NSF/CEE-81064

Building Configuration and Seismic Design

The Architecture of Earthquake Resistance

BUILDING SYSTEMS DEVELOPMENT, INC. FUNDED UNDER A GRANT FROM THE NATIONAL SCIENCE FOUNDATION

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/CEE-81064	2.	3. Recipient's Accession No. PB82-158569

4. Title and Subtitle Building Configuration and Seismic Design: The Architecture of Earthquake Resistance	5. Report Date May 1981 6. 034005
7. Author(s) C. Arnold, R. Reitherman	8. Performing Organization Rept. No.
9. Performing Organization Name and Address Building Systems Development, Inc. San Mateo, CA 94402	10. Project/Task/Work Unit No. 11. Contract(C) or Grant(G) No. (C) (G) PFR7681821
12. Sponsoring Organization Name and Address F. Krimgold, CEE Directorate for Engineering (ENG) National Science Foundation Washington, DC 20550	13. Type of Report & Period Covered 14.

15. Supplementary Notes
Submitted by: Communications Program (OPRM)
National Science Foundation
Washington, DC 20550

16. Abstract (Limit: 200 words)
A study was undertaken to determine how the architecture of a building affects its ability to withstand earthquakes and to provide information that will lead toward good practice in seismic design. Discussed are aspects of ground motion which are significant to building behavior. Provided are results of a survey of configuration decisions that affect the performance of buildings with a focus on the architectural aspects of configuration design. Configuration derivation, building type as it relates to seismic design, and seismic issues in the design process are examined. Case studies of the Veterans Administration Hospital in Loma Linda, California, and the Imperial Hotel in Tokyo, Japan, are presented. Both studies describe the seismic design process, paying special attention to the configuration issues. The need is stressed for guidelines, codes, and regulations to ensure design solutions that respect and balance the full range of architectural, engineering, and material influences on seismic hazards.

17. Document Analysis a. Descriptors

Hazards	Architecture	Design
Building codes	Earthquakes	Structural design
Earthquake resistant structures	Dynamic loads	Structural engineering

b. Identifiers/Open-Ended Terms
Loma Linda (California)
Tokyo (Japan)
Ground motion
C. Arnold, /PI

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS	20. Security Class (This Page)	22. Price

(See ANSI–Z39.18) *See Instructions on Reverse* OPTIONAL FORM 272 (4–77) (Formerly NTIS–35) Department of Commerce

Building Configuration and Seismic Design

The Architecture of Earthquake Resistance

Christopher Arnold and Robert Reitherman

May 1981

A study conducted by Building Systems Development, Inc.,
San Mateo, California
under a grant from the National Science Foundation,
Washington, D.C.

Structural Consultation by Eric Elsesser

Edited and produced by Dianne Whitaker

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Acknowledgments

This study was developed with the financial support of the National Science Foundation under Grant No. ENV 76-81821.

The support of Dr. Charles Thiel and Dr. John Scalzi was much appreciated at the inception of the study; and the encouragement and advice of Dr. Fred Krimgold, program manager for the Foundation, was a continuing assistance.

Valuable advice and encouragement was given by our Review Committee; Joseph Esherick, Gerald McCue, Steve Johnston, and Alexander Tarics. In addition, drafts of the study were read and commented upon by Clarkson Pinkham, Peter Yanev, John Brown, Robert Olson, Robert Nason, Henry Lagorio, Mike Agbabian, and Richard Bender.

Christopher Arnold was Principal Investigator of the study: the report writing was shared with Robert Reitherman, who also contributed many of the graphics, and developed the massive bibliography. His research into the Imperial Hotel, Tokyo, appeared in slightly different form in a paper for the Seventh World Conference on Earthquake Engineering, Istanbul, 1980.

Dianne Whitaker contributed many of the graphics, and was also responsible for the immense task of final editing and production. Rich Newman of Graphic Communications, San Francisco, acted as graphic design consultant. The report was typed by Nancy Hutcheson, more times than any of us can remember.

Our structural consultant, Eric Elsesser, was responsible for the initial suggestion of the subject of this investigation, and was very influential throughout the whole study, but should not be held responsible for any errors that may have crept into our structural thinking.

Engineering information on Loma Linda Hospital in Chapter XIII is based extensively on a paper written by William Holmes, project engineer for Rutherford and Chekene, who also provided many insights to the Principal Investigator during the development of that project.

We would also like to acknowledge the willing and efficient help of Aileen Donovan, librarian, and the staff of the Earthquake Engineering Research Center Library of U.C. Berkeley.

Finally, we would like to recognize the architects whose unending efforts to be different have, over the years, ensured the almost limitless number of building configurations without which this study would not have been possible.

Preface

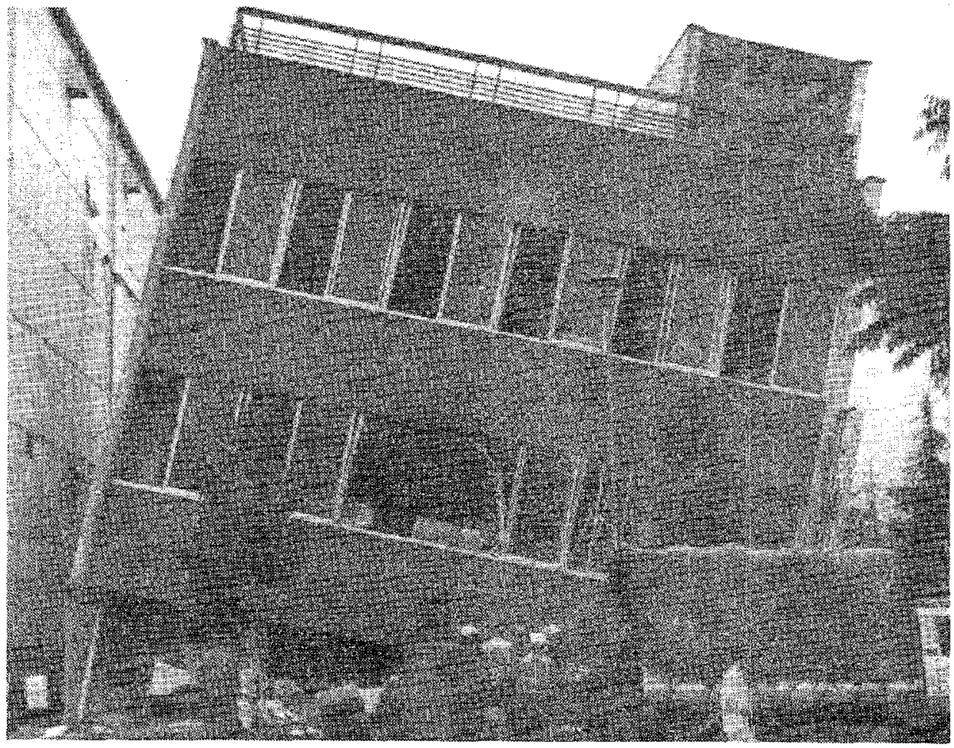
The idea has long been accepted by engineers that building configuration - the size and shape of a building and its component elements - has a significant effect on its behavior in earthquakes, and the principles that govern this behavior are well understood. However, application of these principles has not been effective, and we continue to see the use of hazardous and expensive configurations in seismic areas. There are a number of reasons for this. Some architects are not aware of the seismic importance of their design decisions, and do not seek the advice of their engineers. Some may obtain good advice but, for whatever reason choose to ignore it. And some may, in good faith, seek guidance from their engineers but not receive clear and forceful counsel: not all engineers are skilled in the arts of explanation. Finally, since the engineer is generally employed by the architect, he may feel inhibited in giving advice that is construed as a constraint or a criticism of the creative genius of his employer. So perhaps the failure in application of these important principles lies in the inability of the two professions to talk to one another and in their contractual relationship.

The considerable body of information that exists as to the influence of configuration, tends to take the form either of comments in research papers directed towards larger issues, or as part of the informal lore of practicing structural engineers. The tendency is for this information never to reach the practicing architect, or to reach him in a form that emphasizes design restriction rather than developing the understanding out of which the capable designer creates innovative solutions. Seismic engineering textbooks tend towards mathematical analysis rather than the evaluation of design concepts, and the configuration issue is emphasized and then dismissed in a page or so of constraining directives.

The purpose of this study is not further to constrain the designer's freedom, but to fill the void that exists by making clear, in simple non-mathematical terms, the way in which the architecture of a building affects its ability to withstand earthquakes, and to provide information that will lead the designer towards good practice in seismic design. This study cannot replace the advice and cooperation of the engineer: but by improving the architect's understanding of the seismic problem it can make the joint efforts of architect and engineer more effective. In addition, it provides some information intended to familiarize the engineer with the architect's requirements, and so is addressed to a dual audience.

Information is presented in conceptual rather than analytical form because that represents the designer's way of thinking. In its concern for clear methods of exposition and the extensive use of graphics, this study is quite different from the traditional research study, which is addressed only to a few colleagues who share a special language. The study starts with a discussion of those aspects of ground motion which are significant to building behavior and then explains the ways in which buildings react to this motion. The discussion then moves into a survey of configuration decisions that affect the performance of the building: a focus on the architectural aspects of seismic design. Further sections then review common configuration problems and their solutions, recognizing that because of the many influences on configuration, it may be necessary to accept a seismic configuration that is less than optimal. The following three chapters are devoted to configuration derivation, building type as it relates to seismic design, and seismic issues in the design process. Two case studies of significance to the configuration topic are then presented, followed by discussions of seismic imagery, past and present, and the conclusions of the study. The definition of configuration forms the subject of the first appendix, and a second appendix provides a bibliography.

The general purpose of this study, then, is to act as a bridge between the activities of the researcher and the design professional, and to identify, organize, and present in a useful form information which is presently scattered and difficult to find.



I. Introduction

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A. The Growth of Knowledge

A look at a world seismicity map (Figure I-1) reveals the truly global incidence of earthquakes. Except for Northern Europe, most of the other regions of very low seismicity are largely uninhabited: Greenland, Siberia, Northern Canada, most of Australia, the Amazon basin, the Sahara, and Antarctica.

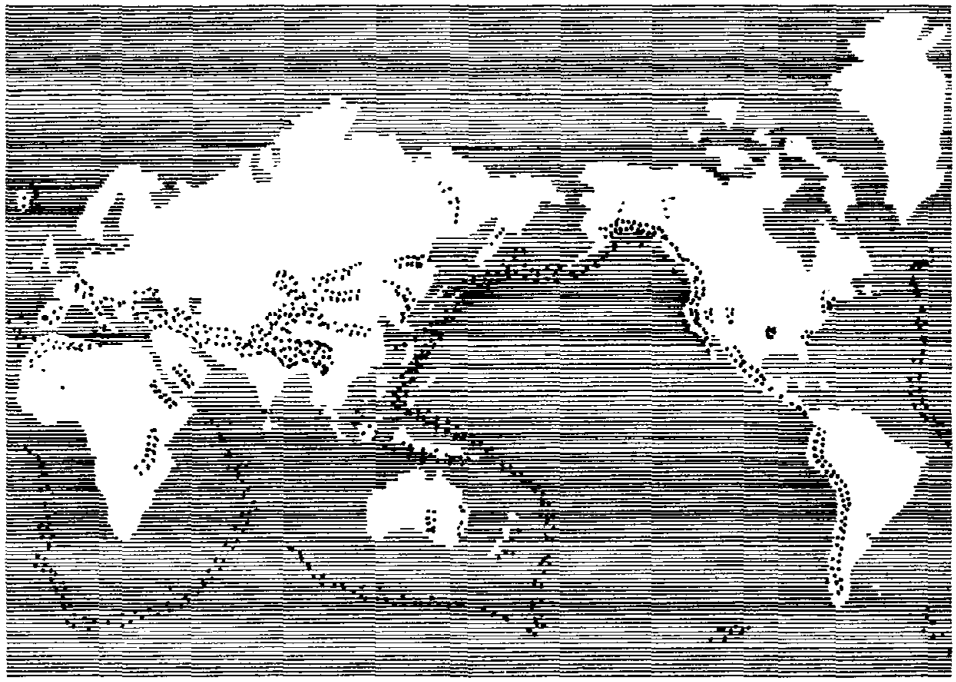

Figure I-1. World seismicity map showing distribution of mid 20th Century seismic events.

Most of the great historic architectural styles have occurred in seismic regions, but it does not seem that the architecture that evolved in these areas consciously expresses seismic design principles. The seismic factor has not induced the evolution of indigenous construction forms as sophisticated and effective as the large variety of climatically-determined responses with which we are now familiar.

Much of the explanation for this lies in the fact that the physical characteristics of earthquakes have not been well understood until this century. Prior to this time earthquakes tended to be regarded as acts of God, and proper mitigation measures consisted of prayer rather than architecture and engineering. By the time of the 1906 San Francisco earthquake, the idea that the design of buildings would affect their seismic performance was beginning to be established, and by the 1923 Tokyo earthquake seismic design principles, based primarily on empirical methods, were firmly established. Following the Tokyo disaster the study of earthquake engineering occupied some of the best engineering and geological minds in the world and analytical methods became highly developed.

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In the United States research into seismic design was concentrated in California, spurred on by periodic events - Santa Barbara 1925, Long Beach 1933, Kern County 1952 - that reminded the public and responsible officials of the seriousness of the problem without, fortunately, providing the impetus of a major disaster. Research was done by a small group of dedicated engineers, supported almost entirely from their own resources, and the occasional large project

Figure I-2. Olive View Hospital following the 1971 San Fernando California earthquake.

that could justify some original investigation. This state of affairs began to change following the Alaska earthquake of 1964 which resulted in major calls on Federal funds to make good the damage. Important studies were made on the effects of this earthquake, supported by Federal and State money.

The San Fernando earthquake of 1971 dramatically changed the picture. Although not a major disaster, severe damage to new buildings designed in accord with prevailing codes (Figure I-2) created major concern for the entire basis of U.S. seismic design and resulted, among other things, in the intensification of national Federally supported programs of earthquake mitigation research.

The study of seismic design has become a major national concern, and as the research continues, the level of seismic design practice is becoming more sophisticated. This has the effect of raising the level of professional responsibility and expanding the knowledge of which the professional is expected to be cognizant. Simultaneously with this change in the range of knowledge has come a revised conception of professional responsibility and liability which is affecting all design areas. As design and construction become increasingly institutionalized, the user demands protection from random hazards of life safety or even discomfort over which he or she has no control. That control rests with the institutions and the professionals and the courts are increasingly reinforcing this view of responsibility.

If the protection from the effects of earthquakes could be segregated and left safely at the door of the engineer, there would be little cause for the architect to show concern for seismic phenomena. Indeed, a pattern of relative unconcern tends to characterize the architectural profession, even in heavily earthquake prone areas such as California. This unconcern stems from several perceptions on the architect's part: one is that seismic design is indeed an engineering exercise which can be added to the architect's concept with little need for concern on his part. This perception is aided by the engineer's tendency often to encourage the architect in this thinking because it enhances the engineer's mystique - his ability to deal with mysterious forces of which the architect is ignorant. As often as the engineer will complain of the seismic design difficulties that the architect's design causes him, at the same time he welcomes the business opportunity created by the wizardry with which he enables the architect to still safely pursue his design images. The architect, in his ignorance, is a nuisance to some engineers, and a source of revenue to others.

But what does the architect do, in his ignorance, that influences the seismic performance of the building, and is a source of variously expressed concerns by the engineer?

B. Configuration Definition

The answer is that the architect conceives and controls the configuration of the building. For this study configuration is defined as building size and shape, but also includes the nature, size and location of the structural elements, and the nature, size and location of non-structural elements than may affect structural performance (Figure I-3). These include such elements as walls, columns, floors, service cores and staircases, and also the quantity and type of interior partitions and the ways in which the exterior wall is left solid or perforated for light and air. This extended definition of configuration is necessary because of the intricate relationship for seismic performance between these three groups of elements of configuration. In this, our definition extends beyond the idea of building form, which tends to limit itself to overall shape or the nature of the building as a sculptural mass. An approach to configuration definition for seismic design is described in Appendix 1.

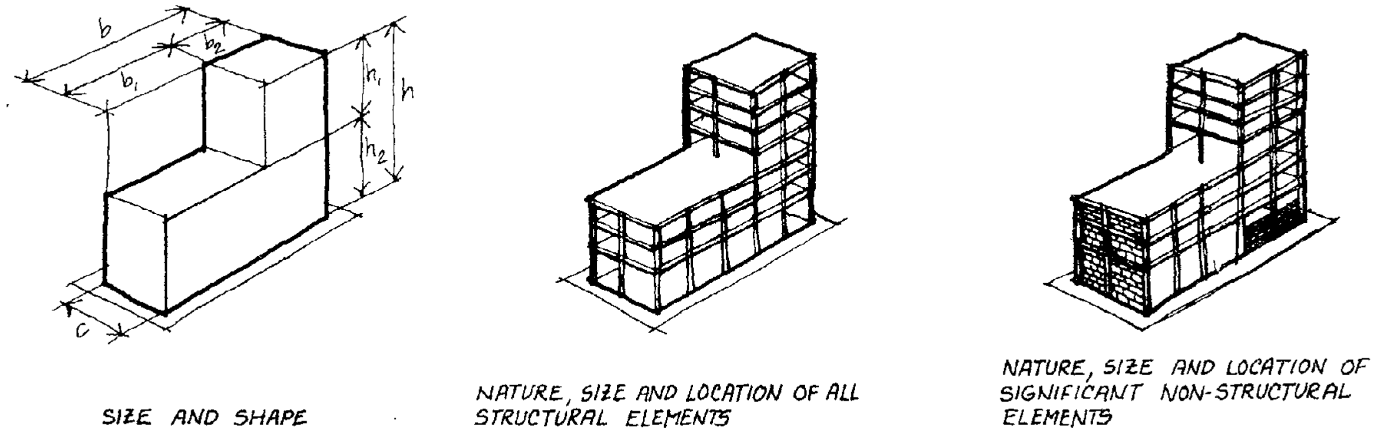

Figure I-3. The extended definition of configuration adopted for this study addresses three distinct issues.

C. Configuration Determinants

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 kinds of space division, connected by a circulation pattern; the combinations of activity spaces and circulation lead to certain dimensions and finally into a building configuration. But there are other determinants of configuration which sometimes may dominate: such things as the geometry, geology and climate at the site, urban design requirements, and architectural stylistic concerns. The final configuration choice is the result of a decision-process which by some means, balances these varying requirements and influences, and resolves conflicts into a single result. The process of configuration derivation is considered more fully in Chapter X.

D. The Importance of Configuration

In conceiving the building configuration the architect influences, or even determines, the kinds of resistance systems that can be used and even the extent to which they will, in the broadest sense, be effective. Further, many failures of engineering detail which result in severe damage or collapse, originate as failures of configuration. In other words the configuration of the building, either as a whole or in detail, is such that seismic forces place intolerable stress on some specific structural material or connection and it fails.

This is not to suggest that configuration is primary, and detailed engineering design and construction techniques secondary or of no consequence: they are obviously related as contributors to the safety and efficiency of the building. But it does mean that the designer's first ideas on configuration are very important, because at a very conceptual stage, perhaps even before there is any engineering discussion, he is making decisions of great significance to later engineering analysis and detail design.

Seismic design, then, is a shared architectural and engineering responsibility. The earthquake attacks the building as a whole, and does not distinguish between those elements conceived by the architect and those devised by the engineer. The architect is a full participant in seismic design, and the importance of configuration has long been recognized by engineers who have studied the behavior of buildings in earthquakes.

Indeed, as the study of building behavior continues, and more empirical data are obtained, the importance of this issue is being increasingly stressed. A recent publication providing directives for the design of buildings for the armed services states (1):

"A great deal of a building's inherent resistance to lateral forces is determined by its basic plan layout...

"Engineers are learning that a building's shape, symmetry, and its general layout developed in the conceptual stage are more important, or make for greater differences than the accurate determination of the code-prescribed forces..."

Structural engineer William Holmes, writing in 1976 (2):

"It has long been acknowledged that the configuration, and the simplicity and directness of the seismic resistance system of a structure is just as important, if not more important, than the actual lateral design forces."

Henry Degenkolb is emphatic in stressing the importance of configuration, but also recognizes that seismic design is but one of many influences on the shape of the building (3).

"If we have a poor configuration to start with, all the engineer can do is to provide a band-aid - improve a basically poor solution as best he can. Conversely, if we start off with a good configuration and a reasonable framing scheme, even a poor engineer can't harm its ultimate performance too much. This last statement is only slightly exaggerated. Much of the problem would be solved if all structures were of regular shape, but economics of lot sizes and arrangements, various planning requirements for efficient use of space, and aesthetically pleasing proportions require the structural engineer to provide for safe constructions of various shapes."

The nature of the problem has been well stated by the Nicaraguan architect Jose Francisco Teran, who studied the effects of the Managua earthquake of 1972 (4).

"The question arises as to whether the building should be designed to meet the functional, social, and aesthetic needs and then be implemented for structural safety or if in seismic areas like Managua, the special problems of stability and overall integrity should condition the design process by which the elements of form such as mass, symmetry, modulation, etc., are decided.

"If we agree that such is the case, how can architects, engineers, owners, and the whole community develop a common design attitude for a phenomenon that occurs critically at considerable time intervals during which many of the design parameters actually change. Besides, in contrast with the automobile, the ship, and the airplane that are designed primarily to be in motion during their functioning periods, buildings are designed to be static but may be subjected to short dangerous periods of violent motions... The more simple, continuous, symmetrical, straightforward and repetitive the solutions the greater will also be the degree of reliability of the motionless structures in which we live and work when they become attacked by seismic motions."

Teran makes two important points: that certain areas in the world demand a rearrangement of the determinants of the design process, so that the rare yet dangerous possibility of seismic forces becomes a contextual issue, within which other design decisions can be made. His other comment could serve as a design directive for work in seismic zones: that solutions should be "simple, continuous, symmetrical, straightforward and repetitive." Note that this directive is expressed not as an absolute, but as a qualitative factor which influences the reliability of the structure. The plea is for understanding and knowledge, not the imposition of mandatory constraints.

E. Configuration and the Code

One final issue: most countries have institutionalized the solution of building problems of life and safety in the form of a code that mandates safe standards for design and construction. How do our building codes deal with the configuration and seismic design issue?

In the United States, until the 1973 edition of the Uniform Building Code, configuration was not dealt with in a specific clause at all, and at present it treats the issue only with a general caveat (5).

"Structures having irregular shapes or framing systems: The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure."

If the subject is important, why does the code treat it in only a general and suggestive way? The problem seems to be that although engineers involved in the seismic field have long recognized that configuration is a key issue, it had been found too difficult to reduce to the relatively simple set of prescriptive rules that is our typical code format. This difficulty is explained in the commentary portion of the Structural Engineers Association of California (SEAOC) Recommended Lateral Force Requirements And Commentary (1975) (6).

"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."

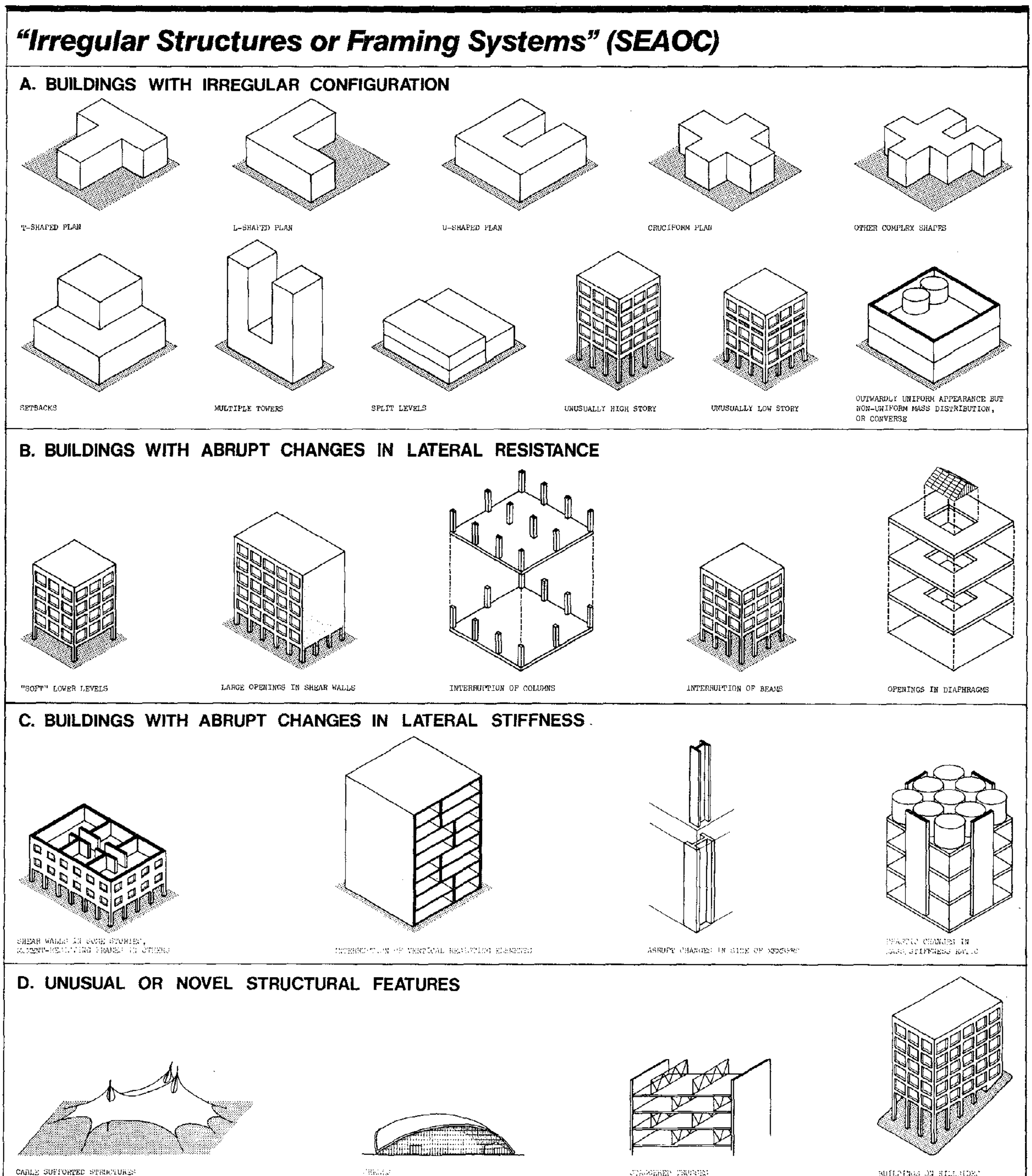

Figure I-4. Graphic interpretation of "irregular structures or framing systems" from the Commentary to the <u>SEAOC Recommended Lateral Force Requirements And Commentary</u>.

SEAOC has produced updated editions of the Recommended Lateral Force Requirements And Commentary since 1959. The "Requirements" of these documents has been adopted almost verbatim into successive editions of the Uniform Building Code, but the Commentary section has not. In this section are listed over twenty specific types of "irregular structures or framing systems" as examples of designs which should involve extra analysis and dynamic consideration rather than use of the normal equivalent static force method. These are illustrated in Figure I-4, which is a graphic interpretation of the SEAOC list.

Scrutiny of these conditions will show that the majority of irregularities are configuration issues within the terms of our definition. Further, our inspections have shown that somewhere between 65-80% of buildings built in the last fifteen years by a typical major national contractor (Figure I-5) fall into one or more of these irregular categories: the percentage range allows for relative judgements in subjectively allocating designs to a category.

Figure I-5. Representative montage of buildings constructed by a major National contractor in 1977. Note irregular forms.

Cursory examination of the plan forms of the ten largest buildings on eight University of California campuses, a sampling which includes residential, office, library, educational, and laboratory buildings, shows that 75% have major irregularities, and on several campuses all ten largest buildings are irregular. It is important to emphasize that these irregularities are ones created by the designers, not by problems intrinsic to the buildings themselves.

It is safe to say that well over half the buildings that have been designed recently do not conform to the simple uniform building configuration upon which the code is based and hence, to a greater or lesser extent, the code forces are inapplicable. The simple equivalent static force method of the code must be augmented by engineering experience and judgement, perhaps combined with a full dynamic analysis. While major building projects will have careful engineering conception and analysis there remain many irregular buildings designed in bare adherence to the code in which, for reasons of cost or ignorance, the modification of seismic performance created by configuration irregularities may not have been carefully considered and accommodated in the design.

A recent review of the purpose of seismic codes provides the historical context necessary to understand the limitations of the code's provisions (7).

"In the 1930's, 40's, and 50's the structural engineers of California (with recognition of the experiences of Japanese engineers) generated the basic earthquake code and design procedures which are employed through-out the world today. It is most important to recognize that these engineers had developed these provisions for the types of building construction which were prevalent in California at that time - specifically structures in Los Angeles and San Francisco. These buildings typically had strong steel (with concrete fireproofing) framing skeletons, filled with very well-constructed brick masonry walls and strong concrete flooring systems. They were usually symmetrical and regular in their configuration, and in most cases they qualified as good tough earthquake resistant structures. It is a most educational experience to walk along Market Street in San Francisco and see some of these structures that survived the motion effects of the disasterous 1906 Earthquake without even significant damage. The California engineers, having a knowledge of the good performance record of these structures, formulated the following type of design philosophy:

- relatively low lateral earthquake forces for the design of structural members.
- relatively strict rules governing the types of allowable materials, the methods of member design and tough connections, and an implied need for symmetry and regularity.

"For the time up to the 1960's, before which much construction in California did not differ substantially from that of the tough buildings, this philosophy was appropriate to provide seismic resistant structure.

"However, architectural configuration along with methods of construction have changed significantly in the past two decades. Frames have become much more open and irregular, and the rugged systems of masonry partition walls and concrete floors have been replaced by largely prefabricated elements with very flexible characteristics. The low seismic design forces which were quite appropriate for the classical old methods of construction were applied without change for the newer structures... The basic error was that the new buildings did not have the regularity, stiffness, and reserve toughness necessary to justify the classical low design values."

It is important to understand the basic philosophy of the code. The code's purpose is to prevent injuries to people, not to reduce damage to buildings, hence the focus of its provisions is to prevent structural collapse. In a large earthquake a building may suffer considerable structural and non-structural damage, but as long as the building does not collapse, the intent of the code has been met.

Hence, we design our buildings recognizing the probability of damage, for complete damage prevention is an unrealistic goal; damage control is the aim. To a large extent, configuration determines <u>where</u> the damage will occur - whether it will be controlled, distributed, and safely absorbed, or whether it will be accidentally concentrated in a way that can lead to catastrophic failure of critical elements.

The above gives some indication of the difficulties experienced in the attempt to codify the influence of configuration. In work currently in progress to develop a sophisticated national code for seismic design (8) these difficulties remain essentially unresolved. It is also clear that bare adherence to the code will not ensure that the influence of configuration has been addressed.

That a situation should prevail in which the basis upon which many buildings are designed is different from that upon which the seismic code is based, is a cause not for alarm but for understanding. Conceptual recognition of problems caused by configuration predates by many decades today's analytical study. Much of the information is empirical: early observers noted the behavior in earthquakes of buildings of certain types of material, construction, and configuration. But this situation emphasizes the danger for the designer of relying exclusively on the code provisions, and not also developing a conceptual understanding of the nature of the dynamic environment and the way in which the building responds.

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II. Ground Motion

A. The Nature of Ground Motion

The following description by Samuel Clemens, who experienced several earthquakes while in California in the 1860's, shows that an acute observer, even with no geologic training, can accurately describe the varied nature of ground motion in earthquakes (1).

"I have tried a good many of them here, and of several varieties, some that came in the form of a universal shiver; others that gave us two or three sudden upward heaves from below; others that swayed grandly and deliberately from side to side; and still others that came rolling and undulating beneath our feet like a great wave of the sea."

If one could hover motionless during an earthquake over a marked spot on the earth it might be seen to move erratically, tracing out a random path resembling that of the wandering of an insect. "Ground motion" is a literal description; the ground moves, to a maximum distance of about a foot, relative to its stationary starting point.

In 1887 Prof. Sekiya modelled in a three-dimensional wire diagram, the motion of a point in the earth during the first 20 seconds of an earthquake (Figure II-1). This was based on the seismogram of the Japanese earthquake on January 15, 1887. The model is made to scale about twelve and a half times greater than the actual earth movement. The actual amount of motion was about 0.29 inch, which is represented full size in the small diagram to the right of the main illustration. It is important to visualize the small size of characteristic earth motion, and to realize that it is the building reaction to this movement that causes the large structural displacements that ultimately lead to damage and failure.

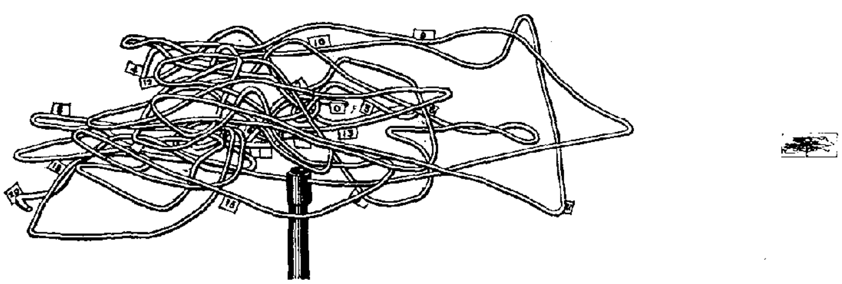

Figure II-1. Professor Sekiya's wire diagram of the motion of an earthquake, enlarged 12-1/2 times. The actual amount of motion is shown full size in the small diagram to the right.

The accompanying diagram (Figure II-2) of the scratch on a kitchen floor left by a kitchen range in the 1933 Long Beach earthquake is another example of the typical complexity of building movement caused by ground motions. However, it must be noted that the path of motion traced by an object is not necessarily a directly translatable record of the motion of the ground, for the object may not remain stationary in space while the ground moves beneath.

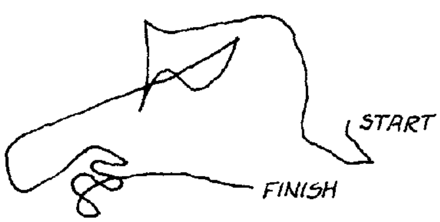

Figure II-2. Scratch left on the floor by a kitchen range in the 1933 Long Beach California earthquake.

Occasionally, the ground motion is predominantly back and forth in one general direction (such as in the 1978 Sendai or 1963 Skopje earthquakes) or is composed of a single shock, (as in the 1960 Agadir event) but this cannot be predicted. A skilled observer later to become eminent in the natural sciences, reported this fact.

Charles Darwin, having rounded South America's Cape Horn on the Beagle voyage, was in Chile during the 1835 earthquake. He noted that the damage in Concepcion suggested that ground motion was predominantly along one axis (2).

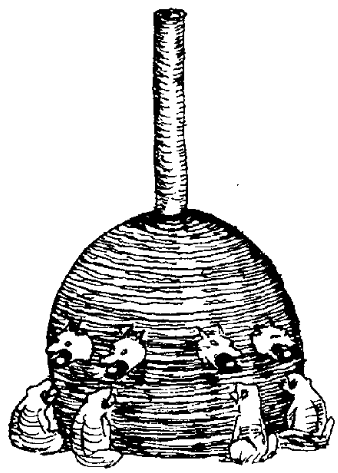

Figure II-3. Chinese seismoscope, 136 A.D.

"The town of Concepcion was built in the usual Spanish fashion, with all the streets running at right angles to each other; one set ranging S.W. by W., and the other set N.W. by N. The walls in the former direction certainly stood better than those in the latter: the greater number of the masses of brickwork were thrown down towards the N.E...."

Darwin was not the first to observe that ground motion is commonly more emphatic along some axes rather than others, for understanding of this phenomenon was reached very early in history. John Milne, one of the European scientists in Japan in the last half of the 1800's who helped lay the groundwork for Japan's subsequent prominence in seismic research, found a contemporary account of an ancient (136 A.D.) Chinese seismoscope (Figure II-3), which elegantly illustrates the point that inertial forces may be exerted in any direction (3).

"When an earthquake occurs, and the bottle is shaken, the dragon instantly drops the ball... With this arrangement, although one dragon may drop the ball, it is not necessary for the other seven dragons to drop their balls unless the movement has been in all directions; thus we can easily tell the direction of an earthquake."

Earthquakes are the result of slippage along a fault plane, often well below the surface of the earth. Geologists have various methods of determining the presence of fault planes and their characteristics. The presence of a fault indicates the possibility of an earthquake, though determination of its likelihood and size is still a very uncertain science. Geologists have a different sense of time from most of us: for them an active fault - which may be expected to cause an earthquake - is one that has moved in the last 10,000 years. In investigating faults related to nuclear facilities, active faults are defined as those that have moved in the last 500,000 years; a more cautious definition related to the potential danger of a nuclear facility.

Slippage along a fault line deep in the earth's surface may eventually result in "surface faulting," the crack or split on the earth's surface that provides the layman's vision of earthquakes. Surface faulting may result in large earth movements - perhaps several yards - and a building located across a surface fault is almost certain to suffer very severe damage however well it is designed. However, the probability of a building location straddling a line of surface rupture is relatively low compared to the probability of a building location that will be affected by ground motion caused by fault slippage.

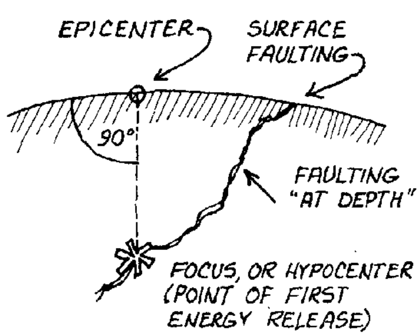

Figure II-4. Definitions used to describe the location of an earthquake.

The epicenter is the point on the earth's surface directly above where the faulting and energy release first begins (Figure II-4). Since the faulting plane is not necessarily exactly vertical, and since the fault may rupture along a considerable distance, shaking at the epicenter may not be the most intense, although it will almost certainly be among the more heavily shaken areas in a given earthquake.

The ground motion that is transmitted through the base of a building, then, has a random form, but sometimes an emphatic direction. The motion originates in four clearly defined types of waves

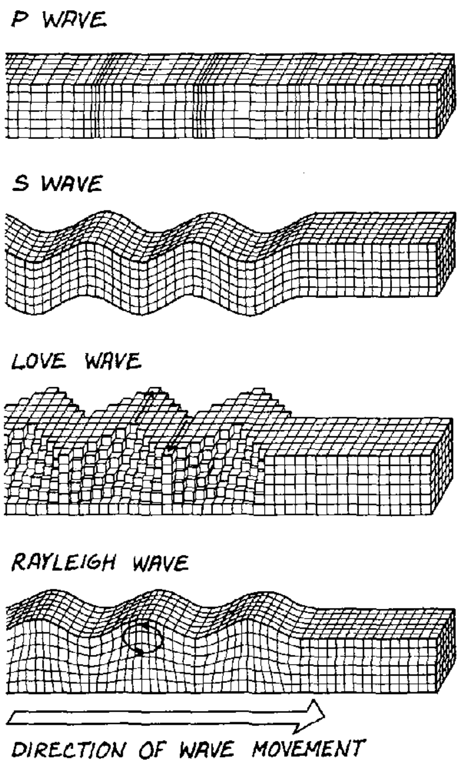

Figure II-5. Four types of earthquake waves created by a fault rupture.

Figure II-6. Liquefaction, 1964 Niigata Japan earthquake.

created by a fault rupture (Figure II-5). These are the primary, or P wave, which is the fastest, traveling at about 8km. per second, or 18,000 mph., and arrives first. It has the form of a sound wave that, as it spreads out, alternately pushes and pulls at the ground material. The second type of wave is the secondary or S wave; this shears the rock sideways at right angles to the direction of travel. The third type is a surface wave called the Love wave, that is similar to a secondary (S) wave with no vertical displacement; it moves the ground from side to side horizontally parallel to the earth's surface, at right angles to the direction of propagation, and produces horizontal shaking. The fourth type of wave, also a surface wave, is known as the Rayleigh wave; in this the disturbed material moves both vertically and horizontally in a vertical plane pointing in the direction in which the waves are traveling. Of the two surface waves, Love waves generally travel faster than Rayleigh.

Earthquake ground motions can be increased or decreased in amplitude or "size," and their rapidity of vibration or frequency can be varied, as the waves travel through various soil and rock layers and by topography as well. Relative amplifications of alluvial soil or San Francisco Bay mud compared to granite are approximately times 4 and times 9 respectively. From this brief conceptual description it becomes clear that, as is often the case with physical phenomena, although the constituent parts are well understood and readily analyzable, their interaction creates effects that appear random, complex, and somewhat unpredictable. Hence the motions that imitate the random wanderings of an insect.

The nature of the ground motion that affects the building can be summarized in a conceptual way as follows. The waves that create motion emanate from the line of fault rupture, and so approach the building from a given direction. The nature of the waves and their interactions is such that actual movement at the ground will be random: predominantly horizontal, often with some directional emphasis, and sometimes with a considerable vertical component. The actual horizontal ground displacement is small, generally measured in fractions of an inch, but in extreme cases the movement may be as much as a foot. These small displacements should be distinguished from displacements of surface fault rupture, which have been measured as large as 20 feet.

The other threatening type of movement of the ground, is the family of geologic hazards. Liquefaction is a condition in which the soil changes temporarily from a solid to a liquid state (Figure II-6). This effect is related to loose granular soils and sand and the presence of water, and hence tends to apply to sites located adjoining rivers, lakes, and bays. Engineering to mitigate the effects of liquefaction involves foundation design, or stabilization of the soil itself. Because of the uncertainties and costs of such measures, avoidance of sites with a potential for liquefaction represents the best design approach.

Landslides, or ground disturbance, can be triggered by earthquake ground motion. Tsunami ("tidal waves") are earthquake-caused wave movements in the ocean, and seiches are similar sloshing in closed lakes or bays.

B. The Measurement of Ground Motion

Ground motion is recorded by a seismograph, an instrument only in existence since the late nineteenth century. The Chinese seismoscope illustrated earlier showed only the principal direction of the earthquake, and did not record its history. The seismograph records the movement over time of a freely supported pendulum within a frame that is attached to the ground. Most of us have seen seismographs in museums and are familiar with the continuous ink trace on a rotating drum.

In modern seismographs, pendulum movement is converted into electronic signals on tape. Strong-motion seismographs, called accelerometers, are designed to record nearby rather than distant ground movement directly, and produce a record called an accelerogram. Instruments are normally placed so as to measure movements along the two horizontal axes and the one vertical. Three measures are of major interest: acceleration, velocity, and displacement. Acceleration is the rate of change of velocity: when multiplied by mass it results in the inertial force which the building must resist. Acceleration is commonly measured in g - the acceleration of a free falling body due to earth's gravity (approx. 32 ft/sec/sec., or 980 cm/sec/sec., or 980 gals, or 1.0g).

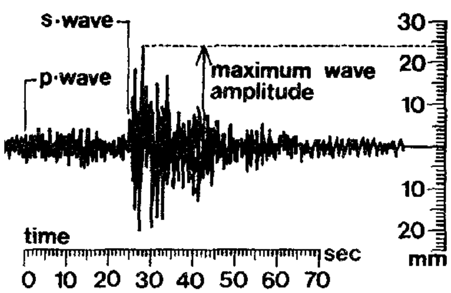

Figure II-7. The accelerogram.

Velocity, measured in inches or centimeters per second, refers to the rate of ground motion. Displacement, measured in inches or centimeters, refers to the distance a particle is removed from its at rest position.

The accelerograph provides a picture of the ground shaking; accurate interpretation of this picture requires skill and experience but the principles are illustrated in Figure II-7. In the accelerogram the arrival of the P wave begins the motion. This is followed by the S wave: the time interval between the two enables the distance from the instrument to the earthquake focus to be calculated. The duration of strong motion shows clearly, and the maximum wave amplitude can be measured directly. The ground acceleration can be calculated by relating amplitude to time. Velocity and displacement are obtained mathematically by integrating the acceleration record once and twice respectively.

The level of acceleration generally taken as sufficient to produce some damage to weak construction is 0.1g. The lower limit of acceleration perceptible to people is set by observation and experiment at approximately 0.001g or 1cm/sec.^{2} ; at between 0.1g and 0.2g most people will have difficulty keeping their footing and sickness symptoms may be induced.

An acceleration approaching 0.50g on the ground is very high. On upper floors of buildings, maximum accelerations will be higher, depending on the degree to which the mass and form of the building acts to damp the vibratory effects. A figure of 1.00g may be reached, or 100% of gravity; diagrammatically equivalent, in a static sense, to trying to design a building that projects horizontally from a vertical surface (Figure II-8). (When the behavior of real buildings is observed, it is seen that several factors modify this diagrammatic equivalence, and structures which could never cantilever from a vertical surface, can briefly withstand 1.0g earthquake shaking).

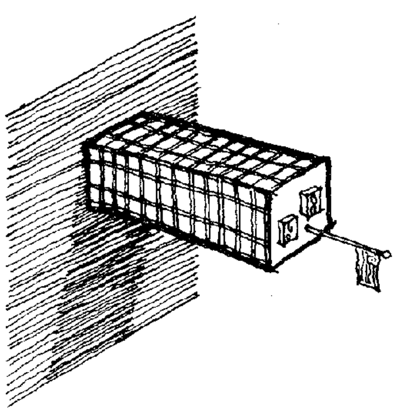
Figure II-8. 1.0g, or 100% of gravity, is equivalent to designing a building which cantilevers from a vertical surface.

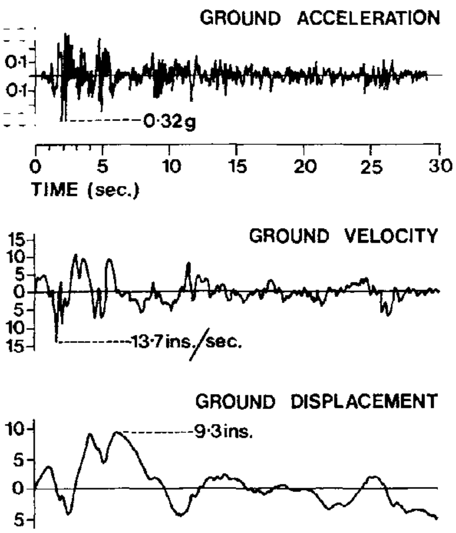

Figure II-9. The 1940 El Centro California earthquake.

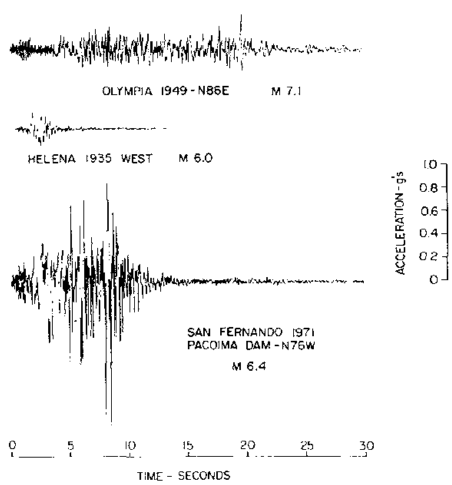

Figure II-10. Comparative accelerograms for several earthquakes. Note that all are to the same scale.

The measure of acceleration is commonly used to indicate the possible destructive power of an earthquake in relation to a building. A more significant measure is that of acceleration combined with duration. This is not hard to visualize intuitively, and it is important to understand that a number of cycles of moderate acceleration may be much more difficult to withstand than a single peak of much higher value. As will be discussed later, frequency is another major parameter of ground motion for design purposes. The instrumentation will also provide a measure of the duration of the strong motion.

The duration is thought to relate to the length of the fault break, and typically will occupy only a few seconds. The 1971 San Fernando earthquake only lasted a little over ten seconds, yet created much destruction. In 1906, San Francisco, the severe shaking lasted about 45 seconds; in Alaska in 1964 the earthquake was over 3 minutes. The record of the 1940 El Centro, California earthquake, for many years the best record available, showed strong motion continuing for approximately 25 seconds, with the major accelerations occurring for approximately 5 seconds (Figure II-9). This earthquake recorded a maximum acceleration of 0.32g, a maximum ground velocity of 13.7 in./sec. and a maximum ground displacement of 9.3 inches. Comparative accelerograms for a number of earthquakes are shown in Figure II-10.

For building design, we are interested in a number of aspects of the measurement of earthquakes. We need a way of comparing one historic earthquake to another. We need to be able to estimate the characteristics of probable ground shaking of a future earthquake, and to be able to relate it to a known historic event so that, by analogy, we can estimate forces and damage. Two earthquake measurement systems are in common use: neither, for various reasons, is really satisfactory to us from the building design viewpoint.

Earthquake magnitude is the first measure: it is expressed as a Richter Magnitude based on the scale devised by Prof. Charles Richter of California Institute of Technology in 1935. Richter selected the term magnitude by analogy with the corresponding astronomical usage for an absolute scale of star brightness independent of the location of the recording station. Richter's scale is based on the maximum amplitude of certain seismic waves recorded on a standard seismograph at a distance of 100 kilometers from the earthquake epicenter. Note that the scale tells us nothing about duration or frequency, which may be of great significance in causing damage.

Because, of course, the instrument will seldom be exactly 100km from the source, Richter developed a method for allowing for the diminishing of the wave amplitude record with increase of distance (just as the light of a star appears dimmer with distance). This method is shown graphically in Figure II-11.

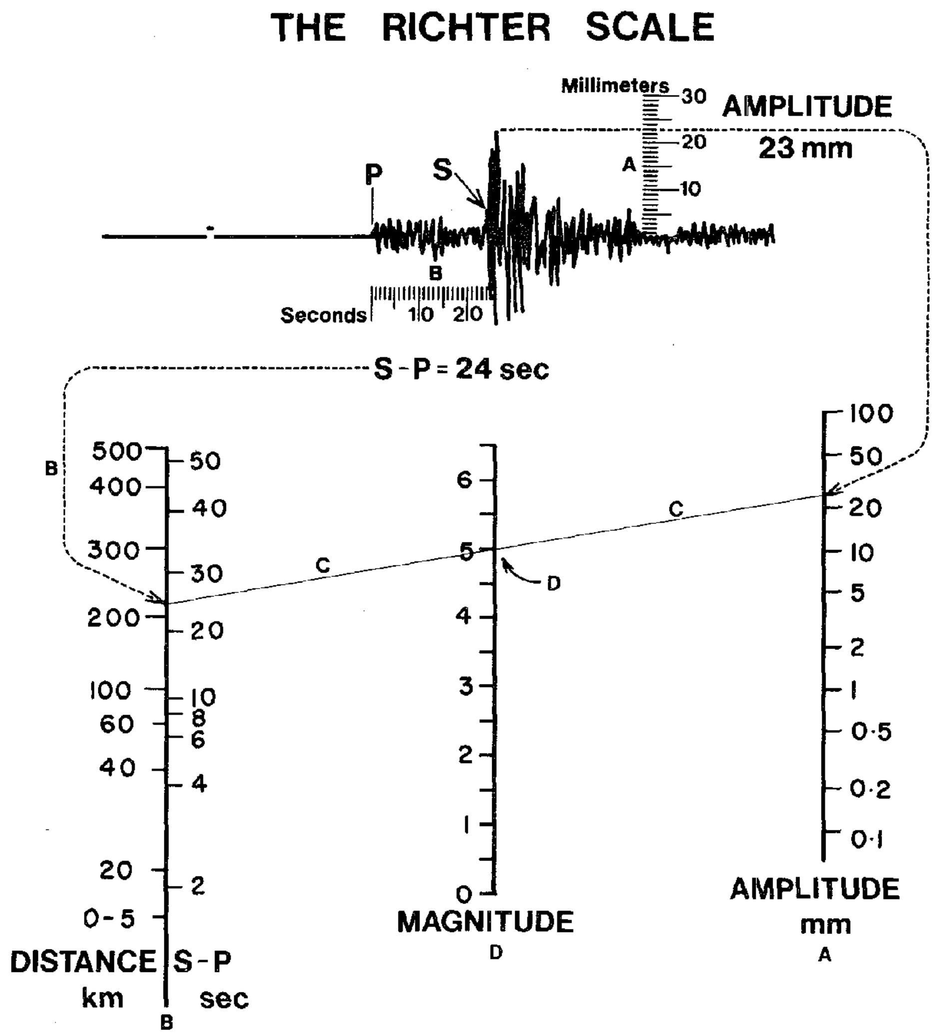

Figure II-11. To determine Richter Magnitude at varying distances from the epicenter, connect on the chart:

A. the maximum amplitude recorded by a standard seismometer, and
B. the distance of seismometer from the epicenter of the earthquake (or difference in arrival times of P and S waves) by
C. a straight line,
D. read the Magnitude off the center scale.

Because the size of earthquakes varies enormously, the graphic range of wave amplitude measured on seismograms is compressed by using as a scale the logarithm to base ten of the recorded wave amplitude. Hence each unit of magnitude indicates a 10 times increase in wave amplitude. But the energy increase represented by each unit is estimated by seismologists as approximately 31 times. Thus the amplitude of an 8.3 magnitude earthquake is 10,000 times that of an 4.3 shock, but its energy release is approximately 1,000,000 times.

The actual energy released by an earthquake is not a significant measure in relation to building reaction. It is of parenthetical interest to realize, however, that the energy release of earthquakes is very large indeed. It is estimated that the total energy released by earthquakes each year throughout the world is between 10^{25} and 10^{26} ergs. This is approximately equivalent to the present total yearly consumption of energy for all purposes in the United States.

The Richter scale has no fixed maximum, but about 9 is the greatest ever recorded. An earthquake of magnitude 2 on the scale is the smallest normally felt by humans; an event with a magnitude of 7 or more is commonly considered to be major. While the Richter scale accomplishes the goal of enabling us to make an objective comparison between earthquakes, it tells little about the local effects. It can also be an inadequate measure of the size of large earthquakes, in terms of the extent of geographical area affected (Figure II-12).

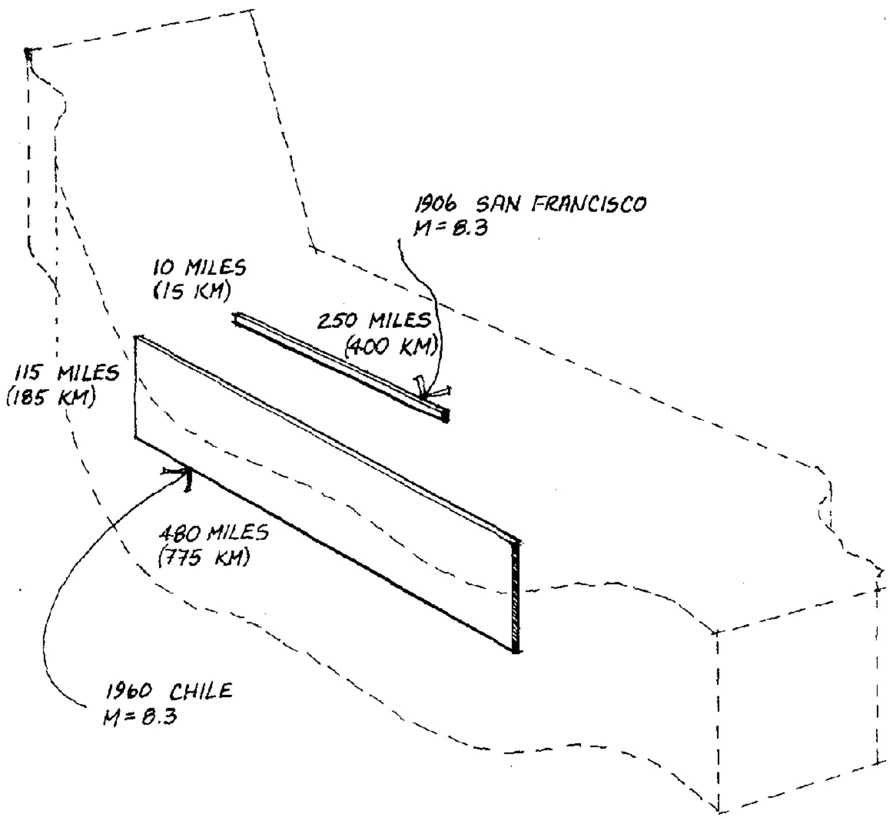

Figure II-12. Two earthquakes may have equal magnitudes but be distinctly unequal in other respects. The 1906 San Francisco California earthquake ruptured rock over a shorter length and shallower depth - only 1/25 the area - as the 1960 Chilean earthquake.

To provide information directly related to local shaking and building damage, several intensity scales are in use. In the United States the commonly used scale is the Modified Mercalli (MM), originally proposed in Europe in 1902, modified in 1931 by Wood and Neuman, to fit construction conditions then prevalent in California and the United States. The MM Scale is based on subjective observation of the effects of the earthquake on buildings, ground, and people. Because these effects will be different depending on distance from the epicenter, nature of the ground etc., one earthquake will have many MM values. An abridged version of the MM Scale, developed by Richter in 1956, is shown in Figure II-13.

MODIFIED MERCALLI INTENSITY SCALE (1956 version)

I. Not felt. Marginal and long-period effects of large earthquakes.

II. Felt by persons at rest, on upper floors, or favorably placed.

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

IV. Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV, wooden walls and frames creak.

V. Felt outdoors; direction 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.

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

VII. Difficult to stand. Noticed by drivers. 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, also unbraced parapets and architectural ornaments. Some cracks in masonry C. Waves on ponds, water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.

VIII. Steering of 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 or wells. Cracks in wet ground and on steep slopes.

IX. 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. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.

X. 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.

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

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

MASONRY A. Good workmanship, mortar, and design; reinforced especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.
MASONRY B. Good workmanship and mortar; reinforced, but not designed in detail to reist lateral forces.
MASONRY C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie at corners, but neither reinforced nor designed against horizontal forces.
MASONRY D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Figure II-13. The abridged Modified Mercalli (MM) scale.

Charles Richter has explained that (4):

"Magnitude can be compared to the power output in kilowatts of a broadcasting station. Local intensity on the Mercalli scale is then comparable to the signal strength on a receiver at a given locality; in effect the quality of the signal. Intensity like signal strength will generally fall off with distance from the source, although it also depends on the local conditions and the pathway from the source to the point."

The MM Scale has been roughly correlated with ground acceleration, for example MM VII corresponds to a peak acceleration of between approximately 0.1g and 0.29g. Other similar twelve point scales are

<table>
<tr><th>U.S.A.</th><th>U.S.S.R</th><th>JAPAN</th></tr>
<tr><td>I</td><td>I</td><td>0</td></tr>
<tr><td>II</td><td>II</td><td rowspan="2">I</td></tr>
<tr><td>III</td><td>III</td></tr>
<tr><td>IV</td><td>IV</td><td>II</td></tr>
<tr><td>V</td><td>V</td><td>III</td></tr>
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<tr><td>XI</td><td>XI</td><td rowspan="2">VII</td></tr>
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</table>

Figure II-14. Approximate conversions for intensity scales.

in use in China and Russia. Japan uses an eight point scale recognizing, perhaps rightly, the difficulties of a twelve point grading scale (Figure II-14).

In our present measurement systems then, we find a paradox. Magnitude based on objective instrumentation and mathematics does not provide the local information about ground shaking that is of most concern to designers. The MM Scale, while being directly oriented to building effects, relies on a methodology of subjective comparisons; its information sources consist of observations, postcard damage reports, and newspaper clippings, expressed in a Roman numeral scale.

Besides the subjectivity of the MM Scale, another problem is that of its age: the listing of construction materials emphasizes masonry and does not refer to many modern methods of construction such as glass curtain walls, hung ceilings, or precast concrete.

References

1. Samuel Clemens, [quote contained in a very old Earthquake Engineering Research Institute Newsletter].

2. Charles Darwin, The Voyage Of The Beagle, (Garden City, New York: Doubleday, n.d.), p. 309.

3. John Milne, Earthquakes and Other Earth Movements, (New York: D. Appleton and Company, 1886), pp. 14-15.

4. Henry Spall, "Charles F. Richter: An Interview," Earthquake Information Bulletin, Volume 12, Number 1 (January-February 1980), p. 7.

III. Building Reaction to Ground Motion

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A. Inertial Forces

Ground motion does not damage a building by impact similar to that of the wrecker's ball, or by externally applied pressure as by wind, but rather by internally generated inertial forces caused by vibration of the building's mass. The building's mass, size, and shape - its configuration - partially determine these forces and also partially determine how well they will be resisted.

Inertial forces are the product of mass and acceleration (Newton's $F = m \times a$). Acceleration is the change of velocity (or speed in a certain direction) over time and is a function of the nature of the earthquake; mass is an attribute of the building. Since the forces are inertial, an increase in the mass generally results in an increase in the force. Hence the immediate virtue of the use of lightweight construction as a seismic design approach.

The other detrimental aspect of mass, besides its role in increasing the lateral loads, is that failure of vertical elements such as columns and walls can occur by buckling when the mass pushing down due to gravity exerts its force on a member bent or moved out of plumb by the lateral forces. This phenomenon is known as the P-e, or P-Δ effect (Figure III-1). The greater the vertical force, the greater the moment due to the product of the force, P, and the eccentricity, e (or Δ).

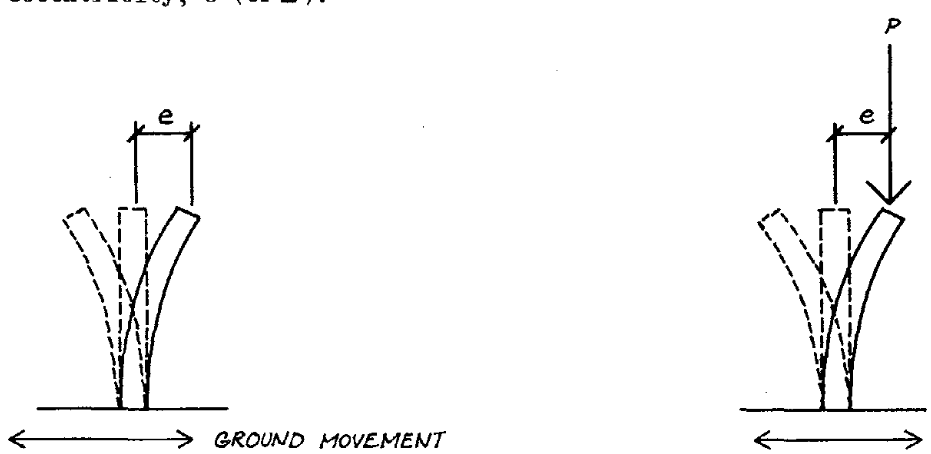

Figure III-1. The P-e effect. The stresses caused by the P-e moment occur concurrently with the other stresses induced by the earthquake and gravity. At a particular instant, the stresses may all be additive.

Although buildings generally have large vertical load carrying reserves due to code gravity load requirements, this safety factor does not necessarily mitigate the P-e problem, which can induce bending in columns.

Earthquakes shake the ground in a variety of directions - including up and down components. Codes generally treat these vertical earthquake forces lightly, although they may be two-thirds as great as the lateral earthquake forces, and "seismic design" and "design for lateral forces" are not really synonymous terms.

It is vertical loads that almost always cause buildings to collapse in earthquakes; however, in earthquakes buildings generally fall down, not over. The lateral forces use up the strength of the structure by bending and shearing columns, beams, and walls, and then gravity pulls the weakened and distorted structure down. Steinbrugge and Moran (1) noted that in the 1952 Kern County earthquakes, even top-heavy elevated water tanks fell down, not over (Figure III-2).

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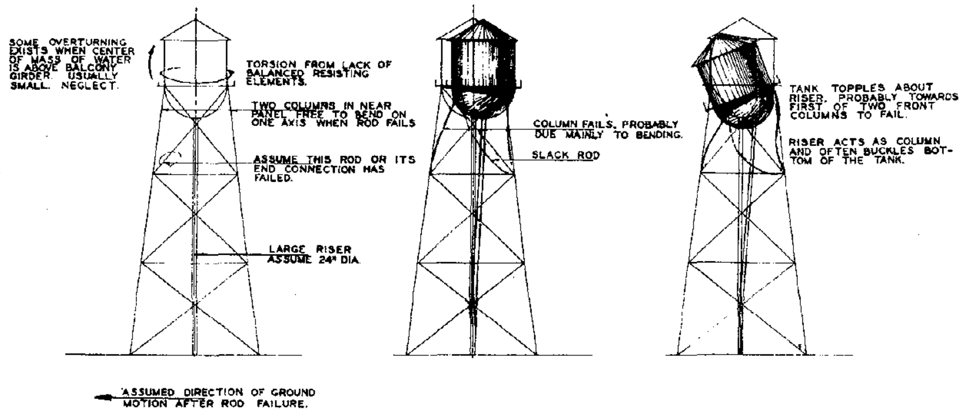


Figure III-2. Typical failure sequence of an elevated water tank, 1952 Kern County California earthquake.

B. Period and Resonance

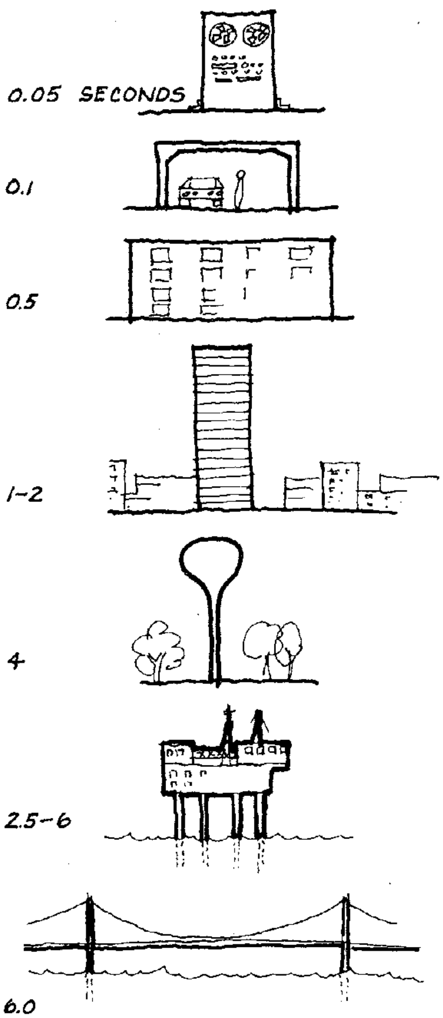


Figure III-3. Fundamental periods of various structures.

If one shook a flag pole with a heavy weight on top in the attempt to break it, one would quickly learn to synchronize one's pushes and pulls with the pole's natural tendency to vibrate back and forth at a certain rate - its fundamental period. If it tends to swing back and forth one complete cycle once a second when "plucked" and allowed to vibrate, it has a fundamental period of one second. If we can predict approximately the rate at which the ground will shake, which is similar to controlling the rate at which one shakes the base of the pole by hand, we could adjust the rate at which the pole will naturally vibrate so that the two either will or will not coincide. If they coincide, then the dimensions of the swing will start to increase, the pole will be said to resonate, and the loads on it will increase.

Ground motion will impart vibrations to a building of a similar nature to our shaking of the flag pole. The fundamental periods of structures may range from about 0.05 second for a well anchored piece of equipment, 0.1 for a one story simple bent or frame, 0.5 for a low structure up to about 4 stories, and between 1-2 seconds for a tall building from 10-20 stories. A water tank on a single support may have a fundamental period of 4, that of an offshore drilling rig will be between 2.5 - 6, and a large suspension bridge may have a period of about 6 seconds (Figure III-3).

Natural periods of soil are usually in the range of 0.5 - 1 second, so that it is possible for the building and ground to have the same fundamental period and therefore there is a high probability for the building to approach a state of partial resonance (quasi-resonance). Hence in developing a design strategy for a building, it is desirable to estimate the fundamental periods both of the building and of the site so that a comparison can be made to see if the probability of quasi-resonance exists. If the initial study shows this to be the case, then it would be advisable to change the resonance characteristics of the building (for the site characteristics are fixed) by methods that will be discussed later.

The natural periods of different types of ground are estimated by methods calling for a great deal of judgement, based on experience in previously recorded earthquakes on sites of like - or supposedly like - ground characteristics. These estimates are expressed by use of a response spectrum, which provides a useful illustration of the expected behavior of the site.

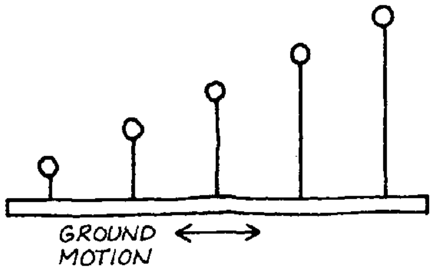


Figure III-4. The principle of the response spectrum.

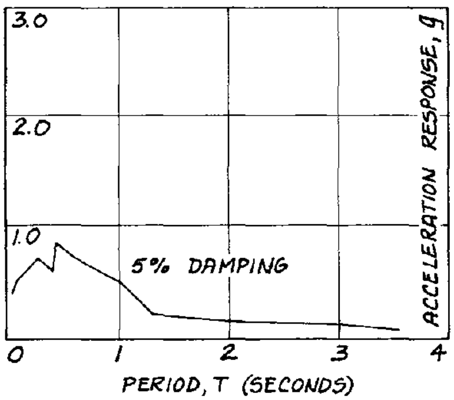


Figure III-5. A typical response spectrum for a particular earthquake and site.

The principle of the response spectrum, developed in 1931 by M. Biot, can be visualized as follows. Figure III-4 shows a series of cantilever pendulums (similar to our flag poles) whose periods lengthen towards the right hand side. If these are imagined as attached to a movable base, and the base is agitated to represent the strong motion of an earthquake as recorded on a seismograph, the maximum response of each pendulum can be recorded - that it, the time and particular frequency during the earthquake at which each pendulum will tend to resonate, with vibration of maximum amplitude. These maximum responses can be plotted against the periods of the pendulum, and will provide a curve, or response spectrum, that relates the nature of the ground motion to a range of natural periods. Note that every site will also show a different response spectrum for each earthquake - in terms of magnitude, type of ground motion, and distance of the fault slippage from the site - that is plotted. A typical curve will appear as in Figure III-5; the horizontal ordinate represents T, or periods, and the vertical ordinate generally represents equivalent acceleration.

In relating the estimated period of a new building to that of a site, curves will be drawn for the site that represent a range of responses; these will show the periods at which a maximum response is likely: part of the seismic design problem, then, is to "tune" the building in such a way that its own period is outside the range of probable site periods, and the possibility of forced amplification by resonance is reduced or eliminated.

How does one "tune" the building in this way? In the case of the simple flag pole paradigm, the pole's period might be altered by any or all combinations of the following: changing the position of the weight to some lower height; changing the height of the pole; changing the sectional area or shape of the pole; changing its material; altering the fixity of the base anchorage (Figure III-6).

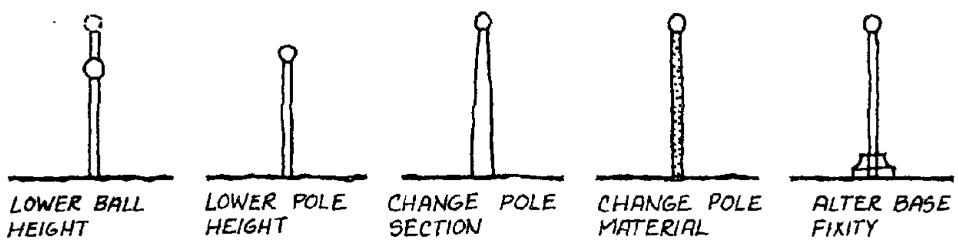


Figure III-6. Five ways to change the fundamental period of a flag pole.

There are analogous possibilities for buildings, though the buildin is much more complex than the simple monolithic flag pole. A struc ture can have more than one period, even if all factors remain constant. There are higher modes of vibration in which the structure will experience increasingly snake-like deflections, rather than just bending back and forth (Figure III-7). Though the first mode, simple to and fro motion, is generally the significant period of structural interest, higher modes can be important for tall slender buildings.

Seed and Alonso (2) concluded that in the 1967 Caracas earthquake, quasi-resonance between the underlying soil and the building itsel provided the best explanation for otherwise puzzling observed damage patterns. Buildings of comparable construction and configuration performed quite differently according to their location, an the soil depth and fundamental period of vibration of the soil in these locations correlated with high or low damage ratios depending upon whether a given building's period was similar or dissimilar to these site periods.

In general, a more flexible, longer period design may be expected to experience lesser forces, proportionately, than a stiffer building if the site is composed of bedrock which will efficiently transmit short period vibrations while filtering out longer period motions. By contrast, since it is difficult for a layer of soft alluvium several hundred feet deep to vibrate rapidly, even though the input motion from the bedrock beneath it may be high frequency, a stiffer building may have much less response than one with a longer period.

It is generally true that locations closer to the fault from where the energy is released will experience higher frequencies of ground motion, and at large distances the motions will probably be of lower frequency type which eye witnesses will call rolling, slowly rocking, swaying, etc.

In the 1954 Dixie Valley earthquake in Nevada, a very few unusually long period structures 185 miles away in Sacramento were selectively damaged while for other buildings and inhabitants of the city, no earthquake motion was experienced (3).

"... Thus, the seismic waves, which originated with the earthquakes, passed beneath Fallon, Reno, and Carson City without damaging them only to damage tanks and reservoirs in Sacramento. Fortunately, a strong-motion seismograph in Sacramento recorded the ground motion. The record clearly shows a number of cycles of ground motion ranging from 5 to 8 sec. This caused quasi-resonance to the liquids in tanks at three different locations in Sacramento and in one case destroyed reinforced concrete walls. The computed natural period of the fluids in the tanks and in the reservoir fell in the same range as the period of the ground motion, and the quasi-resonance that occurred caused large-amplitude water waves."

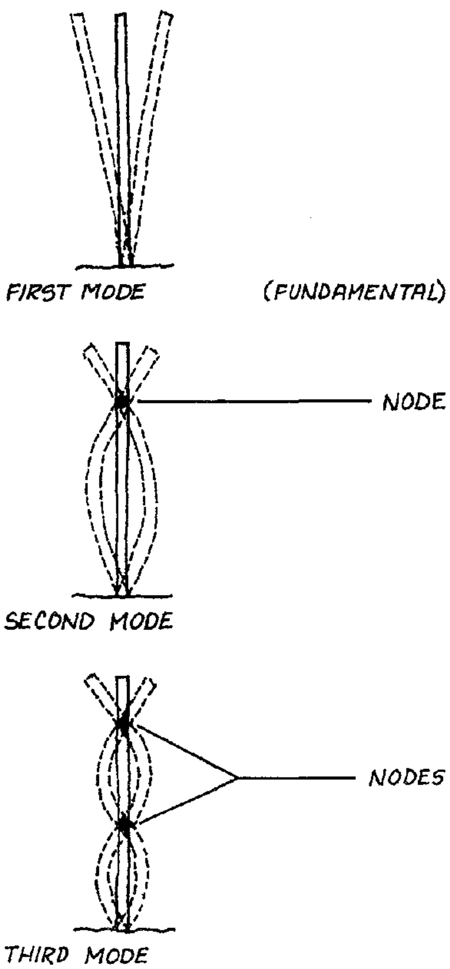


Figure III-7. Higher modes of vibration for a building.

Another related concept needs to be understood: this is the characteristic of damping, which affects the dynamic behavior of the building, and modifies its response to ground motion.

C. Damping

If a building resonates in response to ground motion, its acceleration is amplified - just as the response of certain types of soils may amplify the ground motion. This amplification can be very great: for a pendulum the amplification might be fifty fold, an increase which, for a building, would result in disastrously large forces. However, buildings are prevented from resonating with the purity of a pendulum, because they are damped: that is, they are rather inefficient in their vibration, and when set in motion tend to return to their starting position quickly. The extent of damping in a building depends on its connections, non-structural elements and construction materials, and we make assumptions about our designs based on knowledge of previous structures.

Critical damping refers to the amount of damping which will prevent oscillation from taking place - i.e. a pendulum will simply return to the center when plucked - and damping is measured as a percentage of critical damping. This is an arbitrary assumption because we have no rational approach to the theory of damping, and even the empirical data are less than quantitatively consistent. It is thus useful to modify the ground response spectrum by assuming percentages of damping that represent reasonable figures for buildings - generally of the order of 2% to 15% of critical, with figures at the low end of this scale most commonly used in design.

When damping is introduced, the general shape of the response curve remains the same, but the magnitudes are greatly reduced. Although damping is theoretically subject to alteration, in practice it is not generally regarded as a design variable.

Currently our codes recognize the beneficial aspect of flexibility (long period) by permitting lower design coefficients. However, the amount of motion experienced by these structures means that they may suffer much greater damage to their non-structural components.

This is not a new discovery: in the 1923 Tokyo earthquake, the Yusen and Marunouchi buildings, which were steel frame curtain wall buildings, both suffered mostly non-structural but expensive damage. On the other hand, the Japan Industrial Bank, Kabuki Theatre, and Jitsugyo buildings, (designed by Dr. Tachu Naito), which had rigid frames stiffened with shear walls and braced bents, were almost undamaged (4).

Glen Berg has commented on these issues (5):

"One puzzling anomaly in code development persists. Most earthquake engineers would agree that Dr. Naito was right when half a century ago he advocated the use of rigid rather than flexible structural systems. Experience has demonstrated the validity of his position time and time again. Although properly designed flexible and stiff structures may be equally safe, flexible structures consistently incur greater economic damage than stiff structures, especially non-structural damage. Yet the code provisions consistently grant benefits for flexible structures and penalties for stiff ones... All of these provisions may technically be sound, but whether they lead to optimum structures from the economic or social viewpoint remains an open question."

D. Ductility

Even if resonance is avoided, and the building is well damped, analysis will show that structures will be subject to forces that are much higher then those for which, under the building code, we will design. The code's equivalent static force formula method will produce a design lateral force of about 5% to 20% of the building's mass in high seismic zones, or a theoretical design acceleration of 5% to 20% of gravity (.05 to .2g). Real earthquakes have produced accelerations considerably in excess of this amount but, the fact that, under these conditions, our structures are adequately safe can be partly explained by the material property called ductility. This is the property of certain materials - steel in particular - to fail only after considerable inelastic deformation has occurred. Inelastic deformation is that in which the material does not return to its original shape after distortion. Brittle materials, however, such as concrete - fail suddenly with a minimum of deformation (Figure III-8). Note however, that the steel contained in reinforced concrete can give this material considerable ductility also. The act of deformation absorbs energy and defers absolute failure of the concrete.

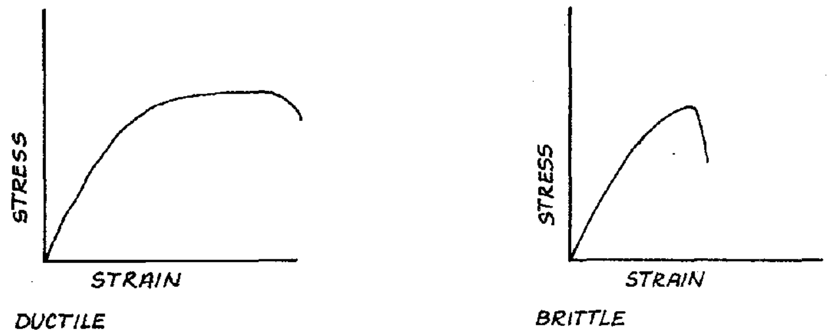


Figure III-8. Variations in ductility: steel is shown in the left curve, and concrete at right.

Ductility and reserve capacity are closely related: past the elastic limit (the point at which loads cause permanent deformation), ductile materials can take further loading before completely rupturing. In addition, the member proportions, end conditions, and connection details will also affect ductility. Reserve capacity is the ability of a complete structure to resist overload, and is dependent on the ductility of its individual members. The only reason for not requiring ductility is to provide so much resistance that members would not exceed elastic limits.

E. Torsion

The center of mass, or center of gravity, of an object is the point at which it could be exactly balanced without any rotation resulting. Uniformly distributed mass results in the coincidence of a plan's geometric center with the center of mass (Figure III-9). An eccentric distribution of mass locates the center of mass away from the geometric center (Figure III-10). This means that since every particle of mass of an object is attracted by gravity toward the center of the earth's mass ("down"), the opposite force exerted upward to counteract this force or "weight" must be precisely located under the object's center of mass to make the object balance without any net moment: the tipping moments along all axes must cancel out.

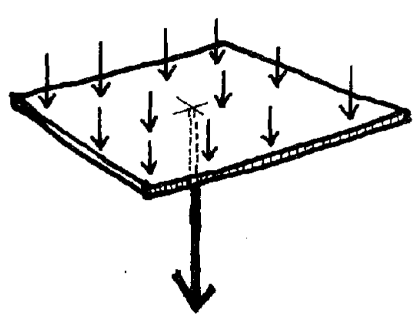


Figure III-9. Uniformly distributed mass: center of mass acts through geometric center.

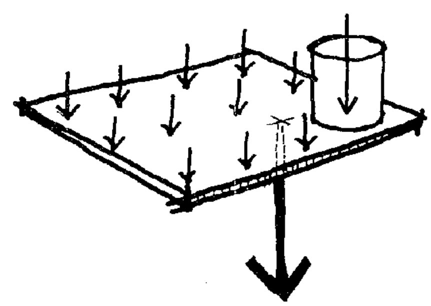


Figure III-10. Eccentrically distributed mass: center of mass acts eccentric to geometric center.

When the particles of mass are accelerated horizontally due to earthquake inertia forces, the same balancing principles apply. Earthquakes create inertia forces which can be likened to a random, pulsating, horizontal equivalent of gravity: every particle of mass is accelerated laterally (and sometimes vertically as well). If the mass within a floor is uniformly distributed, then the resultant force of the horizontal acceleration of all of its particles of mass is exerted through the floor's center (Figure III-11). If the resultant of the resistance (provided by walls or frames) pushes back through this point, and hence meets the resultant of the loads head on, translational dynamic balance is maintained (Figure III-12). Otherwise, horizontal rotation, or torsion, would result.

If the mass is eccentrically disposed, the earthquake load will be eccentric as well since the earthquake only generates a load because of the presence of mass, and the amount of load is directly proportional to the amount of mass. If the load is eccentric, then the resistance must also be eccentric so that the location of the center of mass and the center of horizontal resistance are at the same point (Figure III-13), and torsion is avoided.

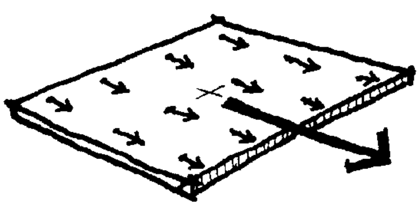


Figure III-11. Uniformly distributed mass: resultant of inertia forces acts through geometric center.

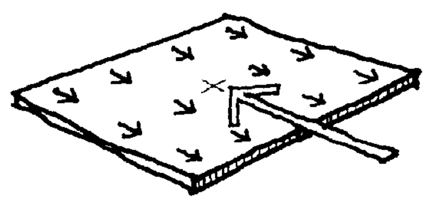


Figure III-12. Uniformly distributed mass: resultant of resistance acts through geometric center - no torsion.

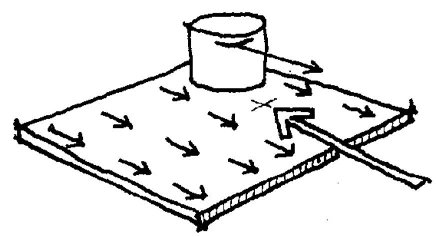


Figure III-13. Eccentrically distributed mass: eccentric resistance acts through the center of mass providing dynamic balance - no torsion.

Figure III-14 shows the torsional effects created in a simple building configuration. Torsion is occurring because a uniformly distributed lateral force is not resisted by a uniformly distributed lateral resistance. Other common examples of torsion (sometimes useful) are illustrated in Figure III-15.

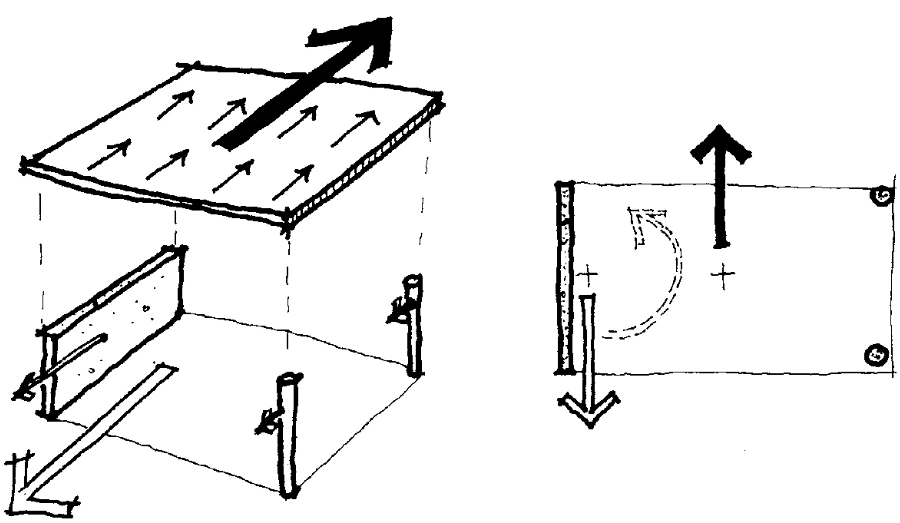


Figure III-14. Torsion in a simple building configuration.

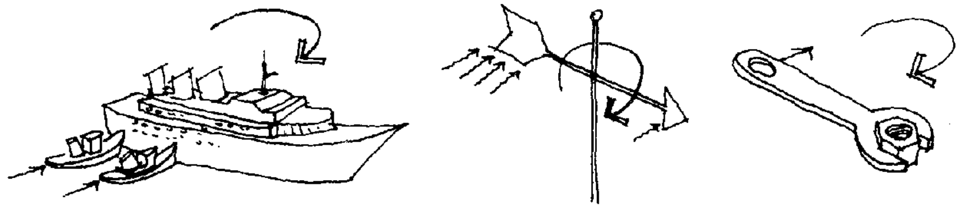


Figure III-15. Useful applications of torsion.

In a building in which the mass is approximately evenly distributed in plan - which would be typical of a symmetrical plan with uniform floor, wall, and column masses - the ideal arrangement is that the earthquake resistant elements should be symmetrically placed, in all directions, so that no matter in what direction the floors are pushed, the structure pushes back with a balanced stiffness which prevents rotation from trying to occur. Hence the general rule is usually stated that symmetry is a valuable configuration characteristic; however, this admonition is a somewhat simplistic directive, as is discussed later.

F. Strength and Stiffness

Strength and stiffness are intuitively two of the most important characteristics of any structure. However, although these two concepts are present in non-seismic structural design and analysis, the distinction between strength and stiffness is perhaps most critical and its study most highly developed in structural engineering as applied to the earthquake problem.

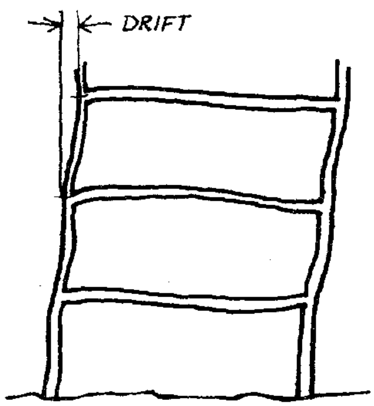


Figure III-16. The definition of drift: horizontal story-to-story deflection.

One measure of stiffness is deflection, and for vertical gravity loads is, in most cases, the only aspect of stiffness which is of concern. In the sizing of floor joists, deflection rather than strength often governs. The analogous lateral force condition is when limitations on drift, the horizontal story-to-story deflection, impose more severe requirements on members than the strength requirements (Figure III-16). The strength problem is how to resist a given load without exceeding a certain stress; the stiffness or horizontal deflection problem is how to prevent the structure from moving out of alignment more than a given amount. In the design of a floor system, the joists may tolerate a certain deflection but the ceiling finish cannot; similarly drift must be limited, even if the structure can tolerate more, because of its effect on non-structural components, particularly partition, skin and ceiling elements, and on the comfort of occupants. Excessive horizontal deflection can also cause loads to be applied eccentrically to their columns, discussed earlier as the P-e effect.

The Uniform Building Code prohibits drift from exceeding 1/2% of the story height (under the design forces, with a multiplier to correct for safety factors). This would be 1" for a 16'-8" high story.

Occasionally in the design of a floor system, it is not possible to assign loads to members merely on the basis of tributary areas without taking into account how stiff the members are. For example: if a beam and a girder interact by being monolithic or framing into one another, as shown in Figure III-17, and if the girder is so stiff that, as it deflects one inch, it becomes highly stressed, while the beam is so flexible that with one-inch deflection it is just beginning to be stressed, it is intuitively obvious that it is

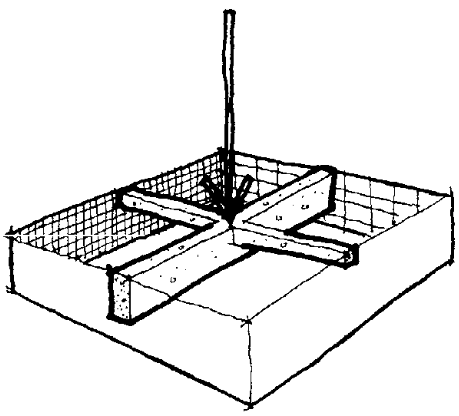


Figure III-17. Stiff and flexible beams.

almost as if the very flexible beam were non-existent: the stiff girder takes almost the entire load. If the flexible beam is incrementally stiffened, it will begin to take a larger and larger share of the floor load.

The relative rigidities of members is occasionally of concern for gravity loads, but it is a major concern in seismic analysis. As soon as a rigid horizontal element, or diaphragm, such as a concrete slab, is tied to vertical resisting elements, it will force those elements to deflect the same amount. (Since the diaphragm is rigid, it can be assumed, for analytical purposes, that it moves translationally the same amount throughout its area without any distortion. "Rigid" in this case refers to certain particular types of diaphragms discussed later.) If two elements, (two frames, walls, braces, or any combination) are forced to deflect the same amount, and if one is stiffer, that one will take more of the load. Only if the stiffnesses are identical can it be assumed that they share the load equally. Since concrete slab construction floors or roofs will generally fit into the "rigid diaphragm" classification, and since it is unusual for all walls, frames or braced frames to be identical, the evaluation of relative rigidities is a necessary part of most seismic analysis problems.

Figure III-18 shows the relative rigidity of a number of concrete walls of different dimensions: the important point here is that, for rigid diaphragm structures, walls take load in proportion to their rigidity. Doubling the length of a wall approximately doubles its shear strength, but more than doubles its rigidity, and hence more than doubles its load.

The other important aspect of stiffness, the overall stiffness of a building as measured by its period, has been previously discussed in connection with response.

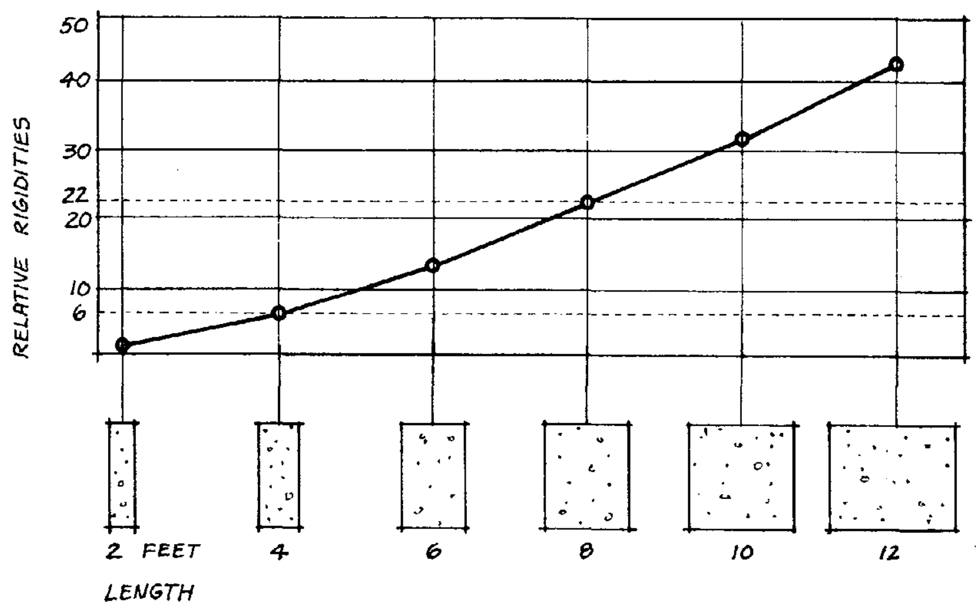


Figure III-18. Relative rigidities of concrete walls which are identical except for length (i.e. 10' high, 10" thick, restrained top and bottom, same modulus). As an example: an 8' long wall is double the length of a 4' wall, but its rigidity is over three times (22/6) greater. The 8' wall will have double the shear strength, but since load is proportional to rigidity, it will carry over three times the load.

G. Resistant Systems

In designing to resist seismic forces, the structural engineer uses a small vocabulary of components which are combined to form a complete resistance system.

In the vertical plane three kinds of components resist lateral forces: shear walls, braced frames, and moment resisting frames (sometimes called 'rigid frames'). In the horizontal plane diaphragms are used, generally formed by floor and roof planes of the building, or horizontal trusses (Figure III-19). These elements are also basic architectural components. Their presence is the result of the schematic architectural design of the building. They may be modified in location or shape as a result of structural analysis, and members may sometimes be added.

It is useful for the architectural designer to acquire an understanding of the way these resistance systems work, in response to the forces that the earthquake generates: the detailed calculations can be left to the engineer. The architect may neither be able nor wish, to acquire the depth of theoretical understanding and experience which the engineer must have, but it is worth attempting to transfer a feeling for structural forces, because once acquired, this feeling can act as an almost automatic guide to the designer.

Most designers have acquired a sense of vertical static forces, if only through the experiences of their own bodies. A sense of dynamic forces is less easy to acquire naturally, but many athletes - skiers, high divers, skate boarders - also have a good sense of how movement modifies the effect of gravity. 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 following sketches represent an elementary course in this approach. However, the reader should remember, as we have seen, that seismic forces are more complex than gravity forces and must always be visualized as dynamic - moving - and as multi-directional rather than operating in a single direction (Figure III-20).

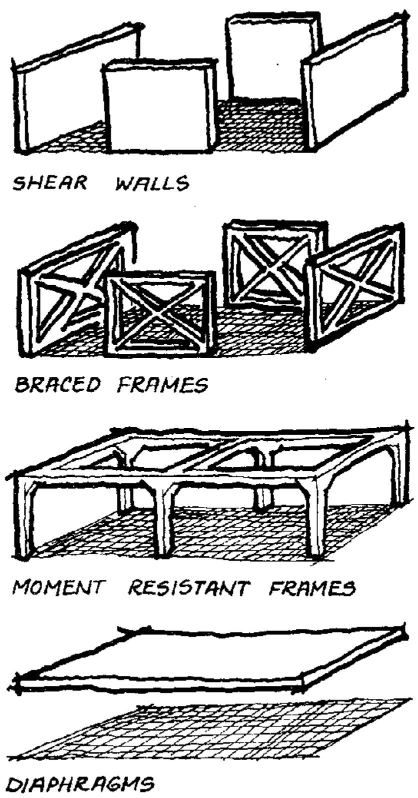


Figure III-19. Components for seismic resistance.

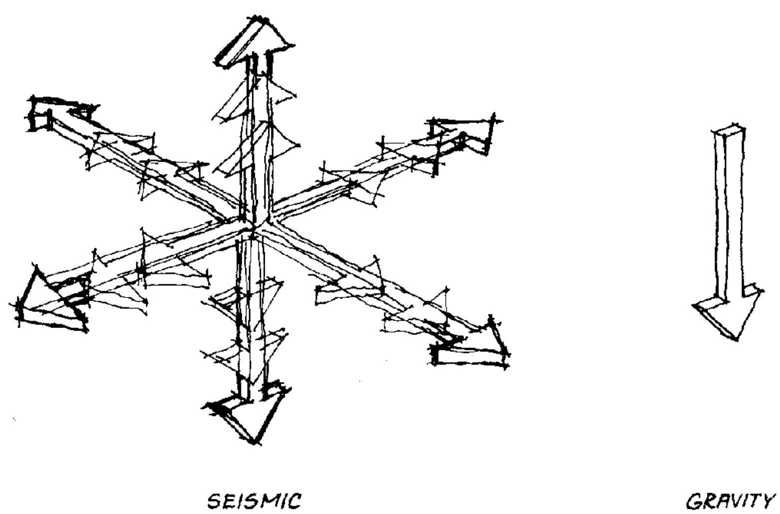


Figure III-20. Gravitational forces vs. seismic forces.

H. Diaphragms

The term 'diaphragm' is used to identify horizontal resistance elements, (generally floors and roofs) that act to transfer lateral forces between vertical resistance elements (shear walls or frames). The diaphragm acts as a horizontal beam: the diaphragm itself acts as the web of the beam, and its edges act as flanges (Figure III-21).

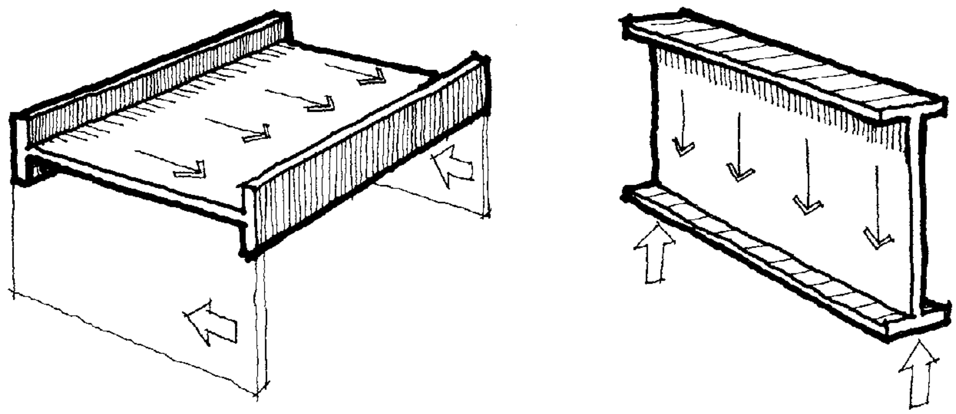


Figure III-21. The diaphragm: a horizontal beam.

Floors and roofs often have to be penetrated - by staircases, elevator and duct shafts, skylights, or architectural features. The size and location of these penetrations is critical to the effectiveness of the diaphragms. The reason for this is not hard to see when the diaphragm is visualized as a beam: we can for example see that openings cut in the tension flange of this beam will seriously weaken its load carrying capability (Figure III-22). In a vertical load system, a penetration through a beam flange would occur in either a tensile or compressive area; in a lateral load system, the hole will be in a region of both tension and compression, since the loading alternates direction.

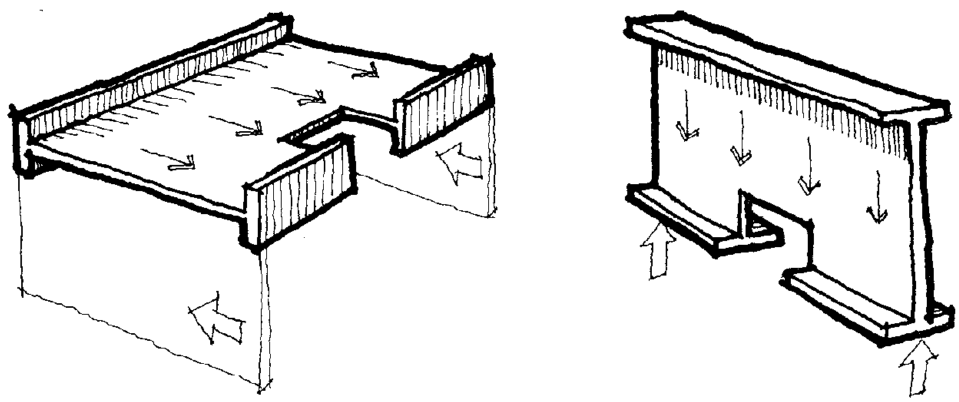
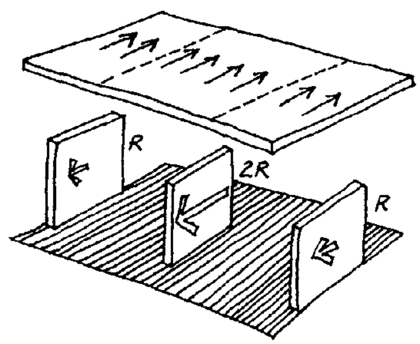


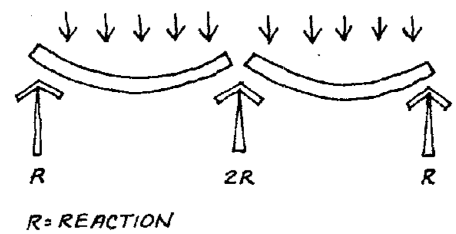
Figure III-22. Diaphragm penetrations.

When diaphragms form part of a resistant system, they may act either in a flexible or stiff manner. This is partly dependent on the size of the diaphragm - its area between enclosing resistance elements or stiffening beams and girders - and also a function of its material. The flexibility of the diaphragm, relative to the shear walls whose forces it is transmitting, also has a major influence on the nature and magnitude of those forces. This effect is shown by Figure III-23.

FLEXIBLE DIAPHRAGMS

USUALLY MADE OF WOOD, STEEL DECKING WITHOUT CONCRETE, OR LIGHTLY-TRUSSED DECKING. LONG NARROW DIAPHRAGMS OF ANY MATERIAL MAY ALSO BEHAVE FLEXIBLY.



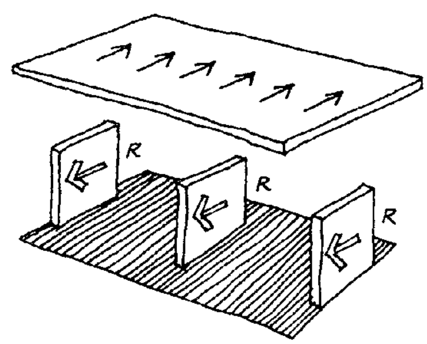


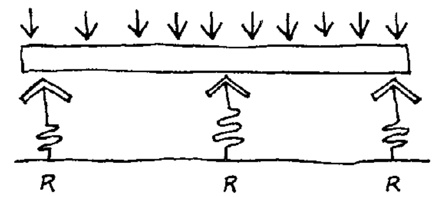
CENTRAL WALL, HAVING TWICE THE TRIBUTARY AREA, TAKES TWICE THE LOAD AS EACH END WALL (EVEN IF THE END WALLS ARE STIFFER).

THE DIAPHRAGM IS MUCH MORE FLEXIBLE THAN THE VERTICAL ELEMENTS AND IS ASSUMED TO ACT LIKE SIMPLY SPANNING BEAMS ON UNYIELDING SUPPORTS. WALLS TAKE LOADS ACCORDING TO TRIBUTARY MASSES (OR TRIBUTARY AREAS IF MASS IS EVENLY DISTRIBUTED). DIAPHRAGM IS ASSUMED INCAPABLE OF CARRYING TORSIONAL MOMENTS.

RIGID DIAPHRAGMS

USUALLY CONCRETE SLABS.





THE WALLS SHARE THE LOADS IN PROPORTION TO THEIR STIFFNESSES-- THE RELATIVE RIGIDITY RULE. IF ALL 3 WALLS ARE EQUALLY RIGID, AND IF THEY ARE FORCED TO STRAIN THE SAME AMOUNT, THEN THEY MUST BE EQUALLY STRESSED AND EQUALLY LOADED. IF ONE WALL IS TWICE AS STIFF AS ANOTHER, IT WILL CARRY TWICE AS MUCH LOAD. (RELATIVE RIGIDITIES, AND HENCE LOAD DISTRIBUTION, MAY CHANGE IF ONE WALL YIELDS EARLIER THAN OTHERS.)

THE VERTICAL ELEMENTS ARE MORE FLEXIBLE THAN THE DIAPHRAGM, WHICH IS ASSUMED TO ACT LIKE A NON-DISTORTING PLATE, (THOUGH FOR OTHER PURPOSES, SUCH AS CALCULATING HOW FAR A MASONRY WALL WILL LEAN, THE DIAPHRAGM RESTRAINING THE WALL IS ASSUMED TO DEFLECT). THE SUPPORTS FOR AN INFINITELY RIGID BEAM DEFLECT THE SAME AMOUNT (BEFORE TORSION IS ADDED IN).

Figure III-23. Horizontal resistant elements: flexible and rigid diaphragms.

Collectors, or drag struts, are diaphragm framing members which "collect" or "drag" diaphragm shear forces from laterally unsupported areas to vertical resisting elements.

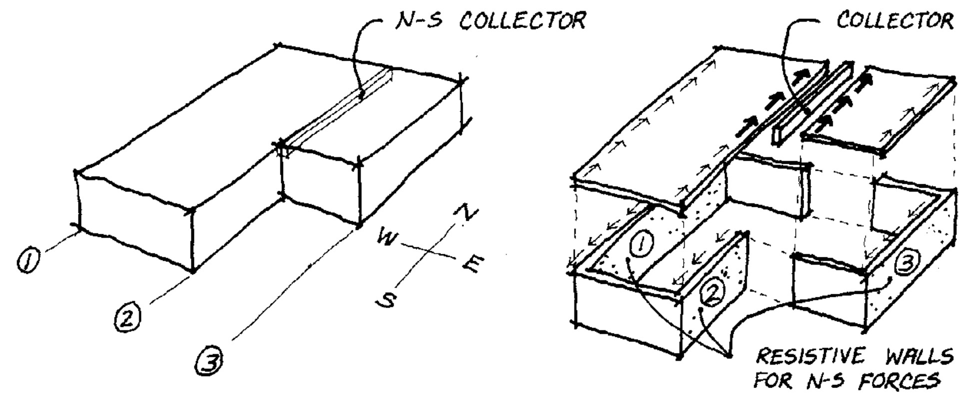


Figure III-24. Collectors.

As the diaphragm attempts to move north (or south), walls 1, 2, and 3 resist via transfer of shear from diaphragm to top of wall (Figure III-24). The forces shown in bold, which will be counteracted by a reaction supplied by Wall 2, cannot be directly transferred to the wall, and hence a collector (which may or may not be a beam for vertical loads) must drag these forces back to Wall 2. The diaphragm on either side of it delivers shears simultaneously, so they are additive. For the case shown, the collector would be in tension and through its anchorage it pulls on Wall 2. For the case when the diaphragm attempts to move south (northward ground movement) the collector would be in compression and would push Wall 2. The same situation occurs on the other axis, so an east-west collector would be used as well (Figure III-25).

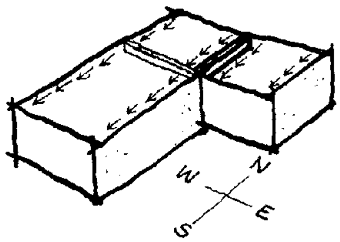


Figure III-25. A system of collectors.

The location of a hole (core, skylight, etc.) at the intersection of the component rectangles would interrupt the collector's load path (Figure III-26), and hence should be avoided.

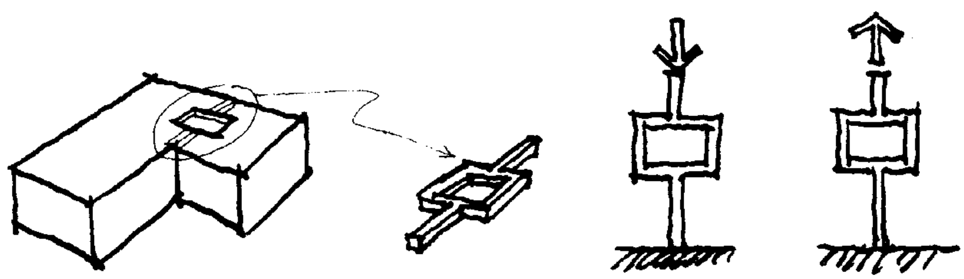


Figure III-26. Holes in collectors: the vertical load analogy.

I. Shear Walls and Braced Frames

Vertical cantilever walls which are designed to receive lateral forces from diaphragms and transmit them to the ground are commonly termed shear walls. The forces in these walls are predominantly shear forces, though a slender shear wall will also incur significant bending (Figure III-27).

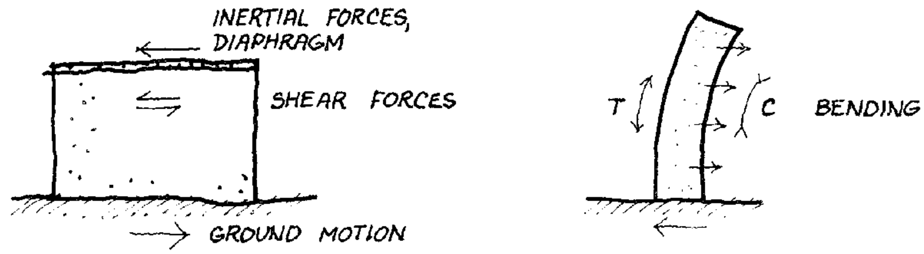


Figure III-27. Forces in shear walls.

Figure III-28 shows a simple building with shear walls at its ends. Ground motion enters the building and creates inertial forces which move the floor diaphragms. This movement is resisted by the shear walls, and the forces are transmitted back down to the foundation.

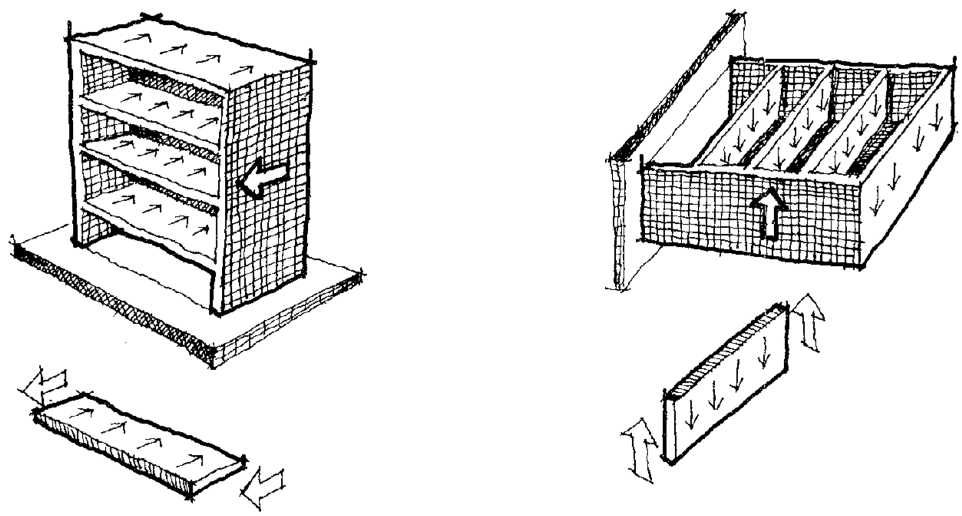


Figure III-28. Shear walls: vertical analogy as cantilever beams.

If the building is visualized as rotated so that it extends horizontally, it is clear that the shear walls are acting as cantilever girders which support beams represented by the floor diaphragms. However, unlike a normal cantilever supporting gravity forces, the shear wall must resist dynamic forces that are reversing their direction, for as long as the strong motion continues (Figure III-29), which is dependent on the characteristics of the earthquake.

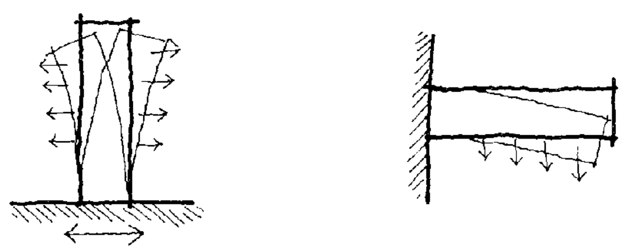


Figure III-29. Direction reversal.

The size and location of shear walls is extremely critical. Plans can be conceived of as collections of resistant elements with varying orientations to resist translational forces, and placed at varying distances from the center of rigidity to resist torsional forces. Some conceptual aspects of wall location within simple geometric plan forms is shown in Figure III-30.

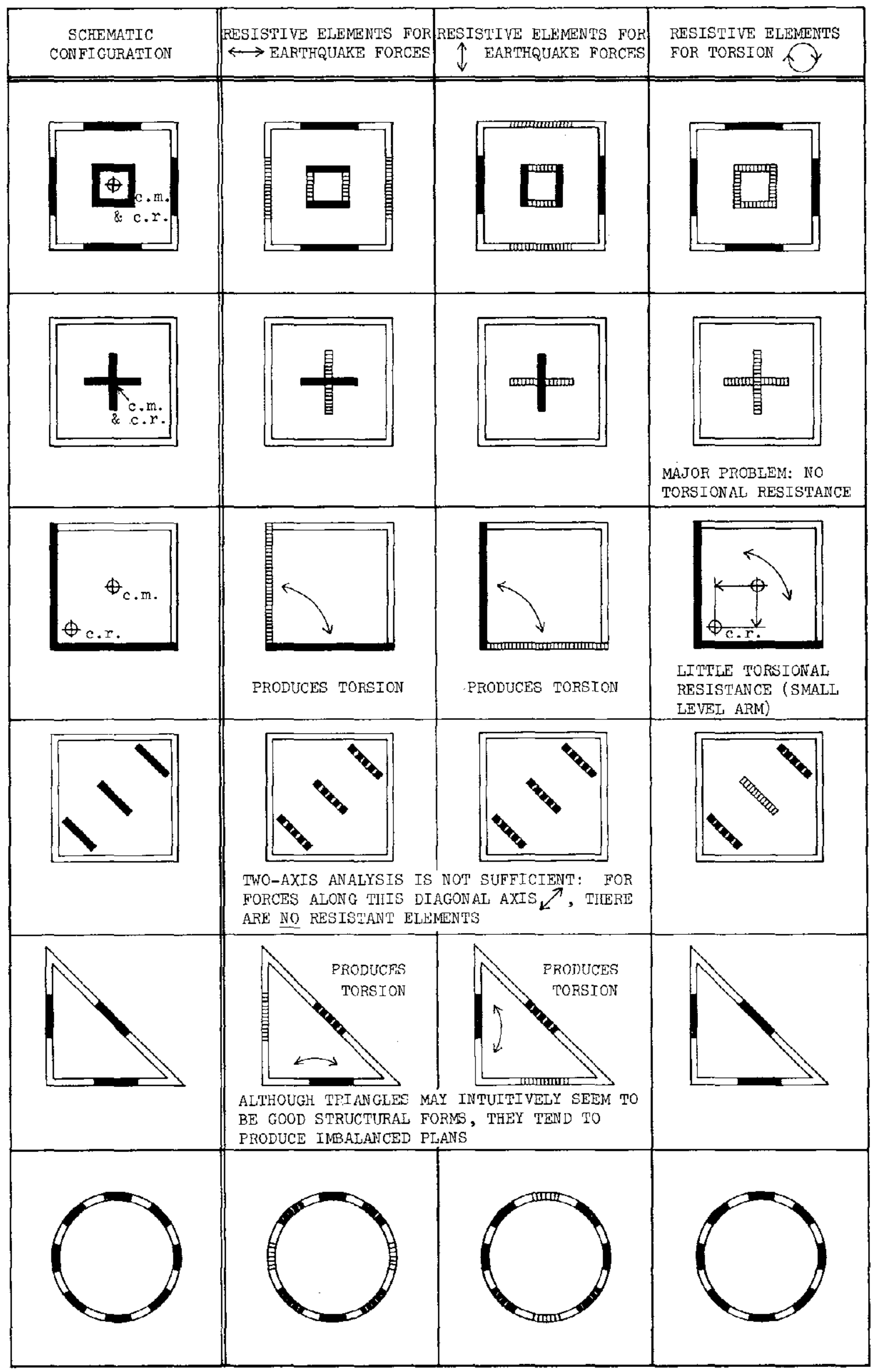


Figure III-30. Shear wall location. Schematic plans can be conceived of as collections of resistant elements with varying orientations (to resist translation) and with varying distances from the center of rigidity (to resist rotation, or torsion).

Braced frames act in the same manner as shear walls, though they may be of lower resistance depending on their detailed design. Bracing generally takes the form of steel rolled sections, circular bar sections, or tubes; vibrating forces may cause it to elongate or compress, in which case it loses its effectiveness and permits large deformations or collapse of the vertical structure (Figure III-31). Inelastic behavior must be designed into the bracing to create a safe assembly.

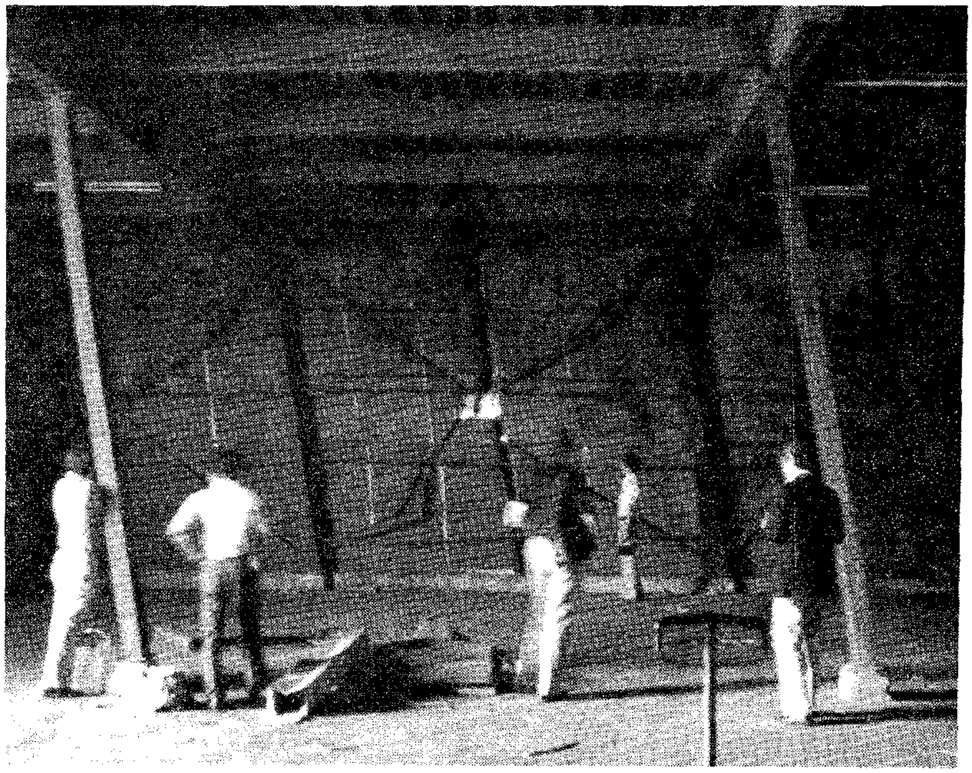


Figure III-31. Near failure of a braced frame warehouse, 1978 Sendai Japan earthquake.

Detailing to ensure complete load paths for the high forces is very important, and detailing which causes eccentricity may greatly reduce the effectiveness of bracing, although some sophisticated bracing schemes now coming into use incorporate offset joints (6). These are designed to ensure that non-linear behavior would occur first in beams rather than columns, and through failure control and the use of ductility, delay the onset of total collapse caused by column buckling (Figure III-32).

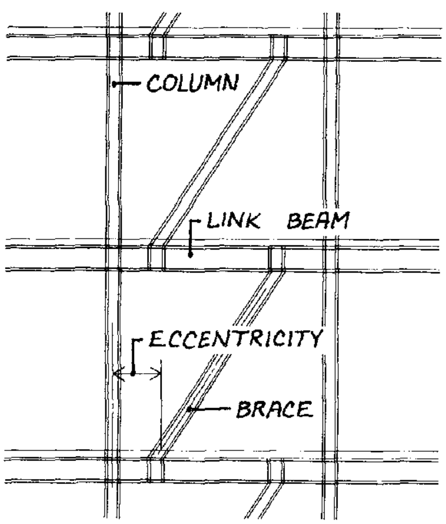


Figure III-32. Bracing with offset joints.

J. Moment Resistant Frames

When seismic resistance is provided by moment resistant frames, lateral forces are resisted by bending and shearing of columns and beams, which are connected by moment connections. Joints become highly stressed, and the details of their construction are important. In addition, behavior of the frame in the inelastic, or plastic, range becomes an important feature in resistance strategy, by using the energy absorption obtained by permanent deformation of the structure prior to ultimate failure. For this reason moment resistant frames are generally conceived as steel structures with stiff welded joints, in which the natural ductility of the material is of advantage. Recently, however, properly reinforced concrete frames have also been accepted as ductile frames: that is, they will retain some resistance capacity in the inelastic range, prior to failure.

The use of moment resistant frames is of architectural significance in two ways. One is that their use obviates the need for shear walls or braced frames, with the possible restricting planning implications of both. The other is that moment resisting frame structures tend to be much more flexible than shear wall type structures, with consequent implications for the design of accompanying architectural elements such as curtain walls, partitions, and ceilings.

Just as in a shear wall resistance system, in which specific walls are assigned to the resistance task, so in a frame design the moment resistant frames may be only a portion of the total frame structure.

K. Non-Structural Elements

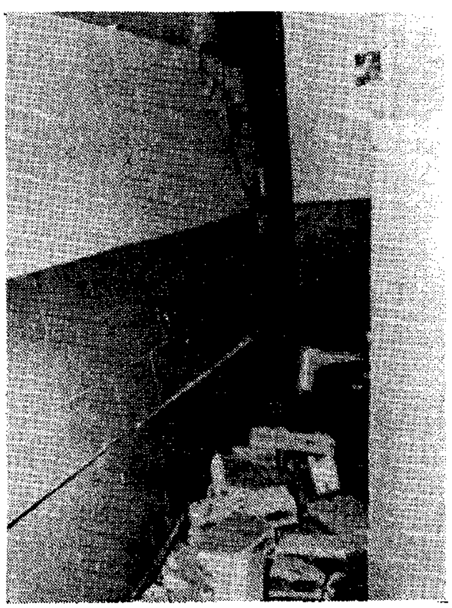


Figure III-33. Damage to non-structural block walls, Elmendorf Air Force Base, during the 1964 Anchorage Alaska earthquake.

Finally, it is important to recognize that non-structural elements may, inadvertently, form part of the lateral resistance system. S.B. Barnes has commented pointedly on this issue as follows (7):

"Some designers not too familiar with the actual response of structures in earthquakes have designed structures to resist earthquakes but ignored the effect of non-bearing but stiff and frangible filler walls. They have said to themselves these are just filler walls or partitions and we will ignore them in our computations. Unfortunately, no one has communicated with these walls and told them that they were to play a passive part in earthquake resistance. The U.S. Government Buildings at Elmendorf Air Force Base and Fort Richardson in Anchorage were full of 4-inch concrete block, non-reinforced walls which tried to act in diagonal compression or diagonal tension (Figure III-33). Some of these exploded like shrapnel and had the earthquake occurred at some other time the loss of life would undoubtedly have been much greater. And, of course, by ignoring the rigidities of these walls the computations made were greatly in error, at least until these walls failed. Isolation of these walls is permissible provided really effective isolation details are used."

If rigid enclosure or separation walls are not isolated from the structure by slip joints, they must be designed as integral parts of the structure, and their location then becomes a structural issue. Because of the tremendous rigidity of walls as compared to frames, a small amount of wall in the wrong place can drastically redistribute loads and change the structure's performance. Asymmetrical wall arrangements can overwhelm a symmetrical frame's

Figure III-34. Staircase damage in 1978 Sendai Japan earthquake.

attempt to respond to lateral forces in a relatively torsion-free manner. Staircases, since they may form diagonal braces, are similarly quite rigid and quickly assume a large structural role, for good or ill, unless isolated from lateral movements (Figure III-34).

However, non-structural elements may also provide a degree of useful redundancy (see Chapter IV, Section K for a discussion of redundancy). A common example of beneficial, uncalculated seismic resistance, demonstrated in a number of earthquakes, is the ability of "non-structural" wood frame partitions to hold up an unreinforced masonry building after the exterior "load bearing" walls have completely collapsed (Figure III-35). In these older apartment or hotel buildings, continuous wooden joists sometimes carried their loads by undesigned cantilever action after exterior walls fell. One of the clear virtues of normal wood frame residential construction, lies in the redundancy of load paths and the multiplicity of joints.

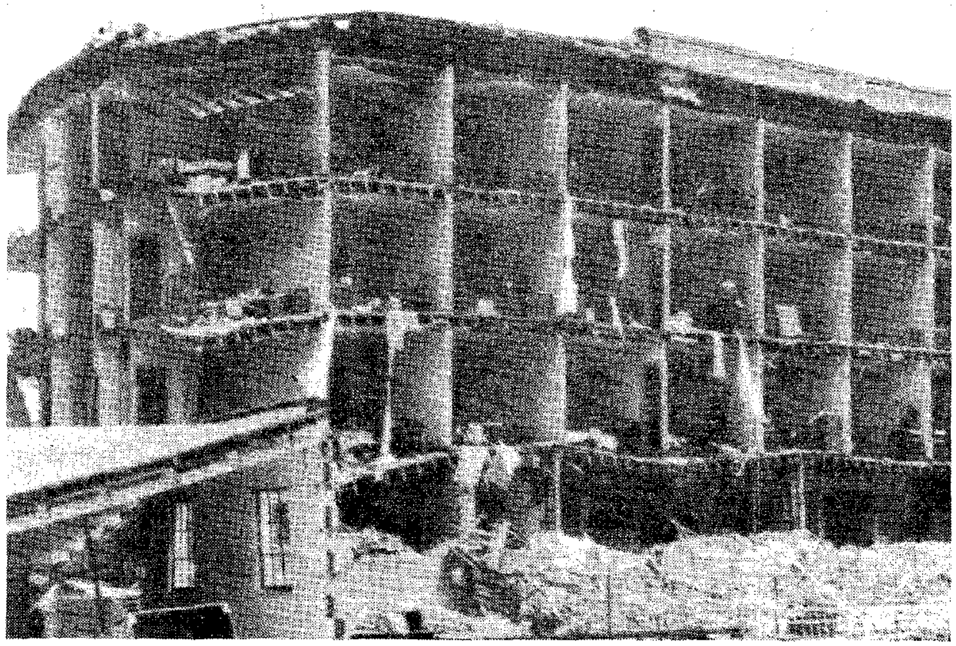


Figure III-35. Undesigned cantilever floors at the Hotel Californian, in the 1925 Santa Barbara California earthquake.

L. Conclusion

We can qualitatively summarize the reaction of the building to ground motion as a set of conditions which, because of the number of unpredictable variables, must be viewed as subject to great uncertainty. This atmosphere of uncertainty has been well expressed by Henry Degenkolb (8).

"The basic problem has its source in the fact that it is much more demanding of the engineer because we are dealing with unknown loads, meager information of material properties and the performance of the structure is determined in the ultimate load range rather than at service loads.

"As far as the Structural Engineer is concerned, the most important aspect of earthquake engineering is this basic difference from all other structural design. Our design forces are only a small fraction of the forces expected to be exerted on a structure in a major earthquake. In other words, the structure will be overstressed many times as defined by usual design standards."

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IV. Configuration Influences on Seismic Performance

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A. Introduction

In identifying those characteristics of configuration that affect the way in which the building responds to earthquakes, the definition of configuration provided earlier may be recalled: that the term refers both to the overall shape of the building, and to the size, nature and location of resisting and non-structural elements within it.

This section provides a general survey of the ways in which configuration influences seismic response, and in so doing, emphasizes the configuration aspects of the behavior discussed in Chapter III.

Before considering building configuration issues in more detail, it is important to re-emphasize the ways in which the building reacts to the dynamic forces of ground motion. The complexity of this reaction can be compared to the simplicity of the building response to the static force of gravity. If a one hundred pound weight is set on a floor there is no way that the structure can avoid carrying that precise weight down to the foundation.

But because earthquakes exert rapidly fluctuating dynamic loads, we cannot begin to determine seismic forces unless we know a building's dynamic characteristics. And even with this knowledge, the sequence of events and the interaction of different elements of the building under dynamic loads are so complex that the exact nature of seismic forces must be subject to great uncertainty.

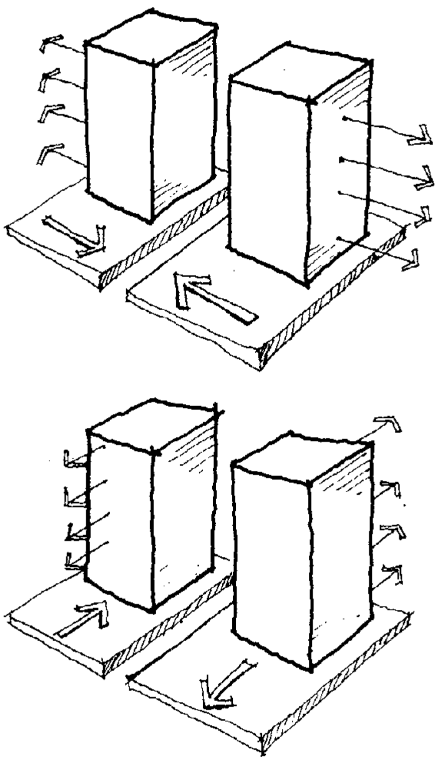

Figure IV-1. Typical analytical diagram of earthquake forces.

This complexity should be remembered when visualizing the lateral forces on a building configuration which are generally modelled by diagrams such as Figure IV-1. This type of diagram originates in the form of the typical seismic design analysis, in which earthquake forces are separately applied to each of the main axes of the building. To choose to consider only two axes is rational in the case of a rectangle; for a circle all axes are the same; for complicated shapes, the building might have to be looked at along several axes (Figure IV-2).

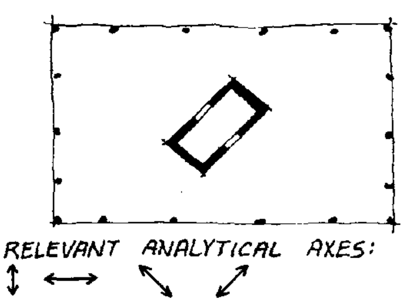

Figure IV-2. For complicated configurations, more than two axes may be used for analysis.

The basic concept is that since earthquake forces may come from any direction, the application of forces perpendicular to the major axes of walls or frames usually simulates the two worst cases. If ground motion, and their corresponding forces occur diagonally, then the walls or frames along both axes can participate in their resistance and the forces in each will be correspondingly reduced (Figure IV-3). These issues were discussed previously in Chapter III, Section I, with specific reference to shear wall location.

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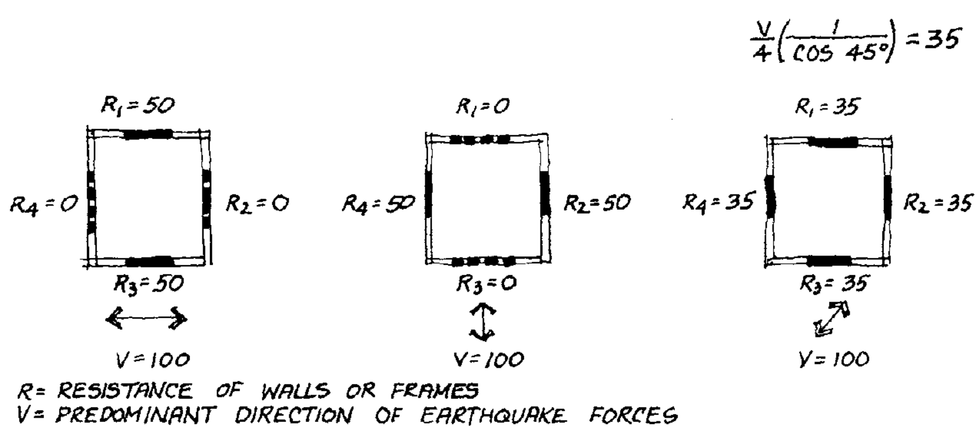


Figure IV-3. Consideration of forces along primary axes generally will include the worst loading cases.

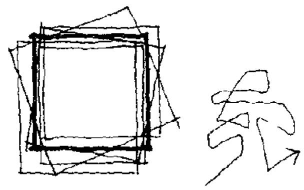


Figure IV-4. Earthquake forces - the reality.

It is important to note that in actuality, earthquake forces are much more complex than our diagrams would indicate. Ground motion is random, and the main direction of emphasis will only be axial by chance. In any event, total ground motion will always include non-axial components. So a better diagram for visualizing configuration reaction to ground motion might be Figure IV-4.

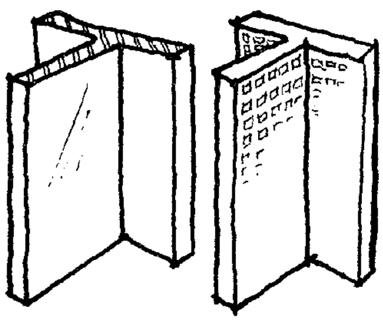


Figure IV-5. The block and the assembly.

A building is not a homogeneous block, but is an assembly of parts. Each part receives forces from adjoining parts through joints, horizontally and vertically. At the scale of a steel section or of wood blocks, the behavior of the T-shaped member is quite different from that of the full-size building (Figure IV-5). In the homogeneous section, the top flange of the T provides useful resistance along the axis of the leg of the T. For the full-size building, the overall T shape contributes nothing in the way of useful resistance. On the contrary, as we shall see, the wings of the T gives rise to torsion and incompatible deformations.

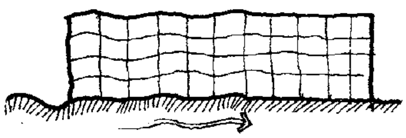


Figure IV-6. The large building.

Because the building is not a homogeneous block, in a larger building the ground motion will affect different parts of the building at times which are different enough to be significant, and may automatically induce torsion or incompatible movement even in a geometrically symmetrical building (Figure IV-6).

The building, being made of parts and connections, will have different localized strengths and stiffnesses, some calculated, some inadvertant caused by the interaction of non-structural elements or configuration influence. This further removes its behavior from that of a homogeneous material (Figure IV-7).

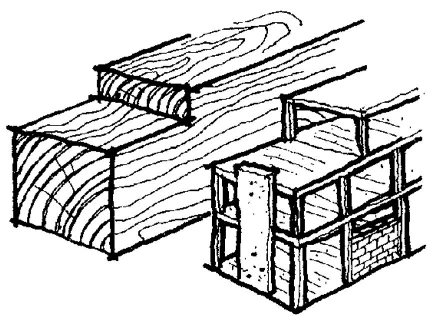


Figure IV-7. Localized strengths and stiffnesses.

B. Scale

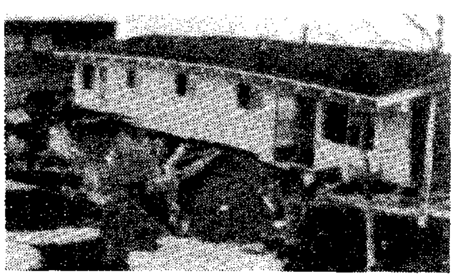


Figure IV-8. Small size allows askew house to remain more or less intact following 1964 Anchorage Alaska earthquake.

It is possible to violate configuration principles in a wood-frame house by introducing irregularities that would be serious problems in a large building, and yet produce a safe building with the inclusion of relatively inexpensive and unobstrusive provisions. This is because a small wood frame house is light in weight and inertial forces will be low. In addition, spans are small and, relative to the floor area, there will be a large number of walls to distribute the loads, and remedial measures, if knowledgeably designed, can be small in scale.

Houses in landslide areas of Anchorage which not only were severely shaken in the 1964 earthquake but which also slid dozens of feet ending up in various askew positions looking like grounded boats (Figure IV-8), probably had their small size, and relatively light

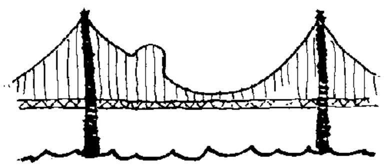


Figure IV-9. The designer's bridge.

weight, primarily to thank for remaining more or less intact, even though they were never designed to protect occupants from such an eventuality.

Alex Tarics has compared the house with a structure at opposite ends of the size spectrum: if the designer had wanted to put a kink in the shape of the Golden Gate Bridge (Figure IV-9), it would have been physically impossible. For a larger building, the violation of basic layout and proportion principles exacts an increasingly severe cost, and as the forces become greater, good performance cannot be relied upon as in an equivalent building of better configuration. This does not mean that small buildings do not pose significant problems as well.

The problem of scale is clearly exemplified by a pendulum: without knowing its absolute dimensions it is impossible to guess at what rate a pendulum will swing back and forth. If the weight is a marble and the string only a few inches long it is easy to imagine the pendulum completing more than one to and fro cycle in a second, whereas if the weight is a wrecking ball and the length 100', one immediately begins to visualize a period of several seconds (Figure IV-10).

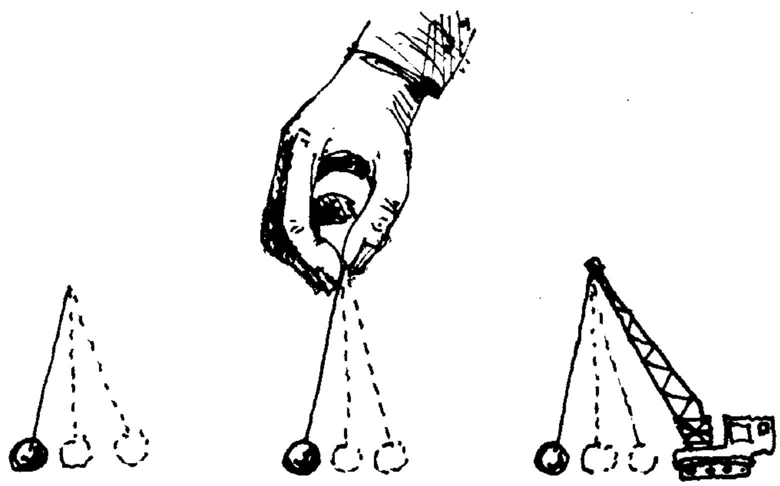


Figure IV-10. The scale of a pendulum.

This interrelationship of variables defeats the attempt quickly to compare one size building with another by simply invoking the ceteris paribus assumption that all variables except one can be held constant. The effects of size on gravity forces is easier to analyze than the effects of size on seismic forces.

With this major qualification, some discussion of size alone can be attempted. As the absolute size of a structure increases, the number of alternatives for its structural solution decrease. A bridge of 300 feet may be built as a beam, arch, truss, or suspension system, but a span of 3000 feet will demand a suspension bridge.

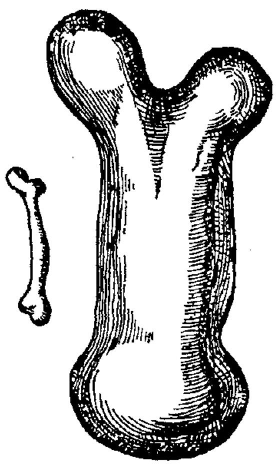


Figure IV-11. Galileo's sketch.

It is not possible to alter the size of a structure and it components and still retain the same structural behavior. This basic principle of physics was not elucidated until Galileo pointed out in 1637 (1):

"...nor can nature produce trees of extraordinary size because the branches would break down under their own weight; so also it would be impossible to build up the bony structures of men, horses, or other animals so as to hold together and perform their normal functions if these animals were to be increased enormously in height; for this increase in height can be accomplished only by employing a material which is harder and stronger than usual, or by enlarging the size of the bones, thus changing their shape... I have sketched a bone (Figure IV-11) whose natural length has been increased three times and whose thickness has been multiplied until, for a correspondingly large animal, it would perform the same function which the small bone performs for its small animal."

When less than full-size models are tested on shaking tables, their dimensions must be taken into account according to the principles of dynamical similitude. The mass, and hence load, must be multiplied, otherwise the structures' strengths would be overscaled, just as an ant carrying a twig cannot serve as a scaled down model of a person carrying a tree.

C. Height

Increasing the height of a building may seem equivalent to increasing the span of a cantilever beam, and it is - all other things being equal. The problem with the analogy is that as a building grows taller its period generally increases, and a change in period means a change - either upward or downward - of the level of response and size of forces. It is unlikely that an earthquake could create sustained high acceleration motions with predominant periods of two seconds, for example, whereas past earthquakes appear to be quite capable of concentrating their energies in the vicinity of one-half second.

Hence a building over twenty stories in height, which would probably have a fundamental period over one second and approaching or exceeding two seconds, would probably experience a lesser effective acceleration of its mass than a five to ten story structure which had a period of half a second. The period of a building is not solely a function of height but also of such factors as height-to-depth ratio, story heights, type of structural systems and materials, and the amount and distribution of mass. Hence changing the size of a building may simultaneously change one or more of these variables, change the period, and hence increase or decrease the seismic forces.

Although a 100 foot height limit throughout Japan was enforced until 1964, a 150 foot/13 story limit was the maximum in Los Angeles until 1957, and 80 feet and later 100 feet in San Francisco, height is rarely singled out as a variable to be controlled to mitigate the earthquake problem. Presently the approach is not to legislate seismic height limits but to legislate more specific seismic design and performance criteria. Generally, urban design, real estate, or programmatic factors will be more significant, and earthquake performance must be engineered with the height predetermined. When there is some latitude in site utilization, and when the earthquake factor is especially important, building height may be strongly influenced: Loma Linda hospital, discussed in Chapter XIII, and the IBM complex in Santa Teresa, California (2) are two recent large buildings that were limited to four stories, primarily in response to seismic planning requirements (Figure IV-12).

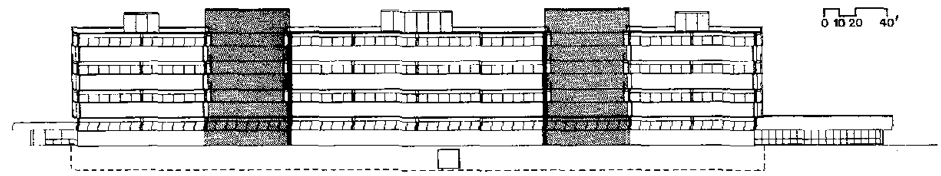


Figure IV-12. Section of Veterans Administration Hospital, Loma Linda, California.

Many engineers are more apprehensive about the performance of 5-15 story mid-height buildings in a major earthquake than that of taller buildings over about 20 stories. While many of the mid-rise buildings are cheaply and speculatively built, the taller buildings generally have more investment in engineering design. In a high seismic zone, the code will force the building over 160' tall to have a ductile moment-resisting frame.

D. Horizontal Size

It is easy to visualize the overturning forces associated with height as a seismic problem, but large plan areas can also be detrimental. When the plan becomes extremely large, even if it is a symmetrical, simple shape, the building can have trouble responding as one unit to earth vibrations.

S. Polyakov, in his 1974 work on Soviet seismic practice, points out that simple assumptions concerning ground motion may not hold true for extremely long buildings (3).

"In determining seismic forces, it is usually assumed that the structure vibrates as a system in which all points of a plan at the same level and at the same moment in time are in the same phase of displacement, velocity and acceleration, their amplitude being the same. In reality, since the propagation of seismic waves is not instantaneous but has a final [finite] velocity depending on the density of the ground and the characteristics of the structural elements, the various parts of the base of the building along its length vibrate asynchronously with different accelerations, thus causing additional longitudinal compressive-tensile stresses and horizontal displacements. Other conditions being equal, the longer the building, the greater the probability of these stresses and the greater their effect..."

The Soviet building code lists specific maximum lengths according to zones and construction types. Pre-existing temperature stresses, and pre-existing or earthquake-induced settlement stresses, are also greater in buildings with large plan dimensions, and these stresses can be additive to the stresses induced by lateral forces.

Polyakov also points out (4):

"An increase in the length of a building increases the stresses in a floor working as a horizontal distribution diaphragm in a transverse direction. The rigidity of the floor may not be sufficient to redistribute the horizontal load during an earthquake from weaker or damaged supporting elements of the building to stronger elements or those with minor damage."

Unless there are numerous interior lateral force resisting elements, large plan buildings impose unusually severe requirements on their diaphragms, which have large lateral spans, and can build up large forces to be resisted by shear walls or frames. The solution is to add walls or frames that will reduce the span of the diaphragm, although it is recognized that this may introduce problems in the use of the building.

For recent seismic rehabilitation of a long, narrow, low, university building with shear walls at the ends, structural analysis concluded that the best approach was to add two interior cross walls to shorten the length-wise span of the diaphragm (Figure IV-13). This recommended step was the most important part of the solution, and comprised 90% of the estimated cost of the renovation. In other words, if the building had been designed initially with better wall/diaphragm ratio, 90% of its problem would have been obviated.

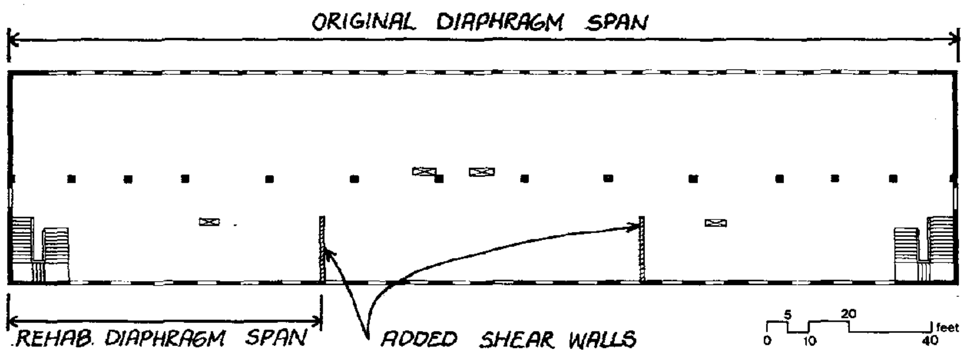


Figure IV-13. Addition of shear walls to decrease span of diaphragm, Veihmeyer Hall, University of California at Davis.

E. Proportion

In seismic design, the proportions of a building may be more important than its absolute size. For tall buildings the slenderness ratio (height/depth) of a building, calculated in the same way as for an individual column, is a more important consideration than just height alone.

Dowrick suggests attempting to limit the height/depth ratio to three or four, explaining (5):

"The more slender a building the worse the overturning effects of an earthquake and the greater the earthquake stresses in the outer columns, particularly the overturning compressive forces which can be very difficult to deal with."

Since interior planning requirements for offices and apartments require that most high rises are at least 60 feet in width, a slenderness ratio of four allows approximately twenty stories, so that for most conventional buildings this rule is automatically observed (Figure IV-14).

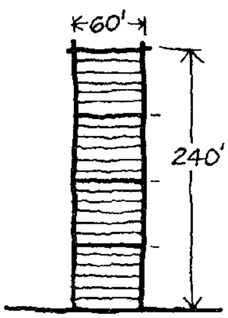


Figure IV-14. 4:1 height/depth ratio.

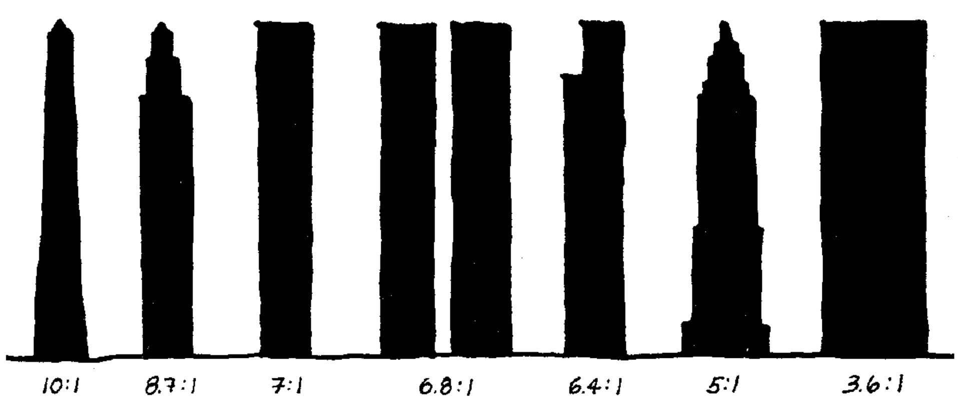


Figure IV-15. Comparative slenderness ratios. From left to right: Washington Monument, Woolworth Building, Pirelli Building, World Trade Center, Sears Tower, Empire State Building, and the U.S. Steel Building. (Note, all are not drawn to same scale).

In general, our tall buildings are not as slender as our recollection would have us believe (Figure IV-15). The New York World Trade Center towers, with a slenderness ratio of 6.8, are exceptional and such unusual buildings can generally afford a very high level of structural design. Unusual sites however, may sometimes produce a high slenderness ratio, even though the building is not unusually high (Figure IV-16).

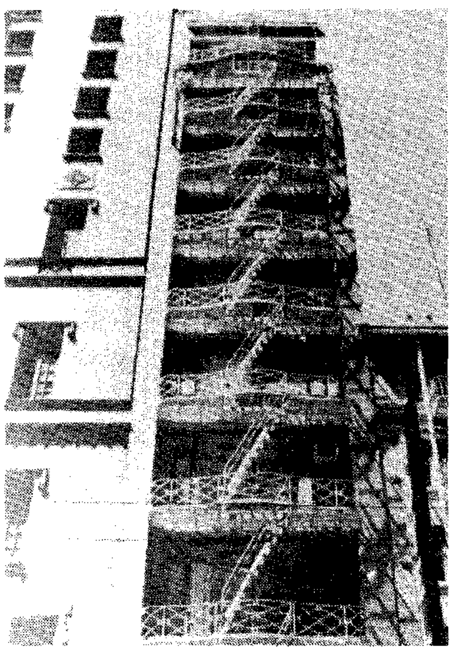


Figure IV-16. Small site leads to a slender building, San Francisco, California.

The plan equivalent of the height/depth, or slenderness ratio, is the aspect ratio. The same generalization holds true: long slender forms are undesirable. If bracing is located only around the exterior, then the longitudinal axis will be quite stiff but the transverse axis, having only two end walls or frames located far apart, will be quite flexible. The diaphragm must span a great distance, and will act like a long slender beam, whereas the assumptions used to analyze diaphrams, presume short shear beam behavior (Figure IV-17).

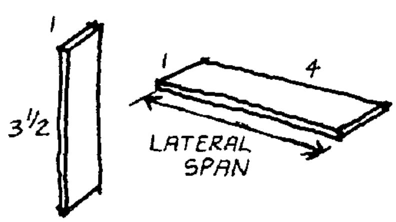


Figure IV-17. UBC proportional limits for plywood diaphragms (nailed all edges).

In the case of 15 California school buildings analyzed by John Blume, Roland Sharpe, and Eric Elsesser, the authors found that the typical shape was a long rectangular building containing side-by-side classrooms. However, the short axis was typically stiffer than the long axis (6).

"Single-story school buildings are generally more rigid in the transverse or short direction because of numerous transverse shear elements, whereas the rigidity of multi-story structures is more nearly equal in each direction. The longitudinal direction of single-story buildings is therefore more critical to the spectral exposure of an earthquake of random origin."

The transverse axis was generally more rigid than the predominant frequency of expected ground motion, and hence escaped the quasi-resonance range by having a shorter period than the expected ground motion. The longitudinal axis, although of greater depth or length, had more glazing area in its walls, whereas the transverse axis had evenly spaced solid cross walls between classrooms, and the period of the longitudinal axis could be long enough to approximate that of the ground motion, with consequent quasi-resonance.

This study cites two other configuration-related aspects of the quasi-resonance problem (7) (8).

"Typical school buildings, unlike taller buildings with longer natural periods of vibration, have improved seismic resistance with greater rigidity as well as strength; conversely, under damage from earthquake motion, their lengthening periods advance into the more critical range of spectral exposure rather than the less critical as is the case for buildings of longer initial period."

"Long, narrow diaphragms tend to have periods that 'tune in' to the most critical part of the earthquake spectrum. Not only are such elements subject to damage, but their reactions affect adjoining parts such as wall supports."

F. Symmetry

The term symmetry denotes a geometrical property of building configuration. A building is symmetrical about two axes in plan if its geometry is identical on either side of whichever axis is being considered. Such a building would be perfectly symmetrical. A building may be symmetrical about one axis only - such a building will be geometrically identical about this axis, but of dissimilar geometry about any other axis that may be drawn (Figure IV-18). Structural symmetry means that the center of mass and center of resistance are located at the same point.

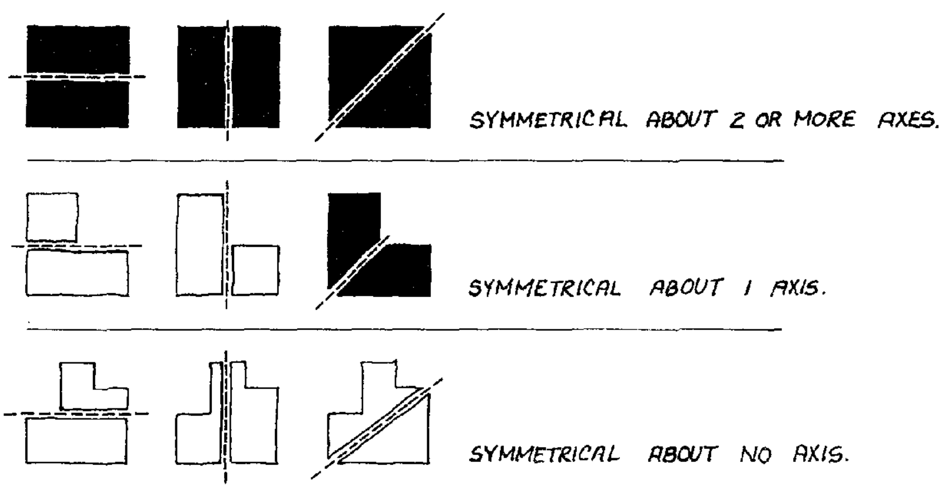


Figure IV-18. Symmetry in plan.

Symmetry about the elevational axis may occur, but it is of less dynamic significance than plan symmetry. In fact, in purely dynamic terms, a building cannot be perfectly symmetrical because it is fixed where it is attached to the ground and free at its other end. Furthermore, one could argue that geometric symmetry about two axes is no intrinsic advantage in the elevational plane and that specific forms of single axis symmetry will be more beneficial (Figure IV-19). For example, the pyramid has the intrinsic advantage that its mass reduces continuously with height.

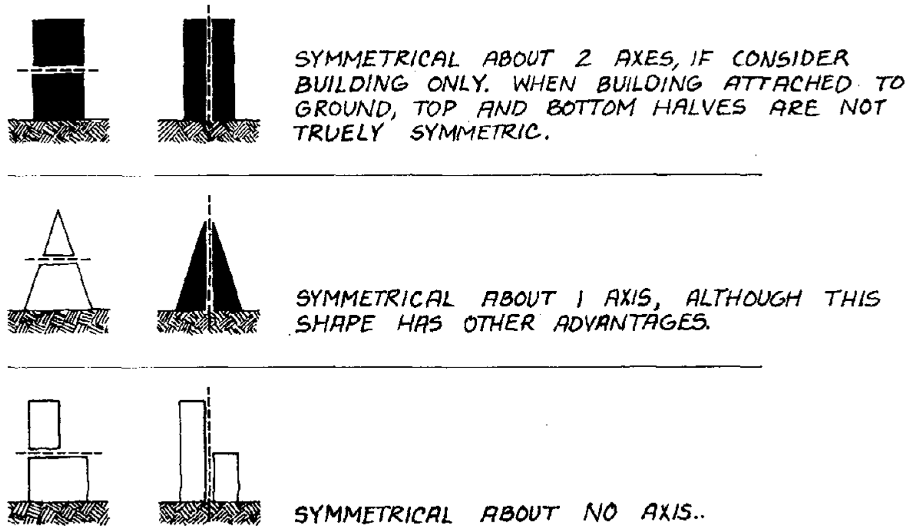


Figure IV-19. Symmetry in elevation.

The single admonition that occurs in all codes and in textbooks that discuss configuration is that symmetrical forms are preferred to those that are non-symmetrical. The two basic reasons for this can be derived from some of the issues discussed earlier. The first is that asymmetry will, purely in geometrical terms, tend to produce eccentricity between the center of mass and the center of rigidity, and torsion will result. Torsion may result from other non-geometrical causes (such as variations of weight distribution in a symmetrical structure) but asymmetry will almost inevitably lead to torsion.

The second reason is that asymmetry will tend to lead to stress concentrations. The most obvious example of this is the concentration of stress at the notch of a re-entrant corner (see Chapter VI, Section B). However, a building with re-entrant corners is not necessarily asymmetrical (a cruciform building may be symmetrical), but it is irregular, as defined, for example, in the SEAOC commentary discussed earlier. Thus we see that symmetry is not sufficent on its own, and it is only when it is combined with simplicity (in the specific way in which it is defined in Appendix 1 of this study as a 'convex' type configuration) that symmetrical form tends to eliminate stress concentrations.

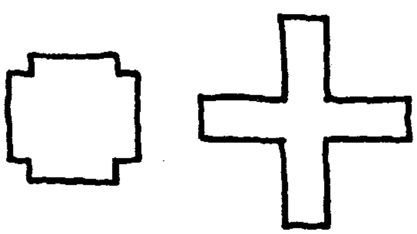


Figure IV-20. Symmetry is not enough - good and bad forms, both of which are symmetrical.

The plans shown (Figure IV-20) are both perfectly symmetrical about 2 axes; if the wings are very short, as on the left, the configuration will approximate the excellent simple symmetrical shape of a square. If the wings are very long, the re-entrant corners will introduce severe stress concentrations and torsion.

Nevertheless, with the above provisio, it is true that as the building becomes more symmetrical, its tendency to suffer torsion and stress concentration will reduce; and performance under seismic loads will tend to become less difficult to analyze and more predictable. This suggests that when safety is to be maintained with economy of design and construction, then symmetrical shapes are *much* to be *preferred*. But these tendencies *must* not be *mistaken* for an axiom that the symmetrical building will not suffer torsion.

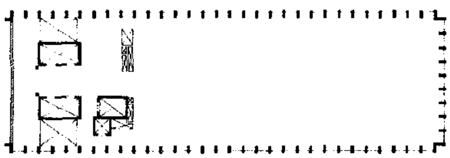


Figure IV-21. False symmetry: Banco Central, Managua, Nicaragua.

The effects of symmetricality refer not only to the overall building shape, but to details of its design and construction. Study of building performance in past earthquakes indicates that performance is sensitive to quite small variations in symmetry. This is particularly true in relation to shear wall design and where service cores are designed as shear walls to act as major lateral resistant elements.

It is not uncommon for major elements, such as service cores, to be arranged asymmetrically within an overall symmetrical configuration (Figure IV-21). The term 'false symmetry' is used to identify this condition and to emphasize that symmetry extends beyond the simple geometry of exterior form into the internal arrangements of resisting elements and non-structural components. This issue is discussed further in Chapter V, Section B.

Conversely, it is possible for a building to appear to be asymmetrical, but its resistance systems to be designed in such a way that dynamically the building acts in a symmetrical manner, and the likelihood of significant torsion is minimized. In fact, this is the correct design approach when faced with an asymmetrical configuration that cannot be changed (Figure IV-22).

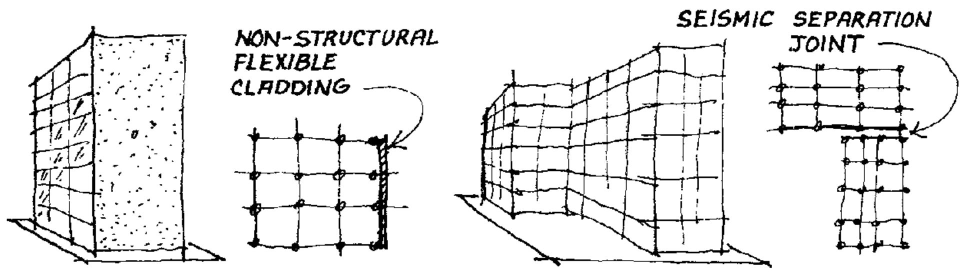


Figure IV-22. Apparently asymmetrical configurations which are actually symmetrical.

G. Distribution and Concentration

Although the two plans shown (Figure IV-23) are both symmetrical, have no re-entrant corners, and are the same size, the design on the right - assuming equivalent materials, details, and construction quality - is intrinsically superior as a seismic design. It has more columns and column-beam joints to share the load, the beam spans are shorter, and the resisting elements are evenly distributed.

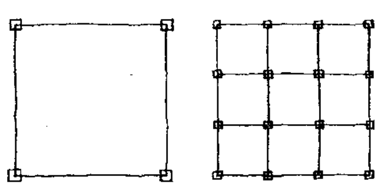


Figure IV-23. Distribution of loads.

Even under the controlled conditions of a testing laboratory, two "identical" reinforced concrete columns will not fail at exactly the same ultimate load. Under real conditions, many such variations can be expected; but in a building with well-distributed resistance, the elements will equally share the loads. If there are many elements rather than few, when one member begins to fail, there will be many other members to provide the necessary resistance. Hence configurations which concentrate earthquake forces in ways that result in the build-up of successively greater forces which are applied to a decreasing number of members, are obviously at an inherent disadvantage.

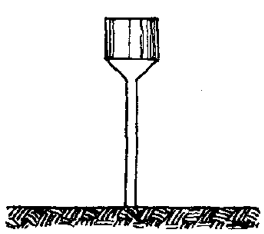


Figure IV-24. The inverted pendulum.

An extreme (though not uncommon) example is that of the single column elevated water tank (Figure IV-24). This represents an inverted pendulum, in which 100% of the lateral and vertical resistance is concentrated in a single member. There is no alternative load path. There may be reasons to occasionally design water tanks this way, but sharing the load widely is always a valid principle.

H. Structural Plan Density

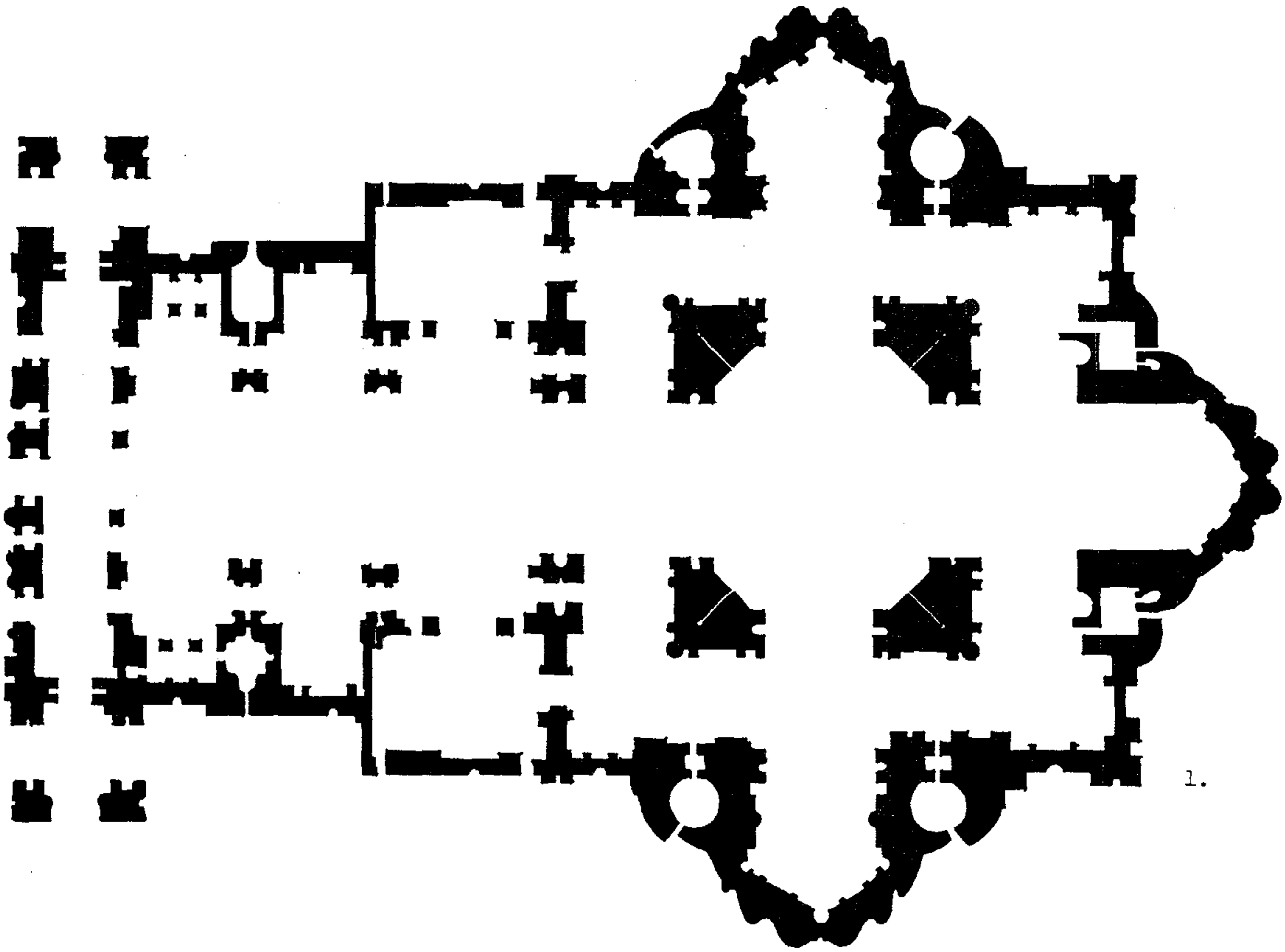
The size and density of structural elements in the buildings of former centuries is strikingly greater than in today's buildings. Structural technology has allowed us, and programmatic, real estate and aesthetic principles have motivated us, to continuously push this trend to further limits.

Although in tall flexible buildings, which can vibrate significantly in their higher modes (snake-like motions), causing maximum forces to occur at places which are not intuitively obvious, earthquake forces are generally greatest at the ground level. The bottom story is required to carry its own lateral load in addition to the shear forces of all the stories above, which is analogous to the downward build-up of vertical gravity loads. At this same lower level, building programmatic and aesthetic criteria are often imposed on the building that demand the removal of as much solid material as possible.

Familiar examples include the hovering cantilevered box; the box on stilts; the many-walled, cellular apartment house or hotel that sits on an open, sparsely columned garage; inverted pyramids or inverted setback shapes. These are all the opposite of the most efficient seismic configuration, which would provide the greatest intensity of vertical resistant elements at the base, where they are most needed. One can also question the validity of 'aesthetic criteria' that ignore good seismic design. To the engineer such buildings are arbitrary and inherently "ugly" buildings designed in ignorance.

An interesting statistical measure in this regard is the ground level "structural plan density," defined as the total area of all vertical structural elements - columns, walls, braces - divided by the gross floor area. In a typical contemporary building, this percentage is at its minimum value in designs that use a moment resistant frame, even including the rectangular fireproofing outline of columns if steel is used.

For instance, a typical ten to twenty story, moment-resisting steel or concrete frame building will touch the ground with its columns over 1% or less of its plan area, and combination frame-shear wall designs will typically reach structural plan densities at ground level of only about 2%. Even for a multi-story office building relying on shear walls alone, the ratio will probably only reach about 3%. The densely filled-in "footprints" of buildings of previous eras present a striking contrast: the ground level structural plan density can go as high as 50% as in the case of the Temple of Khons in Egypt or the Taj Mahal; the ratio for St. Peter's is about 25%; for Santa Sophia, the Parthenon, and the Pantheon 20%; and for Chartres 15%. The 16-story Monadnock Building, built just prior to the advent of the complete metal frame skyscraper and which used exterior bearing walls of brick six feet thick at the ground level, has a ratio of 15% (Figure IV-25).



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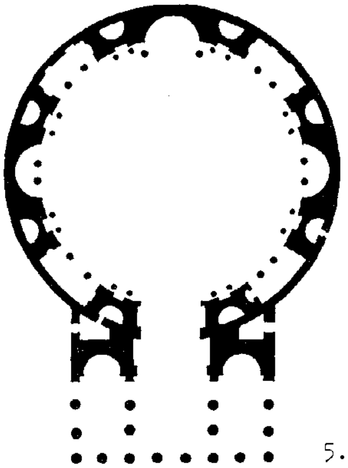

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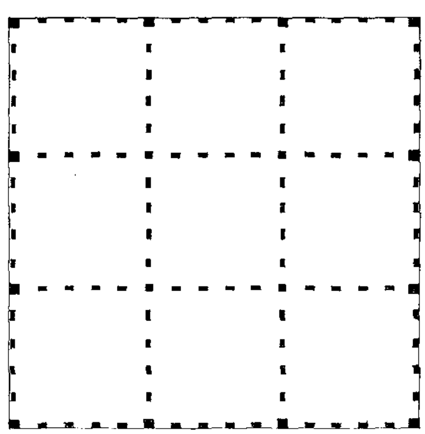

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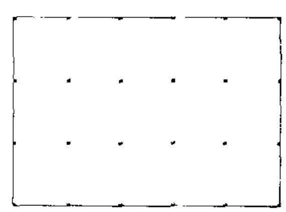

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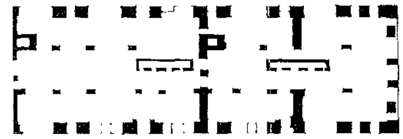

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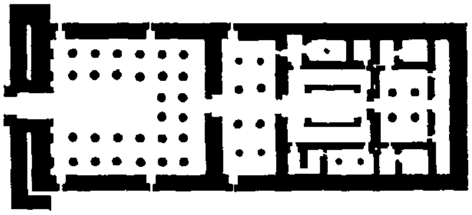

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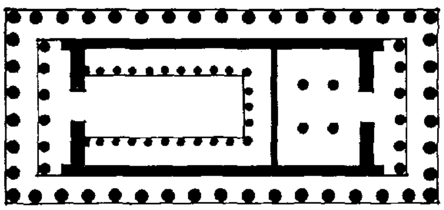

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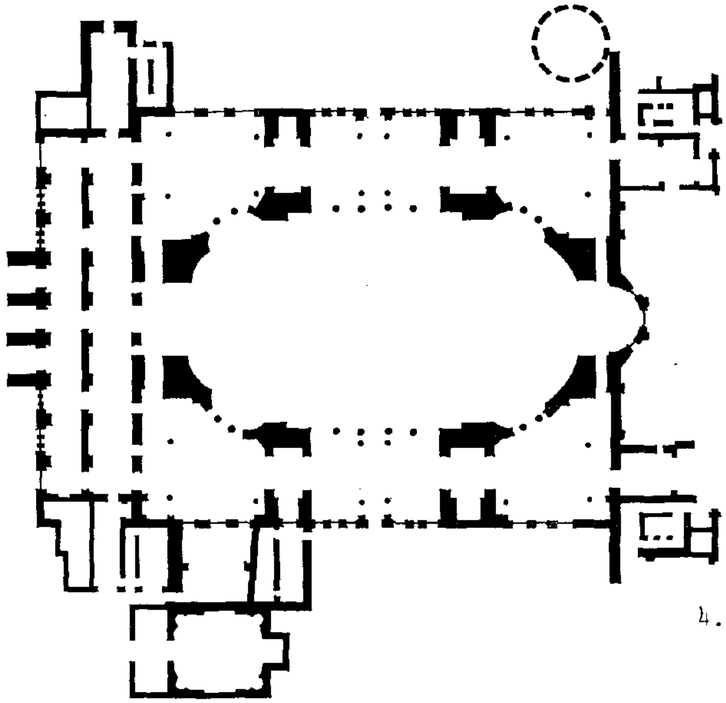

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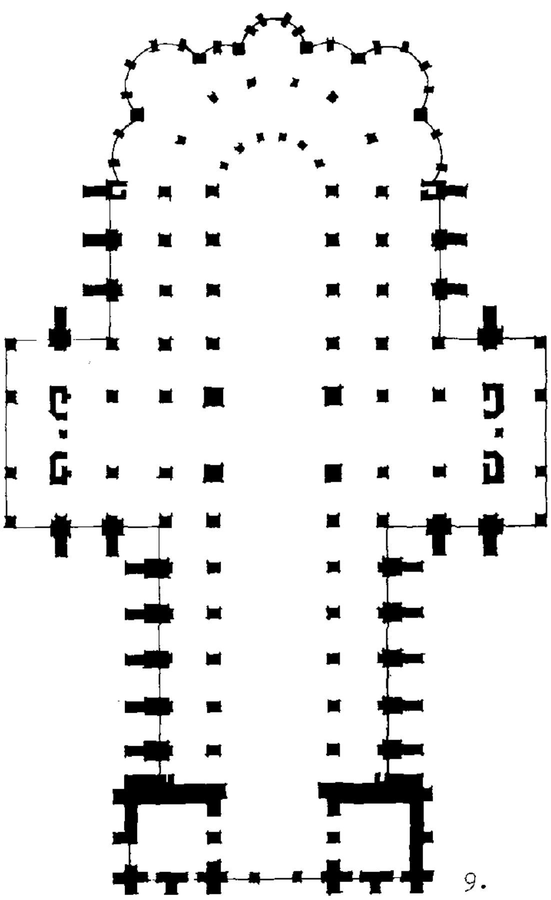

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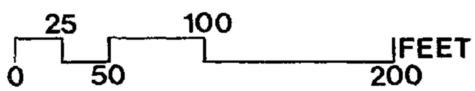
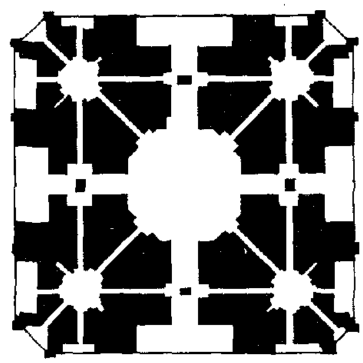




10.

Figure IV-25. Structural plan density.

Building, City, Date	Ground Level Structural Plan Density
1. St. Peter's, Rome, 1506-1626	25%
2. Temple of Khons, Karnak, 1198 B.C.	50
3. Parthenon, Athens, 447-432 B.C.	20
4. Santa Sophia, Istanbul, 532-537	20
5. Pantheon, Rome, 120-124	20
6. Sears Building, Chicago, 1974	2
7. typical contemporary steel high rise, 1975	.2
8. Monadnock Building, Chicago, 1889-1891	15
9. Chartres Cathedral, Chartres, 1194-1260	15
10. Taj Mahal, Agra, 1630-1653	50

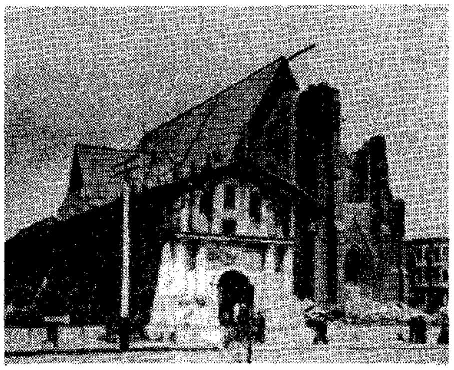


Figure IV-26. Mission Dolores, left, undamaged in 1906 San Francisco California earthquake although constructed in 1777. Newer church, right, was severely damaged.

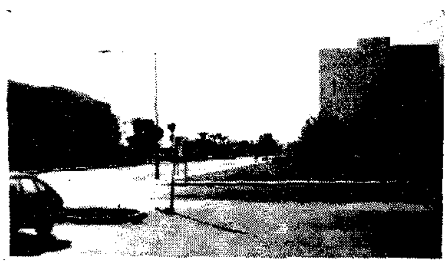


Figure IV-27. 1979 Imperial Valley California earthquake caused no structural damage to the old (1924) Imperial Valley Court House, left, but near collapse of the new (1969) Imperial County Services Building, right.

Structural engineers experienced in seismic rehabilitation work are often struck by the fact that many older buildings are almost seismically safe (Figures IV-26, 27). The "almost" part can be quite significant and quite costly, since for example, inadequate connections may provide the first mode of failure in a particular existing building and may be difficult to replace. But the chief factor which gives these older buildings some degree of earthquake resistance is usually their configuration, and the fact that they bring a great deal of material down to the ground by regular and direct routes. There is no reason for forces to take destructive short cuts since the intended paths are already direct. Simple, structurally logical configurations also often explain why buildings which "should" have collapsed in past earthquakes remained standing.

Analogous to structural plan density is the measure of the extent of walls in a structure. Surveys of damaged buildings in Japan and Turkey have indicated a clear relationship between the length of walls in a box system building and the amount of damage. This relationship has been dealt with in the seismic codes of these and other nations.

After the 1976 Caldiran and 1977 Palu earthquakes in Turkey, the damage to five brick shear wall buildings was assessed and compared to two ratios: the percentage of external wall openings, and the length of shear walls divided by the floor area. (This was done separately for both axes). Construction type was held approximately constant due to similar local practice. It was found that (9):

"an external wall opening ratio more than 40% causes damage in the walls. On the other hand, buildings with wall/floor ratios less than 25 cm/m^2 (about one inch per square foot) were also heavily damaged."

A survey of damaged reinforced concrete buildings in the 1968 Tokachi-oki earthquake in Japan revealed that (10):

"In framed structures having some seismic walls, the extent of damage differed by their quantity of walls. In buildings which had a fair amount of walls, for instance, about 5 cm/m^2 (Approximately .2in./sq.ft.)... arranged in good balance, the walls themselves showed many shear cracks but columns and girders of the frames were kept safe with only slight cracks."

Toshio Shiga (11) refined this basic approach by calculating the nominal average shear stresses in first floor columns and walls that are deduced to have occurred in an earthquake.

To calculate the nominal average shear stresses, the building weight above the ground is assumed to be the floor area times a certain weight factor (1000 kg/m^2 = 200 psf), a base shear coefficient is chosen, and the plan area of first story columns and walls is summated. Undamaged reinforced concrete buildings were found to have either a "wall-area index of more than 30 cm/m^2 (1 in./sq.ft.) or (an) average shear stress of less than 12 kg/cm^2 (170 psi)."

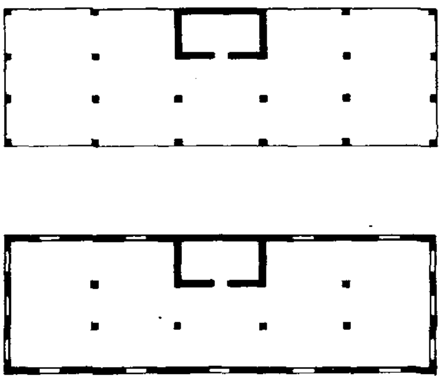


Figure IV-28. Wall length/floor area ratio quickly reveals different load/capacity factors of two buildings which both meet minimum code requirements.

Comparison of the above examples show inconsistencies resulting from differences in building construction type and configuration. Such approaches, at present, must be regarded in the realm of research, but they offer a potentially useful type of measure with which to assess seismic configurations.

The validity of using a simple wall length/floor area ratio depends upon two general factors: the floor area must correlate well with the building's mass and hence loading, and the wall length must be an accurate indicator of the resistance provided by the bracing system. Variations in the exact arrangement of walls (whether symmetrical or not, etc.), material properties, connection and other details, absolute dimensions, and diaphragm characteristics, also may effect performance, but the basic wall length/floor area ratio seems to have a validity that, combined with its simplicity and directness make it too useful to dismiss. Two buildings which both meet the minimum requirements of a code may have different load/capacity factors, and given comparable characteristics discussed above, the wall length/floor area ratio may quickly reveal this fact, whereas merely ascertaining that both designs meet the code will not bring out this difference. Such a ratio, if refined for a variety of construction types, layouts, seismic zones etc., also contains much promise as a useful design tool (Figures IV-28, 29).

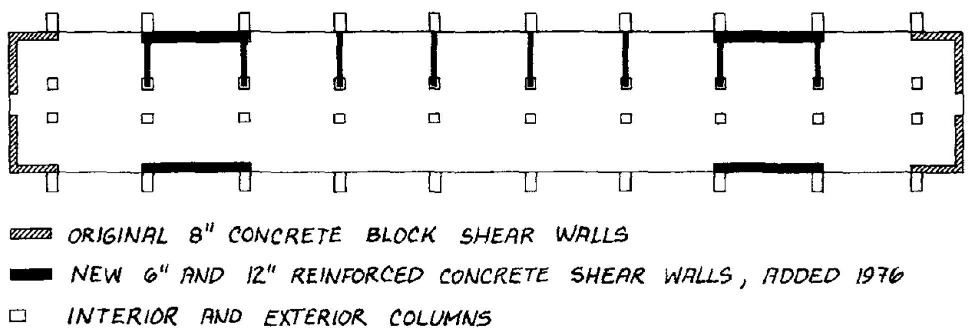


Figure IV-29. North Hall, University of California, Santa Barbara. After rehabilitation, building experienced about 1.0g acceleration at roof level in 1978 earthquake, with repairable though structural damage. How would it have performed without the added walls?

I. Corners

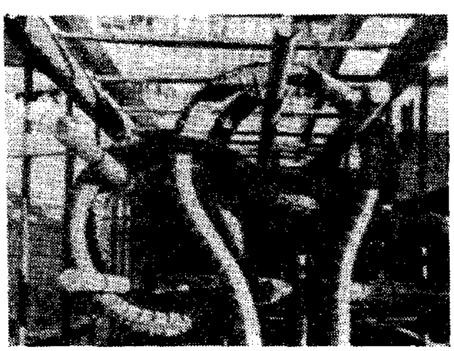


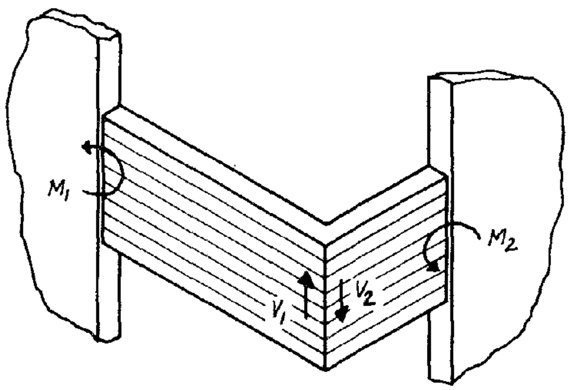
Figure IV-30. Example of special measures taken to reinforce intersection of spandrels at corner, Muir Medical Center, Los Angeles, California.

The corners of a building have their own special seismic problems. The re-entrant or inside corner (as in an L-shaped plan) will be discussed later. Outside corners can also experience problems due to orthogonal effects. Diagonally-oriented ground motion may stress the rest of the structure less, as discussed previously, but it can stress corners more than motion along the principal axes (Figure IV-30).

Clarkson Pinkham, in a summary of lessons learned from the 1971 San Fernando earthquake, has pointed out that (12):

"Particular attention should be given to corner columns of frames, with consideration given to simultaneous motions in both the vertical and horizontal directions. Corner columns should be designed conservatively."

It is also at the corners of a building that the deflection of a wall in one plane must interact with an incompatible deflection of a wall in a plane at a right angle. This may be accentuated by the absence of solid wall at the corner, as shown by a damage example from the 1964 Alaska earthquake illustrated in Figure IV-31.



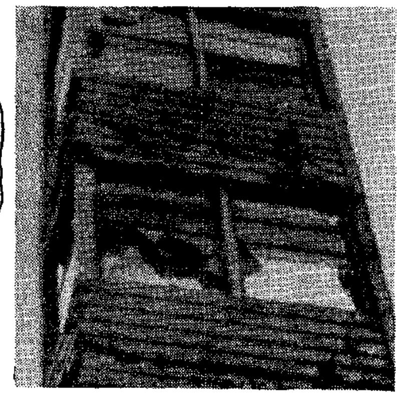


Figure IV-31. Differential motion produced damage at 'soft corner' of the 1200 L Apartment Building in the 1964 Anchorage Alaska earthquake.

J. Perimeter Resistance

In Figure IV-32, although both configurations are symmetrical and contain the same amount of shear wall, the precise location of the walls is significantly different. The walls on the right form greater level arms for resisting overturning and torsional moments.

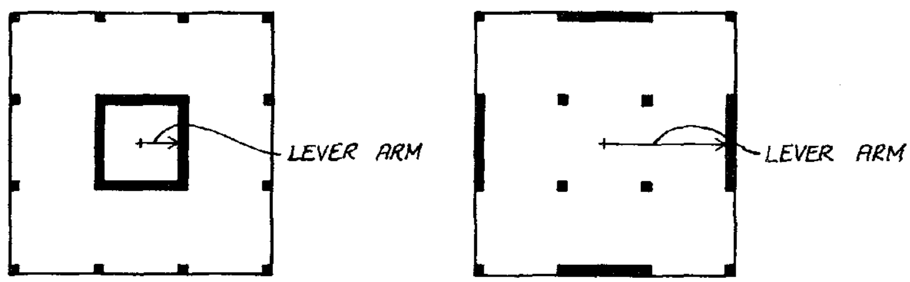


Figure IV-32. Shear wall location for resisting overturning and torsional moments.

Blume, Newmark and Corning advocate strong (though not necessarily shear walled) perimeters (13).

"It is strongly recommended that torsional phenomena be given serious attention in design. It is also recommended that tall buildings have symmetrical moment-resisting frames regardless of any walls, and that every building have as much lateral resistance as feasible in its outermost periphery of structural support. Modern curtain-wall buildings need particular attention to compensate for the lack of periphery resistance inherent in predecessor structures, the good behavior of which played a significant part in the evolution of earthquake codes."

In resisting torsion, with the center of twist of a symmetrical building located exactly in the geometrical center, the further the material is placed from the center, the greater the lever arm through which it acts, and hence the greater the resisting moment that can be generated. This implies that geometrically the most

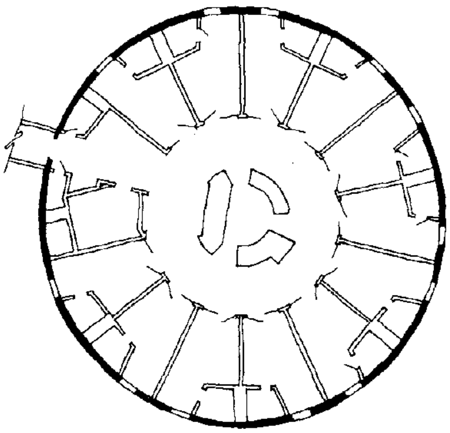
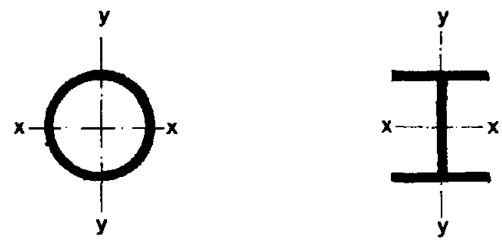


Figure IV-33. Placing resisting members on the perimeter, whenever possible, is desirable.

efficient layout is circular (Figure IV-33), though many other adequately efficient configurations are possible. Putting resisting members on the perimeter whenever possible is, however, always desirable, whether the members are walls, frames, or braced frames, and whether they have to resist direct lateral forces, torsion, or both.

That a circular plan of equal resistant capacity in all directions is theoretically ideal, is only an illustration of the uncertainty inherent in seismic design. To assume that earthquake forces will come from any direction is not also to say that every earthquake will shake the ground equally in all directions.

If one could know that earthquake forces would be exerted along one axis, then one could efficiently arrange material to resist this loading. We do this in dealing with gravitational forces, since their direction is quite predictable; hence we use I-sections for beams, in comparison to which circular beam sections would be quite inefficient (Figure IV-34). In this sense, the circular plan shape is a compromise solution rather than an ideal: it minimizes the maximum negative event that could occur.



TS 12 OD X .375	W12 X 50
$I_{x-x} = 279 in^4$	$I_{x-x} = 395 in^4$
$I_{y-y} = 279 in^4$	$I_{y-y} = 56 in^4$

Figure IV-34. For the same weight of material, the wide flange is 1-1/2 times as stiff as the pipe, along the X-X axis. If loaded sideways however, the pipe is 5 times stiffer than the wide flange section, and many times stronger in torsion.

K. Redundancy

Non-structural members which are forced to play a structural role should not be confused with the idea of redundancy. Redundant members are structural elements that under normal design conditions do not perform a structural function or are understressed relative to their strength, but which are capable of resisting lateral forces if called upon. They provide a useful means of obtaining an additional safety factor where there may be analytical uncertainties in design.

William Zuk (14) has singled out the existence of redundancies as the primary characteristic of fail-safe design. He cites many examples. Spider webs do not collapse even when half their strands are broken. Airplanes with gaping holes in their structures have still managed to fly. One of the most successful of such airplane structures was the geodesic structure designed by Barnes Wallis in England, originally for airships in the 1930's. This structure, analogous to that devised by Buckminster Fuller for geodesic domes, provided many alternative paths for stress relief in the event of damage, and contributed greatly to the success of Britain's Wellington bomber in World War II (Figure IV-35).

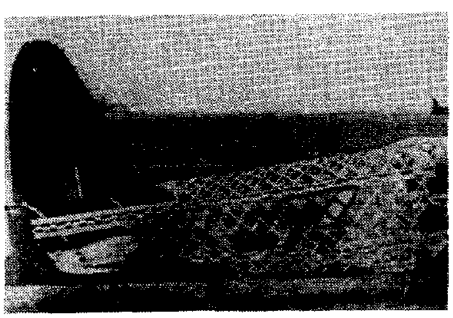


Figure IV-35. Damaged geodesic airplane structure.

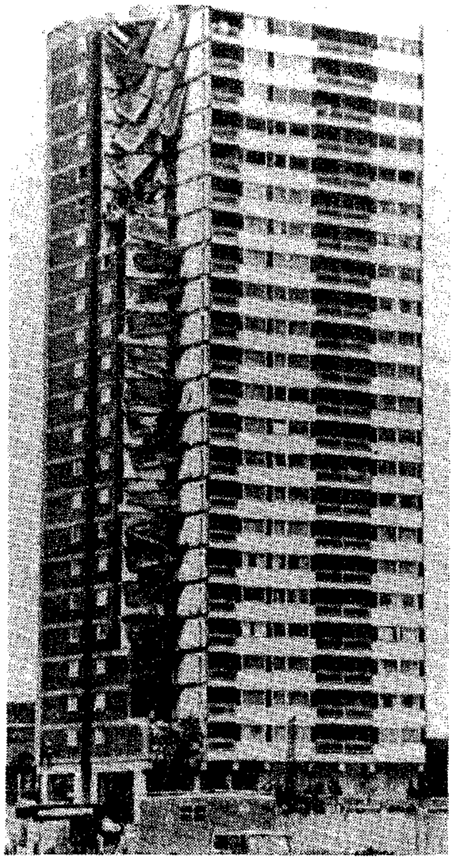


Figure IV-36. Ronan Point, London, England, 1968.

One might argue that the provision of redundancy represents a violation of the concepts of engineering economy and elegance, since it implies that some material will usually be idle or under-stressed. However, the concept recognizes the need to design for the uncalculated disaster as well as everyday service conditions.

A catastrophic example of the absence of redundancy was the progressive collapse of one corner of the Ronan Point apartment building in London in 1968. After a gas explosion knocked out one pre-cast panel on the eighteenth floor, destroying one corner of the building at this level, every corner in the four stories above collapsed because their loads had no alternate path to follow, and the impact of this collapse then progressively destroyed the corner area of each of the seventeen stories beneath (Figure IV-36). Similar seismic problems can arise with non-redundant, large panel structures.

Redundancy in earthquake design is significant in several aspects. The detailing of connections is often cited as a key factor, since the more integrated and interconnected a structure is, the more load redistribution possibilities there are. Configuration is also involved, since the number and location of resisting elements originates in the architectural design and establishes a potential for redistribution which can be made effective by proper structural detailing.

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V. Significant Irregularities in Simple Configurations

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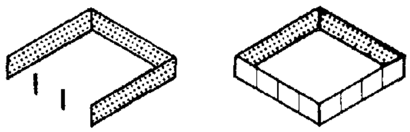
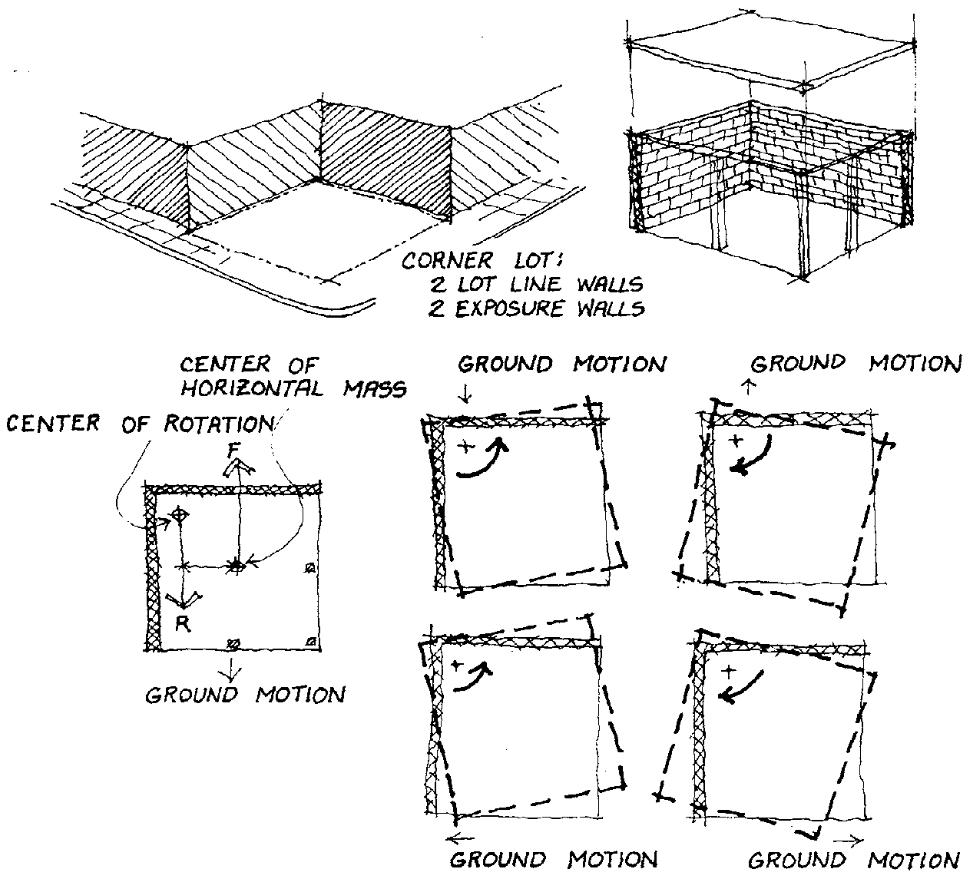
A. Variations in Perimeter Strength and Stiffness



This chapter deals with buildings whose configuration is geometrically simple, but nonetheless irregular for seismic design purposes. (See Appendix 1 for geometric definitions of "simple".)

A building's seismic behavior is strongly influenced by the nature of the perimeter design. If there is wide variation in strength and stiffness around the perimeter, the center of mass will not coincide with the center of resistance, and torsional forces will tend to cause the building to rotate around the center of resistance. This effect is illustrated in Figure V-1.



HORIZONTAL IMBALANCE = TORSION
RESISTANCE (R) & INERTIAL FORCE (F) ACT ECCENTRICALLY, RATHER THAN MEET "HEAD-ON."

Figure V-1. Unbalanced horizontal resistance.

This condition causes much building damage and collapse. Henry Degenkolb has discussed a form of this problem in a way that clearly emphasizes its effect (1).

"The effect of torsion can probably be best illustrated by one of the most common building constructions in the United States if not the world. The side walls are on property lines, the rear wall is either on a property line or faces an alley. The rear wall has minimum openings, if any, but the front wall with its display windows on the street is essentially open. When shaken by an earthquake, the rear and side walls are quite rigid but the front wall is very flexible, and the roof tends to twist. There have been some studies indicating that any columns in the front wall will be highly stressed in torsional shear in addition to the normal deflection loads and shears.

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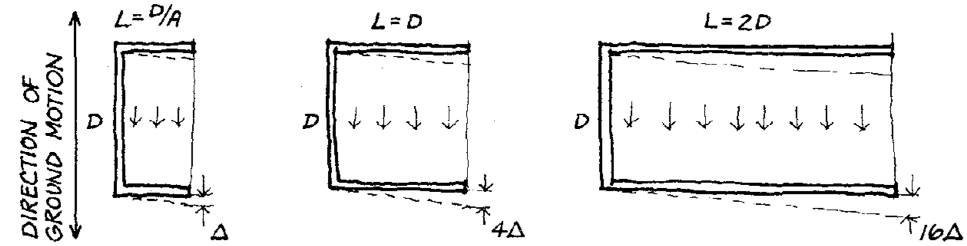


Figure V-2. Torsional deflection of building with 'soft front wall'.

"Figure V-2 shows the plans of three similar buildings, each with three shear walls so arranged that there is an open end and therefore major torsions on the buildings. If the buildings are similar, with uniform shear elements (uniform distribution of stiffness) and considering only shear deformations, it can rather simply be proved that the torsional deflection of the open end varies as the square of the length of the building. It is probable, but not proven, that buildings with a ratio of L/D equal to or about 1/2 or less should have little trouble due to torsion in an earthquake, since the total deflections including torsion will be about the same as the symmetrical loading of the earthquake in the perpendicular direction. With ratios of L/D above 1/2, the torsional deflections increase rapidly and damage will surely occur at the open end, unless specific precautions are taken."

Figure V-3 shows the increase in deflection caused by an increase in the span of a lateral cantilever.

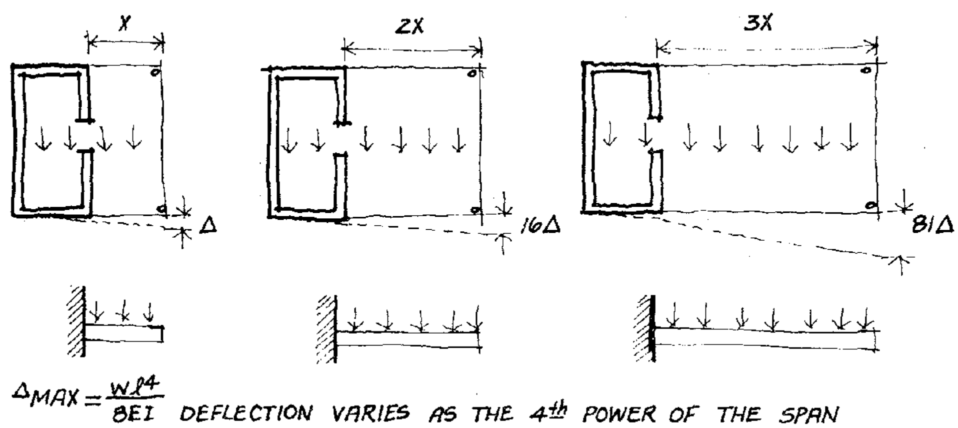


Figure V-3. Torsional deflection of diaphragm performing as lateral cantilever. Columns at free end presumed to have only minimal lateral stiffness. Flexible diaphragms, such as wood-sheathed roofs, are generally presumed incapable of carrying a torsional moment, but this does not apply in the cantilever case.

A classic instance of this kind of effect is that of the J.C. Penney building in Anchorage, in the 1964 Alaska earthquake. The building was so badly damaged by torsional forces that it had to be demolished (Figures V-4, 5, 6). The store was a five story building of reinforced concrete construction. The exterior wall was a combination of poured in place concrete, concrete block, and precast concrete non-structural panels which were heavy, but unable to take large stresses. Steinbrugge, Manning and Degenkolb discuss the source of this torsion (2):

"Torsional forces were not a significant factor in the first story, since shear walls were found along all street fronts. The upper stories, however, had a structurally open north wall, and large torsional forces would arise from the U-shaped shear wall bracing system when subjected to east-west lateral forces."

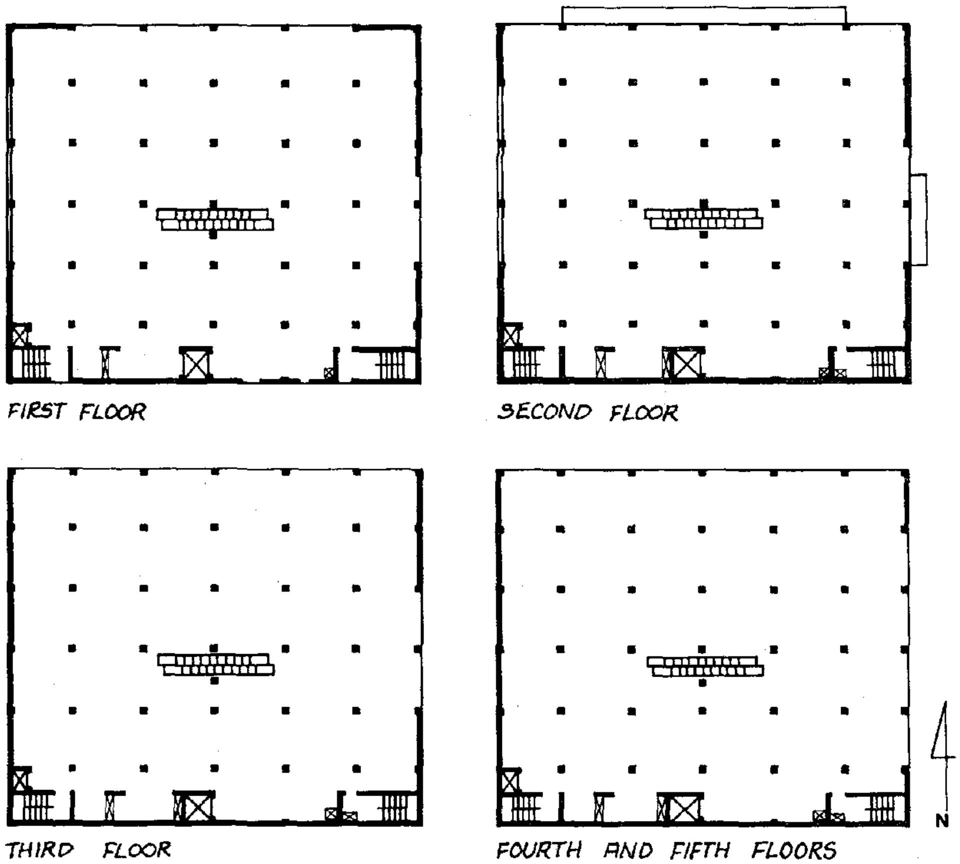
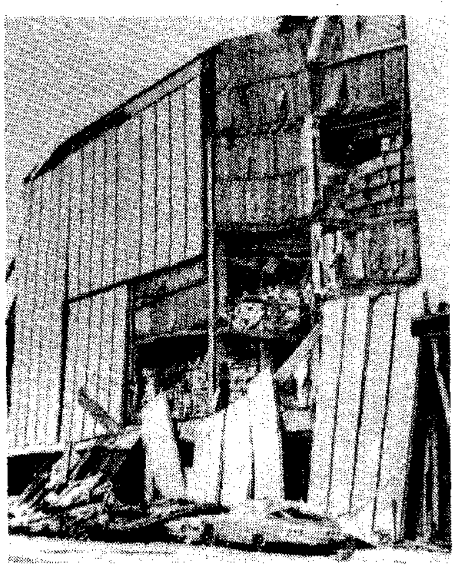


Figure V-4. Plans of J.C. Penney department store, damaged in the 1964 Anchorage Alaska earthquake.



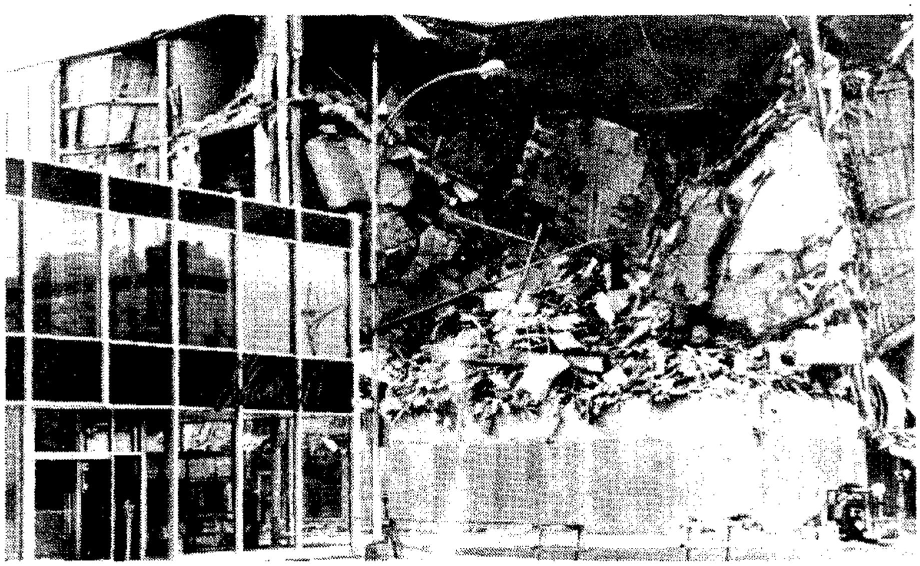


Figure V-5. Failure of heavy precast concrete non-structural panels which formed part of the exterior wall, at J.C. Penney building.

Figure V-6. Severely damaged northeast corner of the J.C. Penney building. Note undamaged curtain wall office building in foreground.

The Penney store is somewhat similar in configuration to Brock's Department Store, which suffered severe damage in the 1952 Kern County, California earthquake for similar reasons. Figures V-7 and 8 clearly show the main irregularity, which was created in response to the location and context of the lot on which it was built.

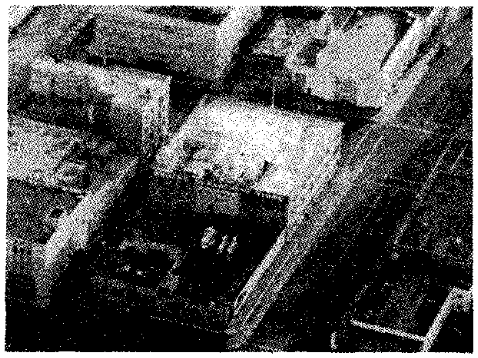
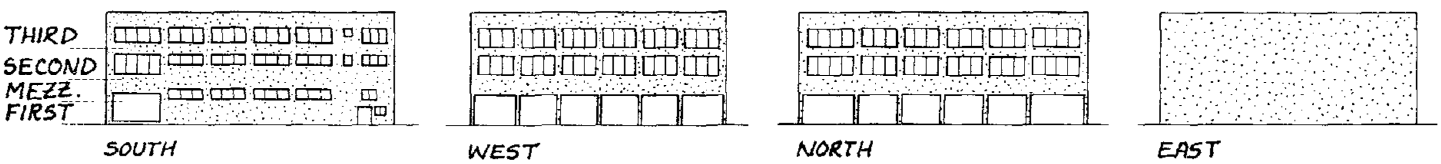


Figure V-7. Aerial view of Brock's Department Store, 1952 Kern County California earthquake.

Figure V-8. Brock's Department Store from the southwest.

The building fronted on 3 streets, and hence these were composed of frames infilled with windows. The fourth side was adjacent to a building next door and this wall was made a solid shear wall (Figure V-9). The inequity of stiffness in plan caused torsional damage. An analysis by Steinbrugge and Moran goes into more detail (3): "Calculations indicate that the south wall of the second story contained perhaps 80% to 90% of the east-west rigidity in that story."

Figure V-9. The four elevations of Brock's Department Store, showing variations in wall perforations.



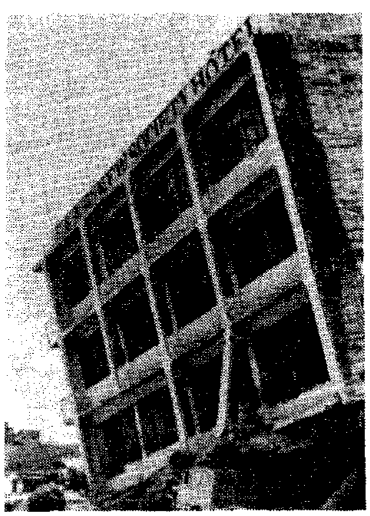


Figure V-10. An example from the 1976 Mindanao Philippines earthquake of a building located on a corner lot. The New Society Hotel had two sides of rigid shear walls and two framed sides of greater flexibility. The amount of torsional movement accompanying ground story collapse is readily apparent.

Figure V-10 shows a building in which torsion resulting from perimeter variation contributed to total ground floor collapse.

Open front design is common in buildings such as fire stations and motor maintenance shops, in which it is necessary to provide large doors for the movement of vehicles. In fire stations, it is particularly important to avoid distortion of the front frame, for if the doors cannot be raised, the fire station is out of action at a time when its equipment may be urgently needed. The fire station in Sendai, Japan, shown in Figure V-12, illustrates a good solution to this problem.

The object of any solution to this problem, is to reduce the possibility of torsion. Four alternative strategies can be employed.

The first strategy is to design a frame structure with approximately equal strength and stiffness for the entire perimeter. Opaque portions of the perimeter can be constructed of non-structural cladding, designed so that it does not affect the seismic performance of the frame (Figure V-11). This can be done either by using lightweight cladding, as in Figure V-12, or by ensuring that heavy materials - such as concrete or masonry - are isolated from the frame, as in Figure V-13.

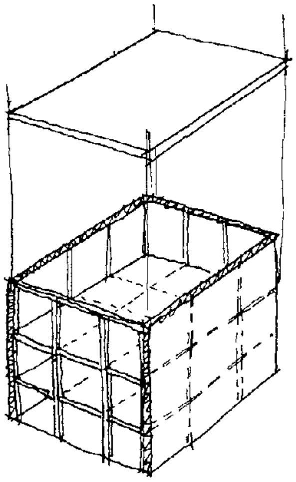


Figure V-11. Solution #1: Frame structure with entire perimeter of approximately equal strength and stiffness. Solid walls should be of non-structural cladding.

Figure V-12. Fire station in Sendai, Japan. The opaque side walls are constructed as lightly clad braced frames, creating an overall framed structural system. The lighter frame structure reduces forces: the frame as a whole must be designed to keep drift down to acceptable limits.

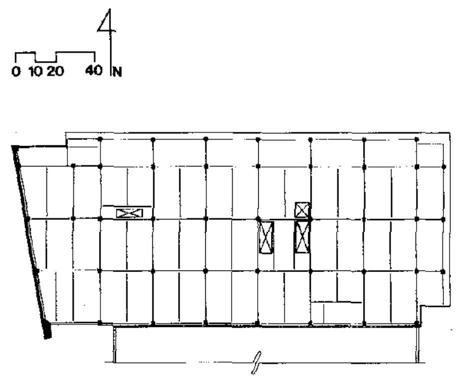


Figure V-13. KB Valley Center, Los Angeles, California. The rigid shear wall is separated from the three flexible walls, braced by frames, by a separation joint which permits independent movement.

A second approach is to increase the stiffness of the open facades by adding shear walls at or near the open face (Figure V-14). This solution is, of course, dependent on a design which permits this addition. An example of this approach is shown in Figure V-15.

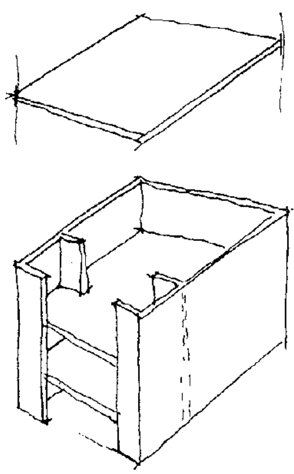


Figure V-14. Solution #2: Shear walls are added at or near the open face.

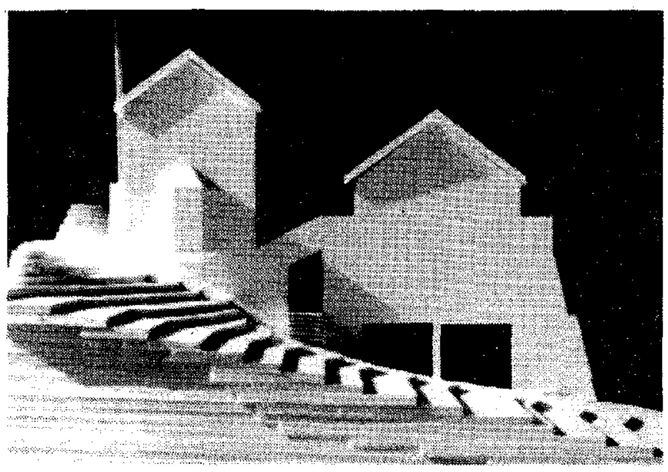


Figure V-15. Model of a house where buttresses have been added to compensate for large openings required by garage entrance.

A third solution is to use a very strong moment resisting or braced frame at the open front, which approaches the solid walls in stiffness (Figure V-16). The ability to do this will be dependent on the size of the facades: a long steel frame can never approach a long concrete wall in stiffness. This is, however, a good solution for wood frame structures, such as apartment houses with a ground floor garage area, because even a long steel frame can be made as stiff as plywood shear walls (Figure V-17).

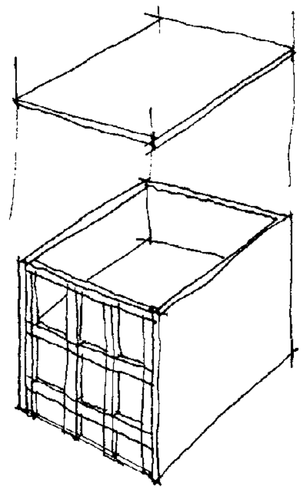


Figure V-16. Solution #3: Strong and stiff moment resisting or braced frame is designed for soft front wall.

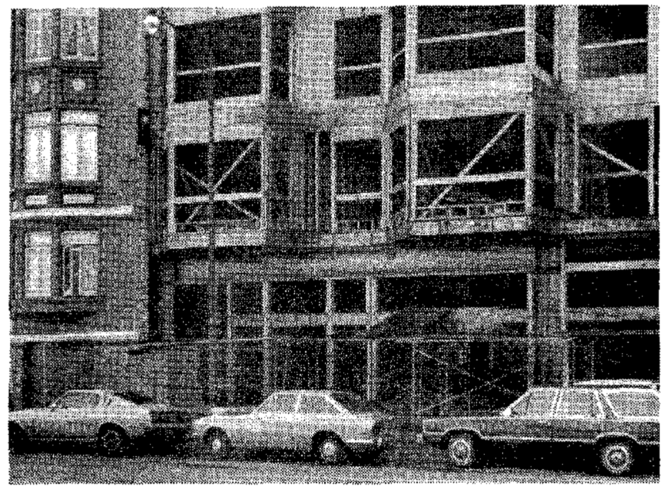


Figure V-17. Steel frame for garage openings allows front elevation to more closely approximate stiffness of plywood shear walls.

Finally, the possibility of torsion may be accepted and the structure designed to resist it (Figure V-18). This solution will only apply to relatively small structures with stiff diaphragms, which can be designed to act as a unit, as shown in Figure V-19.

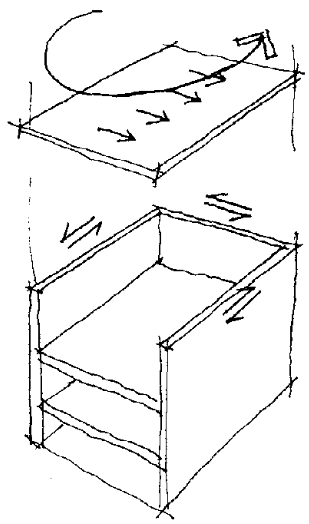


Figure V-18. Solution #4: Torsion is accepted, and building is designed to resist the forces and minimize the distortions they cause.

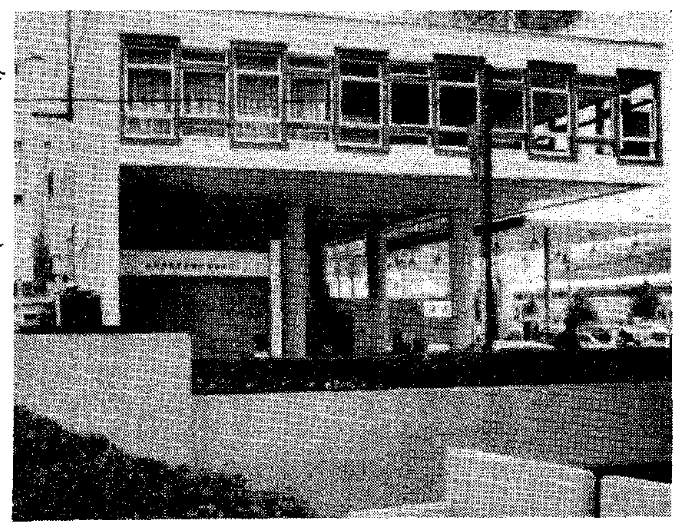
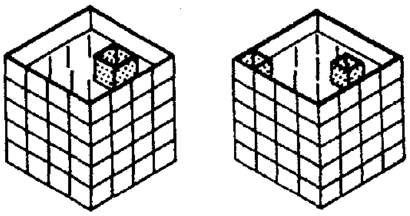


Figure V-19. A stiff structure designed to respond as a unit; undamaged in the 1978 Sendai Japan earthquake.

B. Core Location, False Symmetry



Internal shear walls generally conflict with the use flexibility or openness requirements; exterior shear walls may or may not fit in with other requirements which the building perimeter must perform, and only a relatively small amount of window penetrations are possible before the "shear wall" has been reduced to a frame. Hence, the most common location for shear walls in multi-story buildings is often the core.

However, the location and detailed design of this massive and stiff element then become extremely important in determining the seismic performance of the building. In particular, the location of the core in relation to the overall symmetry of the building is critical because asymmetrical core locations will tend to greatly increase the probability of torsion. Thus, in assessing the symmetry of a building configuration, we must look at more than the overall shape of the building, and should also investigate the location of all significant resisting elements.

We use the term 'false symmetry' to identify buildings which superficially appear to be simple, regular, and symmetrical in configuration but which, because of the arrangement of their resisting elements, are structurally asymmetrical. Two examples illustrate the major issues and their effects.

The Banco Central in Managua, Nicaragua, is a fifteen story, reinforced concrete building constructed in 1962 (Figure V-20). The building is a simple rectangular form, and relies on flexible frame action to resist lateral forces. However, in the east-west direction the elevator core walls, located at one end of the building are stiff enough to resist approximately 35% of the lateral forces.

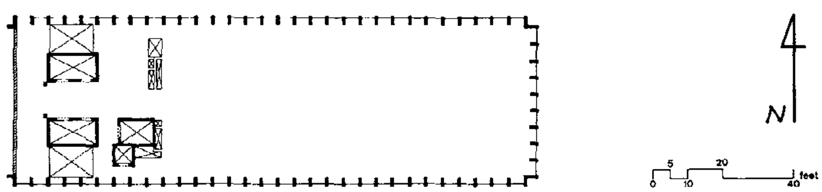


Figure V-20. Plan of Banco Central, Managua, Nicaragua. Overall building shape is symmetric, but placement of cores is not.

In the Managua earthquake of 1972, the building suffered significant damage, both structural and non-structural. The most serious structural damage was the cracking of the floor slab near the elevator cores and stairs. The predominant motion was in the east-west direction; the concrete core walls were stiffer than the frames and consequently the floors 'tore' at these points. Cracks up to 1/2 inch in width were found in almost all floors. In addition, this building suffered extreme non-structural damage due to the heavy shaking of the relatively flexible frame, shaking increased by imbalance caused by the off-center location of the core. Since the major direction of the ground shaking was east-west, the building fortuitously avoided major torsional effects due to the off-center location of the stiff resisting core.

The Four Seasons apartment building in Anchorage, Alaska, was a six-story lift slab reinforced concrete structure of simple, rectangular configuration with two symmetrically placed concrete cores; one for an elevator and one for a stair case. The southern core was partially disconnected from the diaphragm because of an open well at each floor (Figure V-21, plan).

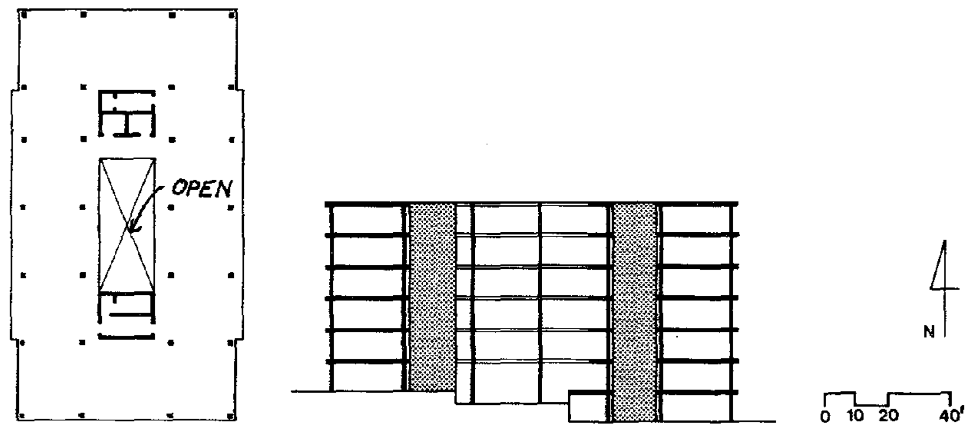


Figure V-21. Plan and section, the Four Seasons Apartment Building, Anchorage, Alaska.

In the 1964 Alaska earthquake the building, which was nearing completion and was unoccupied, collapsed totally. An eye witness reported (4):

"The building collapsed just before the end of the quake, after shaking violently for perhaps 2 to 3 minutes. Just before it fell, it seemed to start crumbling near the second-floor level in the area of its north-east corner. Then with a slight tilt northward it collapsed vertically in a great cloud of dust. The steel support columns fell to the north also, and the slab floors were stacked one on another like pancakes." (Figure V-22).

Figure V-22. Collapse of Four Seasons Apartment Building, 1964 Anchorage Alaska earthquake.

Although the precise causes of collapse are subject to various interpretations, from a configuration standpoint the building met its trial by earthquake with three weaknesses.

1. The two shafts were not tied together via the basement or foundation (Figure V-21, section).

2. Though the two shafts are ostensibly about the same, the walls of the north shaft produced a significantly stiffer section, and thus false symmetry, and its bending capacity was 1/4 more than the south shaft (Figure V-23).

3. The atrium hole in the floors, located between the cores, reduced the contact area of floor and core.

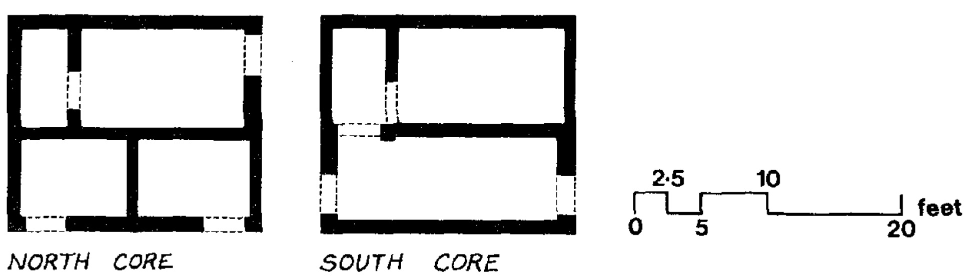


Figure V-23. Disparity between stiffness of ostensibly similar cores. The shear capacities are equal, but the yield moment of the north core is 1/4 greater. Four Seasons Apartment Building.

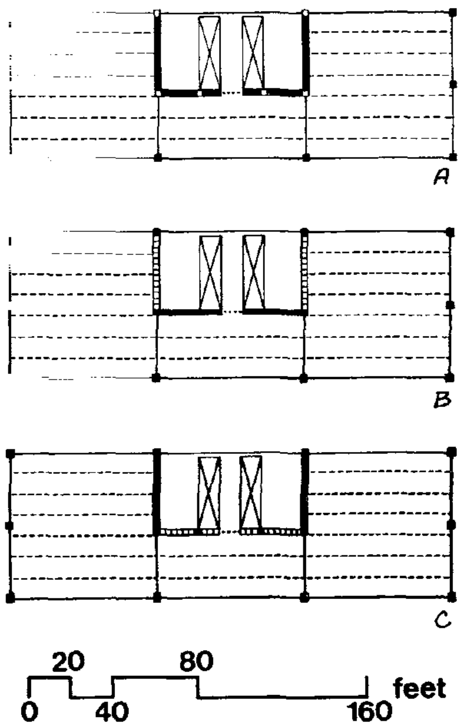


Figure V-24. Norton Building, Seattle, Washington.

It is clear then, that balancing of elements of resistance both in general location relative to the structure as a whole, and in their detailed design, from one floor to another or between separate resistant elements, is of great importance. If the core or cores can be centrally or peripherally located in a symmetrical building, this will reduce the possibility of torsion and decrease the potential for shaking for those parts of the structure farthest away from the core. If, for planning purposes, the core cannot be symmetrically located, some balancing resistance elements must be added - which may be difficult to do recognizing the omni-directional nature of possible shaking.

Alternatively, the core should not be used as the sole resistant element: either the building should rely on rigid frame action or alternative locations for shear walls, preferably on the perimeter, should be found. It is useful to remember that in respect of its own function - elevators, stairs, duct shafts etc - the core is a set of holes in the diaphragm, and it is a matter of the designer's choice as to whether it is a major resistant element or not.

Complete geometric symmetry is not required to ensure sufficient dynamic symmetry. Figure V-24 shows a design for an office building that at first glance appears to be a major case of false symmetry, with a large offset core (A). However, close inspection shows that the longitudinal resisting elements are symmetrically arranged about the building's center and will resist longitudinal forces without creating torsion (B). The transverse shear walls are similarly balanced for transverse forces (C).

References

1. Henry Degenkolb, "Seismic Design: Structural Concepts," Summer Seismic Institute For Architectural Faculty, (Washington, D.C.: AIA Research Corporation, 1977), p. 115.

2. Karl V. Steinbrugge, John H. Manning, and Henry J. Degenkolb, "Building Damage in Anchorage," in Fergus J. Wood, editor, The Prince William Sound, Alaska, Earthquake of 1964 And Aftershocks, (Washington, D.C.: Government Printing Office (Coast and Geodetic Survey), 1967), Volume II, Part A, p. 116.

3. Karl V. Steinbrugge and Donald F. Moran, "An Engineering Study of the Southern California Earthquake of July 21, 1952 and its Aftershocks," Bulletin of the Seismological Society of America, Volume 44, Number 2B (April 1954), p. 340.

4. [Verbal account by Glen L. Faulkner, to] Wallace R. Hansen, Effects of the Earthquake of March 27, 1964 at Anchorage, Alaska, (USGS Professional Paper 542-A, 1966), (Washington, D.C.: United States Government Printing Office, 1965), p. A24.

VI. Re-Entrant Corner Configurations

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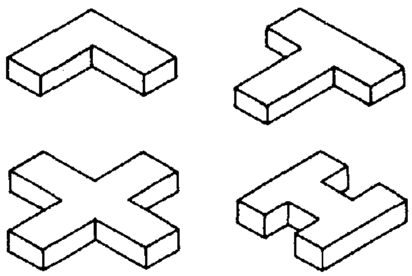
A. Definition



The re-entrant corner is the common characteristic of overall building configuration that, in plan, assumes the shape of an L, T, U, H, + or a combination of these shapes (Figure VI-1). These are complex configurations, as defined in Appendix 1, Figure 1.

This is a most useful set of building shapes, which enable large plan areas to be accommodated in relatively compact form, while still providing a high percentage of perimeter rooms with access to air and light. The advent of air-conditioning reduced somewhat the necessity for perimeter access, and produced the characteristic deep plan form of the mid-twentieth century. Current interest in daylighting and natural ventilation may result in a return to narrow buildings and the traditional set of re-entrant corner configurations. The courtyard form, most appropriate for hotels in tight urban sites, has always remained useful: in its most modern energy conserving form, the courtyard becomes a glass-covered atrium, but the structural form is the same.

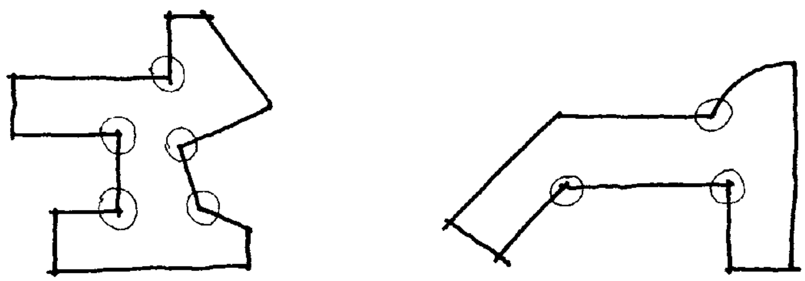


Figure VI-1. Re-entrant corners in combination configurations.

B. Problems

These configurations are so common and familiar that the fact that they represent one of the most difficult problem areas in seismic design may seem surprising.

There are two problems created by these shapes. The first is that they tend to produce variations of rigidity and hence differential motions between different portions of the building, resulting in a local stress concentration at the re-entrant corner.

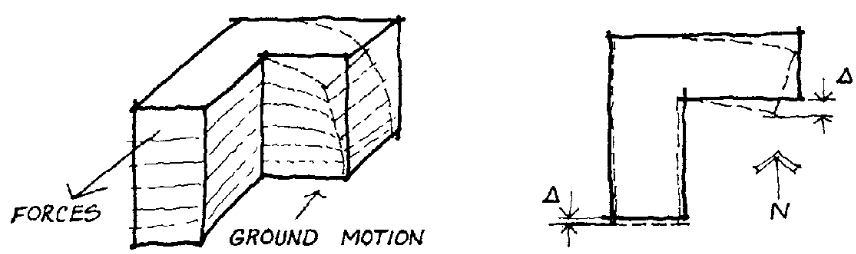


Figure VI-2. Forces on an L-shaped building.

Consider the L-shaped building shown in Figure VI-2. If ground motion occurs with a north-south emphasis, the wing oriented north-south will, purely for geometrical reasons, probably tend to be stiffer than the wing which is located east-west. The north-south wing, if it were a separate building, would tend to deflect less than the east-west wing, but the two wings are tied together and attempt to move differently at their junction, tearing and pushing each other (Figure VI-3). This condition is similar to the notch effect discussed in Chapter VII, Section B. Remember also, that the forces will be dynamic; there will be to- and- fro motion causing further damage. For ground motion along the other axis, the wings reverse roles but the differential motion problem remains.

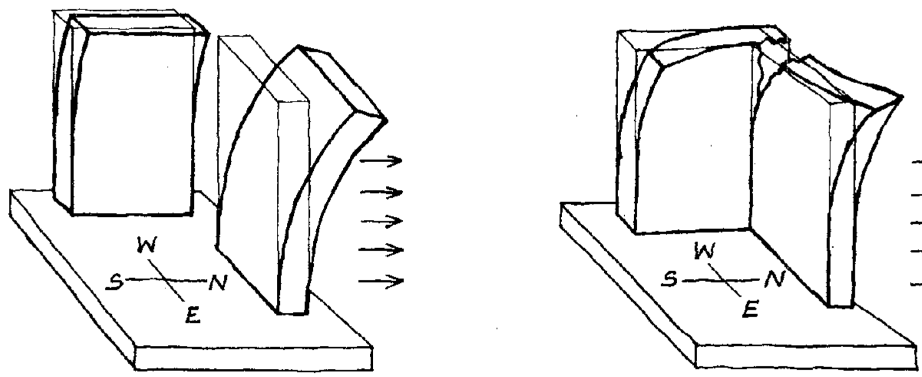


Figure VI-3. Because of their differing orientations, the two wings of an L-shaped building will attempt to move differently when subjected to ground motion, causing damage at their junction. If the wings were separate buildings, they could each move independently.

The second problem of this form is torsion, or twisting. This is caused because the center of mass and the center of rigidity in this form cannot geometrically coincide for all possible earthquake directions. The result is rotation which will tend to distort the form in ways that will vary in nature and magnitude depending on the nature and direction of the ground motion, and result in forces that are very difficult to analyze and predict.

The stress concentration at the notch and the torsional effects are interrelated. The magnitude of the forces and the seriousness of the problems will be dependent on:

. the mass of the building
. the structural systems
. the length of the wings and their aspect ratios
. the height of the wings and their height/depth ratios

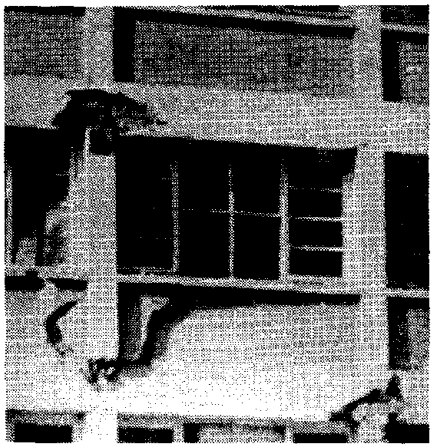
In addition, it is common for wings of a re-entrant corner building to be of different height, so that the vertical discontinuity of a setback in elevation is combined with the horizontal discontinuity of the re-entrant corner in plan, resulting in an even more serious problem.

C. Damage Examples

Examples of damage to re-entrant corner type buildings are common, and this problem was one of the first to be identified by observers. It had been identified before the turn of the century, and by the 20's was generally acknowledged by experts of the day. Naito and Sano (1) attributed significant damage in the 1923 Tokyo earthquake to this factor.

Ray Clough has analyzed and explained the severe damage to an L-shaped school in the 1960 Chile earthquake (2).

"Characteristically, the structures which were damaged were generally of the L shape or at least had many wings, appendages, etc. The nice, rectangular, simply planned shapes behaved very well. In this particular building, the damage that we observed was associated with the interconnection between the two wings. Figure VI-4 shows the kind of damage we observed. This is the right side of that wing which projected toward us is in Figure VI-5. Directly opposite this wall are the floor systems of the other wing. What we observe here is the tendency for the floor slabs to push out as the two wings rotate relative to each other, buckling and breaking off the columns of the wing we are looking at... The breaking was largely due to the changing of the right angle which was built into this L-shaped structure, the floor systems pushing out the wall. This is not a deficiency in design in the sense that changing the seismic coefficient in the code would have improved the situation. It is the kind of thing that designers have to watch out for in the basic structural plan."



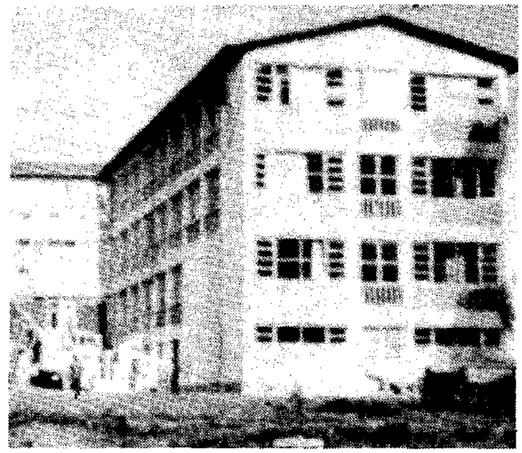


Figure VI-4. Damage to Seminario at Puerto Montt, 1960 Chile earthquake.

Figure VI-5. L-shaped Seminario at Puerto Montt.

The damage to the West Anchorage High School (Figures VI-6-9) in the 1964 Alaska earthquake is typical, and since complete collapse did not occur, the sequence of events can reasonably be reconstructed, as was done by engineer John Meehan (3).

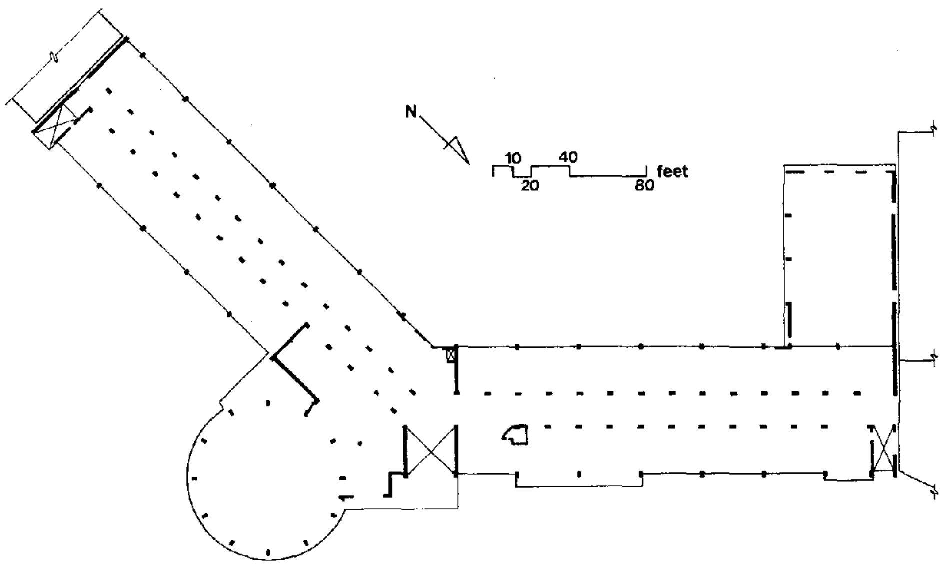


Figure VI-6. Plan of West Anchorage High School, Alaska.

"One cannot be certain of the sequence or path of distress; however, it is believed that the initial damage occurred in the roof diaphragm at the vertex of the angle formed by the two portions of the classroom wing due to torsional moment developed in this diaphragm (Figure VI-7). It is also believed that, after the roof diaphragm separated at this point, each portion of the classroom wing essentially formed individual buildings, thus necessitating a redistribution of load in the shear walls. The shear walls, such as those shown in Figure VI-8, were not capable of resisting this redistribution of load and were apparently damaged next. The exterior second-floor columns were then unable to resist the total load alone, and damage developed in these - such as that shown in Figure VI-9. At this point, the earthquake action stopped. No damage was observed to the more flexible center corridor columns."

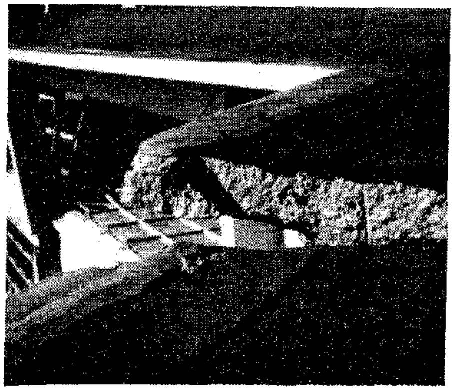


Figure VI-7. Damage to roof diaphragm near junction of two wings, West Anchorage High School, 1964 Alaska earthquake.

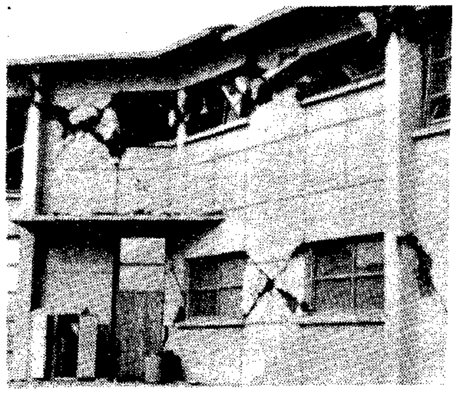


Figure VI-8. Damage to shear walls at intersection of the building wings. Note weakening of these highly stressed walls by insertion of windows. West Anchorage High School.

Figure VI-9. Second floor columns damaged after shear wall failed. West Anchorage High School.

An eye witness report of the earthquake effects on this building adds further graphic credence (4).

"I had just entered my automobile when the earthquake started. As it started, I was looking at the school and kept thinking to myself, Well now, this earthquake has gone on long enough, now is the time for it to stop. However, it continued and grew in intensity. Finally, after I had watched the school for some time, all of the glass in the second story seemed to explode and shattered all at once. Then waves appeared from the two extremes of the V-shaped classroom section and seemed to work toward the center and back again. This continued for some time. Then the roof slab seemed to sigh and rise slightly, and then when it settled all of the second story columns broke."

The concentration of stresses at the notch of a re-entrant corner is graphically illustrated by the San Marcos building in the Santa Barbara earthquake of 1925. The corner of this four story reinforced concrete frame building collapsed when differential shaking of the two wings of this L-shaped building caused them to hammer one another at the notch area (Figure VI-10).

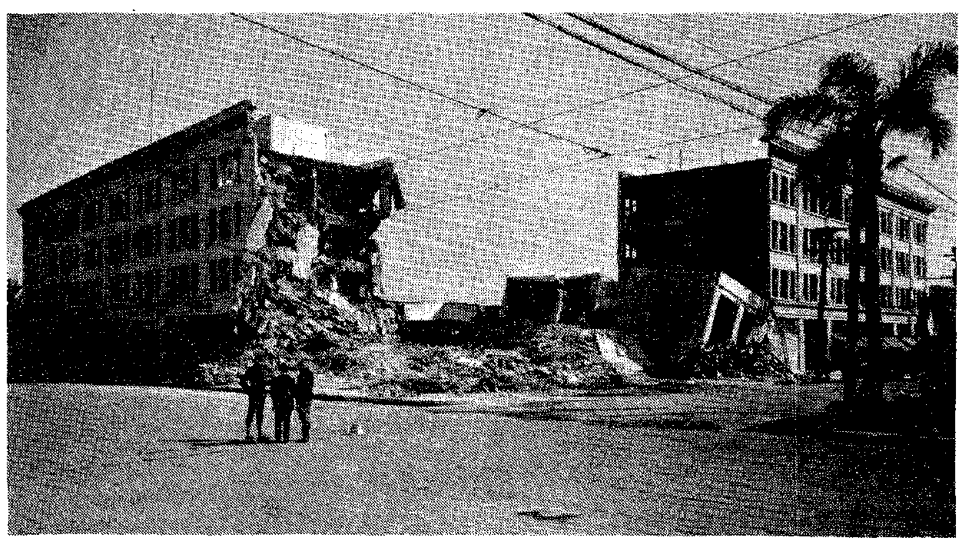


Figure VI-10. Collapse of re-entrant corner of the L-shaped San Marcos Building, 1925 Santa Barbara California earthquake.

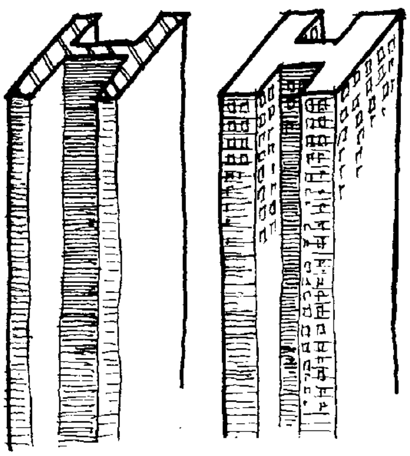


Figure VI-11. The "misleading analogy" of an H-column and an H-shaped building.

The corner and the wing at left were built at a different time than the wing on the right. Poor materials and reinforcing details have also been cited as contributing factors. Bailey Willis and Henry Dewell noted at the time that (5), "The failure was presumably determined by the shape of the ground plan and by its position relative to the earthquake shock."

The re-entrant corner plan well illustrates the dangers of transferring structural behavior from one scale to another. There is no comparison between a solid steel H that is only about one foot square, and a building with wings one hundred or more feet long connected by occasional floor slabs. The latter will not behave homogeneously, and the forces will be transferred through dozens of columns, beams, slabs, and connections, all varying in their strength and stiffness and transferring forces one to the other with varying eccentricity and direction.

Thus it is a totally misleading analogy to suggest that if the flanges of an H-column are the key to its lateral strength and stiffness, then the wings of an H-shaped building must similarly perform like flanges (Figure VI-11).

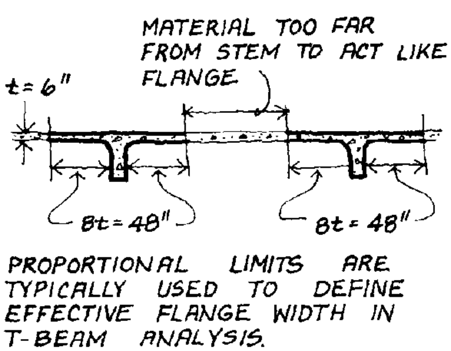


Figure VI-12. Proportional limits for flange/web interaction in a T-beam.

One need only move up one small step in size from the steel H or I-section to begin to face limits for this sort of flange/web interaction. When a reinforced concrete slab is monolithic with or rigidly connected to its concrete beams, a T-beam results. The slab acts as the top flange, efficiently resisting compressive forces in regions of positive moment, and steel reinforcement at the bottom of the beam handles the tension. However, if beams are spaced twenty feet apart, can one assume that twenty-foot-wide flanges are created? This is not the case, because the concrete which is too far from the web fails to interact (Figure VI-12).

The example of the West Anchorage School illustrates that the diaphragm in the region of the notch will sustain the greatest forces and hence strength at that point is crucial. However, it will be found that this is also the most useful area in which to provide vertical circulation - stairs or elevators - in a multi-story building, for the hinge point forms a natural circulation place. Both stairs and elevators result in a hole in the diaphragm at the least desirable point (Figure VI-13).

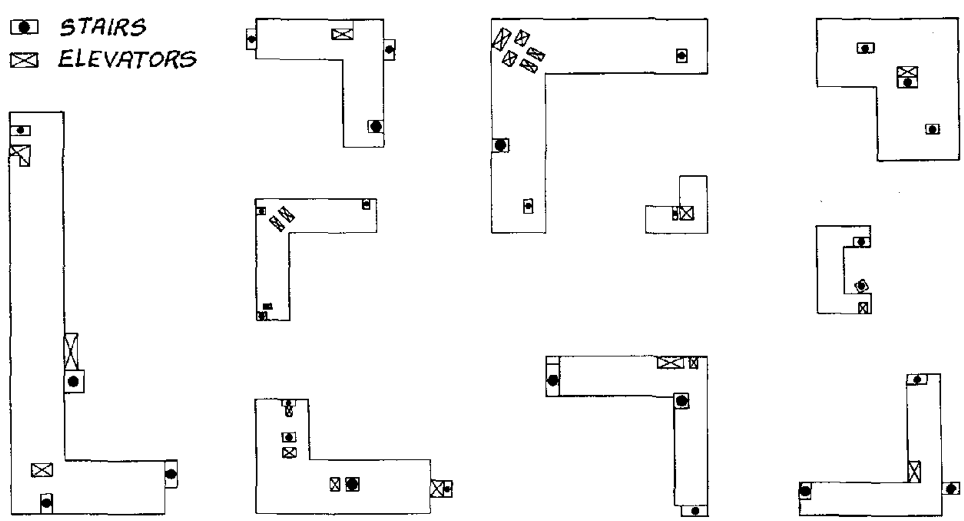


Figure VI-13. Schematic plans of existing L-shaped buildings illustrating recurring patterns of the location of vertical circulation elements.

D. Solutions

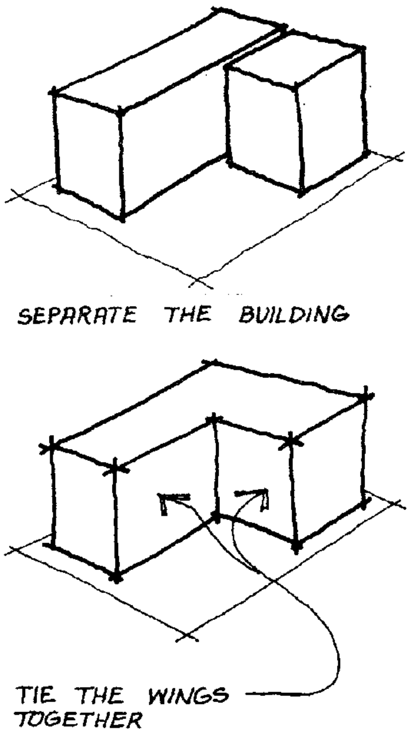
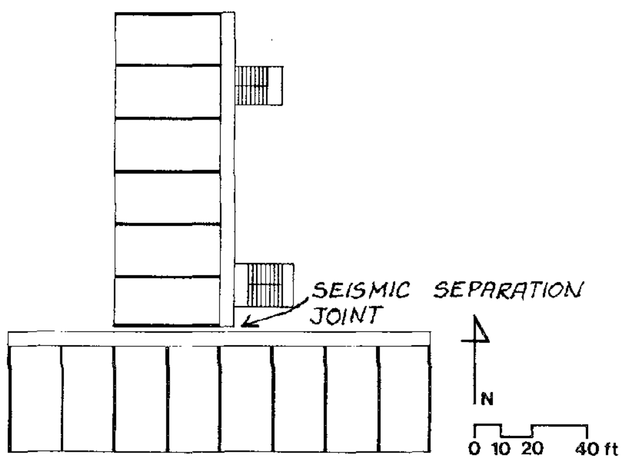


Figure VI-14. Two basic solutions to the re-entrant corner problem.

There are two basic alternative approaches to the problem of the re-entrant corner forms: structurally to separate the building into simpler shapes, or to tie the building together more strongly (Figure VI-14). Once the decision is made to use separation joints, they must be designed and constructed correctly to achieve the original intent. Structurally separated entities of a building must be fully capable of resisting vertical and lateral forces on their own, and their individual configurations must be balanced horizontally and vertically. This problem is demonstrated by the fact that three of the four stairtowers of the Olive View Hospital, separated from the main building, completely overturned in the 1971 San Fernando earthquake as discussed in more detail later in Chapter VIII, Section D. The other concern is that the amount of separation be made adequate so that pounding does not result. This is also illustrated by one of the Olive View Hospital stair towers, in which the 4 inch gap between it and the main building proved insufficient to prevent pounding damage.

The pitfalls of inadequate design for these two alternative approaches were succinctly illustrated in Japan's 1968 Tokachi-oki earthquake, in which Noheji Middle School and Misawa Commerce High School damaged themselves when inadequately separated portions pounded together, and Gonohe Primary School, an unseparated H-plan building, suffered severe cracks in the slab connecting the two wings as it was not capable of responding as a unit.

The Sunnyheights building is a recent example from Japan of a large L-shaped apartment house (Figure VI-15) which was divided by seismic joints into two rectangular structures (Figure VI-16) whose behavior followed a text book pattern in relation to the particular earthquake forces encountered. In this building, the earthquake forces were severe enough to create architectural damage through pounding, but the seismic joints acted to prevent structural damage (Figure VI-17).



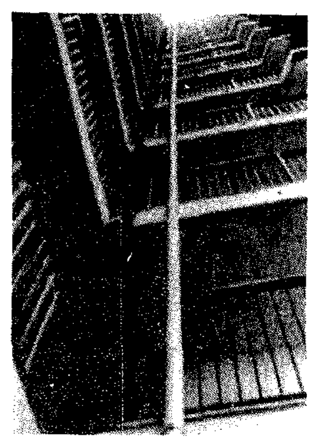


Figure VI-15. General view of the L-shaped Sunnyheights apartment building.

Figure VI-16. Plan of Sunnyheights apartment building, showing location of seismic joint.

Figure VI-17. Damage at seismic joint, Sunnyheights apartments, following 1978 Sendai Japan earthquake. Earthquake forces were predominantly in the North-South direction.

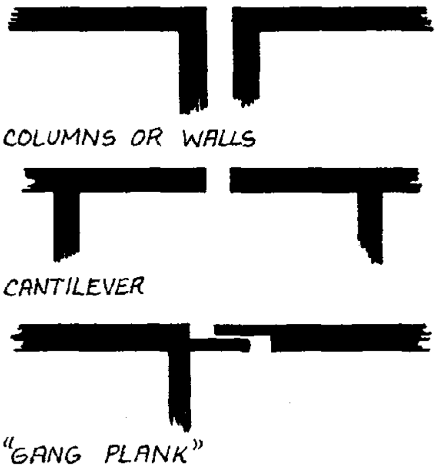


Figure VI-18. Separation joint concepts.

To design a separation joint, the maximum drift of the two units must be calculated by the structural consultant. The worst case is when the two individual structures would lean toward each other simultaneously, and hence the dimension of the separation space must allow for the sum of the building deflections.

Joints can be designed in various ways but must accomplish complete structural separation of floors and walls (Figure VI-18). Non-structural components such as partitions, ceilings, pipes, and ducts must also be detailed to allow for this movement, unless certain components can be safely and economically sacrificed. (A crushible sheet metal section of well-anchored curtain wall might fit the sacrificial criteria, but an ordinary glazed portion of the curtain wall would not.) Seismic separation joints are similar in construction to thermal expansion joints, but are typically larger and must be capable of working smoothly while being horizontally and vertically vibrated. Structures have been damaged by pounding at joints which were intended to function only as thermal joints, but which were forced to behave as inadequately dimensioned seismic joints as well. Figures VI-19 through 22 show examples of buildings containing seismic joints.

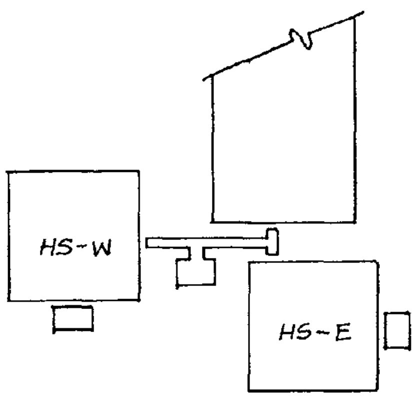


Figure VI-19. The Health Sciences Building, at the U.C. San Francisco Medical Center, shows how a complex plan is divided into independent units.

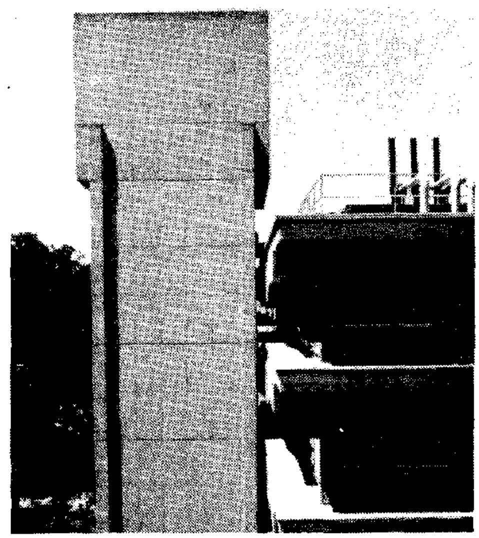


Figure VI-20. Seismic joint between laboratory and mechanical shaft, of the 16-story tower, U.C. San Francisco Medical Center, exterior view.

Figure VI-21. Interior view of seismic joint, U.C. San Francisco Medical Center.

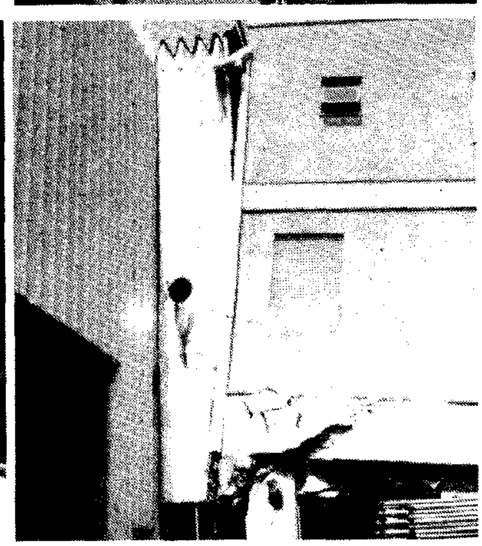


Figure VI-22. Large seismic joint prevented damage to building at left, when commercial building at right collapsed during 1978 Sendai Japan earthquake.

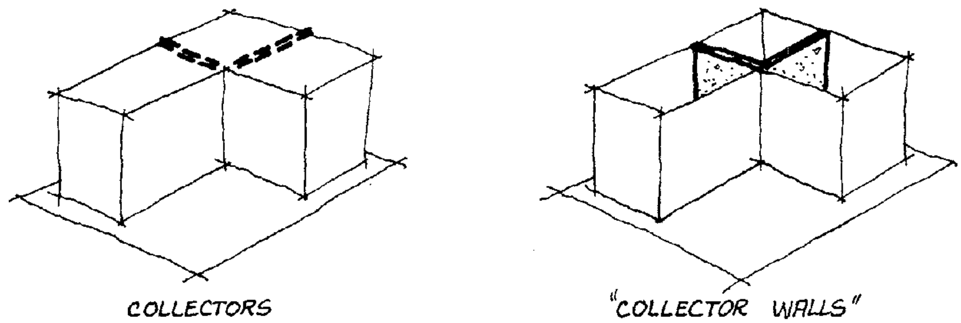


Figure VI-23. Tieing the building together.

Several considerations arise if it is decided to dispense with the separation joint and tie the building together. As discussed earlier in Chapter III, Section H, collectors at the intersection can transfer forces across the intersection area, but only if the design allows for these beam-like members to extend straight across without interruption. Even better than collectors are walls in this same location (Figure VI-23).

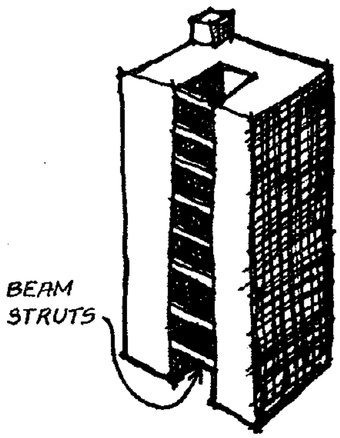


Figure VI-24. Early solution for U-plan weakness in resisting wind, Park Row Building, New York.

Since the portion of the wing which typically distorts the most is the free end, it is desirable to place stiffening elements at that location (Figure VI-24).

Figure VI-25 illustrates the stiffness disparity between the two wings for forces which can occur along either axis, a disparity which can theoretically be balanced by increasing the stiffness of the end bays as shown. (There would still be more likelihood of differential motion than in the simple rectangular configuration.) Only the frames along one axis are shown. Compare this with Figure VI-2 for an intuitive feeling of how this approach mitigates the problem of unbalanced stiffness.

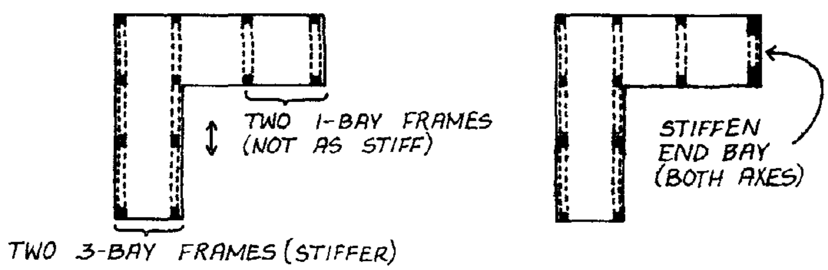


Figure VI-25. Balancing the disparity in stiffness between the two wings.

The use of splayed rather than right angle re-entrant corners lessens the notch effect problem (Figure VI-26), which is analogous to the way a rounded hole in a steel plate creates less stress concentration problems than a rectangular hole, or the way a tapered beam is more desirable than an abruptly notched one.

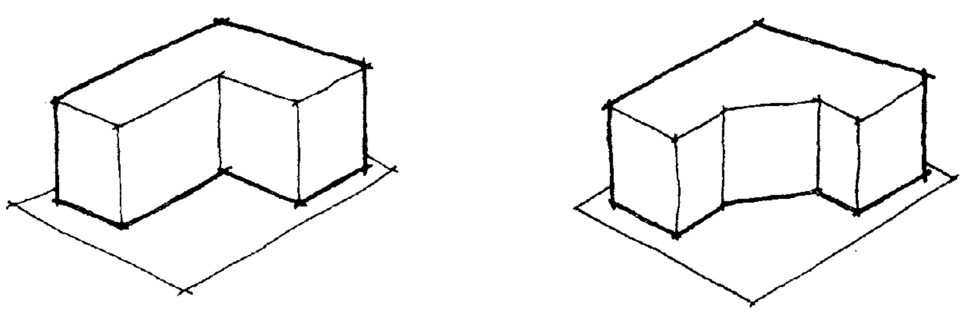


Figure VI-26. The splayed re-entrant corner.

Figure VI-27 illustrates a design subterfuge which combines a slight compromise of form (splayed corners) with innovative framing to turn a highly irregular plan form, the cruciform, into a simple square with small triangular projections.

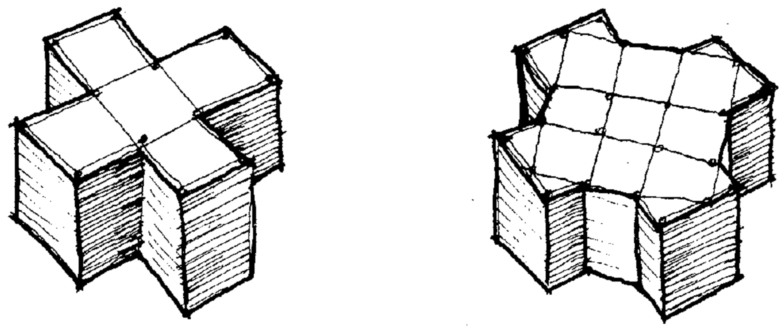


Figure VI-27. Relieving stress by design subterfuge.

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and
John Ripley Freeman, Earthquake Damage and Earthquake Insurance, (New York: McGraw-Hill Book Company, Inc., 1932), p. 478. [Discusses the paper by Dr. Sano, in Report by the Japanese Society of Engineers Upon the Tokyo-Yokohama Earthquake of September 1, 1923, Volume III: Section On Bridges & Buildings, Section On Roads & Streets, (Tokyo: Engineering Society, 1927), p. 610. written in Japanese].

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3. John Meehan, "The Response of Several Public School Buildings in Anchorage, Alaska, to the March 27, 1964, Earthquake," in Fergus J. Wood, editor, The Prince William Sound, Alaska, Earthquake of 1964 And Aftershocks, (Washington, D.C.: Government Printing Office (Coast and Geodetic Survey), 1967), Volume II, Part A, p. 222.

4. [Eyewitness account in] Glen V. Berg and James L. Stratta, "Anchorage and the Alaska Earthquake of March 27, 1964," Earthquakes, (Washington, D.C.: American Iron and Steel Institute, 1975), p. 156.

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VII. Vertical Setback Configurations

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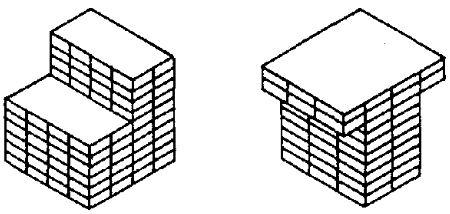
A. Definition



Setback configurations are a common vertical irregularity in building geometry, and consist of one or more abrupt reductions of floor size within the building height. For our purposes, setback configurations are defined as complex elevations; some characteristic types and properties for rectangular plans are defined in Appendix 1, Figure 12, and shown in Figure VII-1. Setbacks may also apply to complex plans, as shown in Appendix 1, Figure 15.

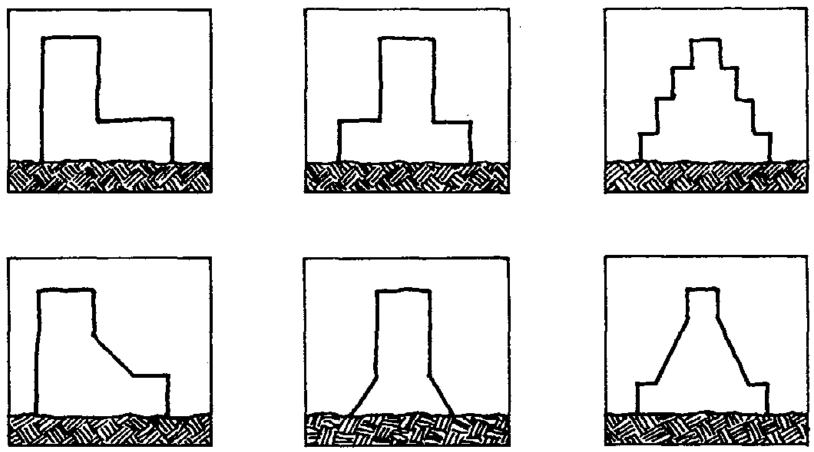


Figure VII-1. Characteristic setback configurations.

Setbacks may vary greatly as to their proportion between tower and base in height and plan, the symmetricality of the base and tower portions of the building, and the types of construction used for each portion.

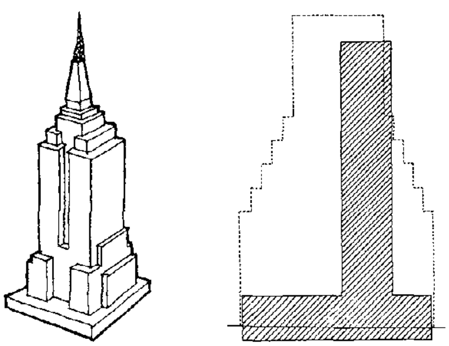


Figure VII-2. Neither the Empire State Building's nor the Lever Building's form was completely determined by setback zoning: both buildings leave some of their zoning-allotted volume unfilled, due to market or image reasons.

Setbacks may be introduced for several reasons: the three most common are; zoning requirements which require upper floors to be set back to preserve light and air to adjoining sites, program requirements which require smaller floors at higher levels, and stylistic requirements relating to the form of the building. In fact, zoning requirements seldom were responsible for complete setback determination as stylistic massing requirements always tended to play a significant role (Figure VII-2).

Setbacks relating to zoning were common a few decades ago when daylighting requirements were of major concern, and resulted in the characteristic shapes of high buildings in New York City. Pressures, primarily of a stylistic nature, replaced these forms by those of simple rectangular solids, made possible by advances in artificial lighting and air conditioning. Now, the requirements of energy conservation, reflected in a new concern for daylighting, appear to herald a renewed interest in setback shapes, a concern which is in tune with a stylistic reversion away from the rectangular solid (Figure VII-3).

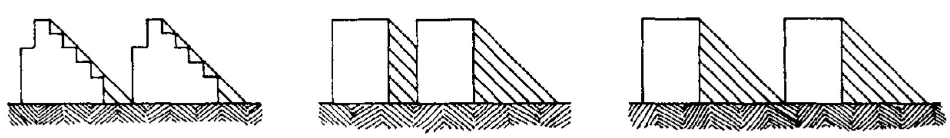


Figure VII-3. Use of setback form to reduce shading of adjacent building.

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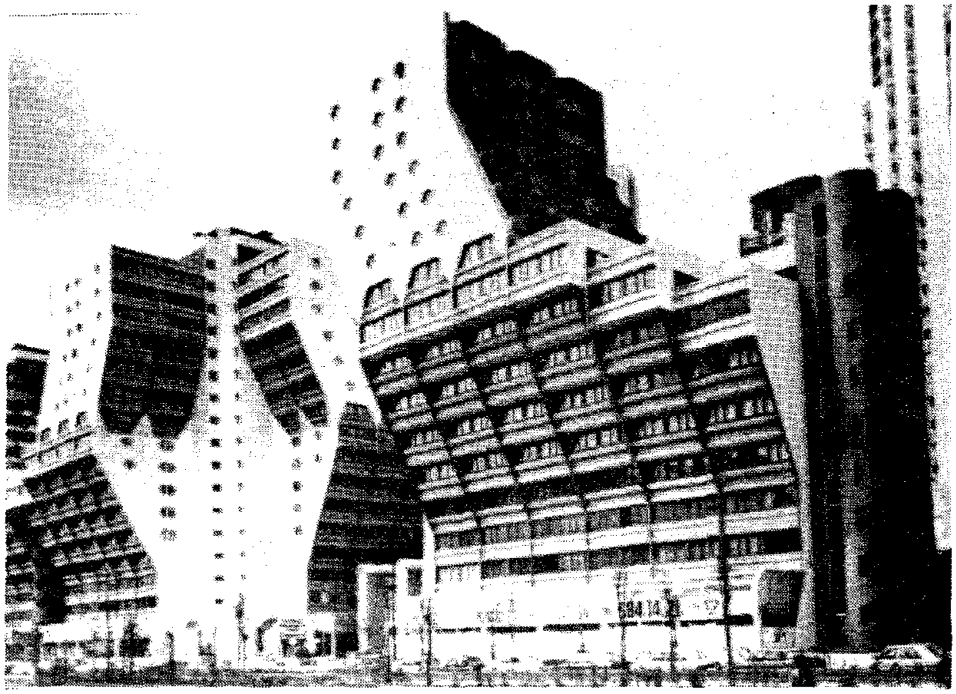


Figure VII-4. The inverted setback.

A new type of setback configuration is that of the building that grows larger with height: this configuration type is termed 'inverted setback'. Its geometrical definition is the same as that of the setback, but because of the problems of overturning its extremes of shape are less. Nevertheless, some surprising, unconstrained demonstrations of this shape have been designed and built (Figure VII-4), and it appears to be a shape whose image has a powerful design appeal (Figure VII-5).

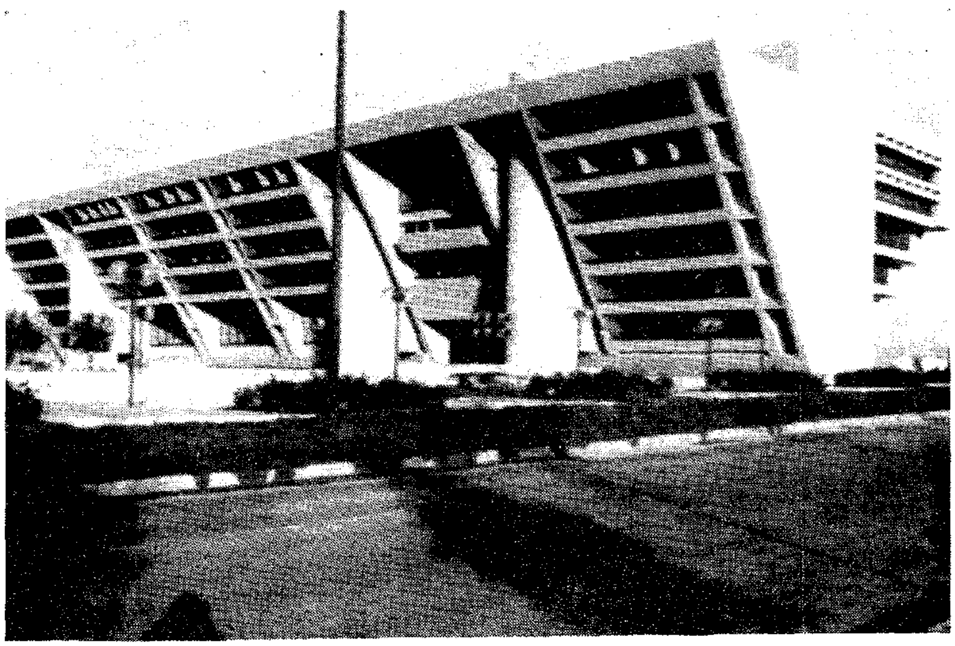


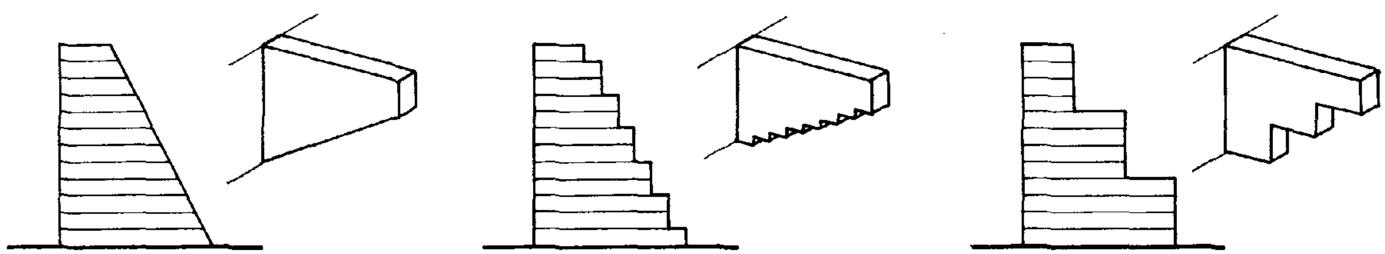
Figure VII-5. The inverted setback: Dallas City Hall.

B. Problems

The problem of the setback shape lies in the more general problem of discontinuity: the abrupt change of strength and stiffness. In the case of this complex configuration, it is most liable to occur at the point of setback, or 'notch'.

The seriousness of the setback effect depends on the relative proportions and absolute size of the separate parts of the building. In addition, the symmetry or asymmetry, in plan, of the tower and base affect the nature of the forces. If the tower, or base, or both are dynamically asymmetrical, then torsion forces will be introduced into the structure, resulting in great complexity of analysis and behavior.

The notch problem can also be visualized as a vertical re-entrant corner. Stresses must go around a corner because a 'notch' has been cut out preventing a more direct route. Thus, the smaller the steps or notches in a setback or inverted setback, the smaller the problem. A smooth taper avoids the notch problem altogether. A tapering beam will not experience stress concentrations, whereas a notched beam will (Figure VII-6). A continually sloping setback or inverted setback (if the framing as well as the skin is actually continuous), completely avoids the problem of abrupt changes in stiffness (although it still may not behave according to code assumptions, and so must be specially analyzed).



TAPERED BEAM

TAPERED BEAM WITH SUPERFICIAL NOTCHES

NOTCHED BEAM: ABRUPT CHANGES IN STIFFNESS

Figure VII-6. Setbacks: the transition from a tapered to a notched beam analogy.

Setbacks with shear walls in the tower portion can introduce other difficulties (Figure VII-7). Besides the possibility of an abrupt change of stiffness where the shear wall enters the base structure, the shear wall will transmit large forces to the top diaphragm of the base. Overturning moments (which are difficult to transfer horizontally) as well as shears must find alternate routes if shear walls are not continuous. As with other combinations of configuration problems, a setback with a discontinuous shear wall creates a situation analogous to a squaring function: the two anomalies interact and form a much greater and more uncertain problem, than would occur if the two variables were independent and merely additive.

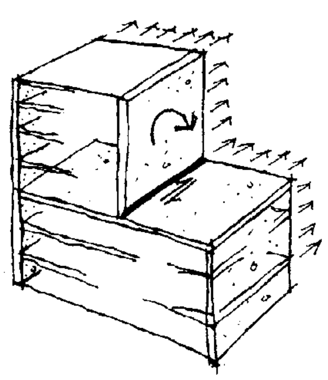


Figure VII-7. Setbacks which contain shear walls may create severe problems in transferring forces to the foundation.

Although the common instances of setbacks occur in a single building, the condition can also be created by adjoining buildings of different heights.

In Managua, in the 1972 Nicaragua earthquake, the six-story Lang building was adjacent to a three-story building on one side, and a two-story building on the other. Extreme damage occurred just above the top of the adjacent two-story building, while there was little damage higher up in the six-story building (Figure VII-8). While this damage was partly due to the adjacent structures vibrating independently and pounding against each other, in large part it was due to the abrupt change in stiffness at the roof level of the adjoining two-story building. With the lower parts of the Lang building constrained by the adjoining building, lateral displacements of the larger building were constrained in its lower portions and damage was severe at this point where a stress concentration occurred.

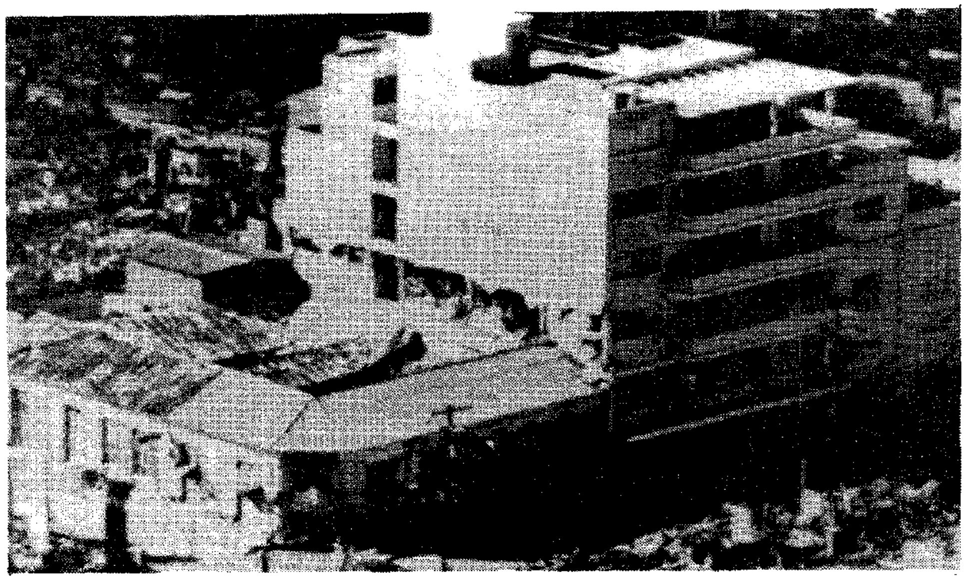


Figure VII-8. Severe damage at the point of setback, the Lang Building, 1972 Managua Nicaragua earthquake.

Engineer S.B. Barnes has discussed narrow towers on broad bases which result from the influence of parking requirements on building form (1).

"This type of structure is encountered frequently in Southern California where parking areas are desirable and required by law. The tower usually has a moment resisting frame with no shear walls. The large base structure usually involves basement stories which obviously must have basement walls which have almost infinite rigidities as compared to the more flexible frame which carries through under the tower (Figure VII-9). At the transition level then we need an especially heavy diaphragm to transfer lateral forces from the tower area to these perimeter basement walls and special attention must be given to strut-tie connections at this level."

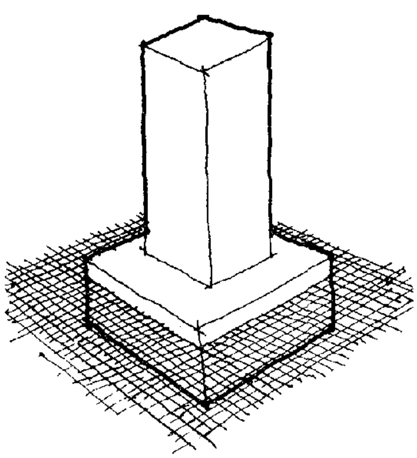


Figure VII-9. Narrow towers on broad underground bases, typifies many buildings with integral parking structures.

Setbacks and inverted setbacks are similar in that they create changes in stiffness and are susceptible to the notch effect, but, as might be expected, are opposite in terms of the characteristics of their overall shape. For although replacing a large notch with several smaller ones, or tapering the elevation may eliminate the problems due to abrupt changes in stiffness for the setback building, the inverted setback building has an additional more serious problem.

Since it is desirable to keep the forces lower to the ground rather than higher, it is desirable to keep the building's mass or center of gravity closer to the ground. Forces at lower heights mean smaller lever arms and hence smaller overturning moments. The setback or pyramid configuration distributes masses in a positive way, while the inverted setback is a significant step in the wrong

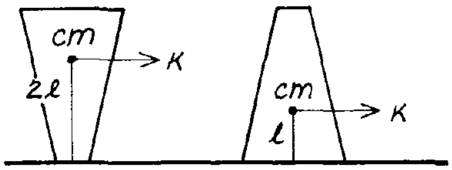


Figure VII-10. With an inverted setback, the lever arm (2L), acting through the center of mass (cm), is twice as long as in a setback, therefore the overturning forces are double.

structural direction (Figure VII-10). Related to the size of the forces, is the question of the amount of the resistance capacity. The pyramid shape increases its amount of material (assuming some uniform distribution of members) as it goes down, in which direction the loads are also increasing. The upside-down pyramid, in addition to stacking its mass up higher off the ground, provides less depth and material as it goes down - the opposite of an effective seismic configuration. An example of damage to an inverted setback building appears in Figure VII-11.

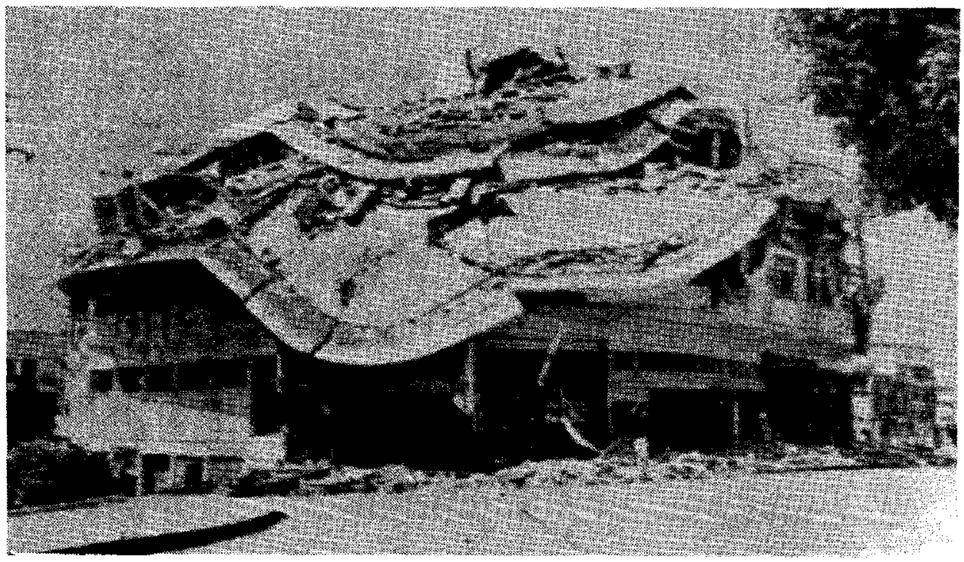


Figure VII-11. Collapse of upper story of inverted setback, Sud Building, 1960 Agadir Morocco earthquake.

C. How the Building Code Deals with Setbacks

Setbacks have long been recognized as a problem, and so the Uniform Building Code has attempted to mandate special provisions for them. Currently, the Earthquake Regulations of the Code refer to setback configurations as follows (2):

"Buildings having setbacks wherein the plan dimension of the tower in each direction is at least 75% the corresponding plan dimension of the lower part may be considered as uniform buildings without setbacks, provided other irregularities as defined in this section do not exist." (Figure VII-12)

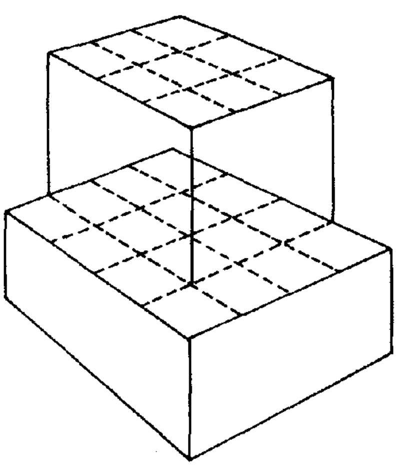


Figure VII-12. SEAOC cut-off point for setbacks: Plan dimension of tower = 75% of corresponding dimension of next lower level. For rectangles, this will occur when the tower area reduces to 9/16 of that of the base.

The SEAOC Commentary (1975) to this section is (3):

"Any horizontal offset in the plane of an exterior wall of a structure is a setback. Sometimes this requires the transfer of shear from the upper wall across the setback to the wall below. Provision for overturning moment also requires special attention at setbacks. The SEAOC Recommendation considers the more usual case of concurrent physical and dynamic setbacks. It can occur that a physical setback will not create a dynamic irregularity. Conversely a dynamic irregularity can be created without a physical setback. These special conditions should be carefully considered by the structural engineer to produce proper design.

"Setbacks in many instances constitute a degree of irregularity that requires consideration of the structure's dynamic characteristics in order to achieve a reasonable distribution of lateral forces."

In addition, the Commentary refers the reader to Appendix C, the report of the 1958 Setback Subcommittee, which is discussed in our next section. Thus, in dealing with setbacks, the code essentially leaves to the designer the evaluation of the problems and the selection of an appropriate design solution.

D. The 1958 SEAOC Setback Subcommittee

In 1958, the Setback Subcommittee of SEAOC, headed by John Blume, attempted to provide a basis upon which code regulations for setback configurations could be written. Their proposal was not adopted, but still appears in the SEAOC "Blue Book" Commentary to the Code, and their approach provides some useful guidance.

Four basic conditions were defined by the subcommittee (4), which are defined below and in the chart following (Figures VII-13, 14).

A. The setback is not of sufficient extent to modify the behavior of the building. To determine period and base shear, the structure is considered as one building of full height, and of a width which is the weighted average of tower and base.

B. "The Base portion predominates and the Tower may be considered an appendage subject to a ground motion which is equal in acceleration to that of the top of the Base portion... The Base shall be considered a separate building of its own height with the Tower weight and base shear applied at the roof level. The Tower base shear coefficient shall be forty (40%) percent greater than that obtained on the assumption that the Tower is a separate building situated on the ground. The other Tower shears shall be determined pro rata from this Tower base shear as for a separate building."

C. "The Tower portion predominates and the portion of the Base not encompassed by the tower extension to the ground is essentially a structural "lean-to" which goes along for the ride, but also contributes additional mass and resistance..." To compute seismic coefficients "extend the Tower through the Base to the foundation level and treat as a separate building unit of" full height. "The additional weight of the portions of the Base not included in the extended 'Tower' shall be used with the seismic coefficients for a fictitious building having the height of the Base only to determine additional lateral forces at the lower levels. At least 70% of all 'Tower' originated forces shall be provided for within the plan limits of the extended 'Tower'."

D. "The intermediate cases which are difficult of determination. In this category we have to consider the situation where the two portions may average out to act as one building of the full height or where they may vibrate as independent units but in such manner as to affect each other... Whichever of the following procedures produces lateral forces which govern the design at any location or member shall be used:

(1) Assume the Tower and the Base constitute one building of the full height, and of a weighted average width, to determine the seismic coefficient: such coefficients shall then be increased twenty (20%) percent; or:

(2) Treat the Base and Tower as two separate buildings and follow procedure B above."

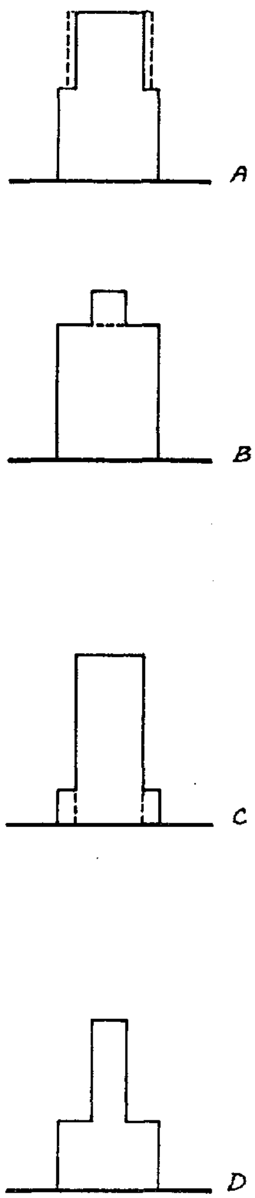
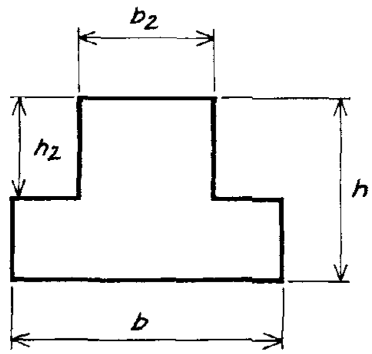


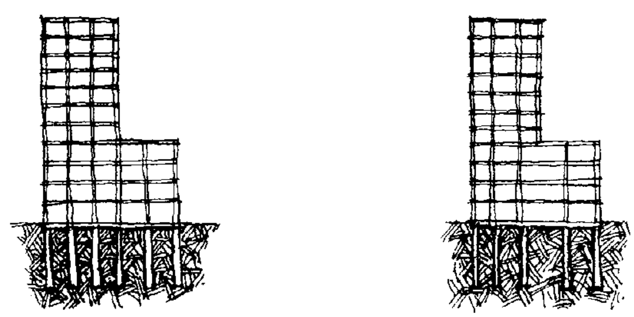
Figure VII-13. The categories defined in the 1958 SEAOC Setback Subcommittee proposal.



$\frac{b_2}{b}$	$\frac{h_2}{h}$	ANALYSIS METHOD
.8 to 1.0	any	A average width
.6 to .8 .4 to .6 0 to .4	less than .35 less than .25 less than .4	B base + appendage
.6 to .8 .4 to .6 less than .4	.65 to 1.0 .75 to 1.0 .8 to 1.0	C tower + "lean-to"
less than .4 .6 to .8	less than .4 to .8 less than .35 to .65	D whichever governs for any member: average width, 120% forces; -or- base + tower, follow B.

Figure VII-14. Analysis procedures proposed by the 1958 SEAOC Setback Subcommittee.

The Setback Subcommittee also recommended that structural frames, either moment resisting or braced, or combinations be required for both tower and base of any setback building with more than one story of setback. In addition, it should be required that the columns supporting the setback be carried straight down from the setback columns all the way to the foundation (Figure VII-15).



CONTINUOUS COLUMNS DISCONTINUOUS COLUMNS

Figure VII-15. Structural framing of setbacks should include continuous columns.

E. Solutions

Solutions to the setback configuration are analogous to those of its horizontal counterpart, the re-entrant corner plan. The first type of solution consists of a complete seismic separation in plan, so that the portions of the building are free to react in their own way (Figure VII-16). For this solution, the guidelines for seismic separation discussed in the previous chapter, should be followed.

Where the building is not separated, the analysis proposed by the SEAOC Setback Subcommittee provides the best guidelines. Particular attention should be paid to avoiding vertical column discontinuity, so that setbacks should be arranged to coincide with normal bay sizes (Figure VII-17). Any large building with major setback conditions should be subject to special analysis, or at least careful investigation of dynamic behavior. Finally, the inverted setback configuration of any extreme form and size should be avoided in seismic areas.

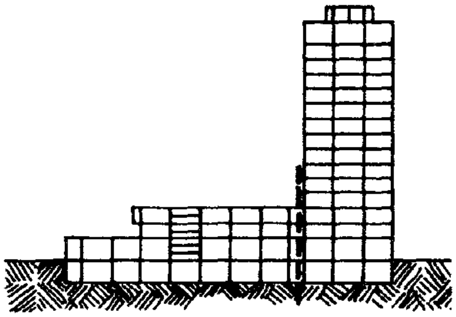


Figure VII-16. Office tower seismically separated from parking structure building, Kajima International Building, Los Angeles.

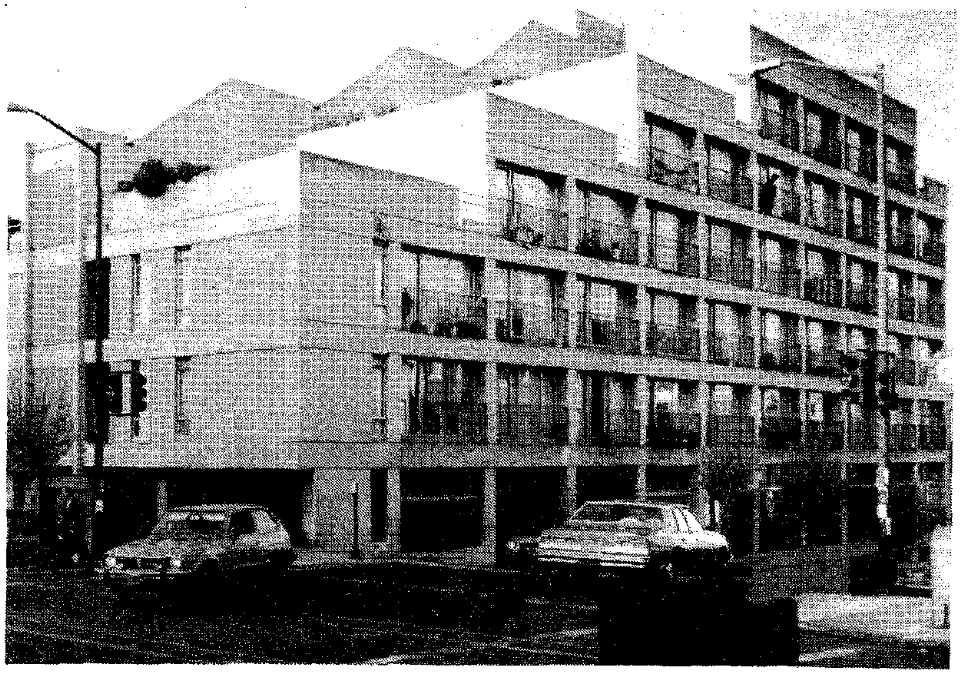


Figure VII-17. Setbacks coincide with bay spacing.

References

1. S.B. Barnes, "Basic Approach To Structural Design For Seismic Forces," Proceedings Of The Symposium On Earthquake Engineering, (Vancouver, B.C.: University of British Columbia, 1966), p. V-7.

2. International Conference of Building Officials (ICBO), 1979 Uniform Building Code (UBC), (Whittier, California: ICBO, 1979), Section 2312(e)2, p. 130.

3. Structural Engineers Association of California (SEAOC), Recommended Lateral Force Requirements and Commentary (The SEAOC "Blue Book"), (San Francisco: SEAOC, 1975), Section 1 (E)2, p. 32-C.

4. 1958 Setback Subcommittee, John A. Blume, Chairman, "Appendix C: Report On Setbacks," in Structural Engineers Association of California (SEAOC), Recommended Lateral Force Requirements and Commentary (The SEAOC "Blue Book"), (San Francisco: SEAOC, 1975), pp. C2-C3, C5.

VIII. Discontinuities of Strength and Stiffness

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A. The General Problem

The set of problems created by discontinuous strength and/or stiffness has been well stated by Hanson and Degenkolb (1).

"If there is a single zone of weakness in the path of force transmission, or if there is a sudden change of stiffness, there is a zone of danger. Even when the structure remains elastic the response will change considerably and the distribution of forces throughout the height of the structure can vary substantially from the assumed triangular distribution. However, it is even more critical when the structure has begun to deform inelastically.

"...If it can be assumed that the code required lateral forces are based on the performance of an older style typical structure where there was no sudden change of stiffness, then the absorption of the earthquake energy is distributed throughout the structure, either uniformly or in some regular continuous pattern. If a structure has a much more flexible portion under a rigid portion, most of the energy absorption is concentrated in the flexible portion and very little is absorbed in the more rigid portion above..."

B. The 'Soft Story'

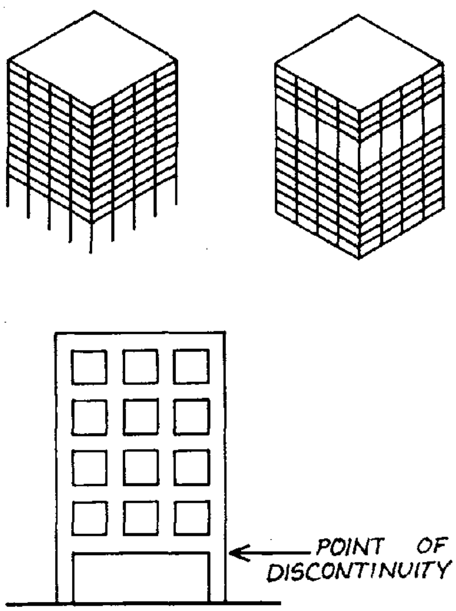


Figure VIII-1. A stiffness discontinuity produces a zone of weakness, or a "soft story".

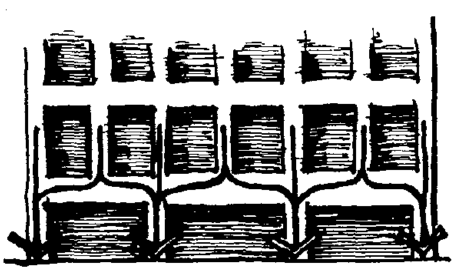


Figure VIII-2. The discontinuous load path.

The most prominent of the set of problems caused by discontinuous strength and stiffness is that of the 'soft story.' This term has commonly been applied to buildings whose ground level story is weaker than those above. However, a soft story at any floor creates a problem, but since the forces are generally greatest towards the base of a building, a stiffness discontinuity between the first and second floors tends to result in the most serious condition (Figure VIII-1).

The soft story occurs when there is a significant discontinuity of strength and stiffness between the vertical structure of one floor and the remainder of the structure. This discontinuity may occur because one floor, generally the first, is significantly taller than the remainder, resulting in decreased stiffness.

Discontinuity may also occur as a result of a common design concept in which all vertical framing elements are not brought down to the foundation, but some are stopped at the second floor to increase the openness at ground level (Figure VIII-2). This condition creates a discontinuous load path resulting in an abrupt change of strength and stiffness at the point of change.

Finally, the soft story may be created by an open floor which supports heavy structural or non-structural walls above. This situation is most serious when the wall above is a shear wall, acting as a major lateral force resistant element: this condition is discussed later in this chapter, since it represents an important, special case of the soft story problem.

The basic problem with all these variations of the soft story is that most of the earthquake forces in the building, and any consequent structural deformity, will tend to be concentrated in the weaker floor or at the point of discontinuity, instead of being more uniformly distributed among all the stories.

Previous statements about flexible elements carrying less load than stiffer ones, referred to the rigid diaphragm which distributed loads according to the relative rigidities of vertical elements, which were uniformly deflected. This is not the situation with soft stories which form the flexible portion of the building. In the soft story condition, deflections of this story will be much greater than that of other floors, and hence this one story will experience more stress and damage.

Figure VIII-3. Taller ground story columns contributed to the damage of this soft-story building, Carrillo Hotel, 1925 Santa Barbara California earthquake.

The soft first story was early identified as a problem. This account of damage to a soft first story building in Santa Barbara was written by Freeman based on the damage observed in 1925 by Henry Dewell and Bailey Willis. Even with more than fifty years of hindsight, an earthquake expert of today would not need to take exception to the summary (2).

"Carrillo Hotel. This was of particular interest because of the tall, first-story columns, which served as a semi-flexible support and were bent by the inertia-resistance of the superstructure, which suffered relatively little damage, while the walls and floors of the lower portion were severely wrecked, so that columns were strained at the point of flexure, and panels distorted at the point where the hollow tile was sheared, and the brick, or tile, veneer was wrenched and shaken off (Figure VIII-3). Joints and connections at top of the tall first story columns were reported badly wrenched and broken. The fact that this building, with first-story columns fractured at top and bottom, did not topple over, can only be explained by the small amplitude of earth motion."

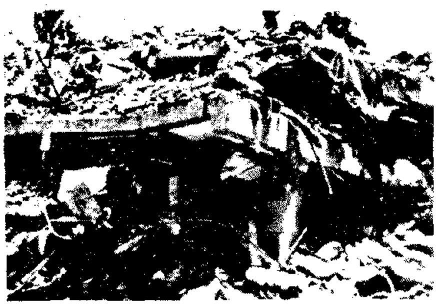


Figure VIII-4. Collapsed wing of 10-story Palace Corvin Building, 1967 Caracas Venezuela earthquake. (The uncollapsed wing contained more ground story partitions.)

Contemporary engineers are clear as to the problems with the 'soft first story' concept. To quote Hanson and Degenkolb (3):

"There is a strong architectural tendency throughout the world to have an open first floor - to place the building on "stilts" as it were... It cannot be emphasized too strongly that current earthquake code requirements are not based on this type of dynamic stiffness distribution, and potentially a great amount of trouble should be expected where these buildings are built to minimum code requirements in areas subject to great earthquake shocks. The damage to many buildings in Caracas [in the 1967 earthquake] gives ample warning as to what lies ahead on the West Coast of the United States." (Figures VIII-4, 5)

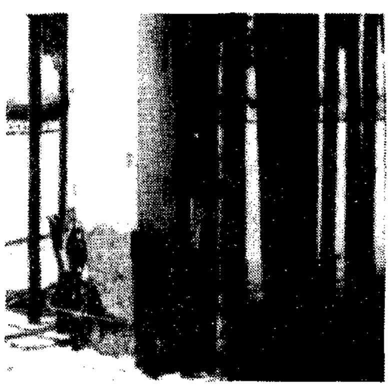


Figure VIII-5. Damage to a soft first story building, the San Bosco building, in the 1967 Caracas Venezuela earthquake.

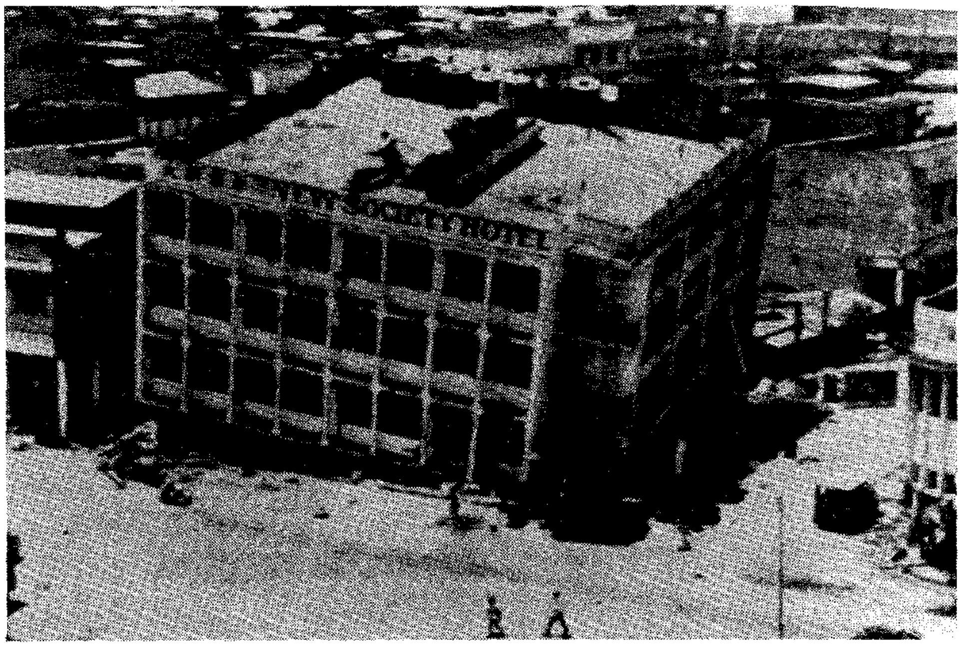


Figure VIII-6. The New Society Hotel, damaged in the 1976 Mindanao Philippines earthquake.

In the New Society Hotel, in Mindanao, 1976, the problem created by the vertical discontinuity of the tall first floor columns was intensified by a horizontal configuration problem leading to torsion. The two sides of the building facing the street were framed, but the two party wall sides were walled (and hence stiffer). The corner by the intersection would be expected to move the most, and hence the ground floor columns here are sheared and bent the most. Figure VIII-6 shows clearly that the building rotated as it fell outward into the street.

Figure VIII-7 shows other recent failures of soft first stories in a variety of structural types.

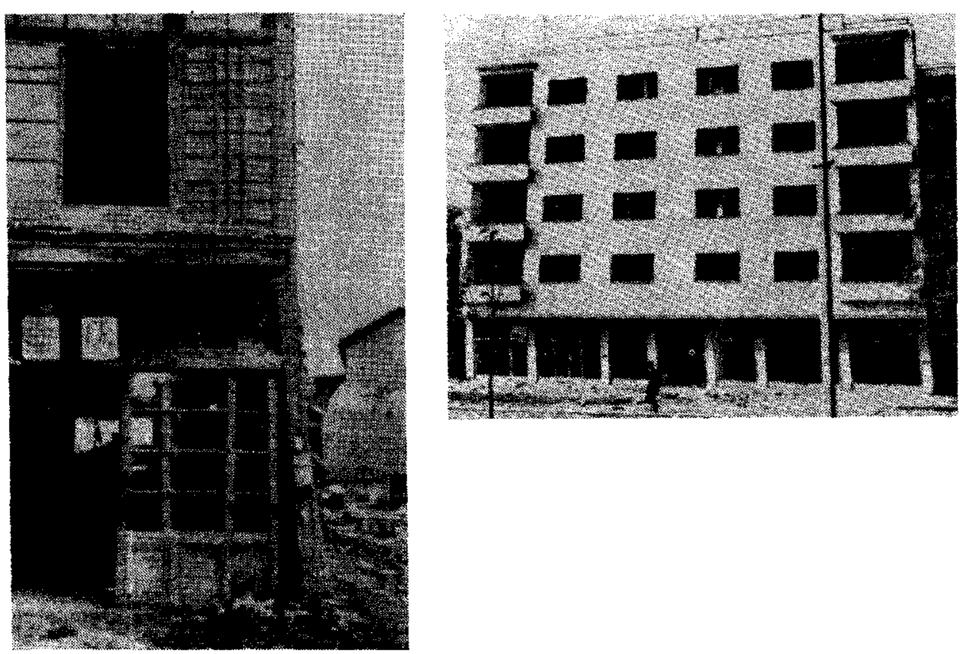


Figure VIII-7. Two examples of soft first story failures. Left, a damaged two-story shop in Misawa City whose column failed at the top of the first floor during the 1968 Tokachi-oki Japan earthquake. Right, distortion of ground story of apartment building on Eleventh of October Street, 1963 Skopje Yugoslavia earthquake.

C. Solutions

Solutions to the problem of the 'soft' story start with its elimination: to avoid the discontinuity through architectural design. If, for programmatic or compelling image reasons this is not possible, the next step is to investigate means of reducing the discontinuity by other design means (Figure VIII-8), such as increasing the number of columns, or adding bracing. Alternately, a high first floor may be attained but dynamic discontinuity eliminated by introducing a vertical super bay in which the main structure has uniformity of stiffness throughout its height and additional lighter floors are inserted in such a way as to have as little effect as possible on the characteristics of the main structure.

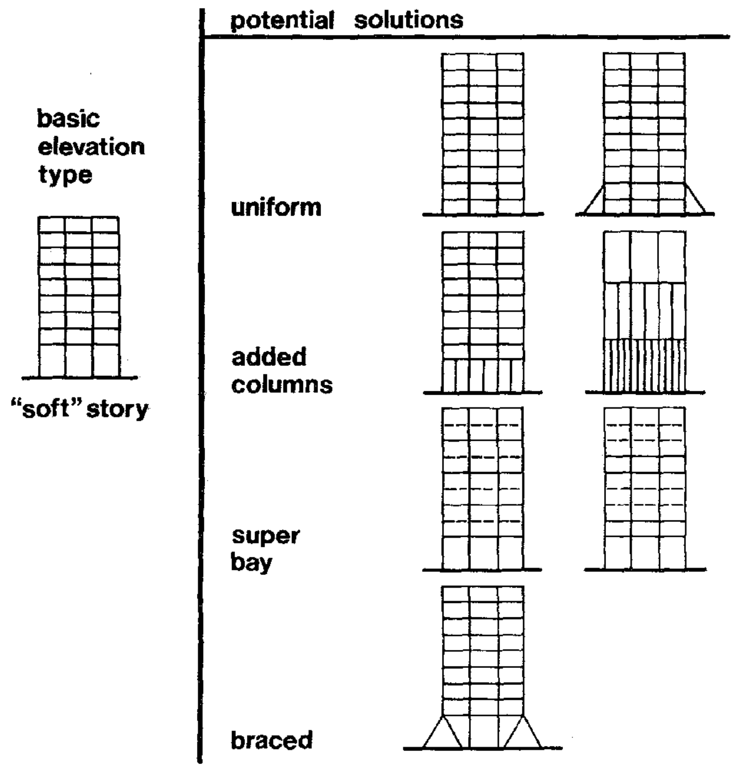


Figure VIII-8. Solutions to the soft story problem.

All these solutions require detailed analysis and refined design to alleviate the problems; the diagrams are intended merely to illustrate generic concepts that may begin a train of investigation.

The possibilities of architectural solutions should not be forgotten: the necessary emphasis to the first floor may be provided by design approaches that do not demand structural discontinuity (Figure VIII-9).

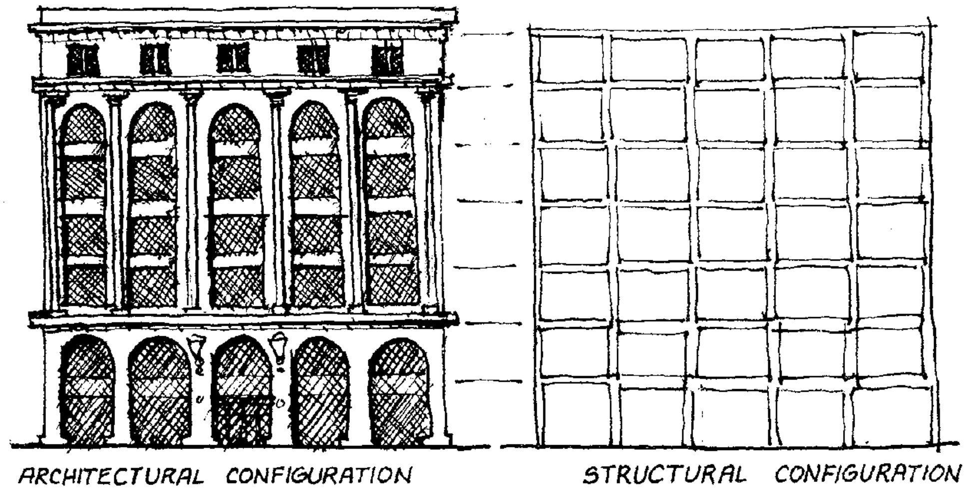


Figure VIII-9. A traditional architectural device for creating a visually non-uniform facade upon a uniform structure.

It should also be noted that the intentional use of a soft first story as a means of disconnecting the building superstructure from ground motion generated in an earthquake is theoretically a valid possibility. In-depth analysis and special construction measures and materials appear to be required to make the scheme work reliably. As Chopra, Clough, and Clough point out in their modeling of a particular first story design (4), "The first story yield mechanism must be designed to accommodate very large displacements, in excess of one foot, if it is to be effective with a flexible structure."

These and other authors have additional caveats. Intentionally designing a soft first story into a building may merit more attention, but at present it is a subject for research rather than for practical application.

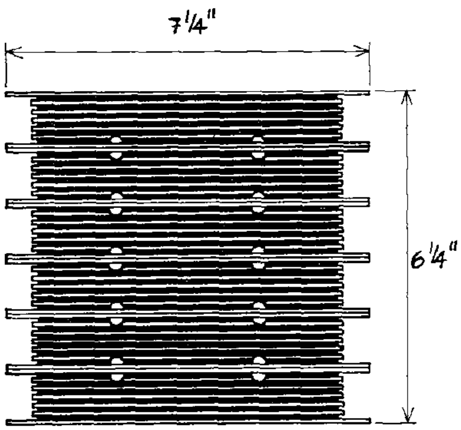
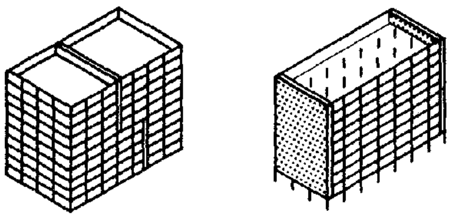


Figure VIII-10. Malaysian Rubber Producer's Research Association base isolation device, a rubber and steel plate sandwich, tested at U.C. Berkeley.

"Base isolation" devices comprise a separate topic, and represent an attempt to achieve a highly controlled soft story that isolates, or partially isolates, upper floors from ground motion. Early (nineteenth century) suggestions that buildings could be mounted on a layer of ball bearings are now being pursued along similar lines of thought, using modern technology: column base plates isolated from the foundation by a shearing sandwich of rubber and steel plates (Figure VIII-10), or a quasi-hydraulic shock absorber which pumps solid lead through an orifice and then back again as the foundation move back and forth, etc. This general approach is still at an experimental stage although French, New Zealand, and Swiss prototype buildings have actually been constructed.

Insofar as such devices may prove practical, they will serve to reduce, not eliminate, the level of loading in the superstructure, and to the extent that lateral forces are still experienced, all of the configuration principles presented here are still applicable.

D. Discontinuous Shear Walls



When shear walls form the main lateral resistant elements of the building, they may be required to carry very high loads. If these walls do not line up in plan from one floor to the next, the forces created by these loads cannot flow directly down through the walls from roof to foundation, and the consequent indirect load path can result in serious overstressing at the points of discontinuity.

Often this discontinuous shear wall condition represents a special, but common, case of the 'soft' first story problem. The programmatic requirements for an open first floor result in the elimination of the shear wall at that level, and its replacement by a frame. It must be emphasized that the discontinuous shear wall is a fundamental design contradiction: the purpose of a shear wall is to collect diaphragm loads at each floor and transmit them as directly and efficiently as possible to the foundation. To interrupt this load path is a fundamental error: to interrupt it at its base is a cardinal sin. Thus the discontinuous shear wall which stops at the second floor represent a 'worst case' of the soft first floor condition.

Olive View Hospital, which was severely damaged in the 1971 San Fernando, California earthquake, represents an extreme form of the discontinuous shear wall problem.

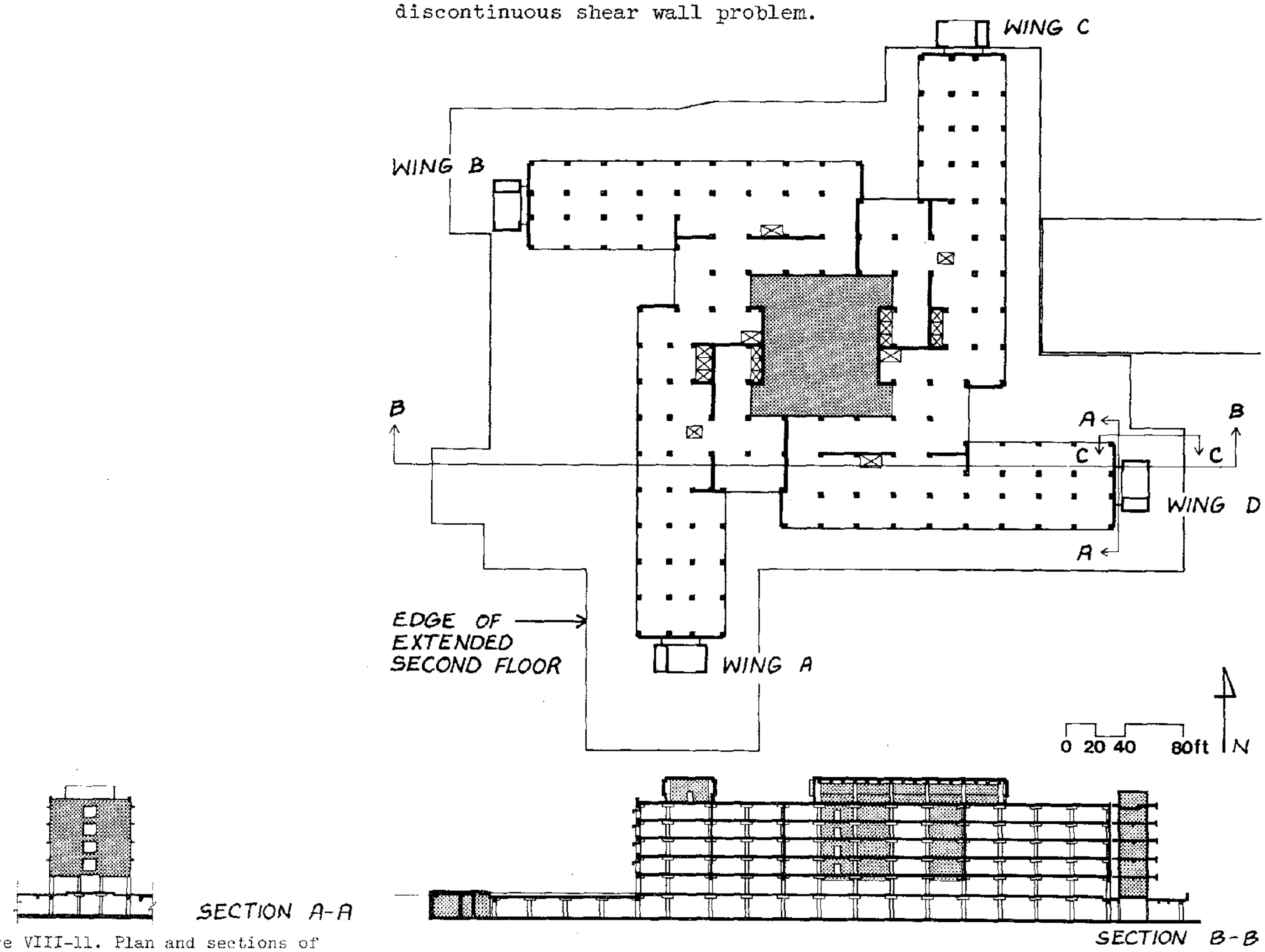


Figure VIII-11. Plan and sections of Olive View Hospital.

The general vertical configuration of the main building was a 'soft' two-story layer of rigid frames on which was supported a four story (five, counting penthouse) shear wall-plus-frame structure (Figure VIII-11). The second floor extends out to form a large plaza: thus, in photographs, the main building appears to have a single soft story, rather than two. The severe damage occurred in the soft story portion, which is generally to be expected (Figure VIII-12). The upper stories moved as a unit, and moved so much that the columns at ground level could not accommodate such a huge displacement between their bases and tops and hence failed. The largest amount by which a column was left permanently out of plumb was 2-1/2 feet. According to Frazier, et al (5), "it is doubtful if the already badly damaged columns could have stood up for another five seconds of strong ground shaking."

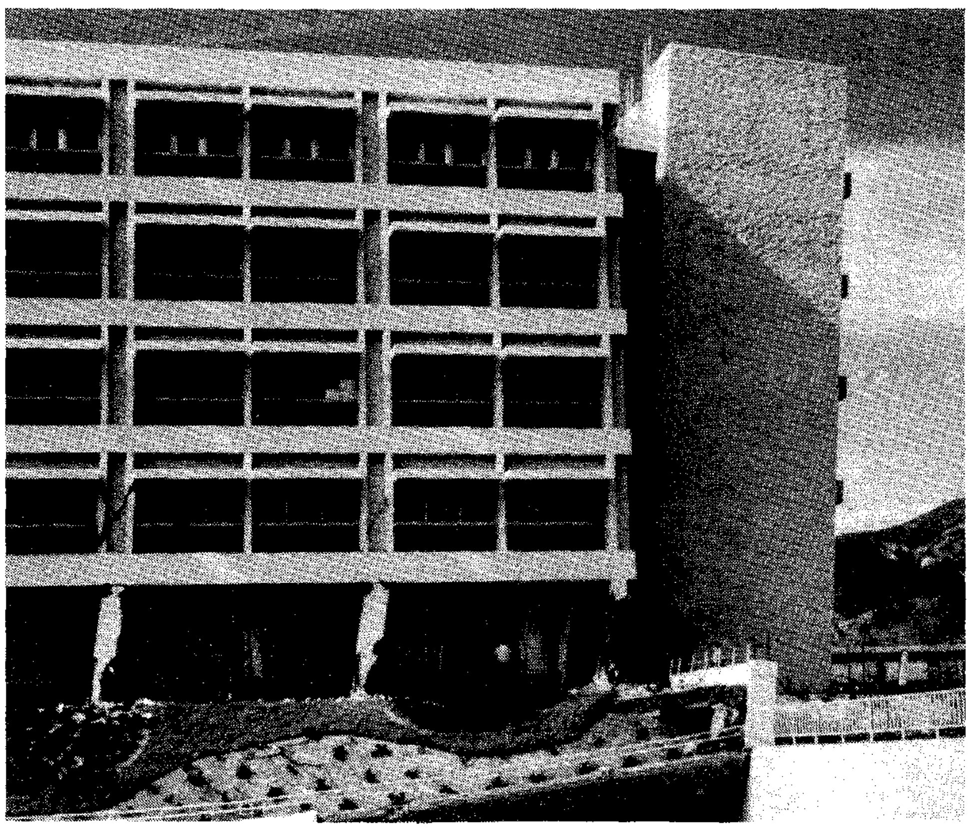


Figure VIII-12. Distortion of the soft story, Olive View Hospital, in the 1971 San Fernando California earthquake.

A discontinuity in vertical stiffness and strength leads to a concentration of stresses and damage, and the story which must hold up all the rest of the stories in a building should be the last, rather than the first, component to sacrifice. Had the columns at Olive View been more strongly reinforced, their failures would have been postponed, but it is unrealistic to think that they would have escaped damage. Thus the significant problem lies in the configuration, and not totally in the column reinforcement.

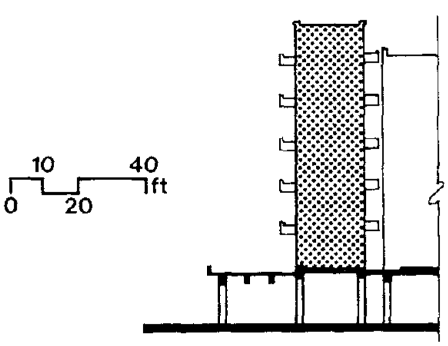


Figure VIII-13. Stair tower section, Olive View Hospital.

Though it is not as widely known, the stairtowers at Olive View also show a clear example of a discontinuous shear wall failure. The nature of this failure is not obvious, since the plaza formed by the extended second floor, gives the towers the appearance of being only six stories in height, when actually they are seven (Figure VIII-13).

These seven-story towers were independent structures, and proved incapable of standing up on their own: three overturned completely, while the fourth leaned outward 10°. The six upper stories were rigidly braced with ample solid reinforced concrete walls, but the bottom, or 'soft' story was composed of six reinforced concrete columns, which failed. The exception was the north tower, whose walls came down to the foundation directly without any discontinuity; this was the only tower which remained standing. Obviously, none of the towers was adequately built to prevent overturning, since the 10° out-of-plumb movement of the north tower might easily be called "failure," but it is clear that this flaw was compounded into total collapse only where the soft story was present (Figure VIII-14).

Figure VIII-14. Stair tower B collapsed outward, Olive View Hospital.

While one may attribute the proximate cause of these stair tower failures to the detailed design of the reinforced concrete columns which failed (such as the inadequacy of their ties) and to the extreme ground motion, it is clear that the configuration factor was responsible for setting up this over-stress situation. No matter how well the reinforcing is designed, a more reliable general solution would have been to eliminate the discontinuity created by the termination of the shear walls.

A common building configuration of the last few decades, is that of a number of repetitive floors of rectangular plan, with blank, or nearly blank, end walls which stop at the second floor level to permit an open ground floor. The Imperial County Services Building, El Centro, California, is a prototypical example of this building type (Figure VIII-15).

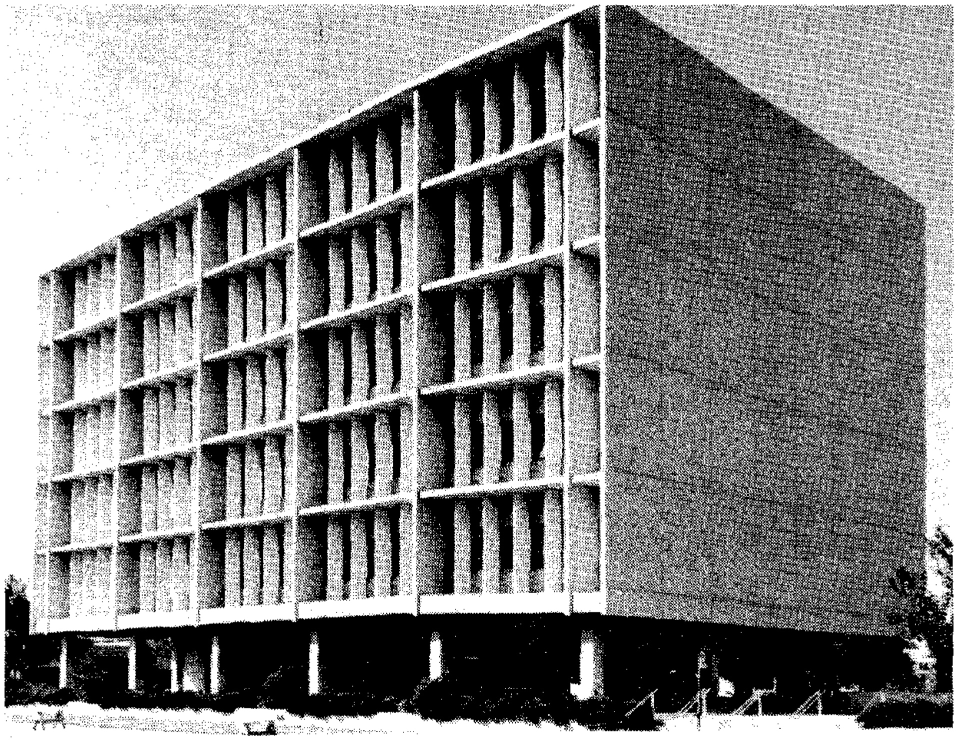


Figure VIII-15. The Imperial County Services Building, located in El Centro California, following the 1979 earthquake.

The behavior of the Imperial County Services Building, El Centro, in the Imperial Valley Earthquake of 1979, provided a textbook example of the effects of architectural characteristics on seismic resistance. The building was a six story reinforced concrete structure built in 1969. In the relatively mild earthquake, in which only a few of the poorest unreinforced masonry buildings suffered structural damage, this building suffered a major structural failure, resulting in column fracture and shortening - by compression - at one end (the East) of the building (Figure VIII-16). The origin of this failure lies in the discontinuous shear wall at this end of the building. The building was subsequently demolished.

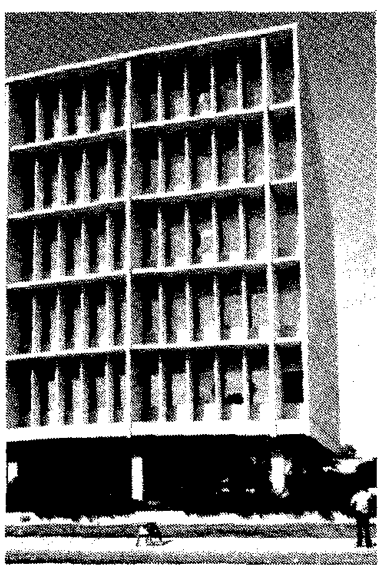


Figure VIII-16. A view of the east end of the Imperial County Services Building. Notice the sagging as a result of the column shortening.

The fact that the failure originated in the configuration is made clear by the architectural difference between the East and West ends, Figure VIII-17. The difference in location of the ground floor shear walls was sufficient to create a major behavioral difference in response to rotational, or overturning, forces on the large end shear walls (Figure VIII-18).

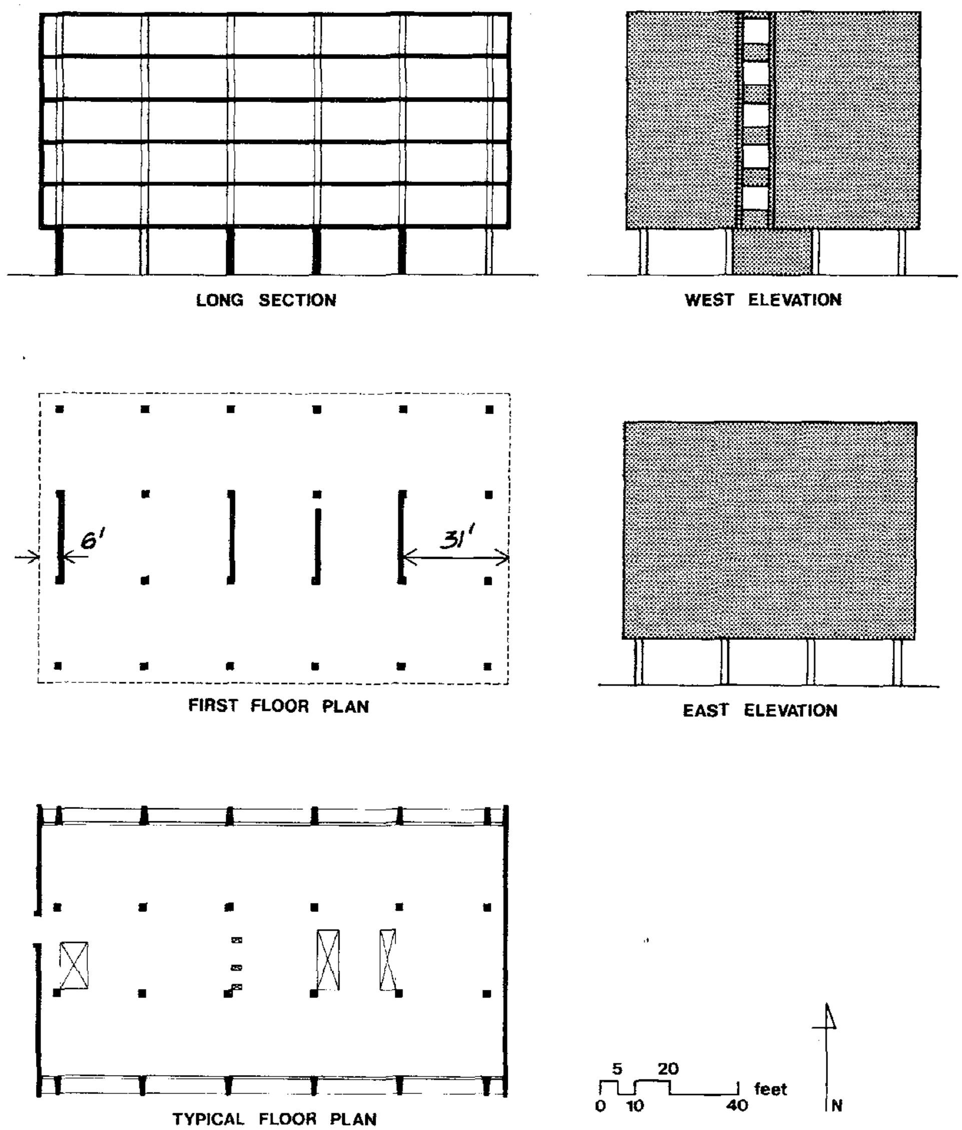


Figure VIII-17. Plans, elevations, and section of the Imperial County Services Building, showing the location of shear walls.

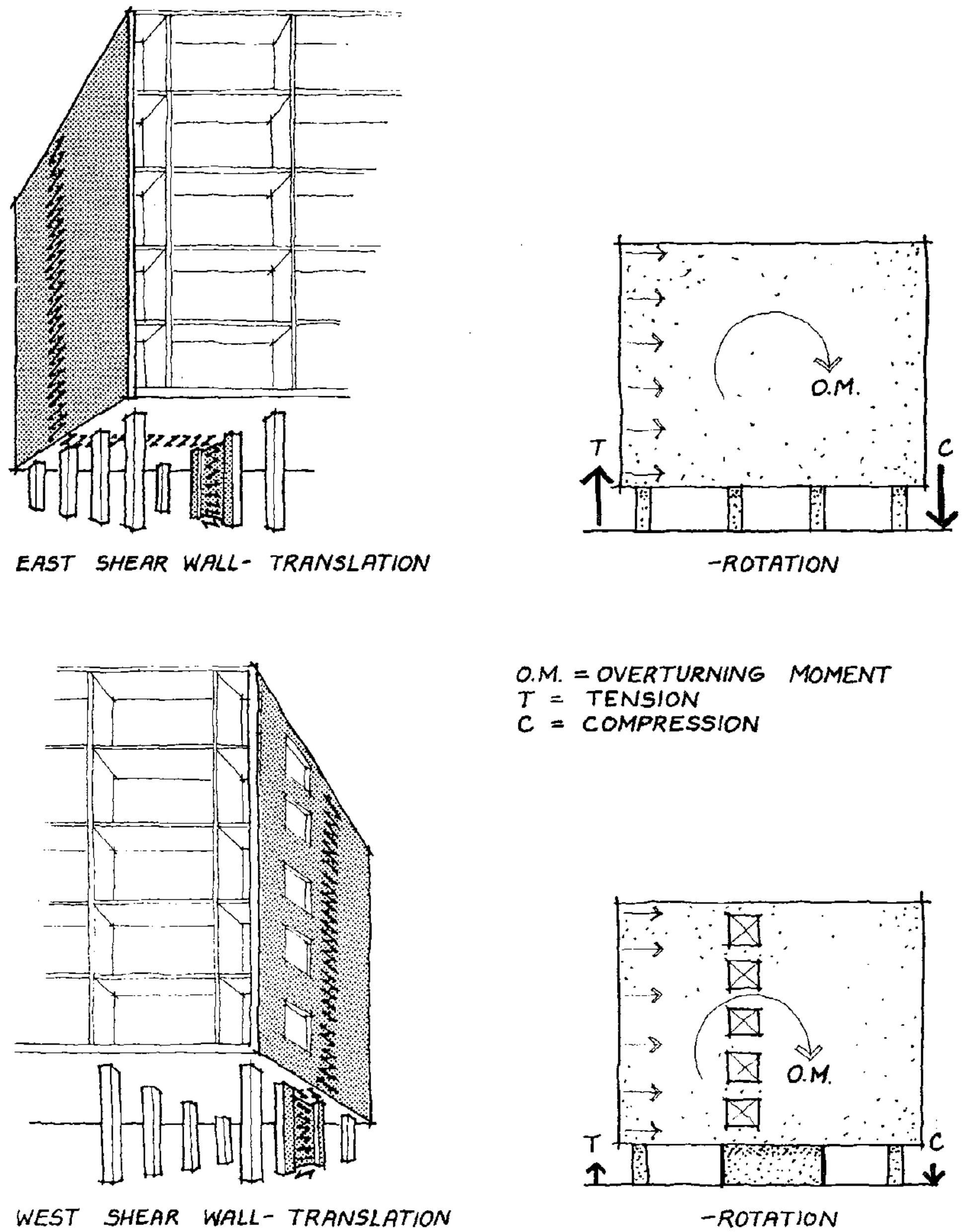
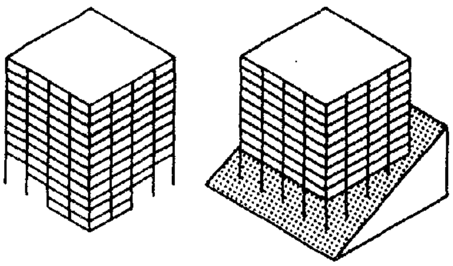


Figure VIII-18. Graphic analysis of the end wall conditions of the Imperial County Services Building.

The solution to the problem of the discontinuous shear wall is unequivocally to eliminate the condition. To do this may create architectural problems of planning or circulation or of image. If this is so, then it indicates that the decision to use shear walls as resistant elements was wrong from the inception of the design. Conversely, if the decision is made to use shear walls, then their presence must be recognized from the beginning of schematic design, and their size and location early made the subject of careful architectural and engineering coordination.

E. Variations in Column Stiffness

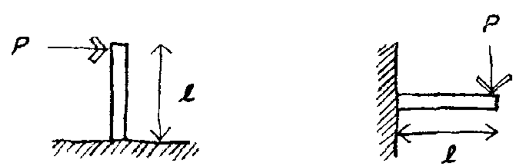


The origin of variations in column stiffness generally lies in architectural considerations: hillside sites, infilling of portions of frames with "non-structural" but stiffening material to create high strip windows, desire to raise a portion of the building up off the ground on tall pilotis while leaving other areas on shorter columns, or stiffening some columns with a mezzanine or loft while leaving others two stories in unbraced height.

The importance of these issues lies in the fact that its effects are counter-intuitive. For example; often infilling may be done as a remodel activity later in building life in which the engineer may not even be consulted, and intuition may suggest to the designer that he is strengthening the column and the structure as a whole, rather than introducing a serious stress concentration.

It would seem reasonable that a short column would be stronger than a longer one of the same cross sectional area: certainly, for vertical loads, it would be less subject to buckling, and hence capable of receiving high loads. But the short column is also stiffer, and under lateral loading, in which loads are distributed according to the stiffness of resistant elements, the short, stiff column will 'attract' forces which may be quite out of proportion to its strength.

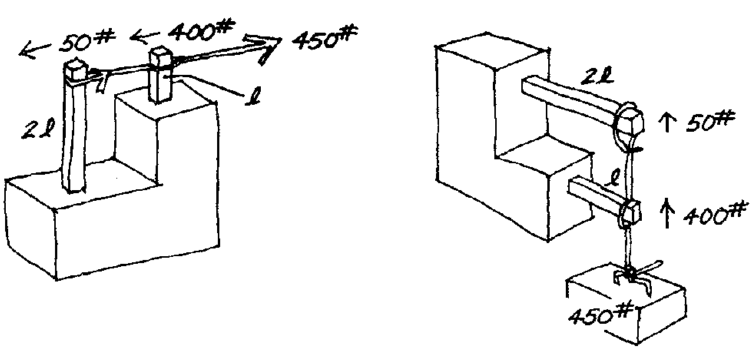
The effect of this phenomenon is illustrated graphically in Figure VIII-19, in which columns of unequal length are also represented, by analogy, as connected cantilever beams of unequal length. The vertical load analogy at once makes clear what is happening. Figure VIII-20 shows an example of damage to a building with this condition.



$$\Delta = \frac{P\ell^3}{3EI} \quad \text{(CANTILEVER)}$$

STIFFNESS VARIES AS THE CUBE OF THE LENGTH.

IF E AND I ARE THE SAME FOR TWO CANTILEVERS, THE ONE TWICE AS LONG WILL BE $2^3 = 8$ TIMES MORE FLEXIBLE.



IF THEY MUST DEFLECT THE SAME AMOUNT, THE COLUMN WHICH IS 8 TIMES STIFFER (i.e. THE SHORT COLUMN) WILL TAKE 8 TIMES THE LOAD OF THE OTHER COLUMN.

Figure VIII-19. The short column vs. the tall column: the results may be counter-intuitive.

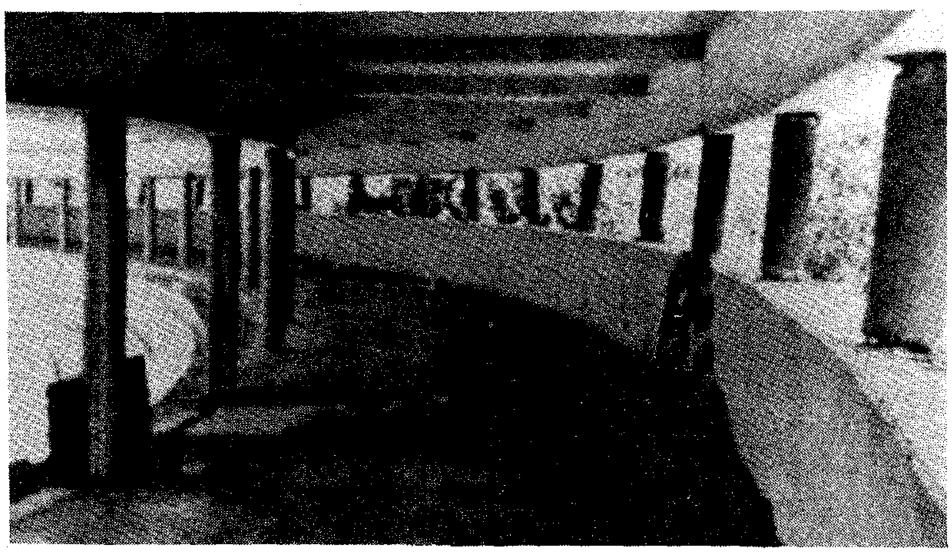


Figure VIII-20. Short column effect results in greater load and greater damage to the shorter columns. Stadium, 1972 Managua Nicaragua earthquake.

If the condition cannot be avoided, a solution is to equalize the stiffnesses of the columns by introducing struts that increase the stiffness of the longer columns. This principle is illustrated in Figure VIII-21, which shows the American Embassy in Tokyo, Japan: the building is on a considerable slope, and it was desirable to maintain a largely open first floor. By connecting columns horizontally at each floor level by a freestanding beam, the stiffness of the columns are made approximately equal.

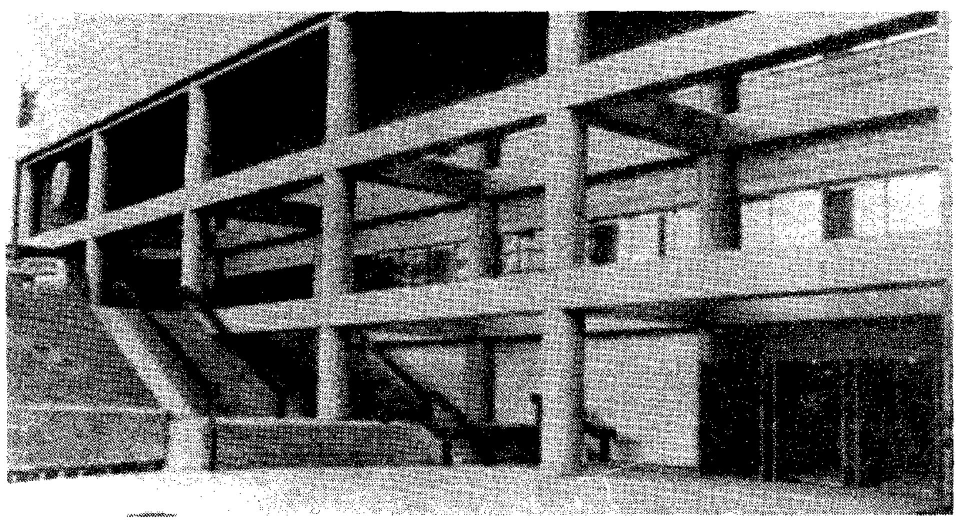
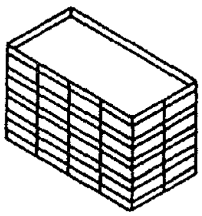


Figure VIII-21. A solution to the problem of unequal column stiffness: the American Embassy, Tokyo Japan.

F. Weak Column, Strong Beam



Even when a building totally collapses, it still has a large amount of relatively undamaged material within its structure. The worst collapse, the "pancaking" phenomenon which occurs when the floors stack up in a pile with only rubble separating them, is due to the destruction of the vertical part of the structure only. The floors still have considerable strength in them, but the fact that they were not destructively overstressed was of no help to the walls or columns.

A basic principle is to design a structure in such a way that under severe seismic forces, beams will perform plastically before columns. This is based on the reasoning that as beams start to fail they will move from elastic to inelastic behavior and start to deform permanently. This action will dissipate and absorb some of the earthquake forces on the same principle as the collapse of the front end of a correctly designed automobile will absorb collision energy and protect the essential structure that surrounds the occupants. Conversely, if the column fails first and begins to deform and buckle, major vertical (compressive) loads may quickly lead to total collapse.

This principle is well understood, but its converse, the design of weak columns and strong beams (Figure VIII-22), is a surprisingly frequent cause of building damage and collapse. The most frequent condition of occurrence is the combination of deep stiff spandrels with reinforced concrete columns in structures such as schools and offices which require (or are assumed to require) long uninterrupted bands of glass inserted between widely separated columns.

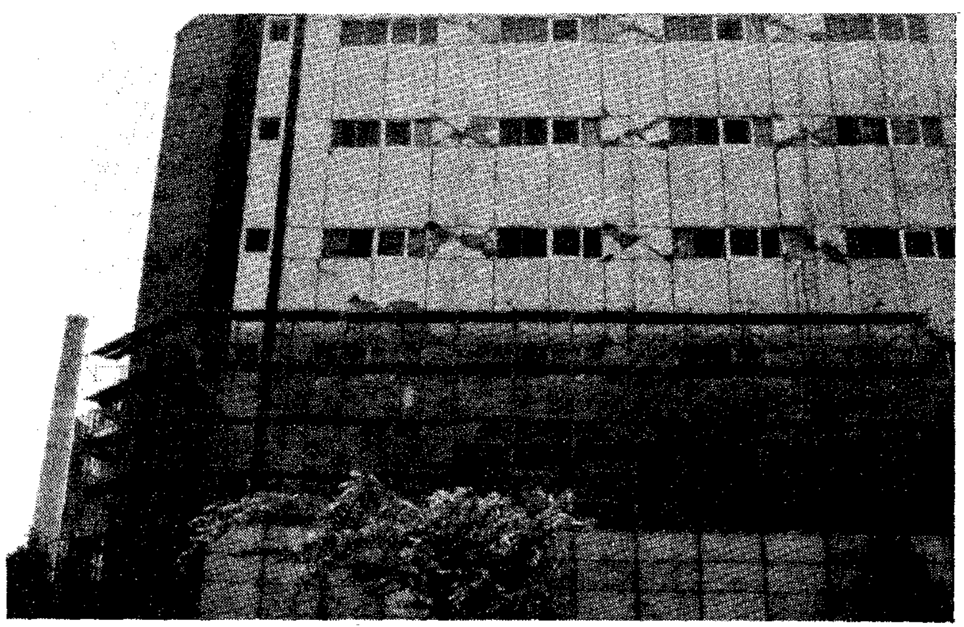


Figure VIII-22. Weak columns and strong beams commonly lead to damage during earthquakes, Maruhon building, 1978 Sendai Japan earthquake.

A detailed example from Japan illustrates this phenomenon. In 1970, Sendai, Japan was the site of the US-Japan Seminar On Earthquake Engineering with emphasis on the Safety of School Buildings. Two different Japanese papers (6) specifically identified a configuration problem common to many Japanese schools: the windows of the north wall are shorter than those of the south.

The spandrels of the north wall are deepened to produce this fenestration condition and with a construction system in which these spandrels are part of the monolithic concrete structure, the north and south walls respond quite differently to earthquakes. The columns of the north wall are shorter and stiffer, and while the more flexible columns of the south wall are accommodating a given amount of movement in flexure the north columns are failing in shear. Since the spandrel beams of the north are so stiff, as the frame deflects, the beams remain essentially straight and most of the deformation is forced into the columns. Differential stiffness of the two longitudinal column lines also causes torsion.

Classrooms are typically arranged in linear fashion with circulation along an exterior gallery corridor. This provides daylighting from two sides, and also results in a relatively narrow (approximately 10m = 33 ft.) building whose frame is one bay wide. If the short columns of the north wall experience enough damage (usually the ground floor columns collapsing the most), the building tilts over toward the north. The 1968 Tokachi-oki earthquake provided many examples of this kind of configuration suffering this type of damage (Figure VIII-23).

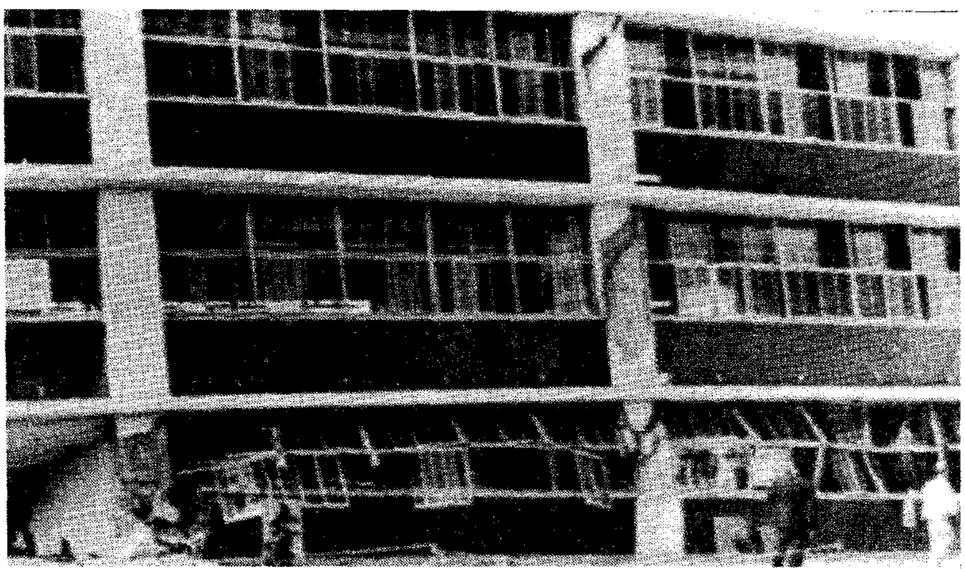


Figure VIII-23. 1968 Tokachi-oki earthquake damaged several school buildings that used the strong beam-weak column configuration. Misawa Commerce High School, Japan.

After the 1970 Sendai conference, many Japanese school buildings were still built in this same configuration. The only difference between the buildings that were damaged in the 1968 Tokachi-oki earthquake and the newer schools of the seventies, was the greater amount of reinforcement required by the building code. When the 1978 Sendai earthquake subsequently occurred, the damage patterns were quite similar: schools of this configuration experienced damage to their short columns. The added strength of members due to revised code provisions for reinforcing, merely delayed the point at which damage occurred, but when subjected to sufficient shaking, the building configuration determined what kind of failure occurred, and where (Figure VIII-24).

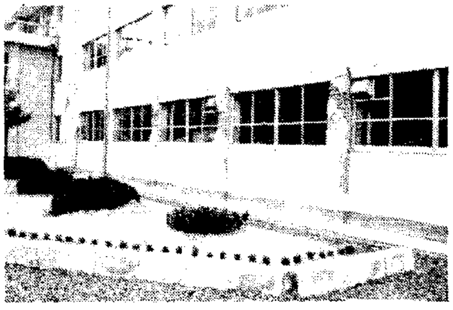


Figure VIII-24. 1978 Miyagi-ken oki earthquake produced similar damage to schools built following a code revision. Izumi High School, Sendai Japan.

The only way to cure the problem is to re-think the basic design, and re-thinking the design necessarily involves the architect, and the educational clients who influence the program, as well as the engineer.

There are a variety of possible solutions. If the original consideration accounting for the facade layout was solar exposure, then a similar-appearing building could be produced which admits light and heat in about the same way but has better structural characteristics. A non-structural curtain wall, part glazed and part opaque, could be used. Alternatively the same structural scheme could be used with a minor revision: provide vertical separation gaps between the spandrel concrete and the columns, so that the columns on both walls are effectively identical in length (Figure VIII-25).

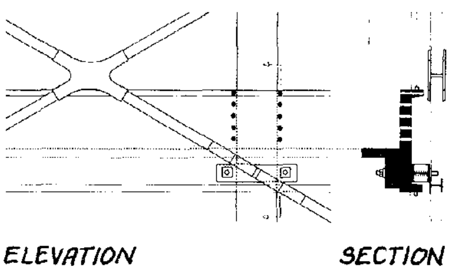
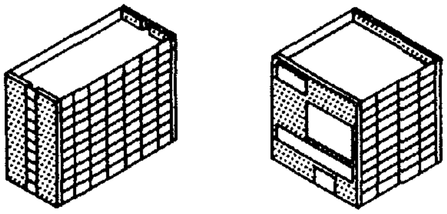


Figure VIII-25. Photo and detail of spandrel-column separation as a remedial measure, Tohoku Institute of Technology, Building #5, Sendai Japan. Remedial measures were undertaken following the 1978 earthquake.

G. Shear Wall and Frame Interaction



The weak column, strong beam condition is a special case within more general problems that arise because of the relationships between shear walls and frames. A weak column, strong beam design can also be defined as a shear wall in which large openings have been placed so as to severely reduce the capacity of the shear wall. As openings are placed in a shear wall, its character may change until it becomes in effect a frame (Figure VIII-26).

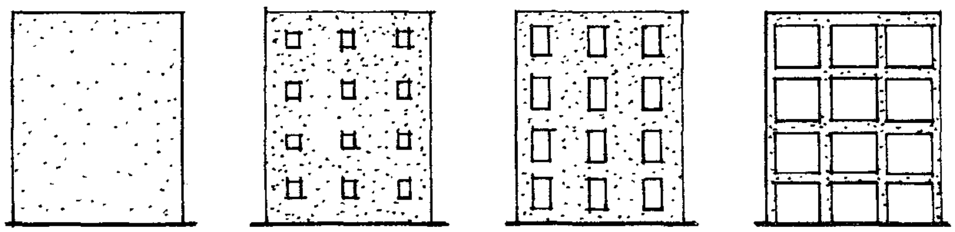


Figure VIII-26. Character of the shear wall changes as size of openings are increased.

The ways in which the shear wall is penetrated or reduced to a frame may cause localized areas of weakness and possible failure (Figure VIII-27). In this diagram, Condition 5 is potentially that of a weak column, strong beam, depending on the precise strength and stiffness of the walls and short columns. If this configuration is further randomized, as in Condition 6, so that a small number of short columns carry the forces, a very poor resistance system is then created.

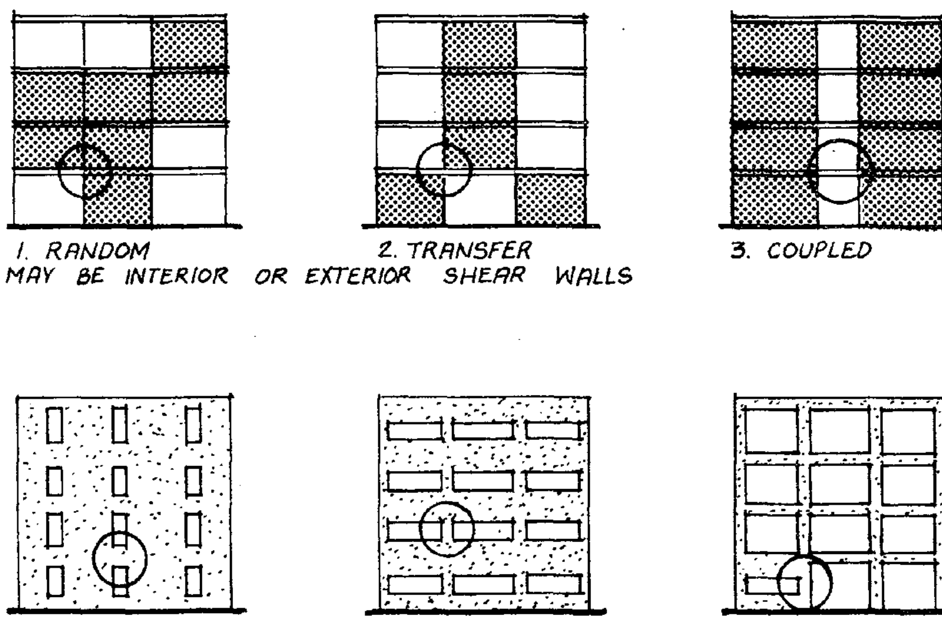


Figure VIII-27. Location of possible failure in shear wall design, caused by size and placement of openings.

The yield behavior of typical shear wall conditions is shown in Figure VIII-28. From this diagram it can be seen that three types of behavior must be contained. The wall must have sufficient capacity to resist shear forces introduced into it at each diaphragm connection; the wall must have sufficient capacity to deal with flexure created by overturning forces; and the wall frame relationships must be able to deal with transfer of forces from wall to frame or from wall to wall through the frame, as in the coupled shear wall system.

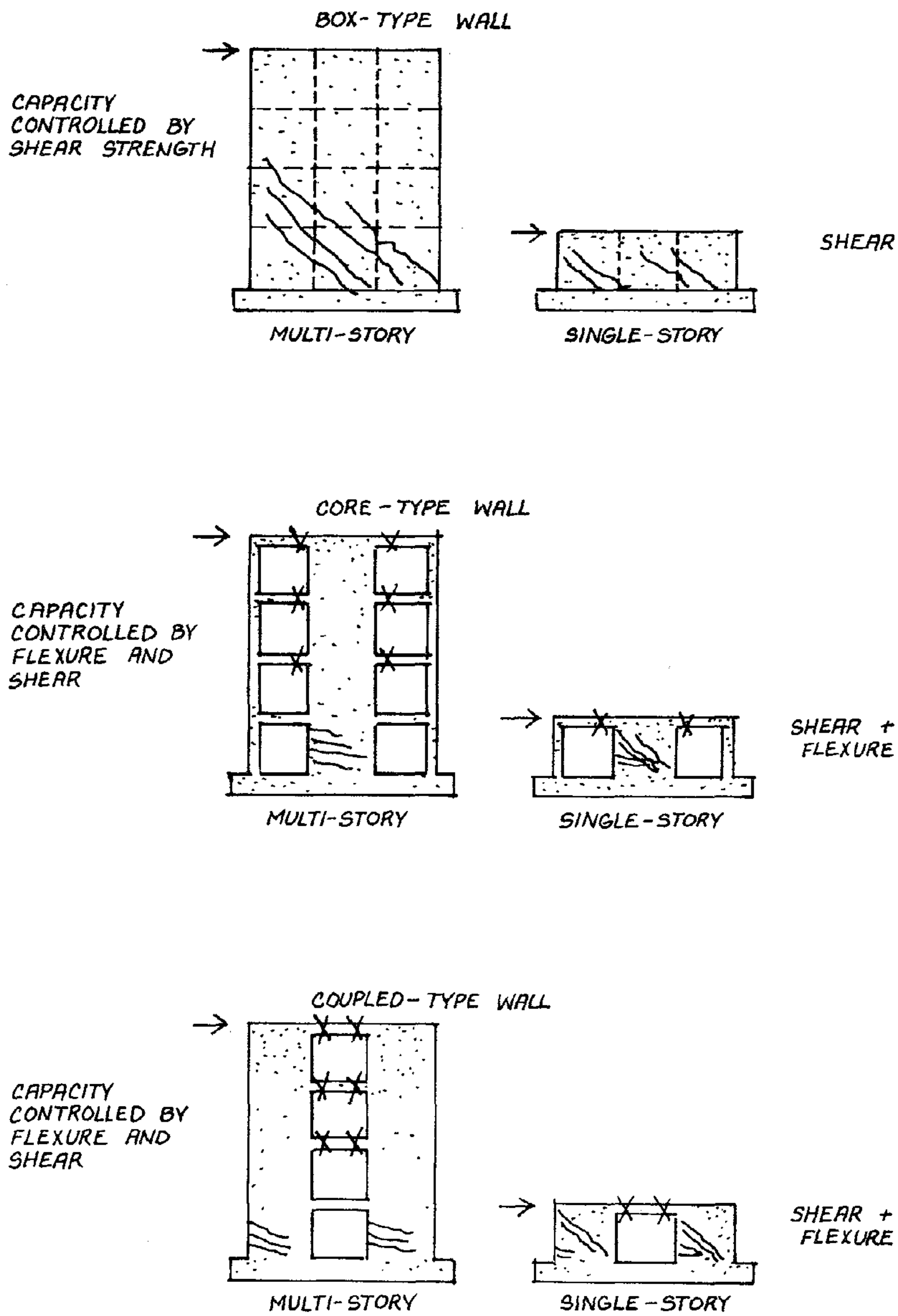


Figure VIII-28. Basic shear wall yield behavior.

The coupled shear wall design is significant because, like the weak column, strong beam solution, it often originates from a useful architectural concept, and from the need to penetrate the perimeter for daylighting purposes, as in the hotel/apartment type configuration shown in Figure VIII-29.

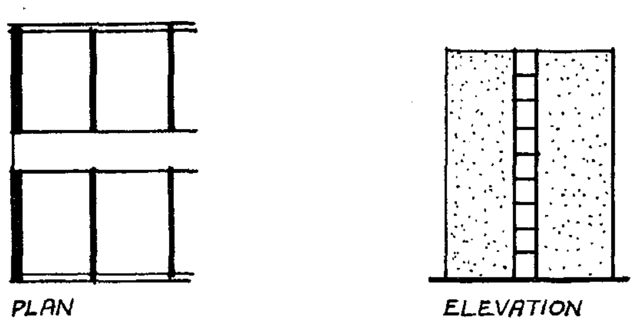


Figure VIII-29. The coupled shear wall is often a result of the architectural program, as in the hotel/apartment configuration.

The coupled shear wall design can also be seen as analogous to the weak column, strong beam condition rotated 90° (Figure VIII-30), although in the coupled shear wall the problems are more likely to occur through flexure of the shear walls particularly if they are tall and slender, and the yielding occurs in the beam rather than in the column, which is obviously more desirable.

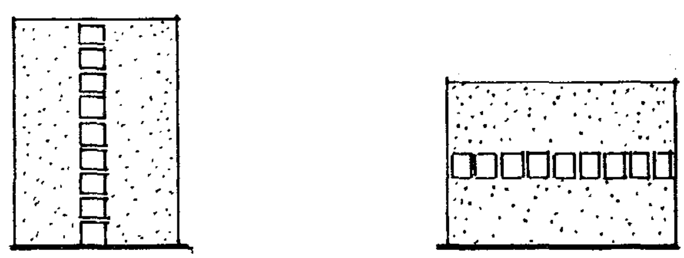


Figure VIII-30. The weak column-strong beam analogy.

Solutions to the kind of problems presented by shear wall frame interaction fall into three groups (Figure VIII-31). The first kind of solution is to prevent flexure damage to the frame by detaching it from the shear wall in such a way that it is free to move without risk of damage. The second solution is to tie frame and shear wall securely together at the foundations, to reduce differential movement. This solution may be suitable for short walls and frames, but will not solve the problems created by tall, slender walls. For these the solution lies in connection also at the top, with a high capacity transfer beam.

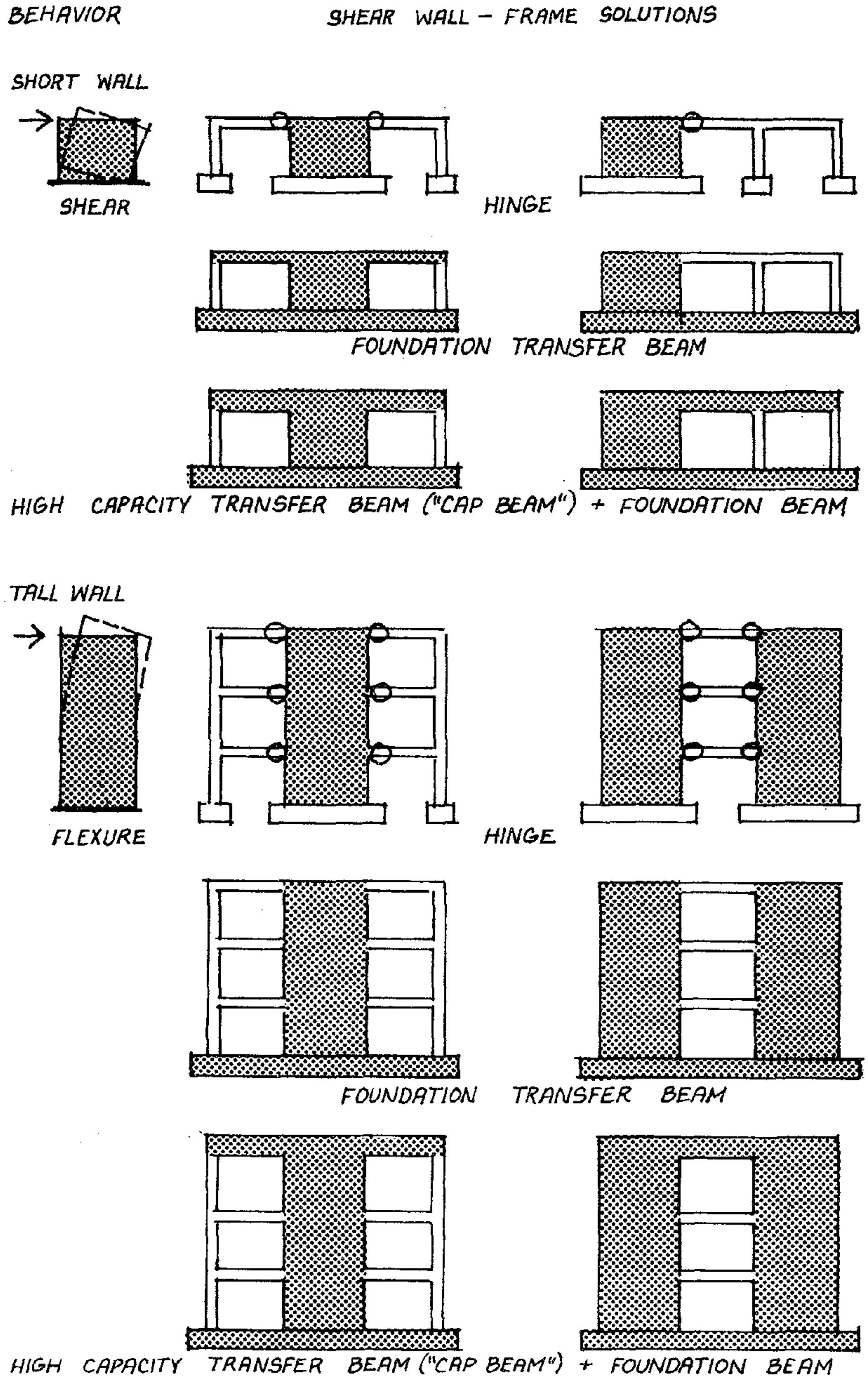


Figure VIII-31. Solutions to the shear wall problem.

It should also be pointed out that in wall-frame designs such as the coupled shear wall, yielding and deflection of the frame may perform a useful energy absorbing function, acting as a second line of defense as the capacity of the shear wall begins to be exceeded.

Sophisticated design solutions can be envisaged in which the size - and hence cost and architectural restraints - of shear walls can be reduced to provide resistance capacity to deal with moderate ground motion, which will occur much more frequently than the great earthquake. When very severe ground motion occurs, additional capacity is provided by energy absorption in a frame, designed to yield in a way that risk of collapse is minimized, and combined with architectural and other non-structural detailing that will limit the economic and functional losses due to non-structural damage.

To adopt this approach with confidence needs further research into the nature of ground motion and building behavior, and more understanding of the interactions between different elements of building structure and other building components. The prime incentive would be economic: to design a structure in which the relationship of its cost and performance is more closely matched to the probability of the conditions that it will encounter during its lifetime.

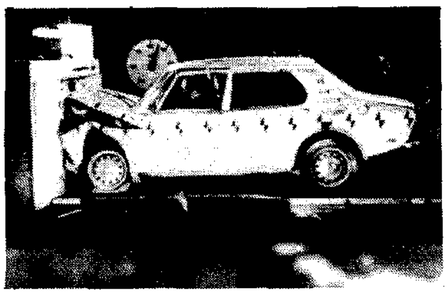
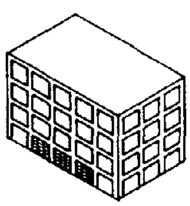


Figure VIII-32. Analogous impact absorption capacity of automobiles.

The approach is analogous to that used in the design of automobiles: rather than the entire vehicle being constructed to remain intact in a major impact, the design is accomplished in such a way that front and rear ends provide major impact absorption capacity, and detailed design of items such as steering columns are designed to break before they present a hazard to the occupants (Figure VIII-32).

H. Non-Structural Modifications



The effects of the addition of non-structural elements which seriously change the dynamic behavior of a structure have already been touched on in looking at variations in column length (also Chapter III, Section K: Non-Structural Elements).

Random stiffening of a frame structure by masonry infill is a common instigator of damage and failure. The mechanism is always the same: the earthquake forces are attracted to the areas of greatest stiffness, and if these are not designed to accommodate these forces, they are prone to fail (Figures VIII-33,34).

Figure VIII-33. Damage to a column which was effectively shortened, and hence stiffened, by "non-structural" infill, Student Union Club, University of Skopje, 1963 Skopje Yugoslavia earthquake.

Olive View hospital included this problem along with its others. Masonry enclosure walls were supposed to act non-structurally but performed structural roles until failure, by accidentally introducing stiffness into the structure at random places. Separation was intended, but (7):

"Seismic gaps provided between the first floor slab and the masonry walls in the ground story, and between the first story elevator walls and the structural members below the second floor level, were not large enough or detailed to achieve the desired degree of isolation. Considerable damage resulted from the pounding or interaction of adjacent components that were intended to be separated. Careful detailing and realistic assessments of relative deformations are required to achieve effective seismic separation."

Unintentional interaction has two detrimental effects: the non-structural component is unnecessarily sacrificed, and, from a configuration standpoint, the stiffness introduced at random places can redistribute loads unequally and produce torsion.

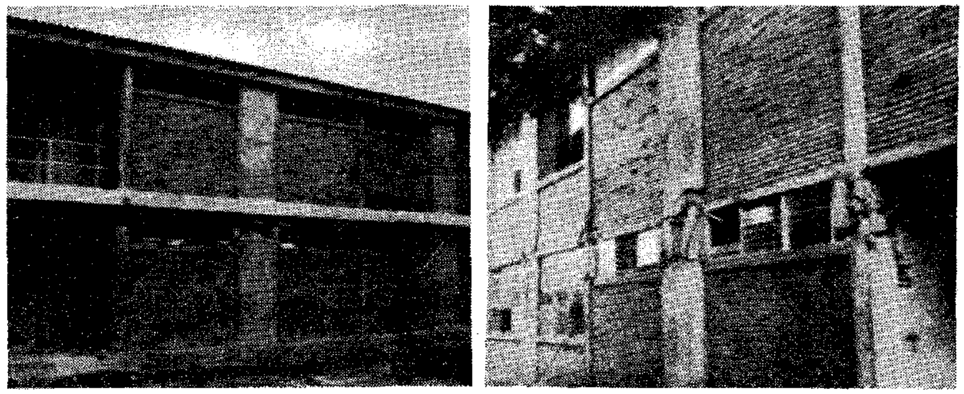


Figure VIII-34. Damage to short, stiff columns created by non-structural infill, Simon Bolivar Central School, 1972 Managua Nicaragua earthquake.

The possibility of accidentally involved modifications is reduced if a careful check of the design is made as working drawings are completed, and all architectural elements are shown which may not have been apparent to the engineer when he worked on the structural design. A special warning should be noted in respect of "fast track" projects, in which additional design is completed before all non-structural design decisions are made. In the haste to meet late program requirements, stiff walls may be added to a structural design which is complete, or under construction.

In general, random infill walls within frames should be avoided, particularly of heavy materials. But even though a stud and gypsum board wall is regarded as non-structural, it may have considerable though unquantified stiffness. Infill walls either should be figured into the structural concept, and detailed accordingly, or detached in such a way that structural distortion will not cause the wall to become stressed. To do this requires some analysis of expected drift, and the development of architectural details that will retain the wall securely in place against normal vertical and lateral loads, and yet allow for movement relative to its frame.

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IX. Adjacency Problems: Pounding

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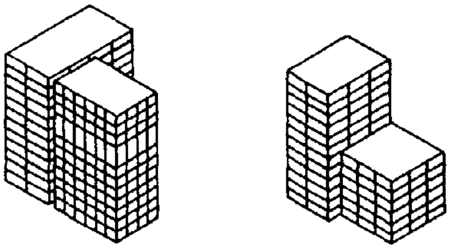
A. The Problem



The problem of two adjoining buildings or parts of the same building pounding together in an earthquake relates to two issues previously discussed: separation joints (if the structures are architecturally connected to form one building) and stiffness (since this affects drift and hence the amount of separation required to prevent contact). Pounding is included in a discussion of configuration issues because it is a matter of where buildings are located, relative to other structures.

Figure IX-1. Gran Hotel damaged in the 1972 Managua Nicaragua earthquake.

Pounding has been noted routinely by earthquake investigators over the past several decades. In the 1972 Managua earthquake, the five-story Gran Hotel suffered a complete collapse of its third story when the battering ram phenomenon occurred at the roof level of the adjacent two story building (Figure IX-1). The six-story Lang Building experienced severe damage where it abutted a lower building at the lower building's roof height, partly due to actually pounding but also perhaps due to the stiffening effect of the adjoining building which momentarily created, in effect, a discontinuity in vertical stiffness (Figure IX-2).

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Figure IX-2. Pounding damage suffered by the Lang Building, in the same earthquake. For another view of the Lang Building, see Figure VII-8.

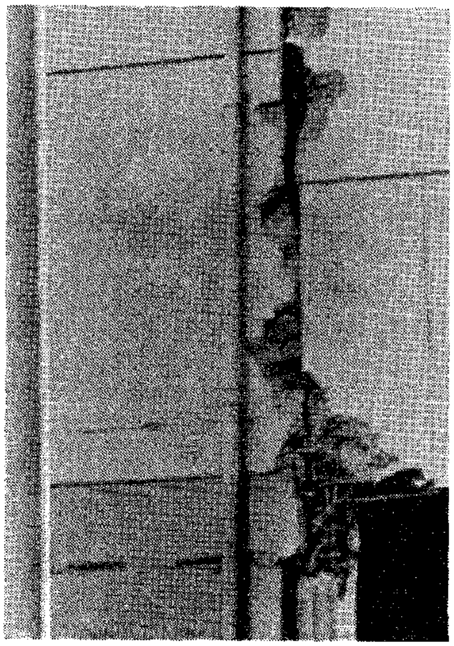

Figure IX-3. Damage due to pounding, Anchorage Westward Hotel, 1964 Alaska earthquake.

In the 1964 Alaska earthquake, the fourteen-story Anchorage Westward Hotel pounded against its low-rise ballroom and an adjoining six-story wing, although separated by a four inch gap (Figure IX-3). The pounding was severe enough in the high-rise to dislocate some of the metal floor decking from its steel beam supports.

Two low (one story and three story) buildings at the Alaska Methodist University (1) were separated by a two-inch gap, but rigid insulation filling this space was apparently stiff enough to transmit compression between the two buildings, because pounding damage resulted.

The 1967 Caracas earthquake provided further illustrations of the pounding problem: the hammering between components of the Macuto Sheraton, between two apartment buildings, and the destruction of one corner of the Nobel building when the adjacent Mijaqual building collapsed.

B. Solutions

Blume, Newmark, and Corning have described the basic problem and suggested possible solutions (2).

"One of the first problems to be settled in the planning for any building is its location in relation to the property lines and adjacent structures. It is generally recognized that buildings sway during earthquakes, but it is not always realized that adjoining buildings can sway out of phase - first away from each other and then toward each other, each in its own natural period of vibration... Buildings that must be separated into units because of temperature movements or for other reasons should have separations so detailed as to avoid the possibility of hammering.

"Building code provisions for separation of adjoining buildings or components have never been too satisfactory, largely because of the various other problems involved. In the congested business portion of most cities, land is extremely valuable. There is usually strong objection when a proposed building code requires a large separation because of the decrease in the usable area and the financial return from the property...

"The SEAOC code [Recommended Lateral Force Requirements] has the following statement regarding building separations: 'All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.'

"The question of what width of separation is sufficient must be considered primarily a matter of engineering judgement. Arbitrary rules could cause severe hardship in some cases and be inadequate in others."

The possibility of pounding, then, is a function of drift, or vertical deflection of adjoining buildings (or parts of a building). Drift is calculated by applying the design forces to the building and then deriving the deflections which result. Since the estimated forces will be less than what we know can occur, calculated deflections must be corrected for this to get a more realistic estimate of how much the building may actually move. Alternatively, an accurate estimate of drift may be made which accounts for all foreseeable factors.

Blume, Corning and Newmark suggest an alternate method(3).

"A less rigorous appearing rule, but one which may in fact be both more accurate and more rational, is to compute the required separation as the sum of the deflections computed for each building separately on the basis of an increment in deflection for each story equal to the yield-point deflection of that story, arbitrarily increasing the yield deflections of the two lowest stories by multiplying them by a factor of 2."

The 1976 Uniform Building Code prohibits drift from exceeding 1/2% of the story height; for a 16'-8" story height this would be 1" of drift.

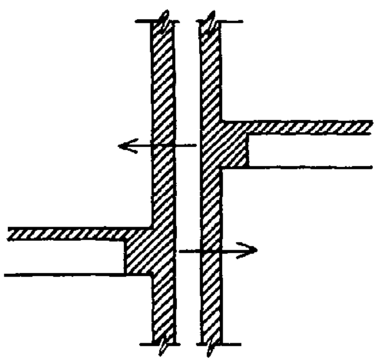

Figure IX-4. Stiff elements, such as floor slabs, not adjacent to each other, will produce more severe damage.

An earlier edition of the Uniform Building Code contained a rule of thumb intended for the relatively stiff structures of that day (4): separations should be "one inch (1") plus one-half inch (1/2") for each ten feet (10') of height above twenty feet (20')." A Russian text (5) suggests 1-1/4" of separation for buildings up to 16' tall, and an additional 3/4" for each additional 16' of height.

Although as noted above, such rules of thumb can be arbitrary, they have a valid purpose for use as a schematic design allowance before analysis can provide more precise figures. Note also that the location of stiff elements is important: if they are not adjacent, the problem becomes worse (Figure IX-4), and particular attention should be paid to ensuring that the buildings will not pound one another.

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X. Configuration Derivation

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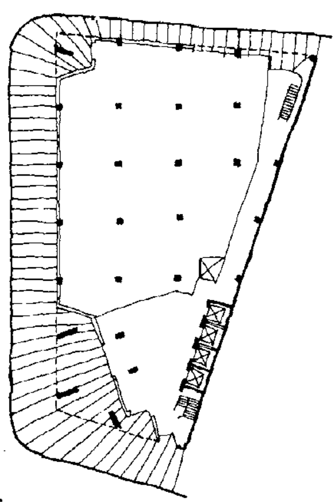
A. Introduction

We have earlier defined configuration as the overall size and shape of a building, together with the size, nature and disposition of those elements of the building that are significant to its seismic performance. These include such elements as walls, columns, floors, service cores, staircases, the quantity and type of interior partitions, and the ways in which the exterior wall is left solid or perforated for light and air.

Just as the principles of seismic design must be understood in order to clarify the relationship between configuration and seismic resistance, it is also necessary to have some conception of the range and nature of the parameters that determine configuration.

We have already noted that building configurations seem to the observer to be so varied that their derivation might seem to be random, even whimsical. We suggest here that this is not so: that there are identifiable determinants of configuration and that even a necessarily brief indication of some of them will improve our general understanding of configuration and the extent to which it can be modified to accommodate seismic requirements.

There are three major influences on building configuration: the requirements of site, the requirements of the building program, and the requirements of imagery, or aesthetic aims (Figure X-1). The first imposes constraints of site geometry and location. The second represents the requirements imposed by the interior planning of the building to respond to its requirements of use, or occupancy. The third requirement represents the designer's desire for physical images that express the aspirations of the building owner, the users, and of course, the designer himself. We will also touch briefly on other influences in order to show the range and complexity of the issues with which the building designer is faced.



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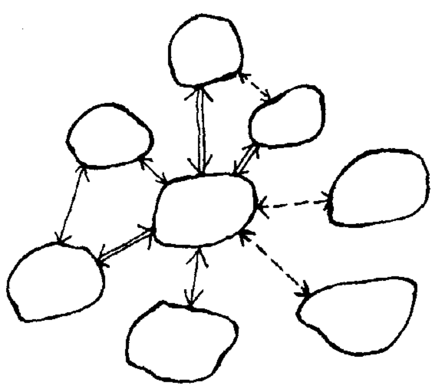

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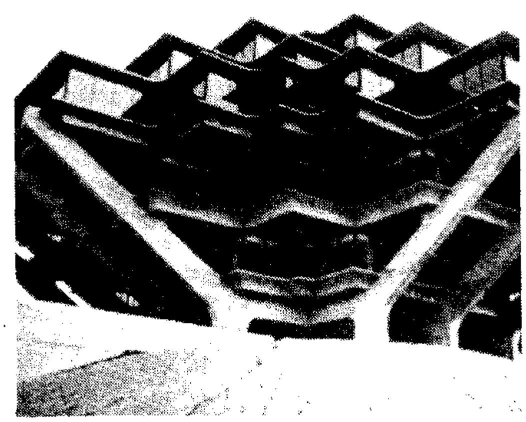

IMAGE

Figure X-1. The three major influences on building configuration: site, program, and image.

The choice of configuration originates in the function of the building. Since building function is a term much used, but seldom understood, it is worth while trying to define function in terms that reflect the multiplicity of building types and intentions. In so doing, we can begin to clarify the whole issue of the purpose of a building.

Confusion as to building function generally stems from definitions that place building function in opposition to building art. This opposition is expressed as antagonistic: design can be directed

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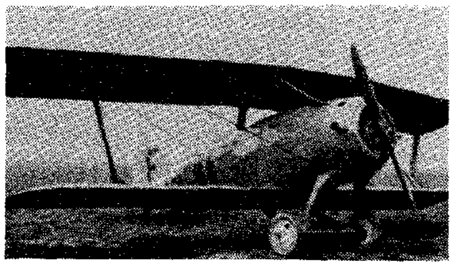


Figure X-2. The co-existence of art and use: an example from Le Corbusier, the SPAD 33 BLERIOT, a passenger plane.

towards use and efficiency, or towards aesthetic expression, but cannot be both. An alternative theory expresses the co-existence of art and use: this theory was embraced by the functionalists of the 1920's in proposing that the art, or aesthetic pleasure of building, lay in the precise expression of building purpose.

Le Corbusier, in his influential books of the 20's and 30's, used examples of co-existence from engineering - ships, cars, planes - to illustrate this view (Figure X-2). A very simplistic version of the idea of aesthetic and utilitarian co-existence became the official posture of the architectural profession: the architect's design will both solve your problems of use, and provide you with a work of art. Such a posture continues today in the face of often overwhelming evidence that such a position is often neither possible for the professional, nor desired by the owner.

B. Determinants

One approach to resolving this kind of simplistic inconsistency is that taken by two English researchers (1) who have suggested that all buildings attempt to perform four functions, which are expressed as modifiers. Thus the building acts as a climatic modifier in providing a special micro climate for its users, and as an economic modifier through the ways in which it modifies the economy by its presence. The building also acts as a behavioral modifier by affecting the ways in which people live, work and play, and finally the building is a symbolic modifier by virtue of its image that affects the building's owners, users, and observers. This last concept includes the traditional 'art' of architecture.

This four function model is useful because it recognizes the co-existence of values in any building concept, and recognizes that any building may have a different mix of emphasis among these attributes. The National Art Museum in Washington is no less "functional" a building than a suburban warehouse, but the emphasis placed on each function is different.

The relative emphasis of the four functions of the building, establishes a context within which the designer goes to work. Of the four, the cost context is usually the critical modifier, and the relations between building configuration and building cost have some parallels to those between configuration and seismic design. The simple, regular, repetitive form will tend to be both the most economical and the most intrinsically trouble-free seismic configuration.

Consideration of a specific functional attribute within the general model may involve considerations of aspects of project development that seem far removed from configuration, yet may have significant effect. Type of ownership and its impact on building form is an example of such a characteristic, within the economic function.

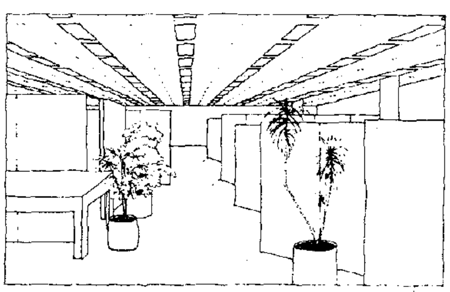


Figure X-3. The easily subdivided floor of an office building.

The building designed for lease is a clearly significant commercial type and the design impact of this ownership concept has been little noted. A whole technology of interior building components has been developed to meet the requirements for easy space and service rearrangement: such elements as the demountable partition and integrated ceiling. But the needs for this kind of space also have direct configuration implications. One result is to provide building plan forms - floors - that can be easily and effectively subdivided geometrically (Figure X-3).

The other result is to establish a preference for frame structures that minimize the extent of large fixed elements (such as shear or braced walls) in the building interior that may inhibit the ability to provide a future tenant with the space he wants. Such structures have an obvious impact on the nature of the seismic resistance system.

As building costs escalate, buildings will tend to become more permanent because the economy cannot afford their replacement. At the same time, the rate of occupancy change requires more rapid rearrangement of the building interior. The impact of improved building standards in seismic areas is already resulting in structures that will have a longer life - as support systems - than any we have known previously in this country. The horizontal, vertical and lateral structure for Loma Linda Hospital (described in Chapter XIII) will probably have a useful life of around 200 years (Figure X-4). At the same time, it is unlikely that even the general planning arrangement within the shell will be valid for more than 40-50 years, and details of planning layout and service systems that respond to medical technology will become outdated within a decade.

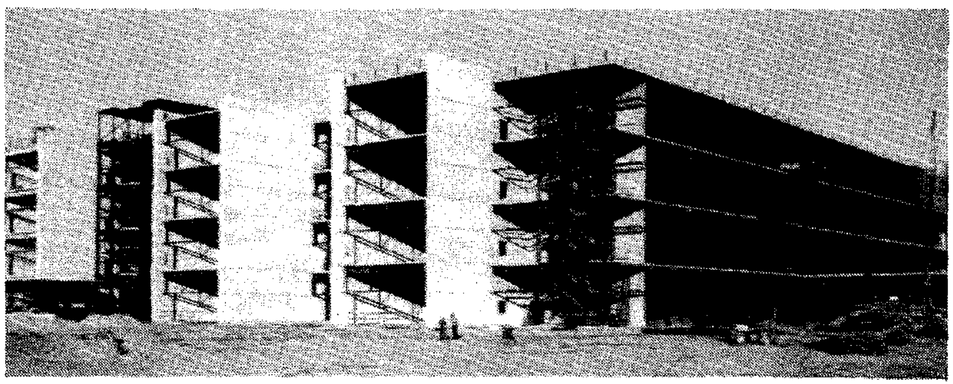


Figure X-4. A possible 200 year structure: Loma Linda Veterans Administration Hospital, Loma Linda California.

The development of modern air-conditioning systems in the last 30-40 years has decreased the impact of climate on building form. However, this era may now be ending and the thermal and daylighting aspects of climate may once more have a significant impact on both general and specific formal considerations. These impacts need not be caused by vast arrays of sloped solar collectors: the changes can be more subtle and related to such traditional design problems of building orientation and the scientific evaluation of the proportions of opaque and insulated wall glazing. We may expect to see the return of the varied facade, that presents a different pattern of solid, void, projection, and recession according to orientation, instead of the uniform facades to which we have become accustomed.

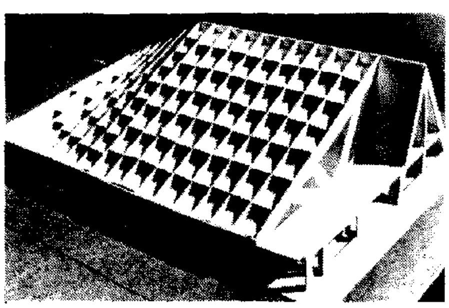


Figure X-5. Form of building is determined by solar access.

Some of the zoning requirements discussed later in relation to office building design have their origin in climatic requirements for light and air. We may expect to see a revival of these factors as the question of solar rights, both for solar energy systems and daylight, again assume importance (Figure X-5). The configuration aspects of these may be of great significance. It would be a safe generalization to say that the impact of all climate considerations and configuration will be to tend towards variety and non-uniformity in facade treatment, and towards a re-evaluation of familiar building forms.

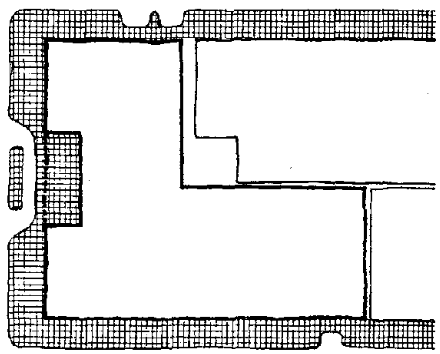


Figure X-6. The shape of the site becomes the shape of the building.

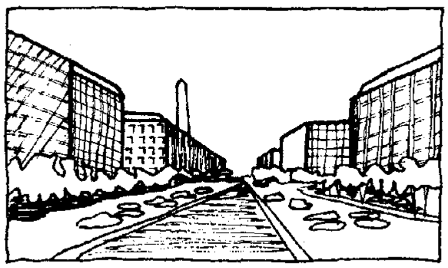


Figure X-7. Zoning frequently produces other characteristic configurations.

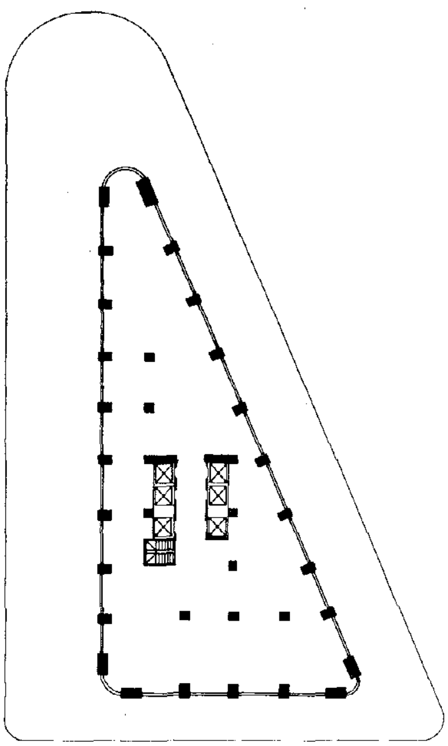


Figure X-8. The Flat Iron Building in New York; a shape produced by a non-gridiron street pattern.

Site geology is well recognized as a determinant of the nature of ground motion. The interactions between site geology, earthquake intensity and location, foundation design, and seismic design are discussed elsewhere in this study.

There are, however, aspects of the site that influence configuration in ways that are unrelated to purely seismic considerations and that may even be in conflict with such concerns. These are such characteristics as those of site geometry and location in relation to urban design considerations, which are expressed specifically in zoning requirements that mandate building setbacks, height limits, floor area ratios and the like.

As building sites become smaller, site geometry becomes more critical as a determinant of building shape. In the suburban setting, there is a greater tendency for buildings - even large buildings - to be free standing structures relatively uninfluenced by site geometry. But in the urban situation, the reverse is true: the shape of the site, as modified by setback requirements, even becomes the plan shape of the building (Figure X-6).

As land costs increase, city sites tend to become smaller, and financial viability demands the maximum site usage - the largest and the most floors that can be developed.

These pressures have resulted in zoning requirements that, to some extent, run counter to real estate economics: in other words, that attempt to limit site coverage, building height, or both. Regulation is generally based on aesthetic grounds: the undesirability from an urban design viewpoint of filling up city lots to unlimited height and maximum coverage (Figure X-7).

The typical geometry of the American city is based on a grid-iron pattern that results in rectangular blocks and rectangular building sites, whether they represent entire blocks or a portion of a block. However, it is also characteristic of city planning that prominent streets may cut diagonally across the grid-iron (as does Broadway in New York City) or different portions of a grid-iron plan may adjoin one another on a non-rectilinear basis, creating a diagonally intersecting street where the two portions meet (as does Market Street in San Francisco). The result is a notable incidence of triangular, or trapezoid shaped, building sites, resulting in building of like form of which the "flat iron" building in New York City is a well known example (Figure X-8).

While site geometry and zoning requirements may, to a varied extent, influence the form of the building envelope, the detailed development of the form is dominated by interior planning requirements.

The interior planning problem can be simply expressed: it is that of arranging appropriate spaces - in size, shape, equipment and quality - for the activities they support, and that of enabling people and materials to move from one activity space to another.

For all the variety and complexity of building plans, their solution to the planning problem of horizontal movement rests on a choice between, or combination of, two basic principles. These are to move from one activity space directly to another, or to move from activity space to circulation space - space dedicated to the activity of movement - to another activity space. The details of planning are familiar to designers. Horizontal planning uses space-to-space planning, single loaded corridors, or double loaded corridors. Figure X-9 shows an example of planning that incorporates all these planning methods.

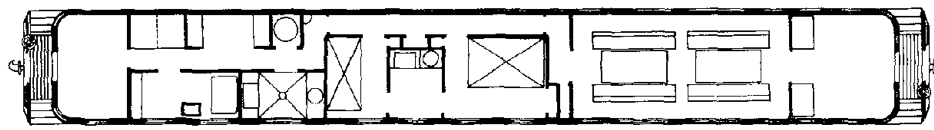


Figure X-9. A proposed railroad car design exhibits a wide variety of planning methods.

Sometimes in the effort to reduce the circulation/activity space ratio, buildings consist of combinations of double loaded corridor space. This kind of planning is particularly characteristic of buildings, such as schools, laboratories, and hospitals, in which efficiency and construction economy are of paramount concern. A particular form of planning of this type was evolved for hospitals, and is sometimes referred to as the 'race-track' plan. Many offices, with an interior core of elevators, and other service elements, also use this planning (Figure X-10).

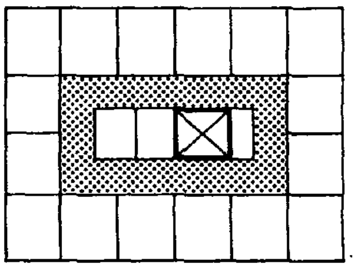


Figure X-10. The race-track plan.

The impact of vertical structure on these planning concepts may take two forms. The structure may define the planning concept, as in the use of load bearing walls to define a cellular repetitive arrangement of rooms for a hotel or apartment. Or, the structure may be independent of the interior space division, as in the office building designed for tenant occupancy (Figure X-11).

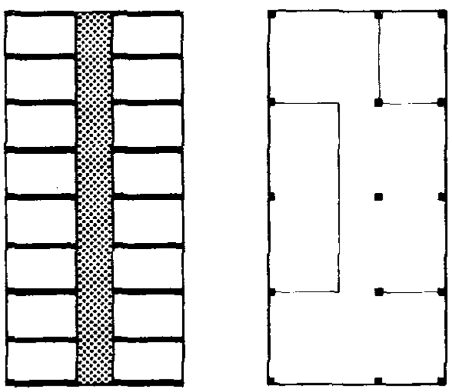


Figure X-11. Alternative approaches for dealing with the planning/structure relationship.

The need for structure to respect safety requirements for clear and direct circulation paths is paramount, and structural obstructions which block public hallways are highly undesirable. While the location of partition walls may change to reflect changes of use, circulation routes will generally remain inviolate. For this reason, the definition of circulation routes by structural elements, which by their nature will remain fixed for the life of the building, is a sound strategy.

Building configuration is three-dimensional: besides the allocation of space horizontally, the planner is faced with a vertical dimension and, in any building over one story in height, with vertical movement. The height of the building is of seismic concern, and is determined by the number of floors and the floor-to-floor (or floor to roof) height.

The number of floors results from the integration of a number of variables such as site size, building cost, building area requirements, and floor area needs. The ceiling heights, or more accurately, floor-to-floor heights (Figure X-12), are determined by activity requirements and economy: the code plays a major part at the low end of the cost spectrum by setting minimums for certain activities.

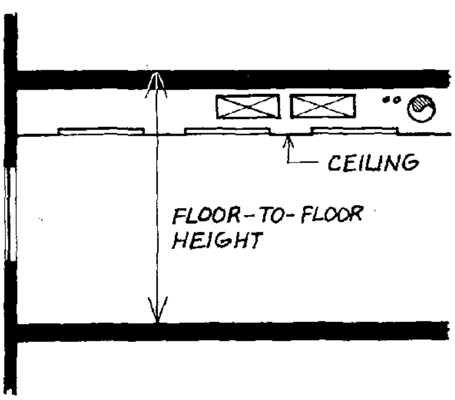


Figure X-12. The floor-to-floor height.

Once a multistory design solution is postulated, two design decisions become fundamental: the way in which floors are layered one above the other, and the way in which vertical movement between floors is arranged.

The design of mezzanines, galleria and other such types of space, tend to demand high ceilings, wide spans, cantilevers, bridges, and other devices that often end up as sources of structural discontinuity and unbalance.

The seismic significance of the stair lies in the fact that it is a fixed element of the building structure, and it may represent a point of localized stiffness, in which case it will receive a disproportionate share of the seismic forces. This is unfortunate because it is also an essential element in the safety planning of the building, and it is particularly necessary that stairs should remain intact in the event of fire or earthquake. A stair may also create an interruption, or 'hole' in a floor diaphragm, unless it is designed as an attachment to the floor outside the main diaphragm.

While we customarily think of elevators as shafts enclosed by walls, in fact the design requirement for elevators is simply that of a hole in the floor, and a consequent diaphragm interruption. The enclosing shaft walls may form part of the building vertical and/or lateral structure, but there is no necessity for this, since the elevator car is supported at the top and requires only guide rails at its sides.

Within the range of rational determinants, there remains ample scope for a wide variety of overall formal solutions. Yet, at any given time, there is a strong tendency for some formal solutions to be selected over others. These impulses - which are stylistic, aesthetic concerns - are very powerful. They serve as driving forces, and their presence quickly resolves a number of variables which, without these would take far longer, or even be impossible, to resolve.

An example of this kind of impulse was the world-wide formal drive towards the simple, rectilinear form of the tall office building between approximately 1950 and 1965. The roots of this drive can be traced back to the influence of certain key figures in modern architectural history: the drive is unmistakable, and it is symbolic rather than economic, climatic or behavioral. In fact, it runs counter to climatic effectiveness.

The building as a rectilinear form represents the concept of the building envelope conceived as a pure geometric element within which functional elements - rooms, departments - are incorporated but not expressed. The opposite of this conception is the building conceived as an assembly of elements, each of which is expressed formally.

This latter approach, of the building as a collection of elements, is clearly a very strong impulse for many designers at present, and the rectangular box is out of favor. Of course, many buildings contain elements of both approaches, but it will be found that aesthetically significant buildings tend to show a specific image without compromise (Figure X-13).

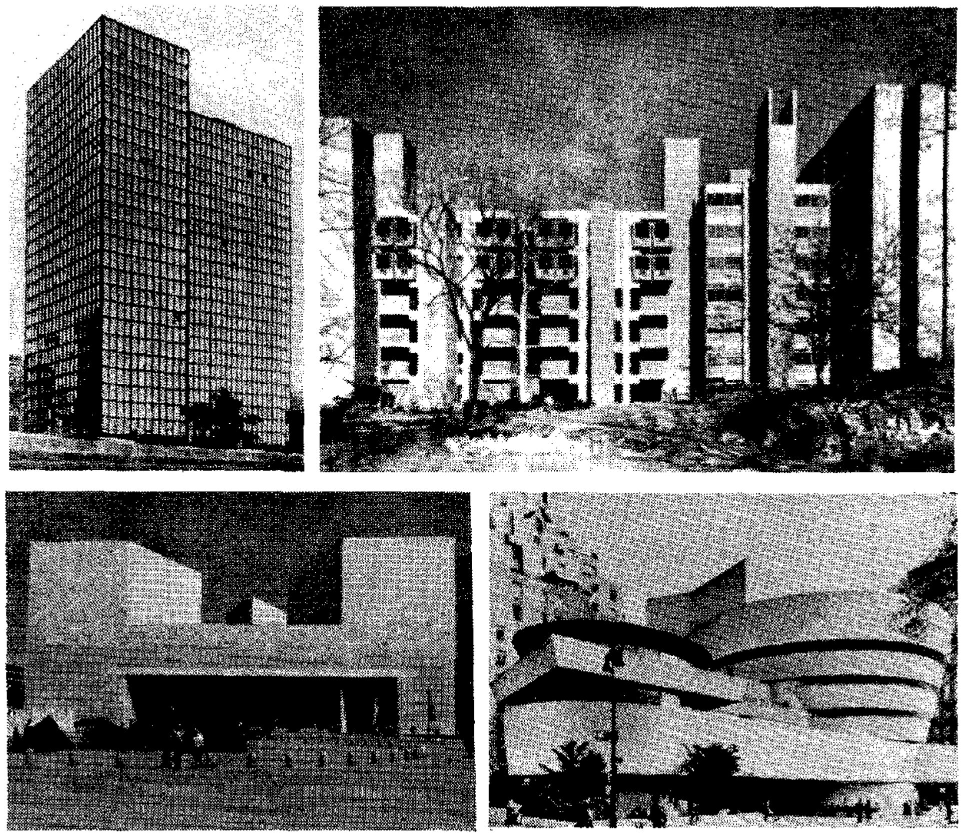


Figure X-13. Image without compromise.

In looking at the derivation of form then, we can identify two determinants: those we may call, for want of a better term, rational: and those we may call symbolic. The rational requirements of the building are accomplished within a symbolic form. To paraphase a famous aphorism, we may say not that form follows function, but that function creates a need for form, and it is our stylistic impulses that provide it. In order to illustrate the interaction of the various determinants that we have discussed, it is useful to study building configuration as it is expressed in the recent history of a familiar contemporary building type: the office.

C. Office Building Configuration Development

Since its origin as a building type in the early nineteenth century, we can identify four distinct phases in the development of office building shape. Our survey is necessarily simplistic, and at any point buildings can be found that do not represent these dominant forms, or which appear as combinations or transitional building types as one phase is replaced by another. The categorization of our four phases is based on issues that significantly influence the three dimensional form of the building (Figure X-14).

The first phase of office building configuration lasted until the early 1940's. During this period, which was also notable for the rise of both public and private bureaucracies, the building form was dominated by the need for natural ventilation, and, even more important, daylight. Although mechanical ventilation existed, artificial cooling, in general, did not: ventilation was accomplished by operable windows augmented, in hot summer locations, by

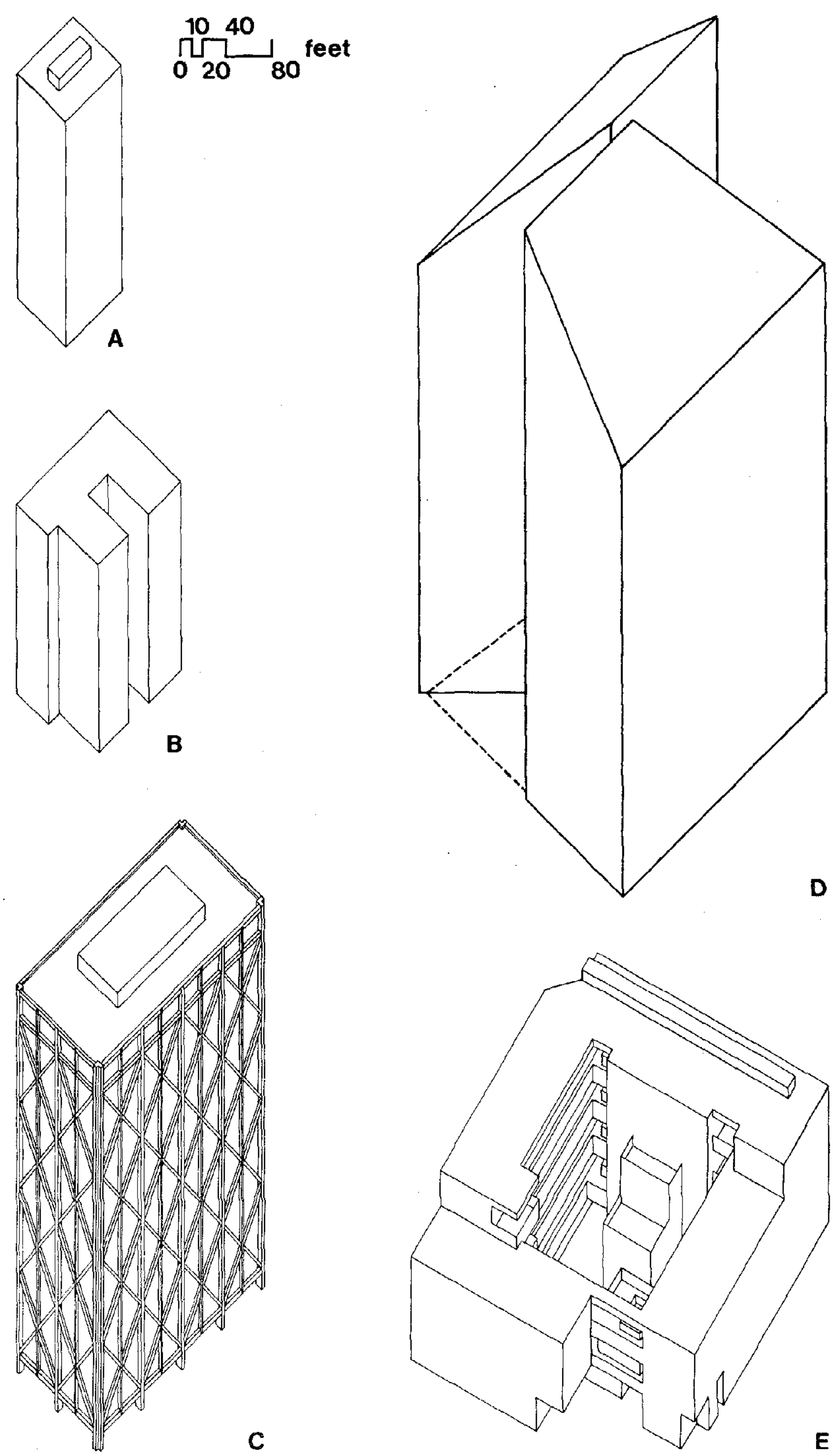
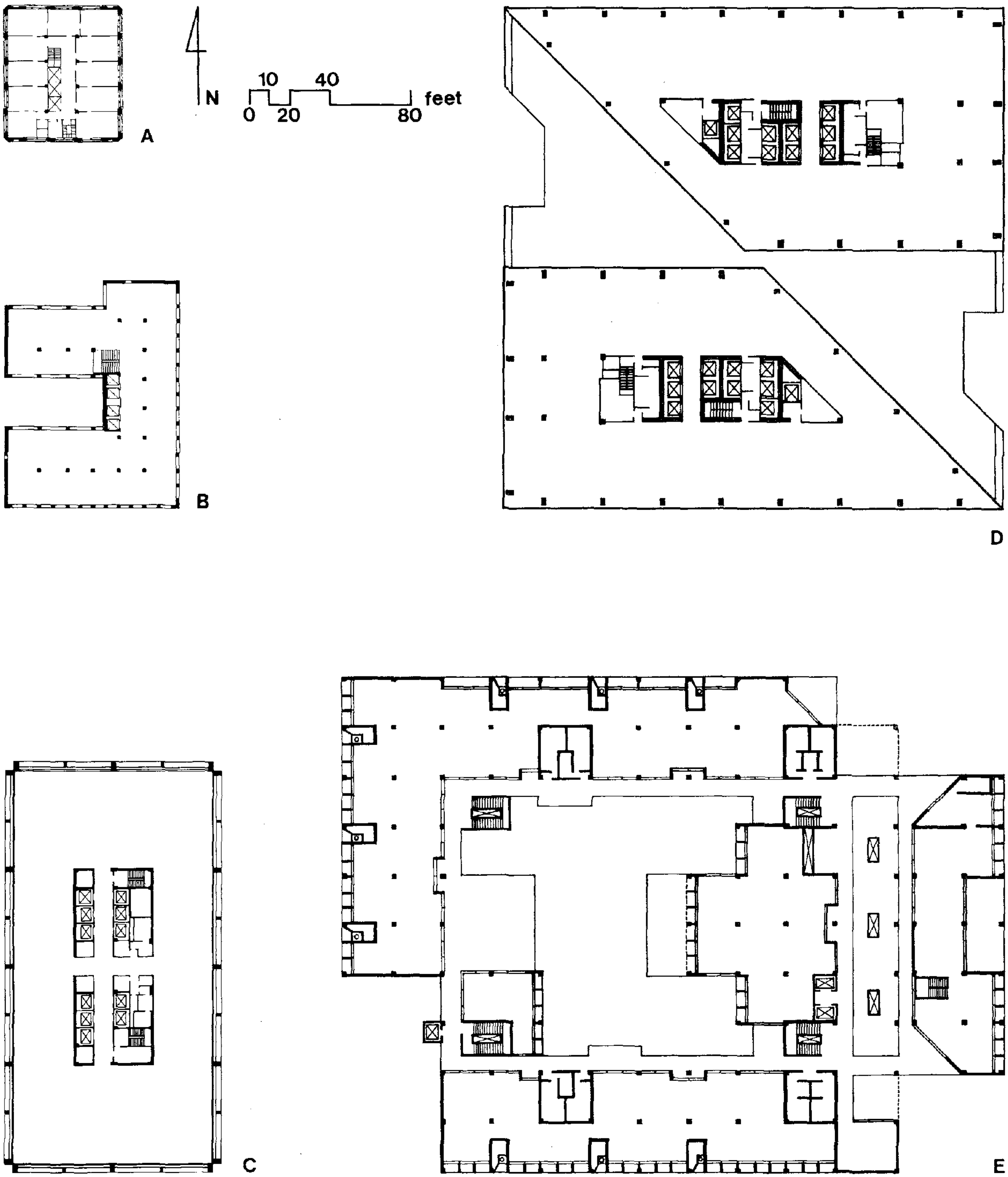


Figure X-14. The four phases of the evolution of office building design. Phase I: small buildings with limited width for light and air; A-Alexander Building, San Francisco, 1920's, B-Guaranty Building, Buffalo, 1895. Phase II: large, cubistic buildings aided by the development of air-conditioning and fluorescent lighting; C-Alcoa Building, San Francisco, 1968. Phase III: large, prismatic buildings emerged as a reaction against the cubistic phase; D-Pennzoil Place, Houston, 1976. Phase IV: a return to humanly-scaled, energy conscious building; E-proposed Sacramento Office Building, 1979.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air moving devices such as fans. Although electric lighting came into general use in the early 20th century, the incandescent systems were inefficient and introduced a great deal of heat. By 1940 the first fluorescent lamps were coming into use, but before that time, daylighting was the major source of daytime lighting.

The result of these few determinants was that buildings were limited in width, and planning was predominantly based on double-loaded corridors. Since the office building was an urban form, the impact of increasing land costs began to be felt very early. The development of the elevator enabled buildings to be built much higher, but the building had to remain narrow.

In order to ensure light and air in small urban sites, buildings consisted of narrow multiple wings or introduced lightwells which, on larger sites, might become courts. This kind of planning predates the modern office building by many centuries: it has been the traditional form of planning for all public and institutional buildings on small sites, and was brought to a high degree of functional and aesthetic refinement in the renaissance buildings of Europe.

In cities like New York, congestion caused the introduction of a secondary configuration element that became characteristic: the setback, which was mandated by building code in order to protect the light and air in streets and between adjoining buildings.

The second phase of configuration is the result of the interactions of four basic influences, one economic, two technological, and one aesthetic. The economic influence was the desire to pack an increasing amount of rentable area into a given site. In trying to do this, the narrow wings of the traditional building limited the area of site coverage, and introduced a number of inside and outside corners that were less useful. Concurrent with this was the owner's demands for larger spaces to accommodate new patterns of office work, and the enormous increase in size of the corporate and public bureaucracies.

The two technological influences were industrial developments that made it possible to build the kind of deep space that economics demanded. One was the development of effective air-conditioning: the other was the development of efficient fluorescent lighting that enabled effective, reasonably economic, illumination to be provided far from the window. A secondary influence was the expansion of the utility company as a business enterprise and its desire to produce and sell energy.

The fourth influence was aesthetic: and it is arguable that this was the most important of all. The rise of the modern movement in architecture, and the influences of the great European masters, Mies Van de Rohe and Le Corbusier, had stressed the aesthetic value of simplicity in facade treatment and simple, pure, cubistic shapes. By the 1940's, many good designers passionately felt that the unadorned rectilinear shape represented the only acceptable style for the contemporary age, and after World War Two, such buildings began to appear on drawing boards all over the world.

In the United States, two seminal designs were brought to fruition by early 1950: the United Nations building and Lever Brothers building, both in New York City (Figure X-15). These represented a new shape for the office building, that was to spread over every city in the world. It is important to realize that in their time they represented a perfect balance of aesthetic intent, technological development, economic need, and provided an easily replicated model.

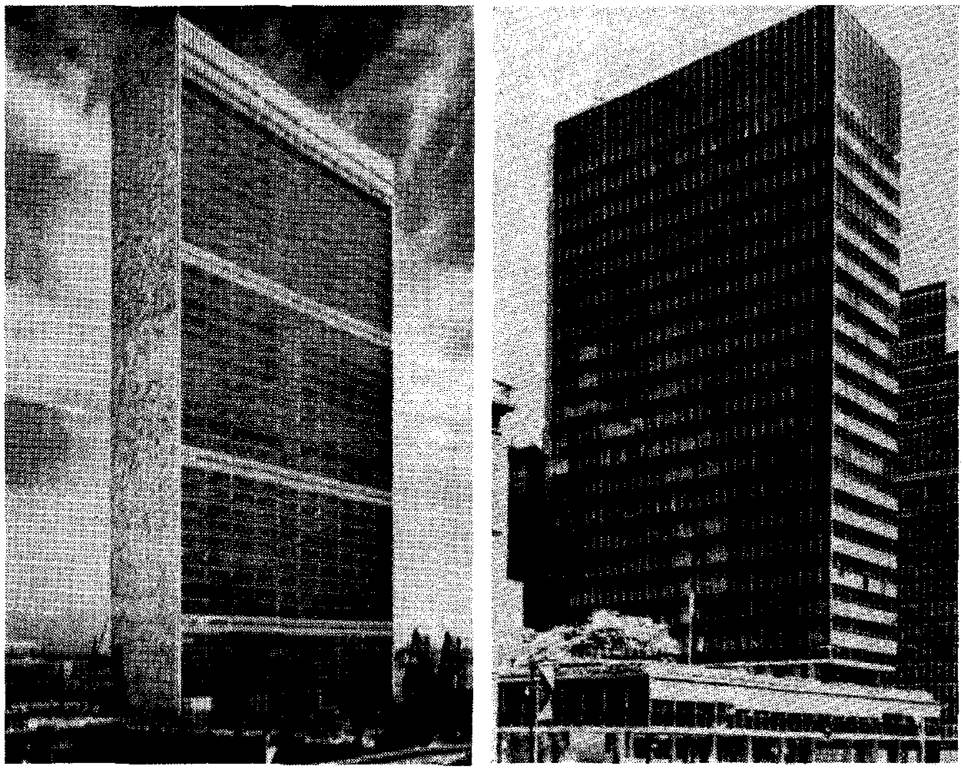

Figure X-15. The U.N. Secretariat and Lever House buildings in New York City.

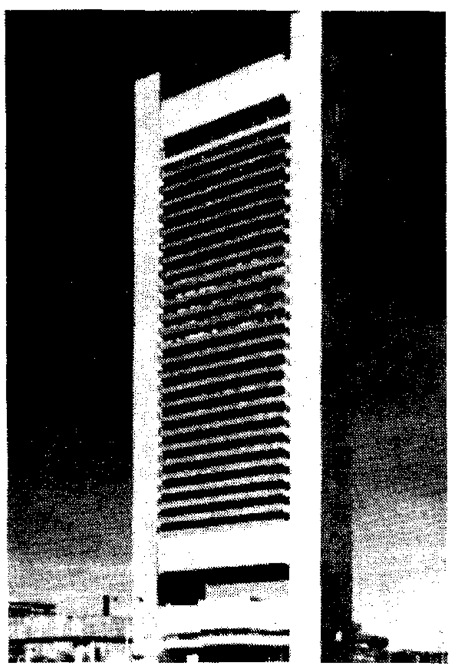

Figure X-16. The unlimited tower.

Two unintentional results of this configuration have proved to be of great importance to seismic design, and their origin is worth tracing. The desire that the building should express the purest kind of cubism meant that if the stepped shapes of the traditional tall block were to be eliminated, the code that related to setbacks must be changed.

The strategy used - successfully - was to persuade code authorities to accept in trade an open plaza area at ground level in exchange for the right to allow the tower to rise, without setbacks, sometimes to heights limited only by the owner's needs and aspirations (Figure X-16).

The open plaza, often created by setting the building on stilts, with only vertical circulation or a glass enclosed lobby at the ground floor, was itself a secondary dogma of the modern movement, much promoted by Le Corbusier. The result of this strategy was the presence, in seismic design terms, of the weak or "soft" first floor, in which either the main structure of the building was not brought down to foundation level, or in other ways, a serious discontinuity of stiffness would be introduced at the second floor level. At the same time, the elimination of setbacks was a positive result, in seismic terms.

The second characteristic also involved a code change; the use of the glass wall only became economic when the codes that required a masonry or concrete spandrel wall (for fire protection) were changed in the late 1940's. This opened the way for the full non-structural curtain wall which, allied with flexible frame structures, raised serious implications for non-structural damage in medium and high-rise buildings.

The third phase of office design, which began around 1965, is primarily an aesthetic reaction to the rectilinear cubism period. In this phase, which is still in effect, the aesthetic desire for pure geometry is still present but dramatic variations from the cube begin to take place. Most notable is the use of non-rectilinear forms, in particular the 45° angle. Forms become prismatic; and this effect is furthered by the development of types of reflective glass that both improve thermal performance and also enable the geometrical purity of the form to be expressed with far greater abstraction than even in the curtain wall buildings of the 1950's and 1960's (Figure X-17).

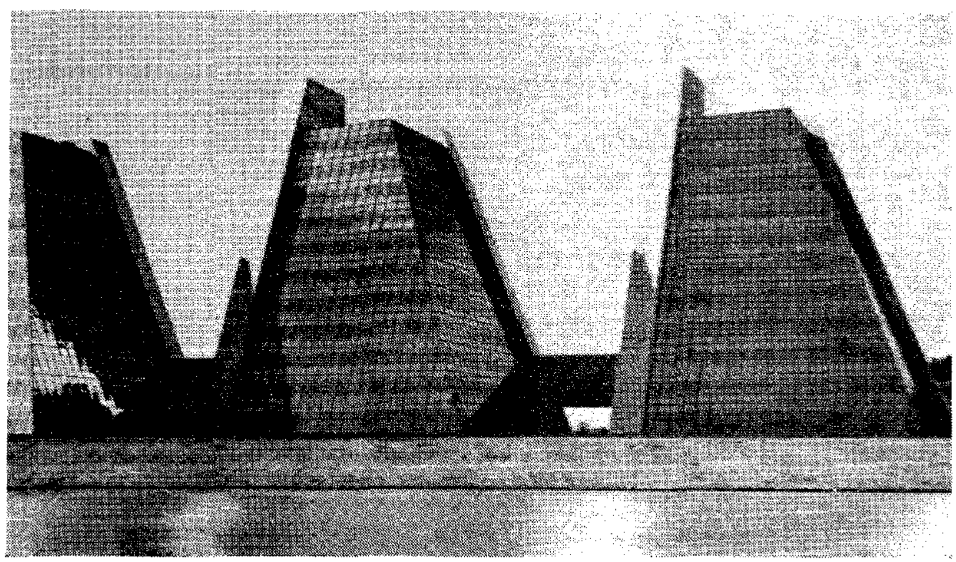

Figure X-17. Prismatic building forms sheathed in reflective glass to emphasize the purity of the form.

In the fourth phase, the building shape is modified by a concern for energy conservation, and the realization that current building shapes are intrinsically wasteful of energy. Nowhere is this more true than in the area of lighting: the fluorescent-lit deep space is intrinsically costly to operate, but in addition, the energy required for lighting adds significantly to the cooling load. At the same time, these technological and economic reservations are accompanied by user irritation with the nature of the office building - more particularly the character of the work space and its use of larger areas of blandly lit and decorated spaces. Some of this irritation is against the bureaucracy itself: and to the extent that the form of the building perfectly expresses the bureaucracy, questions about one inevitably lead to questions about the other.

In phase four, which we are entering, we begin to see a return to the fragmented, smaller scale forms of phase one. We see the narrow building, suitable more for daylighting: we see courtyards, lightwells, and skylights: and we see large building masses begin to break down into small, more humanly scaled units. It begins to look as if the period of monumental geometrical forms is coming to an end. At present, the large prismatic shapes of phase III represent the accepted style for the prestige office, and practitioners of this style are in demand by the large corporations. But there is a chance that the energy conserving, humane, non-monumental environment is on its way.

References

1. Bill Hillier and Adrian Leaman, "A New Approach to Architectural Research," RIBA Journal, December 1972, pp. 517-521.

XI. Seismic Design and Building Type

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A. Introduction

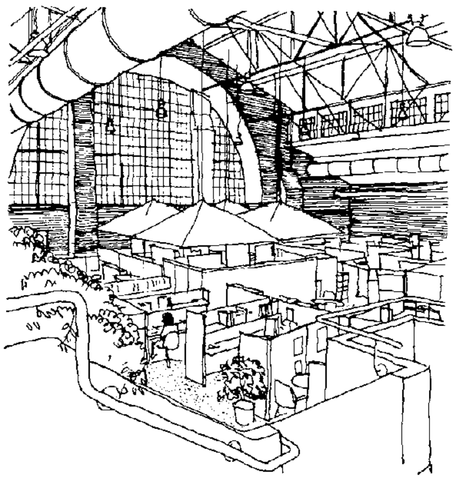

Figure XI-1. Stanford University's Old Pavilion - basketball pavilion to administrative offices.

Up to this point, we have not considered building occupancy type, since it has been more appropriate to identify characteristics that are significant for seismic design, and these characteristics may apply to a number of occupancies. In addition, the design of a building type may change significantly as new organizational patterns develop: for example, the change in school design from an incremental short span box structure to that of an adaptable frame structure, capable of providing a variety of space types at any given time, which was the result of new teaching concepts introduced in the early sixties.

To the extent that building configuration is independent of occupancy requirements, building type is immaterial: the earthquake has no knowledge of, and is not affected by the activities that are taking place in the building. However, from the designer's viewpoint, it is worthwhile in summary form to identify seismic design issues that result from common design solutions to characteristic occupancies. This summary is intended as a schematic presentation of these relationships, and to suggest a fruitful way of relating architectural and seismic issues that is worthy of further development.

In this chapter the emphasis is on characteristic - even simplistic - 'prototype' design solutions: those which are commonly repeated because they represent obvious and economical solutions to the program requirements. While we can identify many 'prototype' design solutions for different occupancies - characteristic configurations for offices, schools, hotels, etc. - we must also recognize that occupancy sometimes runs counter to its prototypical form, and indeed imaginatively conceived buildings often do just this. Much of the evolution of design consists of designing new prototypes. Moreover, the present trend towards building re-use is resulting in surprising adaptations of use to form, such as the successful adaptation of a basketball pavilion to an administrative office (Figure XI-1).

Earlier chapters have built up a long list of preferred configuration approaches in respect of the seismic problem. The designer is not necessarily going to be able to adhere to all these approaches: he must balance many other considerations besides those of seismic risk, and at times the preferred seismic configuration may be in opposition to the preferred programmatic configuration. When this occurs, the designer must perform his most characteristic task: the balancing of alternatives, often based on inadequate information, projecting perhaps half a century into the future, in order to create a single solution that represents a sensible balance of all conflicting requirements.

What follows is a summary, classified under broad headings of building type, that identifies typical architectural characteristics, relates them to the seismic design problem, and outlines alternative solutions. This format serves both as a summary set of guidelines based on the narrative of the study, and also enables the designer to use building type as an entry into the set of guidelines: an early warning system of the major seismic design issues that should be considered as the schematic stage of design begins. Elementary though this warning system may be, it highlights some common and recurring relationships between common building type solutions and seismic design issues.

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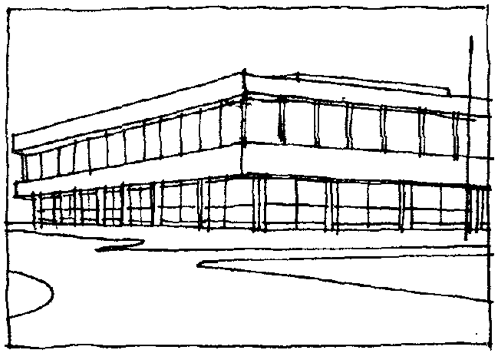
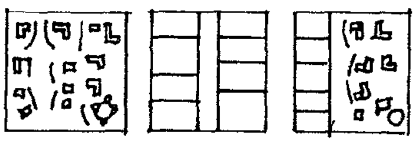
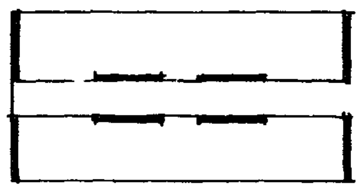
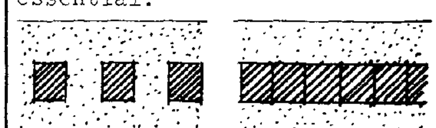
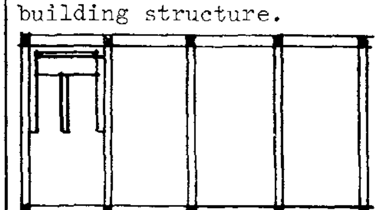
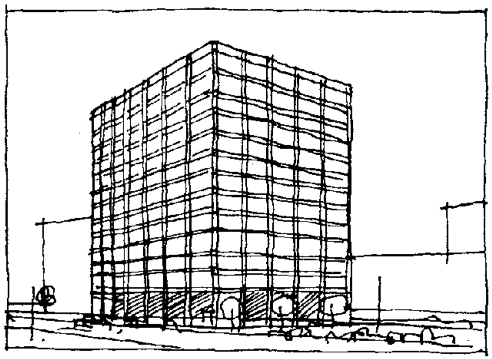
B. Office, low-rise



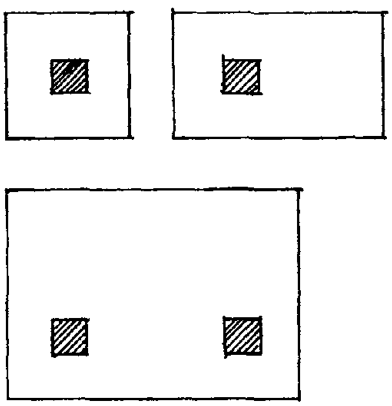
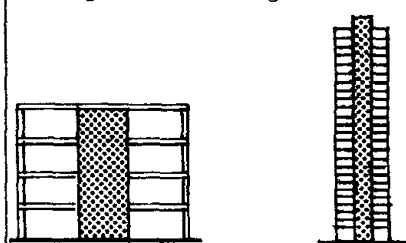
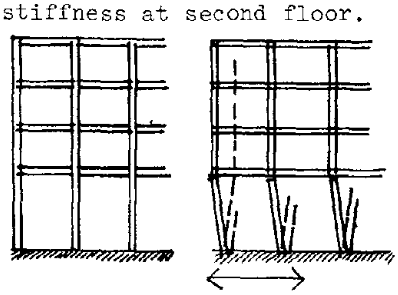
commercial
institutional

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Great variety of configuration alternatives.	Variety of appropriate seismic design approaches.	Shear walls, frames, combinations
May require large unobstructed space for open office layout, or number of small offices, or combination.  Office for single organization will have specific planning requirements, but general adaptability is also required. Offices for rental planned for maximum adaptability.	Interior shear walls may be too obstructive. However, forces in low-rise buildings (particularly steel and wood) are fairly small because of low mass, so extent of bracing and shear walls not great.	Careful location of any shear walls or bracing so as not to limit planning freedom. 
Maximum perimeter exposure for view and daylighting.	Large perimeter shear walls may be difficult to obtain.	Use of perimeter frame. However, use of isolated windows may still enable use of perimeter shear walls, especially in tilt-up concrete: continuous window bands make frame essential. 
Use of staircase access, with limited elevator use for handicapped and deliveries.	Stairs may introduce point of localized stiffness and suffer damage, making them unusable.	Careful design for anticipated forces, or detach stair from building structure. 

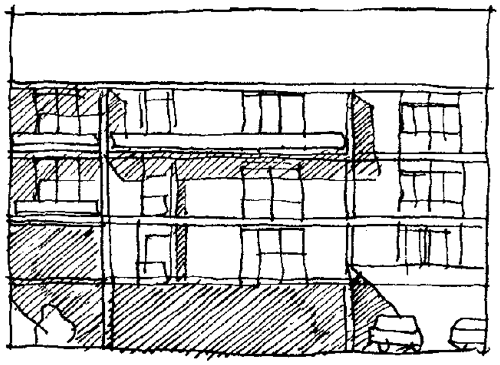
C. Office, medium to high-rise



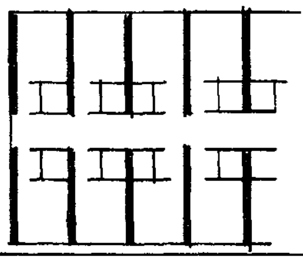
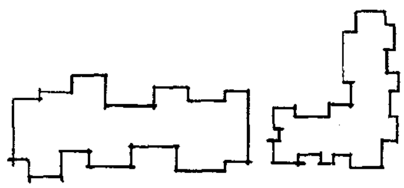
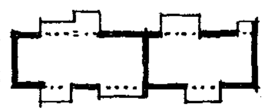
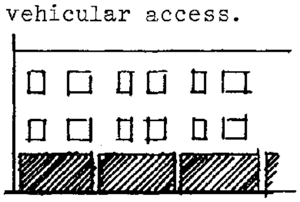
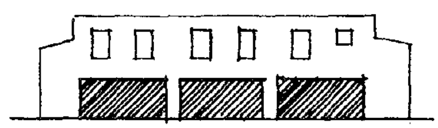
commercial
institutional

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Open perimeter, for daylighting and view for perimeter offices.	Perimeter shear walls may be impossible or limited.	Perimeter frame structure, check for weak column, strong beam.
Vertical circulation and service cores, located to suit building organization and planning requirements. 	Cores may be of suitable size and location to act as shear wall or bracing elements, and provide all or most of the seismic resistance.	Cores must be located symmetrically, and walls wide enough to act as vertical cantilever beams without risk of high overturning forces. 
	Cores may be unsuitable in size (eg. too narrow) or asymmetrically located so that they will produce torsion if designed as stiff element.	Investigate modification of core location or size: if not possible, do not use cores as resistant elements.
High first floor, because of occupancy (lobby, banking floor, commercial).	'Soft story', may be significant discontinuity of framing stiffness at second floor. 	Eliminate discontinuity.
Open first floor, by design choice or mandated by urban design or zoning requirements for open plaza.	O.K. if no framing change. If columns reduced in number or shear walls stopped at upper floor, will result in major discontinuity.	Never stop shear wall above ground floor. Keep other discontinuities to minimum.

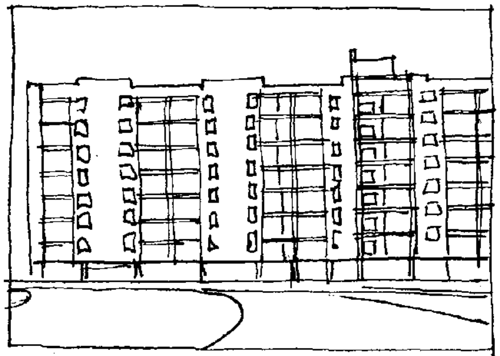
D. Residential, low-rise multi-family



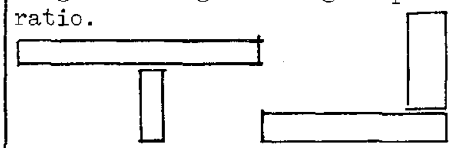
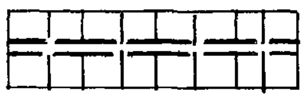
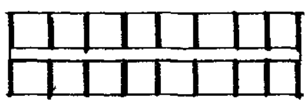
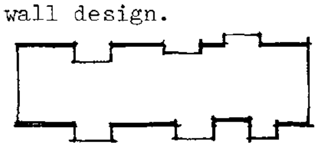
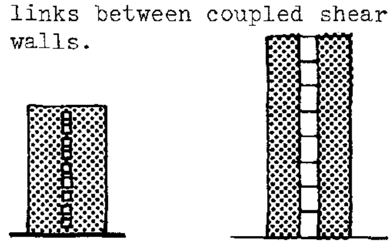
apartment building
motel

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Cellular plan with permanent fire/acoustic/enclosure walls. 	Good opportunities for shear walls.	Care to balance resistance in all directions, and ensure shear walls not weakened by excessive perforation by doors, windows.
Complex configurations for visual appeal and to maximize exposure.	May introduce re-entrant corner problems (if large building), discontinuities of framing or shear walls. 	In large building, separate into simple shapes. In smaller buildings, adjust configuration to provide as many continuous members as possible. 
Open front at grade for vehicular access. 	Soft story. Variations in perimeter strength and stiffness.	External buttress wall or brace.  Internal shear wall near front face, steel rigid frame around open front end. 

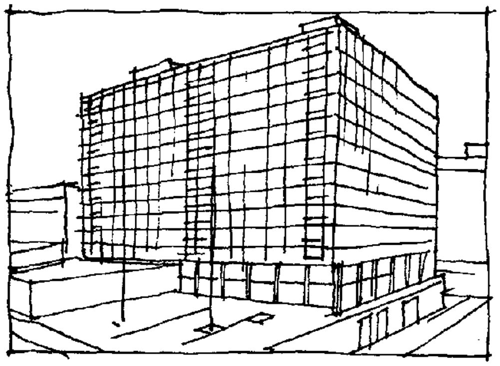
E. Residential, high-rise



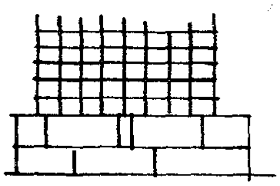
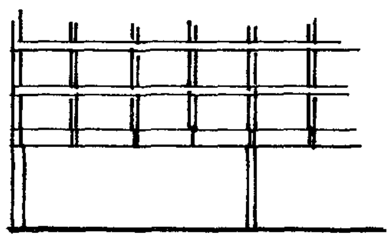
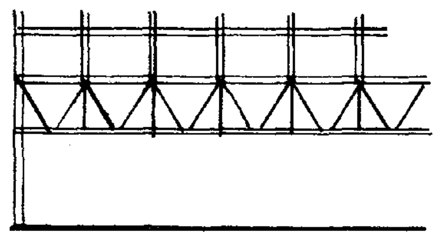
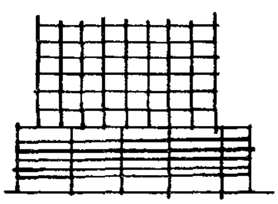
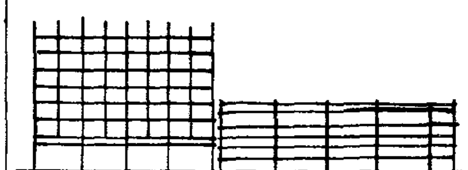
apartment building
hotel

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Often re-entrant corner plan - L or T shape - to ensure high density and perimeter exposure.	Possible stress concentrations at notch, and/or torsion.	Subdivide by seismic joints if wings are long and large aspect ratio. 
Often long narrow wings, with double loaded corridor plan.	Possible use of interior corridor for shear wall. 	Check for openings, and ensure sufficient length for each shear wall.
Good acoustic separation necessary at party walls, and adaptability requirements minimal.	Ideal to use party walls as shear walls. 	Check for location and continuity, particularly vertical. All shear walls must continue to foundation. Use corridor shear wall to balance party wall resistance.
Open perimeter, but window size can be limited. May be balconies and breaks in wall plane.	May be difficult to provide perimeter shear walls. Openings for balconies and wall plane breaks may result in discontinuities and ineffective shear wall design. 	Perimeter shear wall may be possible with careful opening sizing and placement. Check for adequate length and proportion of shear walls.
End walls of narrow building may be solid, or nearly so.	End walls may be useful as shear walls.	Check shear wall proportion for overturning and design of links between coupled shear walls. 

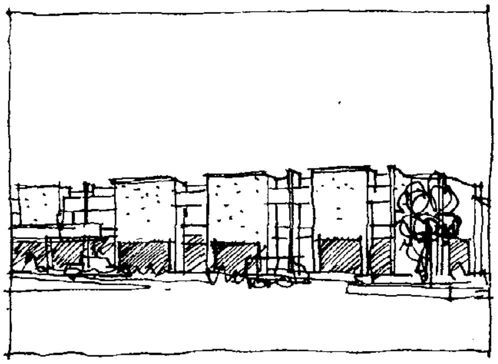
F. Residential, high-rise



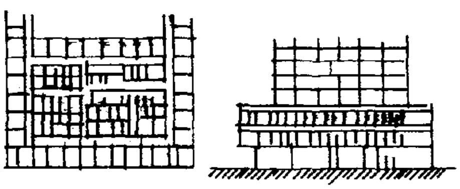
convention hotel

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
see type 4 (high-rise residential) and add following:		
Large rooms (banquet, meeting rooms) at lower level. 	Need for wide spans and change in framing. Vertical framing to accommodate may result in severe vertical framing discontinuities. 	Possible use of large transfer frames or trusses, creating spaces that may accommodate mechanical equipment. 
Possible integral parking structure at lower levels. 	Change in framing (bay size) to accommodate vehicle parking space requirements, resulting in severe vertical discontinuities, including discontinuous shear walls or vertical setback.	Use of transfer frames or trusses. Better if parking structure built alongside lower levels so that hotel framing not impacted. Provide adequate separation. 

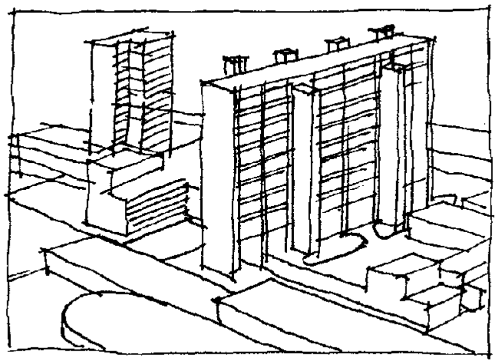
G. Medical Facility, low-rise



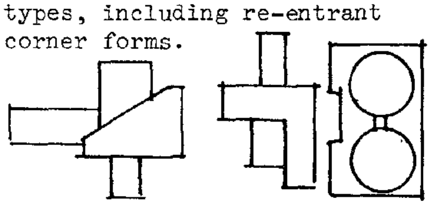
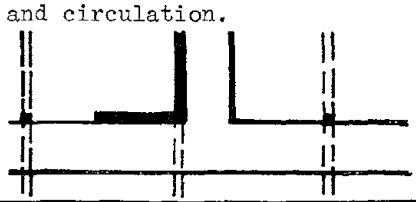
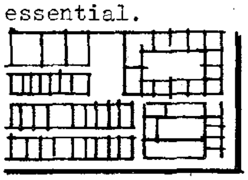
hospital
health clinic

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Great variety of configurations.	Variety of appropriate seismic design approaches.	Shear walls, frames, combinations.
Predominately small rooms. Often complex planning requirements. May be non-repetitive plan from floor to floor. 	Possible difficulty in maintaining uniform framing, shear wall location within each floor and floor to floor. However, forces in low-rise structure are fairly small because of low mass, so extent of bracing and shear walls not great.	Moment resisting frame structure ideal to provide maximum adaptability of planning, but check drift limits. Shear walls must be continuous and manipulate plan to achieve this.
Facility function dependent on equipment and utilities.	Structural design to reduce seismic effect on non-structural components. Design seismic protection for equipment and utilities.	Design building for stiffness and low drift limits. Careful detailing of equipment and utility relationship to building structure. Check for overturning.
Extreme seismic code standards (California) if provides overnight care.	Rigorous plan checking and site inspection by state increases cost and design time.	Serious consideration of seismic issues from design inception essential.

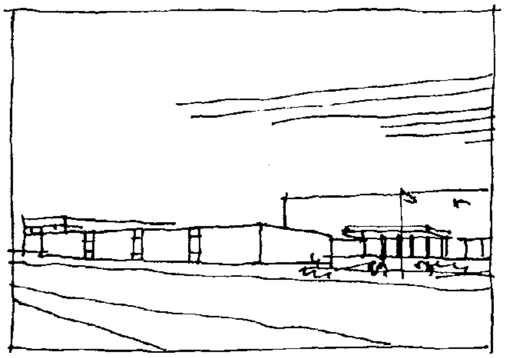
H. Medical Facility, medium to high-rise



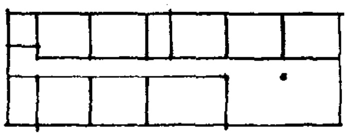
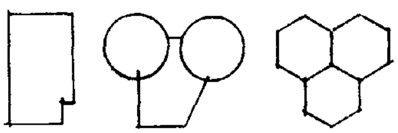
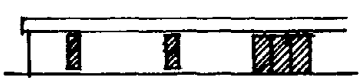
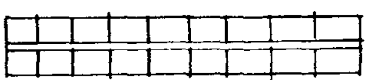
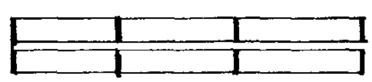
hospital

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Large variety of configuration types, including re-entrant corner forms. 	Possibility of stress concentration, torsion.	Subdivide by seismic joints.
Complex planning requirements: much horizontal and vertical movement of people, materials and equipment.	Limitations on placement of shear wall and bracing; must be related to circulation.	Careful planning relationships between shear walls, bracing and circulation. 
Large elevators result in large vertical circulation cores.	See high-rise offices, but larger cores increase shear wall possibilities.	See high-rise offices.
Large clinical and diagnostic areas need many small rooms, perimeter location not essential. 	Generally large floor area may result in need for interior shear walls, braces.	Care in locating shear walls, braces to permit planning function.
Hospital function very dependent on equipment and utilities.	Structural design to reduce seismic effect on non-structural components.	Design stiff structure to limit drift, best done by shear walls or frames. Interstitial framing may be beneficial in limiting story drift.
	Design seismic protection for equipment and utilities.	Careful detailing of equipment and utility relationship to building structure. Check for overturning.
Extreme seismic code standards (California).	Rigorous plan checking and site inspection by state increases cost and design time.	Serious consideration of seismic issues from design inception essential.

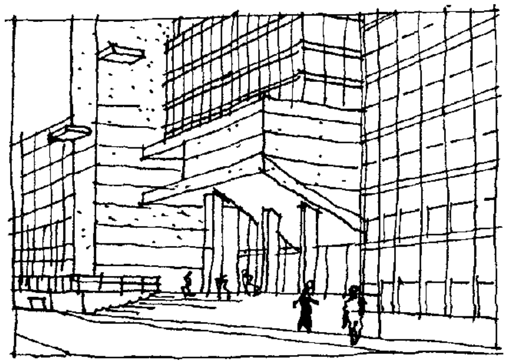
I. Educational, low-rise



school
community college
small college
university

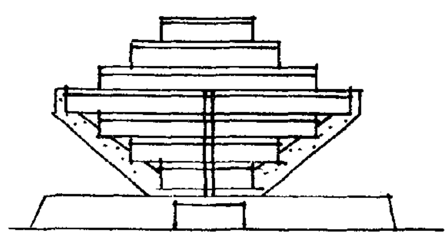
Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Mix of space sizes, with classroom size (30 x 30±) predominating. 	Fairly wide spans, but since forces are small (low mass), often possible to locate all shear walls or braces at perimeter. In large structures, interior shear walls or frames may be necessary.	Moment frame structure ideal for large structures, but shear walls and braced frames may generally be located at fixed areas, such as toilets, with little planning or flexibility impact.
Desire for adaptability.	Framing must accommodate planning variations, or use of a perimeter seismic solution to keep interior open for changes.	Check for framing continuity. Locate braced frame or shear wall on perimeter; make sure they are continuous.
Great variety of overall configurations: lofts, pods, clusters, etc. 	Variety of appropriate seismic design approaches.	Shear walls, frames, combinations.
Non-uniform perimeter, generally low need for openings. 	Provides many alternatives for perimeter structure with few limitations.	Ideal for perimeter shear wall or braced frames, either as entire seismic resistance (small buildings) or in combination with interior elements.
Buildings (particularly older) may have long elevations, large aspect ratio. 	Building stiff in transverse direction, but flexible in longitudinal direction, leading to possible quasi-resonance. Diaphragm spans a great distance.	Balance rigidities: decrease building length, subdivide building with seismic joints, etc. Add interior cross walls to shorten diaphragm span. 

J. Educational, high-rise

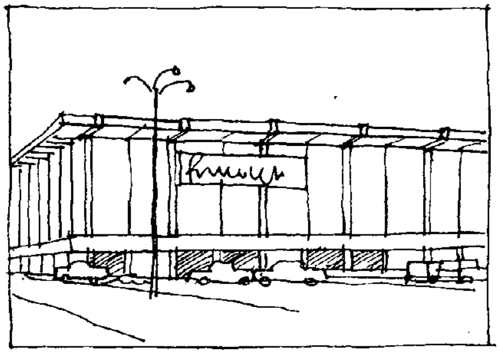


city school
large college
large university

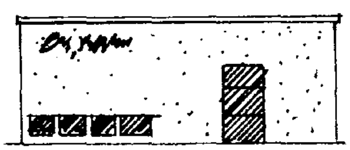
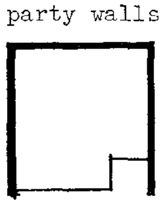
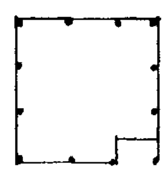
Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Mix of space sizes, with classroom size (30 x 30$\pm$) predominating.	Fairly wide spans.	Interior shear walls or frames may be necessary, locate at toilets, cores, etc.
Generally, variation in floor to floor layout.	Possible difficulty in maintaining uniform framing, or continuous shear wall from floor to floor.	Check for framing continuity. Shear walls must be continuous: manipulate plan to achieve this.
Non-uniform perimeter, generally low need for openings.	Provides many alternatives for perimeter structure with few limitations.	Ideal for perimeter shear walls or bracing.
Often large complex building with intricate planning that reflects organization's structure.	Results in complex configurations, and consequent framing complications, with discontinuity.	Manipulate planning and configurations to reduce discontinuities.
University buildings sometimes place extreme emphasis on image.	May result in unusual configurations.	Check for discontinuities, torsion, etc.



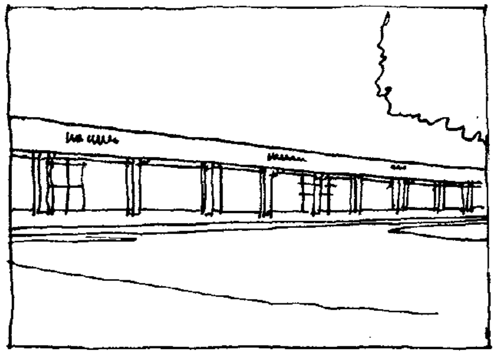
K. Commercial, low-rise



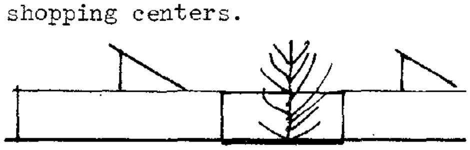
store
department store
market

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Large, minimally obstructed interior space.	Fairly wide spans limit use of interior shear walls.	Use of frame: if interior shear walls necessary, locate around fixed elements such as stairs, escalators, toilets, etc.
Vertical circulation often by moving stair, elevators. Periodic dense occupancy.	Extreme importance for safety to preserve structural integrity of stairs. Stair may be point of localized stiffness, creating torsion, and be particularly vulnerable to damage.	Careful relationship of stair design to overall structure. Structural separation of stair from main structure may be best.
Elevator core area generally small relative to building area.	Less possibility of torsion. Core less likely to be useful as shear element.	Perimeter shear walls, or use frame system.
Perimeter generally solid, occasionally non-uniform openings for display. 	Possible economical use of perimeter shear wall box.	Ensure non-uniform openings do not result in localized stress concentration or weakness.
Smaller stores on street often have open front, closed rear or party walls. 	Major variation in perimeter strength and stiffness, possible torsion.	Balance perimeter strength and stiffness. 
Tend to be low in cost.	Tend to design to code minimum.	Simple building to reduce seismic design costs.

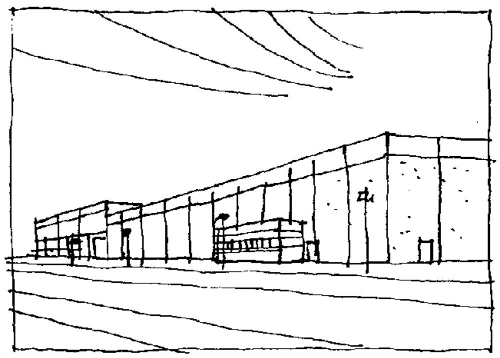
L. Commercial, single story



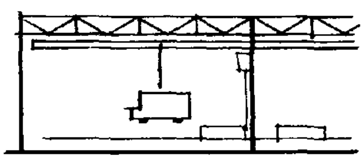
warehouse
shopping center

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Large, minimally obstructed interior space.	Wide spans, roof loads only, often low mass.	Use of frames with perimeter shear walls, or bracing.
Variation in occupancy: low occupancy in warehouses, high occupancy in shopping centers.	Increased seismic risk in shopping centers.	Variation not recognized in code; designer should use judgement.
Often large plan area.	Build-up of large diaphragm forces.	Check diaphragm forces: in very large structures, sub-divide by seismic joints.
Predominately closed perimeter, with possible large openings for display or access.	Perimeter may be suitable for shear wall, provided building not too large in plan. If large openings, may result in perimeter strength and stiffness, leading to torsion and/or distortion.	If perimeter shear wall, check forces at openings. If very large building, or expansion anticipated, use frames. Balance perimeter strength and stiffness.
Use of skylights, atria, roof monitors. Currently rare in warehouses, use increasing in shopping centers. 	May result in serious deni-gration of diaphragm capacity and create local stress concen-trations.	Careful check of location and size of diaphragm openings. Provide adequate collectors, and if diaphragm ineffective, substitute horizontal bracing system.
Tend to be low in cost.	Tend to design to code minimum.	Simple building to reduce seismic design costs.

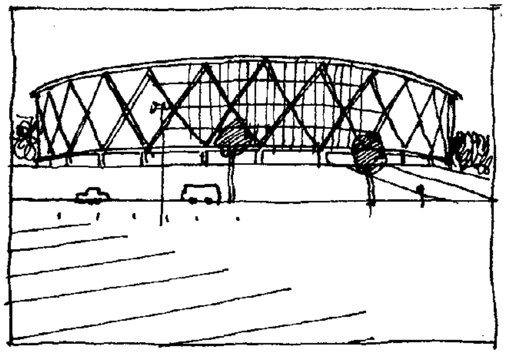
M. Industrial, single story



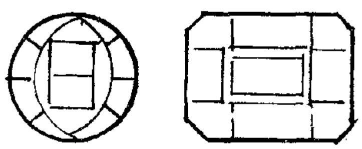
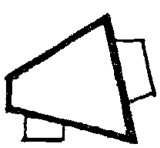
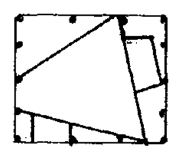
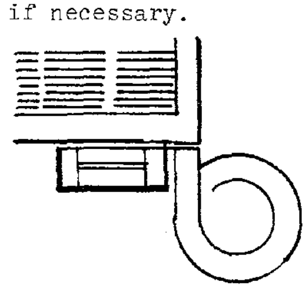
production
assembly

Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Generally large, minimally obstructed interior space. May have high ceiling height, and mezzanine areas.	Wide spans, often low mass.	Use of frames with perimeter shear walls or bracing. Building use may require moment frame to avoid obstruction by interior shear walls or bracing.
May be very large (auto assembly) in plan.	May build-up large diaphragm forces.	Check diaphragm forces: in very large structures, subdivide by seismic joints.
May have large roof loads, or roof-hung moving loads. 	May seriously affect dynamic characteristics of vertical structure if resulting loads are large in proportion to building size.	Careful check on effects of different load conditions.
Closed perimeter, generally uniform, except for loading-bay areas.	Perimeter may be suitable for shear wall, provided building not too large in plan. If large openings, may result in variation in perimeter strength and stiffness, leading to torsion and/or distortion.	If perimeter shear wall, check forces at openings. If very large building, or expansion anticipated, use frames. Balance perimeter strength and stiffness.
Roof generally unbroken except for relatively small ventilators, smoke vents, etc.	Makes for good diaphragm.	

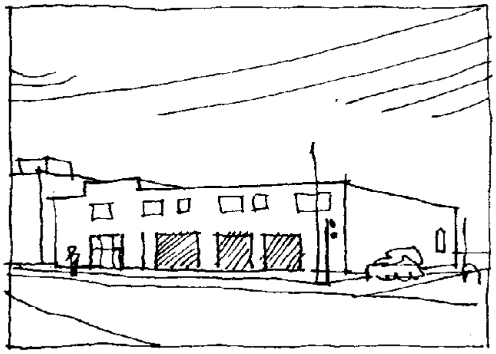
N. Long-Span Roof

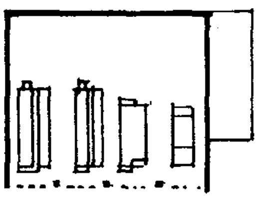
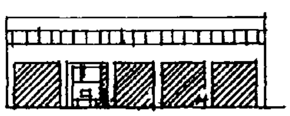


auditorium
stadium
large theater

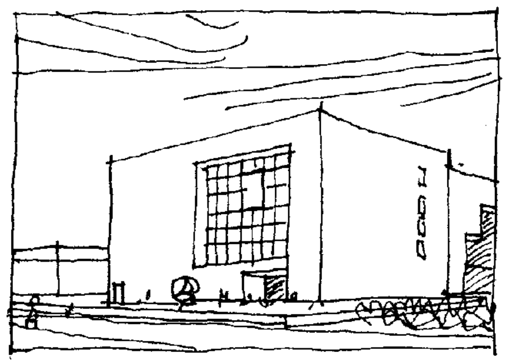
Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Large unobstructed space, high ceiling.	Very large roof diaphragm forces to be transferred to perimeter.	Careful design of diaphragm and diaphragm to perimeter connection.
Some uses (general purposes, sports) require simple configuration. 	Ideal from seismic design viewpoint; both safety and economy of design and construction.	
Some uses (theaters) or locations often require asymmetrical forms. 	May result in configurations liable to torsion or stress concentrations.	Design asymmetrical space within symmetric structural configuration. 
Perimeters generally closed (for thermal and lighting reasons), but good public access needed.	Good for shear walls or bracing, but need for access may interfere with structural integrity.	Check for openings and discontinuities in shear walls. Manipulate planning and configuration to reduce discontinuities.
Large stadia have extensive staircase and ramp access.	Important for safety to preserve integrity of stairs.	Check for relationship to main structure, and detach if necessary. 

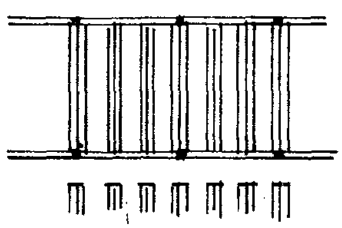
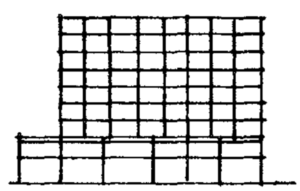
O. Fire Station, Vehicle Maintenance



Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Open elevation (doors for vehicle access) often combined with blank walls on other sides. 	Variation in perimeter strength and stiffness, with weak walls with large openings. Increased possibility of torsion and distortion even if collapse is avoided. Even small distortion may result in unusable facility during critical period.	Balance perimeter strength and stiffness. Design strong framed opening to minimize distortion and allow minimal permanent deformation.
Urban fire stations often provide residential and office space on second floor, above appliance floor. 	May result in addition of mass over soft first floor (wide span with large openings produces weak wall). Mass in wrong place increases possibility of torsion.	Replan to eliminate condition, or if not feasible, keep super-imposed mass as light as possible, and check design carefully.

P. Library, medium to high-rise with integral book stacks



Typical Architectural Characteristics	Seismic Design Implications	Seismic Design Solutions
Great variety of configuration alternatives.	Variety of appropriate seismic design approaches.	Shear walls, frames, combinations.
Great variety of spaces. General need for large open areas (reading) and stack areas with low ceiling.	Moderate spans. Sometimes wide discrepancies in room height (stack, reading, lobbies, galleries) may lead to framing discontinuities.	Use of frames. Careful check for framing continuity, and manipulate plan if necessary.
Structural framing module often set by stack requirements. 	Repetitive framing beneficial, provided does not conflict with framing at other levels resulting in discontinuity. 	Careful check to eliminate framing discontinuity.
Very high mass in stack areas. Since book acquisition is continual, location of mass will slowly change. Though design may show symmetrical stack location, <u>loading</u> of stacks may be asymmetrical.	Asymmetrical mass, particularly if large and on upper floors, may seriously affect dynamic characteristics of building: and produce large torsional forces.	Careful analysis and simulation of dynamic conditions over building life.
Open stack layout.	Restricts use of interior shear walls or bracing.	Moment resistant frame, or sufficient bracing around perimeter.
Perimeter requirements: open in reading areas, closed in stack areas.	May result in variations in perimeter strength and stiffness, leading to stress concentrations and torsion.	Careful check to balance perimeter resistance system.

XII. Seismic Issues in the Design Process

A. Introduction

Just as structural design is an integral element of the overall building design process, seismic design is an integral element of structural design, and should not be regarded as a separate process. Models of the design process always tend to suggest a sequential set of activities that does not truly represent the way in which design is done. In theory, we need a formal process in which the interactions of all the elements can be considered concurrently. This does not lend itself to an intelligible model, but in fact the designer approximates this condition by very rapid recycling of information and concepts, and by frequent shifts of emphasis and solution.

Because the designer habitually works with information which is uncertain and often ambiguous, it is possible for him to manipulate and modify information very quickly. In this the designer's method varies greatly from the kind of linear process exemplified by a computer program in which each piece of information is precisely defined and unambiguous.

The concern here is with real world building design processes. For this reason, no attempt is made here to present a sophisticated model. Instead, a set of issues relating to seismic design is outlined for each of the conventional phases of building design, that are appropriate for review between the architect and engineer. This set, in turn, is related to a broad list of the structural decisions that would normally be expected for each phase.

B. Program

Architect should review program requirements and his initial design ideas briefly with engineer to ensure that no needless conflicts will later arise because of programmatic or design assumptions or constraints.

issues for review

building size
- gross area
- floor area
- probable number of floors

site characteristics
- geology
- zoning restrictions:
  - plan area
  - height limit
- orientation
- foundation characteristics

interior planning requirements
- types of spaces:
  - large
  - small
- circulation requirements:
  - vertical
  - horizontal
- special planning requirements

fire standards
- code construction type options

budget
- general level of quality

structural decisions

seismic code
- determination of applicable code

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C. Schematic Design

Architect should review with engineer before beginning schematic design, or very early in the process, as soon as possible configurations begin to appear. Complex plans or significant configuration issues should be brought to engineer's attention at earliest possible point so that the degree of their effect can be assessed.

issues for review

configuration
. shape
. size
. number of floors
. significant configuration problems
. floor to floor heights; variations

vertical circulation
. stairs
. elevators
. cores:
size
location

h.v.a.c.
. general type
. distribution pattern
. required space for ducts

materials
. code requirements
. cladding

structural decisions

general structural strategies
. horizontal framing
. vertical framing
. lateral systems:
moment resisting frames
shear walls
braced frames
. perimeter requirements
. special aesthetic requirements

During this phase, the architect and structural engineer should confer frequently, particularly on the issues of general structural strategies, vertical circulation, configuration, and their interrelationships.

D. Preliminary Design/ Design Development

This is perhaps the typical phase at which major engineering input is obtained, and yet the opportunity to avoid major seismic problems is largely present only in the earlier stages. The detailed design gives the engineer the necessary data to pursue an analysis, and on the basis of this, the design is revised to take into account member sizes or even more basic aspects of the structure. Detailed analysis may be required at earlier stages for complicated or unusual designs if the usual approximations cannot be applied.

issues for review

architectural systems
. exterior cladding

preliminary details
. interior partitions
. ceilings
. depressions in floor slabs
. vertical transportation

	mechanical, electrical, plumbing systems design	. h.v.a.c. and utilities distribution systems . prelim. duct sizes and locations . openings in floors, walls, beams, girders . equipment locations: roof floors basement . vertical shafts . lighting
structural decisions	structural system design	. bay size . horizontal framing: materials foundation requirements . vertical/lateral framing . shear wall/braced frame locations
	preliminary structural analysis	. preliminary member sizing . preliminary seismic details

E. Contract Documents

While it is obvious that the architect will want to know the engineering decisions - exact column dimensions, separation joint detailing, etc. - the engineer also needs to know what the architect is doing. If it has been assumed that partitions will carry no lateral load and will not be deformed by frame deflections, the architectural detailing must allow for this. The engineer should review the seismic aspects of non-structural features designed by others - anchorage of ceilings and shelves, heavy equipment location, exterior cladding, etc.

issues for review	architectural systems: final details	. interior partitions . exterior cladding . ceilings . vertical shafts . stairways . floor slab depressions
	mechanical, electrical, plumbing systems: final details	. responsibility for seismic safety . duct size and locations . piping size and locations . size, weight, location of all major equipment . all required penetrations of floors, roofs, walls, shafts, beams . lighting systems
structural decisions	structural design: final details	. member sizes, locations . connection details
	final structural analysis	. review of shop drawings

F. Construction

The standard practice of requiring the design engineer to also supervise construction to ensure conformance with his drawings and specs needs no change, but it is valid to have the engineer and architect review other relevant features as well, such as ceiling attachments, exterior cladding, partition supports, and seismic joint details.

issues for review

- conformance to drawings and specs
- construction quality, workmanship

G. Operation

Not normally included in the usual professional services contract, consultation to make the building more efficient after construction is now becoming more common. From the seismic viewpoint, any changes resulting in alteration of original framing, including the addition of stiff non-structural walls, or new holes in existing walls, may be significant. Moving, or addition of heavy equipment, particularly in upper floors or non-symmetric location, may also be significant and the engineer should be consulted.

XIII. Case Studies of Seismic Design

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A. Introduction

This chapter presents a description of the seismic design process, with particular reference to configuration issues, for two buildings with particularly stringent seismic requirements.

The 500 bed Veterans Administration Hospital in Loma Linda, California opened in September 1977. Since it was designed as a replacement facility for the V.A. Hospital lost in the 1971 San Fernando earthquake, and since demographic considerations necessitated that it be in an area of extremely high seismicity, the building represents an interesting case study in careful seismic design.

The project was remarkable in the extent to which the configuration of a large and complex building was influenced by seismic design concerns. At the same time, the project provides a lesson in showing that early recognition of seismic design determinants by the whole design team, and a serious interdisciplinary approach from the inception of design, can enable requirements both of seismic design and hospital planning and economy to be achieved with equal effectiveness.

The Imperial Hotel, Tokyo, Japan, designed by Frank Lloyd Wright in the early 1920's, presents a startling contrast. For this large building, standing on very poor ground in a known highly seismic area, the architect assumed full responsibility for an innovative seismic design concept which was diametrically opposed to the conventional wisdom of the day. A myth of infallibility has grown up around this design, which was tested in the great Tokyo earthquake of 1923. The facts are a little different from the myth, and the most innovative features of the design, widely publicized by the architect, were probably the least effective. But the building obeyed many of the configuration rules that have been discussed in these pages, and the building's good performance can be attributed primarily to that fortunate situation.

B. Veterans Administration Hospital, Loma Linda, California

The San Bernardino Valley is seismically very active and the final site selected for the new hospital had 11 known active faults within a 65 mile radius, including the San Jacinto fault and two segments of the San Andreas fault.

The potentially active Loma Linda fault was believed to be located in close proximity to the site, and after intensive studies it was concluded that the most likely location of the fault was 200 to 400 feet south-west of the site, that surface rupture in the site was not likely, and that soil amplification was not of significance.

The consultants recommended that design earthquakes should be considered of magnitude 8+ and duration of 35-40 seconds on the San Andreas fault, and magnitude 6.5-7.25 and duration 20 seconds on the San Jacinto fault, at distances from the site of 7 and 1.25 miles respectively. The building should be designed for a peak acceleration of 0.5g. Essential or potentially damaging non-structural components should be designed for an acceleration of 2.0g. Response spectra were calculated at the surface for structural damping of 5% and 10% of critical, which showed that the peaks of the response spectra occurred at 0.3 sec. for the San Jacinto fault and 0.8 sec. for the San Andreas fault.

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For the low horizontal configuration considered likely for Loma Linda, the San Jacinto spectrum was used because its peak approximately coincided with the probable building period. It was not considered reasonable to assume that the building period could lengthen to the peak of the San Andreas spectrum.

Some of the design force determinants which were evaluated are dependent on a building configuration concept. Prime considerations of building configuration were as follows:

1. Site geometry. The large 40 acre site enabled the consideration of a free standing building unconstrained by site geometry. The site area was sufficient to allow consideration of a relatively low, horizontally planned building.

2. Programmatic. Research studies on hospital organization and planning, performed by the architects prior to the Loma Linda project, had established some general benefits of horizontal planning-defined as plans in which clinical and diagnostic areas are placed on the same floor as nursing areas rather than being concentrated into a base structure with a vertical connection to the bed relating functions.

The advantages related generally to the transportation issue, which was being separately studied by the architects during the schematic design phase of Loma Linda hospital under a separate contract from the V.A. Experience in vertically planned V.A. hospitals had indicated some problems in ensuring adequate circulation, since the concentration of vertical circulation into a single tower tended to result in over or under-capacity depending on the time of the day. There were also indications of a general preference by staff for horizontal movement over vertical, and an indication that a reduction of vertical circulation for severely ill patients - pre and post operational, for example - would also be desirable.

3. Aesthetic. The design of hospitals tends to be dominated by the solution of very complex planning, service and equipment problems, and appearance tends to be a secondary concern. The city of Loma Linda was anxious that the setting of the hospital should be 'park like': in response to this desire, and to the generally small scale of the immediate site surroundings, the image of a low, non-assertive building, placed towards the center of the site, seemed appropriate. The building, because of its nearly 700,000 gross square feet would be very large, but its relatively low height and the large size of the site would help to reduce the community impact of the building (Figure XIII-1).

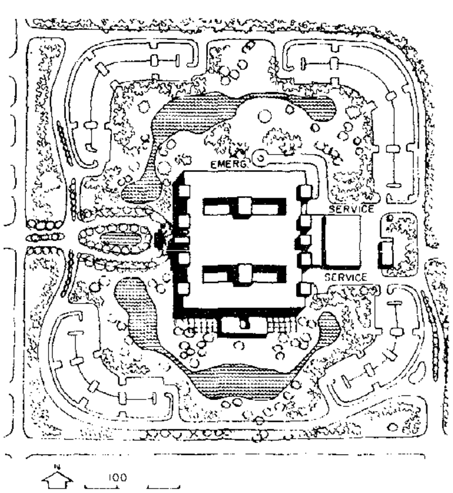

Figure XIII-1. The large, park-like, site plan of Loma Linda Veterans Administration Hospital.

4. Building System. Loma Linda Hospital was intended as a demonstration of the Veterans Administration Hospital Building System, which had been developed over a period of several years by the same consultant team responsible for the hospital design. The building system consisted of a carefully conceived set of design concepts intended to rationalize and organize the preliminary hospital design.

Structural aspects of the system were: a moderate span, simple post and beam, shallow floor framing system; large floor-to-floor heights; and lateral force resistance elements concentrated in the service tower at the end of each service module, a number of which form each floor of the building. The possibility of the use of the system in the extreme seismic conditions encountered at Loma Linda had not been anticipated, but this approach enabled the severe requirements to be successfully accommodated.

Planning and aesthetic requirements pointed towards a low, deep plan building, which coincided well with a low, stiff seismic design that would minimize story drift and consequent architectural, mechanical, electrical and contents damage, and loss of operational capability. In addition, the low stiff building would have a shorter period and a possibly lower response from the projected response spectra peaks of 0.3 sec. and 0.8 sec. from the two nearby faults. The only way of moving the building response well away from the ground response, would be to develop a flexible high rise structure that would be undesirable from all the other view points considered.

The above requirements were specifically defined by the structural engineers as a preferred design of not more than four stories, symmetrical on two plan axes and in section. Any complex configuration should be subdivided so that, if possible, each component met the above requirements. Accordingly, the architects studied in some detail a number of schemes using multiple and single buildings of 3, 4, and 5 stories, using full basement, half basement, and no basement (Figure XIII-2). Symmetry, shear wall availability, separation joint requirements, and continuity of vertical stiffnesses were considered in evaluating seismic resistance.

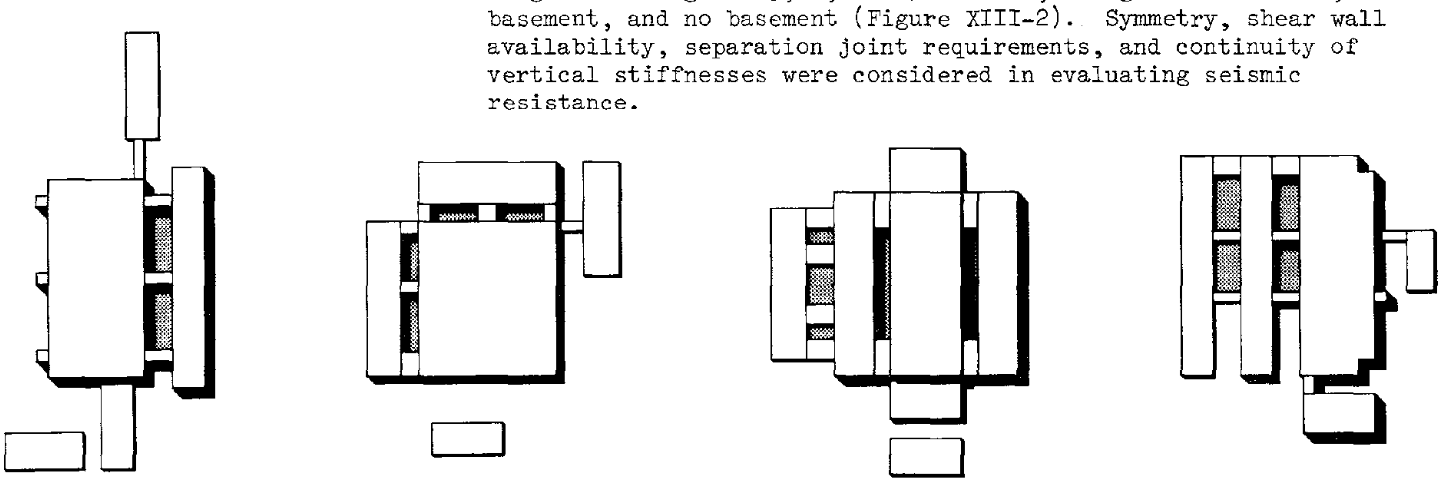

Figure XIII-2. Preliminary alternative schematic designs studied by the architects.

All solutions which used basements produced vertical stiffness discontinuity at the first floor level. Multi-building solutions required many connecting bridges to preserve reasonable circulation, which in turn would require many seismic joints.

The chosen configuration was the simplest of all those studied: it took the form of a simple block, almost square in plan with no basement, and a symmetrical pattern of four courtyards within the block. The courtyards were relatively small. The plan has an even distribution of shear walls throughout, which run uninterrupted from roof to foundation and have direct continuity in plan with the framing members (Figure XIII-3 thru 5).

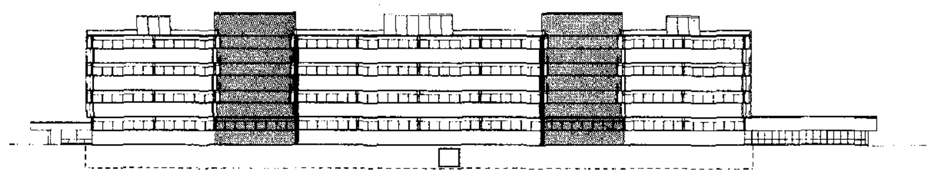

Figure XIII-3. Section through courtyards, showing shear walls at end.

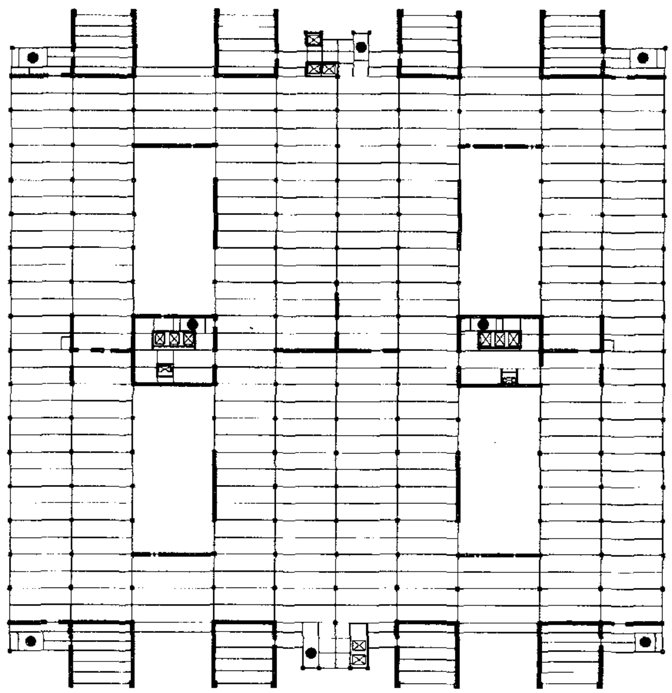

Figure XIII-4. Typical structural framing plan.

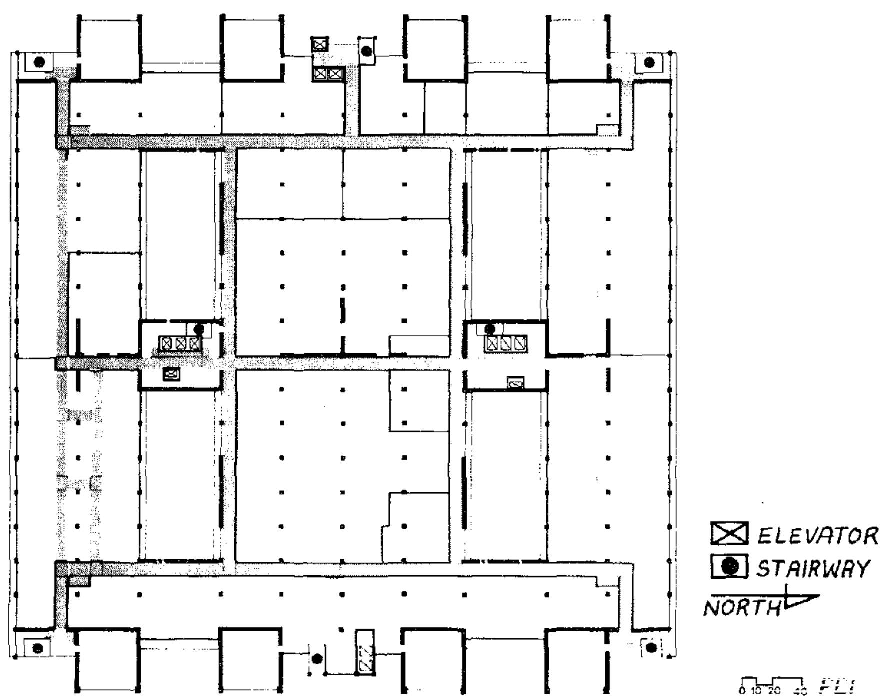

Figure XIII-5. Third floor plan, showing circulation pattern.

Figure XIII-6. Entrance elevation.

The planning and circulation of the building were carefully related to shear wall layout to achieve minimum shear wall penetration with clearly defined, highly accessible public and departmental planning. The final result is remarkably uncompromised on both courts. The eight service towers (four at each end) provide a location for major shear walls (Figure XIII-6). Each tower provides two shear walls in the east-west direction, and one in the north-south direction. The latter wall is an interior wall and is penetrated by large ducts and other horizontal services. However, these openings are repetitive and carefully controlled, and the use of a interior wall enables these shear walls to remain continuous with the perimeter framing of the building which would not be the case if the end walls of the tower had been used (Figure XIII-7).

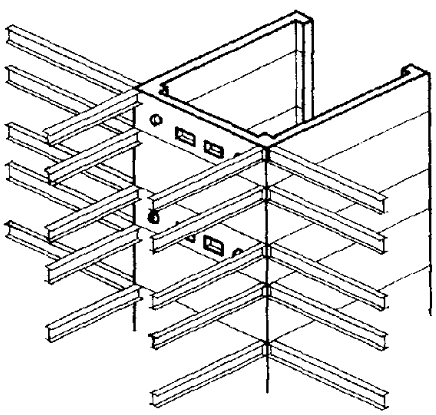

Figure XIII-7. Careful penetration of shear walls with openings for corridors and service ducts.

The general lateral resisting system uses concrete shear walls and a ductile moment resisting "back-up" frame. The stiff primary shear wall system is designed for a high force level so that the structure will tend to have low lateral deflections for the design earthquakes described above. The calculated maximum story-to-story lateral deflection was approximately 0.004H, well within presently accepted desirable ranges for hospitals.

The chances of the back-up frame being forced to work to its full capacity were considered small, but in view of the size and importance of the facility, and the uncertainties of estimating the nature of ground motion, the possibility could not be ignored.

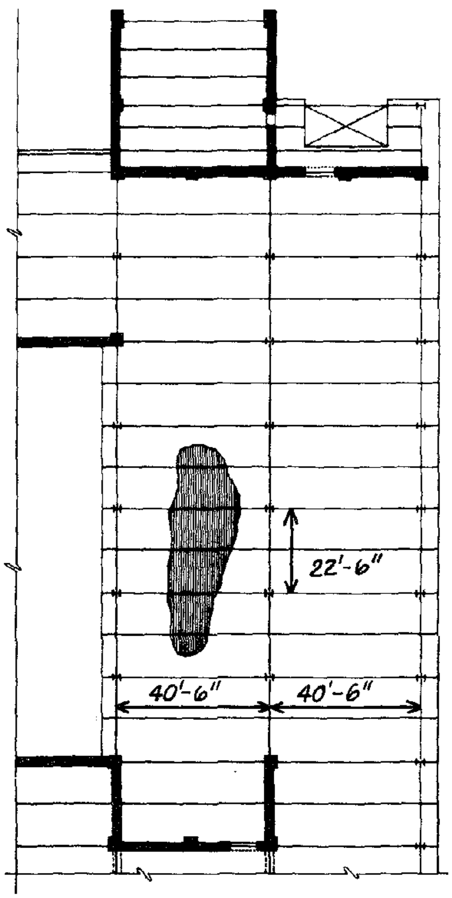

Figure XIII-8. Detail of typical structural framing.

A detail of the basic structural framing is shown in Figure XIII-8. Three steel girders, placed at 40'-6" o.c., run longitudinally spanning the 22'-6" column spacing. Steel beams, composite with the steel deck and concrete floor, span transversely at 11'-3" o.c. The 22'-6" X 40'-6" column spacing is consistent throughout, except for 54'-0" spans at the inside courts.

Sixteen bays, each 22'-6" X 40'-6", and a service tower make up a service module. There are a total of eight service modules per level.

Shear walls are always placed at the perimeter of service modules to minimize planning interferences. Interior girders are dropped below the beams to minimize interference with plumbing service running across the girders, between the beams, and to allow beam continuity across the module. As a result of the service organization, all beams and girders are free of penetration. These framing characteristics are all features of the systems design that were worked out in the earlier research study.

The shear walls were all designed as "infill" walls that simply enclose portions of the overall steel framing pattern. The advantages of this arrangement are:

1. There are always beams or girders parallel and on line with the walls to serve as lateral force collectors.

2. The continuation of these members through the wall allows direct transfer of forces from the diaphragm to the wall.

3. The columns at the end of walls form the required ductile flange members for wall bending.

4. Frame members are in the correct position to provide vertical support for shear wall dead load.

Thickness of walls varied from a minimum of 12" to a maximum of 24". The steel frame weight was 16.5lb/sq.ft.

C. The Imperial Hotel, Tokyo

In the short history of seismic design, one building stands alone in capturing the imagination of architectural historians and journalists, largely as a result of a theatrical piece of good fortune brilliantly exploited by the building's creator. The Imperial Hotel in Tokyo (Figure XIII-9) designed by Frank Lloyd Wright, was opened in July 1923. Foremost in the publicity attending the evolution of the project, was the innovative seismic design, devised by the architect, in direct contradiction to prevalent engineering principles. In September of that year, Kanto province was hit by one of the greatest earthquakes in history, and the Imperial Hotel stood firm and undamaged, alone amid desolation, a triumphant vindication of its designer's genius. This was the myth, carefully nurtured by Wright: the facts are somewhat different.

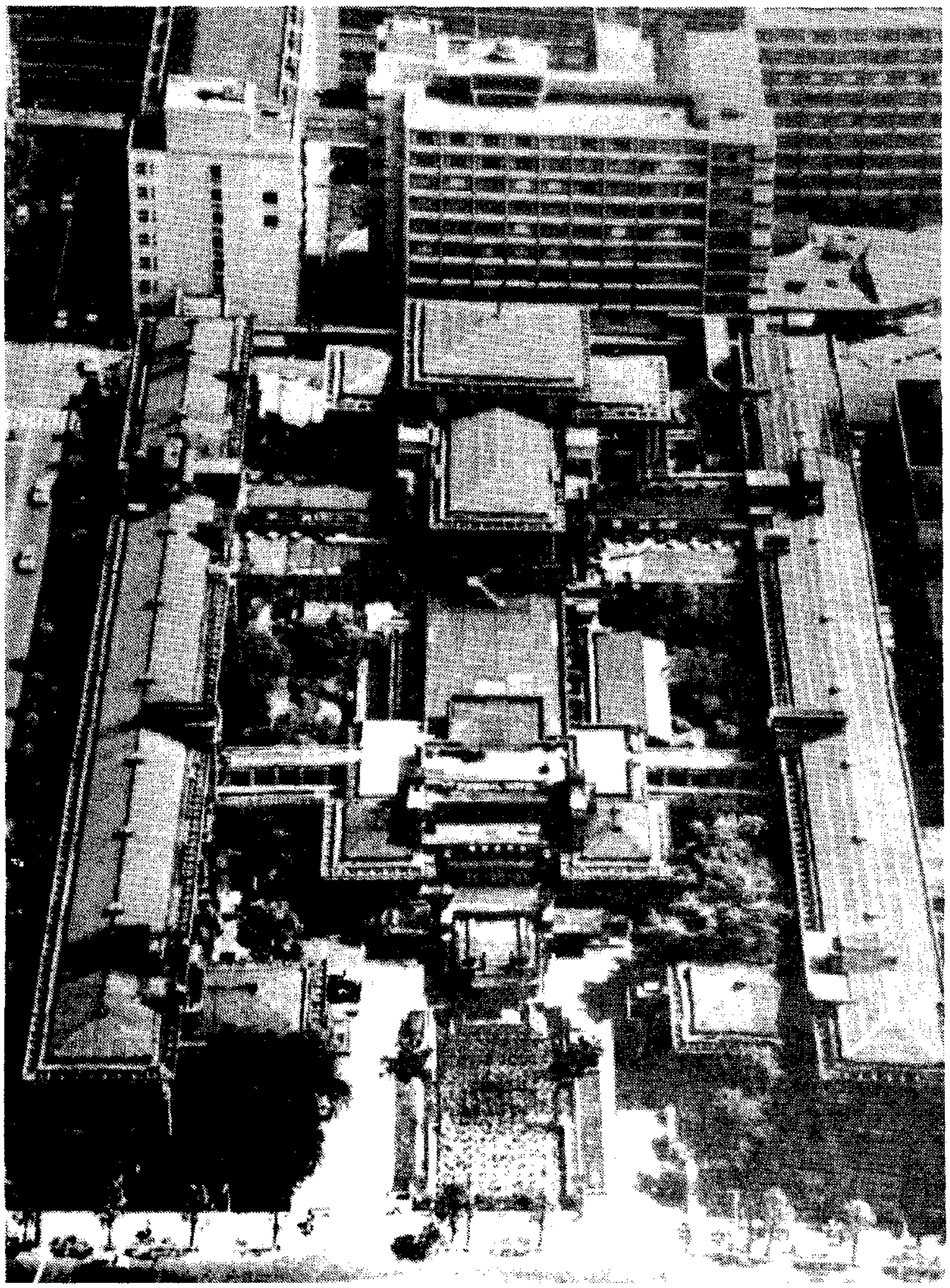

Figure XIII-9. Aerial view of the Imperial Hotel, Tokyo, in the 1960's, prior to demolition to make way for new high-rise hotel on the same site.

The design of the Imperial Hotel was a remarkable episode in the history of seismic design, not least for the mythology that has grown up around the story. An extraordinary degree of responsibility was assumed by the architect in the seismic design, responsibility that would be unthinkable in today's climate of liability and lawsuits. Paradoxically, the aspects of seismic work that are part of the mythology emphasized by its creator, were the least effective. Yet, the building nevertheless performed well, for reasons which were traditional rather than innovative. Of these reasons, the excellence of the configuration was the most significant.

In 1916, when Wright began designing the Imperial Hotel, seismic design was in its infancy. Wright attempted to devise a basic seismic design rationale, based on his understanding of the earthquake phenomenon, and to derive specific design ideas from his basic assumptions. This in itself was innovative, especially so since Wright was an architect not an engineer.

Wright took total responsibility for the design. There was a Japanese structural engineer for the project, Julius Hoto, who executed a structural design based on the Chicago building code. Hoto discovered later that Wright had taken all of his calculations for the reinforced concrete members and then personally adjusted them (1).

"He tells me now that, in building, my computations were disregarded and that much lighter sections were everywhere substituted, making in effect a design which eliminated all the strength usually provided for the live loads. In this connection, the writer would like to comment that this reduction was entirely logical... Many engineers agree that the live load requirements of our building codes are too severe..."

Much confusion about the performance of the Imperial Hotel stems from the fact that Wright, with a strangely modern understanding of the media, personally managed the release of information on his projects, and often for the sake of publicity and effect, would make statements that were dramatic, witty, but off the top of his head as to their technical accuracy.

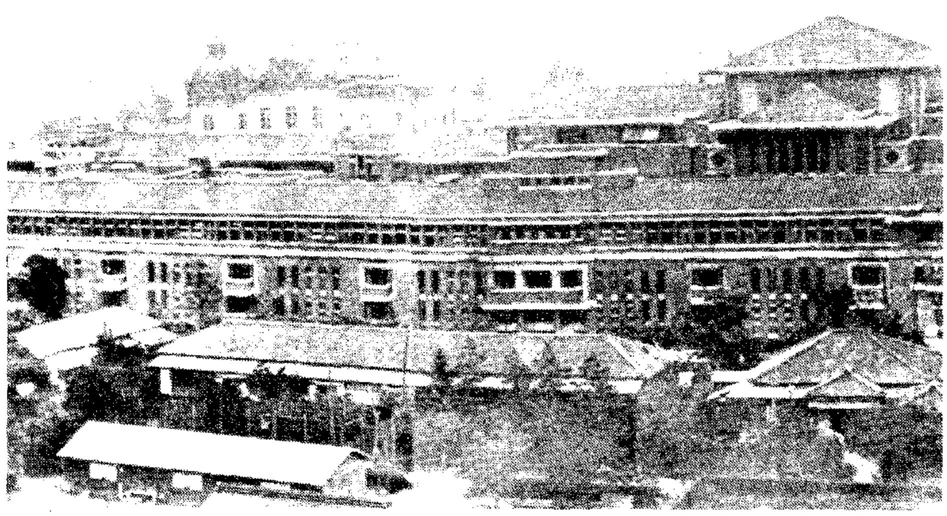

Figure XIII-10. View of Imperial Hotel following the 1923 earthquake, showing an insurance firm's brick building beyond as well.

What happened to the Hotel? One factual source is the insurance underwriter's reports on the damage. Underwriters divided all buildings in Tokyo into five categories of damage, ranging from Number 1, undamaged, to 5, total damage. The Imperial Hotel was rated Category 2, a small amount of damage (Figure XIII-10). As Bradshaw points out (2): "a respectable position in which to be; however, many buildings, including some of Tokyo's largest, were in Category No. 1 (Figure XIII-11)."

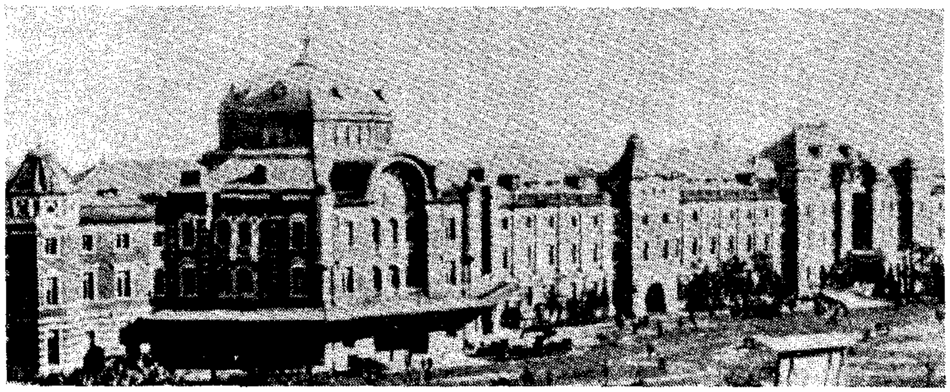

Figure XIII-11. Tokyo Railroad Station - according to contemporary reports, "undamaged."

The Imperial Hotel had some cracked corridor walls, the dining room floor bulged and required cutting or shimming concrete columns to re-level it, and fans, kitchen equipment, lights, and other non-structural elements were damaged. Piping and wiring, which hung free in shafts or was laid loosely in concrete trenches beneath the building, apparently performed well. This separation of non-structural components was quite precocious for its time. The central portion of the building subsided about two feet and subsequently sank about one third of an inch per year. When the building was demolished in 1968, the rear part of the central section had sunk three feet-eight inches.

Baron Okura, the prime backer of the development, immediately sent a wire to Wright saying (3):

HOTEL STANDS UNDAMAGED AS MONUMENT OF YOUR GENIUS. HUNDREDS OF HOMELESS PROVIDED BY PERFECTLY MAINTAINED SERVICE.
CONGRATULATIONS, OKURA

As Farr notes (4):

"The publication of this message in the newspapers was the start of the widely believed and printed myth that the Imperial Hotel was the only building in Tokyo to withstand the earthquake. This however, was far from the truth."

For example, three large structures designed by Dr. Tachu Naito, one of which was 100 feet tall (the height limit at the time), were virtually undamaged. Naito was a leading figure in seismic design, advocating 'stiff' structures, and the concepts introduced by him and other Japanese engineers are still influential today.

Nevertheless, the performance of the building was certainly acceptable. An important aspect of the performance lay in the construction of the exterior bearing walls. The walls were composed of an exterior wythe of solid bricks, an interior wythe of hollow

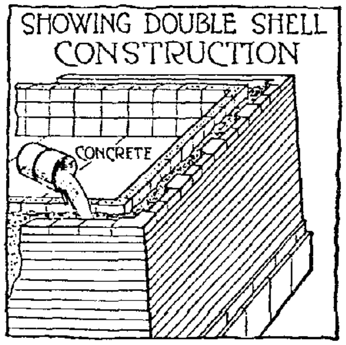

Figure XIII-12. Method of wall construction. Note that this sketch omits steel reinforcing.

patterned bricks, and a solidly filled cavity of concrete. Because of some statement by Wright, there is some confusion as to whether the walls were reinforced, but it seems probable that, in fact, they were, and that Wright either for dramatic effect or from ignorance (writing many years after the design and construction) confused the issue (Figure XIII-12).

Wright's own publicity placed more emphasis on the foundation system that he devised, a combination of short piles and spread footings (Figure XIII-13). Full discussion of this feature is not relevant here, but it may be noted that since there was a soft eight-foot layer of water-saturated surface soil underlain by at least seventy-five feet of alluvium, it is likely that the underlying mud, rather than being "a merciful provision - a good cushion," probably amplified the ground motion. At the same time, it probably filtered out the short period component and transmitted a long period motion to the surface. Since the building had a short period, quasi-resonance of structure and soil was minimized. This was not part of Wright's expressed strategy, but must be assumed to be a part of the explanation for the way the building performed.

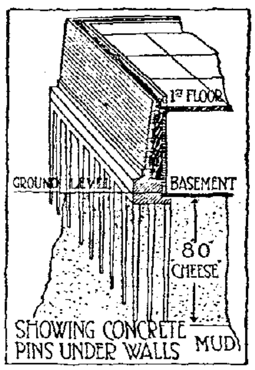

Figure XIII-13. Foundation construction technique.

Probably the most important seismic design factor in the hotel was its division into small component units, changing a complex plan with many re-entrant corners into a set of small rigid boxes, each about 35'x60' in plan. Wright's explanation is, however, more dramatic than analytical. Wright wrote that (5):

"We solved the problem of the menace of the quake by concluding that rigidity couldn't be the answer, and that flexibility and resiliency must be the answer. ...Why fight the quake? Why not sympathize with it and outwit it?"

However, by these comments Wright did not envision, as we would today, some kind of flexible, ductile frame. Wright's method of creating a (6):

"flexible structure instead of a foolish rigid one" was to "divide the building into parts. Where the parts were necessarily more than sixty feet long, joint these parts, clear through floors, walls, footings, and all, and manage the joints in the design."

The simple rectangularity and symmetricality of the component units of the building were undoubtedly helpful in both avoiding undue torsion and in equalizing the disturbance of lateral and vertical loads (Figure XIII-14).

What is immediately apparent today is that Wright used seismic separation joints in a very thorough manner. He seems to have done this out of consideration for the differential displacements that would be caused by a surface wave traversing the site, undulating beneath his long building, rather than out of concern for incompatible vibration movements within, and at the intersection points, of the long wings.

The component units of the building were extremely rigid, not "flexible" and their performance fully vindicated Naito. Their fundamental period was probably less than 1/4 second. The shear walls of the perimeter, along with the rigid diaphragms, and perhaps with some rigidity added by the interior columns, created a very stiff box. The numerous longitudinal and transverse partitions must have further stiffened the structure.

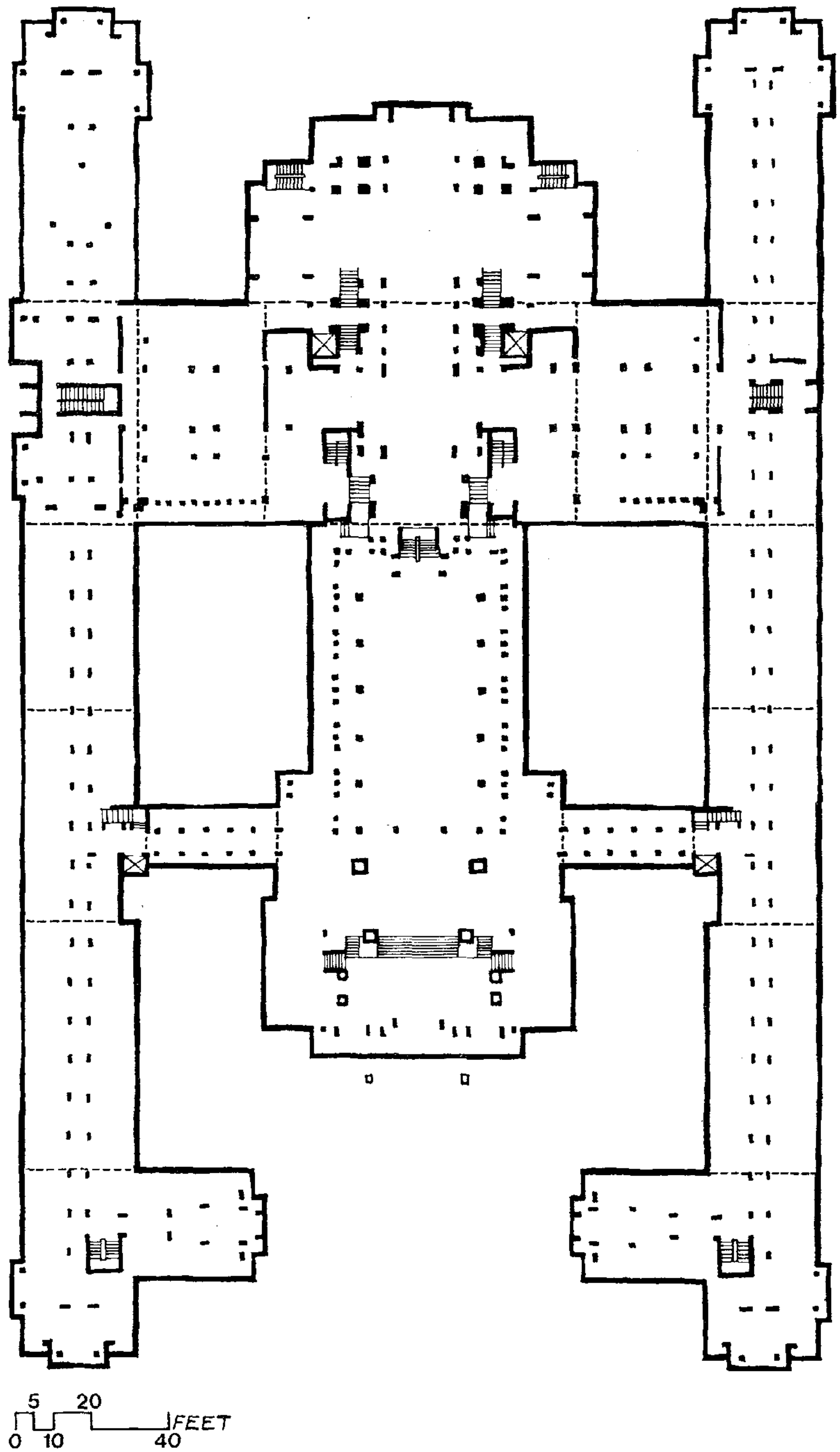

Figure XIII-14. Ground floor plan of the Imperial Hotel, Tokyo. None of the published floor plans show the location of seismic joints described in the text: dashed lines have been suggested by the

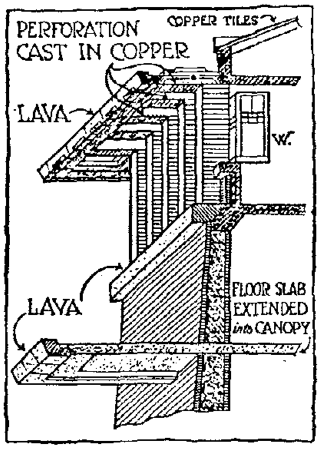

Figure XIII-15. Exterior wall and roof construction.

The hotel units, besides being small and rigid, disposed their masses vertically in a very favorable manner (7).

"The outer walls were spread wide, thick and heavy at the base, growing thinner and lighter toward the top. Whereas Tokyo buildings were all top-heavy, the center of gravity was kept low against the swinging quake movements and the wall slopes were made an aesthetic feature of the design."

The walls were perforated with small windows in the first two stories while more abundant openings were put in the third story. By lightening the mass as the elevation above grade increased, this facade design also effectively disposed more material in the lower stories, where forces are greater (Figure XIII-15). The height/depth ratio was approximately one. Since the Imperial Hotel's fenestration and massing is recognizably related to Wright's earlier mid-western works, it would be unwise to assert that the Imperial Hotel's aesthetic form was purely a functional adaptation to the structural problems of designing a building in a seismic region, but the result was effective nonetheless.

Wright's use of light weight copper sheet rather than the traditional Japanese tile lightened the roof by a factor of about ten, and further lowered the center of gravity and reduced the period of the structure.

We must conclude that Wright's heroic effort to change the course of seismic design failed, for his concepts had little influence in Japan or elsewhere. His building performed well, but his most innovative idea - of foundation design - was probably insignificant in affecting the amount of ground shaking imparted to the building and created major settlement problems. His emotional plea for flexibility in fact concealed a design concept that used rigidity as well as its most staunch advocates could wish, and hidden also is the fact that the building was ahead of its time in the use of seismic separation joints to provide "flexibility." The construction and the massing of the building employed traditional concepts that are perennially effective. Cool analysis of Wright's rhetoric must not dimish his achievement, for the Imperial Hotel remains as a memorial of design courage that must be unique to our century.

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XIV. The Imagery of Seismic Design

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A. Introduction: the Historical Precedents

To the designer, the recitation of problem configurations that has occupied much of this study may appear as overwhelmingly constraining. Does conscientious attention to the guidelines expressed here mean that the seismically economical and safe structure must be visually symmetrical, uniform, regular, repetitive and bland?

We would argue to the contrary: that if the precepts of good seismic design are applied not with superficial adherence to rules but with understanding and imagination, then there are opportunities for design expression that have yet to be realized. It seems reasonable that buildings built in heavy seismic areas might look somewhat different from those built elsewhere; and that this difference in appearance far from being the whimsical straining for attention that mars so much contemporary design, might have its roots in the kind of discipline that can be found in all the great historic works of architecture. To the extent that good seismic design extols simplicity, no good designer need be worried, for simplicity need not imply boredom. Indeed, it is easy to forget the fundamental simplicity of structure of our most enduring architectural masterpieces.

Although most architectural styles and traditions, and civilizations itself, have developed in seismic areas (Mesopotamia, China, Mediterranean, Indus Valley), it is not possible to identify consciously understood and expressed seismic design principles that distinguish the architecture of these areas from that of non-seismic regions. The analytical approach to seismic design is a twentieth century phenomenon, and in historic times earthquakes were regarded as events too mischievous and random to be subject to design control. But the structural basis for design is undeniable: one can encapsulate much of the history of architecture, until the late 19th century, as the struggle to create suitable spaces under the constraint of materials that were only effective against compressive forces. As structures became taller and more delicate, the designer's response to the nature of the lateral forces of wind and buckling created an analogy for seismic design and, in fact, the familiar wonder of gothic architecture is based largely on this response.

Before speculating on the imagery of seismic design for the future, it is instructive to investigate a few familiar historic monuments from a structural viewpoint, in particular with respect to the problem of traditional lateral forces. In some cases - at Santa Sophia for example - resistance to earthquakes has formed part of architectural history. In this we may find it remarkable that so many vulnerable monuments have survived, lacking, as they do, the tensile materials of steel and reinforced concrete that will strengthen and tie the joints of the building together, and having been designed prior to the advent of virtually all of the analytical and quantitative basics of earthquake design. The answer is in the configuration, in the use of simple, generally symmetrical shapes, that reduce the earthquake loads, and the intuitive use of compressive materials in such a way that tensile, shear, and overturning forces are further reduced. In historic structures, configuration is almost the only available tool of seismic design.

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B. The Perfect Configuration: the Parthenon of Athens

The Parthenon has stood for 24 centuries in an area of moderate seismicity. Its plan is symmetrical about both axes. It has a dense perimeter colonnade enclosing almost completely solid shear walls, within which are more columns. A great deal of material is brought down to the ground, a feature which was facilitated by the lack of desire and ability to create a large clear-spanned interior space. Like other examples of pre-Roman architecture, the building needed to have only a small hollow area inside to fulfill its requirements (Figure XIV-1).

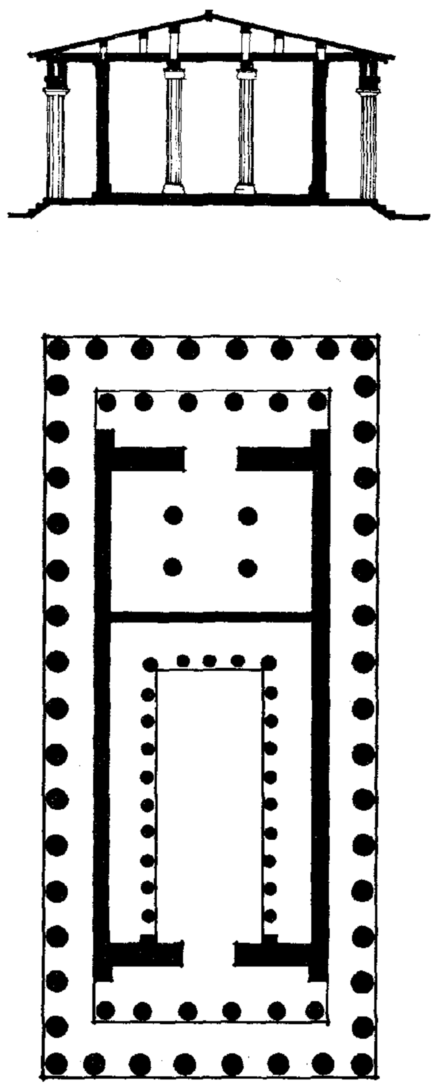


Figure XIV-1. Section and plan, the Parthenon.

The changes that would be made to this design to increase its earthquake resistance were it built in its generic form today, would involve changes of material (changing the unreinforced stone walls into homogeneous shear walls; using a stronger, stiffer roof diaphragm; and changing the stone columns into an earthquake resistant material), an alternative structural system (to transform the post and beam peristyle into a rigid frame), and better structural details (connections between walls and roof, anchorage of any appendages, etc.). However, the configuration - where the resistant elements are rather than what they are - is admirably well-suited for earthquake forces. If one wished to build a building of this size out of blocks of marble without the aid of modern earthquake resistant practices, the Parthenon's configuration could scarcely be bettered. It has been suggested that wooden ring beams in the Palace of Knossos in Crete had a seismic function, but there seems little evidence to assert that earthquakes had any major influence on Greek, or pre-Hellenic architecture.

In comparing Egyptian temples with those of Greece, Rowland Mainstone has noted that while unfinished temples without any walls have stood for centuries (1),

"... in all completed [Greek] temples the walls were also present, and when favorably aligned they probably made a larger contribution to the stability of the whole in the event (more likely than in Egypt) of earthquakes... Primarily perhaps as an added safeguard in the event of earthquakes, the individual blocks were further extensively tied together by iron cramps."

Greek architecture generally did not develop internally generated lateral forces due to arching thrusts, while Roman architecture did so extensively.

C. A Roman Superdome: the Pantheon

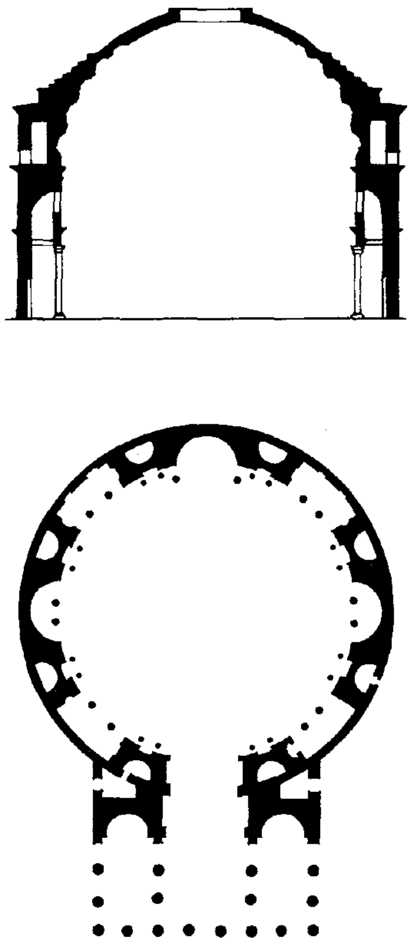


Figure XIV-2. Section and plan, the Pantheon.

Since the grand scale of Roman interior spaces is one of the characteristics which differentiates Roman from Greek classical architecture, it is instructive to study the Pantheon: at one hundred and forty three feet, the largest span building of antiquity, and a record for the world which it maintained for the seventeen centuries following its construction.

The configuration of the Pantheon is simple to an extreme, consisting of a cylindrical drum with a large span dome, to the front of which is added an entrance-forming rectangular portico (Figure XIV-2).

Though historically notable for its height and span, the tremendous mass of the Pantheon is also a prominent structural feature. Unlike the strategic point-for-point buttressing of lateral thrusts in some Gothic buildings, the Pantheon's walls resist the thrust through sheer mass. Although it is true that mass is generally a seismic liability, extremely thick masonry walls can be more stable than this truism might lead one to believe. The vertical gravity force of the Pantheon's mass is so great, relative to any lateral forces, that the entire structure is kept firmly in compression at all times.

This strategy represents a simple form of the prestressing in which today we induce very high compressive forces (by use of a highly stressed tensile component) which has the effect of greatly diminishing the tensile forces in that member.

Friction, which is typically neglected in the design of structures today, is significant in such a massive masonry structure. Friction between the large surface areas of masonry elements provides the connections which would today be made with positive mechanical fastenings, and though one would not wish to count on friction to provide reliable connecting force values, it is part of the explanation for the stability and strength of massive, well built, unreinforced masonry buildings.

The dome is completely solid except for the oculus at the top, and the walls are punctured by only the one monumental doorway. Coffers and hollow spaces are uniformly distributed around the building. This solidity provides unbroken stress paths for lateral forces. Though the walls are built of unreinforced concrete, brick, and stone, and hence can sustain only small shear stresses, their tremendous horizontal cross-sectional areas make up for the deficiencies of the material properties.

D. The Seismic Survivor: Santa Sophia, Istanbul

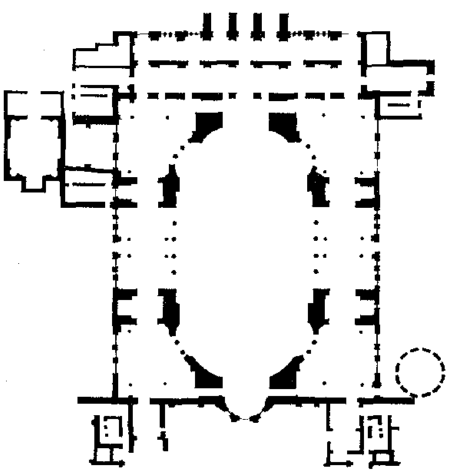


Figure XIV-3. Plan of Santa Sophia.

From the standpoint of building configuration and seismic design, Santa Sophia is one of the most interesting buildings to analyze because in large part, it can be considered as a large scale and skillful attempt to handle the problem of lateral forces.

The designers, Anthemios and Isodorus, arranged the building's configuration to efficiently handle the horizontal forces created by the domes and arches, rather than to deal with the horizontal forces created by earth shaking, but the simplicity, symmetricality and mass disposition for dealing with vertical loads proved also to be effective in dealing with horizontal forces (Figure XIV-3).

The lateral thrust problem induced by the large span shallow dome of Santa Sophia is handled quite directly by the structure's configuration. Pendentives bring the dome's circular plan down to a square drum, and convert the uniform outward thrust in all directions into forces at the corners. Huge buttresses take the thrust at the two sides, and half domes lean against the two ends (Figure XIV-4). The volume and mass of the building spreads out toward the base, efficiently distributing lateral and vertical resistance.

The half domes on the east and west provide a continuous line of reactions for the central dome, and they in turn have smaller half domes at their bases, but on the two buttressed sides, the thrust forces must be carried sideways by the north and south pendentive arches to reach the reactions provided by the buttresses. The pendentive arch is widened on these two sides - prominently protruding on the exterior - so that the masonry can act like an arch or arch-beam on its side. The buttresses proved to be undersized - a quantitative error that could not be foreseen in that era - and their outward deformations allowed a portion of the dome to collapse during repairs following an earthquake.

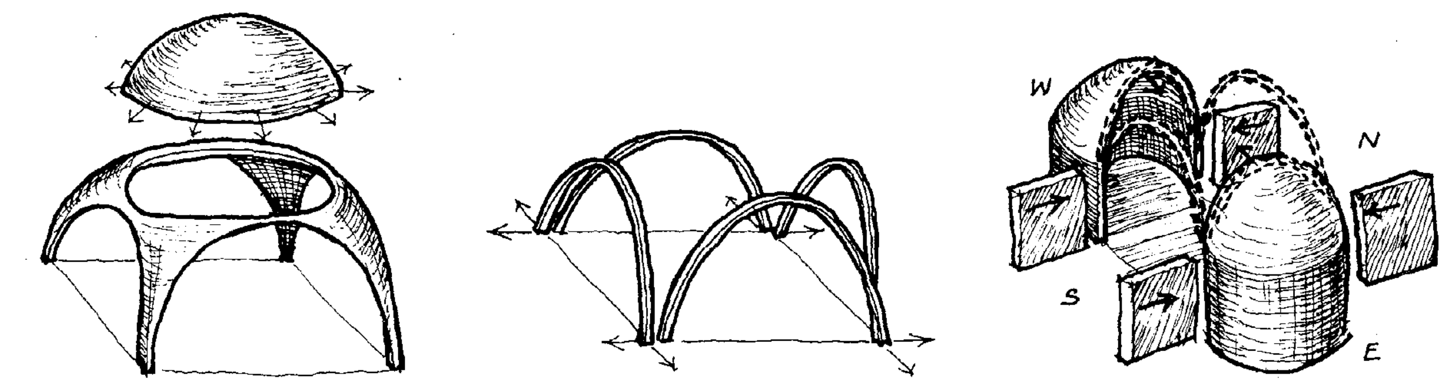
Of all the great architectural monuments, Santa Sophia has perhaps endured the most seismic activity. Although the building's configuration is good, the structure has experienced partial collapse due to lateral forces induced by gravity (arching thrusts), as well as earthquake. These failures can be attributed to inadequate member sizes, material properties, and connections, however, rather than configurational weaknesses. It is interesting to note the configuration changes which were made in response to the damage caused by the frequent earthquakes in Constantinople (2).

In 558A.D. an earthquake damaged the eastern portion of the church. Paul the Silentiary recorded at the time that the top of the eastern semi-dome and part of the central dome collapsed. Perhaps one of the smaller semi-domes abutting the east semi-dome collapsed as well. Since the central dome is ribbed, each rib springing from between each of the forty windows at the dome's base, it would have been possible for only a portion to fall. The damaged central dome was dismantled and re-constructed 20-25 feet higher. The spherical shape was kept, and hence this taller dome would have been similar to, but more stable than, the original, since the shallower the dome, the greater the thrust. Material was added to the upper part of the buttresses on the north and south.

During the reign of Basil I in the ninth century, extensive repairs were made, and in an earthquake in 975 A.D., the western semi-dome and its pendentive arch were severely damaged and partially collapsed.

Thirty-six earthquakes have been recorded at Istanbul between 366A.D. and 1894. In a major earthquake in 1346 the eastern pendentive arch, the dome on one side of it and a portion of the semi-dome on the other side, all collapsed, in a recurrence of the 558A.D. event. An iron tie was placed around the dome's circumference in 1847.

In spite of this formidable seismic history, this huge unreinforced masonry structure still survives to amaze us with its sense of structural mystery.



DOME PRODUCES LATERAL FORCES IN ALL DIRECTIONS, BORN BY 4 PENDENTIVES.

PENDENTIVES RE-CHANNEL FORCES INTO ARCHES, ALONG TWO PERPENDICULAR AXES.

LONGITUDINAL LATERAL FORCES RESISTED BY HALF DOMES, TRANSVERSE THRUSTS BY BUTTRESSES.

Figure XIV-4. Analysis of lateral force-resistant system of Santa Sophia.

E. The Expression of Lateral Resistance: the Gothic Cathedral

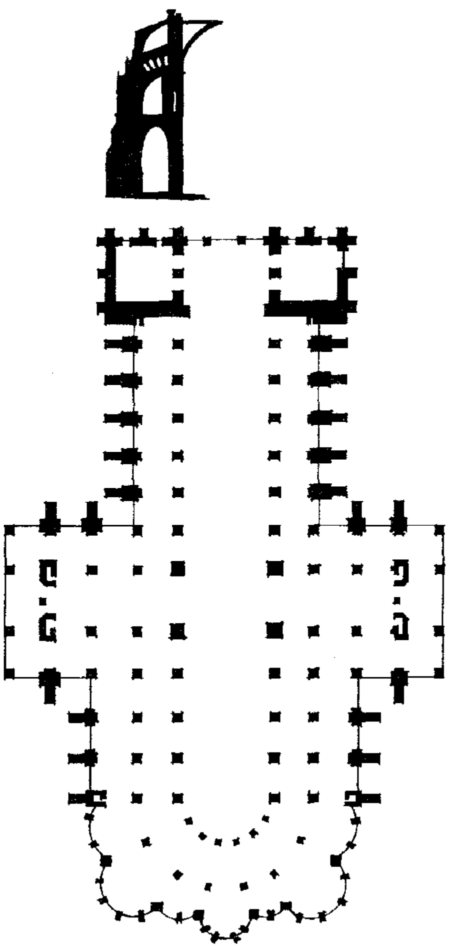


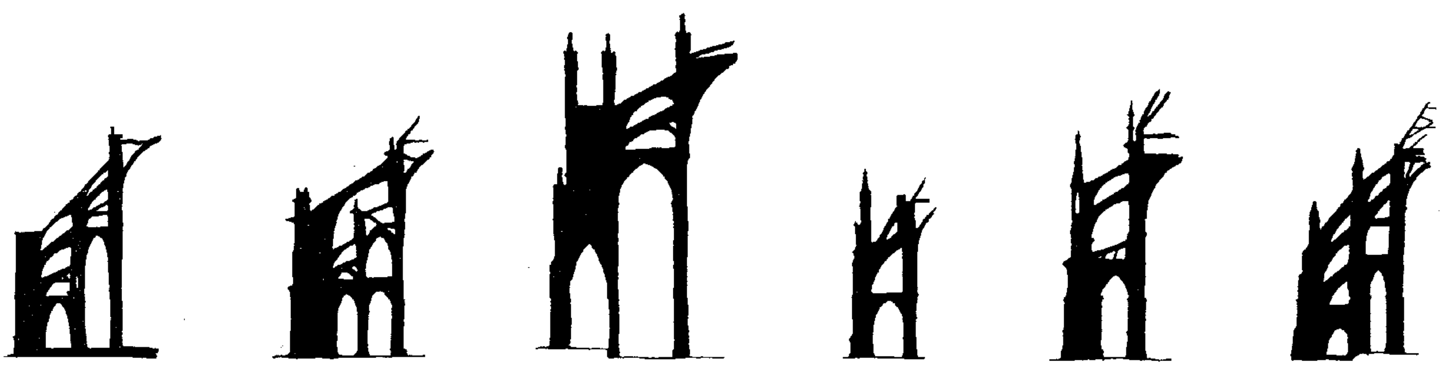
Figure XIV-5. Section through buttress, and plan of Chartres Cathedral.

Medieval cathedrals have long been noted for their structural expression. The bold and clear sense of structure in a Gothic cathedral is cited as one of the key characteristics that distinguishes Gothic architecture from the more "static" image of classical architecture. Gothic buildings were impressively tall. Although the tower of Beauvais Cathedral stood only four years before collapsing, it was the first structure to break the 500 foot barrier. Strasbourg's more stable tower reached 466 feet, which was only exceeded at that time by the Pyramid of Khufu (originally 482 feet tall). The contrast between the laborious structural concept of the pyramid and the daring creativity underlying the design and construction of Gothic spires and vaults emphasizes one of the reasons, apart from mere statistical records, why we think of "structure" when we mention "Gothic." The other main reason for this identification is perhaps the pervasive structural-aesthetic interplay which is more obvious in Gothic architecture than in other traditions.

Though both Gothic and classical buildings rely on mass to make their unreinforced masonry structures hold together, Gothic buildings display an effort to significantly reduce the mass employed by using such rationalizing techniques as buttresses, pinnacles, and more efficiently shaped arches.

The most characteristic structural image of the Gothic cathedral, is that of lightening of the wall, achieved by turning it at right angles to form buttresses. Chartres Cathedral is chosen to illustrate the characteristic features of a Gothic Cathedral (Figure XIV-5). This is a direct response to a sense of lateral forces, both wind loads due to the steep gable roof and arching thrusts, and the flying buttress, which relies as much on depth - by increasing the effective width of the structure against overturning - as on mass, in a remarkable instance of the refined development of a structural concept (Figures XIV-6, 7).

The buttresses of the Gothic cathedral are at once straightforward structural solutions as well as successful and evocative architectural forms, and it is somewhat surprising to note the magnitude of the aesthetic change which occurred when the formerly rather "clean", exterior shell had pronounced articulated buttresses added to it, to handle lateral arching thrusts.



BOURGES NOTRE DAME PALMA ELY RHEIMS WESTMINSTER

Figure XIV-6. A variety of expression in flying buttress construction.

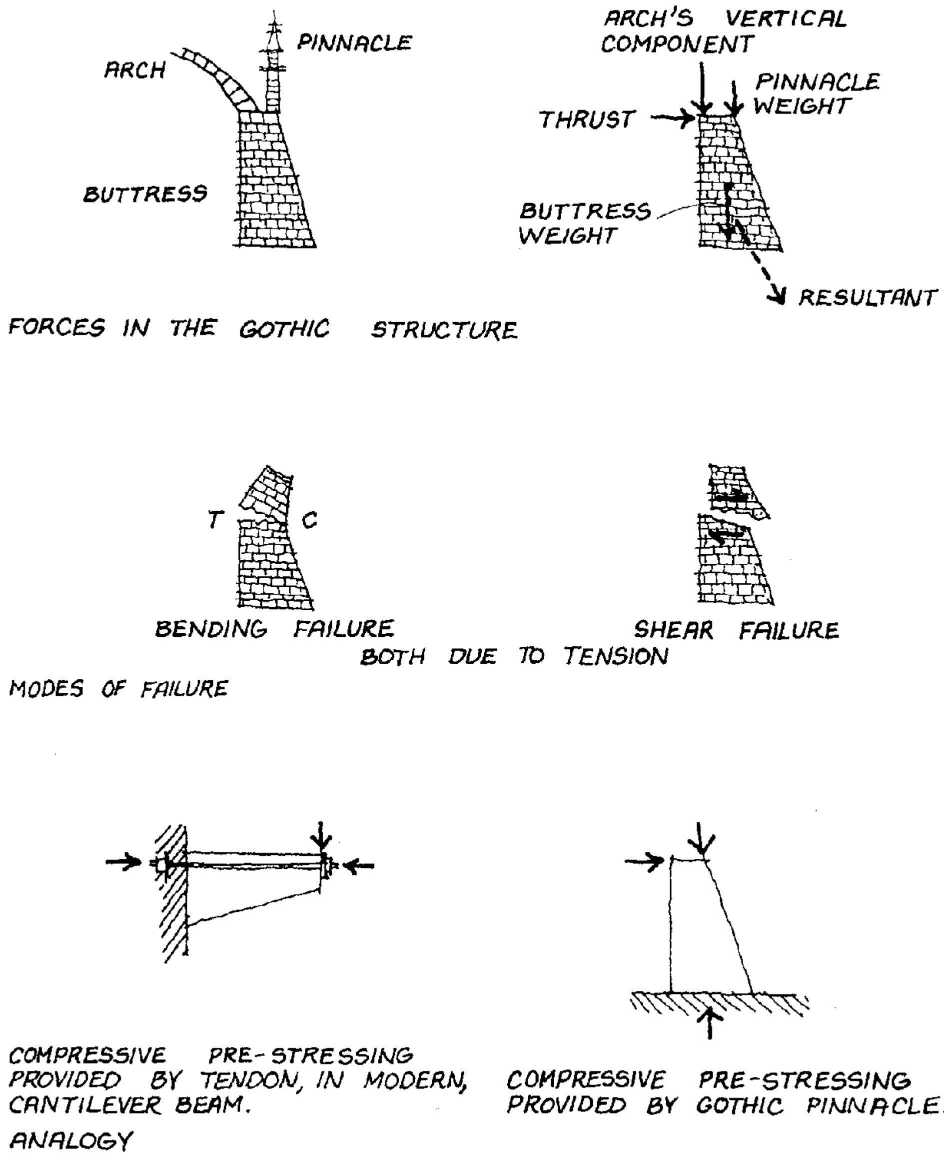


Figure XIV-7. Lateral force-resisting role of the pinnacle in Gothic architecture. (Not all pinnacles were necessary to stabilize buttresses, and some were added during nineteenth century renovations.)

F. The Wooden Structures of Japan

Historic Japanese design and construction has developed along lines quite different from that of the large masonry structures of the west. The typical structure consists of a wooden frame, with deep overhanging eaves to protect the wood structure from decay.

Contrary to legend, traditional Japanese temples and castles have suffered considerable earthquake damage and loss, and architectural history is replete with comments on rebuilding and repair. For example, the Hokoji was a hall erected in Kyoto in 1589 to enshrine a colossal Buddha. The building was destroyed by an earthquake in 1596, rebuilt in 1614, and destroyed by another earthquake in 1662. The earlier earthquake of 1596 also damaged the Fushimi Castle at Shigatsu, causing it to be rebuilt on a different site (3).

For these buildings, the immense and heavy tile roof was a major culprit, providing a heavy mass towards the top of the building. This problem persists today in the light wood frame house with its tile roof. However, the traditional residential structure can withstand large deformations before collapse. In tests on a full-scale, single story wooden model carried out at Tokyo University in 1939, the model did not collapse until the deflection of vertical-resisting elements due to lateral loads reached up to 17.5 centimeters per meter of height of the elements, or almost 2 feet in a 10 foot high structure.

However, the wooden pagoda, generally part of a temple complex, represents an especially interesting case, for there has been no reported instance of serious damage to these structures. The three and five storied pagodas in Kyoto, shown in Figure XIV-8, are typical, dating from the 14th and 15th century.

Glen Berg has commented on these structures (4):

"Pagodas are relatively flexible structures, having natural periods in the range of 1 to 1.5 seconds, considerably longer than the periods of most other structures in Japan and longer than the dominant period of ground motion in Japanese earthquakes. Wooden structures are relatively light in weight and hence incur smaller inertia forces than some other types of structures. But their remarkable ability to withstand earthquakes must be attributed largely to their structural damping, for any deformation of a pagoda is accompanied by the friction of timber sliding on timber and wood on wood in the contact surfaces of timber joints."

One theory for the unusual resistance of pagodas, is based on the peculiarity of their construction, which uses a central column independent of the surrounding structural frames, suspended like a pendulum from the top of the pagoda. This technique was developed in the 17th century in order to eliminate the difference between the small shrinkage of the central column in its longitudinal direction, in comparison to the large shrinkage of the surrounding girders and beams across their grains. This construction is seen in the five story pagoda of Figure XIV-8. However, there are many examples of pagodas in which the central column stands directly on the ground or is supported by a girder at the second floor, as in the three story pagoda of Figure XIV-8. Consequently, one cannot infer that the good earthquake performance of pagodas is entirely due to the pendulum-like central column.

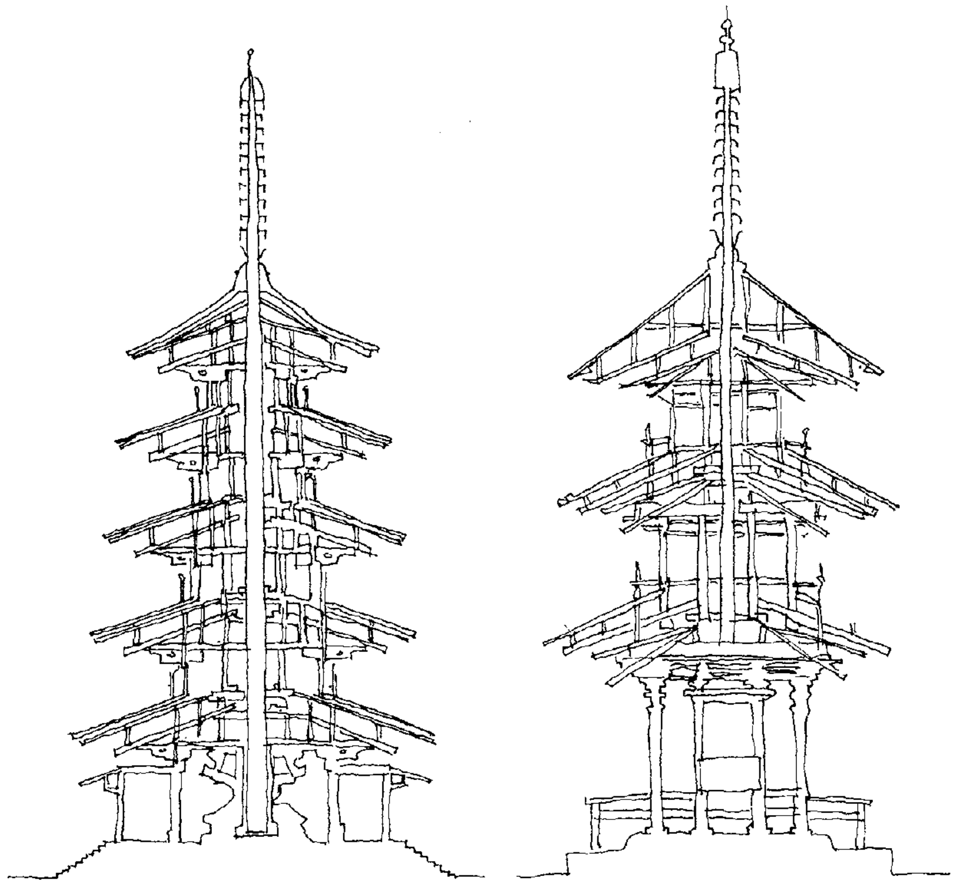


Figure XIV-8. The 5-story Horyu-ji Temple at Nara and the 3-story Sammyo-ji Temple at Toyokawa-shi, Aichi.

Prof. Tanabashi of Kyoto University (5) summarized a set of reasons for the good performance of the pagoda in terms that closely resemble those of Glen Berg. He has suggested four features of the pagoda:

1. The natural period of the pagoda is very long (1 to 1.5 seconds) compared with other traditional structures, and generally considerably longer than the ground period.

2. Pagodas have sufficient strength to withstand considerable lateral forces.

3. Pagodas can suffer large deformations before failure.

4. Pagodas provide a large amount of structural damping.

These four characteristics, as Prof. Tanabashi comments, represent something of an ideal set for earthquake resistance. The material and the design allows a strategy analogous to that of the ductile frame building of today.

G. Symmetry: the Hangars of Nervi

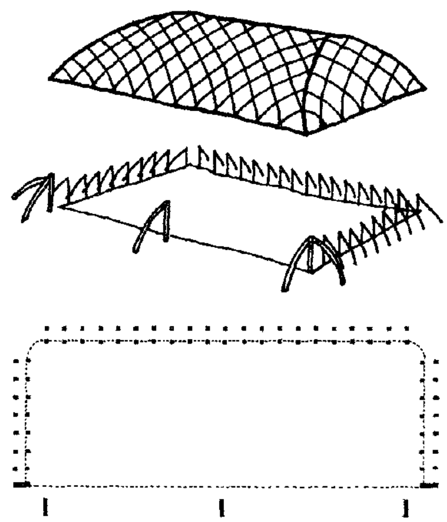


Figure XIV-9. 1936 Italian Air Force hangar designed by Nervi.

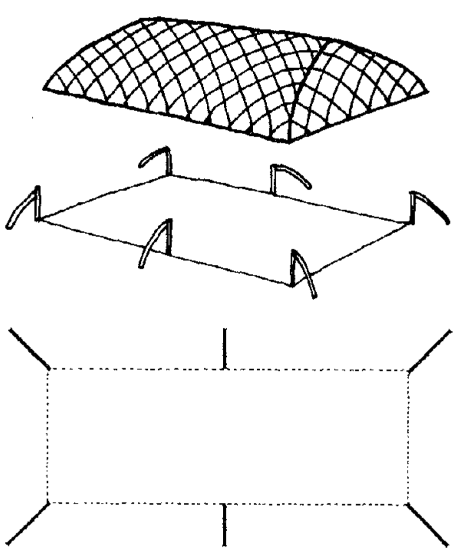


Figure XIV-10. 1939 hangar was based on the re-design of earlier hangar by Nervi.

The great Italian architect/engineer Pier Luigi Nervi seldom contended with the problems of seismic design, but he exploited the aesthetic delight in rationalized structure to a degree unparalleled in the twentieth century. His structural design was always remarkable for its simplicity and regularity, and he achieved the greatest beauty of form from the imaginative expression of structural forces, always within a realistic context of construction economics. As one example of Nervi's methods, his analysis of the virtues of symmetry is instructive.

The hangars built by Nervi's construction firm for the Italian Air Force in 1936 were asymmetric due to the requirement to be able to move planes in and out through one side. Closely spaced column-buttresses were distributed around the other three sides (Figure XIV-9).

The indeterminancy of the structure made physical model testing necessary, but Nervi's initial static analysis, using intuitive assumptions where required, proved to be substantially correct. The design-and-build bid was made on the basis of Nervi's simplified analysis and further refined when his firm was awarded the contract.

In 1939 a competition was held for similarly sized hangars, with similar programmatic requirements. On the basis of his experience with the earlier hangars, Nervi concluded that three improvements should be made: the roof should be lightened by the addition of beams or purlins; the arch ribs themselves should be lightened by using pre-cast trusses rather than solid members; and the third point, having to do only with the configuration itself, was to make the arrangement of supports symmetrical in order to equalize the stresses within the structure (Figure XIV-10).

Although it might seem that one would take advantage of every possible point of support in the design of a long span roof, Nervi's improved 1939 design eliminated most of the columns of the earlier scheme - rather than 40 supports only six were used, but the six supports were located to produce complete symmetry. This simplified design (6),

"less complicated than the previous one, was restudied and calculated using simplifying hypotheses which allowed me to achieve more accurate results than those of 1936. These results were checked by means of model analysis at the Laboratory of the Polytechnic Institute of Milan. The two methods gave comparable results."

H. Seismic Imagery of the Present and Future

In view of the interest in structural expression in twentieth century architecture, it is surprising that this movement largely confined itself to the expression of vertical forces. Examples of the expression of wind and earthquake bracing are infrequent. There are probably two main reasons for this. One is that in the last few decades, architects have tended towards formal conceptions that were sculptural in origin rather than structural, and the help of the engineer was invoked to make their forms achievable, rather than architect and engineer working together in concert from the beginning to generate solutions.

The other reason is ignorance: while the architect often had some concept of the nature of vertical forces and of the characteristic expression of resistance to gravity, his knowledge of seismic forces tended to be very limited. In this circumstance he tended, again, to rely on his engineer to make his building forms possible rather than to enjoy the exploration of the explicit illustration of seismic resistance. Consulting engineers tend to be limited in proposing structural solutions with strong aesthetic content and generally efface their own aesthetic preferences in favor of their employer architect.

This attitude would seem to result in aesthetic loss. There is the loss occasioned by the universalization of forms that appear in the same guise across the country, even across the world. However dramatic the individual forms, the repetition of their idioms, unrelated to national culture, topography, climate, or geology, results in true blandness.

There is the loss resulting from the lack of a firm basis upon which to build a sense of form, the kind of basis that the constraints of load bearing masonry provided, and the forms of wall, buttress and arch that resulted from understanding of the material and an imaginative response to its limitations. All great art and artists thrive on limitations: no one is more disciplined than the truly great creator, whether a Michelangelo, Shakespeare, or Einstein. As our economies become strained, and we begin to sense the finite nature of our resources, we must acquire a new respect for the limits of our materials and methods and, just as the masters of the past, turn these to our aesthetic advantage.

As mentioned above, contemporary examples are few in which the design response to seismic forces is used as a source of imagery. Some examples are shown following, together with some examples in which design for the other common lateral force - wind - is also made explicit. These examples are shown not to suggest that all answers have been explored, but to indicate the possibility of an exploration that might yield rich benefits in the aesthetic opportunity created by the problem of seismic design.

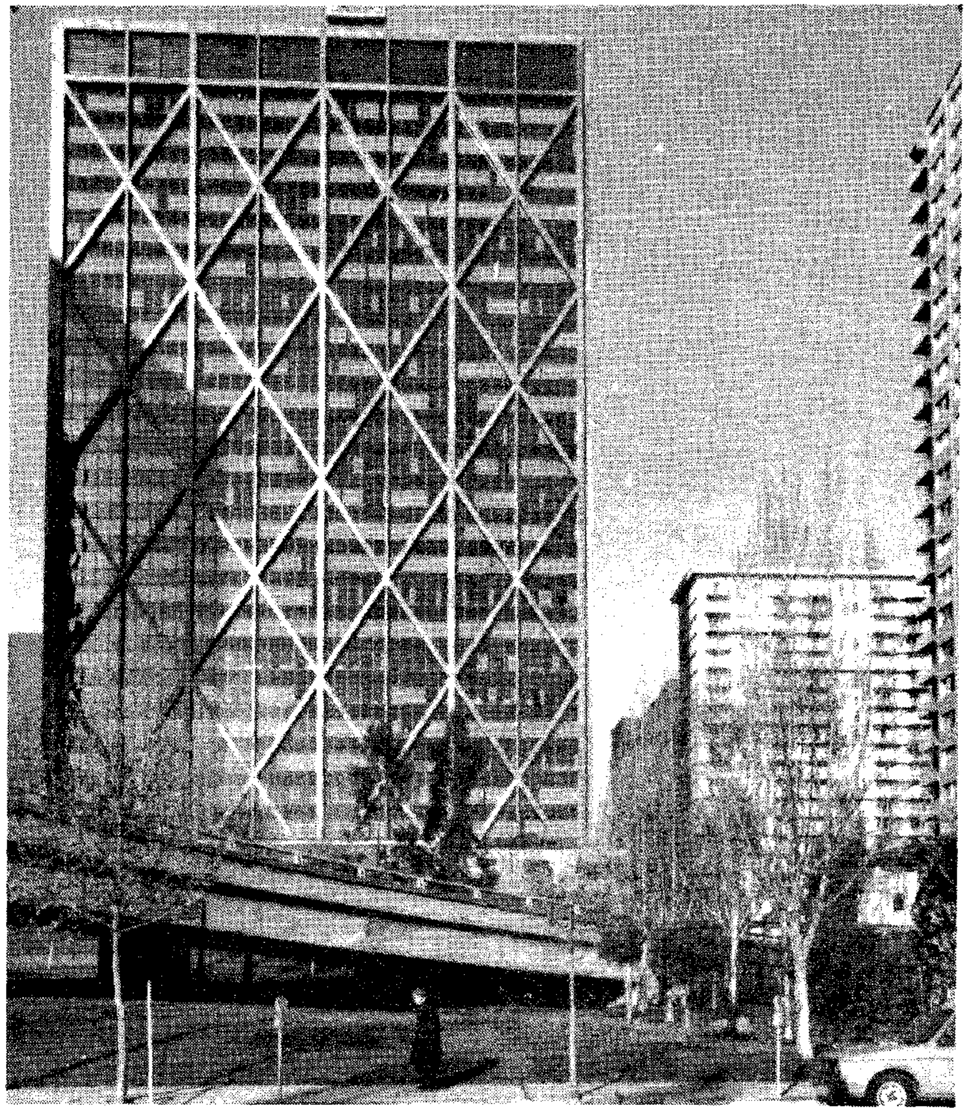


Figure XIV-11. Exposed braced frame. Alcoa Building, San Francisco, California. Skidmore, Owings & Merrill, architects.

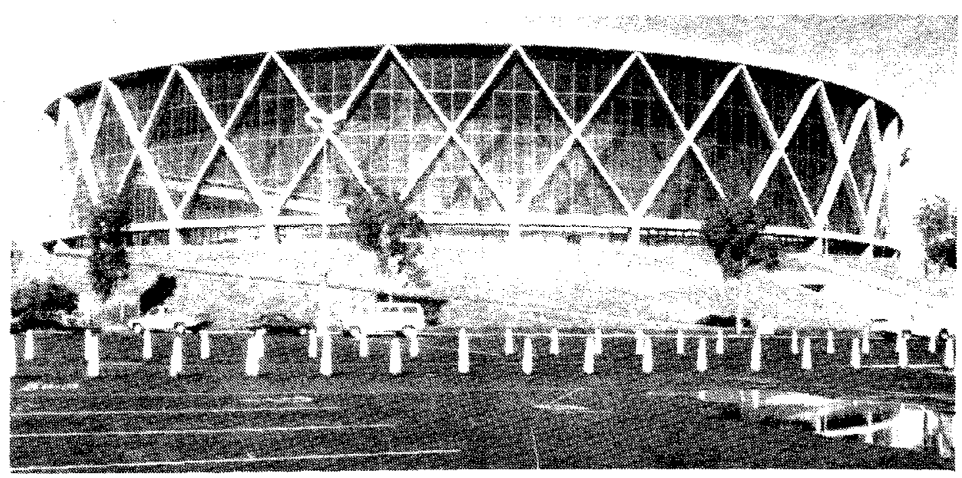
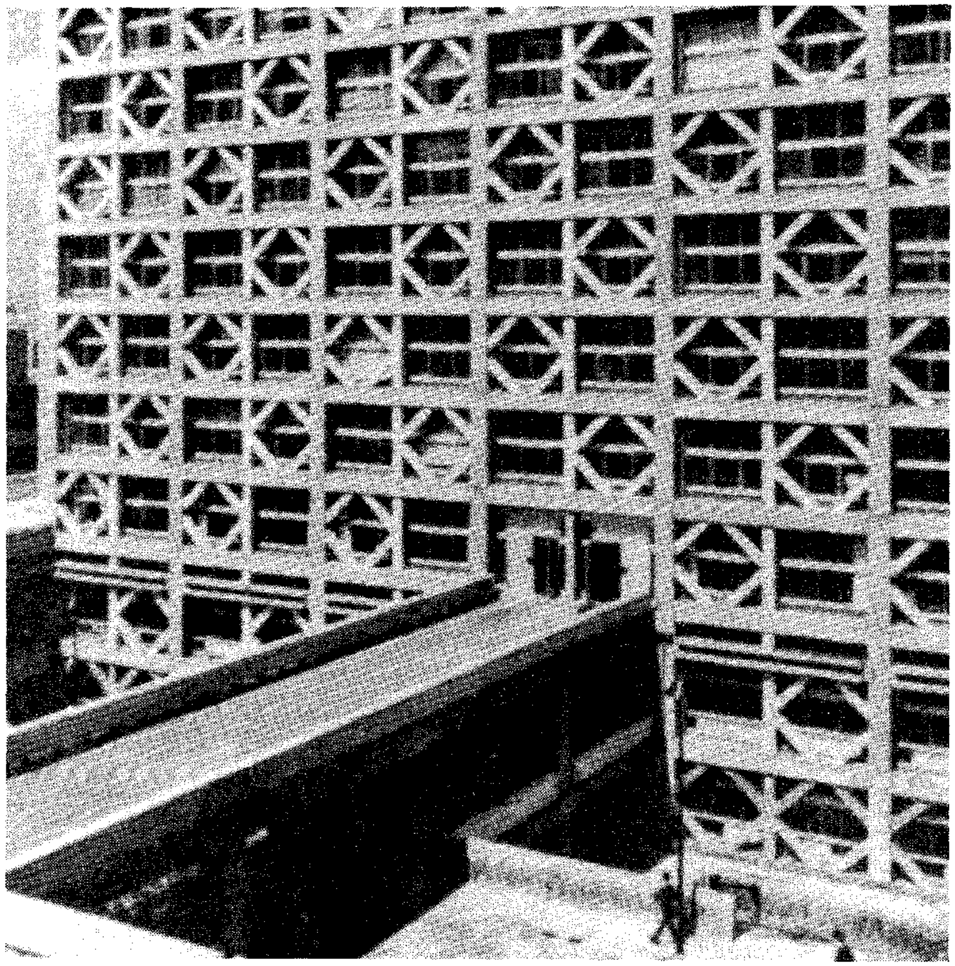


Figure XIV-12. Exposed braced frame. Oakland-Alameda County Coliseum Complex, Oakland, California. Skidmore, Owings & Merrill, architects.



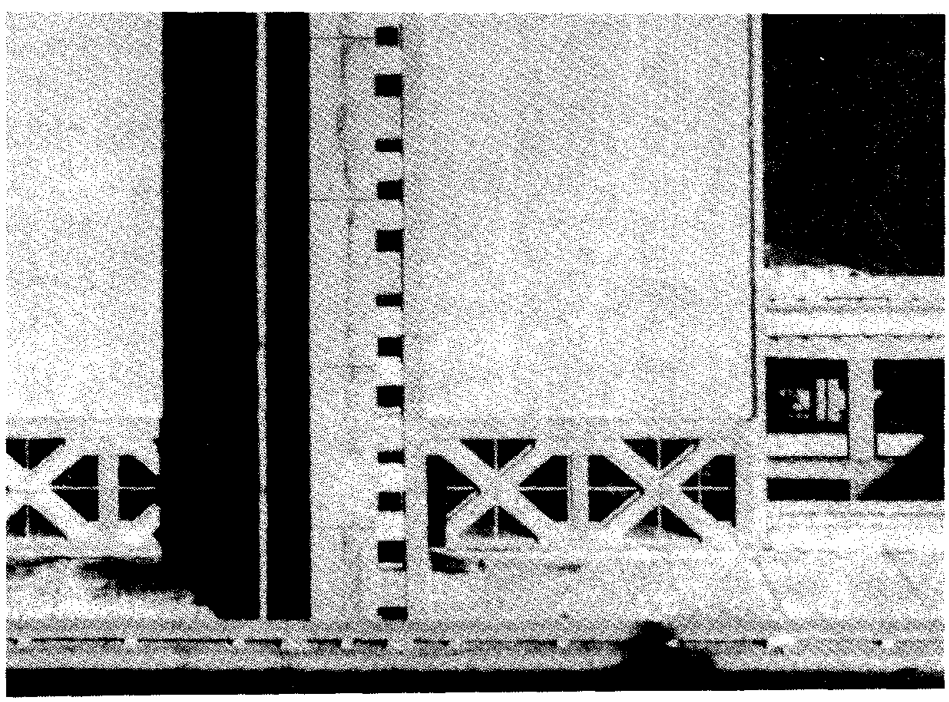


Figure XIV-13. Inset checkerboard bracing, and braced, open, first floor. Physics and Engineering Building, Waseda University School of Science & Engineering, Tokyo, Japan. Katsuo Ando and his staff, architects.

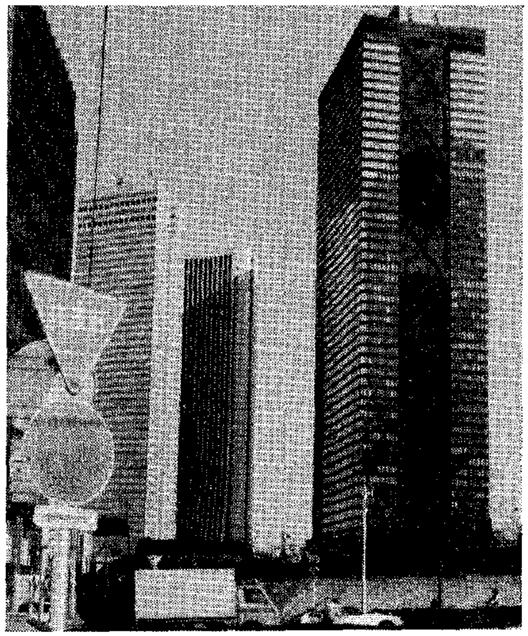


Figure XIV-14. Exposed bracing to the core. Mitsui Building, Tokyo, Japan.

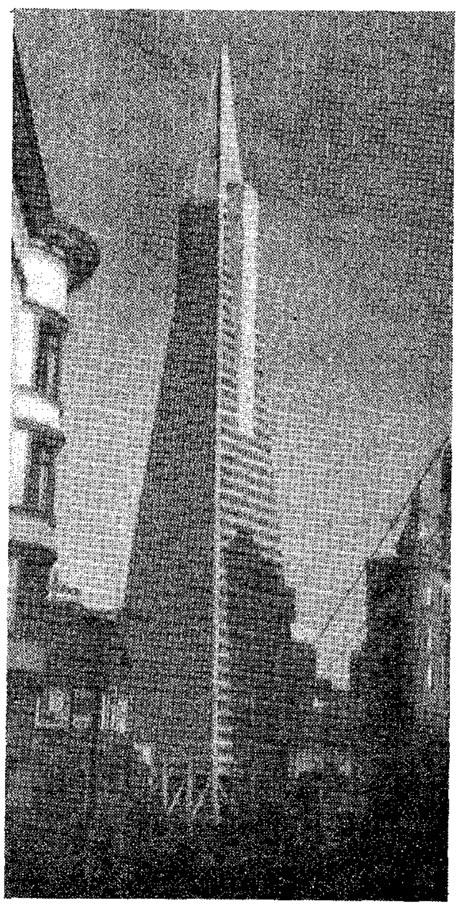


Figure XIV-15. Braced transition from first floor to tower, and tapering elevation. Transamerica Building, San Francisco, California. William Pereira, architect.

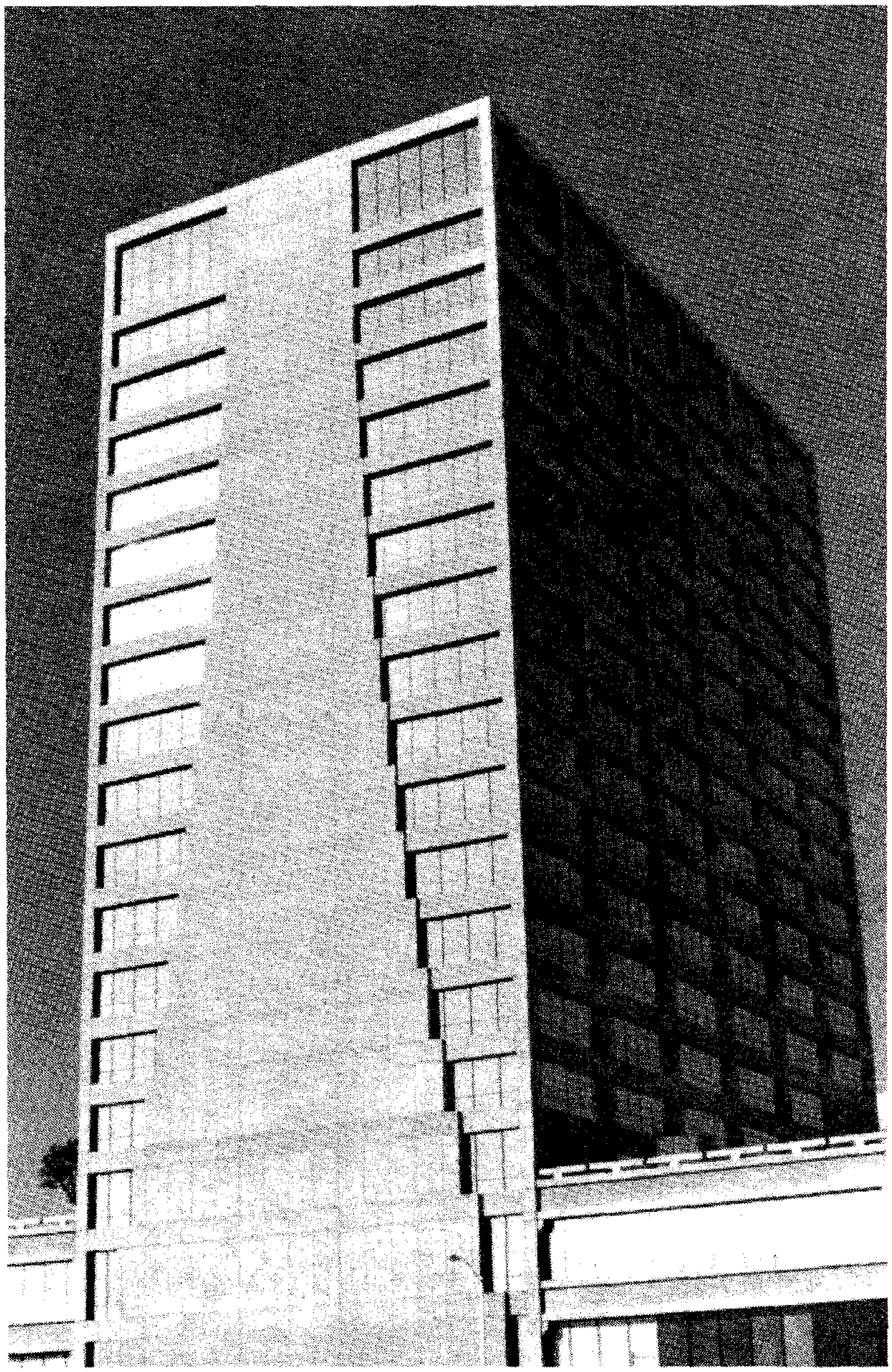


Figure XIV-16. Shear wall diminishes with height. Tandy Center building, Fort Worth, Texas. The lateral forces-whether wind loads or seismic-which must be transmitted by a shear wall, increase towards the building base. This simple and economical office building is a rare example of shear wall design that reflects this: the design also increases the available window area. Growald Architects.

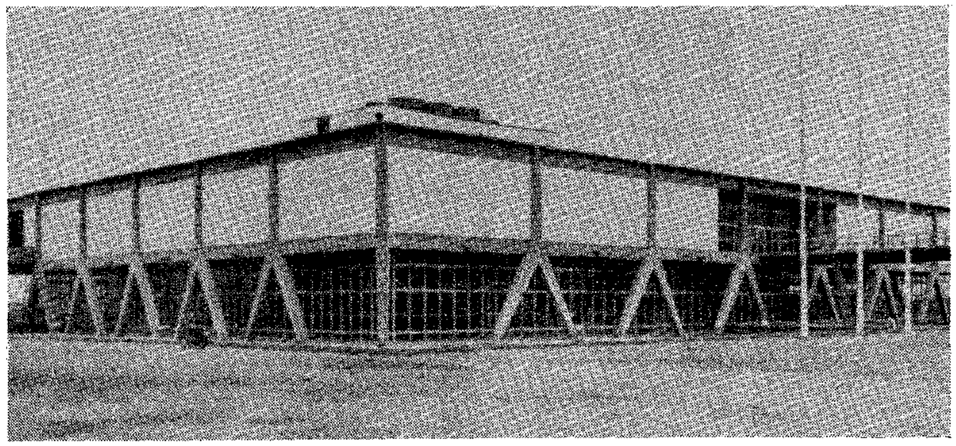


Figure XIV-17. Braced first floor. Tokyo International Trading Center, Tokyo, Japan. Masachika Murata, architect & associates.

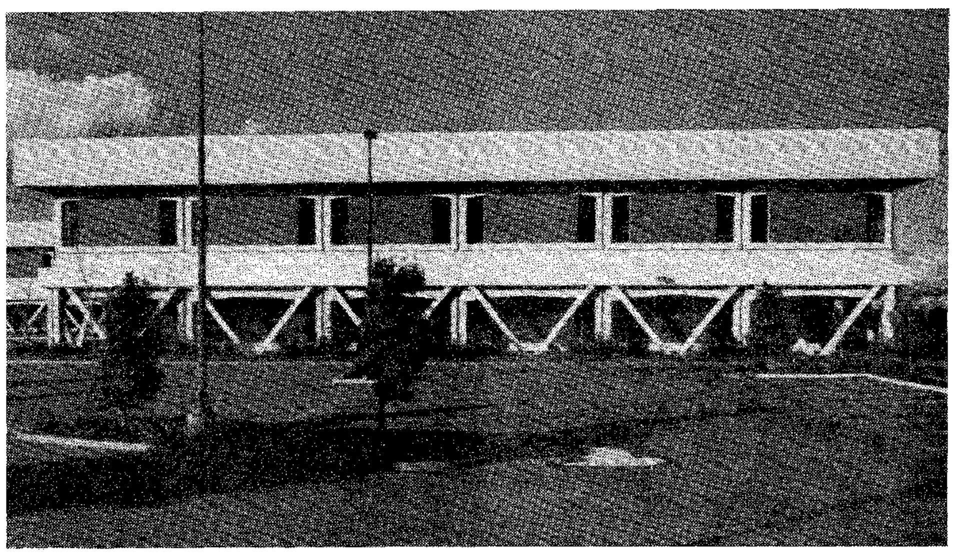


Figure XIV-18. Braced first floor. Office building, Sunnyvale, California.

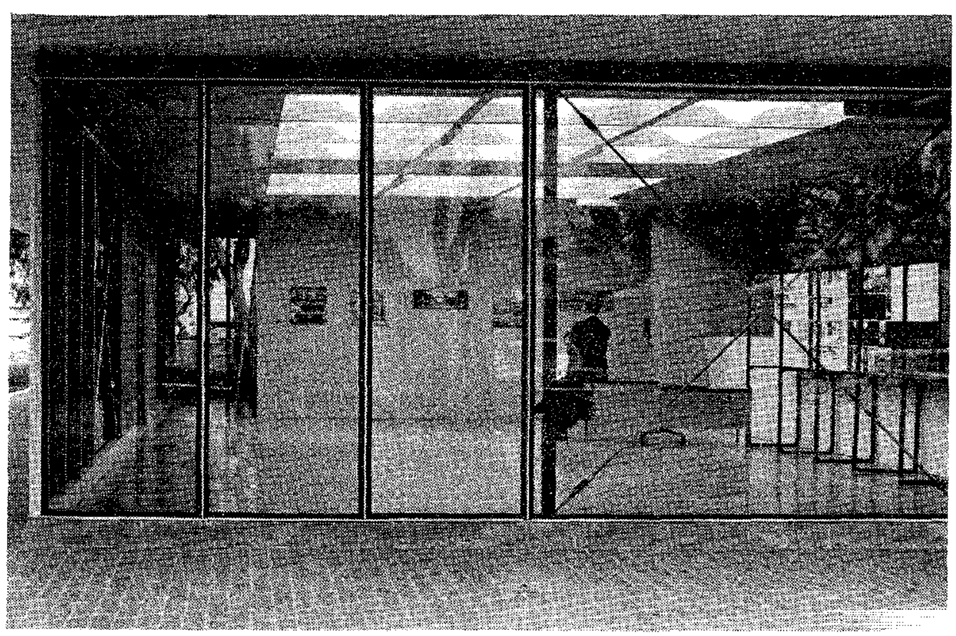
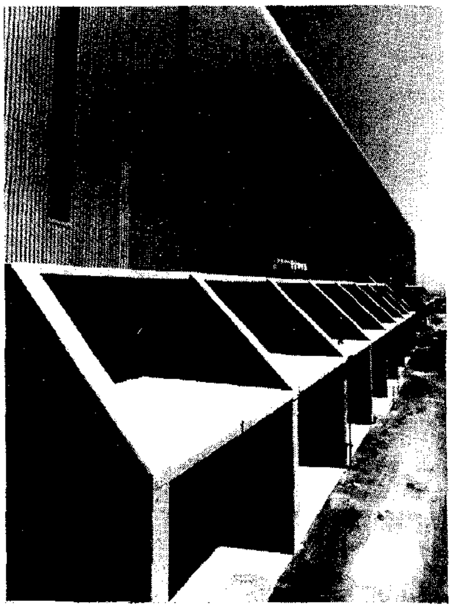


Figure XIV-19. Braced light steel frame. SCSD prototype building. Stanford University, California. BSD, Inc., architects.



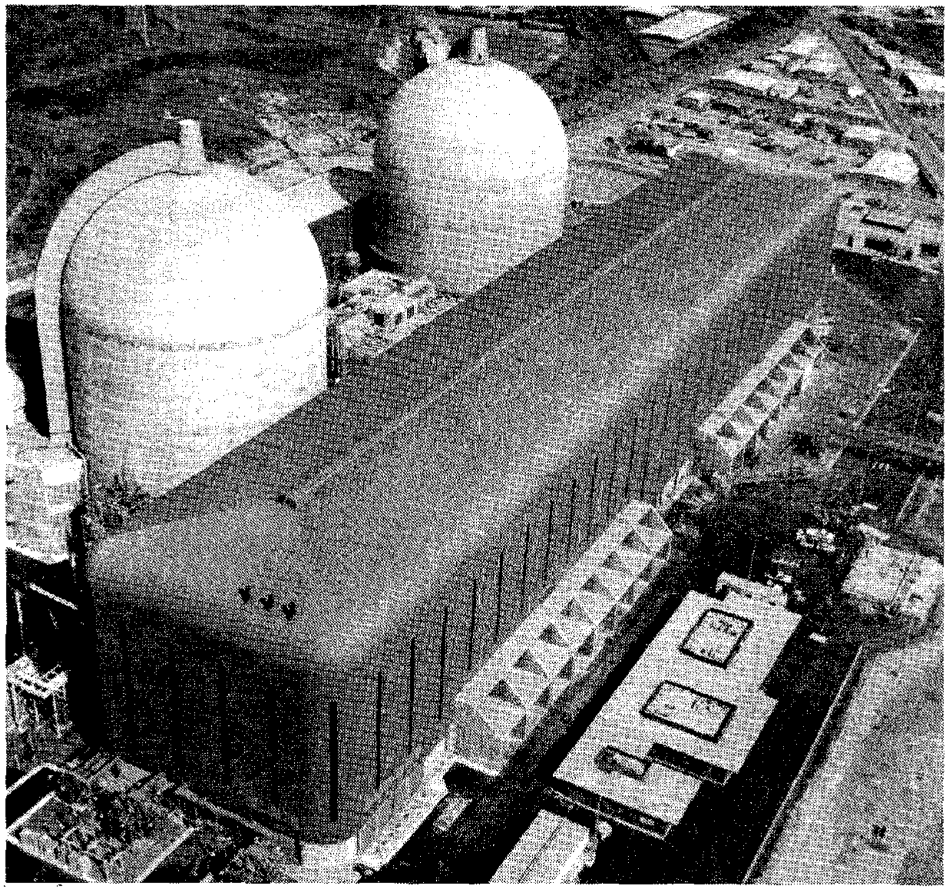


Figure XIV-20. External buttresses. Diablo Canyon Nuclear Power Plant, San Luis Obispo, California. PG&E.

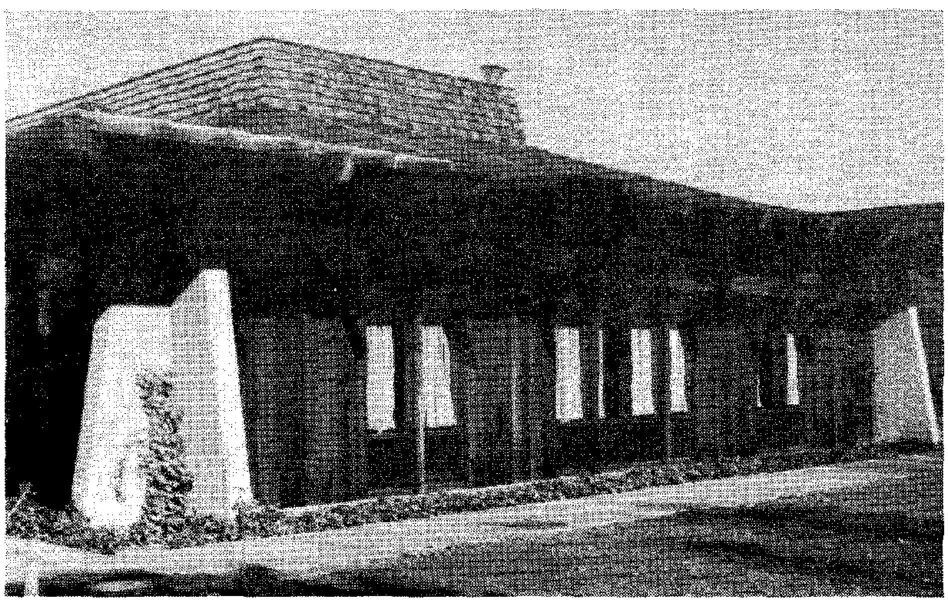


Figure XIV-21. External buttresses at corners. Foothill College, Los Altos, California. Kump Associates, architects.

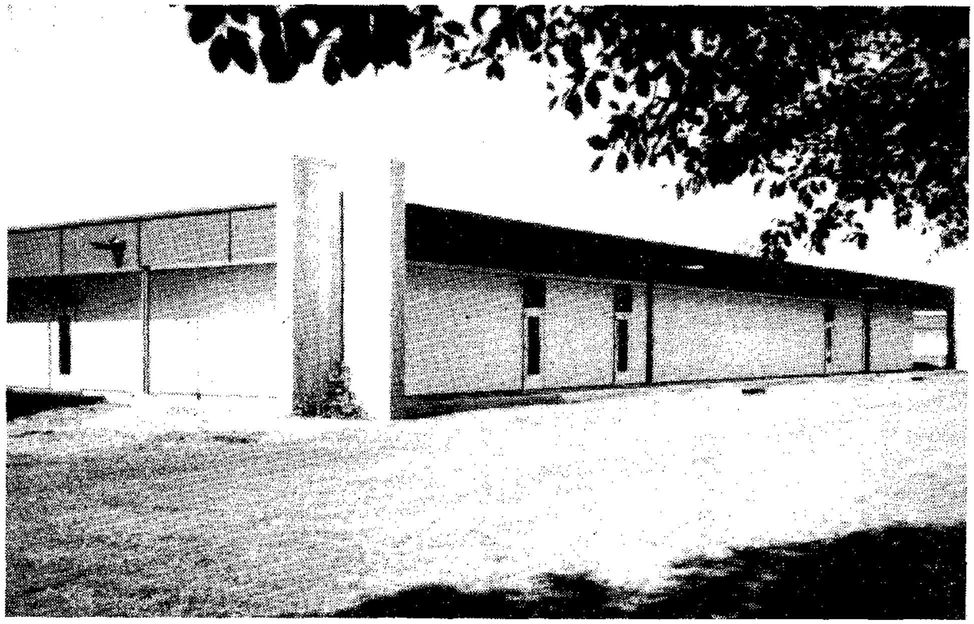


Figure XIV-22. External buttresses handle all lateral forces. Oak Grove High School, San Jose, California. Allan Walter, architect.

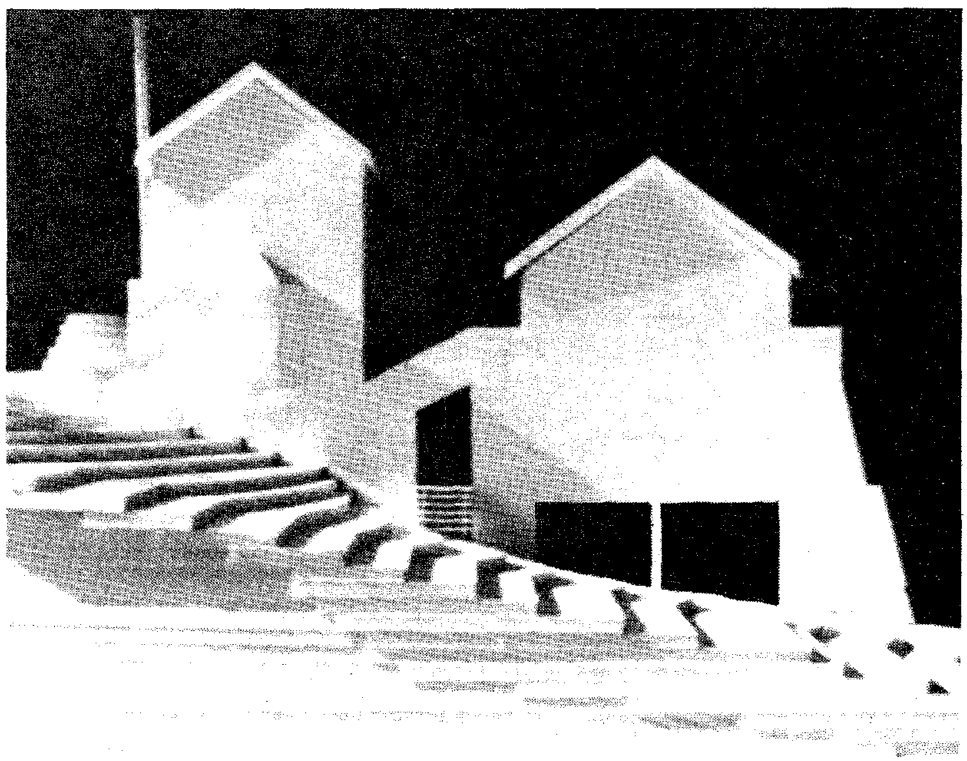


Figure XIV-23. External buttress provides lateral resistance at garage opening. Zelver House project, Santa Cruz, California. Chris Arnold, architect.

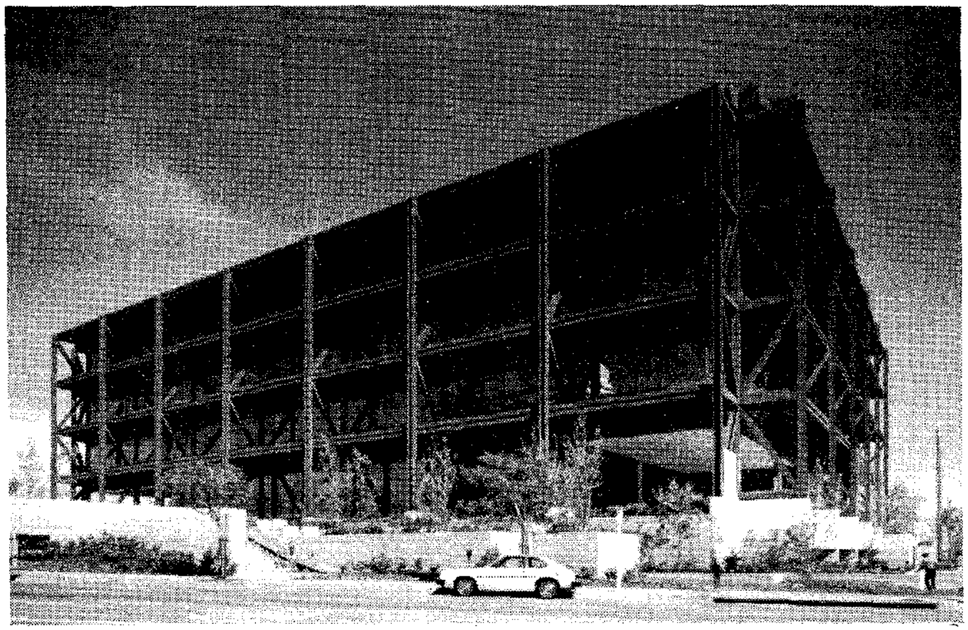


Figure XIV-24. Exposed seismic trusses, water-filled for fire protection. Los Angeles Harbor Department Administrative Office Facility, Los Angeles, California. John Carl Warnecke & Associates, architects.

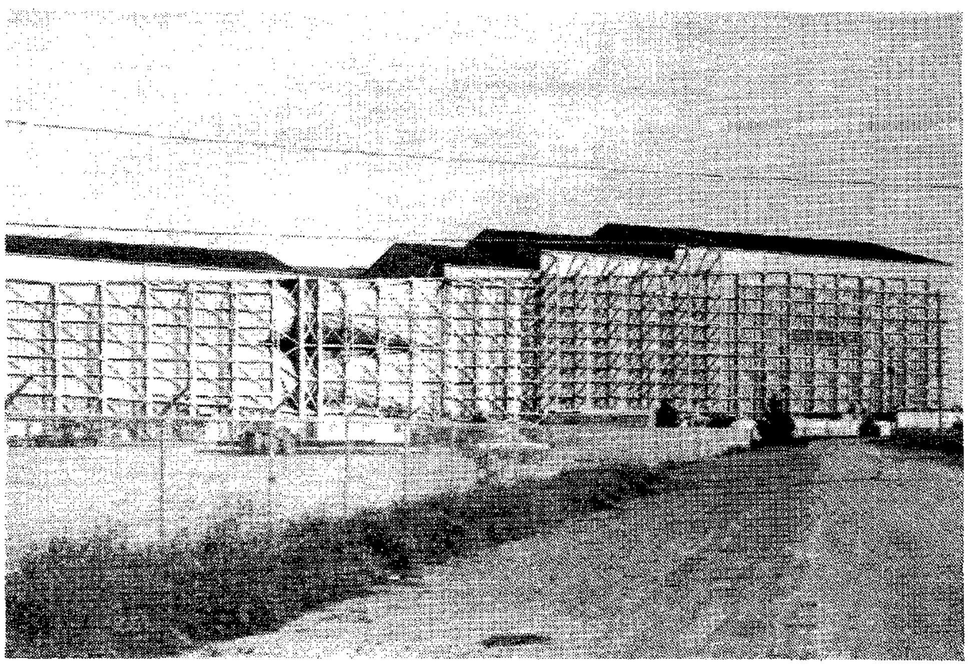


Figure XIV-25. Exposed seismic trusses. Wind Tunnel Structure, Ames Research Laboratory, NASA, Mountain View, California.

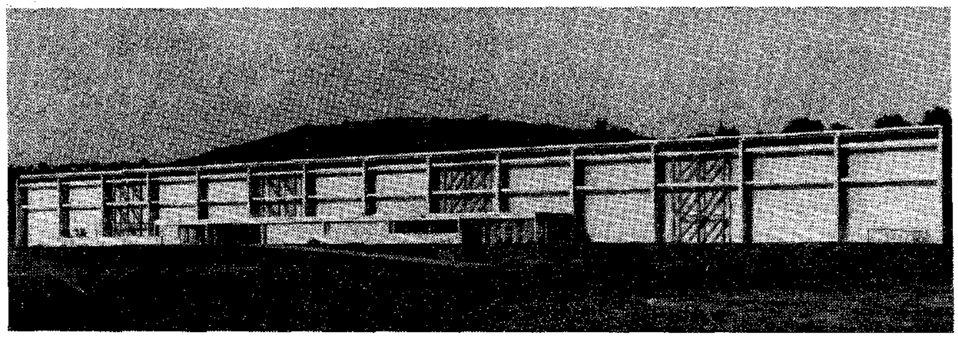


Figure XIV-26. Exterior seismic bracing, part of 1955 original design, on longitudinal wall of gymnasium. Hillsdale High School, San Mateo, California. Reid & Tarics Associates, architects.

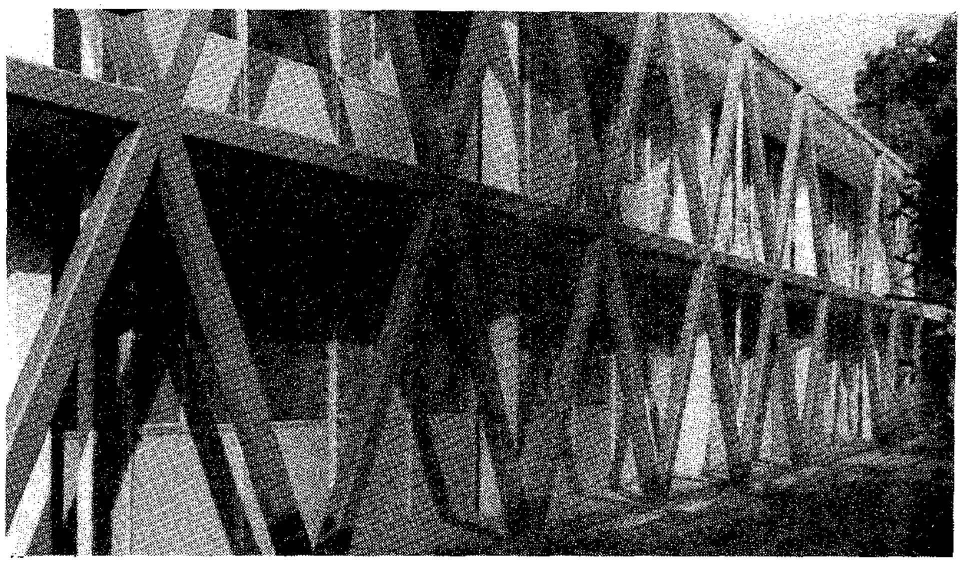


Figure XIV-27. Seismic rehabilitation of long, narrow, concrete office building: cross bracing provides longitudinal lateral resistance. United States Geological Survey, Menlo Park, California. Richard C. Marshall, architects. Forell/Elsesser Engineers, Inc.

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XV. Conclusion and Summary

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In addressing the architect this study has had three main objectives: to provide the interested designer with a basis for understanding the problems and nature of seismic design; to emphasize and make clear the role that architectural configuration plays in determining the seismic resistance of a building; and to identify and explain the main seismic problems created by characteristic configuration solutions and provide some conceptual methods by which these problems may be reduced or avoided. A graphic summary of these problems and solutions appears at the end of this chapter.

In its limited intentions this study has introduced a set of problems in a conceptual way: the aim is to open up an area of investigation, rather than close it down. The justification for this lies in the general agreement on the importance of the configuration issue, combined with the general lack of systematic analysis and exposition of its effects on seismic behavior and its relationship to other issues such as materials and construction quality.

We have stressed that seismic design is a shared architectural and engineering responsibility. It is shared in the physical relationships between architectural forms and structural resistant systems, and ideally an understanding of these relationships would be present in the mind of any designer working in a seismic area. Unfortunately our methods of education and practice have tended to diminish the opportunity for such understanding to become ingrained, as it should, in the designer's way of thinking, for we separate our architects and engineers in their education and, for the most part, in their practice. In fact, some architects, by intuition and thinking pattern, do have an excellent sense of structure, but they are rare, and such fortunate understanding tends to be in spite of education and practice rather than because of them.

Indeed, our conditions of practice are such that, for all but small structures, it is virtually impossible for the architect also to assume a structural design role. The combined role of architect and iconoclastic engineer that Frank Lloyd Wright played in the design of the Imperial Hotel would be unthinkable today: even if the building officials permitted it, liability issues might daunt even so bold a spirit as Wright. Similarly, a few engineers appear to have an excellent sense of the integration act that is the essence of architecture, but most engineers are content to practice their specialized trade, employed by the architect, with the role of advisor but seldom the authority to ensure that their advice is followed.

The interrelationships between issues of form and issues of seismic engineering demand that architect and engineer work together from the inception of a concept to ensure that these relationships are given full respect. The idea of the engineer participating in early design concepts is neither new nor controversial, yet it often does not happen. There are three main reasons. The first is due to the business climate in which the architect works, which often means that the economics of a job will tend towards reduction in early design costs until the future of the project is assured, by which time the important aspects of the design concept may be determined. The second reason, stemming perhaps from traditional design education - training for a role as 'master designer' - combined with a designer's temperament, is that the architect may feel that the engineer's advice is not necessary to help him with his conception. The third reason is that by virture of the engineer's training and his own tendencies to focus on specific material solutions rather than alternative general concepts, his advice may, in fact, be of little use to the architect.

So the issue is not only that of the two professionals coming together at design inception, but also that of their being able to communicate within a common conceptual framework using a shared language. This leads to a fourth major objective of this study; the attempt to sketch such a framework. One of our traditional problems of communication has been - and continues to be - that designers tend to think visually and express themselves in sketches that are almost a visual shorthand, whereas engineers like the precise but abstract language of mathematics, and their visual language is that of curves and algebraic formulae. Because these two languages are almost completely incompatible, this study, leaning towards the mode of the designers, reverts to the shared language of English, and the use of a variety of graphic and photographic devices which it is hoped will prove effective. It is not to be expected that this study will at once transform the level of understanding of whoever reads it: but at least a document now exists where previously there was only a void.

To this time the information from which the principles of configuration-based building behavior have been derived has been almost entirely empirical: knowledge has come from the observed behavior in earthquakes of buildings with specific formal characteristics. From the architect's viewpoint the analytical and experimental work of research engineers has been directed at structural abstractions that are grossly simplified models of the buildings that he designs. The kinds of formal irregularities discussed here are rarely part of sophisticated analysis. The need for simplicity in analytical models is understood: the number of variables that can be studied is limited and the design of real buildings seems too whimsical and haphazard a process to justify extensive analytical work on real configurations. And so the analysts bring back ever more precise data about the kind of simple building concepts that represent a minute percentage of the buildings that architects will design. In reducing design to the kind of abstract form that responds to analysis and from which general conclusions will be reached, the analysts eliminate almost all those characteristics that the architect uses to make a functioning building.

One objective for the future, then, is that analysis and experimentation begin to look at, and devise the techniques for, buildings as they are built rather than simplifying them to suit the current constraints of computer or shaking table. This involves the development of much more complex models, in which not only are common irregularities of configuration introduced, but also all those 'non-structural' elements that convert a building from a structural frame to a usable entity. To do this, the architect must be brought into the research arena to contribute his knowledge of the nature of buildings, and to enable a further and exciting interdisciplinary research effort to develop. The other objective must be that the research provides useful knowledge that can be translated into guidelines, suggestions and even codes and regulations that ensure design solutions that respect and balance the full range of architectural, engineering, and material influences on seismic hazard.

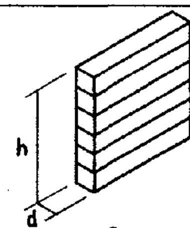
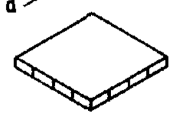
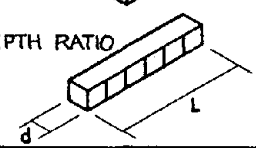
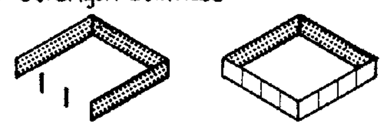
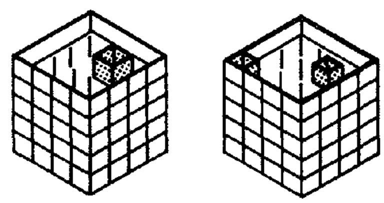
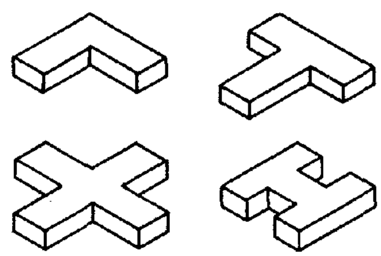
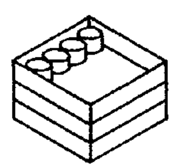
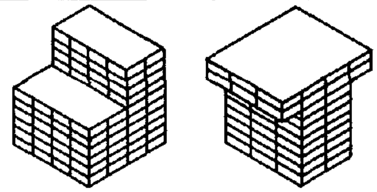
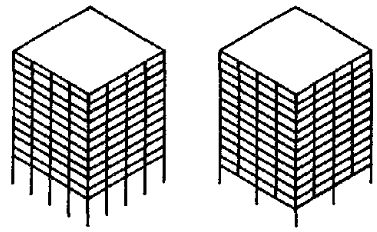
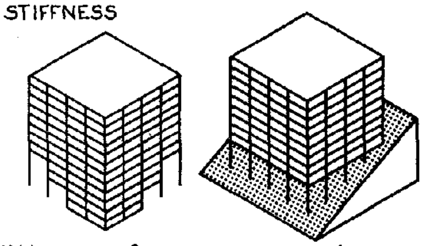
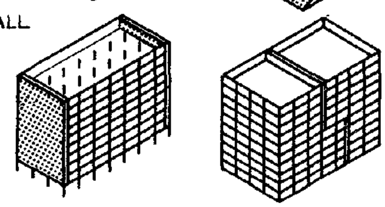
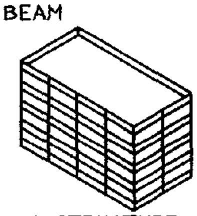
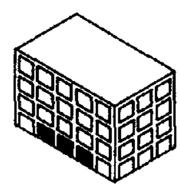
PROBLEM CONFIGURATION		
A. PROBLEMS IN EXTREME DIMENSIONS		
IV-C, IV-E	1. EXTREME HEIGHT-DEPTH RATIO	
IV-A, IV-D, XI-L, XI-M	2. EXTREME PLAN AREA	
IV-A, IV-D, IV-E	3. EXTREME ELEVATION LENGTH-DEPTH RATIO (ASPECT RATIO)	
B. PROBLEMS OF HORIZONTAL LAYOUT	1. SIMPLE PLAN CONFIGURATION	
V-A	a. variations in perimeter strength-stiffness	
IV-F, V-B	b. false symmetry	
VI	2. RE-ENTRANT CORNERS	
IV-F, XI-P	3. MASS ECCENTRICITIES	

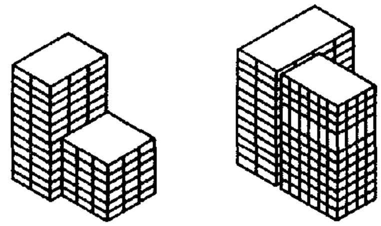
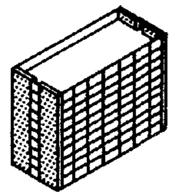
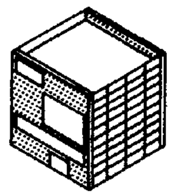
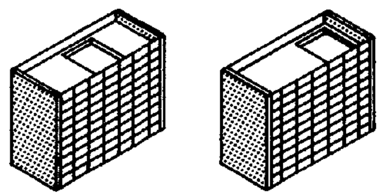
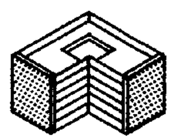
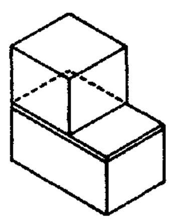
Figure XV-1. Graphic summary of configuration problems and solutions.

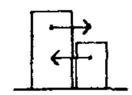
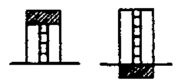
ARCHITECTURAL STATEMENT	STRUCTURAL PROBLEM STATEMENT	SOLUTION
function of planning or constricted site	high overturning forces, large drift causing non-structural damage	revise proportion or special structural system
common in warehouses, industrial buildings, shopping centers	build-up of large diaphragm forces	subdivide building by seismic joints
common in older schools, multi-story residential	build-up of large lateral forces in perimeter: big difference in resistance of two axes	subdivide building by seismic joints
often result of program: e.g. fire station, store front. need for blank walls on corner	torsion caused by extreme variation in strength and stiffness	add frames and disconnect walls, or use frames and lightweight walls
program requirements, relating vertical circulation to use spaces	torsion caused by stiff asymmetric core	disconnect core, or use frame with non-structural core walls
program requirements for narrow wings, e.g. residential, hospital, and tight urban site. common in older buildings, pre air-conditioning and fluorescent lighting	torsion, and stress concentration at the notches	separate walls  uniform box  center box  architectural relief 
programmatic requirements: book stacks in libraries, special equipment, elevated swimming pools	torsion, stress concentrations	reprogram, or add resistance around mass to balance resistance and mass

PROBLEM CONFIGURATION

C. PROBLEMS OF VERTICAL LAYOUT VII	1. VERTICAL SETBACKS & INVERTED SETBACKS 
VIII-A, VIII-B, VIII-C	2. SOFT STORY -frame 
VIII-E	3. VARIATIONS IN COLUMN STIFFNESS 
VII-B, VIII-D	4. DISCONTINUOUS SHEAR WALL 
VIII-F	5. WEAK COLUMN - STRONG BEAM 
VIII-E, VIII-H	6. MODIFICATIONS OF PRIMARY STRUCTURE 

ARCHITECTURAL STATEMENT	STRUCTURAL PROBLEM STATEMENT	SOLUTION
vertical setbacks result of program or site: inverted setback almost always an image requirement (fashion)	stress concentration at notch, different periods for different parts of building, high diaphragm forces to transfer at setback	special structural systems, careful dynamic analysis
programmatic: need for open first floor plazas or large spaces at any floor: often image requirement (fashion)	causes abrupt change of stiffness at point of discontinuity	add bracing  add columns  braced 
programmatic: need for variety of spaces and ceiling heights: often image requirements	causes abrupt change of stiffness, much higher forces in stiffer columns	redesign structural system to balance stiffnesses
result of program: planning restricts use of shear walls on entrance floor, or image "floating cube"	results in discontinuities in load path and stress concentration for most heavily loaded elements	NO
common in buildings with large window areas - schools, hospitals, offices - wide span beams, deep spandrels	column failure occurs before beam, short column must try and accommodate story height displacement	NO add full walls to reduce column forces, or detach spandrels from columns, or use light weight curtain wall with frame
programmatic: requirement for high window: common as remodel, sometimes by building management (may be interior condition also)	most serious when masonry in-fill modifies structural concept. creation of short, stiff columns results in stress concentration	detach in-fill, or use light-weight materials

PROBLEM	CONFIGURATION
D. PROBLEMS OF ADJACENCY VI-D, VII-B, IX	1. BUILDING SEPARATION 
E. SHEAR WALLS VIII-G	1. COUPLED 
VIII-G	2. RANDOM OPENINGS 
F. DIAPHRAGM III-H	1. OPENINGS 
III-H, VI-C	2. SHAPE 
VII-B	3. TOWER 

ARCHITECTURAL STATEMENT	STRUCTURAL PROBLEM STATEMENT	SOLUTION
may be different parts of same building (setback) or buildings on adjacent sites	possibility of pounding dependent on building period, height, drift, distance	ensure adequate separation, assuming opposing building vibration 
common expression for end of double-loaded corridor plan	incompatible deformation between walls and links	$\boxed{\text{NO}}$ with weak link. design adequate link  or repairable system
requirement for windows, doors, holes for ducts	seriously degrade capacity at point of maximum force transfer	careful design, adequate space for reinforcing design for non-linear behavior
need for vertical circulation, light wells, skylights	seriously degrade diaphragm capacity	$\boxed{\text{NO}}$ unless careful design for non-linear behavior
planning almost always requires vertical circulation at 'hinge' of re-entrant corner plans	weakens diaphragm at most critical location	$\boxed{\text{NO}}$ unless careful design for non-linear behavior
see setbacks	diaphragm at setback must transfer full tower loads	careful design, recognizing diaphragm problem

References

An earlier version of the problems and solutions chart appeared in Christopher Arnold and Eric Elsesser, "Building Configuration: Problems and Solutions," Proceedings of the Seventh World Conference on Earthquake Engineering, (Istanbul, Turkey: 1980), Volume 4, pp. 153-160.

Appendices

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A1. Configuration Definition

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This appendix forms a grammar of configuration. It is extremely selective in that the classification is judgmentally based on a combination of pure geometry, seismic significance, and building use, i.e. configurations which because of their usefulness constantly reoccur in practice. Through the combination of these three characteristics it becomes possible to reduce the display of a mathematically infinite number of configurations to a fairly small quantity, and to establish a basis for definition without the necessity for illustrating every basic shape and its variations. If desired, it would be easy to move from these definitions into appropriate forms of computer generation.

The basis for this classification system is provided by the geometrical concepts of convexity and concavity, Figure 1. By using these concepts in relation to building plan and building elevation (or section), a useful distinction is at once made between buildings of simple shape, and those of complex shape involving re-entrant corners or curves, in both plan and elevation.

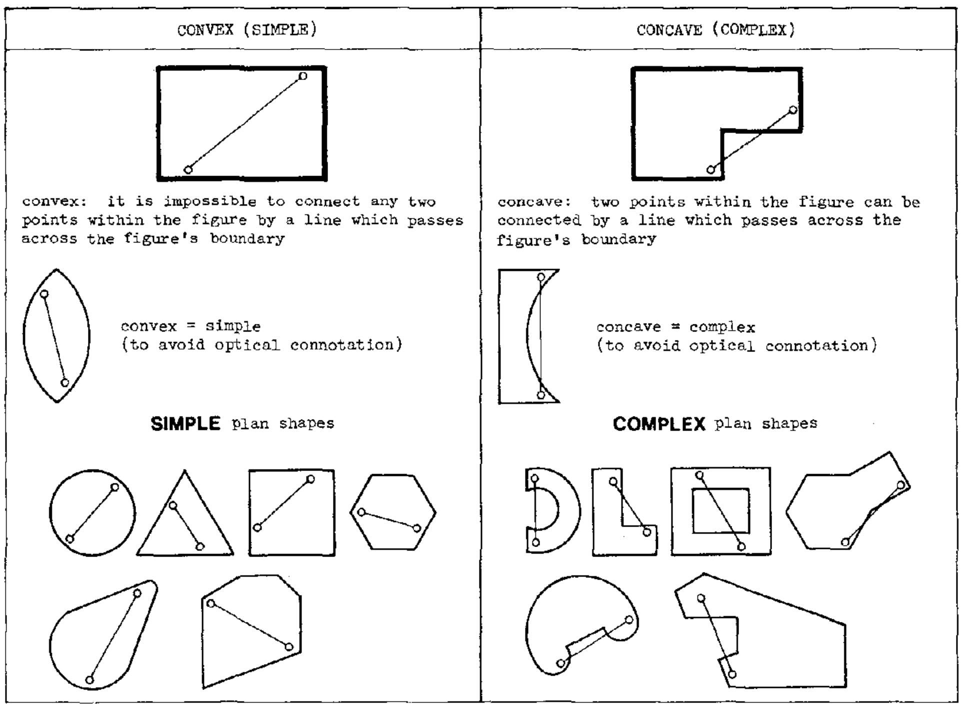

Figure 1. The concept of simple and complex.

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Figure 2 shows examples of shapes defined separately as simple and complex in plan, and simple and complex in elevation. The shapes shown have been selected as representative of those common in building design: in this diagram the geometrical shapes at once become recognizable as buildings.

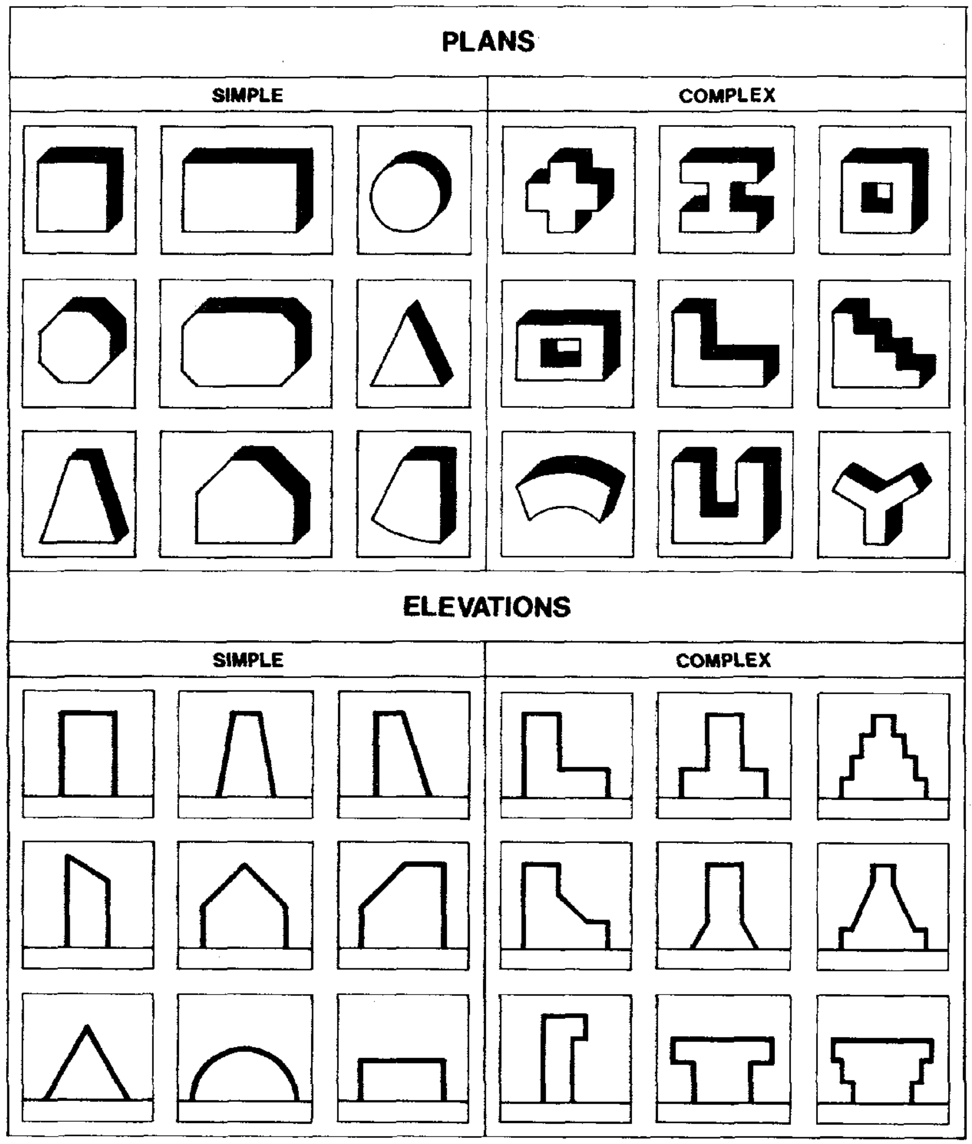


Figure 2. Simple and complex shapes in plan and elevation.

Moving from two-dimensions, into three, Figure 3 shows how these two pairs of characteristics are combined in a matrix that defines the four basic categories of building shapes. All building configurations can be related back to this matrix.

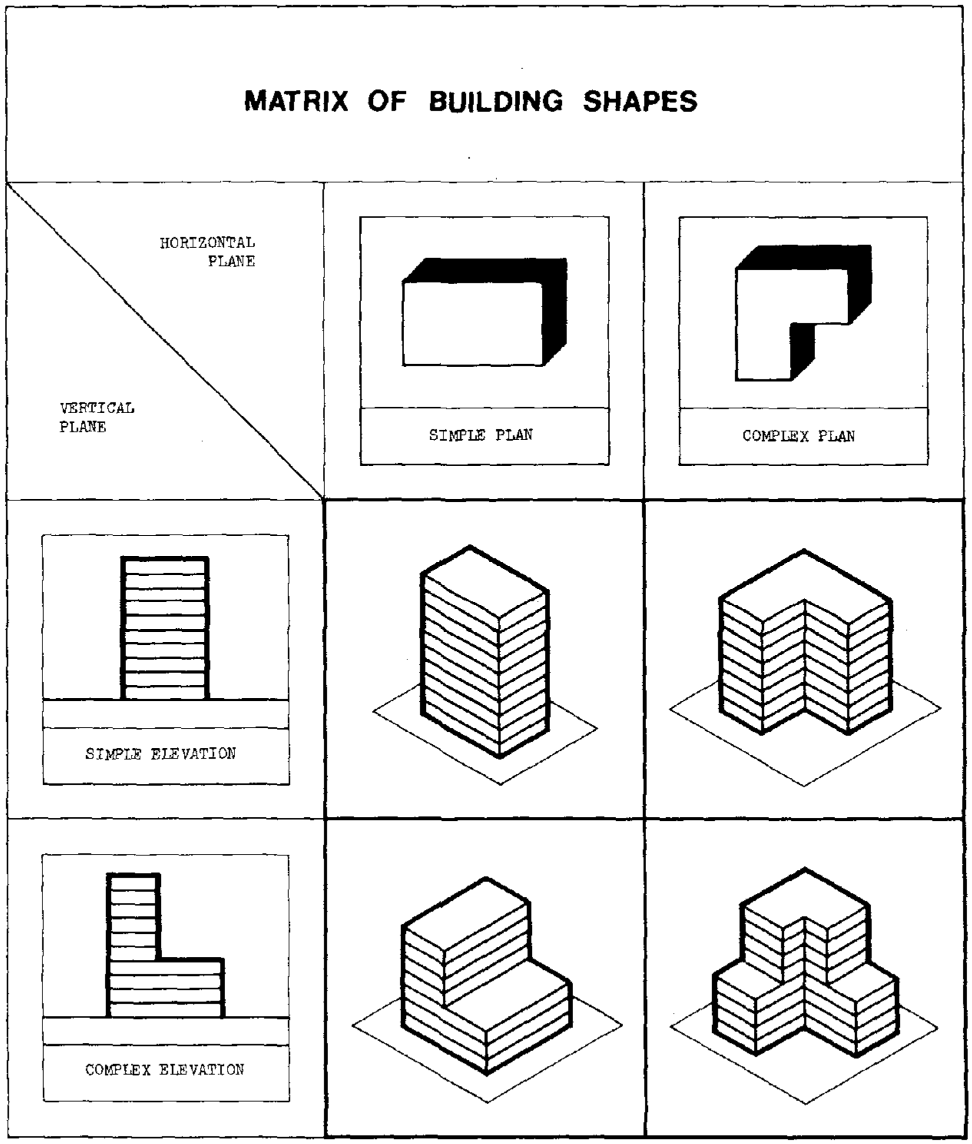


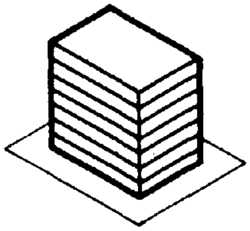
Figure 3. The matrix composed of the four basic building shapes.

In the layouts that follow, first the four basic categories, or families, of building shapes are described. This is done by photo examples, defining the basic dimensions, and showing dimensional variations for each family. No attempt is made to show every conceivable shape and variation: it is only necessary to establish a basis upon which shapes can be clearly defined for comparative purposes (Figures 4 - 15).

After the four basic shapes are defined, the characteristic of the splay is described that applies to simple and complex shapes in plan and elevation. In addition, the way in which the splay becomes, progressively, a shape composed of a large number of small steps, leading ultimately back to the pure L-shape in plan or elevation, is shown. The significance of this concept lies in the fact that shapes with the appearance of steps may be achieved with a splayed structure, and so a stepped geometry does not necessarily mean a re-entrant corner structure in plan or a setback structure in elevation (Figure 16).

Finally, three components of configuration are defined. These are chosen because they are of significance in seismic design and are also the result of very early decision making in the schematic design of the building. First, the nature of the perimeter design is defined in terms of openness, and uniformity. Second, the nature of interior space division is defined in terms of intensity and adaptability. Third, the important element of the core is defined. For each of these the general significance of each component and its characteristics are outlined in the graphic display (Figures 17-20).

The photographs shown in this section are intended only as examples of categorization of buildings and no comment is implied as to their possible seismic performance.



SIMPLE PLAN / SIMPLE ELEVATION

The family of forms which are 3-dimensionally convex in plan and elevation.

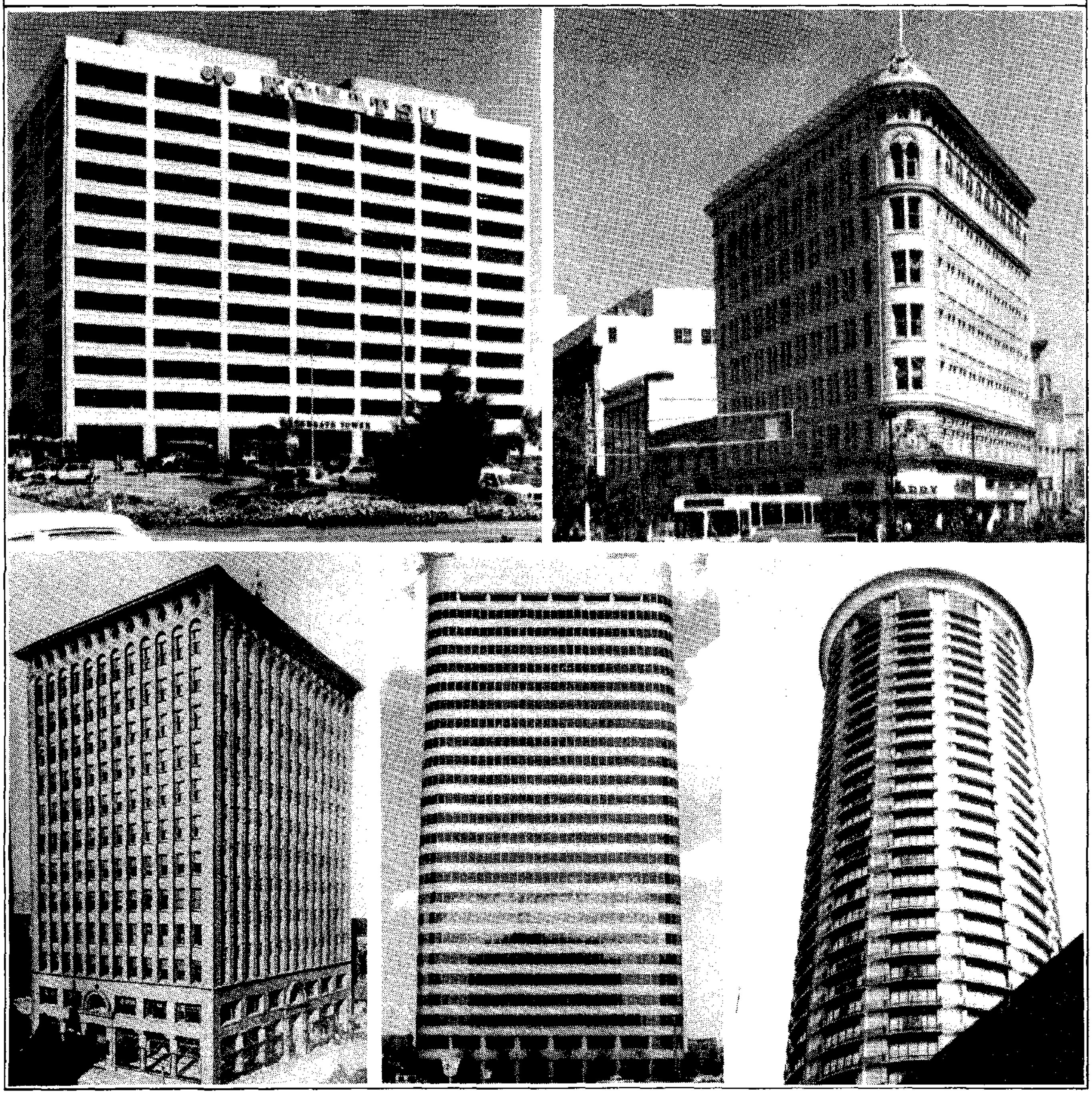


Figure 4. Examples of simple plan/simple elevation buildings.

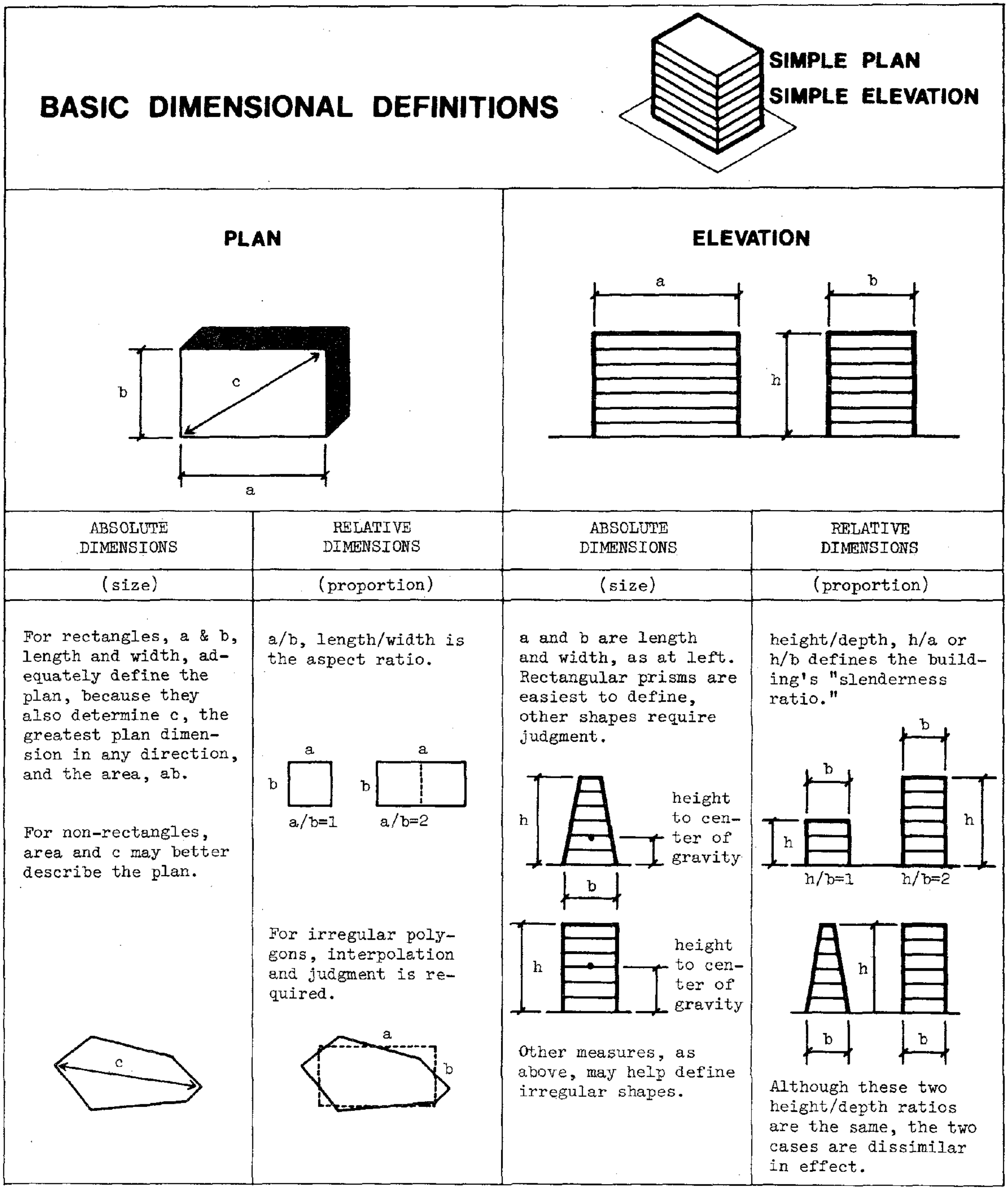


Figure 5. Simple plan/simple elevation: basic dimensional definitions.

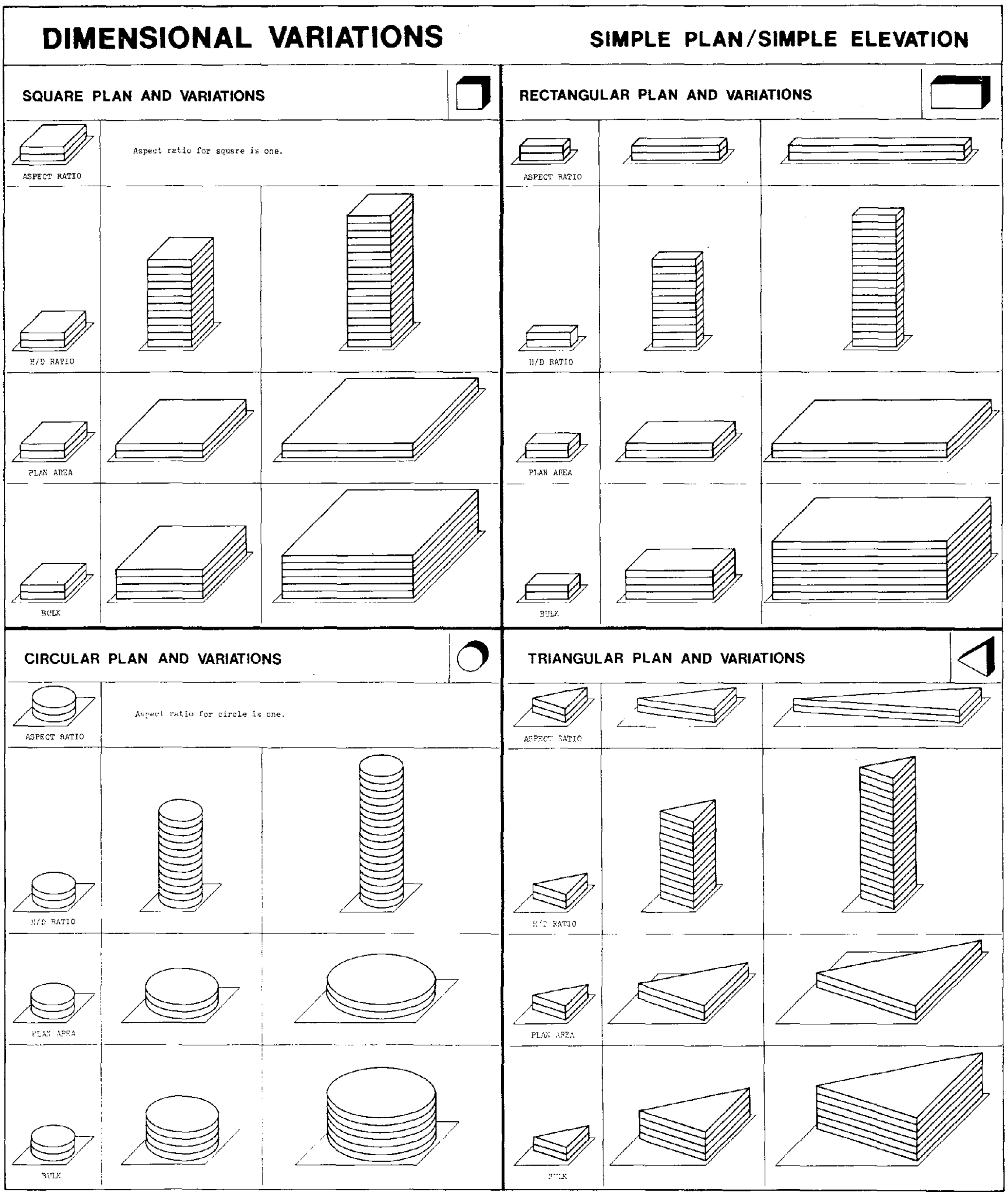
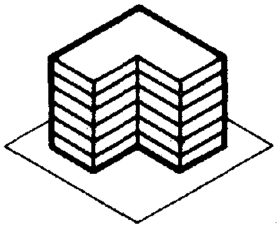


Figure 6. Simple plan/simple elevation: dimensional variations.



COMPLEX PLAN / SIMPLE ELEVATION

The family of forms which are 3-dimensionally concave in plan, but convex in elevation.

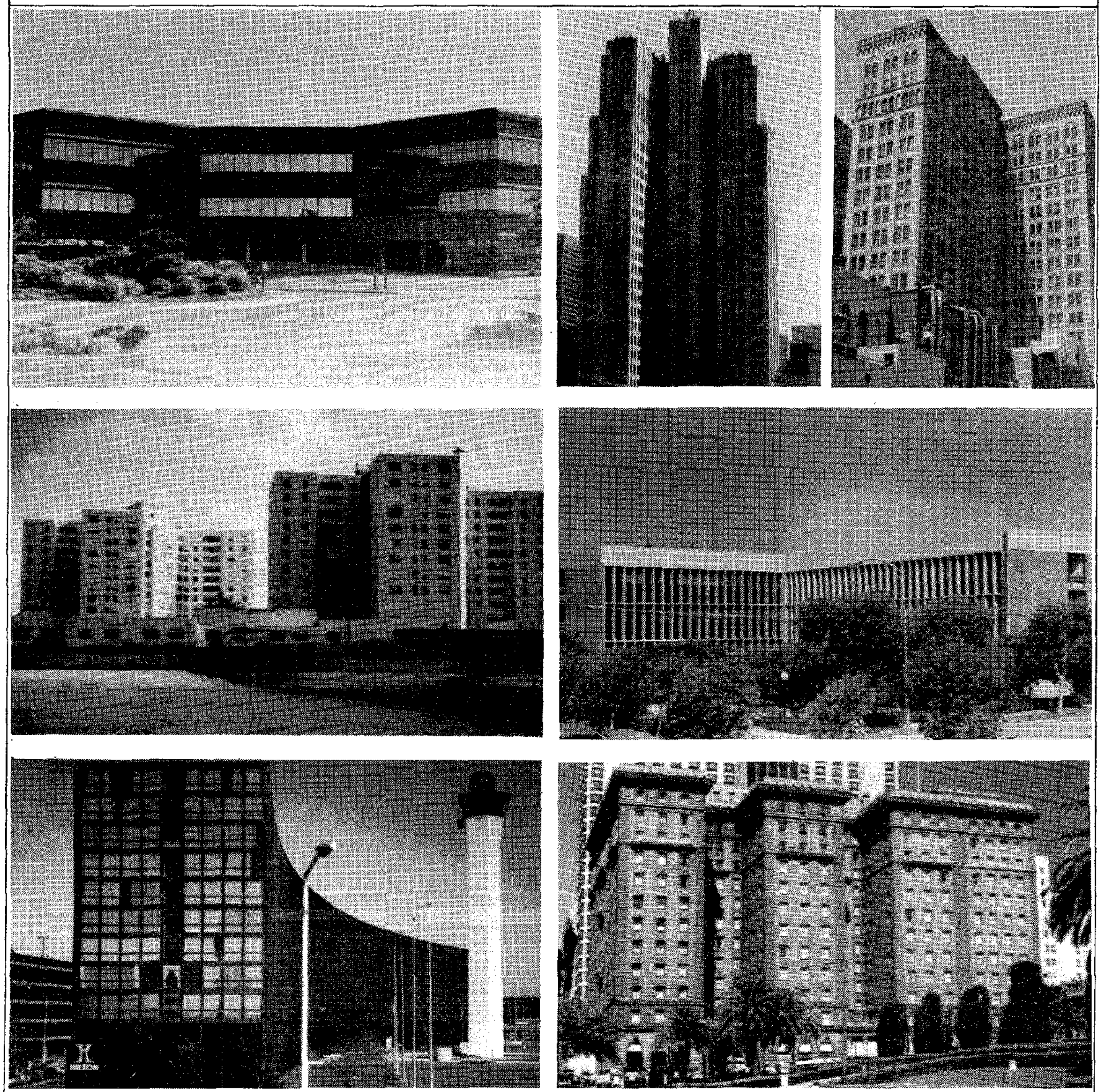
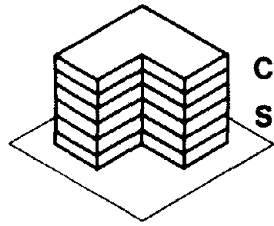


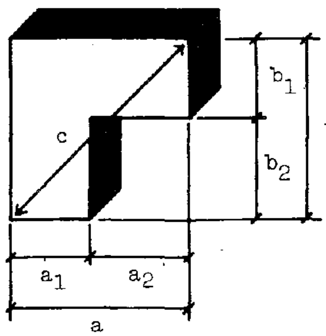
Figure 7. Examples of complex plan/simple elevation buildings.

BASIC DIMENSIONAL DEFINITIONS

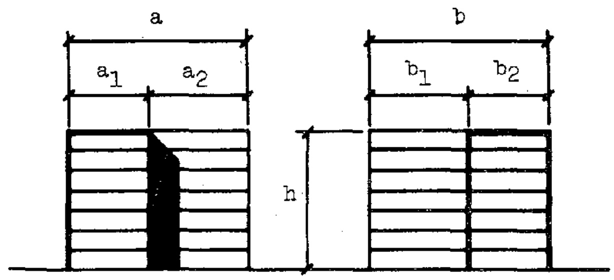


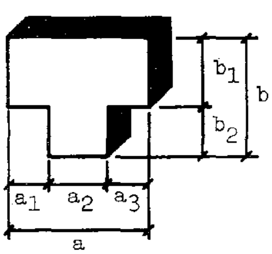
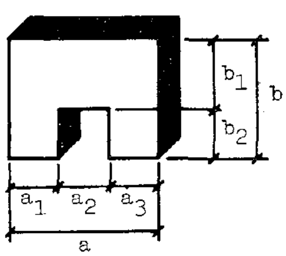
COMPLEX PLAN
SIMPLE ELEVATION

PLAN



ELEVATION



ABSOLUTE DIMENSIONS	RELATIVE DIMENSIONS	ABSOLUTE DIMENSIONS	RELATIVE DIMENSIONS
(size)	(proportion)	(size)	(proportion)
a_2 & b_2 define the absolute dimensions of the extent of the legs. c is the maximum distance between any two points in plan. 	a_1/b & b_1/a define aspect ratios of component rectangles. a_1/a & b_1/b define the legs proportionally; their values range from zero to one. 	h is overall height. Other height dimensions may be useful as well: see simple plan/simple elevation case.	h/a_1 & h/b_1 are the height/depth or h/d ratios for the component rectangles forming the legs. h/a & h/b are the h/d ratios for the overall form. h/a_2 & h/b_2 are the h/d ratios for the projecting portions of the legs.

More complex plans defined by extension of above definitions.

Non-rectilinear components defined as for simple plan case. See also "splays" discussed on a following page.

Figure 8. Complex plan/simple elevation: basic dimensional definitions.

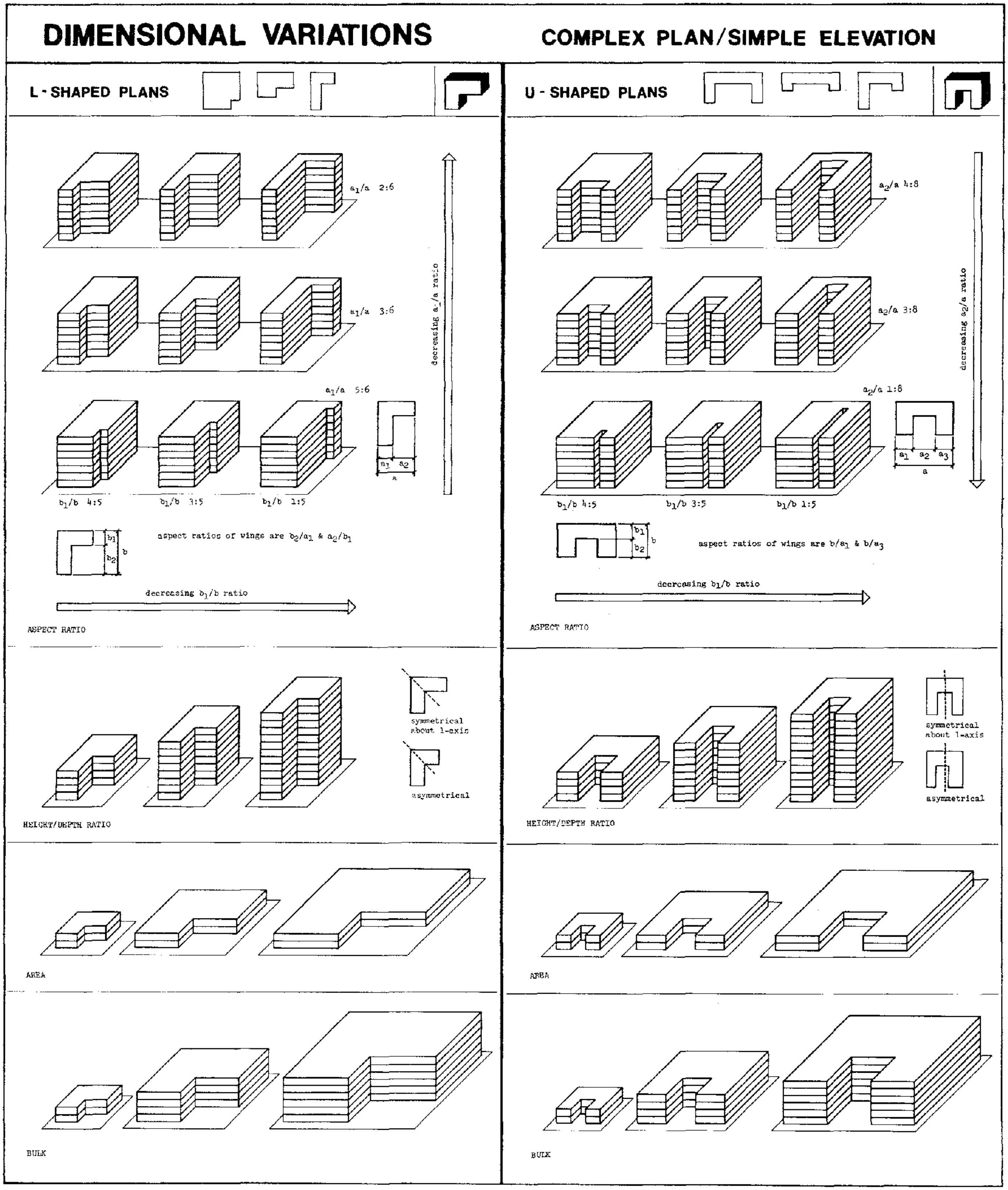
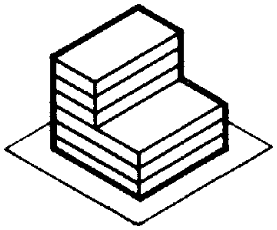


Figure 9. Complex plan/simple elevation: dimensional variations.



SIMPLE PLAN/COMPLEX ELEVATION

The family of forms which are 3-dimensionally convex in plan, but concave in elevation.

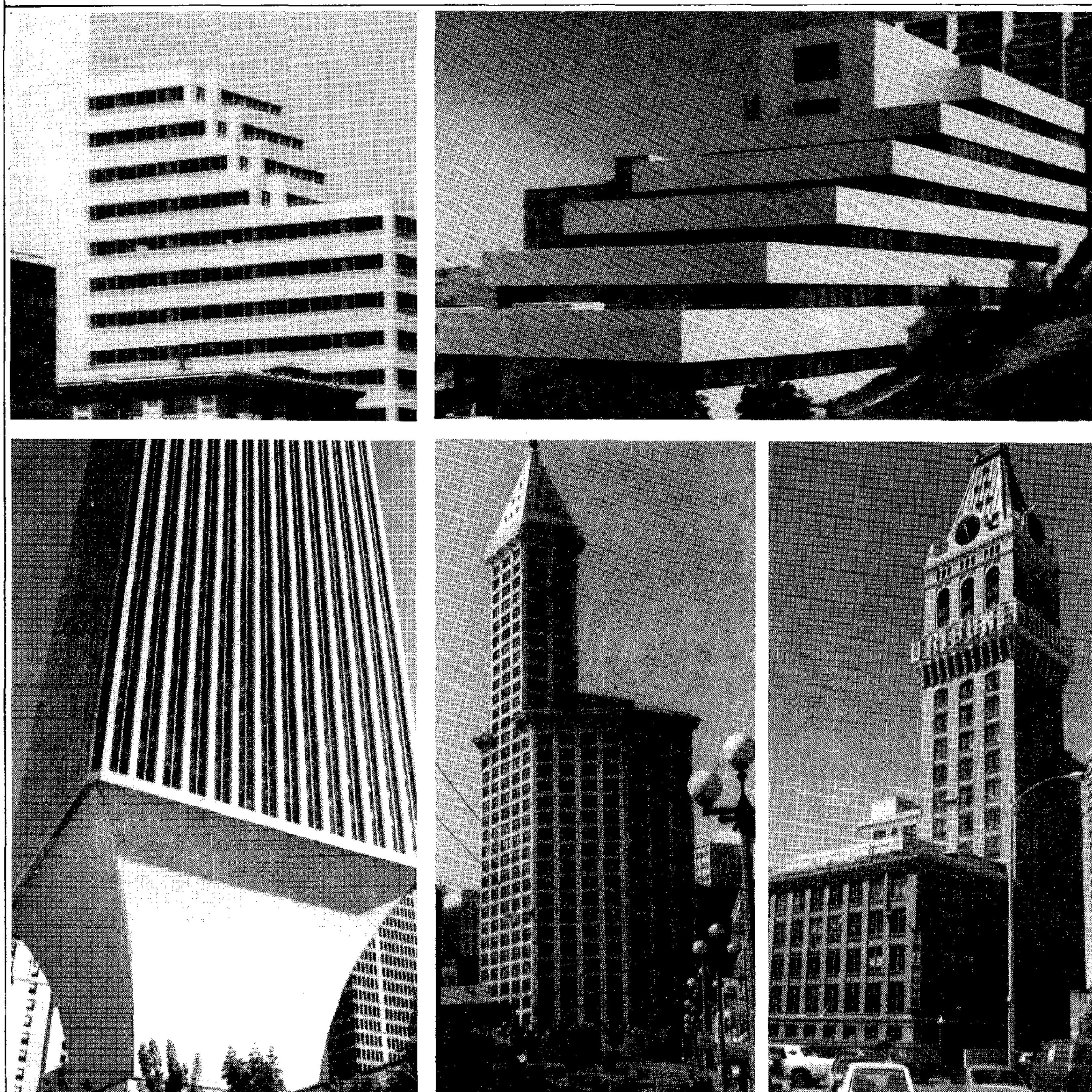
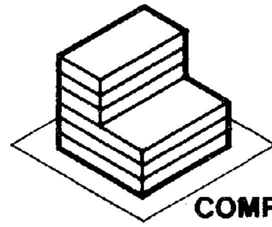


Figure 10. Examples of simple plan/complex elevation buildings.

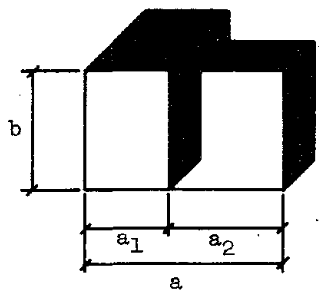
BASIC DIMENSIONAL DEFINITIONS



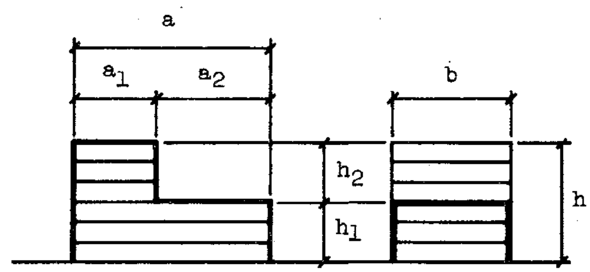
SIMPLE PLAN
COMPLEX ELEVATION

PLAN		ELEVATION	

PLAN



ELEVATION



ABSOLUTE DIMENSIONS	RELATIVE DIMENSIONS	ABSOLUTE DIMENSIONS	RELATIVE DIMENSIONS
(size)	(proportion)	(size)	(proportion)
The plan is simple, therefore it is defined as for the simple plan/simple elevation case, except that since the plan form varies at different levels, the process is applied to two or more plans.	As for the simple plan/simple elevation case, the overall aspect ratio is a/b. b/a_1 is the tower aspect ratio. a_1/a assesses the relative amount of setback.	h_2 is introduced to define tower height, h_1, the base height. Similar extensions of the definitions can be made as for the simple plan/simple elevation case.	h/a & h/b define the overall H/D ratios. h_2/a_1 & h_2/b define H/D ratios for tower only and h_1/a & h_1/b for base only. h_2/h indicates relative vertical amount of setback.

Figure 11. Simple plan/complex elevation: basic dimensional definitions.

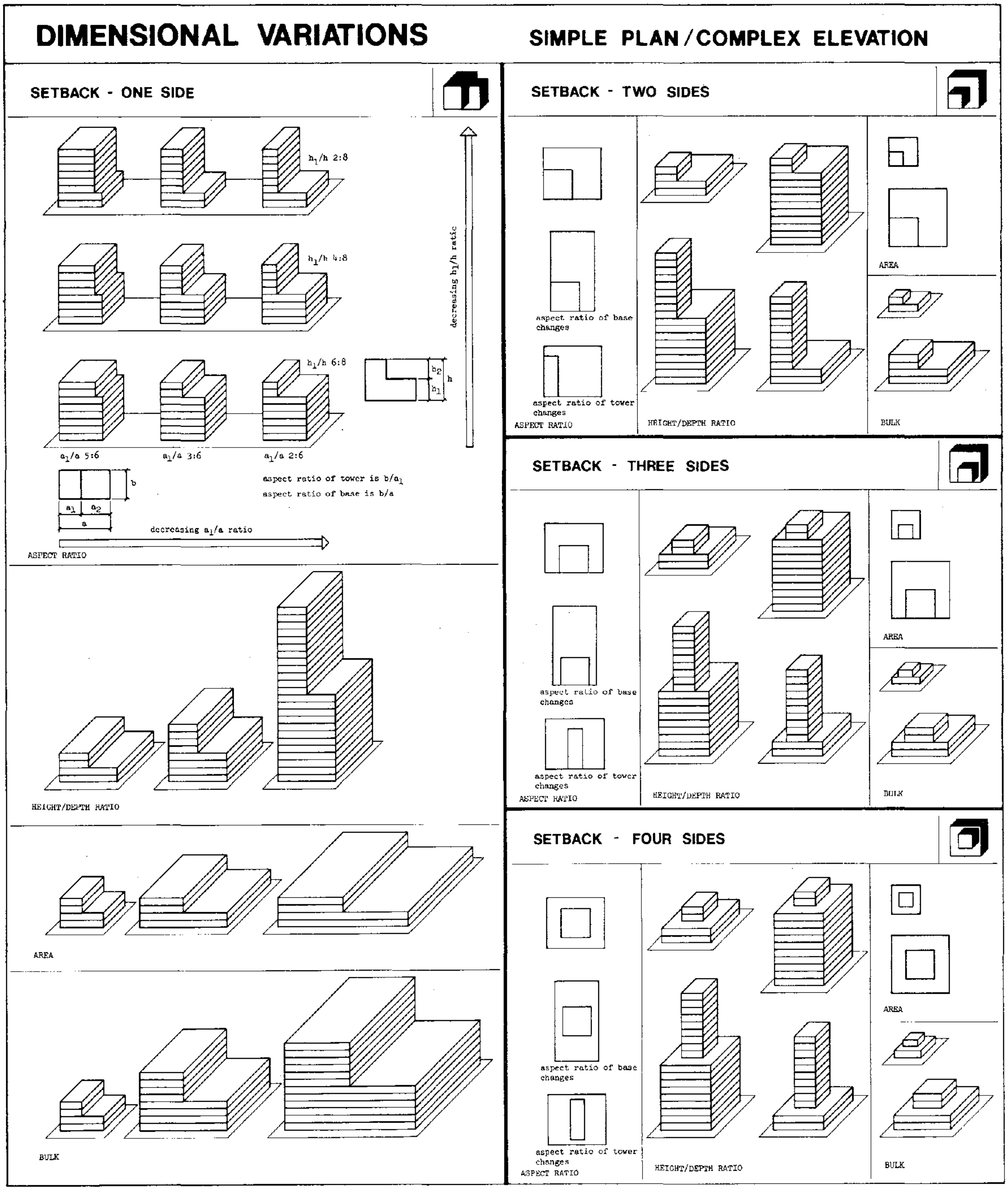
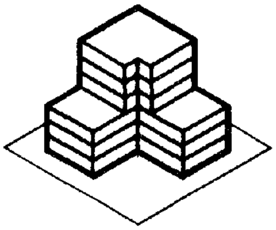


Figure 12. Simple plan/complex elevation: dimensional variations.



COMPLEX PLAN/COMPLEX ELEVATION

The family of forms which are 3-dimensionally concave in plan and elevation.

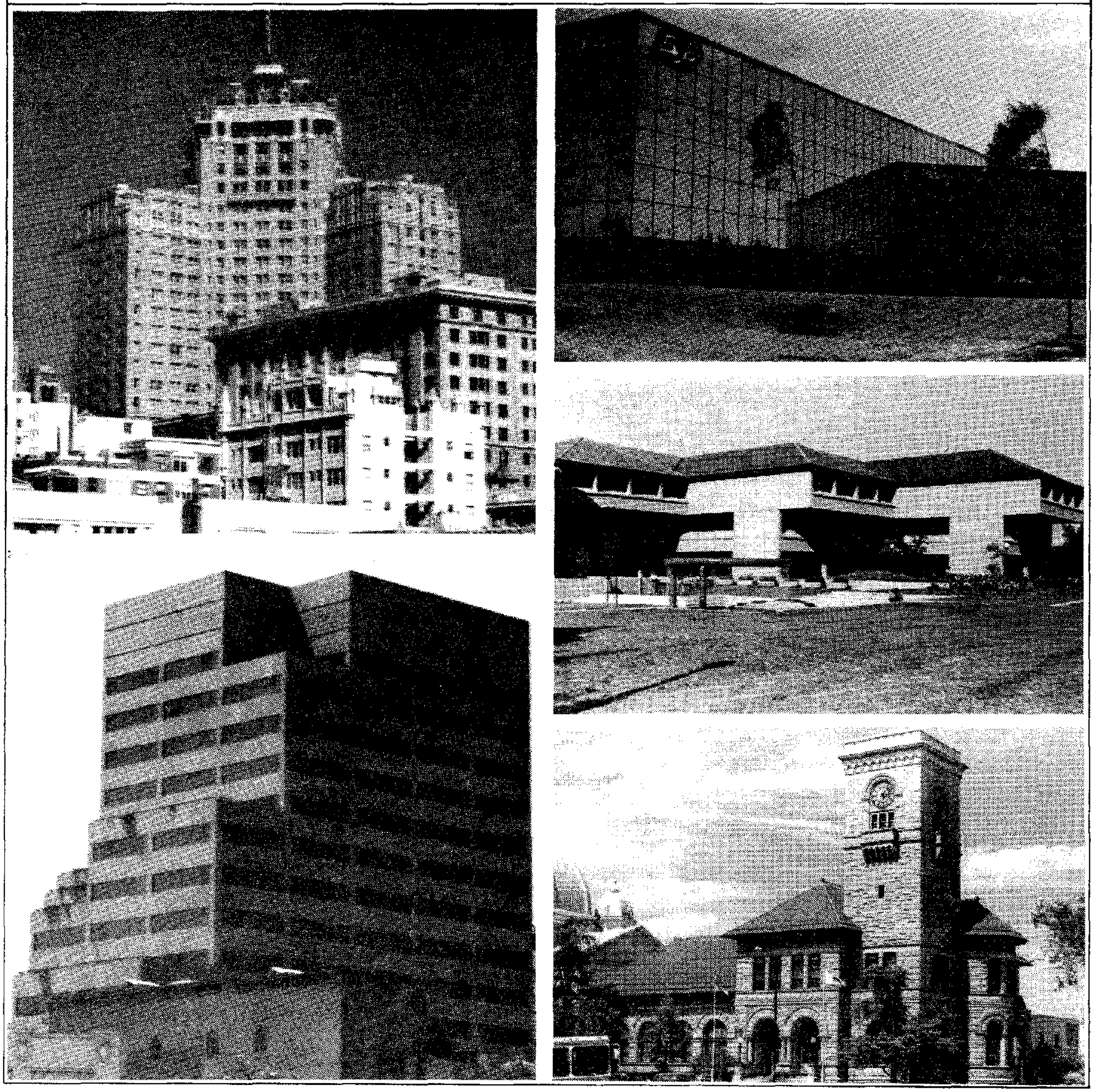
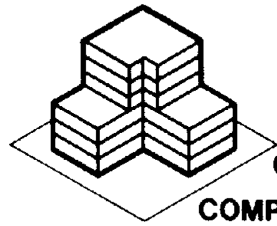


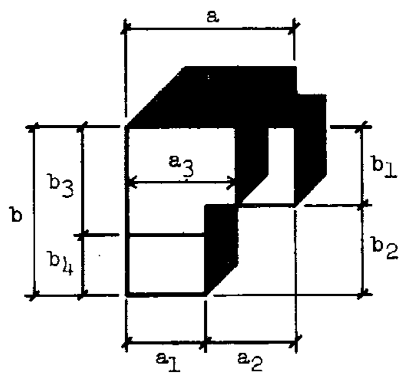
Figure 13. Examples of complex plan/complex elevation buildings.

BASIC DIMENSIONAL DEFINITIONS

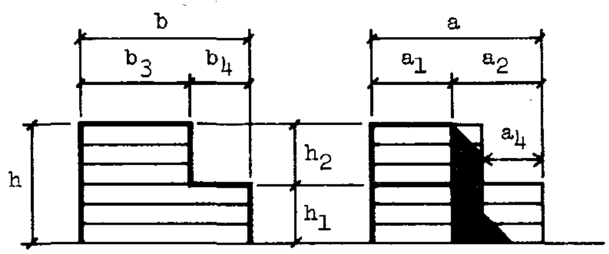


COMPLEX PLAN
COMPLEX ELEVATION

PLAN



ELEVATION



ABSOLUTE DIMENSIONS	RELATIVE DIMENSIONS	ABSOLUTE DIMENSIONS	RELATIVE DIMENSIONS
(size)	(proportion)	(size)	(proportion)

These definitions are the same as for the other complex plan case (complex plan but simple elevation---the L-shaped plan family).

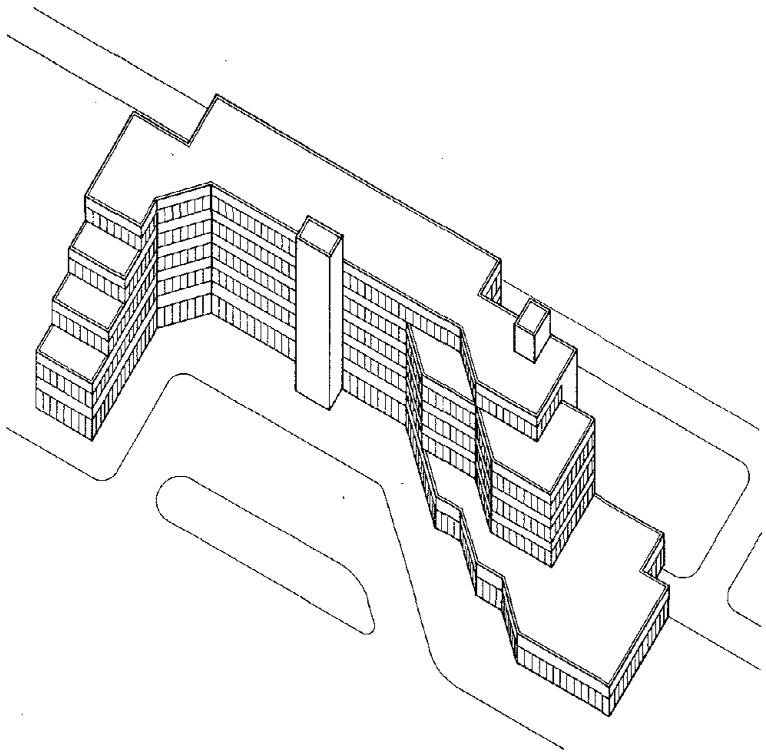
With the addition of a setback condition to the complex plan, it is necessary to define the plan at every level where it changes form.

These definitions are the same as for the other complex elevation case (complex elevation but simple plan--- the setback family).

With the addition of complexity in plan to complexity in elevation, it may become necessary to define the elevations along various axes (h, h_1, & h_2 may not be the same on all wings).

Figure 14. Complex plan/complex elevation: basic dimensional definitions.

VARIATIONS AND COMBINATIONS COMPLEX PLAN/COMPLEX ELEVATION



An infinite variety of forms are possible, but they are definable by a sequential application of the variables previously defined.

These variables are summarized below, as they relate to the illustration at left.

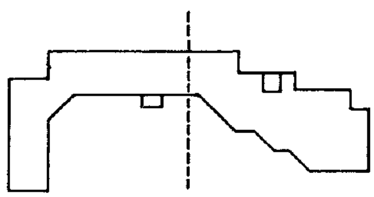
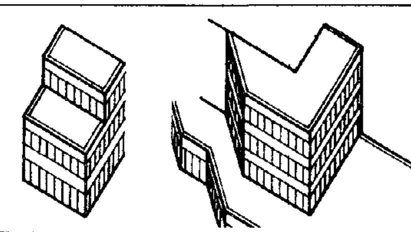
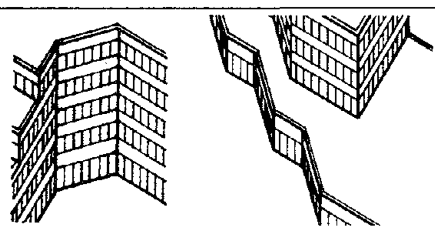
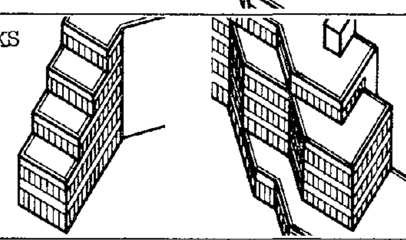
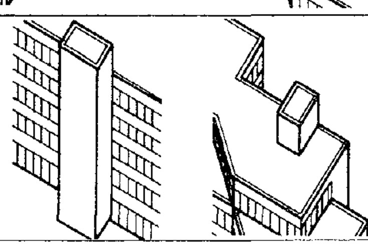
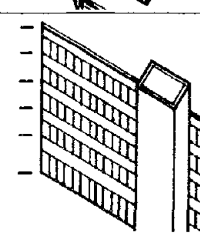
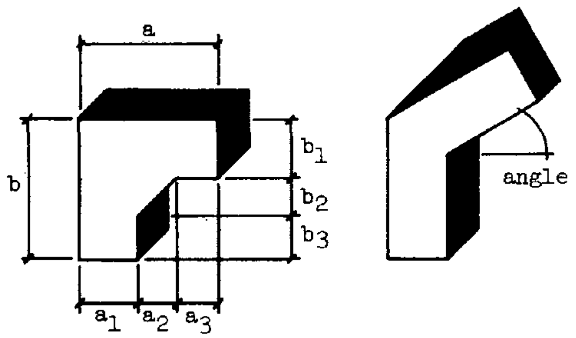
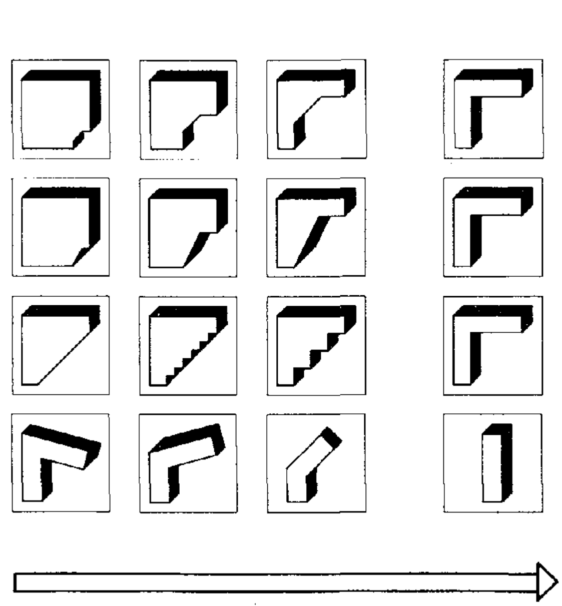
PLAN		ELEVATION	
RE-ENTRANT CORNER - U-shape (1) - asymmetrical (2)		SETBACK - 1-sided (3) - 3-sided (4)	
SPLAY - single (5) - multiple (6)		MULTIPLE SETBACKS - 1-sided (7) - 3-sided (8)	
CORE - internal (9) - external (10)		PERIMETER DESIGN - high percentage open (11) - uniform openings (12) - high first floor (13)	

Figure 15. Complex plan/complex elevation: variations and combinations.

DIMENSIONAL VARIATIONS APPLICABLE TO SIMPLE AND COMPLEX SHAPES

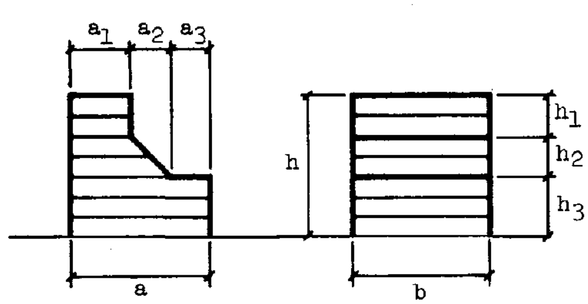
SPLAYS IN PLAN

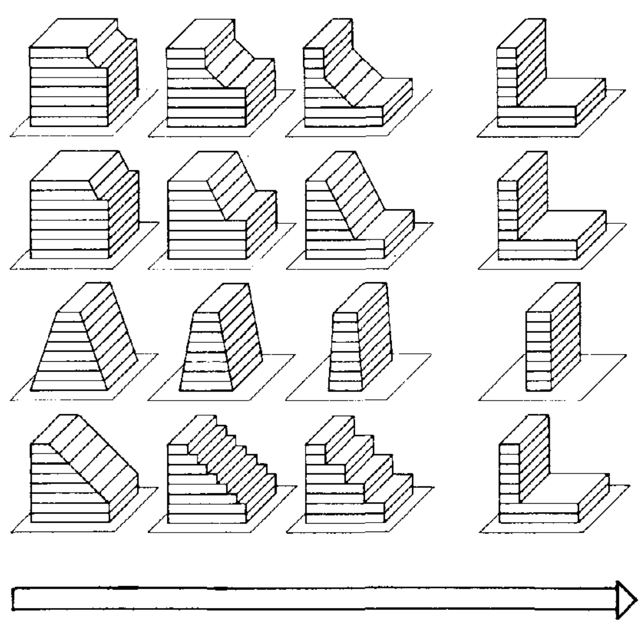




Various splays in plan are shown progressing from splay, back to one of the four basic families of building shapes.

SPLAYS IN ELEVATION





Various splays in elevation are shown progressing from splay, back to one of the four basic families of building shapes.

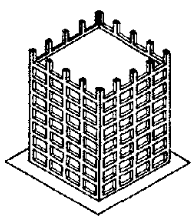
Figure 16. Dimensional variations applicable to simple and complex shapes; in plan or elevation, the splay.

CONFIGURATION COMPONENTS

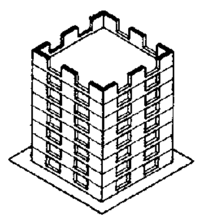
PERIMETER DESIGN REQUIREMENTS

PERCENTAGE OPEN

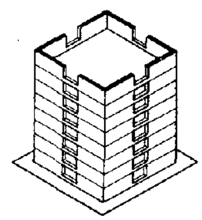
Defines the extent to which open and opaque exterior elements are uniformly distributed between building facades or floors.



High



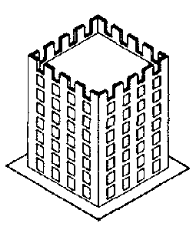
Medium



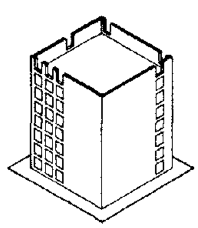
Low

UNIFORMITY

Defines the extent to which perimeter must be open for daylighting, view, or other purposes.



Uniform



Non-uniform

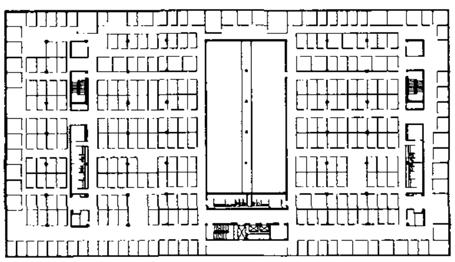
Figure 17. Configuration components: perimeter design requirements.

CONFIGURATION COMPONENTS

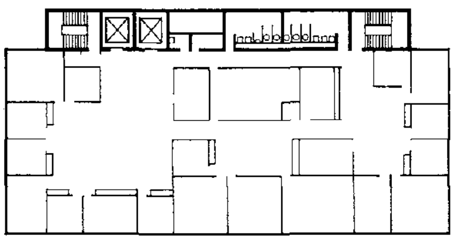
INTERIOR SPACE DIVISION

ITENSITY

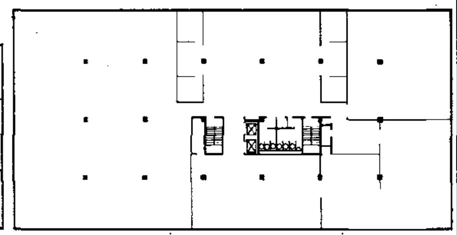
Quantity of interior walls and partitions, expressed as a ratio of linear footage of partition to plan area.



High



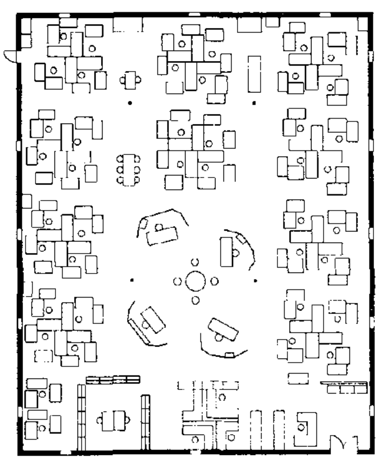
Medium



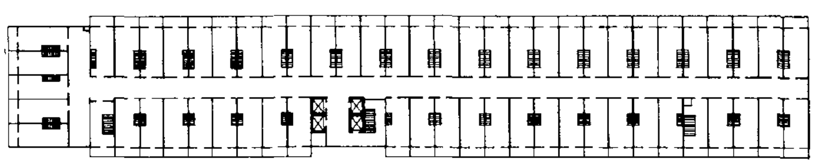
Low

ADAPTABILITY

Defines the extent to which interior space division elements are permanent.



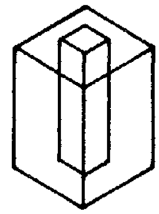
Low

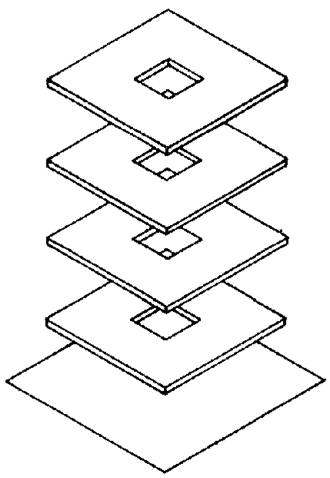


High

Figure 18. Configuration components: interior space division.

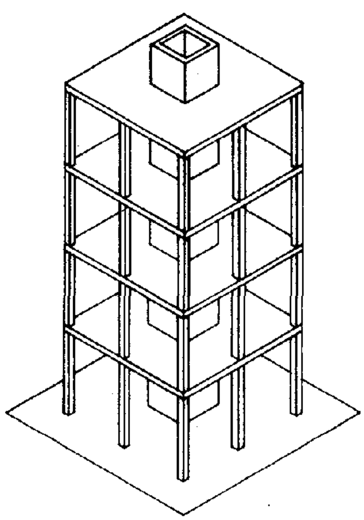
CONFIGURATION COMPONENTS





CORE -- A series of holes in the floors which are in line vertically to enable vertical mechanical services, elevators, or stairs to penetrate the building.

A core may or may not be enclosed with structural material to resist vertical and/or lateral forces.



Diagramatically, a core is represented as a single tube, but typically it is a more complex form.

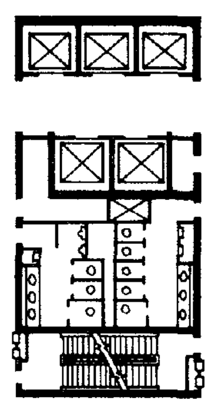


Figure 19. Configuration components: cores.

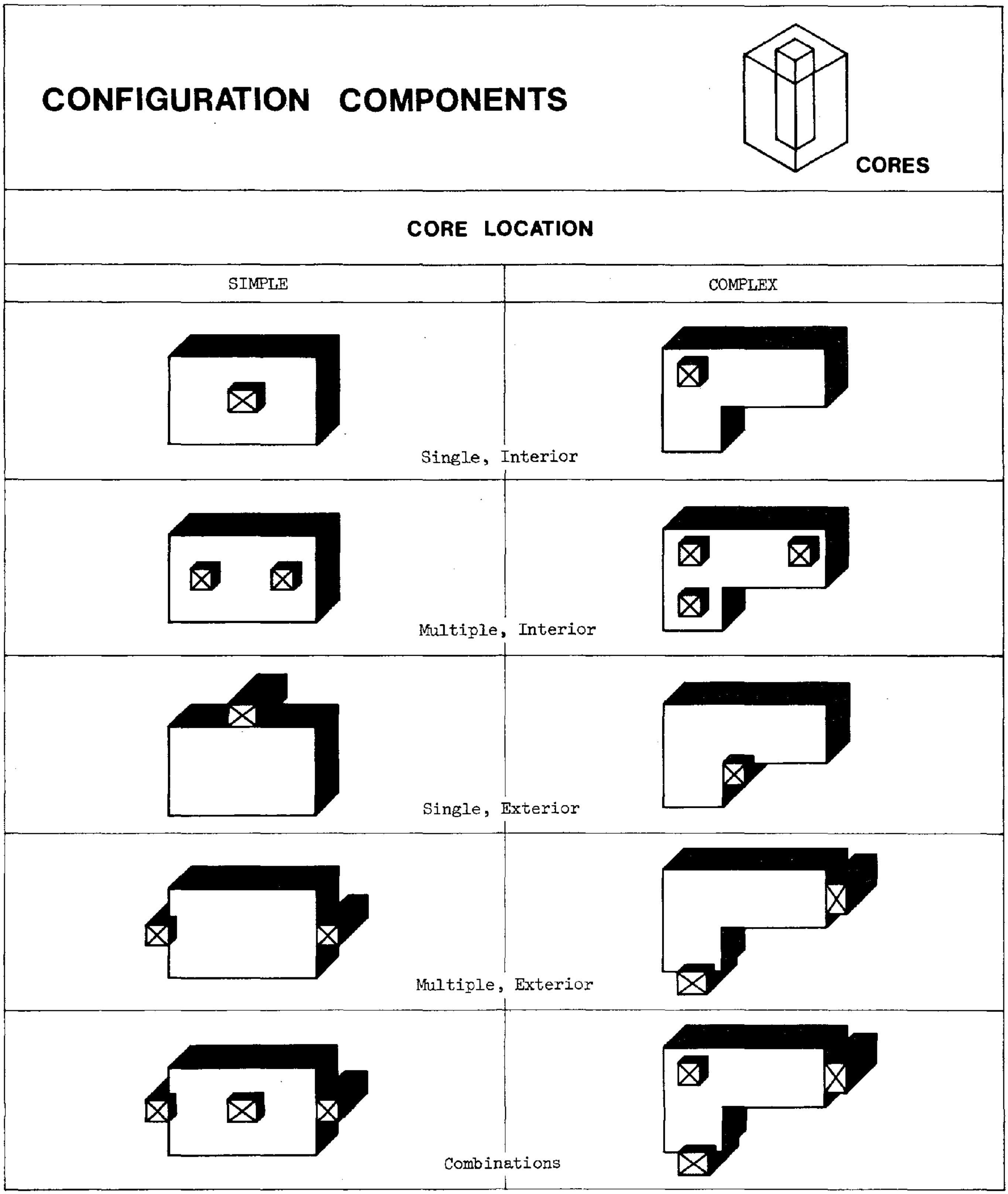


Figure 20. Configuration components: core location.

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A2. Bibliography

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A. Introduction

This bibliography has been designed to be of use to a varied audience: academic, research, and practicing architects and engineers; code officials; architectural historians and critics; planners; and others. It is both topically arranged and annotated. It is striking that almost no writings have been devoted entirely to the subject of seismic design and building configuration. Generally the subject is discussed in passing, or a particular technical sub-topic such as torsion, for example, has been chosen for study. This does not mean that no information concerning the subject has been published but merely that the information is widespread and has not been focused on this issue.

Hence, the topical structuring has been provided to pull together a wide variety of sources into categories which make sense in the context of this study. The annotation is offered only to help indicate the nature of the configuration-related contents of some references but is not intended to be judgmental, nor are the comments meant to be summaries of the overall scope of the works. Some cited sources, for example, will be readily recognized as major works whose reputations are based on their contributions in other subject areas and which only tangentially relate to the subject at hand.

The four divisions of the bibliography are:
-- 1. configuration in seismic design
-- 2. performance of specific buildings in past earthquakes
-- 3. research reports
-- 4. codes

B. Configuration in Seismic Design

Since one of the bibliographic conclusions of this study is that almost no papers or books have been devoted solely to the subject of building configuration and seismic design, the following works generally discuss this topic tangentially in the process of dealing with other issues. From a design standpoint, the other primary engineering categories are materials, detailing, and systems, and each of these topics has accumulated a much larger literature than the configuration subject.

Alexander, Robert L., "Where The Architect Stands On The Team: An Introduction To Aseismic Design," AIA Journal, December 1964.
"As an experienced architect conceives the spatial and esthetic requirements for a multistory building in a seismic area, he thinks in terms of structure intuitively. From his rough sketches of inspiration to his preliminary drawings, he has determined the structural form, column and story spacings, shear wall locations and surface openings... If he does a thorough job, his structural consultant will not be placed in the unfair position of having to choose between seeking revisions or resorting to the use of deformed structural solutions to the problems of bad architecture." (p. 42)

Architectural Graphic Standards, Joseph Boaz, editor, 6th edition, New York: John Wiley & Sons, 1970.
"The configuration of a structure and its fundamental period affects its earthquake resistance considerably. Symmetry in plan is very desirable. Unusual shaped plans result in high stress concentration areas and must be specifically designed for. Structural elements must be tied together to make them respond as a unit; or structural separations may be required." (p. 562)

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Architectural Institute of Japan, Design Essentials In Earthquake Resistant Buildings, New York: Elsevier, 1970.
"Bearing walls should be proportionately arranged in the plan. This is common to all wall construction. If the distribution of wall is one-sided, divergence of the location of center of mass of the building from that of rigidity of the walls become large and the building as a whole is twisted at the time of earthquake and dangerous stresses occur. For buildings in this country, there is a general tendency to provide large openings on the south side, therefore the amount of wall on the south side decreases thus causing easily one-sided distribution of the walls and these facts require attention. In case there is an inevitable need of such, it is recommended that all the walls in that direction be made of reinforced concrete frame construction." (p. 256)

Army, Navy, and Air Force (S.B. Barnes & Associates and John A. Blume & Associates, consultants), Seismic Design For Buildings, (The "Tri-Services Design Manual"), April 1973.
"ARCHITECTURAL CONSIDERATIONS: The seismic design begins with and is dominated by the architectural concept of the building. The architectural and functional requirements must be tailored to the concept of safety and damage control. Too little time is often spent on the architectural/structural concept, and too much on the stress analysis. We need a clear distinction between design (selection of a basically sound and economical structural system) and analysis (computation of stresses and deflections). Establishment of the configuration, fenestration, materials and details of construction must be a joint effort of the structural engineer and the architect...
Architect and structural engineer must work closely together to come up with the most desirable building that will be seismic resistant as well as functionally and aesthetically acceptable. Every effort shall be made to appreciate the characteristics of earthquake action and to recognize that a building of regular shape and symmetrical structural elements will provide more satisfactory response. Torsional effects due to earthquake forces create major points of discontinuity causing extreme high stress concentrations. Non-structural elements can profoundly alter the anticipated performance of a structure by either absorbing energy or concentrating energy at a point not designed to withstand it. Engineers are learning that a building's shape, symmetry, and its general layout developed in the conceptual stage are more important, or make for greater differences, than the accurate determination of the code-prescribed forces. These criteria have been viewed too long as prerogatives of the engineering professions; but they must have an increasingly greater bearing on architectural design in all its facets. It is the architect's and engineer's joint responsibility to design consistent with risk to life and property. Architects are advised to work closely with engineers at early consultative stages of the design process." (p. 3-12 & 3-13)

Arnold, Christopher, "Configuration And Seismic Design: A General Review," Second U.S. National Conference On Earthquake Engineering, 1979.
Touches upon all the major themes covered in the present study, including recurrent problem examples and the architect-engineer relationship.

Arnold, Christopher, "Building Configuration: Characteristics For Seismic Design," Seventh World Conference On Earthquake Engineering, September 1980.
Geometric definitions which usefully describe configuration for seismic purposes are introduced and illustrated. The history of the evolution of office buildings is specifically discussed to exemplify the variety of factors which affect building configuration, only one of which is seismic concern.

Arnold, Christopher, and Elsesser, Eric, "Building Configuration: Problems And Solutions," Seventh World Conference On Earthquake Engineering, September 1980.
A general matrix or summary of the relationships between specific building types and their architectural and structural implications in the seismic context. The Imperial County Services Building, badly damaged in the 1979 Imperial Valley earthquake, is used as an example.

Berg, Glen V., "Design Procedures, Structural Dynamics, And The Behavior Of Structures In Earthquakes," U.S. National Conference On Earthquake Engineering, 1975.
"Preserve symmetry. The inertia force in a building acts through the center of mass. The counterbalancing internal force is generated by the rigidity of the structure and acts through the center of rigidity. If the building is symmetric the center of mass and the center of rigidity coincide and torsion is eliminated. No building can be perfectly symmetric, of course, but accidental eccentricity is not apt to be very bothersome.
Avoid structural discontinuities. Just as a notch in a metal bar becomes an area of stress concentration, so will a structural discontinuity in a building be a region of potential structural damage. Often architectural considerations require setbacks or other abrupt changes in building stiffness. It is better to avoid them whenever possible. Occasionally the effect of a structural discontinuity may be caused by adjacent structures. A tall building may be stiffened in its bottom stories by lower adjacent buildings, and the effect of an abrupt change of stiffness at roof level of the adjacent building may lead to a concentration of earthquake damage in the tall building just above that level." (p. 74)

Bertero, Vitelmo V., "An Overview Of The State-Of-The-Art In Earthquake-Resistant Reinforced Concrete Building Construction," Second U.S. National Conference On Earthquake Engineering, 1979.
Selection of structural layout is emphasized as a very important step in this overview of the total design process. Applicable to non-concrete design as well.

Blake-Kelly, J.R., "The Effects Of Seismic Engineering On Architecture In New Zealand," Third World Conference On Earthquake Engineering, 1965.
The "Napier style," based on the least expensive concrete system, "a very plain wall type of architecture," briefly followed the 1931 Napier earthquake and was simply a substitution of concrete for brick. In the mid-30's a government building concrete style evolved. "When using reinforced concrete in low-rise buildings, because fire resistance measures established stiff walls about lifts, staircases, etc., the great majority of all internal walls were in reinforced concrete too, so that a heavy and rigid compartmentation was introduced with minimal openings. This system worked externally

with the type of Georgian proportion elevations, for the relation of solid to void allowed easy two-way reinforcement... The early 1950's saw a more liberal easing of the rigid shear-wall architectural strait-jacket. Information was now becoming available again from America and Japan..." (p. IV-38). Smaller design firms "have not established any discernible continuing development of an architecture conditioned by seismic engineering. Their structural consultants are hired job by job with very little likelihood of continuity of design development... In the private field there is no parallel to the State's 30 years of endeavor broken though it was by war and controls..." (p. IV-39)
"The private field has more difficult sites in city commercial work than the many island sites of the State. The small frontages... in conjunction with lengthy side boundaries dictate stiff side walls, with deep-membered spandrels front and rear, from which most of the natural daylight must come too." (p. IV-40)
The Ministry of Works "has shown over recent years developments of an architecture influenced by seismic engineering - in non-technical terms, to display buildings that look as though they can withstand earthquakes." (p. IV-51)
Typical examples of buildings from the 30's to 60's are illustrated.

Blume, John A., Newmark, Nathan M., and Corning, Leo H., Design Of Multistory Reinforced Concrete Buildings For Earthquake Motions, Skokie, Illinois: Portland Cement Association, 1961.
General seismic design reference, as well as containing specific concrete design guidance. Torsion is covered with an example in Section 4.9.

Blume, John A., in Joint Committee On Seismic Safety, California Legislature, "Public Hearing On Seismic Hazards Of High-Rise Buildings In The San Francisco Bay Area," October 24, 1972.
"I am especially concerned with: (1) some old structures not having adequate strength or ductility, lacking integrity of floor-wall-framing connections, and lacking redundancy; and (2) some modern structures where the designers put all their faith in a few first story vertical members- columns and/or walls - which barely pass the code requirements without reserve capacity in the inelastic range." (p. 26-27)

Blume, John A., "A Message From EERI President John Blume," Earthquake Engineering Research Institute Newsletter, November 1979, Vol. 13, No. 6.
A brief summary of some major configuration problems: "... the special risks involved with 'core' type resistance providing very low polar moment of inertia in stiffness and strength; of what I have termed 'vagrant' architecture in which there is no visible means of support; of very flexible first stories where the lateral forces can only be estimated with great uncertainty; of poor seismic layout or geometry having structural offsets in plan and/or elevation; of the special demands on corner columns; and of the fact that no building will retain its structural symmetry, if indeed it has this to start with, and remain without torsion when it inevitably enters the inelastic range under severe motion." (p. 2)

Borges, J. Ferry, and Ravara, A., Earthquake Engineering: Seismic Design Of Reinforced Concrete Buildings, (course syllabus), Laboratorio Nacional De Engenharia Civil, Lisbon, 1969.
"As regards the structural conception, attention should be drawn to the advantage of reducing the eccentricity due to the asymmetry in plan of the structural elements. Such asymmetries give rise to torsion moments which can be sufficiently high to cause, alone, the collapse of the structure." (p. 117)

Botsai, Elmer E., et al, Architects And Earthquakes, Washington, D.C.: Government Printing Office, n.d., (AIA Research Corporation). The problems of the L- and T-shaped plans, eccentric core, and inverted setback configurations are discussed pp. 40-43 and 46-47.

Clay Products Institute of California, Earthquakes And Building Construction, 1929.
While obviously motivated by a concern for the materials represented by this trade group, configuration also stands out sharply as a major issue in this summary of the state-of-the-art in the twenties. This work's emphasis on the percentage of wall opening, and on the ratio between lineal wall footage and floor area square footage now seems anachronistic: It is perhaps unfortunate that the intelligent use of configuration to proportion a building's resistant capacity, as compared to its load, has been neglected in recent years, and also that our sophisticated quantitative state-of-the-art today often leads us to fail to perceive the forest for the trees, and to incline architects especially to conclude that earthquake resistance begins with analysis (rather than design) and is achieved only through the engineer's selection of systems, materials, and details (rather than choice of configuration as well).

Dean, R. Gordon, and Zacher, Edwin, "Structural Considerations," Architects And Earthquakes: Research Needs, AIA Research Corporation.
"...the choice of structural system, symmetry of the system, presence of abrupt changes in rigidity and often choice of materials is greatly influenced by and sometimes dictated by the initial concept and configuration of a building. The Structural Engineer cannot provide continuous shear walls if there are no places in the basic layout where they can run. A completely open first story eliminates the possibility of continuing exterior walls to the ground. The combination of exterior concrete walls over an open first floor automatically creates an abrupt change in stiffness. Open street fronts in a building on a corner lot with solid property line walls on one or two sides in the rear create an almost unmanageable torsion problem. Effective moment resisting frames require sufficient vertical space for the large girders necessary to make frames rigid...
...continuous skylights, pipe chases, or other openings through roof or floor diaphragms adjacent to shear walls make the proper connection of a diaphragm to a shear wall difficult, if not impossible. A stairwell or elevator shaft straddling the line of a diaphragm chord will substantially weaken a diaphragm. A series of story high openings in each story through an otherwise continuous shear wall literally cuts the shear wall into separate walls and greatly reduces the effectiveness of the wall. Placing rigid but non-structural exterior panels on an otherwise flexible building, invites damage to the rigid panels, etc. Although the actual design of the structural components of a building to resist earthquakes is the responsibility of the Structural Engineer, the

successful design requires close cooperation with the Architect. This cooperation must begin with the first schematic studies before the initial architectural concept is established." (p. 58)

Degenkolb, Henry J., Earthquake Forces On Tall Structures, Bethlehem Steel, 1970.
"Much of the problem would be solved if all structures were of a regular shape, but economics of lot sizes and arrangements, various planning requirements for efficient use of space, and esthetically pleasing proportions require the structural engineer to provide for safe constructions of various shapes. L, T, or U shapes or variations of these must be accommodated in many building designs. Engineers must realize that re-entrant corners are areas of great stress and must reinforce their structure accordingly. In code terms, the amount of stress is difficult to define, therefore each case must be individually analyzed by the engineer, considering the magnitude of forces to be resisted. Provisions must also be made to resist various combinations of force transference that are likely to occur. All buildings, no matter how regular theoretically, are subject to torsion and those non-symmetrical shapes will be subjected to major torsional stresses." (p. 16)
Contains an example of a particular cruciform plan problem.

Degenkolb, Henry J., in Joint Committee On Seismic Safety, California Legislature, "Public Hearing On Seismic Hazards Of High-Rise Buildings In The San Francisco Bay Area," October 24, 1972.
"If earthquake safety is really a high priority item in the design of high-rise structures, the concept and design of the building configurations should be suitable for providing dynamic lateral resistance and not subject to the latest architectural whims and fashions." (p. 18)

Degenkolb, Henry J., "Earthquake Engineering And The Practicing Engineer," The Future Of Earthquake Engineering: Proceedings Of The Inaugural Symposium Of The John A. Blume Earthquake Engineering Center, September 17, 1976.
"Even though our technical knowledge is not adequate, the engineer can design reasonably safe structures, just as other engineers of the past were able to design serviceable structures without even a small portion of our present advantages of materials and analytical methods.
In order to provide reliable structures, the most important requirement is that the engineer be truly professional and take pride in his work to satisfy both client and society. He cannot rely heavily on codes or standards of practice. Lengthy, complex codes may mislead the engineer as to the extent and reliability of our current earthquake knowledge and will tend to make him more of a technician than an engineer.
In the past, our detailed earthquake codes have contained errors or have omitted important factors to the extent that the collapse of some recently designed buildings may be assured in the event of a major earthquake. We must expect that present codes or that codes propounded in the next few years will contain similar deficiencies." (p. 128-129)

Degenkolb, Henry J., "Seismic Design: Structural Concepts," Summer Seismic Institute For Architectural Faculty, Washington, D.C.: AIA Research Corporation, October 1977.
Emphasizes importance of configuration; explains soft story and

other problems in the process of providing an introduction to principles of structural engineering as applied to earthquakes.

Dewell, Henry, "Earthquake Design," Engineering News Record, May 3, 1928.
Plans should be rectangular, no setbacks, solid walls at corners, heavy towers or appurtenant structures avoided.

Dowrick, David J., Earthquake Resistant Design, London: John Wiley & Sons, 1977.
See Chapter Four, "Determination Of Structural Form." "This chapter is addressed to architects as well as engineers because the structural engineer cannot make a poor structural form behave satisfactorily in an earthquake...
Briefly, the structure should be simple; be symmetrical; not be too elongated in plan or elevation; have uniform and continuous distribution of strength; have horizontal members which form hinges before the vertical members; have its stiffness related to the subsoil properties." (p. 80)

Dowrick, David J., "Structural Form For Earthquake Resistance," Sixth World Conference On Earthquake Engineering, 1977.
"Many engineers recognize that structural form is of great importance for earthquake resistance. It is somewhat surprising that this aspect of design has not been discussed in the literature as a subject in its own right until now." (p. 1826)
A brief version of the same material which appears in Dowrick's book cited above.

Earthquake Engineering Research Institute, Learning From Earthquakes: Planning And Field Guides, Earthquake Engineering Research Institute, 1977.
This reference is designed for use by post-earthquake observation and research teams.
"Irregular Systems. Commentary: Re-entrant corners, insets, setbacks and similar breaks in the continuity of the lateral-force-resisting system tend to result in areas of localized damage. These irregularities may be due to discontinuous and inadequate force paths, unrecognized force components, and/or construction variabilities.
Torsional responses are expected in buildings having marked asymmetry in geometrics, stiffnesses, and masses. Torsion can also arise from other sources, such as the presence (and participation in the building response) of stairs, partitions, and masonry infill walls. Structural failures and shifting in the structural response characteristics due to damage can contribute to a torsional response. Restricted deformation as evidenced by pounding against adjacent structures can induce torsion.
It is possible to overcome partially the problems of irregular systems, but this requires careful and thorough design and execution in the field. Measures which are effective or ineffective in controlling or reducing damage in irregular systems should be noted.
Checklist:
1. Irregular plans and setbacks in elevation.
2. Changes in the lateral-load-resisting system, in materials, masses, or stiffnesses.
3. Evidences of torsional response.
4. Relative behavior of regular and irregular systems in the same general area.

5. Good and poor design details and construction procedures." (p. 51-52)

Eiby, G.A., "A History Of Anti-Seismic Measures In New Zealand," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 8, No. 4, December 1975.
Brief reference to a commission report on the South-West Wairarapa earthquake of 1855 in which C.R. Carter, a contractor, produced a theoretical argument that pyramids were the ideal architectural form for earthquake resistance.

Engle, H.M., "The Earthquake Resistance Of Buildings From The Underwriter's Point Of View," Bulletin of the Seismological Society of America, Vol. 19, No. 2, June 1929.
"The shape of the building is of some importance. As bracing should be of equal strength in all directions and be symmetrical about the center of mass of the building, a closed shape is desirable, preferably square or not too badly rectangular, as these shapes lend themselves most easily to equal and symmetrical bracing. It is more difficult to get a proper distribution of bracing in odd-shaped buildings, such as those of L or U shapes. If the building is to resist most successfully, the natural period of vibration of all parts of the building must be about the same, and this is more difficult to get with the odd-shaped buildings with long wings. It is preferable to avoid the odd shapes, if possible, and where used they should be carefully designed so that bracing will be the same in all directions and the natural period of vibration of each part about the same." (p. 91)

Fattal, S.G., "Structural Performance Of Low-Cost Housing And Community Buildings Under Earthquake Conditions," in William F. Reps and Emil Simiu, editors, Design, Siting, And Construction Of Low-Cost Housing And Community Buildings To Better Withstand Earthquakes And Windstorms, Washington. D.C.: Government Printing Office, 1974, (National Bureau of Standards).
"The geometry of a building may have a marked effect in distributing seismic forces and influence the type and degree of failure. For example, twisting and warping are more likely to occur in buildings of irregular shape...or in buildings having nonuniform arrangement of walls and openings. Failure commonly occurs at corners of openings...at the top junctions of abutting walls, near attachments of projecting elements, or, as a result of "hammering" between adjacent walls of detached buildings. In general, buildings marked with sharp transitions in mass and/or geometry tend to develop structurally unfavorable force distribution patterns." (p. 13-15)

Freeman, John Ripley, Earthquake Damage And Earthquake Insurance, New York: McGraw-Hill, 1932.
A classic work, summarizing the state-of-the-art at that time (and still valid even today in most respects). Includes a variety of configuration topics in its discussion of seismology and engineering.
Concerning re-entrant corner plans: In reference to the Pacific Telephone & Telegraph Building in San Francisco: "The ground plan is not a closed rectangle and it remains to be seen if the projecting wing in the rear...will develop difficulties at the unbraced rear wall when, if ever, violent earthquake oscillations attack the structure. The rigid concrete floors are doubtless relied upon for distribution of any exceptional horizontal stress from the wing or ell. This example is of particular interest be-

cause it is not always practicable to avoid wings or ells, or to provide heavy internal partition walls at their intersection, to act as braces." (p. 380)
End wall, interior partition, and corner wall bracing are discussed p. 812-816: overall stiffness and strong corner walls, end walls and interior partitions are considered advisable; torsional motion resulting from asymmetric wall layout is diagrammed; "The architect should plan his window and door openings with careful reference to resistance to earthquake stress by the wall, which may be greatly weakened by these openings unless great care is taken in placing them and in reinforcing around them." Pros and cons of the soft story are discussed.
Tachu Naito's contemporary seismic design principles are summarized, which include: even and symmetrical distribution of walls, braces, and bents in plan and vertical continuity of these elements throughout the height of the structure; combining stiff and flexible components creates the problem of incompatible deflections; possibly negative effects of infilling frames without regard for increased stiffness; corners should be walled as solidly as possible; plan forms should be closed.

Green, Norman B., Earthquake Resistant Building Design And Construction, New York: Van Nostrand Reinhold, 1978.
Torsion is discussed, with examples, in Chapters Three and Five.

Hauf, Harold D., "Minimizing Earthquake Hazards: II. Architectural Factors," AIA Journal, July 1968.
"Wherever possible, buildings should be designed so there will be no substantial built-in eccentricity... It is believed that torsional phenomena may be of greater significance in the resistance of tall framed buildings to earthquakes than has been recognized generally. This applies particularly to modern curtain wall buildings because of their lack of peripheral resistance, whether the curtain wall consists of precast concrete panels or metal and glass panels....
The problem is more difficult to handle architecturally in buildings where the service core is asymmetrically placed with respect to the entire building envelope. Since the walls surrounding the core are usually available for the structural purposes, symmetrical core location is obviously advantageous, but it is not always feasible. The ingenuity of both architect and structural engineer may be taxed to minimize potential torsional displacement in buildings with asymmetrical plans...
Imaginative design thinking can produce compatible solutions to this highly significant problem if it is recognized as such during the early stages of a project." (p. 70-71)

Hunt, Sumner, "Committee On Building For Safety - Preliminary Report On Construction Of Walls And Floors," Bulletin of the Seismological Society of America, Vol. 16, No. 4, December 1926.
"Plan.---In studying the action of earthquake shock on walls, consideration of the plan or shape of the building is important. Thinking of the Pyramids, one almost wonders if the Nile country might not have been subject to earthquakes and if the Pyramid builders might not have designed them especially to withstand earthquake shock. Certainly the pyramidal is the ideal shape of building to withstand such a shock - the maximum weight at bottom, the minimum at top is the ideal to strive for.
The square type of plan, with uniform weight on all sides of the center is also an ideal form." (p. 268)

International Conference On Planning And Design Of Tall Buildings, Tall Buildings: Systems And Concepts, Vol. Ia, New York, 1972.
Although extensive consideration of the seismic issue is absent from these proceedings (wind is given more attention concerning lateral forces) several papers are relevant in their discussions of the structural layouts of tall buildings.

Long Beach, California, City of, Subdivision 80 of the Building Regulations.
Deals with the problem of how to abate the hazard posed by existing unsafe buildings, typically those of unreinforced masonry construction. The checklist rating system used by building inspectors specifically lists and penalizes these configuration conditions: "Poor dimensional plan, "L" and "T" shapes, (non-rectangular). Lack of symmetrical bracing or shear walls. Excessive length to width and height ratios (greater than 4:1)...Excessive wall openings & wall heights without adequate design...Excessive wall openings include percentages of 50 and over for any one wall in any story." (Appendix A-5)

Montel, Alfredo, Building Structures In Earthquake Countries, London: Griffen, 1912.
"It is therefore advisable to construct very low buildings, with very few stories and a light roof which cannot be deformed and is rigidly united with the walls. In addition to that, the various parts ought to show as few interruptions of continuity as possible, and the transition from one to the other ought never to be made in a sudden manner. The circular form is theoretically the most suitable one for a building, the more so as, when using it, no account need be taken of the direction of the seismic action, which is very often ill defined or wholly unknown. The good resistance of circular buildings to earthquakes has, besides, already been observed in practice." (p. 81)

Page, Robert A., Blume, John A., and Joyner, William B., "Earthquake Shaking And Damage To Buildings," Science, Vol. 189, No. 4203, August 22, 1975.
In the process of comparing old and new buildings reference is made to the "fortunate geometry" and extensively-walled plans of the former.

Polyakov, S., Design Of Earthquake Resistant Structures, Moscow: Mir Publishers, 1974.
"Horizontal torsional moments that are always present in a vibrating building make the wings highly vulnerable to earthquakes when they are structurally connected to the building. In planning a building, such wings should be avoided. When architectural planning makes it impossible to avoid this type of construction, the wings should be simple in form and separated from the main part of the building by separation joints to prevent contact of the wings with the building during vibration.
Analyses of earthquakes have shown, the most desirable configuration of a building (or its parts) is a circle, a polygon, a square or a configuration close to these. The strength and rigidity of the walls and other structural elements of such buildings in any position are approximately the same and hence their resistance to a horizontal seismic shock in any direction is the same. Such buildings also perform better when they are subjected to horizontal rotation." (p. 151)

Soviet code provisions which set dimensional and proportional limits for buildings (broken down by construction type and geographic zone) are discussed.

Portland Cement Association, Analysis Of Small Reinforced Concrete Buildings For Earthquake Forces, Skokie, Illinois: 1955 (and later editions).
A general introduction as well as an explanation of topics unique to concrete. An analysis example of an asymmetrical (open-front) building is provided. The drastic change in rigidity that results from relatively slight changes in the dimensions of piers is illustrated via nomographs.

Prendergast, J.D., and Fisher, W.E., Seismic Structural Design/ Analysis Guidelines For Buildings, U.S. Army Construction Engineering Research Laboratory, February 1977.
"Preserve symmetry. Avoid irregularly (L, T, U, and +) shaped building layouts unless adequate design precautions are taken to subdivide the building into regularly shaped integral units which can respond independently and are structurally separated by sufficient distance to avoid contact under the expected maximum lateral deflections. Furthermore, avoid mixed framing systems such as a shear wall on one side of a building and a steel frame on the other.
Minimize building torsion. The distance between the center of mass and the center of rigidity should be minimized.
Provide direct vertical paths for lateral forces. Avoid transferring lateral forces over long distances through diaphragm action or through complicated structural systems that require the lateral forces to be transferred through setbacks, overhangs, and other geometrical irregularities before reaching the foundation.
Avoid abrupt discontinuities. Minimize abrupt changes in the lateral resistance or stiffness such as large openings in shear walls, interruption of columns and beams, diaphragm openings, or changes in structural systems between stories." (p. 11)

Reitherman, Robert, "Frank Lloyd Wright's Imperial Hotel: A Seismic Re-evaluation", Seventh World Conference On Earthquake Engineering, 1980.
Rather than the much publicized foundation system, it is primarily the Hotel's configuration which is singled out as a positive seismic design contribution.

Shah, Haresh C., Zsutty, Theodore C., and Padilla, Luis, "The Purpose And Effects Of Earthquake Codes," Internal Study Report No. 1, John A. Blume Earthquake Engineering Center, August 1977.
"It is most important to realize that an earthquake can have widely differing effects on different types of buildings depending upon their qualities of symmetry and regularity. If a building is well-braced by walls, regular and symmetrical, drift is easily controlled. If, however, there are drastic irregularities from floor-to-floor, or if the plan is grossly non-symmetrical in its floor plan or with walls on one side and flexible framing on the other - then severe localized drift and torsional twisting distortions will occur." (p. 6)

Siegel, Curt, Structure And Form In Modern Architecture, New York: Reinhold, 1961.
A large number of rules or principles interrelating structural behavior and aesthetic design are formulated. This basic text on

structuralism neglects not only seismic but also wind loading, and hence its generalizations, such as the statement that corner columns have less tributary area for gravity loading and therefore should be smaller than all the rest, may seem simplistic, and the approach emphasizes only theoretical structural principles, rather than the practical aspects of constructing structures, but within these limits it thoroughly examines a multitude of specific structuralist issues and stimulates interest in the interrelationship of structure and architecture.

Simonds, George, P., "Building Configuration And Earthquakes As An Architect Looks At Design," Earthquake Engineering Of Buildings Symposium, February 5-6, 1968, San Francisco, Earthquake Engineering Research Institute, 1968.
"Form in buildings is determined by the Architect. Traditionally architecture has been influenced by "firmness, commodity and delight" and by style or fashion as well.
Today's style forms do not have those inherent qualities of earthquake resistance as did the solid wall, the cellular space, the symmetrical arrangement of the past. Such stylistic forms place added burden on the structural consultant-designer."
One of only a handful of papers or articles which have ever been devoted to this topic. (from abstract submitted for conference)

Téran, Jose Francisco, "Historical Context Of Building Forms In Managua," Managua, Nicaragua Earthquake Of December 23, 1972, November 29-30, 1973 conference proceedings, Earthquake Engineering Research Institute.
A brief historical review of Nicaraguan architectural form in the seismic context. Emphasizes that building form is a result of many influences. A building form should be created early in the design phase "that at least will not hinder or handicap the structural solution." Modern architecture, even though introduced following the 1931 Nicaraguan earthquake, often introduced problematic forms. "The question arises as to whether the building should be designed to meet the functional, social and aesthetic needs and then be implemented for structural safety or if in seismic areas like Managua, the special problems of stability and overall integrity should condition the design process by which the elements of form such as mass, symmetry, modulations, etc. are decided...how can architects, engineers, owners, and the whole community develop a common design attitude...?" (p. 322). Photos illustrating historical trends are included. Téran's brief paper is one of the very few works devoted to the issue of the evolution of architectural trends and forms in the context of the earthquake problem.

UNESCO Working Group On The Principles Of Earthquake Resistant Design, Intergovernmental Meeting On Seismology And Earthquake Engineering, Paris, April 21-30, 1964.
"It is desirable that a structure have a simple layout in plan and elevation. The plan should be as nearly symmetrical as possible... the center of mass of the structure should be kept as low as feasible by avoiding heavy loads at or near the roof level, or in the upper portions of tall structures...the center of mass in the horizontal plane at all levels should preferably coincide with the center of rigidity of the structure in order to minimize torsional effects. Torsional effects should be considered in the analysis where they cannot be eliminated. Careful consideration should be given to irregular shaped structures and to portions of structures

having different rigidities. When an irregularly shaped structure is unavoidable, it should be designed to act as an integral unit, or different portions of the structure should be adequately separated to avoid collision due to seismic forces acting in any direction. Also adjacent structures should have adequate separations." (p. 179)

Weidlinger, Paul, "Visualizing The Effect Of Earthquakes On The Behavior Of Building Structures," Architectural Record, May 1977. "Ideally, a structure should have two axes or symmetry in plan. This is easily accomplished if the architectural plan has these properties, but, even so, structural symmetry can be achieved sometimes within a slightly unsymmetrical plan, by intelligent disposition of the framing systems. The symmetry avoids torsion in the structure caused by lateral seismic forces... An irregular plan also usually leads to complex, higher mode shapes which can, somewhat unpredictably, result in very high design moments and shears in unexpected parts of the structure. And these may lead to loss of economy and to unpleasant and unanticipated dimensions of structural members." (p. 147)

Yanev, Peter, Peace Of Mind In Earthquake Country, San Francisco: Chronicle Books, 1974.
Yanev states that "Most such buildings [split-level and multi-story houses or apartment buildings with garage underneath] have been constructed since the early 1950's, and in the only three earthquakes to strike heavily populated areas of California since 1952 - all moderate shocks - these buildings suffered a very disproportionate amount of the total damage.
The reason for the failure of these buildings is obvious. They are inherently weaker than conventional buildings resting wholly on the ground, because the large garage door and any other windows and doors on the lower garage level constitute a large portion of the wall area which must carry and resist the brunt of the earthquake forces. The garage level is, in effect, a foundation with only three walls." (p. 197-199)

C. Performance of Specific Buildings in Past Earthquakes

Although any failure must involve materials, the materials science aspect to a particular failure may be quite unimportant while the issues of detailing, for example, may be quite pertinent. Similarly, since all buildings have configurations, all examples of good or bad performance could be logically related to the configuration topic, but this would not usefully identify those cases in which anything useful concerning configuration has been learned.

Generally the following cases involve a poor configuration which contributed significantly to damage, although in some cases engineers who have studied the building would consider configuration to be only one of several contributing factors, and a few examples of good performance are included as well. Olive View Hospital is a good example of a case involving multiple and major configuration issues as well as quite significant issues concerning reinforcement, detailing, ground motion parameters, design force levels, and ductility. Hence this is not a list of buildings whose damage was "caused" by configuration, although in many cases configuration was the paramount factor. In other cases, there is a suggestive link between configuration and performance which has not yet been verified due to lack of existing information. Even in cases where

configuration is obviously a major if not the controlling factor, subsequent research generally emphasizes other subjects, such as materials, at the expense of any in-depth analysis of configuration: More attention should be focused on the configuration subject in future earthquake analysis.

1835 Chile

Darwin, Charles, The Voyage Of The Beagle, Garden City, New York: Doubleday.
Directionality of ground motion in Concepcion and damage to walls of various orientation among dwellings and in the cathedral are noted. (p. 303-310)

1886 Charleston, South Carolina

Dutton, Clarence Edward, The Charleston Earthquake Of August 31, 1886, Washington: Government Printing Office, 1890, reprinted 1979 (U.S. Geological Survey).
- Hastie House, Summerville and similar houses in Charleston (soft story).

1906 San Francisco

American Society Of Civil Engineers, "The Effects Of The San Francisco Earthquake Of April 18, 1906 On Engineering Constructions," ASCE Proceedings, Vol. 33, No. 2, March 1907.

Gilbert, Grove Karl, Humphrey, Richard Lewis, Sewell, John Stephen, and Soulé, Frank, The San Francisco Earthquake And Fire Of April 18, 1906, Washington, D.C.: 1907 (reprinted edition by San Francisco Historical Publishing Co., n.d.).

Lawson, Andrew C., et al, The California Earthquake Of April 18, 1906: Report Of The State Earthquake Investigation Commission, Washington, D.C.: Carnegie Institution, 1908, reprinted 1969.
- Morrell Ranch, Santa Clara County (soft story). (p. 276-277)
- Bolinas Lagoon waterfront structures (soft story due to piles)

Page, Robert A., Blume, John A., and Joyner, William B., "Earthquake Shaking And Damage To Buildings," Science, Vol. 189, No. 4203, August 22, 1975.
More noticeable than the presence of configuration-induced damage in 1906 was its absence, due to the inherently desirable nature of the architecture of that time from the seismic standpoint. This paper comments on the good performance of the large mid-rise buildings in San Francisco, which it partly attributes to the extensive use of non-calculated but actually lateral load bearing partitions in cellular office plans, and overall simple configurations.

1925 Santa Barbara, California

Dewell, Henry D., and Willis, Bailey, "Earthquake Damage To Buildings," Bulletin of the Seismological Society of America, Vol. 15, No. 4.
- San Marcos Building (L-shaped plan)
- El Camino Real Garage and Hotel (soft story)
- Carrillo Hotel (soft story)

Olsen, Phil G., and Sylvester, Arthur G., "The Santa Barbara Earthquake, 29 June 1925," California Geology, June 1975.
- San Marcos Building (L-shaped plan)

1927 Kita-Tango

Okamoto, Shunzō, Introduction To Earthquake Engineering, New York: John Wiley & Sons, 1973.
"There were houses in the area of most violent earthquake (shaking) along the Gōmura Fault which escaped significant damage, and these were without exception houses with light roofs and room arrangements providing a great number of internal walls." (p. 67)

1933 Long Beach, California

Binder, R.W., "Engineering Aspects Of The 1933 Long Beach Earthquake," Proceedings of the Symposium on Earthquake and Blast Effects on Structures, Los Angeles, June 1952.
"Emphasis should be placed on the fact that in some cases the architectural treatment gave certain structures initial inherent resistance to seismic disturbances..." (p. 191)
From the unpublished report of the Earthquake Committee of the Structural Engineers Association of Southern California: "...Aside from many schools, collapse was confined largely to stores, markets and garages, on corners. Ground motion parallel to either street front of a corner building with two walls cut up by show windows will tend to produce twisting about a vertical axis. This tendency to twist accounts in part for the more frequent collapse and generally greater damage to corner buildings. It did not, however, adversely affect all such buildings, many of which suffered no more than those on interior lots... although most of the buildings of this type had walls of unit masonry, there were a number with bearing walls of reinforced concrete. These latter suffered much less than the former and none of the reinforced concrete walls collapsed, but the damage was nevertheless sufficient to indicate that merely the substitution of reinforced concrete for unit masonry will not render this type of building completely immune from earthquake hazard... the first story of a very old three-story hotel on Pier Avenue in Naples spread out and let the upper two stories suddenly settle down vertically. Not originally intended for this purpose, this building had been converted for use as a hotel by the removal of most of the partitions in the first floor." (p. 193 and 195)
From the Millikan Report: A dozen important conclusions based on observations of damage include the importance of "considering the proportions and shape of the structure," "eliminating unnecessary dead weight," "providing for torsional effects," "providing for the necessary separation from adjacent structures." (p. 205)

1940 Imperial Valley, California

Ulrich, Franklin P., "The Imperial Valley Earthquakes Of 1940," Bulletin of the Seismological Society of America, Vol. 31, No. 1, (January 1941).
- Woodrow Hotel, Brawley (soft-story)

1952 Kern County, California

Steinbrugge, Karl V., and Moran, Donald F., "An Engineering Study Of The Southern California Earthquake Of July 21, 1952 And Its Aftershocks," Bulletin of the Seismological Society of America, Vol. 44, No. 2B, (April 1954).
- Brock's Department Store (asymmetric wall layout, torsion)
- Kern General Hospital (re-entrant corners)

The authors recognized several single family house design trends in their infancy at that time which have subsequently grown steadily: "the excessive number of exterior wall openings often found in many modern homes. Many residences, for example have continuous picture windows in addition to a double garage door opening. The lack of balanced resisting elements can cause damage due to twisting and racking... Steep hillsides are now being covered with homes. One side of the residence may be one-story

while the other may be two or three stories... Construction in some instances amounts to no more than a building on stilts...it is probable that the overall behavior of wood frame dwellings in future shocks may not be as good as it has been in past shocks." (p. 225)
- Haberfelde Building (pounding)
- Pacific Fire Rating Bureau file #'s 194, 397, 556, and 2251 (pounding damage to medium-rise buildings in Los Angeles)

1954 Fallon, Nevada

Steinbrugge, Karl V., and Moran, Donald F., "Damage Caused By The Earthquakes Of July 6 And August 23, 1954," Bulletin of the Seismological Society of America, Vol. 46, No. 1, (January 1956).
- Main Street commercial buildings (pounding)

1957 Daly City, California

Steinbrugge, Karl V., Bush, Vincent R., and Zacher, Edwin G., "Damage To Buildings And Other Structures During The Earthquake Of March 22, 1957," in Oakeshott, Gordon B., ed., San Francisco Earthquakes Of March 1957, California Division of Mines and Geology Special Report 57, 1959.
- single family two story houses: "Many of these wood frame structures are located on 25 foot - or somewhat wider - lots, and often no space exists between the sides of adjoining dwellings. Also these homes are conventionally two-story. The garage, laundry areas, storage areas, etc., are in the first story; the second story is living space. An arrangement of this type leads to numerous partitions in the second story with a minimum number of partitions in the first story. The front wall in the first story has numerous openings. The rear wall of the story has fewer openings than the front. Many of the side walls have no openings. The result is a building which in general is weak in the first story especially against transverse lateral forces. This basic house design is characteristic of a great many homes in San Francisco and Daly City and has been used by many builders. The degree of lateral force stability will vary somewhat with the design used by the individual builder, but the transverse lateral force resistance of the first story is theoretically the weakest region in practically all cases." (p. 76-77)

1960 Agadir, Morocco

Kirkland, W.G., Binder, R.W., Clough, R.W., and Higgins, T.R., "The Agadir, Morocco Earthquake, February 29, 1960," Earthquakes, Washington, D.C.: American Iron and Steel Institute, 1975.
- Sud Building (L-shaped plan)
- apartment building across the street from Sud Building (soft story)
- La Reserve Restaurant (soft story)
- Immeuble Paternal (soft story)
- New City commercial buildings (soft story, soft front wall, bracing provided accidentally by roll-down steel doors)

1960 Chile

Clough, Ray W., and Jenschke, Victor, A., "The Effect Of Diagonal Bracing On The Earthquake Performance Of A Steel Frame Building," Bulletin of the Seismological Society of America, Vol. 53, No. 2, (February 1963).
"Diagonal bracing, intended to increase the lateral strength of two steel frame buildings at the University of Concepcion, was broken by the Chile earthquake of May 21, 1960. The unbraced structures then resisted the earthquake of May 22 with no further structural damage, thus raising the question whether the bracing was either necessary or beneficial. The results indicate that for some earthquakes, the bracing may be beneficial. However, for other cases, the bracing induces increased forces in the frame which exceed "

strengthening effect of the bracing. Thus the unbraced structure actually may be less earthquake resistant than if it were braced." (p. 389)
Although confined to elastic analysis, the authors suggest that if inelastic behavior is considered, the bracing's connections could suddenly fail and the bracing would contribute little energy absorption.

Portland Cement Association, "The Behavior Of Reinforced Concrete Buildings In The Chilean Earthquakes Of May 1960," Skokie, Illinois: Portland Cement Association, 1963.
- hotel, Puerto Montt (pounding)
- Seminario, Puerto Montt (L-shaped plan)
- Prales Building (good performance; yet had an H-shaped plan)
- Valdivia post office (asymmetrical shear wall layout)
- Traumatologico Hospital (L-shaped plan)
- Regional Hospital (pounding)

"Torsional effects were often computed precisely and painstakingly and the columns and walls arranged in such a manner as to eliminate computed torsion. In other words, the center of rigidity was made to coincide with the center of mass. Nevertheless, as previously reported, there were instances of torsional distress... Provisions for accidental torsion are necessary even for symmetrical structures because of the random character of seismic motion and structural resistance." (p. 30)

Steinbrugge, Karl V., and Flores, Rodrigo A., "The Chilean Earthquake Of May 1960: A Structural Engineering Viewpoint," Bulletin of the Seismological Society of America, Vol. 53, No. 2, (February 1963).
- Carlos Anwandter School (torsion, "nonstructural" masonry infill creating short column problem, lack of longitudinal as compared to transverse walls)
- Regional Hospital, Valdivia (torsion, pounding, holes in shear walls, reliance on interior core for lateral resistance)
- Seminario, Puerto Montt (L-shaped plan)

1963 Skopje, Yugoslavia

Berg, Glen V., "The Skopje, Yugoslavia Earthquake, July 26, 1963," Earthquakes, Washington, D.C.: American Iron and Steel Institute, 1975.
- Skopje Fairgrounds Hall (unequal column heights)
- Feminine Secondary School (unequal column heights)
- Zeleznicka Street Apartments (soft ground story)
- Student Club (asymmetry)
- New Technical Faculty, Orce Nikolov apartment building, Student Club, Zeleznicka Street apartment building ("nonstructural" infill stiffened columns and attracted more load to them)

Muto, Kiyoshi, Okamoto, Shunzō, and Hisada, Toshihiko, Report Of The Japanese Earthquake Engineering Mission To Yugoslavia, Tokyo: Overseas Technical Cooperative Agency, October 1963.
"The most remarkable feature of the damage to the buildings whose first story was used as shops was the big distortion of that story caused by the failure of isolated columns." (p. 10). As in the case of Caracas in 1967, the combination of a reinforced concrete frame/masonry infill construction system and soft first story configuration was shown to be especially vulnerable.

1964 Alaska

Ayres, J. Marx, Sun, Tseng-Yao, and Brown, Frederick R., "Report On Non-Structural Damage To Buildings Due To The March 27, 1964 Alaska Earthquake," National Academy of Sciences, August 15, 1967.
- Hodge Building (seismic separation joint and piping detail performed well)
- Elmendorf Air Force Base Hospital (damage to piping crossing joints)
"In tall structures the seismic separations between structural units often can exceed 12 inches and design engineers should try to avoid all pipe crossings on the upper floors." (p. 74)

Berg, Glen V., and Stratta, James L., "Anchorage And The Alaska Earthquake Of March 27, 1964," Earthquakes, Washington, D.C.: American Iron and Steel Institute, 1975.
- J.C. Penney Building (torsion)
- West Anchorage High School (L-shaped plan)

Meehan, John F., "The Reponse Of Several Public School Buildings In Anchorage, Alaska, To The March 27, 1964 Earthquake," in Fergus J. Wood, editor, The Prince William Sound, Alaska, Earthquake Of 1964 And Aftershocks, Washington, D.C.: Government Printing Office, 1967, (Coast and Geodetic Survey).
- West Anchorage High School (L-shaped plan)
- Clark Junior High School (T-shaped plan)

Steinbrugge, Karl V., Manning, John H., and Degenkolb, Henry J., "Building Damage In Anchorage," in Fergus J. Wood, editor, The Prince William Sound, Alaska, Earthquake Of 1964 And Aftershocks, Washington, D.C.: Government Printing Office, 1967, (Coast and Geodetic Survey).
- Hodge Building (separation joints)
- Pepsi-Cola Plant (setback, or unequal roof heights)
- Knik Arm Plant, Chugach Electric Association (asymmetric disposition of mass)
- Mt. McKinley Building (weak corners, holes in shear walls)
- Anchorage-Westward Hotel (pounding)
- J.C. Penney Building (asymmetric wall layout, torsion)
- Elmendorf Hospital (separation joints, re-entrant corner)
- Alaska Methodist University (pounding)
- Knik Arms Apartment House, Hodge Building, St. Mary's Residence, Alaska Native Hospital (re-entrant corner forms, but good performance)
- Four Seasons Apartment House: In addition to rebar splice and slab connection details, the authors emphasize the inequality of strength and stiffness between the building's two cores. "There is really no accurate way of determining the relative stiffness of these walls and applying deflection formulas to them. Judgement must inevitably play a large role... Any one of the following factors may change a designer's estimate by 100 percent." (1) foundation rocking, (2) vertical load distribution, (3) effect of holes in walls, (4) inelastic, cracked section behavior. "It would appear that the safest approach to this design problem, at present, is to use shear walls that are completely determinate... (1) A single shear wall centrally placed, (2) two or more symmetrically placed shear walls of identical dimension and layout, (3) two shear walls so placed that each carries a determinate amount (such as the reactions at the ends of a simple span beam), and (4) a series of shear walls where conditions of (1), (2), or (3) would result after some of the existing walls not needed in the analysis had failed... This is the first known collapse of a multistory

structure in North America due to reinforced concrete shear wall failure and also the first case of shear wall tower complete failure by turning over." (p. 191-192)

1967 Venezuela

Hanson, Robert D., and Degenkolb, Henry J., "The Venezuela Earthquake, July 29, 1967," in Earthquakes, Washington, D.C.: American Iron and Steel Institute, 1975.

Portland Cement Association, "Preliminary Report: The Behavior Of Reinforced Concrete Structures In The Caracas, Venezuela Earthquake Of July 29, 1967," Portland Cement Association, n.d.
Palace Corvin: "consisted of two similar buildings connected by a structurally independent elevator and stairway core. The west wing, which withstood the earthquake, had partitions extending to the ground floor... The collapsed wing was reported to be similar in construction except that the lower floor was open for parking with freestanding columns." (p. 13)
- Neveri (asymmetry)
- Macuto Sheraton (short column, discontinuous shear wall)
- Mobil (exterior asymmetric core)
- Plaza I (good performance, H-plan)
- Naiguata Beach House (unequal column heights)
- Atlantic (inverted setback)

Caromay: "The shape of the building has a profound effect on the response to lateral forces. For example, the curved Caromay building responded as a vertical cylindrical shell and not as individual radial frames as designed." (p. 26)
"Very frequently the partitions and exterior walls are terminated in the first story to permit an open ground floor for commercial use... The resulting sudden change in stiffness from infilled partitions to freestanding columns affected the behavior of these structures..." (p. 8)
"Corner columns were most susceptible to compression failures, since the infill walls and partitions created a rigid shear wall that responded like a cantilever with high overturning moments and resultant high axial forces in corner columns." (p. 10)
"The staircase cast monolithically with the frame behaved like a truss, and because of its rigidity, it attracted large lateral forces." (p. 12)

Steinbrugge, Karl V., "Appendix A: Comparative Building Damage," in A.F. Espinosa and S.T. Algermissen, A Study Of Soil Amplification Factors In Earthquake Damage Areas, Caracas, Venezuela, Washington, D.C.: Government Printing Office, 1972, (National Oceanic and Atmospheric Administration).
Stair/elevator cores were often externalized, sometimes forming the link between two dwelling unit towers to produce a dumbell or H plan; or complex plans were devised to produce light courts: Palace Corvin, San Jose Building, Coral, Cyress Garden, Covent Garden, Mene Grande, Royal San Bosco, Union, Pasaquire, Amalfi, Guipelia, Le Roc, Club Puerto Azul. The Mobil building had an eccentric external core which was intentionally left as a frame, rather than strengthening with walls, following the earthquake, so that torsion problems would not be created.
Soft ground stories were endemic due to the typical masonry infilling of the frame, (for external enclosure and for internal room separations) in all upper stories while the ground story columns were left free standing (to allow for parking, shops, or lobby): San Bosco, Laguna Beach Club, Blue Palace, Altamira, Sucre, Catia La Mar, Amalfi, Petunia II, Maria Luisa, San Jose,

Palace Corvin, Mijagual, Neveri, Marco Aurelio.
- Naiguata Beach Club (unequal column height)
- Macuto Sheraton, Gran Hotel, Lang, Nobel (pounding)
- Caromay Building (asymmetric shear wall arrangement at basement level but no apparent torsional damage)

From Hanson's and Degenkolb's conclusions: "The addition of the shear walls [to a damaged building], unless judiciously placed, could introduce torsion problems where none had existed and the building could have lower total strength than it had before reinforcement." (p. 309)

"The haphazard placement of non-structural elements can introduce torsion where none was intended." (p. 312)

"Most of the taller apartments had many tile partitions and tile exterior walls that acted as shear walls, at least until the tile failed. The ground floor, however, was often devoted to commercial space or automobile parking so the tile walls were not continued to the ground. This concentrated the forces, deformation and energy absorption in the first story with the consequent damage at that point... There is a strong architectural tendency throughout the world to have an open first floor - to place the building on "stilts" as it were. As one structural engineer put it - 'Architects like to build their buildings with no visible means of support.' It cannot be emphasized too strongly that current earthquake code requirements are not based on this type of dynamic stiffness distribution, and potentially a great amount of trouble should be expected where these buildings are built to minimum code requirements in areas subject to great earthquake shocks. The damage to many buildings in Caracas gives ample warning as to what lies ahead on the West Coast of the United States." (p. 314)

1968 Tokachi-oki

Ogura, Koichiro, "Outline Of Damage To Reinforced Concrete Construction," Ziro Suzuki, editor, General Report On The Tokachi-Oki Earthquake Of 1968, Tokyo: Keigaku Publishing Co.,Ltd., 1971.

Buildings are divided into configuration categories: 1. "Box type reinforced concrete structures whose quantity of walls amount to more than 10cm/m^2 [one lineal foot per 32-1/2 square feet] or so in both span direction and longitudinal direction, remained unhurt without any remarkable cracks on their walls..." 2. In reinforced concrete wall/frame structures having about 5 cm/m^2 wall length/floor area ratios, "arranged in good balance," the walls were cracked but frames were only slightly cracked. Strong beam/weak column situations, such as commonly occurred in schools and in a railway station, more serious column damage occurred. 3. Reinforced concrete frames having either weak walls or no walls performed the worst. (Frame design not necessarily state-of-the-art.) (pp.484-485)
- Misawa Commerce High School (separation joint, strong beam/weak column)
- Hachinohe Higashi High School (strong beam/weak column)
- Hachinohe City Office (asymmetry, amount of walls, soft top story)
- Okamisawa Primary School (exterior core)
- Kitazato University (more longitudinal walls than usual along corridor, good performance)

Okada, Tsuneo, et al, "Analysis Of The Hachinohe Library Damaged By '68 Tokachi-Oki Earthquake," Proceedings of the U.S.-Japan Seminar On Earthquake Engineering With Emphasis On The Safety Of School Buildings, 1970, Tokyo: Japan Earthquake Engineering Promotion Society, 1971.
- Hachinohe Library (asymmetry, torsion)

Shiga, Toshio, Shibata, Akenori, Onose, Junichi, and Hagio, Kenji, "Torsional Response Of Structures To Earthquake Motion," Proceedings Of The U.S.-Japan Seminar On Earthquake Engineering With Emphasis On The Safety Of School Buildings, 1970, Tokyo: Japan Earthquake Engineering Promotion Society, 1971.
The damage pattern experienced by the Kamikita Agricultural Cooperative Office in the 1968 Tokachi-oki earthquake indicates torsional motion, and this could be predicted from the asymmetrical wall layout.

Shiga, Toshio, "Earthquake Damage And The Amount Of Walls In Reinforced Concrete Buildings," Sixth World Conference On Earthquake Engineering, 1977.
Analysis of 245 buildings in terms of wall area/floor area and column area/floor area indexes and average nominal ground story shear stress/damage correlations. With the exception of the addition of the shear stress index, this study is similar to an earlier statistical study of the 1923 Kanto earthquake (K. Nakagawa, I. Kamei, and S. Kokusho, "Relation Between Earthquake Damage And The Amount of Walls In Reinforced Concrete Buildings," Transactions of the Architectural Institute of Japan, No. 60, 1958 (in Japanese).

1970 Peru

Fattal, S.G., "Structural Performance Of Low-Cost Housing And Community Buildings Under Earthquake Conditions," in William F. Reps and Emil Simiu, editors, Design, Siting And Construction Of Low-Cost Housing And Community Buildings To Better Withstand Earthquakes And Windstorms, Washington, D.C.: Government Printing Office, 1974, (National Bureau Of Standards).
- apartment house, Chimbote (excessive wall openings) (p. 16)
- house, Casma (unbalanced wall arrangement)

1971 San Fernando, California

Barnes, S.B., and Pinkham, C.W., "Holy Cross Hospital," in Leonard Murphy, co-ordinator, San Fernando, California, Earthquake Of February 9, 1971, Washington, D.C.: Government Printing Office, 1973, (National Oceanic and Atmospheric Administration).
- Holy Cross Hospital (L-shaped plan)
"The north wing was of sufficient size to be a significant dynamic irregularity. Separation of this wing with an independent lateral force - resisting system would have improved the overall performance of the building." (p. 253)

Frazier, G.A., Wood, J.H., and Housner, G.W., "Earthquake Damage To Buildings," in Paul C. Jennings, editor, Engineering Features Of The San Fernando Earthquake Of February 9, 1971, Earthquake Engineering Research Laboratory, California Institute Of Technology, June 1971.
- Olive View Hospital (soft story)
- house-over-garage dwellings in Sylmar (also split-level)
"The large garage area with a nearly open wall for the door has insufficient lateral strength to support the horizontal inertial loads of the overhead structure." (p. 290)

Gates, William E., "KB Valley Center," in Leonard Murphy, co-ordinator, San Fernando, California, Earthquake Of February 9, 1971, Washington, D.C.: Government Printing Office, 1973, (National Oceanic and Atmospheric Administration).
KB Valley Center: Seismic separation joints were used on two sides of the building to isolate an adjacent parking structure and a masonry fire wall and to hence maintain symmetricality.

Goers, Ralph W., A Methodology For Seismic Design And Construction Of Single-Family Dwellings, Palo Alto, California: Applied Technology Council (under contract to HUD), September 1976.
- houses in Sylmar (asymmetric wall layout because of garage openings, soft story)

Johnston, Roy, G., and Strand, Donald, R., "Olive View Hospital," in Leonard Murphy, co-ordinator, San Fernando, California, Earthquake Of February 9, 1971, Washington, D.C.: Government Printing Office, 1973, (National Oceanic and Atmospheric Administration).
- Olive View Hospital (soft story, discontinuous shear wall, column shape, seismic separations, torsion)

Lew, H.S., Leyendecker, E.V., and Dikkers, R.D., Engineering Aspects Of The 1971 San Fernando Earthquake, Washington, D.C.: Government Printing Office, 1971, (National Bureau of Standards Building Science Series 40).
- V.A. Hospital (lack of cross walls)
- Olive View Hospital (soft story)
- house-over-garage dwellings in Sylmar
"Collapse frequently occurred due to a wide open-space provided by the garage door in the end wall, which offered little resistance." (p. 244)

Mahin, Stephen, Bertero, Vitelmo V., Chopra, Anil K., and Collins, Robert G., Response Of The Olive View Hospital Main Building During The San Fernando Earthquake, Earthquake Engineering Research Center, University of California, Berkeley, October 1976.
- Olive View Hospital (mass distribution, discontinuous shear walls, pounding, torsion, column shape, soft story)

McClure, Frank E., Performance Of Single Family Dwellings In The San Fernando Earthquake Of February 9, 1971, Washington, D.C.: HUD, May 1973, (NTIS #PB-226-293).
- several houses in Sylmar, California, (house-over-garage and split level)
"The performance of the dwellings was influenced primarily by the number of stories and the design configuration of the dwellings... The primary cause of the overall damage to the dwellings was the lack of adequate lateral bracing in the bracing walls. This was caused by excessive door and window openings..." (p. 6)

Pinkham, Clarkson W., "Summary Of Conclusions And Recommendations," in Leonard Murphy, co-ordinator, San Fernando, California Earthquake Of February 9, 1971, Washington, D.C.: Government Printing Office, 1973, (National Oceanic and Atmospheric Administration).
Several subjects are configuration-related: split-level houses, torsion, corner columns, differing dynamic characteristics among building portions, separation joints, weak links due to complex stress paths, opening locations in walls.

Steinbrugge, Karl V., Schader, Eugene E., and Moran, Donald F., "Building Damage In San Fernando Valley," in Gordon B. Oakeshott, editor, San Fernando, California, Earthquake Of 9 February 1971, Bulletin 196, California Division of Mines and Geology, 1975. Statistical analyses of damage point to interesting conclusions, such as "One-story dwellings performed substantially better than did two-story dwellings, and much better than combination one-and-two-story dwellings." (p. 334)

Wheeler & Gray, Consulting Engineers, "Vector Electronics" in Leonard Murphy, co-ordinator, San Fernando, California, Earthquake Of February 9, 1971, Washington, D.C.: Government Printing Office, 1973, (National Oceanic and Atmospheric Administration).
- Vector Electronics building (L-shaped plan)

Yanev, Peter, Peace Of Mind In Earthquake Country, San Francisco: Chronicle Books, 1974.
- house-over-garage and split level dwellings (soft story, torsion) (p. 127, 199, 201, 208)
- mobile homes (soft story) (p. 177)
- commercial buildings, San Fernando Valley (pounding) (p. 119)

1972 Managua, Nicaragua

McLean, Ralph, S., "Three Reinforced Concrete Frame Buildings, Managua Earthquake, December 1972," Managua, Nicaragua Earthquake Of December 23, 1972, EERI Conference Proceedings, Earthquake Engineering Research Institute, November 1973, Vol. 1.
- Social Security Building (T-shaped plan, slab shear failure along line of junction of legs where shaft holes were located)
- Other papers dealing with the core layout of the Banco Centrale and Banco Americano Buildings are relevant as well.

Wyllie, Loring A., Jr., and Poland, Chris D., "A Documented Vertical Acceleration Failure," Second U.S. National Conference On Earthquake Engineering, 1979.
This paper, concerning a building damaged in the 1972 Managua earthquake, is relevant to the configuration subject because it deals with vertical, rather than horizontal earthquake forces. Although vertical seismic forces are generally considered to be adequately accounted for via gravity loads safety factors, seismic forces can be exerted upward as well as downward, and unusual configurations may be especially vulnerable to their action. A reminder that seismic forces should be presumed to come from any direction, even if the vertical components may often take care of themselves.

1974 Lima, Peru

Moran, Donald, et al, Engineering Aspects Of the Lima, Peru Earthquake Of October 3, 1974, Earthquake Engineering Research Institute, May 1975.
Agricultural University (masonry infill creating short column effect caused damage in the previous 1966 earthquake; subsequently the stiffening effect of the enclosure wall was eliminated by cutting out a vertical strip of masonry on each side of each column and filling the gap with compressible filler). "These modifications were successful in reducing damage" in the 1974 earthquake. (p.39)

1975 Lice, Turkey

Yanev, Peter I., The Lice, Turkey, Earthquake Of September 6, 1975, URS/John A. Blume & Associates, published in Earthquake Engineering Research Institute Newsletter, Vol. 9, No. 6B, November 1975. (Lice is pronounced Lee-ja).
Minaret: spiral cracking pattern subsequently identified by Yanev as due to spiral stairway location.

The Turkish seismic code, and a plan for a mass education campaign for self-built housing, are reprinted in appendices. The Turkish code is one of several codes in use in the world (but not including any U.S. codes) which offers detailed prescriptive rules of thumb for the layout of small non-engineered buildings: size of wall openings and rooms, length of unperforated walls at corners, etc.

1976 Mindanao, Philippines

Stratta, James L., et al, Mindanao, Philippines Earthquake, August 17, 1976, Earthquake Engineering Research Institute, August 1977.
- Sultan Hotel (torsion due to 3-sided box layout, soft story)
- New Society Hotel (torsion due to two adjacent sides of the box being shear walled, while the other two sides which had exposure due to the corner lot condition were frames; soft story due to first story height)
- Imperial Hotel (soft story due to tall columns, pounding)
Ground motion was apparently predominantly east-west. "It should be noted that the major streets run east to west and most of the structures noted above were located on these streets. Since buildings have large openings on the streets they face, their weaker direction will be parallel to the front of the building. This configuration undoubtedly contributed to some of the problems encountered." (p. 66)
- typical elevated pole-framed, thatched roof dwellings (soft story condition apparently mitigated by ground story diagonal braces which may have been intended as livestock barriers rather than structural members).
- well-to-do wood frame dwelling, lumberyard, Cotabato City (soft story: second level living area was shear walled by wood paneling; ground level consisted only of wood posts to allow space for lumber storage beneath dwelling).

1976 Friuli, Italy

Stratta, James L., and Wyllie, Loring A., Reconnaissance Report: Friuli, Italy Earthquakes Of 1976, Earthquake Engineering Research Institute, August 1979.
- houses: "Because of addition upon addition, they had developed L or U shapes or other irregular configurations which compounded the problem of seismic resistance. The homes were also usually two or three stories in height, frequently with shops or restaurants in the first story." (p. 7)
- Elite Condominium (soft ground story due to lack of ground story masonry infill)
- Zorutti Condominium (L-shaped plan separated into two rectangles; unit with fewer ground story infill walls leaned after the May 6 earthquake and collapsed after the September 15 earthquake).
- Snaidero Office Building (holes in shear walls, soft upper story, asymmetric core, coupled core-column)
- Gemona Hospital (soft second story, asymmetric core, torsion, discontinuous shear wall)
- Artegna 3-story structure (soft ground story)

1976 Caldiran & 1977 Palu, Turkey

Bayülke, Nejat, "Behavior Of Brick Masonry Buildings During Earthquakes," Seminar On Constructions In Seismic Zones, Bergamo-Udine, Italy, 1978, International Association For Bridge And Structural Engineering.
Wall length/floor area ratios are correlated with damage. Turkish practice and regulations concerning proportional rules for plan layout, wall openings and vertical distribution of material are discussed.

1977 Romania

Fattal, George, Simiu, Emil, and Culver, Charles, Observations On The Behavior Of Buildings In The Romania Earthquake Of March 4, 1977, Washington, D.C.: Government Printing Office, 1977, (National Bureau of Standards).
Pre-World War II eight - twelve story concrete frame apartment buildings had typical characteristics: "The first story is generally higher than all the rest and is almost void of walls to accommodate stores and other non-residential facilities. In the upper stories, masonry infill walls and partitions are used liberally to provide enclosure for apartment or office space, and to function as lateral bracing against wind action. As a result, the structure is characterized by laterally stiff upper stories resting on relatively flexible columns at the ground level... The end and corner units tend to be particularly non-uniform in layout." (p. 12). Columns were not always aligned from one story to the next.
Extensive damage to this type of construction occurred in both this earthquake of 1977 and in the previous major Bucharest earthquake of 1940.
- Wilson Apartments (soft story)
- Snagov Street residential building (large story height, holes in walls, slender piers)
- Dunres Building (soft story, setbacks)
- Lido Building (U-shaped plan)
- Casata Building (pounding, setbacks)
- Turist Building (soft story)
- Metalimport Building (setbacks, L-shaped plan)
- Mercerie Building (soft story)

Post-World War II poured-in-place or pre-cast panel concrete systems tended to be designed with symmetrical, cellular plans that were rationalized for structural and production reasons. Only three of the more than 30 buildings which collapsed in Bucharest were in this category.
- Computer Center (lack of redundancy)
- Building 17, new concrete frame construction (unequal column lengths, pounding)

In addition to concrete and masonry detailing and construction practices, the authors singled out soft stories, irregular plans and vertical discontinuities, setbacks, and pounding as major issues in their conclusions. (p. 68-69)

Iordăchescu, E., Zorapapel, T., Marcovici, D., and Danci, G., "Statistical Survey Of The Performance Of One Standard-Design Type Of High-Rise Reinforced-Concrete Shear-Wall Apartment Buildings, In Bucharest, During The March 4, 1977 Romania Earthquake," Second U.S. National Conference On Earthquake Engineering, 1979.
This earthquake is distinguished by the relatively unusual conjunction of two conditions: ground motion with marked directionality, and a sample of over 100 approximately identical large apartment buildings. The significance of the orientation of the buildings, and their transverse and longitudinal plan characteristics, are hence emphasized to a great extent.

1978 Miyagi-ken-oki, Japan

Arnold, Christopher, "Architectural Aspects," in Peter I. Yanev, editor, Miyagi-ken-oki, Japan Earthquake, June 12, 1978, Earthquake Engineering Research Institute, December 1978.
- Kinoshita Building (shear wall layout)
- Obisan Building (staircase location, torsion)

- Tonan High School, Tohoku Institute of Technology, Izumi High School (strong beam, weak column)
- Sunnyheights apartment building (L-shaped plan divided into two rectangles via seismic joint; shear wall layout; lack of longitudinal bearing/shear walls)

Cooper, James D., Ellingwood, Bruce R., and Yanev, Peter I., "Engineering Aspects," in Peter I. Yanev, editor, Miyagi-ken-oki, Japan Earthquake, June 12, 1978, Earthquake Engineering Research Institute, December 1978.
- Obisan Building (soft first story, torsion)
- Paloma Building (asymmetry; columns at one end, shear wall at the other)
- Tonan High School (shear wall layout; walls were used along the less heavily shaken axis only, resulting in extensive damage. A nearby similar structure, oriented at right angles, was undamaged)
- Concrete Batch Plant, Sendai (structural units of disparate dynamic properties damaged at their interfaces)

Kawamata, Shigeya, and Ohnuma, Masaaki,"Strengthening Effect Of Eccentric Steel Braces To Existing Reinforced Concrete Frames," Seventh World Conference On Earthquake Engineering, 1980.
Describes damage suffered by a large concrete frame building at Tohoku University as well as the innovative rehab system devised to strengthen it: external steel x-bracing. Elimination of the strong beam/weak column problem caused by deeper north spandrels was accomplished by drilling holes in these spandrels near the column joint.

1978 Santa Barbara, California

Miller, Richard K., and Felszeghy, Stephen F., Engineering Features Of The Santa Barbara Earthquake Of August 13, 1978, Earthquake Engineering Research Institute, December 1978.
Several buildings discussed in Chapter 7, (North Hall, Anacapa Residence Hall, Engineering Building) illustrate interesting issues concerning shear wall layout. North Hall's seismic rehabilitation, involving the addition of shear walls in a very symmetrical layout, was completed two years before the earthquake, which produced approximately 1/2g peak ground acceleration and 1g upper story acceleration and which was directionally weighted along the building's transverse axis, which was the axis most strengthened by the rehab project. Damage was moderate during this severe but brief ground shaking. Comparison of pre- and post-rehab plans provokes speculation on the question of whether the pre-rehab plan, without the extra interior transverse shear walls, would have suffered extreme damage.

1979 Imperial Valley, California

Arnold, Christopher, "Architectural Implications," in David J. Leeds, editor, Reconnaissance Report: Imperial County, California, Earthquake October 15, 1979, Earthquake Engineering Research Institute, February 1980.

D. Research Reports

Analytical or experimental work concerning configuration-related topics.

general

Blume, John A., and Jhaveri, Dilip, "Time History Response Of Buildings With Unusual Configurations," Fourth World Conference On Earthquake Engineering, 1969.
"The earthquake response analyses of unusual buildings - whether 'unusual' refers to geometry, to size, to shape, to asymmetry, and/or to risk and the consequences of failure - can now be, and should be, accomplished by time-history or other realistic methods... Generally, the design values obtained from normal seismic code provisions applied to unusual structures, are much less than those obtained from specific response analyses." (Volume II, p.A3-164)

Blume, John A., "On Instrumental Versus Effective Acceleration, And Design Coefficients," Second U.S. National Conference On Earthquake Engineering, 1979.
The paper is primarily concerned with the broader question of why buildings which are designed for ostensibly small base shears perform well even though earthquakes produce what appear to be excessive acceleration. Tangentially, two configuration topics are mentioned: ground story ductility as a convenient indicator of overall building strength, and redundancy as a significant and desirable attribute.

Clough, Ray W., Benuska, K. Lee, and T.Y. Lin And Associates, FHA Study Of Seismic Design Criteria For High-Rise Buildings, (HUD TS-3, August 1966).
Section 2 analyzes a hypothetical high-rise design (Building E/20) with varying amounts and layouts of shear wall. The analysis of the total collapse of the Four Seasons Apartment Building in the 1964 Alaska earthquake, although it emphasizes a non-configuration cause (bond failure at splice points in the core wall reinforcement), is of configuration interest in showing the configuration-caused structural disparity between two ostensibly similar cores.

Clough, R.W., "The Finite Element After Twenty-Five Years - A Personal View," Engineering Application Of The Finite Element Method, A.S. Computas, Norway, 1979.
"Almost a decade ago I became concerned that the advancement of structural analysis capabilities was progressing much more rapidly than was knowledge of the basic material and structural component behavior mechanisms, at least for the non-linear response range. This deficiency of experimental data was particularly evident in the field of earthquake resistant design, where the structural performance must be evaluated during large cyclic excursions into the non-linear range. Therefore, during most of the past decade I have followed this alternative path of dynamic experimental research, and have been involved only peripherally with recent developments in the finite element field.
At the outset of this review, it is important to express my concern over the tendency for users of the finite element method to become increasingly impressed by the sheer power of the computer procedure, and decreasingly concerned with relating the computer output to the expected behavior of the real structure under investigation." (Volume I, p. 1.2)

Leutheusser, Hans J., "Influence Of Architectural Features On The Static Wind Loading Of Buildings," in R.D. Marshall and H.C.S. Thom, editors, Proceedings Of Technical Meeting Concerning Wind Loads On Buildings And Structures, January 27-28, 1969, Washington D.C.: Government Printing Office (National Bureau Of Standards), 1970. Wind tunnel tests of variously shaped building models: rectangular prisms, cylinders, shapes with and without parapets and overhangs, and effects of protruding mullions. Makes no reference to seismic, and wind forces and seismic forces are distinctly different in nature, but form is of comparable importance in both fields and the use of dynamic model testing (wind tunnel tests in this case, or shake table tests for the earthquake problem) to study the significance of configuration seems closely analogous.

Okamota, T., and Yokoyama, M., "Earthquake-Resistant Design Theory For Prefabricated Structures Using Checkered Shear Wall Frames," Sixth World Conference On Earthquake Engineering, 1977. Discussion of structural, architectural, and constructional aspects.

Penzien, Joseph, and Chopra, Anil K., "Earthquake Response Of Appendage On A Multi-Story Building," Third World Conference On Earthquake Engineering, 1965. Relationships between the mass of the building and mass of appendage (equipment, penthouse, etc.) and between periods of building and periods of appendage are discussed.

Severud-Gruzen-Turner, Seismic Design: Cost Impact On High-Rise Residential Structures, NTIS #PB278352 (HUD), September 1977. Comparative analysis of steel, reinforced concrete, brick, and concrete block mid-rise apartment buildings of identical plans and elevations.
"It would be more economical, and have less cost impact on seismic upgrading if a structural framing scheme were employed keeping seismic resistance in mind at the design stage, regardless of material, rather than forcing a solution on a pre-determined non-seismic resistive framing system." (p. 80)

Stephen , R.M., and Wilson, E.L., "Dynamic Behavior Of A Pedestal Base Multistory Building," Second U.S. National Conference On Earthquake Engineering, 1979. The configuration of this existing building is quite novel. The bottom twelve stories comprise a smoothly tapering inverted pyramid of concrete wall construction; the upper thirty stories are steel frame. This configuration can be clearly "read" from the forced vibration and analytical mode shapes. In a generic sense, the inverse of the soft story case.

torsion

Ayre, R.S., "Interconnection Of Translational And Torsional Vibrations In Buildings," Bulletin of the Seismological Society of America, Vol. 28, No. 2, (April 1938). Analysis and small scale dynamic testing of physical models at Stanford. "The chief purpose is to show that horizontal translations and rotations about vertical axes are usually interconnected." (p. 89). Ayre points out that symmetry can mean both geometric symmetry of a plan form and also the coincidence of the center of rigidity and center of gravity. Symmetrical buildings are presumed desirable for three reasons: 1) elimination of torsion, 2) simplification of analysis and 3) less chance of resonance. "Certainly, in designing to resist such phenomena as earthquakes, for which there are relatively few engineering data, simplicity seems worth striving for." (p. 130)

Bouwkamp, J.G., and Blohm, J.K., "Dynamic Response Of A Two-Story Steel Frame Structure," Bulletin of the Seismological Society of America, Vol. 56, No. 6, (December 1966).
Forced vibration tests of a building with an asymmetrical wall layout due to corner lot location: two adjacent walls concrete block, the other two sides steel rigid frame only.

Bustamente, Jorge I., and Rosenblueth, Emilio, "Building Code Provisions On Torsional Oscillations," Second World Conference On Earthquake Engineering, 1960.
"Damage from torsional oscillations due to earthquakes has been observed in many buildings." "Dynamic eccentricity may exceed statically computed values." (p. 879). Applicability of one-story analysis procedures to the multi-story case is discussed with an example.

Cardona, R., and Esteva, L., "Static Analysis Of Asymmetric Multi-Story Structures," Sixth World Conference On Earthquake Engineering, 1977.
Applies single-story analytical techniques to multi-story case.

Elms, D.G., "Seismic Torsional Effects On Buildings," Bulletin Of The New Zealand National Society For Earthquake Engineering, Vol. 9, No.1, March 1976.
"Horizontal torsional effects in buildings are not well understood. However, they are thought to have contributed to a number of serious failures... During dynamic testing even completely symmetric buildings or models have usually shown considerable torsional effects at some stage." (p. 79). New Zealand code provisions are more severe than those in use in the U.S.

Irwin, A.W., and Andrew, N., "Performance Of Model Reinforced Concrete Cores For Tall Buildings Cycled In Torsion," Sixth World Conference On Earthquake Engineering, 1977.
Static inelastic tests of small scale cores while undergoing applied torques.

Ishac, M., and Heidebrecht, A., "Dynamic Response Of Asymmetric Shear Wall-Frame Building Structures," Sixth World Conference On Earthquake Engineering, 1977.
A symmetrical frame with a shear-walled core at one end of the plan is analyzed. "The dynamic reponse of eccentric core buildings can give eccentricities much larger than normal values calculated in the code; even doubling the calculated value to get a design eccentricity may not be adequate." (p. 1363)

Keintzel, Einar, "On The Seismic Analysis Of Unsymmetrical Multi-Storied Buildings," Fifth World Conference On Earthquake Enineering, 1973.
Coupling of translational and torsional vibrations.

Krishnamoorthy, G., Young, G.A., and Hegemier, G.A., "Prediction Of The Torsional Response Of A Multi-Story Reinforced Concrete Masonry Building By A Three Dimensional Dynamic Analysis," Fifth World Conference On Earthquake Engineering, 1973.
"(I)nnovative architectural features may result in complex, dynamic response and consequent structural overstress not predictable by a static analysis..." (p. 129)

Mazilu, P., Sandi, H., and Teodorescu, D., "Analysis Of Torsional Oscillations," Fifth World Conference On Earthquake Engineering, 1973.
Considers implications of the fact that "seismic motion is non-synchronous at different contact points of ground and structure." (p. 154)

Medearis, Kenneth, "Coupled Bending And Torsional Oscillations Of A Modern Skyscraper," Bulletin of the Seismological Society of America, Vol. 56, No. 4, (August 1966).
Description of analysis used for the L-shaped 43 story Wells Fargo Building in San Francisco.

Newmark, Nathan M., "Torsion In Symmetrical Buildings," Fourth World Conference On Earthquake Engineering, 1969.
Concludes that a uniform accidental torsion provision (typically 5% eccentricity) is not rationally related to differing torsional situations. "Buildings with short periods or high frequencies should be designed for a larger value of eccentricity than buildings with long periods or low frequencies. The UBC, for example, may not provide sufficient torsional resistance for framed buildings with periods shorter than about 0.6 sec. or shear wall buildings with periods shorter than about 1 sec. A value of accidental eccentricity of 10 percent of the longer plan dimension would be reasonable for shorter periods, possibly even increasing to 15 percent at a period of 0.2 sec." (p. A3-29). "When yielding occurs because of local weakness it almost certainly will occur at one extreme point [in plan]. Then the symmetry of the structure is impaired and further eccentricities are caused which makes yielding develop further and faster in the already yielded member, since the center of resistance moves further away from the yielded member. This indicates that yielding in torsion may be much more serious than yielding in flexure or in linear displacement, and the design should provide greater assurance of resistance to torsional yielding than to other types of yielding. For this reason, corner columns or end shear walls should be more conservatively designed than other members of the structure." (p. A3-28)

Penzien, Joseph, "Earthquake Response Of Irregularly Shaped Buildings," Fourth World Conference On Earthquake Engineering, 1969.
"To reduce the seismic forces in a building caused by an eccentricity, it should be designed so that its fundamental period in torsion differs considerably (preferably higher) from the first lateral vibration mode of the building." (Volume II, p.A3-83)

Prasad, B.K. Raghu, and Jagadish, K.S., "The Inelastic Torsional Response Of Structures To Earthquake Motions," Sixth World Conference On Earthquake Engineering, 1977.
Eccentricity is varied in increments to see the results on frame ductility requirements.

Rutenberg, A., Tso, W.K., and Heidebrecht, A.C., "Dynamic Properties Of Asymmetric Wall-Frame Structures," Earthquake Engineering And Structural Dynamics, Vol. 5, No. 1, January-March 1977.
"The purpose of this paper is to present a simple approximate method for the evaluation of the natural frequencies and mode shapes for a class of uniform asymmetric wall-frame structures." (p. 42)

Shiga, Toshio, "Torsional Vibration Of Multi-Storied Buildings," Third World Conference On Earthquake Engineering, 1965.
The analytical example is a five-story L-shaped building.

Skinner, R.I., Skilton, D.W.C., and Laws, D.A., "Unbalanced Buildings, And Buildings With Light Towers, Under Earthquake Forces," Third World Conference On Earthquake Engineering, 1965.
"Torsionally unbalanced buildings, and buildings surmounted by relatively light towers, have special sensitive features which influence the seismic forces and movements in them. This sensitivity arises when a natural period of the building in translational movement is close to a torsional period in the one case, or to a period of the tower in the other case. Such buildings call for special consideration during dynamic analysis." (p. 586)

Tso, W.K., and Biswas, J.K., "Torsional Analysis Of Core Wall Structures," Fifth World Conference On Earthquake Engineering, 1973.
A simplified core layout, two "channel sections" facing each other, is analyzed for torsion.

size

Barstein, M.F., "Dynamics Of Extended-In-Plan Structures In Strong Earthquakes," Fourth World Conference On Earthquake Engineering, 1969.
Considers the scale effect of large structures (masts, power line supports, large-span single and multi-story buildings) whose dimensions can become as large as that of seismic waves.

Housner, George W., "The Behavior Of Inverted Pendulum Structures During Earthquakes," Bulletin of the Seismological Society of America, Vol. 53, No. 2, (February 1963).
"It is shown that there is an unexpected scale effect which makes the larger of two geometrically similar blocks more stable than the smaller block. It is also shown that the stability of a tall slender block subjected to earthquake motion is much greater than would be inferred from its stability against a constant horizontal force. In light of these facts, the occasional survival of a slender structure that is apparently highly unstable is not surprising." (p. 417).
The unexpectedly good performance of "golf-ball-on-a-tee" water tanks in the 1960 Chile earthquakes is the case in point.

Iguchi, Michio, "A Basic Study On The Behavior Of Long Dimensional Size Buildings During Earthquakes," Sixth World Conference On Earthquake Engineering, 1977.
A "theoretical, though appropriate, analysis of the vertical behavior of long dimensional size structures on the ground, when subjected to the obliquely incident earthquake motion." (p. 1490)

Khachian, E.E., "A Study Of Seismic Influences On Structures Considering Their Length And Height," Sixth World Conference On Earthquake Engineering, 1977.
Considers possible implications of the fact that waves do not propogate instantaneously along the breadth or up the height of a structure.

Korchinskiy, I.L., "Seismic Resistivity Of Extended Structures," Third World Conference On Earthquake Engineering, 1965.
Torsion-generating ground motion in symmetrical structures is discussed.

Polyakov, S., Design Of Earthquake Resistant Structures, Moscow: Mir Publishers, 1974.
Soviet code provisions which tabularize size limits according to construction type and geographic zone, and possible implications of non-instantaneous wave transmissions are discussed.

'soft story'

Chopra, A.K., Clough, D.P. and Clough, R.W., "Earthquake Resistance Of Buildings With A 'Soft' First Story," Earthquake Engineering And Structural Dynamics, Vol. 1, No. 4, April-June 1973.
Under the presumed loading, the sample design's "first story yield mechanism must be designed to accommodate very large displacements, in excess of 1 foot, if it is to be effective with a flexible structure." (p. 4)

Fintel, Mark, and Khan, Fazlur R., "Shock-Absorbing Soft Story Concept For Multistory Earthquake Structures," American Concrete Institute Convention, Los Angeles, March 7, 1968.
A proposed soft ground story shock-absorbing concept in which ground story reinforced concrete columns are designed to completely hinge at relatively low force levels, with ground story walls, isolated from the second floor by a neoprene pad detail, providing stability. Upper levels are supposed to remain elastic. Although the soft story concept in any form, even when advocated by sophisticated engineers as in this case, has been for decades and still is a very controversial idea, it is important to note that it is the unintentional, or accidental soft story which has been built in large numbers and which has contributed to significant damage in many cases, rather than intentional soft story designs. "Base isolation" devices are not essentially different from "soft stories" in theory, but their scale is quite different. Base isolation devices have been generally proposed as taking the form of relatively thin layers of structure (rubber blocks, ball bearings, etc.) inserted between foundation and superstructure, rather than forming an entire story which has isolative characteristics. Whether the future "verdicts" on soft stories and base isolation devices will be similar or not, the soft story is basically a configuration-related variable whereas the base isolation device is considered here more a question of detailing alone without overall building configuration being involved. Unintentional and perhaps intentional soft stories will remain a major seismic configuration topic for the foreseeable future.

Healey, Timothy J., and Sozen, Mete A., "Experimental Study Of The Dynamic Response Of A Ten-Story Reinforced Concrete Frame With A Tall First Story," University of Illinois at Urbana-Champaign, August 1978, (Report No. UILU-ENG-78-2012).
Scaled-down ten-story concrete frame with first and tenth stories 20% taller than the others; shaking table results compared with analysis.

Martel, R.R., "Effects Of Earthquakes On Buildings With A Flexible First Story," Bulletin of the Seismological Society of America, Vol. 19, (September 1929).
A generally negative report on the analysis of one soft story prototype design.

Moehle, Jack P., and Sozen, Mete A., "Earthquake-Simulation Tests Of A Ten-Story Reinforced Concrete Frame With A Discontinued First-Level Beam," University of Illinois at Urbana-Champaign, August 1978, (Report No. UILU-ENG-78-2014).
A scaled-down ten-story concrete frame structure was subjected to 1940 El Centro N-S motion on a shaking table, and performance was compared with analysis. The "deletion" of one second floor beam span created one two-story high ground level column.

strong beam/weak column

Higashi, Yoichi, and Ohkubo, Masamichi, "Static And Dynamic Loading Tests Of Reinforced Concrete Frames With Thin Spandrel Or Wing Walls," Proceedings Of The U.S.-Japan Seminar On Earthquake Engineering With Emphasis On The Safety Of School Buildings, September 21-26, 1970, Sendai, Japan. Tokyo: The Japan Earthquake Engineering Promotion Society, 1971.
In the 1968 Tokachi-oki earthquake, strong beam/weak column damage to schools appeared as a recurrent theme: The typical long, narrow, two or three story reinforced concrete frame structures had very wide spandrels on the north side but on the south, where more glazing was desired, the beams became narrower. Hence one line of columns were shorter than the other side's, and these took more load and failed prematurely in shear. The research program described in this paper was conducted to see the effects of varying the beam/column stiffness and strength ratios.
It is ironic to note that Sendai, the very city where this conference was held during which several papers pointed out this configuration-caused school building problem, later experienced a major earthquake in 1978 (Miyagi-ken-oki earthquake) and several newly constructed schools of the same configuration suffered the exact same patterns of damage, even though reinforcement requirements in the code had been increased in the interim. Integrating engineering research with practice -- architectural as well as engineering practice -- seems to be a world-wide problem.

Kawamata, Shigeya, and Ohnuma, Masaaki, "Strengthening Effect Of Eccentric Steel Braces To Existing Reinforced Concrete Frames," Seventh World Conference On Earthquake Engineering, 1980.
Two Tohoku Institute of Technology buildings were damaged in the 1978 Miyagi-ken-oki earthquake. North-wall frames were stiffer, since spandrels were deeper for less exposure, and hence the strong beam/weak column problem resulted in column shearing damage. One building was demolished, the other was strengthened by adding transverse shear walls and longitudinal exteriorly mounted x-bracing. In addition, to avoid recurrence of the strong beam/weak column effect, spandrels were perforated with holes at intersections with columns to eliminate the stiffness disparity.

diaphragm proportion & size

Blume, John A., Sharpe, Roland, and Elsesser, Eric, "A Structural Dynamic Investigation Of Fifteen School Buildings Subjected To Simulated Earthquake Motion," John A. Blume & Associates, for the California Division Of Architecture, 1960.
"Long, narrow diaphragms tend to have periods that 'tune in' to the most critical part of the earthquake spectrum. Not only are such elements subject to damage, but their reactions affect adjoining parts such as wall supports." (p. XVI-7)

Rutherford and Chekene, Consulting Engineers, "Evaluation Of Seismic Resistance Of Existing Buildings, U.C. Davis Campus: Veihmeyer Hall," May 1973.
Proposed seismic rehab consisted largely of reducing diaphragm span by adding interior shear walls.

Steinbrugge, Karl V., and Schader, Eugene E., "Earthquake Damage And Related Statistics" in Leonard Murphy, editor, San Fernando, California, Earthquake Of February 9, 1971, Washington, D.C.: Government Printing Office, 1973, (National Oceanic And Atmospheric Administration).
Figure 18, p. 707, correlates roof damage with aspect ratios and areas for 40 light industrial buildings. The authors conclude that several different interpretations of the data are possible.

setbacks

Berg, G.V., "Earthquake Stresses In Tall Buildings With Set-Backs," Proceedings, Second Symposium On Earthquake Engineering, University of Roorkee, 1962.
Stepped cantilever beam analysis.

Humar, J.L., and Wright, E.W., "Earthquake Response Of Steel-Framed Multistorey Buildings With Set-Backs," Earthquake Engineering And Structural Dynamics, Vol. 5, No. 1 (January-March 1977).
A comprehensive review.
A brief survey of previous work followed by a dynamic analysis of setback designs with varying configurations, with conclusions as to the significant variables, including: "In some cases the contribution of the second or the third mode may be higher than the contribution of the fundamental mode. A direct consequence of this behavior is that the approximate code method of distributing the base shear on the assumption that the building vibrates predominantly in the fundamental mode cannot be applied to set-back frame buildings with slender towers." (p. 37)
"The provisions of the present codes for the distribution of the base shears throughout the building height underestimate the shears in the upper one third of the building even for a uniform building. For a set-back building, the discrepancy is larger." (p. 37)
"In set-back buildings with very slender towers, the shear coefficient shows a sudden and marked increase in the transition region between the base and the tower - this can be appropriately termed a 'notch effect'." (p. 38)
SEAOC set-back provisions are compared to the authors'.

Jhaveri, D.P., "Earthquake Forces In Tall Buildings With Set Backs," Ph.D. thesis , University of Michigan, 1967.
Two models were used: stepped cantilever beam, and multi-degree-of-freedom with infinitely rigid floors.

Pekau, O.A., "Inelastic Behaviour Of Frame Structures Under Static And Earthquake Forces," Ph.D. thesis, University of Waterloo, 1970.
Includes inelastic considerations and allows for girder flexibility.

Penzien, Joseph, "Earthquake Response Of Irregularly Shaped Buildings," Fourth World Conference On Earthquake Engineering, 1969.
A setback building and an eccentrically stiffened example are analyzed for 1940 El Centro N-S ground motion. "To greatly reduce the seismic forces in a set-back structure (or appendage), it should be designed so that its fundamental period of vibration differs considerably (preferably higher) from the first lateral vibration mode of the building..." (Volume II, p.A3-83)

Skinner, R.I., Skilton, D.W.C., and Laws, D.A., "Unbalanced Buildings, And Buildings With Light Towers, Under Earthquake Forces," Third World Conference On Earthquake Engineering, 1965.
Analyzes effect of similarity or dissimilarity of base periods and tower periods.

separation joints

Holmes, William T., "The Rehabilitation Of History Corner Of The Stanford University Main Quad," Second U.S. National Conference On Earthquake Engineering, 1979.
Configuration problems are emphasized in the discussion of design decisions: L-shaped plan, collectors, contiguous arcade, alternative seismic separation strategies, discontinuous walls.

Rea, Dixon, and Bouwkamp, Jack G., "A Source Of Damping Produced By The Interaction Of Close-Standing Buildings," Bulletin of the Seismological Society of America, Vol. 58, No. 3, (June 1968).
Dynamic field tests during construction of the Health Sciences building at the University of California, San Francisco Medical Center revealed that the sliding plate separation joint added significant damping via friction.

Willsea, Frederick, "Reconstruction Of Margaret Jacks Hall, Stanford University," Second U.S. National Conference On Earthquake Engineering, 1979.
Specific trade-offs involved with two seismic separation decisions are discussed.

E. Codes

Portions of these codes or commentaries deal specifically with configuration. In addition, configuration is indirectly or generally involved in other sections of these codes, since any calculation procedure must inevitably take into account the dimensions of a building and its members, the disposition of its mass, and other configuration variables. The following indicated code sections generally attempt to point out when this "automatic" consideration of configuration variables is not sufficient.

Applied Technology Council, Tentative Provisions For The Development Of Seismic Regulations For Buildings ("ATC-3"), June 1978.
This document is intended to provide the basis for a nationally-applicable seismic code, and includes commentary as well as code-language provisions in its 500 pages. Configuration is mentioned in section 3.4 to differentiate the regular from irregular building. For certain levels of hazard exposure, the irregular case requires "special consideration" of its dynamic characteristics.
"Past earthquakes have repeatedly shown that buildings having irregular configurations suffer greater damage than buildings having regular configurations. This situation prevails even with good design and construction. These provisions are designed to encourage that buildings be designed to have regular configurations." (p. 339, section 3.4 of commentary).
Irregular characteristics in plan include asymmetry, re-entrant corners "with significant dimensions," large torsional eccentricity, diaphragm interruptions; in section: asymmetry about vertical axes or horizontal offsets, variations in mass-stiffness ratio between adjacent stories.

Also briefly discussed in the code and commentary sections are orthogonal effects (3.7.2), vertical discontinuities (3.7.3), redundancy (3.7.4), support for discontinuous components (columns supporting stiff walls or trusses) (11.5.4), holes in shear walls and diaphragms (11.8.3).

Architectural Institute of Japan, Design Essentials In Earthquake Resistant Buildings, New York: Elsevier, 1970.
A commentary on recommended practice: includes a discussion of torsion, vertical discontinuities, proportional guidelines for the amount of shear wall in small non-engineered buildings. The latter section, written with particular traditional Japanese construction systems in mind, raises interesting possibilities concerning the development of simple rules, such as shear wall lineal footage/floor area square footage ratios, for specific U.S. situations such as wood-frame dwellings.

Army, Navy, And The Air Force, Departments Of, (S.B. Barnes & Associates, and John Blume & Associates, consultants), Seismic Design For Buildings, Washington, D.C.: Department of The Army, April 1973.
The "tri-services" manual, a combination design manual/textbook, contains several strongly worded statements on the significance of configuration, a before-and-after example of how seismic awareness can improve a plan layout, a discussion of torsion. "The seismic design begins with and is dominated by the architectural concept of the building... Engineers are learning that a building's shape, symmetry, and its general layout developed in the conceptual stage are more important, or make for greater differences, than the accurate determination of the code - prescribed forces." (p. 3-13). Designed to be used alongside the SEAOC Blue Book.

Berg, Glen V., "Historical Review Of Earthquakes, Damage, And Building Codes," Methods Of Structural Analysis, New York: American Society Of Civil Engineers, 1976.
Discusses the sensitivity of design forces to calculated periods, which in turn are highly configuration-dependent.

Binder, R.W., and Wheeler, W.T., "Building Code Provisions For Aseismic Design," Second World Conference On Earthquake Engineering, 1960.
Discusses historical evolution of the C factor, including the fact that when Los Angeles abolished its 150-foot/13 story height limit in 1957 it had to rewrite the C factor formula to take into account the different behavior of taller structures. Evolution of torsion and setback regulations in the UBC is also covered.

California Department of Health, Title 22 and 24, California Administrative Code, (Hospital Act of 1972).
Passed in 1972 partly in response to the casualties and damage resulting from hospital performance in the 1971 San Fernando, California earthquake, Senate Bill 519 did for California hospitals what the Field Act had done for schools. The code itself is different, and involved the added aim of maintaining the functionality of hospitals immediately following earthquakes, but it is carried out by the same division of the Office of the State Architect. Like the Field Act, it contains many provisions similar to the basic UBC provisions, and also like the Field Act, its general caveat on configuration is more forcefully stated than in the UBC:
"When the design of a structure, due to unusual configuration of

the structure or parts of the structure, does not provide at least the same safeguard against earthquakes as provided by the applicable portions of this section when applied in the design of a similar structure of customary configuration, framing and assembly of materials, the Department shall have the authority to withhold its approval." (T22-2312 (a)2).
The idea of "equivalence" is not usually stated in codes, merely "adequacy." Recently an L-shaped plan was divided into two rectangles with a seismic joint to avoid the more costly dynamic analysis and extra strengthening that would have been otherwise required.

California Office of Architecture And Construction, Title 21, California Administrative Code, (Field Act).
Passed one month after the 1933 Long Beach earthquake, the Field Act regulates the construction of public schools (but not public universities) in California, and subsequent legislation eventually made it retroactively applicable to existing school buildings. Although similar to the Uniform Building Code, its general clause on configuration is more strongly worded:
"When the design of a structure or parts of a structure result in unusual configuration 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."

California Seismic Safety Commission, Task Committee On The Hospital Act Of 1972, Report, May 12, 1977.
The fire vs. seismic conflict concerning separation joints is discussed p. 20-21, and in the letter by Donald Axon, Appendix.

General Services Administration, and Pregnoff, Matheu, Beebe, Inc. consultant authors for structural contents, Earthquake Resistance Of Buildings: Vol.1, Design Guidelines; Vol. 11, Evaluation Of Existing Structures; Vol.111, Commentary On Design Guidelines, 1976.
Based on the UBC although in some respects concerning configuration more cautionary. "If the building is irregular in plan or elevation and/or if resisting elements are not symmetrically placed, or if there are sudden changes in resisting element stiffness" then, for buildings of high exposure occupancy/zone categories, a dynamic analysis is indicated. (For low exposure regular buildings, the UBC is used without modification). (p. I-9). "A significant torsion inducing factor to be considered in the analysis of any structure is the possibility of asymmetric resisting forces of the lateral load resisting elements after some of these elements have yielded or failed." (p. III-6). The commentary, as with the case of the SEAOC Blue Book, is as important as the regulations themselves.

Glogau, O.A., "Some Comments On The New Zealand Earthquake Loading Provisions," Sixth World Conference On Earthquake Engineering, 1977.
Comments on 1976 code (N2S 4203:1976). "For a number of reasons torsional situations are discouraged both by the fairly severe provisions of the code and the commentary. Even the most sophisticated analysis does not account for out of phase ground motions which may damage the junctions of L, T and U shaped buildings...in torsional situations uniform dissipation of energy is rare..." (p. V-52).

Goers, Ralph W. & Associates, A Methodology For Seismic Design And Construction Of Single-Family Dwellings, Palo Alto, California: Applied Technology Council (under contract to HUD), September 1976. A thorough study containing information on damage to houses (including the house-over-garage configuration-induced problem which was prominent in the 1971 San Fernando earthquake), construction details, design/analysis methods, and suggested plan review procedures. Examples of houses with irregular configurations, such as hillside houses on stilts or certain complex roof arrangements, are illustrated to indicate the type of case which must be individually engineered rather than analyzed according to the procedures included in the report. Tributary area/tributary load relationships, shear wall location and proportion, and other configuration factors are emphasized in this approach.

International Association For Earthquake Engineering, Earthquake Resistant Regulations, A World List, 1973, and Supplement, 1976. Concerning the following configuration issues, the codes in use in these nations are of particular interest, especially because there are often significant variations from what is considered standard practice in the U.S.
- torsion: New Zealand, Canada, Mexico, Peru, Argentina, Venezuela, India.
- irregular configurations (complex plans, abrupt vertical changes in strength and stiffness, setbacks): Japan, Yugoslavia, USSR, New Zealand, Spain.
- specific configuration rules for small non-engineered buildings of particular indigenous construction characteristics (window and door opening size and location, cross-wall spacing, corner treatment, etc.): Japan, Argentina, Turkey, Peru, Bulgaria, Italy, Iran.
- absolute and relative dimensional characteristics (seismic height limits, height/depth ratios, maximum plan dimension limits and aspect ratios): Bulgaria, Argentina, USSR.

International Conference Of Building Officials (ICBO),1979 Uniform Building Code (UBC), Whittier, California (5360 S. Workman Mill Road): ICBO, 1979. Earlier editions at three-year intervals. Chapter 23 contains the seismic regulations. (Other chapters also contain provisions specific to the various materials). First enacted 1927. In addition to being the model code (one of four in the US) which predominates in the West, its seismic provisions (derived from the Structural Engineers Association of California document separately listed) have been a model for this aspect of many of the codes in use in the rest of the US and the rest of the world. The configuration references in Chapter 23 are rather brief and are not meant to handle all the problems that can arise.
"Structures having irregular shapes or framing systems. The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure." (2312 (e)3)
Torsion is referenced in 2312 (e)5: 5% eccentricity for accidental torsion.
Setbacks are covered in 2312(e)2: "Buildings having setbacks wherein the plan dimension of the tower in each direction is at least 75 percent of the corresponding plan dimension of the lower part may be considered as uniform buildings..."
Analytical assumptions to use for setback cases are listed as an

appendix in the SEAOC Blue Book, but have never been included in the UBC.

Moran, Donald F., and Bockemohle, L.W., "History And Philosophy Of California Earthquake Codes And Elements Of Lateral Force Design," in Leonard Murphy, co-ordinator, San Fernando California, Earthquake Of February 9, 1971, Washington, D.C.: Government Printing Office, 1973, (National Oceanic And Atmospheric Administration). "Symmetrical structures with simple geometrical plans generally perform better in earthquakes than those with irregular plans and unsymmetrical resisting elements. This does not mean that it is impossible to design the latter to perform equally well. However, the chances for mistakes in both design and construction are more likely in complicated, irregular structures. Also, more assumptions as to actual seismic behavior must be made for unsymmetrical designs." (p.35)

Shah, Haresh C., Zsutty, Theodore C., and Padilla, Luis, "The Purpose And Effects Of Earthquake Codes," Internal Study Report No. 1, The John A. Blume Earthquake Engineering Center, August 1977. "In the 1930's, 40's and 50's the structural engineers of California (with recognition of the experiences of Japanese engineers) generated the basic earthquake code and design procedures which are employed throughout the world today. It is most important to recognize that these engineers had developed these provisions for the types of building construction which were prevalent in California at that time - specifically structures in Los Angeles and San Francisco. These buildings typically had strong steel (with concrete fireproofing) framing skeletons, filled with very well-constructed brick masonry walls and strong concrete flooring systems. They were usually symmetrical and regular in their configuration, and in most cases they qualified as good tough earthquake-resistant structures... However, architectural configurations along with methods of construction have changed significantly in the past two decades... The basic error was that the new buildings did not have the regularity, stiffness, and reserve toughness necessary to justify the classical low design values." (p. 8-9)

Steinbrugge, Karl V., and Degenkolb, Henry J., "Lateral Force Provisions Of The Uniform Building Code, And Practical Design Problems," in Fergus Wood, editor, The Prince William Sound Earthquake of 1964 And Aftershocks, Washington, D.C.: Government Printing Office, 1967, (Coast And Geodetic Survey).
The majority of the practical design problems discussed are configuration issues: location and size of holes in shear walls, core layouts, stair location.

Structural Engineers Association of California, Recommended Lateral Force Requirements And Commentary (The SEAOC "Blue Book"), 1959, 1960, 1963, 1966, 1967, 1968, 1973, 1974 and 1975 editions, San Francisco: SEAOC.
The "Requirements" portion is written in code language, and except for generally minor changes is adopted word-for-word by the International Conference Of Building Officials to provide the seismic portion of the Uniform Building Code. The "Commentary" portion, which is keyed to the "Requirements," explains the reasoning behind the regulations and it must be understood to use the specific rules and calculation procedures intelligently. The Commentary throughout is relevant to the configuration subject but is especially per-

tinent, p. 33-C to 36-C, wherein are described over 20 specific problem configurations ("irregular structures or framing systems"). Specific solutions are assumed to be best devised by reliance on good engineering judgment and practice rather then via specific regulations.
"Due to the infinite variations of irregularities 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." (p. 33-C)

Veterans Administration, Earthquake Resistant Design Requirements For VA Hospital Facilities (H-08-8 and occasional up-dating memoranda), Washington, D.C.: Office of Construction, Veterans Administration, 1973.
The VA adopted a comprehensive aseismic program for its facilities following the 1971 San Fernando earthquake. Although basically similar to the UBC, the VA code involves detailed site studies, higher force levels, some more stringent non-structural requirements, and a team design process in which seismic concerns are strongly voiced by the client's engineering staff and geotechnical and structural consultants. (Typically a special seismic structural consultant is involved in addition to the usual structural engineering consultant). See the case study of the design process involved for the VA hospital in Loma Linda, California, Chapter XIII.

Figure Credits

Complete citations for figure credits marked by an asterick (*) may be found elsewhere in the book, either as references or in the bibliography. Names appearing in brackets, are those of the photographer, if known, and if different than the author.

Those figures for which no credit is given, are by Building Systems Development, Inc. (Christopher Arnold, Robert Reitherman and Dianne Whitaker).

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