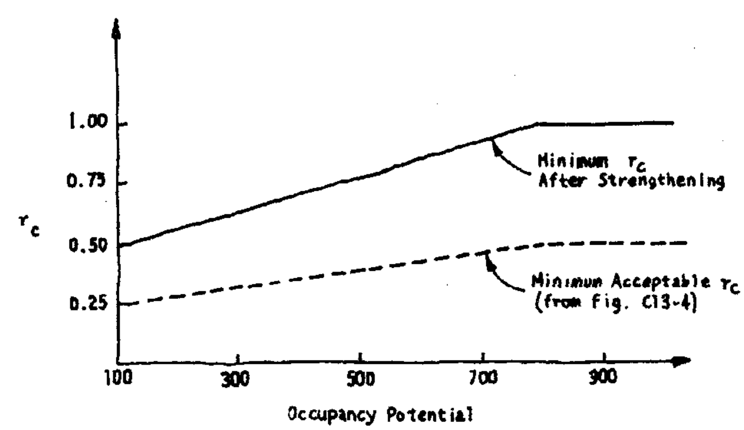
ANALYTICAL EVALUATION
Figure 16.4

As I have stated before, older construction is often not going to comply with current code requirements. The ATC/NSF/NBS provisions therefore allow buildings in Group III—essential buildings designed over twenty years ago—to remain as they are if their capacity is at least half that required by the present code. Buildings with lesser capacity must be strengthened to present code requirements. This would be primarily a policy, not a technical decision.

The provisions for buildings in Groups I and II are somewhat less severe. Buildings with low occupancy potential, on the order of 100 which is approximately the occupancy potential of a small apartment house, and whose capacity is at least 25 percent of the present requirement, need not be strengthened. Any building in Groups I and II with less than 25 percent capacity must be strengthened to at least 50 percent of code-mandated capacity. While clearly not ideal, such strengthening would represent a substantial reduction in hazard; for buildings with limited occupancy, rehabilitation to full strength may simply be economically unfeasible. Buildings with an occupancy potential of 800 or greater must be strengthened only if their capacity is below 50 percent, in which case they will have to be brought fully into compliance with current code requirements.

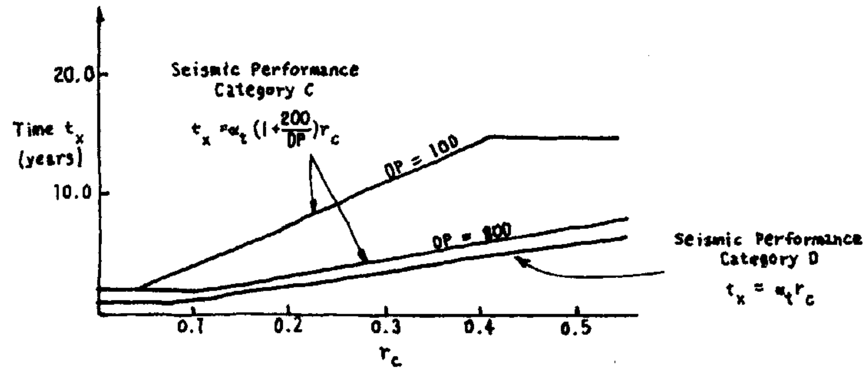


MINIMUM EARTHQUAKE CAPACITY RATIOS (CATAGORY C BUILDINGS)
Figure 16.5



STRENGTHENING REQUIREMENTS (CATEGORY C BUILDINGS)
Figure 16.6

Clearly, policy as well as technical considerations have gone into formulating the above. I have, for instance, described a method whereby buildings are selected out for strengthening, a method that involves both a technical assessment of capacity and a policy decision on what level of capacity is acceptable. Another issue is that of what period of time owners and communities will be allowed to complete required strengthening; this will be a policy decision. In the ATC/NSF/NBS document, it is recommended that mandated strengthening of buildings in Group III, essential buildings, whose capacity ratios are low must be completed within one year. If the capacity ratio of the building increases, additional time, not to exceed six years altogether, can be granted.



TIME TO STRENGTHEN OR DEMOLISH BUILDING ($T_X = 12$)
Figure 16.7

Time limits on the rehabilitation of buildings in Group I and II were based on the number of occupants in a building. If the number is small, say approximately 200, and the hazard is severe, owners would be allowed two years to complete strengthening. For a lesser number of occupants and a lower degree of hazard, a longer period of time, up to twelve years, would be allowed. As the occupancy of buildings in Group II approaches 800, close to that of buildings categorized in Group III, the limit decreases to no more than seven years.

Historical buildings are a special problem. Buildings that are designated as important to the history, architectural heritage, or culture of an area frequently do not comply with code requirements. In ATC-3, it was proposed that such buildings be evaluated individually, that no code provision be written. In San Diego, however, a special code provision has been created for historical buildings. The city now requires that the capacity of historical buildings can be calculated according to the 1949 code, and that value compared to the demand calculated according to the 1975 code. If the capacity of the building is 25 percent of calculated demand, the building need not be strengthened. If the capacity is 25 percent or less, however, the building must be strengthened to approximately two-thirds of the current requirement. While this may be as good a policy decision as that made in the ATC provisions, it is essential to remember that the capacity specified in the 1949 code is not well related to the method of calculating demand in the 1975 code.

Many particularly hazardous buildings are three- or four-story unreinforced masonry apartment buildings. The rent for apartments in these buildings is typically quite low; thus, when provision is made to rehabilitate or demolish those that are unsafe, those least able to relocate, the poor and the elderly, must find other housing. This is a socio-economic problem that must be heard and addressed.

Another policy decision that must be made is the allocation of funds for hazard reduction versus funds for disaster relief. For example, when a disaster occurs, relief funds become available. If funding agencies were willing to invest a fraction of that amount of money every year to hazard reduction, the need for funds after a disaster might well be substantially reduced. Also, apart from this source of funding, we must decide what portion of funds for hazard reduction should come from the public and what portion from the private sector of the economy. I believe that it will be difficult to mandate hazard reduction funds primarily from public coffers. On the other hand, I think that a great deal can be done through private financing provided that some economic incentive, such as a tax benefit or write-off, is offered.

If we consider the country as a whole, we will find that earthquakes are not everywhere the major hazard. In many areas it is fires, in others flood and storms that constitute far greater hazards. What is essential is that the critical hazard or hazards for different types of construction in different locations can be identified so that priorities based on the overall severity of hazard can be established and the level of hazard reduced. The alternative to such an integrated approach to hazard will, in the end, mean greater expense to all.

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Chapter 17. SEISMIC RENOVATION/RETROFIT TECHNIQUES

Daniel Shapiro

FACTORS INFLUENCING SEISMIC RENOVATION OR RETROFIT

Let us begin a discussion of seismic renovation and retrofit by distinguishing between the two terms. As I use it, retrofit means to bring up to higher standards of performance an already existing seismic-resisting system. In other words, if a building has been designed to a lower set of standards than we think appropriate for seismic design today, then we might go into that building and provide some details that will help the building be more seismic resistant. Renovation, on the other hand, implies that we are looking at an older building that perhaps was not designed for any seismic resistance at all. To extend its life, our efforts will be aimed at introducing into it an entirely new seismic system.

The factors that influence the selection of the seismic system for renovation or retrofit obviously overlap. But for our purposes we can consider them separately so that we can identify some of the important aspects of each.

One reason for a seismic renovation or a retrofit program would be to extend the life of a building. Sometimes this is in response to a request by the owner. The owner may be making a large investment in the building and therefore think it appropriate to add to its seismic resistance. So without any agency telling him what he has to do, he may decide on his own to bring the building up to higher standards.

Another reason for bringing a building up to higher standards might be in response to a new law passed by a regulatory agency. In California, for example, the state legislature mandated that all public schools built before a certain date had to be brought into conformance with modern seismic resistance codes. Each school had to be studied and brought into conformance with the standards.

Still another reason might be to conform with code requirements. Perhaps an occupancy change is being made. In that instance, certain jurisdictions require that an existing non-conforming building must be brought into conformance with modern seismic standards.

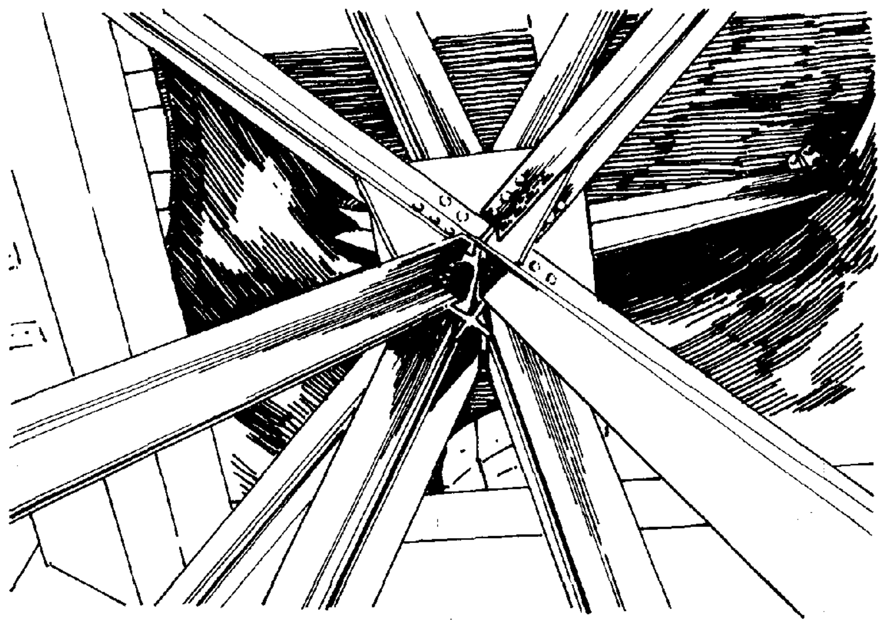
Another reason for seismic upgrading would be the relocation of old buildings in a redevelopment area to new sites. These buildings are generally of some historic significance. It is desirable, therefore, to preserve them, but the redevelopment plan may not provide suitable requirements for this type of building. So buildings may actually be picked up and moved to new sites. Now, it may be that the city where these buildings are located requires that buildings that are moved from one site to another be brought into conformance with the seismic standards of that area.

One example of a voluntary seismic renovation was an old church building that was being retained as part of an expansion program for a new facility being built alongside it. The building was of some historic interest; thus, the people using the church were very interested in preserving it as part of the heritage of that particular site. For that reason, it was necessary to do some work on the church to bring it up to more current standards. It was decided voluntarily to put in a seismic-resisting system, although there was no law in the city requiring it.

Another example of a seismic renovation to meet prevailing code requirements was an old waterfront warehouse in San Francisco that dated from before the earthquake of 1906. It was probably badly damaged in the earthquake and reconstructed. Now a change of occupancy was being made; the building was being turned into an office structure. As a result, because of the laws of the city, it was necessary to design in a seismic-resisting system where one did not previously exist.

The level of safety required for the seismic rehabilitation or retrofit of a building may be a requirement of the building owner or may be imposed by the building code. This has special implications for public or historic buildings in which access to hazardous areas can be controlled.

When an owner faces minimal legislated restraints he may choose a scheme which adds merely enough interior bracing to meet the minimum code requirements. The building owner may not ask that the building be designed to extremely high standards, so the designer usually opts for the absolute minimum required by code to brace the building seismically.



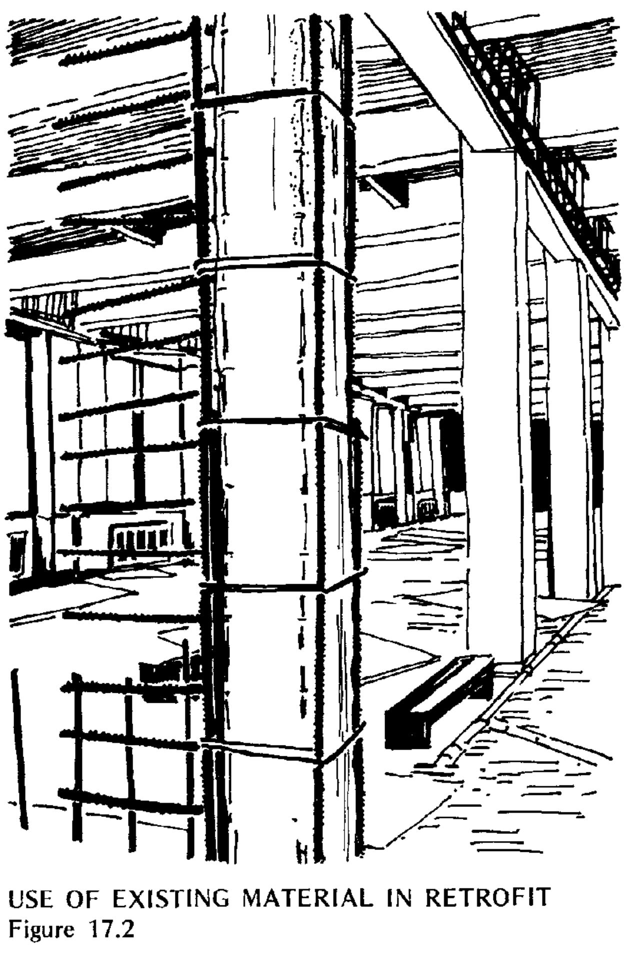
TOWER BRACING
Figure 17.1

By contrast, Figure 17.1 shows very complicated bracing that has been added to the decorative tower of a building. It was needed to keep the tower from being a falling hazard. In this case the owner had no choice but to put in this bracing; it was required by law. The level of safety had to be such that the tower was safe against falling.

The problem posed by that tower raises the larger issue of ornate buildings. Frequently, such buildings are, in and of themselves, safe from collapse; but because of their ornate nature, they have many falling hazards. If access can be controlled so that the public cannot reach an area that might be jeopardized by a falling hazard, then it is possible that the building need not be renovated or strengthened or the falling hazards be replaced. Frequently, however, the most ornate features of a building occur right over the access and egress areas, right over the doors and windows. Of course the owner must either repair these or must offer some additional protection to the people going in or out because the areas of access and egress are most important in terms of protection.

The type and condition of existing construction is also an important aspect that every owner must consider in the retrofit or renovation of a building for seismic safety. For clearly, the type of existing construction is going to have an influence on the selection of the system to be used. In many instances, particularly for concrete and masonry construction, it is essential to know the quality of the existing construction. Frequently (particularly in older buildings) concrete and masonry are of a very low quality. It is necessary, therefore, to set up a testing program in which the actual strength of the material can be determined.

Testing will reveal if there has been any deterioration. This can happen in any kind of building–concrete, masonry, steel, wood frame. Such deterioration can have a significant effect; a system that might ordinarily work can be deteriorated to a point where it will not perform satisfactorily. As a matter of fact, there have been instances of collapse under just dead load simply because of the deterioration of reinforcing bars of a reduction of the concrete strength.



USE OF EXISTING MATERIAL IN RETROFIT
Figure 17.2

Not every test will turn up deterioration. Figure 17.2 shows an instance where the quality of existing material permitted its use as part of the lateral force-resisting system. In the case of this concrete frame building, most of the concrete was in very good shape. A shear wall system was added to what had previously been a frame building because the frame was felt to be especially vulnerable to earthquakes. However, the old concrete columns were suitable to be used as cores for new concrete columns. All the existing beams were also used; new shear walls were simply added between the existing columns.

In another instance, an entirely new foundation and ground floor framing system were required. Although the upper floors of the building were in good shape, the wood used of excellent quality, and the building itself showed very little sign of deterioration, it was poorly fastened to its foundation. So the building was moved to a new reinforced concrete foundation and the entire lower floor was rebuilt. This offered the opportunity to open up the front with doors and windows and put in a new seismic resisting system. In this instance, a rigid frame system was selected to replace what had been a shear wall lower floor.

Functional space requirements also influence the selection of seismic resisting systems. The system must be consistent with the space utilization needs of the user of the building. The effects of different systems on flexibility of plan layouts, of heating and ventilating, as well as light and air are also obviously important considerations. Renovation plans, have had to be abandoned because of the functional requirements of the buildings. There was, in one instance, a school building where a shear wall system was introduced to replace an inadequate frame system. So many shear walls had to be introduced that all of the flexibility for future changes of the layout was lost. It was finally decided after the design was completed to abandon the building.

If an open area is desired with plenty of light, it is not possible to add shear walls to the exterior of the building where the windows are located. For that reason, shear walls can be added along interior corridors or in cross-walls between rooms. Corridor walls of stud dry wall construction can be removed and replaced by new shear walls.

In other instances where it is important to preserve the ability to admit light into a building, a light-weight framing system can be chosen as the shear-resisting element for the building.

Frequently, there is a need for the preservation of ornamental and other structural and non-structural features of a building. This is a consideration particularly in buildings of historical or architectural interest. A system must be selected that will be consistent with this preservation and not destroy the very features that the designer is trying to preserve. For that reason, it is sometimes necessary to develop special details or methods to provide seismic protection.

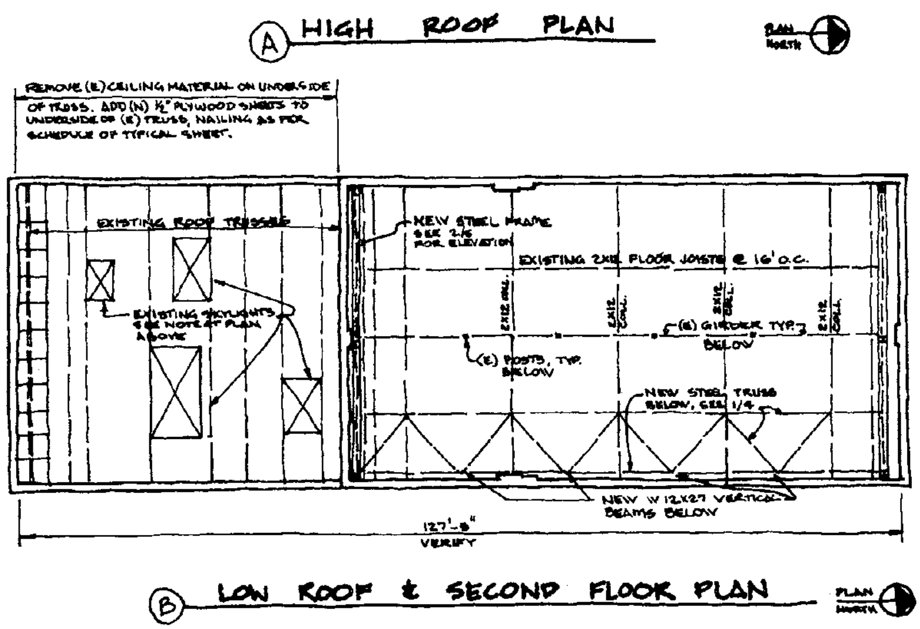
In one instance it was necessary to develop a system for saving a highly ornamented facade of a school building. The facade was made up of cast stone that was erected in rather small pieces for the size of the facade. The pieces were mortared together but not joined in any other way. It was decided to anchor each piece or as many pieces as deemed necessary to the backing material which was reinforced concrete. In order to determine how many anchors should be used, and how frequently they had to be spaced, it was necessary to make a test to see how much force was required to pull the ornamentation from the building. Since there were no standards for such a procedure, it was necessary to develop the standards.

The economics of a preservation or rehabilitation effort are perhaps the most important aspect of both activities. No effort can be mounted without a consideration of the economic problems that are going to be encountered. Of course, in many cases where buildings are of a historical nature, it is not necessary to prove that there is a cost benefit. But in buildings that are going to have extended life or are going to be utilized in some other manner other than preserving their historical aspects, the cost benefits have to be considered. Morever, it is necessary to consider each aspect of the rehabilitation or renovation effort in terms of the cost.

TECHNIQUES FOR SEISMIC RENOVATION/RETROFIT

Since this is a new field, there is plenty of room for new ideas for both renovation and retrofit techniques. However, there are some techniques that have been successfully used.

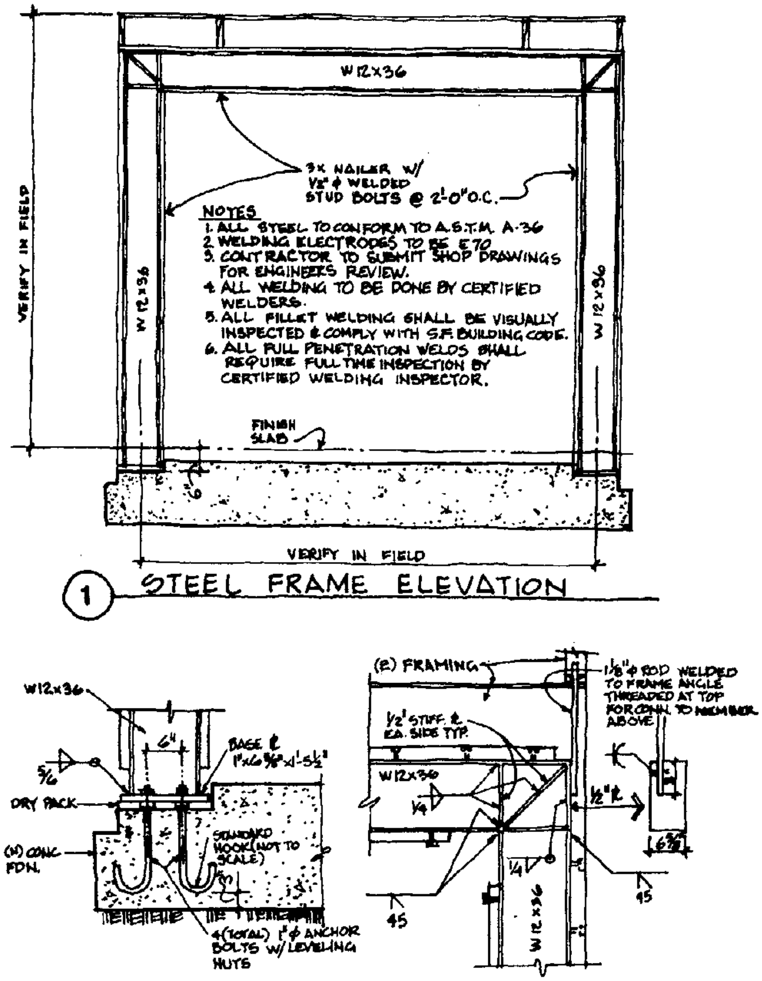
First of all, new ductile moment-resisting space frames with complete horizontal load transfer systems can be effectively used as new elements in buildings that are deficient in transverse seismic support. Infrequently, partial frames exist that can be completed for effective use. More often it is necessary to put entirely new frames into the building. In Figure 17.3 we have a two-level building that required a seismic renovation. Along the right hand wall, the middle wall, and the left hand walls, new rigid frames were built into the building. Since there was not an adequate horizontal diaphragm at the second floor level, it was necessary to put in a bracing frame at that level. The truss system introduced at the second floor level spanned between the new rigid frame that were built at the right hand and the center walls. The existing roof diaphragm to the left of the center ductile frame was adequate, with added sheathing, so no additional horizontal bracing frame was needed at that point.



USE OF NEW RIGID FRAME IN RETROFIT
Figure 17.3

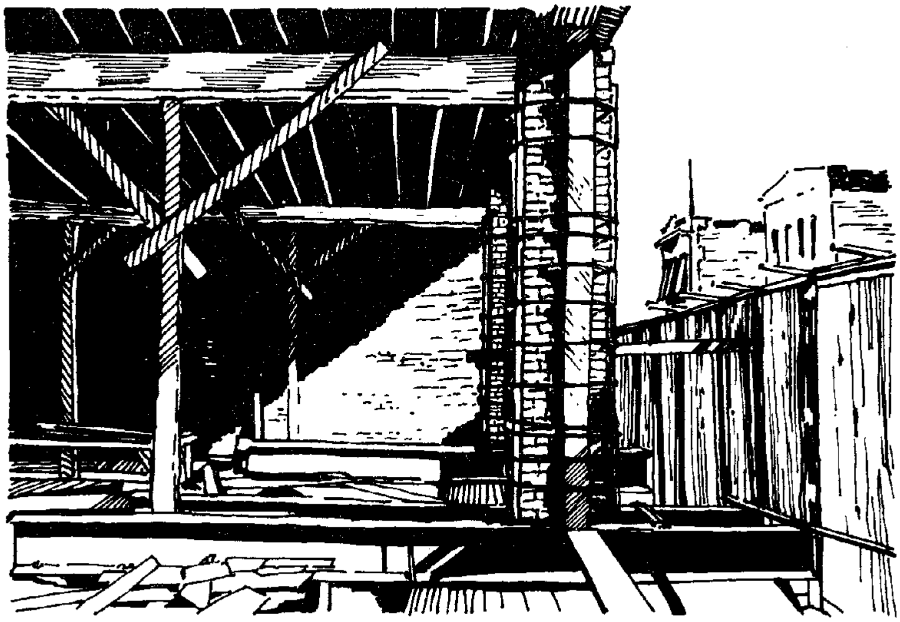
USE OF BRACED FRAME IN RETROFIT
Figure 17.4

How else can this technique be used? Figures 17.4 and 17.5 are an example of a rigid frame that was used in a Victorian residence that was relocated. Because there was a need to open up the front of that building and allow for new doors and windows, a rigid frame was used as the resisting system. A new foundation was required because when a frame such as this is introduced, it has an effect on the foundation loads. This must be considered in the design of the frame.



USE OF BRACED FRAME IN RETROFIT
Figure 17.5

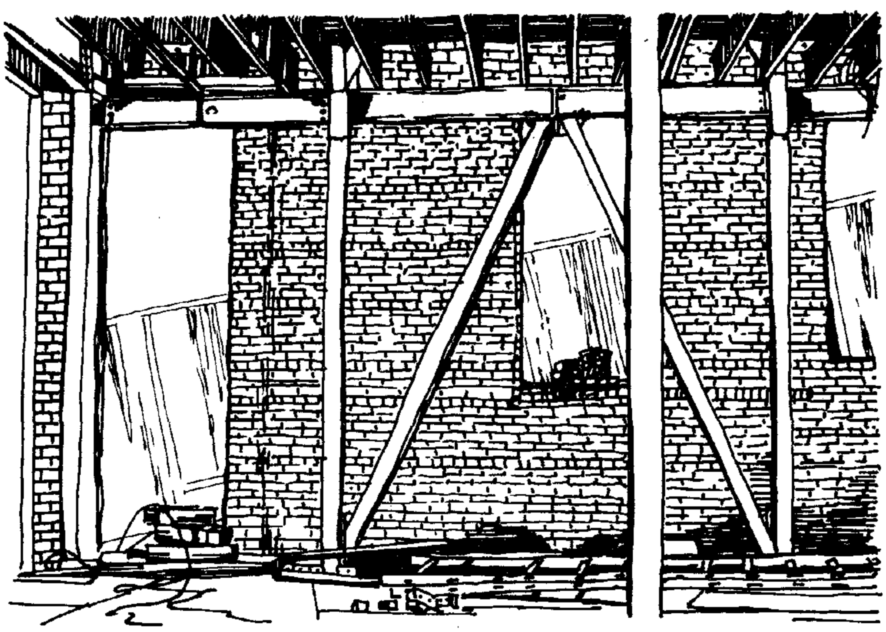
Figure 17.6 shows an example of the effort that sometimes has to be expended to put in a new foundation when the existing foundation is inadequate to react with the new braced frame. Here, shoring is used to hold up the columns while new foundations are constructed. In this instance an imaginative contractor tied the columns to the shoring with the same type of binding used to bind up bales. Finally, a completely new foundation system was inserted beneath the columns, after which a rigid frame was to be inserted at every column.



NEW FOUNDATION IN RETROFIT
Figure 17.6

A second technique utilizes a new box type system by providing shear walls or braced frames together with complete horizontal load transfer systems. A common way of providing new seismic support for existing buildings, this method can have important impact on space, appearance, and function requirements.

Figure 17.7 shows a new braced frame system in an old building. This is an appropriate use for a braced frame system because the wall is frequently penetrated by openings which the owner of the building wants to preserve. These make the wall fairly flexible. Notice that the top member of the frame appears to continue along the wall. This is the collector that is built in all along the length of the wall to drag shear forces back to the bracing.



USE OF NEW BRACED FRAME IN RETROFIT
Figure 17.7

Figure 17.8 shows how a new shear wall will be tied to the columns. Note that the wall is inserted directly under the girders and the new dowels are drilled through and grouted into the girders. The collector beam is mounted alongside the existing girder and the heavy reinforcing is being used to drag the forces to the shear walls. Of course it is necessary to connect the collector to the existing girder by shear transfer dowels. By being able to keep the heavy collector bars below the bottom of the beams, it was not necessary to drill them through the beams. But in some instances it is necessary, in the absence of sufficient room, to drill those bars through the beams so that they can be made continuous.

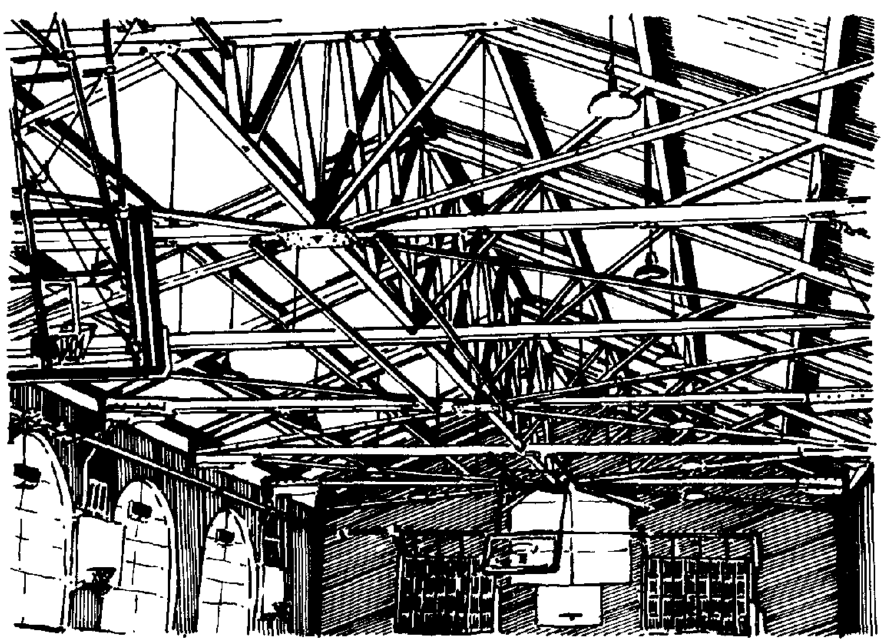
COLUMN TIES FOR NEW SHEAR WALL
Figure 17.8

In Figure 17.9 a braced frame is being used instead of new shear walls to strengthen this building. Again, the collector can be seen as the dark line that goes directly under the connection of the roof trusses into the side walls. The braced frame is appropriate in this building because it was necessary to preserve the light from the windows; also it is a flexible building.

USE OF BRACED FRAME
Figure 17.9

Collectors are required in all those cases where the diaphragms are used to transfer the shears into the side walls and the frames, or where the shear walls are located periodically along the length of the wall. These collectors can also act as chords of the diaphragm which are generally lacking in existing wood diaphragms of old buildings.

Figure 17.10 is an example where horizontal bracing was needed to replace or add to the strength of an ineffective horizontal diaphragm. The diaphragm itself was not adequate to transfer the shears to the two end walls. So a new steel bracing system was added at the bottom chord level of the trusses. It did not interfere with any of the uses of the building and was able to work effectively as a horizontal load transfer method.



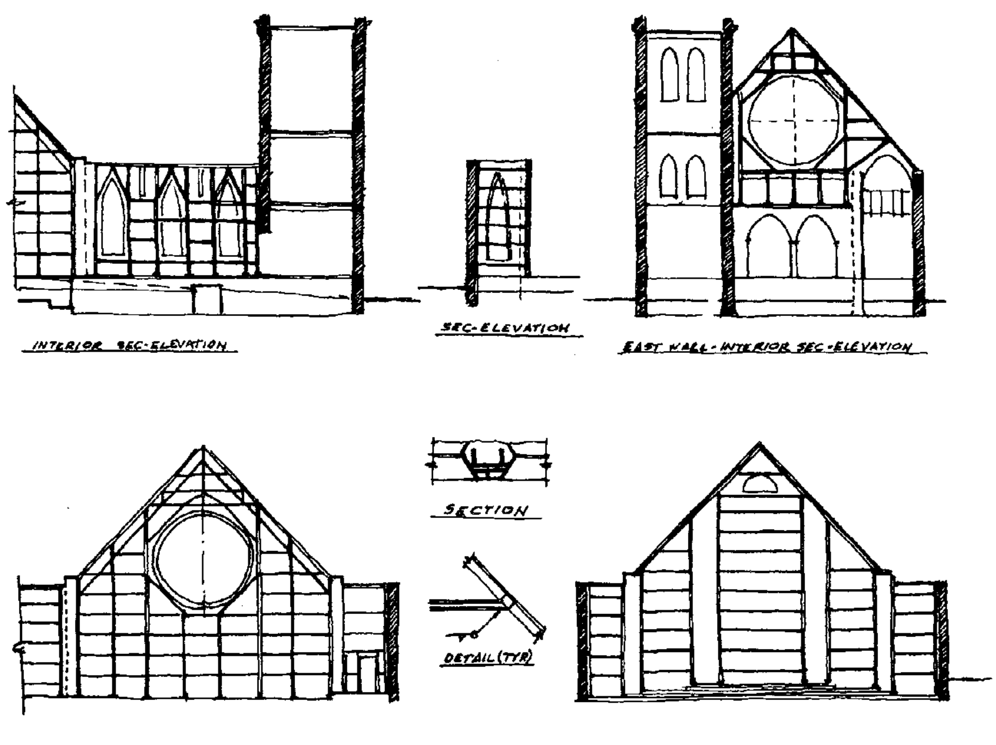
HORIZONTAL BRACING OF DIAPHRAGM
Figure 17.10

New braced frames can also require added foundations and sometimes this requirement can be rather extreme. Considerable force can be brught to the foundation by the frame. As a result, it is not only necessary to build a new foundation, but to buttress it so that it will not slide along the old foundation.

Added shear walls usually do not create local eccentricities and their use may prevent the need for work on the horizontal diaphragm. However, in braced frames where the frames are usually added inside or outside of existng walls and the collectors are then put on the face of the walls, some torsion can be generated at the connection between the collector and the braced frame itself. For this reason, it is necessary to consider what that torsion will do the diaphragm. Usually it is necessary to carry out some torsional-resisting members into the diaphragm and anchor them so that the connection cannot twist when the load is applied.

Until now we have been considering bracing in the in-plane direction of the wall. But sometimes it is necessary to brace the walls perpendicular to that plane. This can be accomplished by applying a gunite skin to the face of the wall with studs or grooves cut into the wall so that a vertical beam can be established in the gunite or, as an alternative, with a steel bracing grid.

Figure 17.11 shows a case where the walls span a long distance between the foundation and the roof diaphragm. It was thought necessary to provide extra strength perpendicular to the face of the walls; however, this approach is more typical in single-story buildings than in highrises where the compressive weight of the dead load on the outside walls helps to resist the lateral forces perpendicular to their plane. In this particular building the bracing was added on the interior because the soft granite exterior could not be marred. A gunite skin was ruled out because cutting grooves in the building would have been a very difficult operation. Instead, a steel frame was inserted on the inside. The heavy lines identify the vertical elements of this steel frame. The lighter lines are horizontal elements spanning between what might be called soldier or vertical beams. The frame is anchored into the wall. It was necessary to anchor the existing wall to the frame so that it would interact with this new frame.



USE OF INTERIOR STEEL FRAME
Figure 17.11

The anchorage of frames to walls can become a complicated process. First of all, it is sometimes necessary to drill into material which presents special problems. Secondly, it is frequently necessary to decide what kind of material can be used to bond these anchors into existing material. In most instances, it is not possible to drill through for a connection on the outside; the appearances of the building would be destroyed. So almost every case is a special instance that requires a unique system. In some instances oversized holes can be drilled, bolts inserted into the oversized holes, and epoxy grout then injected into the holes.

A third technique for seismic renovation and retrofit is to upgrade existing systems by correcting deficient details and strengthening existing elements. Performances of seismic resisting systems and materials are tested from time to time by severe earthquakes. Certain accepted design techniques and details sometimes are found to be deficient. In retrofitting for better performance, information gained from past earthquakes is invaluable.

Figure 17.12 is an example of a modern structure in which the plywood diaphragm failed. Here, the end wall fell out of the building. The structure was designed to modern standards, but it had a detail in the roof diaphragm that was deficient. The roof diaphragm came to a ledger which was bolted to the wall. The deficiency in the detail was that the diaphragm was simply brought over and nailed to the leder without adequate provision to drag the load into the diaphragm. The detail failed under seismic loading. The appropriate way to make the diaphragm sustain the load at the wall is to take the load from the wall into the diaphragm. It is necessary to make a positive connection between the purlins or joist members which frame to the wall and the wall itself. This can be done by tieing the sub-purlins or blocking across so that it extends out into the diaphragm and becomes a more effective drag member. This is a very simple retrofit of a deficient detail that can make an otherwise unsatisfactory system function appropriately.

FAILURE OF PLYWOOD DIAPHRAGM
Figure 17.12

A fourth technique is to utilize existing elements not ordinarily accepted for lateral force resistance. When actual expected performance is the criterion, rather than conformance to code, it is possible to evaluate the seismic resisting effect of all elements in the building not ordinarily accepted for seismic resistance. Special thought must be given to expected and desired performance levels under these circumstances.

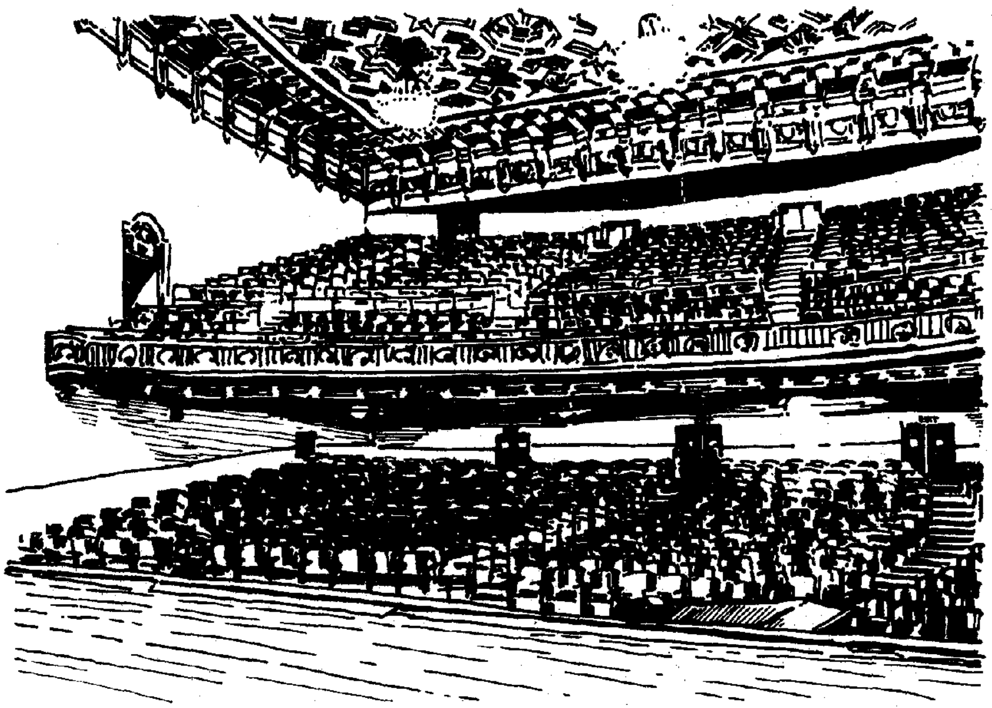
Figure 17.13 is an example of a building that was severely damaged by an earthquake; but as can be seen, although the exterior walls fell out of the building, the building did not collapse. Why? Because the interior non-structural walls functioned as shear walls. It might be argued that if the walls had been better fastened to the roof and floor lines, this building would have survived the earthquake intact. However, this conclusion should consider the possibility that the added load of the wall remaining attached to the building might have overstressed the seismic resisting nonstructural walls. We can, in certain instances, utilize non-structural elements to resist lateral forces. Many governmental agencies that are not under any particular requirement to follow seismic codes frequently ask us to make sure that we consider every possible resistance to lateral force before we spend money to introduce new frames or new shear walls into buildings. But almost always, these nonstructural elements are used only in voluntary programs. If a program is mandatory, we usually have to follow a code and the code generally does not permit the use of nonstructural seismic resisting elements.

NONSTRUCTURAL WALLS FUNCTIONING AS SHEAR WALLS
Figure 17.13

PRESERVATION OF NON-STRUCTURAL FEATURES

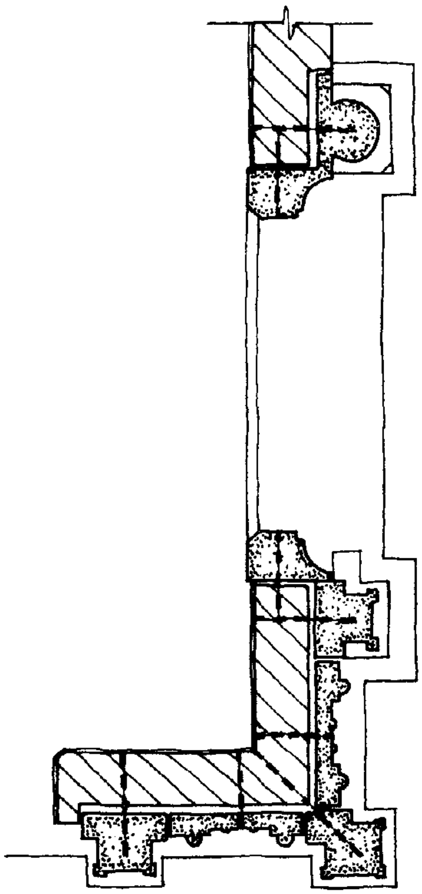
Sometimes the preservation of non-structural features, particularly ornamental features, becomes a major problem requiring the highest degree of innovation. As a matter of fact, some of the most difficult task we have had to face in renovations have been in the saving of the ornamental features of a building.

Figure 17.14 is an auditorium in a school building; it has a highly ornamented ceiling and considerable ornamentation around the walls where they meet the ceiling. It was decided that it would be impossible to recreate this ceiling, yet considerable structural strengthening had to be done to make this space safe. Fortunately, we were able to get in above the ceiling and put in bracing. The walls had a gunite skin put on the outside without touching the interior.



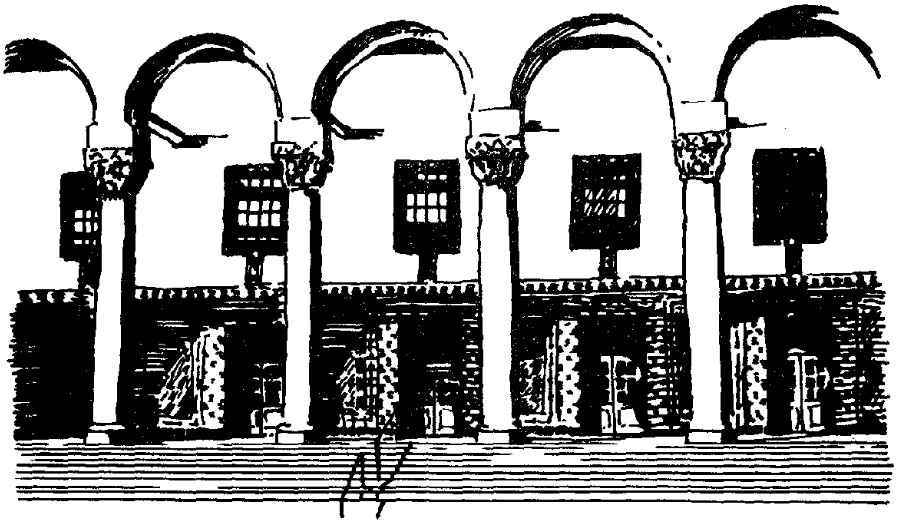
ORNAMENTED CEILING IN AUDITORIUM
Figure 17.14

The anchoring of the heavy ornamentation was one of the most difficult tasks that we had to face. There are many decisions that can be made in such situations. Fortunately, in that case of most of the exterior ornamentation we were able to drill completely through reinforced concrete walls with anchors that were then grouted into the heavy ornamentation. Figure 17.15 is a picture of what that ornamentation looks like on the building itself. In some cases, it actually sticks out more than two feet from the face of the building. Two other types of pins were also inserted to anchor the ornamentation. Some of them were drilled-in inserts commonly called expansion bolts; in other words, bolts were inserted into holes drilled to the exact size of the bolts. The other type of pins used oversized holes. In these cases, the pins were inserted and grouted. But in neither case did we penetrate through the wall. Where the bolts are going to be left exposed on the exterior, it is almost always necessary to put some kind of non-deteriorating washer under them so that both bolt and the ornamentation are protected.

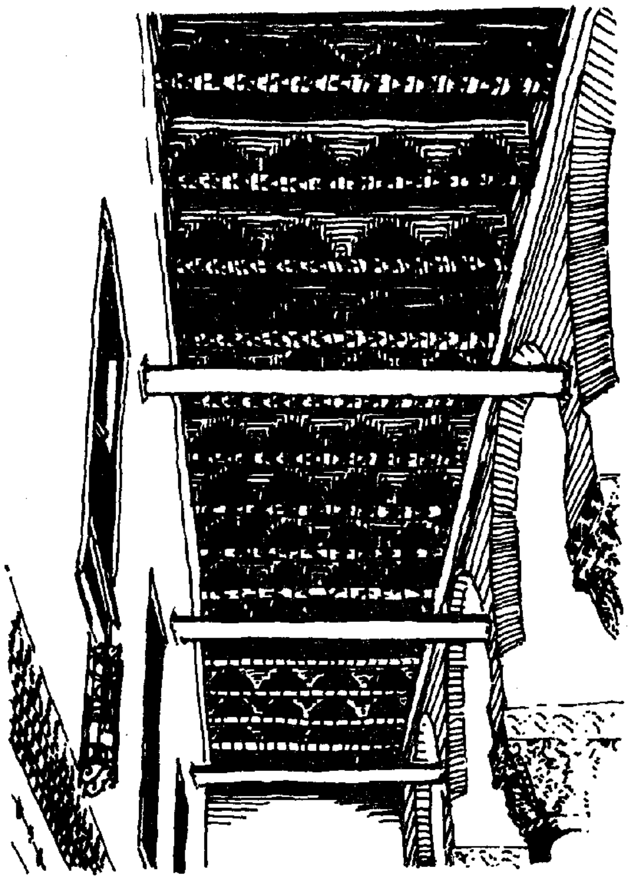


ANCHORING OF HEAVY ORNAMENTATION
Figure 17.15

The columns shown in Figure 17.16 were not structurally sound in terms of resisting lateral forces. It was necessary to provide a bracing system to keep them from buckling at the joint between the columns and the arches above. However, back in the corridor there was a beautiful ceiling. It was decided, therefore, not to destroy the view of that ceiling from the inside of the colonnade. So the braces had to be kept fairly light and unobtrusive. Figure 17.17 shows the scheme that was eventually chosen. The braces were carried back into the building, and additional bracing was included inside the building where it could be hidden in the ceiling area. Thus, the columns were saved as well as the view of the ceiling.



RETROFIT OF COLONADE
Figure 17.16



BRACING OF COLONADE
Figure 17.17

Sometimes there are interesting spaces that are very, very difficult to preserve. All of the counters in Figure 17.18, for example, are heavy marble. To our knowledge they are not fastened by anything and are very vulnerable. The heavily ornamented arch ceiling is also unbraced and vulnerable. This particular ceiling also has to be braced in a manner that will not disturb the exterior appearance. Methods for saving the counters and ceiling without marring their appearances has not yet been devised.

RETROFIT OF HEAVY COUNTERS
Figure 17.18

These are all quite difficult tasks. They make the point that seismic retrofit and renovation is a new field open to new ideas. Many problems have been solved, but for the most part solutions have been unique to each situation. We have not solved every problem that we have faced; many still remain.

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Chapter 18. BUILDING RETROFIT FOR SEISMIC SAFETY

James Cagley

INTRODUCTION

The basic procedure that should be followed to evaluate the earthquake resistance of existing buildings involves several different aspects, but the initial step is to make a determination of what the potential earthquake might be. This could be called a site evaluation survey.

Shortly after the San Fernando Earthquake in 1971, the Veterans Administration (VA) embarked upon a rigorous and organized program to evaluate the structural integrity of all their hospitals, during which this writer was personally involved in several projects in the Southeast. The first step was to obtain site evaluation surveys of about 69 sites. For instance, in August, Georgia, the VA came up with a maximum ground acceleration of 18 percent g. or 18 percent of the weight of the building. At Atlanta, they came up with 13 percent g; at Birmingham, 11 percent g.

A site survey can also be carried out on a community-wide basis as has been done, for instance, in Long Beach, California. Long Beach has an ordinance that is bringing many existing buildings up to a reasonable degree of seismic resistance. This is being done on the basis of a procedure that gives greater priority to buildings that would have a tendency to affect more people. The ordinance will also involve key emergency facilities such as hospitals, police and fire stations as well as buildings that house large numbers of people. The Long Beach earthquake regulations have been the most rigidly enforced on the West Coast.

As another example, several years ago, the City of Los Angeles made a determination that they had in excess of 10,000 buildings that were pre-code unreinforced masonry buildings. These buildings were potential earthquake hazards. Yet despite approximately two years of debate, nothing has yet been done. No doubt another good earthquake in the United States will make everybody understand again that there is, in fact, a significant problem.

Of course, the West Coast is not the only area in the continental United States subject to earthquakes. In the late 1800's there was a large earthquake in Charleston, South Carolina. This earthquake affected much of the Southeastern part of the United States. The point is that the ground moves all over the country.

SITE EVALUATION

The first thing that is called for, wherever the location, is to determine the potential seismicity of an area. Generally, this can be done by looking at the history of earthquakes. An invaluable resource is the U.S. Geological Survey Center in Boulder, Colorado. They have on record a list of earthquakes—depending upon the geographic area and timespan under consideration—that will provide information on every earthquake of any magnitude that has occurred within the last several hundred years.

Once the potential magnitude of the occurance is known, determine whether there are specific geologic or soil problems, such as liquefaction. Then an estimation of the probable effect of an earthquake on the foundation system of the building is possible. Finally, it is possible to come up with an estimate of the kind of ground forces that may occur during the potential earthquake.

In their site surveys, the Veterans Administration chose to use what is called peak ground acceleration. The peak ground acceleration becomes in reality a dynamic criterion that is applied statically to the building.

SUMMARIES

The following list includes the type of information that is needed to prepare a site evaluation survey for existing facilities:

- A list of historical earthquakes for the region, which in the opinion of the consultant would have produced ground intensities near the site, of intensity V on the Modified Mercalli Earthquake Intensity Scale (MM) or over. This list may be supplemented by a map of the region when appropriate, showing the centers (greatest damage or epicenter) of the earthquake in relation to the site.
- A brief review from the available historical earthquake record of reports of ground shaking, damage, etc., near the site.
- An estimate of the maximum intensity (MM) of ground shaking on firm ground near the site which has occurred in historical times, and a statement on the earthquake which caused it.
- An estimate of the location, magnitude, and faulting mechanism of the earthquake which can reasonably be expected during the planned life of the facility to produce the maximum ground shaking near the site.
- An estimate of the frequency of occurrence of the maximum intensity and the earthquake that could be reasonably expected.
- A geological map with discussion giving a summary account of the structural geology in the general region of the site.
- A list of any major faults in the region and the type of displacement on them. Evidence of tectonic movement on any of these faults in Holocene time should be cited.
- A discussion of the stratigraphy and rock structure at the site itself. Attention should be drawn to any geological conditions at the site that may constitute a special hazard to the facilities.
- An estimate of the probable effect of earthquake motion on the foundation soils.
- An estimate of ground shaking at site for the earthquake that could be reasonably expected.

STRUCTURAL EVALUATION

Assume that it has been determined that the site of an existing building has a potential for an earthquake. The next step is to look at the building and see if it has a seismic-resistance problem.

One of the best ways to review existing buildings is to use the construction drawings. If the drawings are not available, a cursory visual examination of the building is necessary. Try to establish that the building is either conforming or nonconforming to earthquake requirements. Is it nonconforming because it has a parapet wall that might fall, or it is nonconforming because the structural system is not tied together? In other words, the degree to which a building does not conform to current requirements has to be established. Consideration should be given to the following potential structural hazards:

- Unreinforced masonry interior and exterior walls,
- Pounding,
- Framing system,
- Diaphragm and bracing systems,
- Shape and configuration of building,
- Settlement and state of repair,
- Connection of walls to frame or diaphragm, and
- Others identified by the A/E.

In reviewing the drawings or in an on-site inspection of the building, look for unreinforced masonry interior and exterior walls. A sheet rock partition may go to pieces during an earthquake, but it probably will not kill anybody. However, if a masonry wall comes down it can kill or injure anyone around it. Also, look for masonry partitions that are not supported laterally. A lot of buildings in this country are built that way.

Another major problem is pounding. When designing to resist earthquakes, the fewer expansion joints the better. For every time there is an earthquake, expansion joints are going to cause problems in pounding, since adjacent parts of the buildings do not usually move at the same time or in the same direction. Of course, with lateral loads, it is important to have everything symmetrical, particularly the resistance to the loads, and if possible the load itself. Consider, too, that when looking at existing buildings, even if a building was designed for a high lateral wind load, it does not necessarily mean that it will resist an earthquake. For example, a VA hospital in Atlanta had a welded steel frame in the transverse direction which worked fine for resisting earthquake loads. However, in the longitudinal direction, the building had a series of columns. Since the building had a narrow face in this direction, the computed wind load was very low. To increase the earthquake resistance in this direction, it was necessary to design a bracing scheme for the building.

Therefore, look at the framing system itself and determine whether it has some inherent characteristics that either do or do not lend themselves to resisting lateral loads. In most buildings built today, there is usually a good floor diaphragm. Any cast-in-place concrete building inherently has a good floor diaphragm that will carry loads to the existing lateral load-resisting elements.

If these elements do not exist, they can be put in. Even the old masonry bearing wall buildings with cast-in-place floors are probably pretty good. Steel buildings, if the diaphragms are somehow tied to the steel, are fine, but often this is not the case. Look at the diaphragms and look at the bracing systems if there are any.

Investigate to find out if there has been any serious settling of the building. It is important when starting out to retrofit a building to resist an earthquake not to inherit other structural problems.

The state of deterioration of the building is another critical area. As far as the structural system is concerned, this is usually not a serious problem, unless it is an exposed steel building and has not been painted.

Consideration should also be given to how the walls are connected to the frame—if they are connected to the frame. If the walls and frame are tied together, the building is not going to fall apart. Most concrete beams do not fail, simply because the designers put in number six bars instead of number seven bars. The important thing is that there is a bar there in the first place. It may deflect, but it probably will not fail.

Do a certain amount of testing of the materials. For example, test the concrete to determine what the realistic strength of the concrete is in the building. What it was designed for is one thing; what its strength is, in fact, may be something else.

We have discovered that it cost about $2,000 a test to take a masonry panel out of a wall and test it in a racking shear. This is for a panel about 30 x 30 inches to 4 feet square. Of course the major cost is not the testing; it is putting a wall back in the building. A less expensive alternative may be to drill six inch horizontal cores through the wall for the shear test. You can get pretty good correlation—and for only a few hundred dollars.

NONSTRUCTURAL EVALUATION

A list of exterior and interior falling hazards should be prepared with consideration given to the following potential hazards:

- Decorative tile or stone,
- Unbraced parapets, gables, balconies, etc.
- Terra cotta roof tile or slate
- Suspended ceilings,
- Lighting fixtures,
- Unbraced partitions, and
- Others identified by the A/E.

Take a look at decorative tile or stone. Actually, the older the building, usually the better the stone anchor details. One problem can be unbraced parapets or gable ends. It is not uncommon to have a masonry gable in a building that stands maybe 20 feet high at the center with the top joist running parrallel to it. Since there is nothing bracing the top of that wall, these gable end walls are apt to fall out during an earthquake, and they are usually placed over the entrance to a building.

What about suspended ceilings? A light-weight acoustical tile ceiling is probably not a serious problem. However, plaster ceilings can be a problem if they are not braced in some way. Some problems may also occur with hanging light fixtures. The answer is not to eliminate such fixtures altogether; just brace them in some way.

Also take a look at unbraced partitions and the major elements of the mechanical system. One problem is elevator equipment. In the San Fernando Earthquake, out-of-service elevators were one of the major problems. The Los Angeles County Medical Center, which was about 30 miles from the center of the earthquake, had 33 elevators; 32 of them went out of service. One of the basic problems is that elevator rails are typically attached at every floor because that is where the framing is. However, when loads are applied horizontally, they need bracing at closer intervals.

Moreover, the weights for the elevator counter balance are bars that are set in a strap. If they shake loose, they can come right through the roof of the elevator cab. However, that particular problem can be easily solved by banding them in place. In fact, often a designer does not have to do a whole lot on some of these things. It is just a matter of being aware of the problem.

RETROFIT SOLUTION

The non-conforming structures are then assigned into one of the following categories:

- Lateral force-resisting elements need immediate major corrective action,
- Lateral force-resisting elements need major corrective action; or
- Lateral force-resisting elements need corrective action.

Now, once the study has identified the problem, the next consideration is what might be the solution to that problem and what that solution might cost. The entire procedure falls into a three-phase operation:

- Determine if there is a problem;
- Determine what the solution to that problem is and how much it will cost; and
- Draw up the working drawings and actual construction of the solution.

The following scope of work details the information that is needed to proceed with the retrofit:

- Diagrammatic sketches and cost estimates (including architectural, structural, mechanical and electrical work) for a system of strengthening the buildings and correcting seismic deficiencies to meet the standards set by independent study shall be prepared to accomplish the following: (1) Provide a structural system capable of resisting the earthquake forces; (2) Assure that floor and roof diaphragms can distribute the earthquake forces; (3) Brace unreinforced masonry walls as necessary to prevent collapse under strong ground motion; and (4) correct other major seismic deficiencies identified. In arriving at recommended solutions, the A/E shall investigate various alternate methods in order to arrive at the most economical total construction cost.

  No effort shall be made to rehabilitate, modernize, or update the building's functions or systems which are not directly affected by the changes required under the above. The A/E shall confine his work to updating the buildings to meet the safety standards set forth for seismic criteria and the related architectural, mechanical, electrical, and structural work involved.

  The structural portion of the work will include the preparation of seismic calculations which will become the basis for strengthening the buildings. Studies will be made for the location of any required additional shear walls within the buildings. From these studies the most economical solutions will be developed.

  The architectural portion of the work will include the necessary planning of spaces as they are affected by the structural changes to the buildings. A study of the material that will be affected by this work will be made as well as a determination as to the replacement or patching of these materials.

  The scope of the electrical work will include the investigation and solution for proper support and attachment of major electrical equipment. Elevator equipment that is found to be unsafe and dangerous due to improper anchorage will be redesigned. All electrical work affected by the structural changes will be noted and costed.

  The mechanical work is similiar in this scope to the electrical: Investigation and design solutions will be made for proper support attachments of major mechanical equipment that is found to be hazardous. Similarily, all mechanical work affected by the structural changes will be noted and costed.

- An outline specification and descriptive analysis sufficient for cost estimates shall be prepared. Cost estimates prepared for each building shall be by major line items and unit prices.

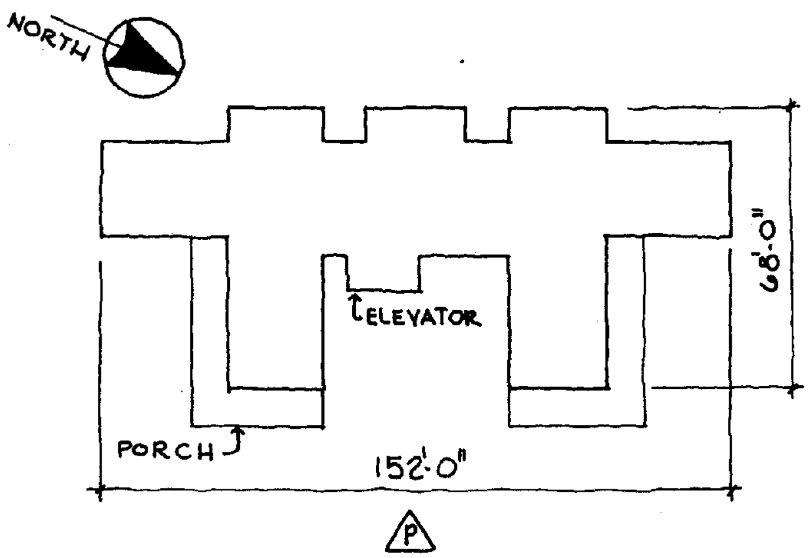
- A time schedule for the development of plans, review meetings, bid periods and the construction will be established for this work. A crucial element to this time schedule will be the phasing of the construction.

The solution does not merely involve the structural system. The architectural ramifications have to be considered as well. Try to avoid major replanning in the building, particularly in a medical facility. For example, if there is a major pipe running down a corridor which is an exitway, it could be serious if it broke. If it is in a tunnel, however, perhaps it does not make any difference. Consider, in other words, what the ultimate ramification will be and, as in the case of a fire, try to protect the exists. Also, in developing solutions, be sure they can be built, that some sequence of events can be developed that will allow the system to be constructed. Remember that it is not always possible to close down the operation of the facility being worked on. One possibility is to work by floors when retrofitting a building for earthquakes. If so, do not start at the top. There could be serious problems if an earthquake occurred in the interim.

RETROFIT PROJECTS

At this point, I would like to discuss several specific seismic retrofit projects.

In Augusta, Georgia, we looked at a complex with some 24 buildings which together had about a million and half square feet. The buildings were of many types, but the typical building was a two-story or three-story clay-tile bearing wall building with a joist system formed with tile.



PLAN-AUGUSTA HOSPITAL
Figure 18.1

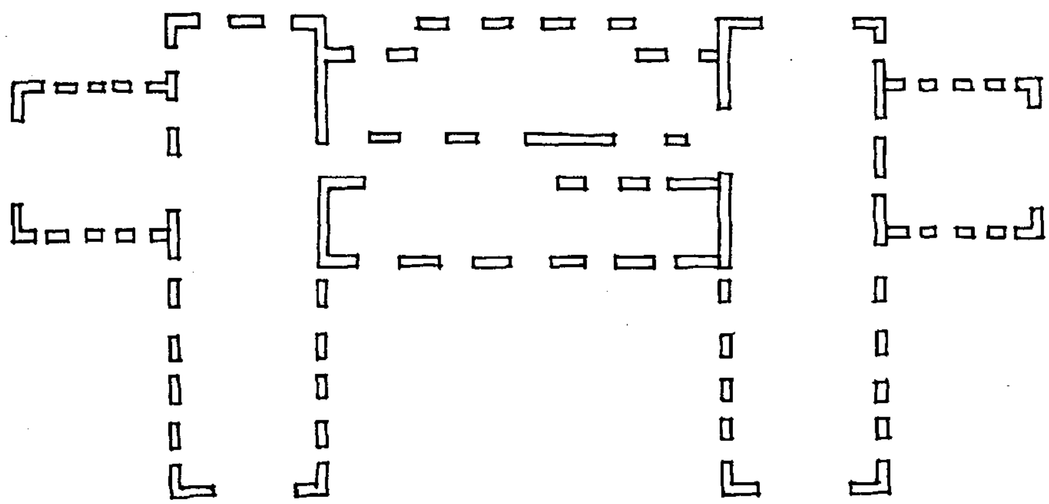
These buildings were examined using the potential of an 18 percent g earthquake. The 18 percent g came from an evaluation of the earthquakes in the area, primarily the 1886 Charleston earthquake. The VA looked at the Charleston site with 25 percent g. When we used their criteria in this building, we came up with a lateral force equivalent to 38 percent of the weight of the building. However, the Uniform Building Code would have given 13.3 percent g as a box structure. That is a big difference. Some factor had to be applied to the 13.3 percent g, but not necessarily 3.

This is, again, a clay tile bearing wall structure with joists. The floor-to-floor height is about 10 feet. These buildings do not have much mechanical equipment because they are neither heated nor air-conditioned.

The existing building can be looked at as a series of masonry piers because of the amount of window openings. The floors were concrete and acted reasonably well as diaphragms. Also, some rebars ran all around outside of the slabs so there was accidentally some chord reinforcing in the diaphragms.

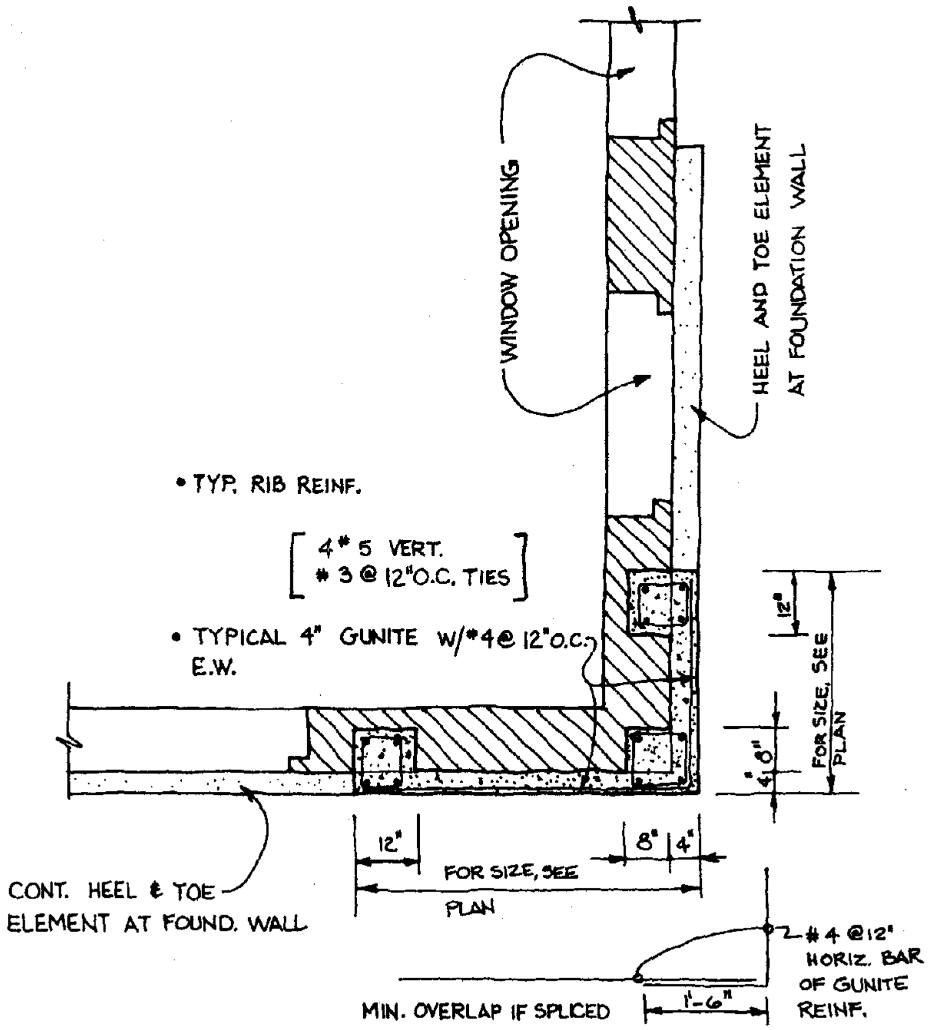
After we studied it, we saw that we had a building with very brittle wall elements. The exterior wall was basically 12-inch clay tile with cement stucco on the outside and plaster on the inside. We tested these elements in shear and came up with values on the order of 50 - 60 PSI times the gross area of the section. The fact that we had gotten good test results led us to wonder if we could confine the masonry.

If there is a way to chase the loads through the buildings, then the building will work. We took the building, split it up and looked at it in pieces. We tried to locate the elements that were going to take the loads in the same location as the mass was generated. We then put gunite membranes at these locations, though we could have replaced the whole wall.

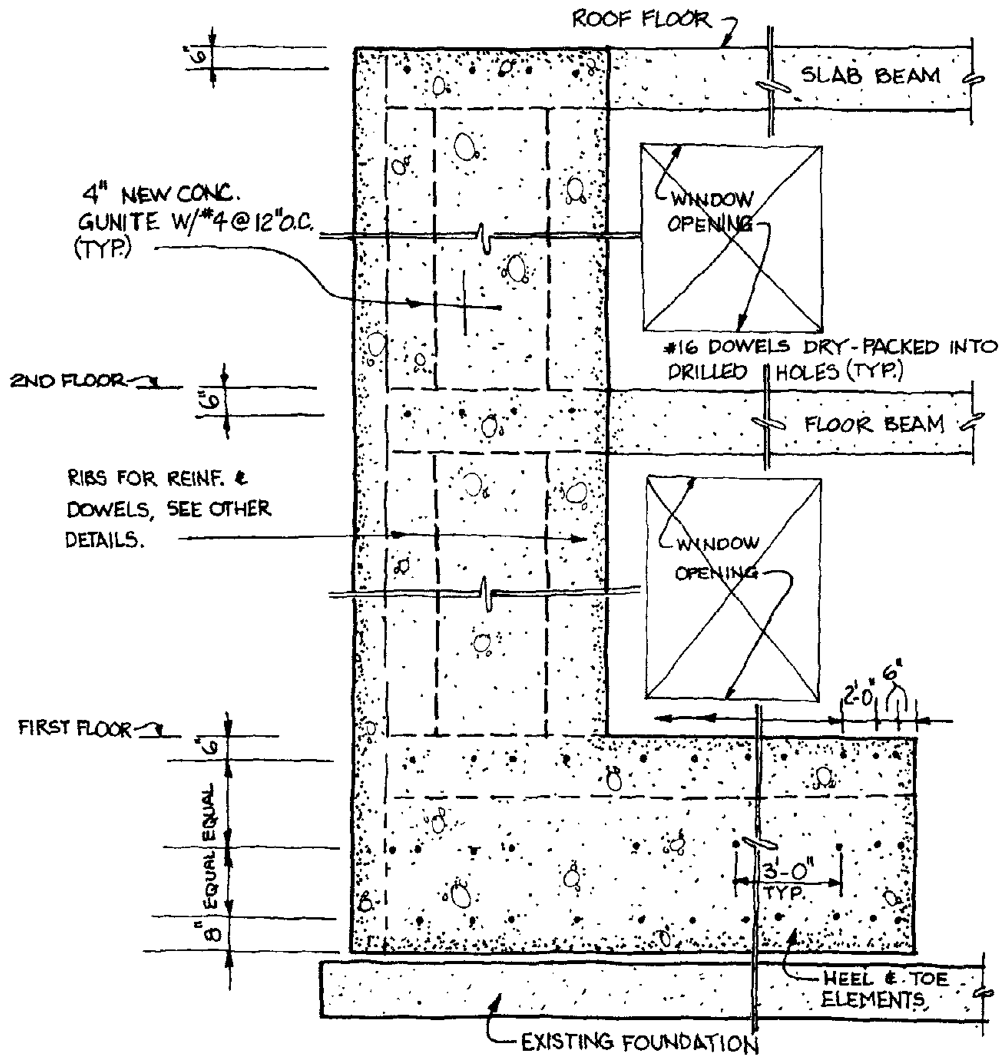


LAYOUT OF EXISTING MASONRY PIERS
Figure 18.2

Figure 18.3 shows a typical corner detail. We put in roughly a 4-inch gunite membrane. The columns were cut back into the wall. Figure 18.4 shows how the problem of overturning was handled by putting a heel or toe element on the existing foundation which expanded the area at the base.

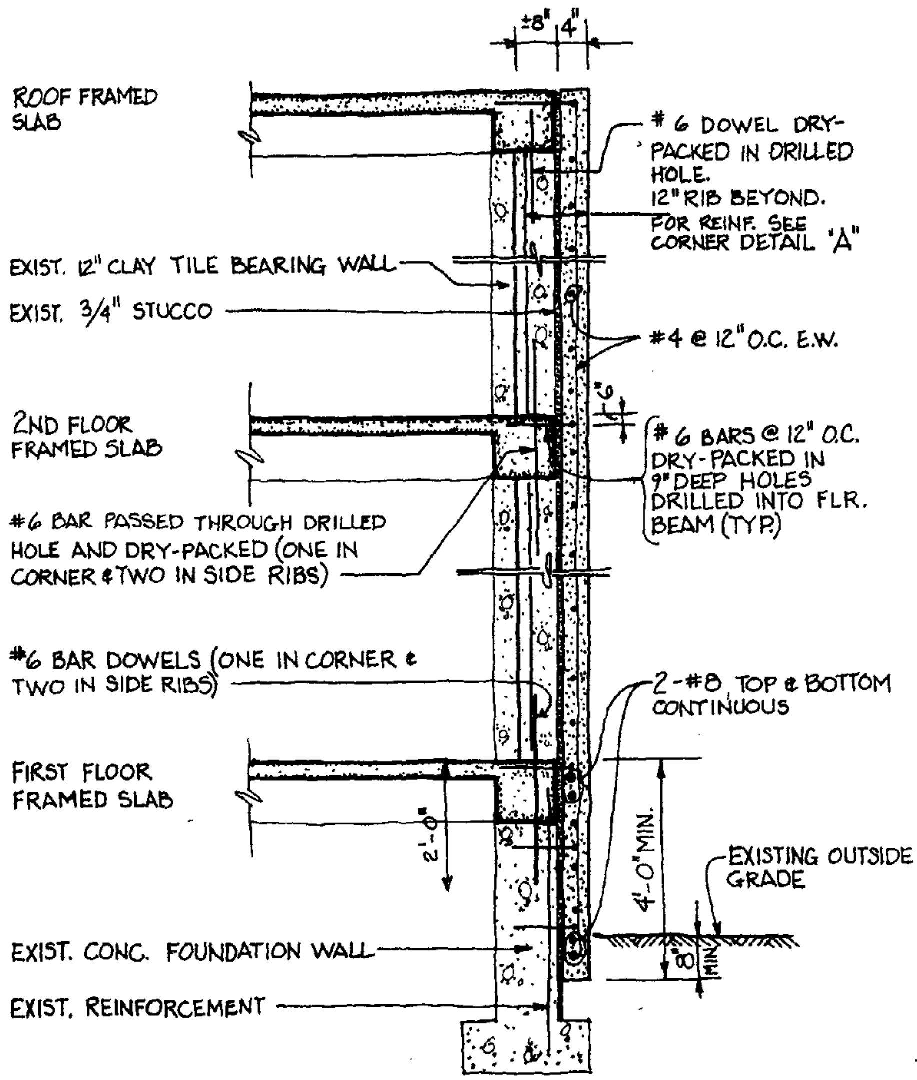


TYPICAL CORNER DETAIL
Figure 18.3



TYPICAL CORNER ELEVATION
Figure 18.4

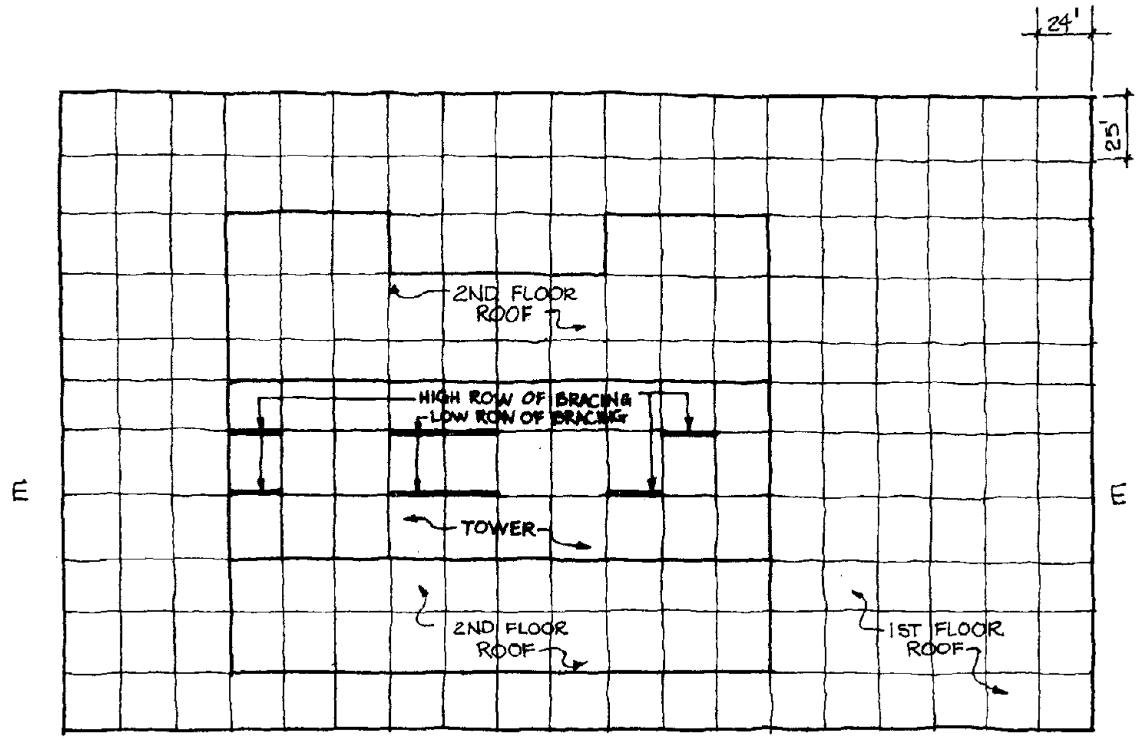
Note that these elements have to be tied into the floor systems and carefully tied together with bars. Again, the approach is related to the specific building that we had, but the point is always the same —tie the building together. The cost was somewhere between six and eight dollars a square foot to retrofit these buildings.



TYPICAL SECTION THRU GUNITE ELEMENT
Figure 18.5

The next project was in Atlanta, Georgia. Finished in 1966, the Atlanta hospital is a 12-story welded steel frame with half a million square feet. It is welded in both the transverse and the longitudinal directions, but there is not much strength in the longitudinal direction.

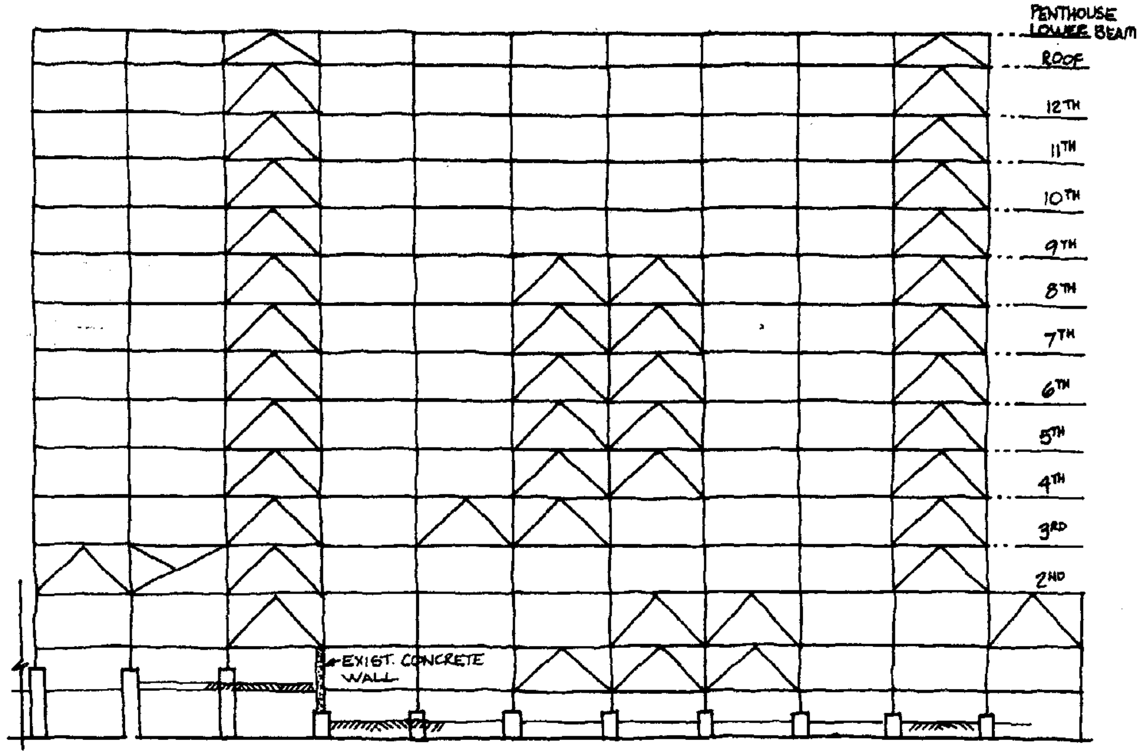
Whereas we used a force of 18 percent g in evaluating the facility in Augusta, here we used 13 percent. Again, the force was developed in relation to the structure's proximity to Charleston.



PLAN-ATLANTA HOSPITAL
Figure 18.6

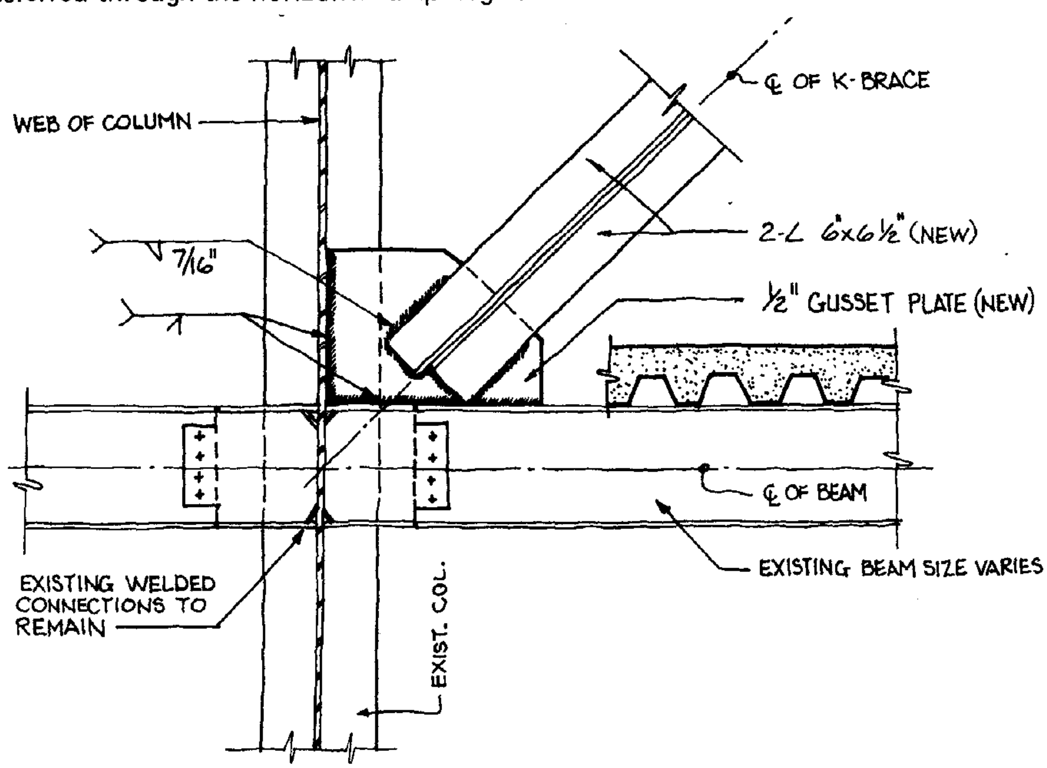
We ran a computer analysis of the transverse section which indicated that we had some overstressing in these members. But it was an overstressing of 20 to 30 percent, which is not going to happen. What is more likely is that the members are going to redistribute the loads.

Now, in the longitudinal direction we ended up having to add some elements because the frame was designed for wind, but the wind force on the end of the building was small. We introduced K-bracing in the areas of the elevator and stairwell cores in the building. However, we did run into some problems at the lower levels.



LOCATION OF K-BRACING (COLUMN LINE E)
Figure 18.7

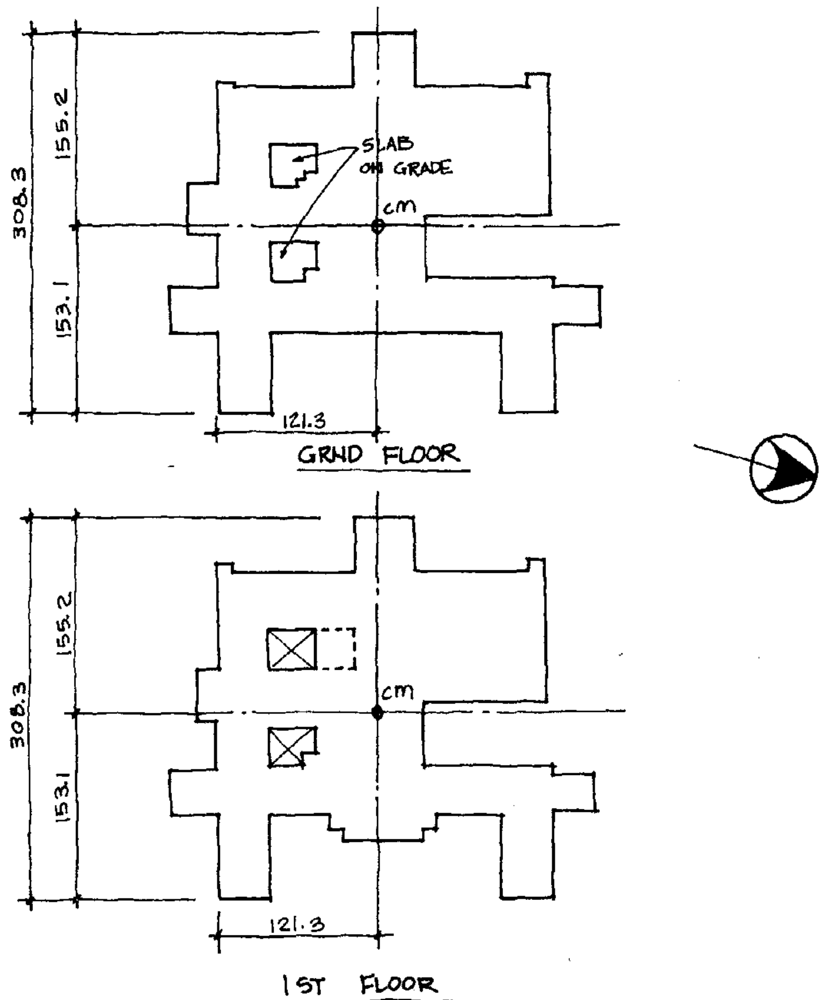
For one, we could not carry the K-bracing down at certain points because of functional requirements. In one case the corridor changed locations at the first floor; and in another, we had the kitchen. It was possible to locate the braces at different points; however, when the bracing force is transferred, it has to be transferred through the horizontal diaphragm.



TYPICAL K-BRACE CONNECTION
Figure 18.8

The foundations are pile foundations. We were able to transfer the tower load out to the foundations. It happened that the floor system was so designed that these were all well tied together. Therefore, we had a good set of ties to transfer the load. Also, we had a basement all the way around so that we could take a certain amount of lateral force by transferring it into the earth. The cost of retrofitting this building was $1.50 a square foot.

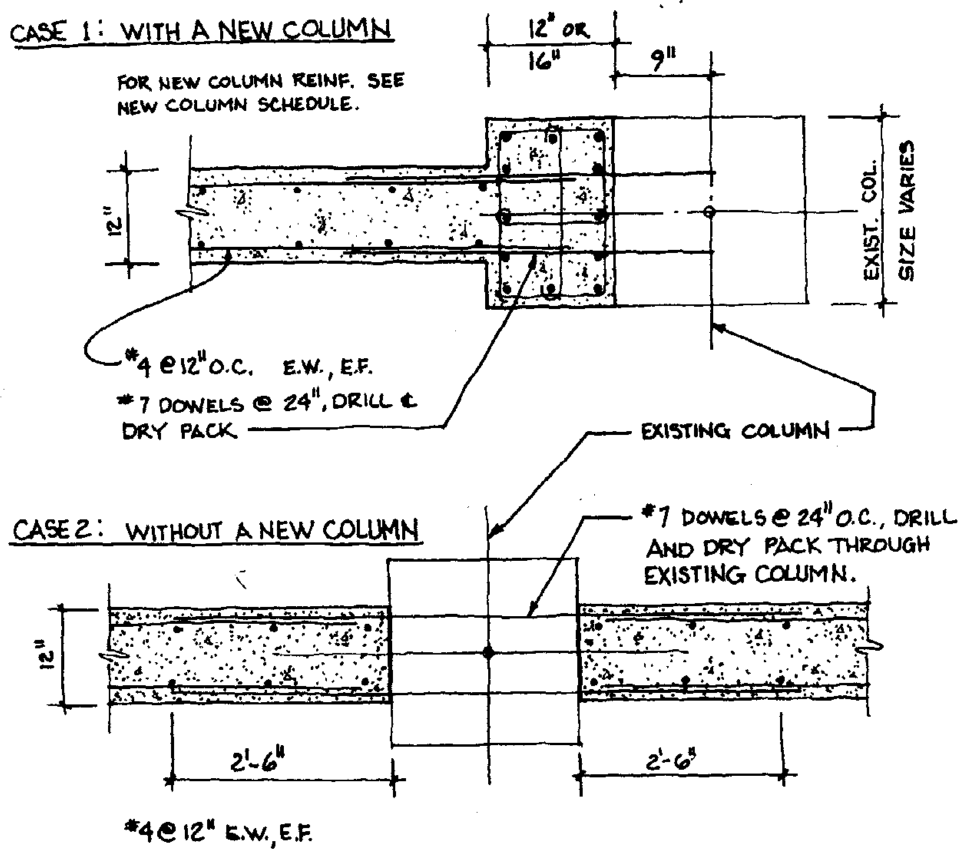
Figure 18.9 is the Birmingham Hospital. There were three buildings involved. The main building was completed in the early 50's and is a nine-story, reinforced concrete frame. It was a challenging design problem. Our solution was to put in new concrete shear walls on the interior and at the lower level in various locations.



PLAN - BIRMINGHAM HOSPITAL
Figure 18.9

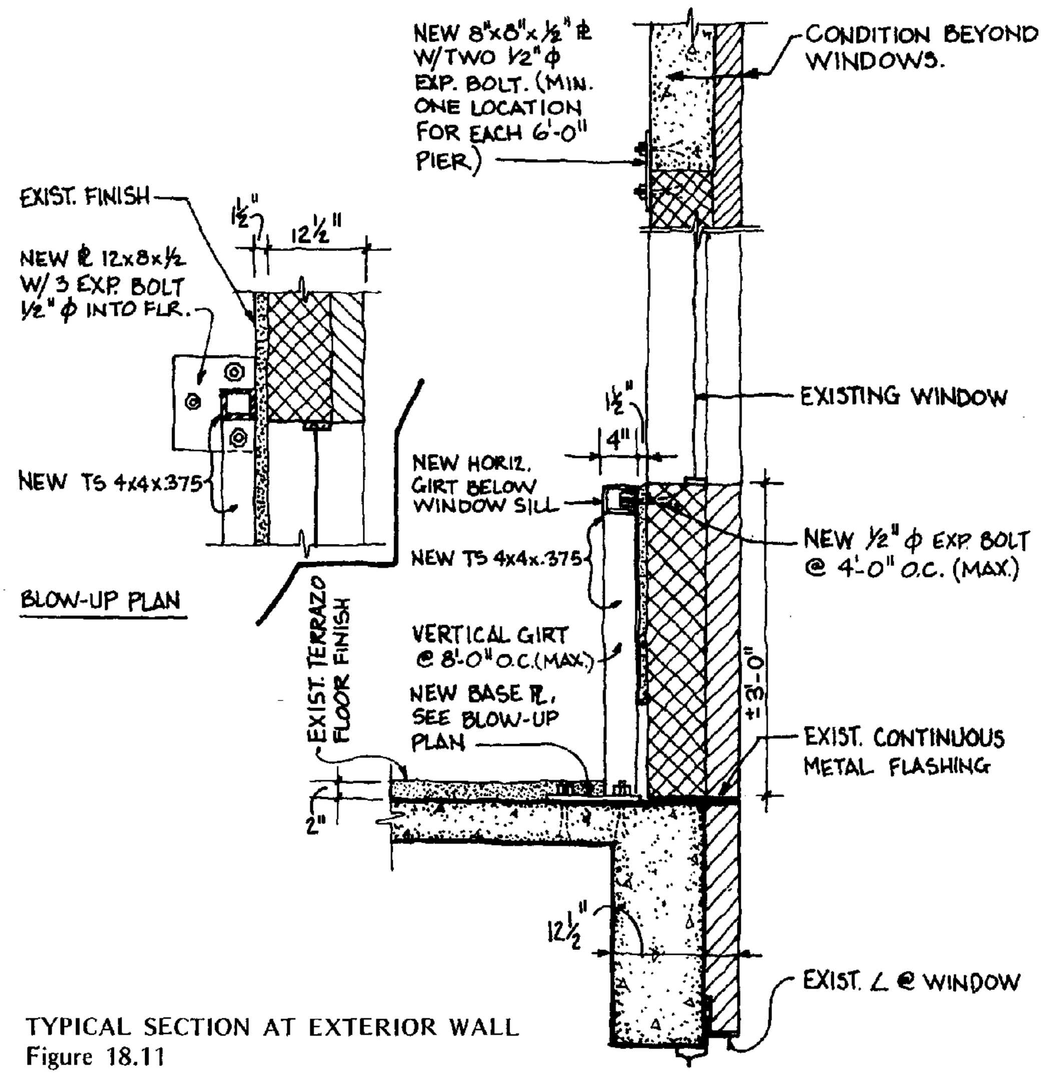
To combat overturning where we had a caisson foundation, we proposed the use of rock anchors. Then, going up the building, we reduced the number and length of the walls.

There can be problems putting in shear walls within existing buildings. Sometimes an additional chord element is needed, and another column has to be added inside, which means drilling into the existing column. This was not a cheap solution; it cost about $10.00 - 11.00 a square foot.



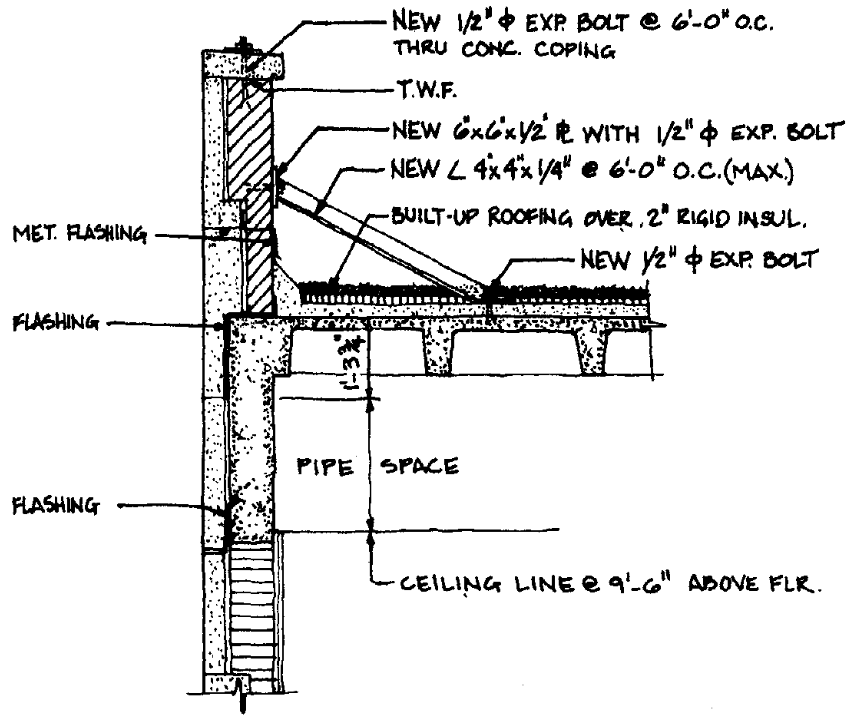
CONNECTION OF NEW SHEAR WALL WITH COLUMN
Figure 18.10

Figure 18.11 shows a typical exterior wall. It has through wall flashing at the base of the wall. Because it has continuous strip windows, we proposed a frame behind the wall along with a filler pan at about the center of the bay. There were some vertical tubes at the center of the bay to make a frame that would carry the whole wall together.



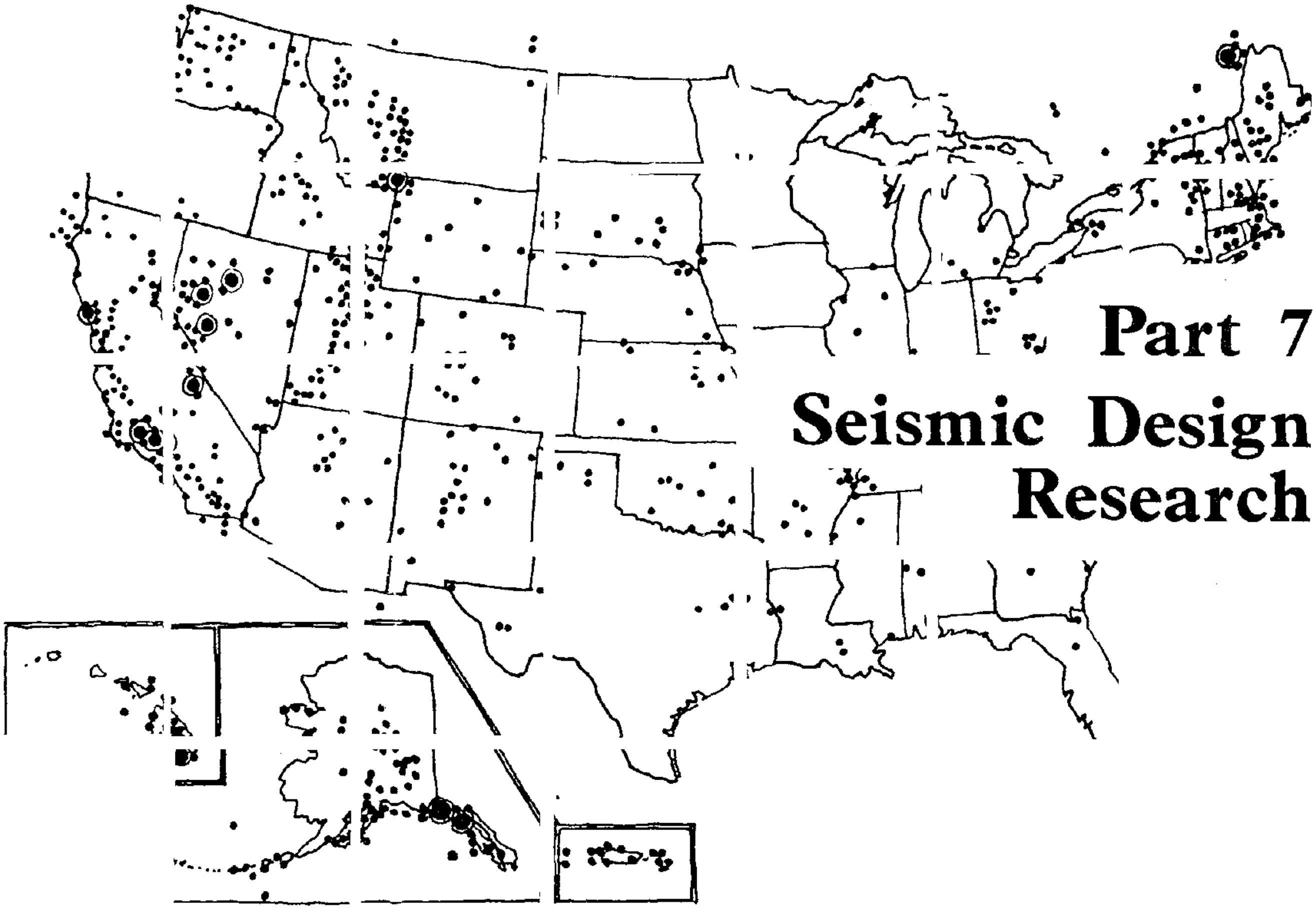
TYPICAL SECTION AT EXTERIOR WALL
Figure 18.11

This building also had high parapet walls. Here, too, there was through wall flashing, so we proposed steel braces. This is what is being used in the Los Angeles City schools. What is important in the parapet, is that there is now some kind of working triangle instead of a collapse mechanism.



TYPICAL PARAPET STRENGTHENING DETAIL
Figure 18.12

In summary, what I would really like to get across is that I think in the retrofit of buildings, the important thing is to tie the building together. I do not think the force used is so important as tying all the elements together to provide a way for loads to disperse laterally. When an earthquake occurs, forces that are five times what the building was designed for are possible; therefore, there must be a way for the lateral loads to be handled by the building. Unfortunately, there are a lot of buildings today that cannot handle such loads, with the obvious potential for costly and tragic results.



Part 7
Seismic Design Research

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Chapter 19. THE NSF EARTHQUAKE HAZARDS MITIGATION PROGRAM

Dr. Frederick Krimgold

INTRODUCTION

The Earthquake Hazards Mitigation Program of the National Science Foundation is part of the Problem Focused Research Division of the Directorate for Engineering and Applied Science. Earthquake engineering research was originally initiated in the Engineering Division of the Foundation in the early 1960's. With the occurrence of important damaging earthquakes in Alaska (1964) and San Fernando, California (1971) research interest and the national importance of earthquake mitigation research increased.

In the fall of 1977, the President signed into law the Earthquake Hazards Reduction Act of 1977.

The purpose of the Act is to reduce the risks of life and property from future earthquakes in the United States through the establishment and maintenance of an effective earthquake hazards reduction program. The objectives of the program include the development of technologically and economically feasible design and construction methods and procedures to make new and existing structures, in areas of seismic risk, earthquake resistant. Emphasis is placed on the development of research on a) ways to increase the use of existing scientific and engineering knowledge to mitigate earthquake hazards; b) the social, economic, legal, and political consequences of earthquake prediction and; c) ways to assure the availability of earthquake insurance.

The two agencies of the Federal Government which have had major responsibility for the research component of the Earthquake Hazards Mitigation Program have been the National Science Foundation and the United States Geological Survey. The U.S. Geological Survey has had primary responsibility for the basic geological and seismological research relating to hazard mapping and earthquake prediction. The National Science Foundation has had primary responsibility for earthquake engineering research and research on societal response to natural hazards.

The Earthquake Hazards Reduction Act includes research elements relating to development of information and guidelines for zoning land in light of seismic risk in all parts of the United States, and preparation of seismic risk analysis useful for emergency planning and community preparedness. It also specifically emphasizes research for development of methods for planning, design, construction, rehabilitation, and utilization of man-made works so as to effectively resist the hazards imposed by earthquakes. In the area of social science research, the Act calls for exploration of possible social and economic adjustments that could be made to reduce earthquake vulnerability and to effectively exploit existing and developing earthquake mitigation techniques.

The Act has provided the basis for a significant expansion of earthquake research by the Federal Government. A very important part of this expansion has been the extension of topic areas for research to include architectural and planning research. The NSF program is now structured under two headings: "Engineering, Architecture and Urban Planning" and "Societal Response."

The Engineering Architecture and Urban Planning Elements in subdivided into fifteen subtopics as follows:

1. Analysis of destructive earthquake ground motions.
2. Instrumentation for strong earchquake ground motion.
3. Geotechnical earthquake engineering.
4. Analytical methods for structural response.
5. Structural properties for experimental tests.
6. Existing hazardous buildings.
7. Non-structural and architectural systems.
8. Architectural and planning influences on earthquake vulnerability.
9. Seismic effects on coastal and inland waterways.
10. Seismic effects on lifeline facilities.
11. Post-disaster earthquake studies.
12. International cooperative research.
13. Information transfer.
14. Dam Safety.
15. Earthquake effects in relation to other natural hazards.

Of these subtopics, six are of particular relevance for Architecture and Planning researchers:

EXISTING HAZARDOUS BUILDINGS

The greatest threat to life safety in earthquakes is associated with the existed failure of those structures which predate the establishment of modern aseismic regulations. Many of the residential and commercial buildings in older downtown areas of cities on the west coast such as Los Angeles, San Francisco, and Seattle are considered highly vulnerable to earthquake shaking because the first regulatory approaches to aseismic construction date only from the 1930's. East of the Rocky Mountains, the existing building problem is even greater because, though the level of seismic activity is relatively lower than in the west, actions to improve construction for earthquake resistance have only been initiated in the past decade. The strong trends toward historic preservation and recycling of existing structures stand in direct conflict with the ambition of earthquake hazard reduction. Considerable research is necessary to resolve this conflict and to develop safe and economical retrofit techniques.

NON-STRUCTURAL AND ARCHITECTURAL SYSTEMS

The greater bulk of earthquake engineering research in the past has been directed to avoidance of structural failure of collapse as this was considered the major treat of life safety. However, it has been demonstrated in recent earthquakes that major economic loss has been incurred as well as severe injury in buildings which did not suffer significant structural failure. In the San Fernando Earthquake, it has been estimated that the greatest dollar value of damage was suffered in non-structured elements of buildings. These finds indicate that considerable research must be directed to reducing the failure probability of building components, electrical, mechanical and architectural systems.

ARCHITECTURAL AND PLANNING INFLUENCES ON EARTHQUAKE VULNERABILITY

The disciplines of architecture and planning combine understanding of the physical and natural science aspects of construction and resource management as well as the social and behavioural aspects of occupants and users.

Architectural configuration must accommodate both the intended socially determined uses of buildings as well as the physically determined structural system. As these objectives often appear in competition for limited resources, means need to be developed for guiding design decisions to provide maximum safety at feasible cost. At the larger scale of urban and regional planning, methods must be developed for introducing geotechnical information in to land use management and to the direction of development pattern to minimize further earthquake exposure.

SEISMIC EFFECTS ON LIFELINE FACILITIES

Concern for seismic effects on lifelines facilities has expanded significantly in recent years. While early work in earthquake hazards mitigation was confined to the avoidance of structural failure in individual buildings, it is now recognized that the secondary effects of urban systems failures may be as great or greater than direct losses due to earthquake shaking. Fire following earthquake in the absence of water and power supply systems poses a very serious problem. Lifeline facilities by their nature as extended network systems suffer a high probability of experiencing earthquake effects. Furthermore, it is the nature of network systems that the failure of a single element will impair the function of the entire system. Particularly important in the evaluation of seismic risk in lifeline systems is the understanding of expected social and economic impacts of failure. Such evaluations require a thorough understanding of the complex interaction of urban systems and the social consequences of systems failure.

LEARNING FROM EARTHQUAKES

The analysis of damage following earthquakes has provided in most important data for improvement of understanding of earthquake effects and structural failure patterns. The structural engineering research community has made extensive studies of earthquake damage around the world. To date, however, there has been only very limited participation by architectural and planning researchers. Post earthquake damage analysis is possibly the best source of insight into the patterns and mechanisms of non-structural failure. It also provided an understanding of how systems interact in the real world where materials and construction techniques may vary considerably from those of the laboratory. Architects and planners working in conjunction with social science researchers should also be able to develop a better understanding of the social significance of damage impacts.

INTERNATIONAL COOPERATIVE RESEARCH

Fortunately, earthquakes are relatively infrequent in any particular country. However, this makes them difficult to study. In order to accumulate adequate data earthquake researchers must utilize an international network of data collection. As much as possible in the way of new knowledge must be extracted from each earthquake experience. Bilateral research programs are currently being developed with Japan, China, and the Soviet Union. Efforts are also underway to facilitate the development of cooperative research efforts with researchers in Europe and the Middle East. Proposals are currently mitigation in developing countries.

For Architecture and Planning researchers these international cooperation research activities have added importance. Not only here in the United States is there the potential of significantly expanding earthquake mitigation policy approaches developed in other countries. There is a general benefit to the derived from the establishment of cooperative research agreements between planning and design researchers in this country and their counterparts abroad.

The National Science Foundation Earthquake Program will receive unsolicited proposals at any time. The proposals should follow the format provided in the Guidelines for Preparation of Unsolicited Proposals. Proposals are subject to peer review and program evaluation. The processing of proposals generally require six to nine months.

For further information, please contact:

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