ST/ESA/23

Department of Economic and Social Affairs

LOW-COST CONSTRUCTION RESISTANT TO EARTHQUAKES AND HURRICANES

UNITED NATIONS
New York, 1975

NOTE

The designations employed and the presentation of material in this publication do not imply the expression of any opinion whatsoever on the part of the Secretariat of the United Nations concerning the legal status of any country, territory, city or area or of its authorities, or concerning the delimitation of its frontiers or boundaries.

ST/ESA/23

UNITED NATIONS PUBLICATION

Sales No. E.75.IV.7

Price: $U.S. 9.00
(or equivalent in other currencies)

CONTENTS

CONTENTS (continued)

CONTENTS (continued)

LIST OF FIGURES

Page

PART ONE: EARTHQUAKES

LIST OF FIGURES (continued)

PART TWO: HURRICANES

LIST OF TABLES

FOREWORD

Throughout the centuries natural disasters have taken a high toll of human lives and caused great property losses throughout the world. It is estimated that during the last century alone about 1 million persons perished in earthquakes, and over 600,000 in hurricanes, typhoons and cyclones. The cost in property damage has been enormous. In 1966, for example, 58 disasters, mostly in developing countries, caused losses amounting to $2,400 million. Typhoons in South-East Asia are estimated to cause damage amounting to approximately $500 million per year.

Disasters, wherever they occur, concern the whole world. The United Nations therefore gives high priority to social and technical problems associated with disasters, both from the point of view of prevention or preparation and from that of post-disaster emergency measures and the rehabilitation of stricken areas, with particular emphasis on the reconstruction of housing and community installations.

The General Assembly, the Economic and Social Council, the Committee for the Housing, Building and Planning and other United Nations bodies have dealt with the question on various occasions and have adopted important resolutions on the subject. For example, General Assembly resolution 2717 (XXV), adopted unanimously in December 1970 and entitled "Assistance in cases of natural disaster", urges the adoption of measures designed to strengthen the capacity of various organizations of the United Nations system to provide assistance in cases of natural disasters; ways and means of ensuring better mobilization and co-ordination of assistance to be provided through the United Nations and non-governmental organizations; the application of technology and scientific research to prevent and mitigate the effects of disasters; and pre-disaster planning at both the national and the international level.

In recent years various organizations of the United Nations system have given active consideration to the work of mitigating the effects of disasters by providing assistance to Governments to enable them to prepare more effectively for that work. Among others, mention should be made of the Secretariat, the regional economic commissions, the United Nations Educational, Scientific and Cultural Organization (UNESCO), the United Nations Development Programme (UNDP), the Food and Agriculture Organization of the United Nations (FAO) and the World Food Programme (WFP), the World Health Organization (WHO), the United Nations Children's Fund (UNICEF), the World Meteorological Organization (WMO) and, since 1972, the Office of the United Nations Disaster Relief Co-ordinator.

Also of special importance are the efforts of various intergovernmental and non-governmental organizations such as the League of Red Cross Societies, recognized for its key role in organizing relief measures in the first stages of an emergency following a disaster, the Organization of American States, which provides technical and financial assistance, and other organizations.

The United Nations and its specialized agencies have published studies and reports on the social, economic, scientific and technical aspects of natural disasters, which are designed to serve as a guide to Governments, institutions and experts.

The Centre for Housing, Building and Planning of the Department of Economic and Social Affairs is studying various documents on the subject; these include proposed standards for urban planning and a manual on the repairing and strengthening of structures affected by earthquakes, which will be published in the near future.

The present work deals with the technological aspects of design and construction which must be taken into account when putting up new low-cost buildings and auxiliary structures in areas stricken by earthquakes and strong winds. Its purpose is to set forth principles and basic techniques which can serve as a guide to administrators and technicians to enable them to prevent substantial damage to structures subjected to stresses resulting from earth tremors or violent movements of air masses.

The application of the principles set forth in this document does not necessarily mean higher construction costs. In any event, it implies a possible saving of lives and investments in buildings, thus offsetting any small increase in the original cost which might prove necessary, as has already been shown by experience. Both individuals and the authorities are responsible for adequate preventive measures against future disasters. Experts and technicians must see to the proper design and execution of construction projects according to these guidelines. The authorities have the duty to lay down regulations and require compliance therewith, in accordance with the preventive measures suggested in this study.

The present document consists of two parts, prepared by a number of advisers specializing in the subject. The first part is devoted to earthquakes, and the second to hurricanes.

Part One

EARTHQUAKES

INTRODUCTION

1. Earthquakes are natural phenomena which occur frequently in certain regions of the world. When they strike populated areas, the consequences are disastrous in terms of loss of life and property damage.

2. Although the regions of greatest seismic activity are clearly defined and although there are scientific theories on what causes earthquakes, it is not yet possible to forecast the time and place of a destructive earthquake. Moreover, even if such forecasting were possible, it would only reduce the number of lives lost but would not prevent property damage.

3. Hence the best way to deal with seismic activity is to design and construct buildings, including dwellings and infrastructure works in such a manner that they will adequately withstand strong earthquakes. In addition, people must be educated concerning the nature of earthquakes and, in particular, concerning safety conditions in their physical surroundings, so that they will be able to react rationally and not panic when a tremor occurs.

4. Studies conducted by the United Nations 1/ have established that the great loss of life and vast destruction caused by earthquakes are due, for the most part, to insufficient attention being given to earthquake resistance in construction design and to the use of inferior construction techniques or materials of sub-standard characteristics or quality.

5. Other causes include unfavourable soils and geological structure of the terrain and the lack of adequate studies that could help in choosing a site or designing foundations to take account of terrain characteristics.

6. Because of this experience, investigations to determine earthquake-resistant design and construction methods have been carried out and standards have been laid down. These, when strictly observed, have considerably increased the safety of buildings. An example is the case of Concepción, Chile, which was almost completely destroyed by the 1939 earthquake. In 1960 a second earthquake occurred, but this caused practically no damage to the buildings and houses which had been built in accordance with the building code enacted in 1939.

7. These considerations demonstrate the importance of spreading a knowledge of earthquake engineering - whose systematic advances in recent years make it possible to design earthquake-resistant structures safely and economically - and of the benefits that can be obtained by incorporating the relevant recommendations and restrictions into national and local codes and regulations.

8. The present study outlines in a simplified form the problem of making buildings earthquake-resistant, with particular emphasis on one-storey low-cost housing or multi-family dwellings. Its purpose is to help the professional responsible

---

1/ UN/UNESCO report on international co-operation in the field of seismological research, seismology and earthquake engineering (E/3617).

for designing, constructing or repairing buildings in seismic regions to evaluate the various determining factors in the light of current scientific and technical knowledge.

9. The material in this document has been set out in a logical order. Chapter I gives a series of basic facts about seismology, in particular, the theories most widely accepted today concerning the origin of earthquakes, their magnitude and intensity and the most common scales for measuring them.

10. Chapters II, III and IV deal with the effect of earthquakes on buildings and the behaviour of simple and complex structural elements when subjected to horizontal forces. In these chapters an attempt has been made to review the basic concepts of structural analysis, resistance of materials and structural dynamics.

11. Chapter V deals with the foundation soil. The soil and its characteristics are fundamental to a structure's earthquake-resistance, and the influence of soil on the seismic characteristics of a location makes it a factor of the first importance in earthquake engineering. The complexity of the behaviour of the soil and its interaction with the structure have led to the development of soil mechanics, a highly technical field of specialization. The chapter concludes with some general recommendations for single-storey low-cost housing. Buildings of more than one storey require a special design which takes account of soil characteristics.

12. Chapters VI, VII and VIII deal with the basic aspects of design and construction. Chapter VI describes the static and dynamic methods of analysis, with a view to determining quantitatively the magnitude and characteristics of the resistance which a structure must have in order to withstand an earthquake.

13. Chapter VII contains recommendations that the planner should take into account in designing a building in order to withstand the destructive forces of an earthquake.

14. Chapter VIII includes a series of simple recommendations concerning anti-earthquake construction techniques; buildings constructed without the use of such techniques have suffered serious structural damage during earthquakes.

15. Most of the design and construction recommendations in this document are based on the experience of the authors, who have inspected and reported on the damage caused by various earthquakes, including those which occurred at Concepción, Chile (1960), Valdivia, Chile (1960) and Chimbote, Peru (1970).

16. The document also contains a bibliography, which lists various works of interest to any professionals wishing to go deeper into certain aspects of earthquake engineering.

17. In addition, Venezuela's Provisional Standard for Earthquake-Resistant Buildings (1967) is annexed hereto as an example of building legislation in a country located in a seismic zone, together with a selection of photographs taken by the authors to illustrate some of the topics developed in the document.

18. The document has been prepared for the United Nations Secretariat by the following group of Chilean consultants: Mr. Hernán Ayarza E., Dr. Gonzalo Castro S., Mr. Luis Crisosto A., Mr. Carl Luders S., Mr. Sergio Rojas I. and Dr. Patricio Ruiz T.

Part One

EARTHQUAKES

I. ELEMENTS OF SEISMOLOGY

A. Causes and origin of earthquakes

19. Earthquakes are natural geological phenomena which may cause great disasters when they strike densely populated areas if the engineering structures built in those areas do not have adequate resistance characteristics.

20. The study of the causes, propagation and effects of the movements of the earth's crust which result from an earthquake have given rise to seismology, a branch of geophysics, which today is a firmly established science.

21. Almost everything that is known or is believed to be known concerning seismic phenomena originates in the study of ground movements that are felt as vibrations during an earthquake, since nothing contributes more to a good understanding of seismic phenomena than the experience of an earthquake and the analysis of its effects.

22. The rapid development of precise measuring and recording instruments during the past few decades and the importance attached by universities, governmental departments and international organizations to research in this field have made possible a systematic study of ground vibrations and the establishment of hypotheses on the internal composition of the earth and the causes and origin of earthquakes.

23. On the basis of an analysis of seismological observations, it can be estimated that the internal structure of the earth consists of three basic concentric layers: the crust, the mantle and the core (Fig. 1).

24. In relation to the size of the planet, the crust is a thin, solid but fragile shell whose thickness varies between 30 and 60 km. This crust rests upon the mantle, which extends to a depth of about 2,900 km and consists of molten rock at enormous pressure and temperature. Under the mantle is the core, extending to the centre of the earth and consisting of the inner core and the outer core.

25. Although the origin of earthquakes is not fully understood, enough is known to affirm that some earthquakes are of volcanic origin, others are caused by cave-ins, landslides or other minor phenomena, and still others are of tectonic origin. Tectonic earthquakes are the most interesting from the seismological point of view, since they are the most powerful, liberating vastly more energy than the other types. Owing to their great power, these tectonic processes are the ones that have contributed most to the formation of natural features such as mountains and valleys, which characterize the earth's landscape and contribute to its beauty. Such tectonic earthquakes are so violent that they are capable of causing disasters in areas where no attention has been given to preventive measures for avoiding destructive effects.

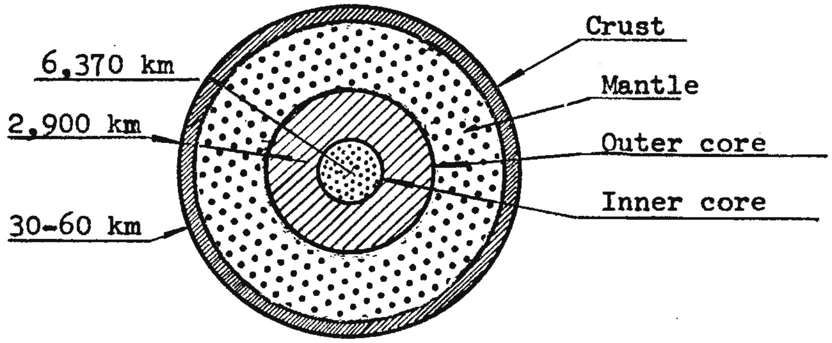


FIG. 1. SCHEMATIC CROSS-SECTIONAL VIEW OF THE INTERNAL STRUCTURE OF THE EARTH

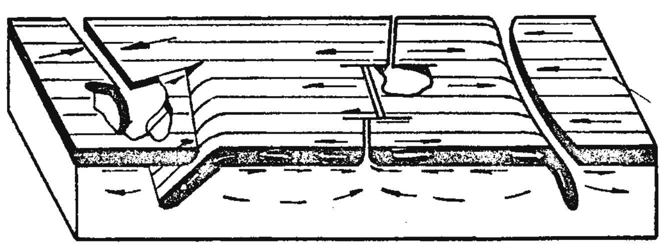


FIG. 2. AN ILLUSTRATION OF THE CURRENT THEORIES OF OCEANIC TECTONICS

Plates of lithosphere (crust) are formed symmetrically on spreading ridges, move horizontally as rigid blocks and are destroyed by undercrusting beneath island arcs. The arrows on the plates show the directions of relative motions. Arrows showing the flow in the mantle serve to indicate that return flow may have taken place (D. P. McKenzie).

26. Most of the tectonic earthquakes detected thus far have originated at depths of no more than 60 km. They are considered surface earthquakes, and their origin is associated with deformations of the earth's crust, which is in a state of constant deformation owing to the convection currents in the plastic zones of the mantle and to the contractions resulting from the cooling of the earth. The accumulation of these deformations in the crust of the earth is the source of the energy released in earthquakes of tectonic origin. When the tensions associated with these deformations reach the rock's limit of resistance, there is a fracture, accompanied by a sudden liberation of the energy accumulated during the deformation process. The fracture takes place in areas in which the resistance of the rock has been reduced by the presence of geological faults. These faults constitute a potential earthquake danger, defining zones of seismic activity in which slow tectonic processes are taking place. Although the perturbation in these zones is general rather than isolated, there are points at which greater or lesser amounts of accumulated energy are liberated.

27. In recent years the theory of plate tectonics and continental drift has gained increasingly wide acceptance among geologists and geophysicists as a hypothesis than can explain the structure of the continents. The most recent version of these ideas, known as the "plate tectonics" theory, assumes that the top 50 to 100 km of the earth consists of a number of rigid plates which are in constant motion relative to one another.

28. These plates move very slowly; many of their boundaries cannot be directly mapped by geologists because they are below sea level, and therefore they must be determined by indirect methods, one of the most important of which is the localization of earthquakes. Various studies of major surface earthquakes occurring in many parts of the earth have shown that the earthquakes resulted from sudden movements of major faults. Some of the measured displacements are large, reaching 10, as in the Alaskan earthquake of 1964. These displacements are a direct expression of the movement of two plates separated by the fault forming the boundary between them. Although not all earthquakes produce slip on faults at the earth's surface, many are due to relative movements of the edges of the plates taking place by a series of jumps, each of which is an earthquake, rather than by a steady creeping motion.

29. In some regions the plate margins cross continents and can then be studied directly, as in the case of the San Andreas Fault in California, along which there is slip between the North American and Pacific plates.

30. The possible movements between adjacent plates, which are illustrated in the diagram of Fig. 2, are of three types. If two plates move apart, mantle material from beneath them wells up, cools, hardens and regenerates the plates. Since the upwelling material is hot, it is less dense than the cold plates and therefore wells up above the deep ocean floor to form a linear ridge; such ridges, together with trenches and fracture zones, constitute the most important characteristics of the ocean bottom, marking the boundaries between the plates. Earthquakes occur along the axis of the ridge which is the boundary between the two separating plates.

31. Another type of movement takes place when two plates slide past each other without destruction of either; this corresponds to the case in which the edges of the plates are parallel to the direction of relative motion. The San Andreas Fault in California is an example of this type of fault, known as a transform

fault. The mechanism giving rise to earthquake motions in this case is schematically illustrated in Fig. 3, in accordance with the ideas on elastic rebound formulated by H. F. Reid on the basis of the earthquake which occurred at San Francisco, California in 1906.

32. The third and last type of movement is the movement of two plates towards each other; this happens in the most seismically active regions of the earth, where one plate overrides the other and drives it down into the mantle. In this case, since the cold plates are denser than the surrounding mantle, the lower plate tends to sink, so that it warms up and its mechanical properties become similar to those of the surrounding mantle. The trenches and island arcs are the surface features associated with plate destruction, much of which is at present taking place on the Pacific margins. This is the case of the coast of Chile and Peru, a region of high seismic activity, in which the crust is disappearing into the Pacific trench along the line of contact of the South Pacific plate and the American plate. The tectonic movement caused by the sliding of the oceanic plate under the continental plate is the cause of the high seismic activity in the region. A schematic cross section of the tectonic model is shown in Fig. 4; this is a cross section of the coastal fault through the epicentre of the Peruvian earthquake of 31 May 1970.

B. Propagation of seismic disturbances; earthquake waves

33. During the process of deformation of the earth's crust, energy is accumulated, and when this energy exceeds a specific value, it is suddenly liberated by the fracture of the crust.

34. The point at which the fracture begins is called the focus of the earthquake and corresponds to the centre of the mechanical perturbation from which the seismic energy begins to radiate. The earthquake focus may lie at various depths in the interior of the earth: shallow earthquakes have foci at depths down to 60 km, intermediate earthquakes have focus depths of 60 to 300 km, and deep earthquakes have foci at depths below 300 km.

35. The point of the earth's surface directly above the focus is called the epicentre.

36. If the planet is regarded as an elastic body, the mechanical perturbations produced at the focus of the earthquake will tend to propagate themselves in the form of seismic waves travelling through the earth and giving rise to the vibratory motions of the ground which are the typical manifestation of earthquakes.

37. These wave motions consist basically of two types of waves: longitudinal waves, which are movements of contraction and dilatation in the direction of propagation, and transverse waves, which are movements perpendicular to the path of propagation.

38. The longitudinal waves, or P waves, propagate normal stresses, tension and compression, at velocities of the order of 5 to 6 km/sec in the crust. The transverse waves, or S waves, are slower than the longitudinal waves but have greater amplitudes; they are propagated at velocities of the order of 3 to 4 km/sec. The S waves transmit tangential tensions.

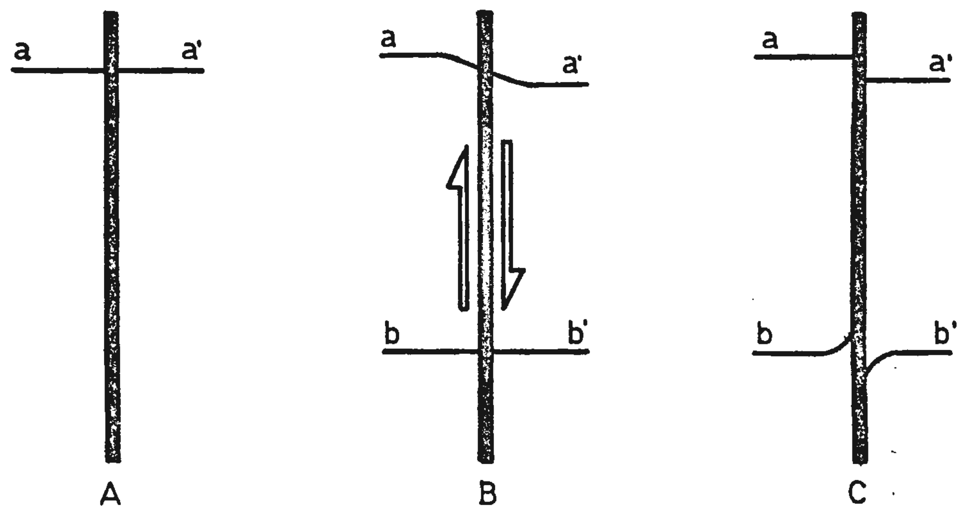


FIG. 3. ILLUSTRATION OF THE THEORY OF ELASTIC REBOUND

The thick line represents the geological fault. Figure A shows the undeformed state, in which a reference line a-a' has been marked; this line may be a road, an iron railing or a row of trees. Figure B shows the deformed state of the region before a rupture takes place; the arrows show the direction of the relative movement which has deformed the reference line a-a' in the vicinity of the fault. At this stage a new reference line b-b' can be drawn. Figure C shows the situation after the rupture has taken place along the fault and also shows the elastic rebound of the deformed sections; the shape of the reference lines indicates that the elastic rebound, like the accumulated deformation, is concentrated in the vicinity of the fault.

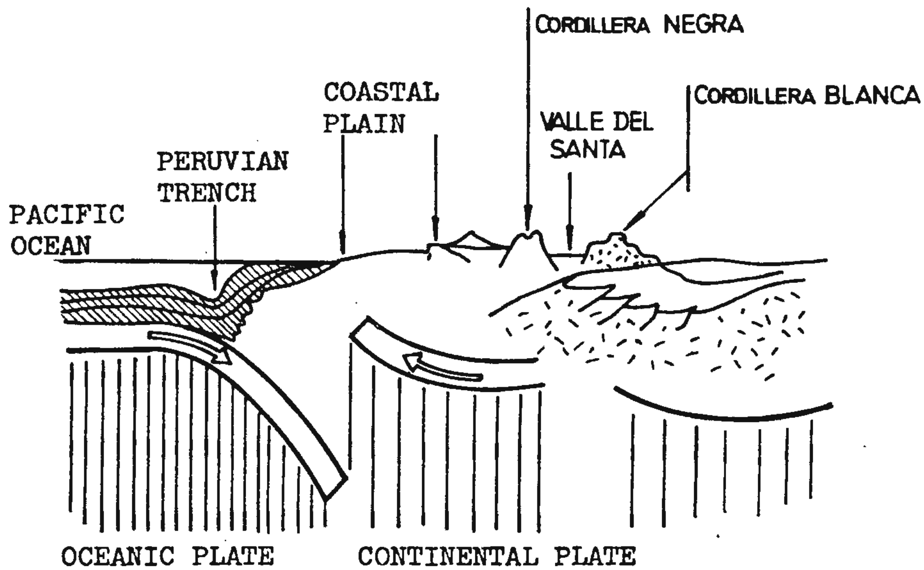


FIG. 4. SCHEMATIC CROSS SECTION OF THE TECTONIC MODEL

The figure shows in schematic form the situation existing in the zone of contact between the South Pacific plate and the American plate off the coast of Peru, on a cross section passing through the epicentre of the May 1970 earthquake (Lomnitz).

The tectonic movement caused by the sliding of the oceanic plate under the continental plate is the cause of the high seismic activity in the region.

39. When earthquake waves arrive at a surface of discontinuity, either in the interior of the earth or at its surface, they undergo multiple reflection and refraction, giving rise to new types of waves.

40. The trajectories of seismic waves in the interior of the earth are not straight lines but curves which are concave on the side towards the surface, as shown in Fig. 5, owing to the fact that the medium through which the waves are propagated is heterogeneous and increases in density towards the centre of the planet.

C. Earthquake records and measuring instruments

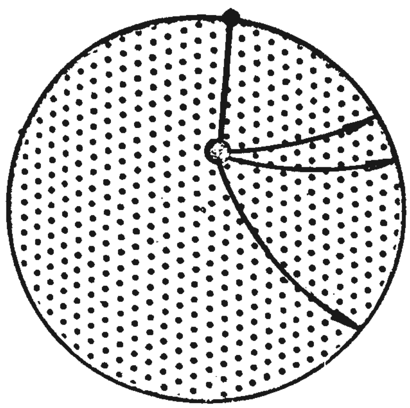
41. Ground movements during an earthquake can be measured by special instruments known as seismographs, Fig. 6, which record the ground movements graphically in a seismogram. By an analysis of seismograms, it is possible to determine the velocity of propagation of the various types of seismic waves for each particular earthquake, and a systematic study of such records has made possible a fairly exact analysis of the physical characteristics of the earth. Seismographs also make it possible to locate the focus of the earthquake by determining the difference between the times of arrival of the seismic waves at different seismological stations.

42. For the purposes of disciplines related to earthquake-resistant construction, the earthquakes of greatest interest are those which take place at an epicentre distance of no more than 200 km and whose seismograms are very complex in appearance, owing to the superposition of different types of waves of various frequencies. These records reflect the selective characteristics of local geology, which acts as a filter on the seismic waves, amplifying some waves and decreasing the amplitudes of others, depending on their frequency. In very general terms, it may be said that soft soils increase the amplitudes of waves with low-frequencies, of the order of 1 cycle/sec, and reduce the amplitudes of high-frequency waves, of the order of 3 cycles/sec. Similarly, the attenuation of amplitudes with distance is greater for high-frequency waves, so that the records obtained at distant stations show a higher proportion of long-period waves.

43. The instruments used in the measurement of seismic movements are very simple in concept but require a high degree of refinement in their operation. Conceptually, a seismograph consists of a pendulum which is free to oscillate when excited by a seismic movement and which, when it oscillates, inscribes a record of the ground movement on a strip of paper that moves at a uniform rate under the stylus (Fig. 6).

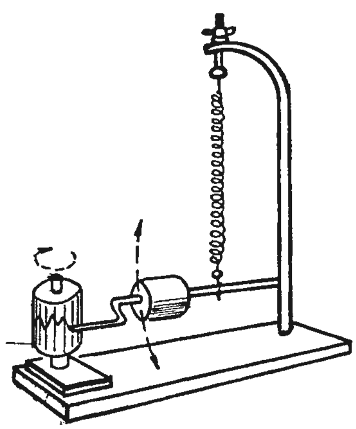
44. These instruments have been steadily improved since the middle of the nineteenth century; various ways have been found for simulating the effect of the pendulum and translating its movement into a record, called a seismogram, which represents the movement of the ground. Today seismographs are so sensitive that they can detect movements of the order of a billionth of a millimetre, making seismology the branch of geophysics which has made the greatest contribution to a quantitative knowledge of the interior of the earth.

45. What is of special interest in earthquake engineering is the measurement of earthquakes which have destructive effects on structures. This requires



Earthquake waves behave in accordance with the laws of reflection and refraction whenever they encounter different strata of the earth, and consequently their trajectories do not follow straight lines in the interior of the earth, since the density of the earth increases with depth.

FIG. 5. DIAGRAM SHOWING THE PROPAGATION OF EARTHQUAKE WAVES



The figure shows an elementary seismograph which can be used for recording on a rotating drum the vertical vibrations caused by an earthquake. The recording of such vibrations uses the principle of the dynamometer: an inert mass is suspended from a strong spring which elongates and contracts with the vertical vibrations.

FIG. 6. SCHEMATIC DIAGRAM OF A SEISMOGRAPH

instruments capable of recording with sufficient precision the most violent oscillations that take place in the vicinity of the epicentre, unlike the delicate instruments used by seismologists, which are made highly sensitive in order to be able to detect small earthquakes occurring in remote areas. These delicate instruments are so sensitive that during very violent earthquakes they often jump out of their supports and produce records which run off the scale.

46. Since the vibratory motion of the ground is manifested in structures in the form of inertial forces directly related to the acceleration of the ground, scientists have designed instruments called "strong-motion accelerographs" (Fig. 7), which make it possible to record in a graph called an accelerogram (Fig. 8) the motions of the ground during an earthquake. Unlike seismographs, these instruments do not operate continuously but have a special actuator which turns them on when the accleration of the ground exceeds a certain threshold, so that they can record the most important portion of the accelerogram.

47. There are at present a number of international programmes designed to install a sufficient number of these instruments in some of the most seismically active regions of the earth.

48. Seismographs, designed for measuring the properties of the interior of the earth, are usually installed on bedrock in order to avoid the local influence of the foundation soil. In contrast, accelerographs, which are designed to measure excitations in structures and the behaviour of these structures during earthquakes, are usually installed in river fills or on terrain typically used as the foundation for structures. In some countries there are even regulations requiring the installation of such instruments in structures of special importance. Normally the instruments installed for such purposes can record simultaneously the three components of the seismic motion, two horizontal and one vertical.

D. Seismic intensity

49. In investigating the effects of earthquakes, it is helpful to correlate observations of the resulting damage by identifying areas in which such damage is of uniform intensity. The isoseismic lines demarcating these areas normally extend in the direction of the fault and are strongly influenced by local geology. Figure 9 shows a typical example of isoseismic curves which make it possible to form a general impression of the destructive effect and of the size of the area affected.

50. For an analysis of the damage, each of the defined areas is assigned a degree of seismic intensity by simple comparison with some previously established scale of seismic intensity. The seismic intensity thus represented indicates the degree of severity of seismic movement at a given place, based on the sensations of individuals and on observation of the effects of the earthquake on man-made structures. Consequently, unlike other measurements of seismic phenomena, intensity data give a qualitative rather than an instrumental 2/ description of seismic movement.

2/ Arturo Arias proposed an instrumental method of measuring seismic intensity whereby the damage caused to structures would be measured in terms of the energy dissipated by a group of uniform structures (First National Congress of Seismology and Earthquake Engineering, Lima, Peru, 1969).

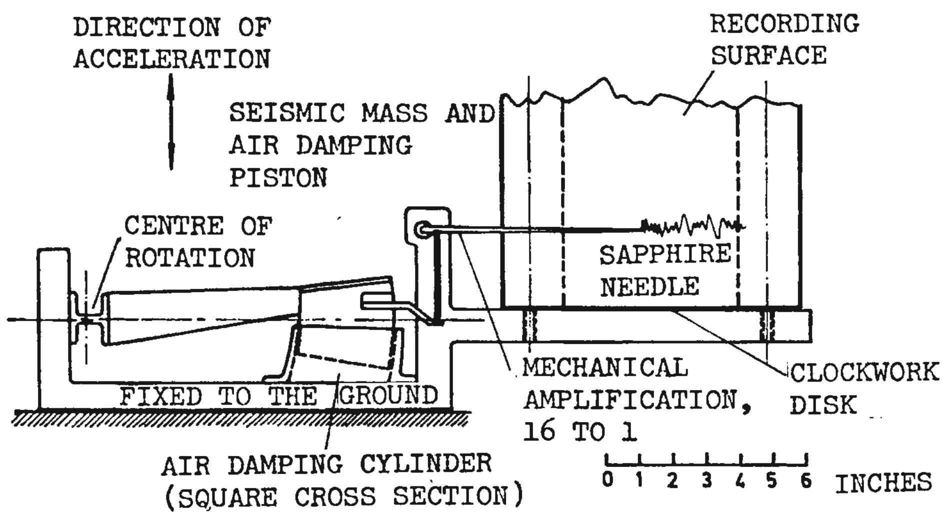


FIG. 7. SCHEMATIC DIAGRAM OF STRONG-MOTION ACCELEROGRAPH

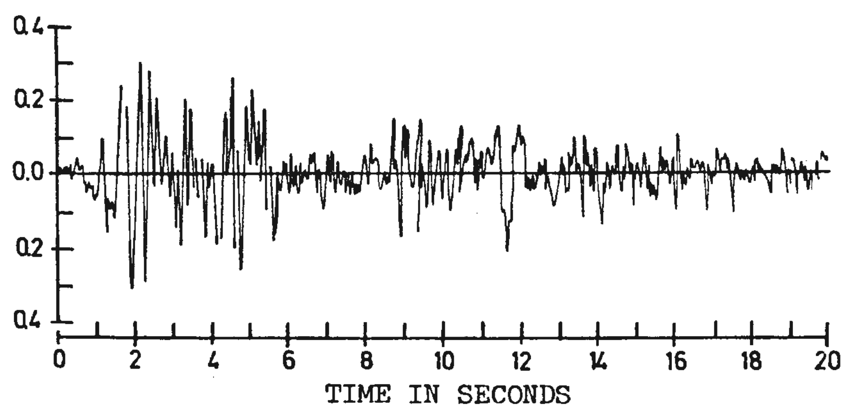


FIG. 8. ACCELEROGRAM OF THE NORTH-SOUTH COMPONENT OF THE EL CENTRO EARTHQUAKE OF 1940

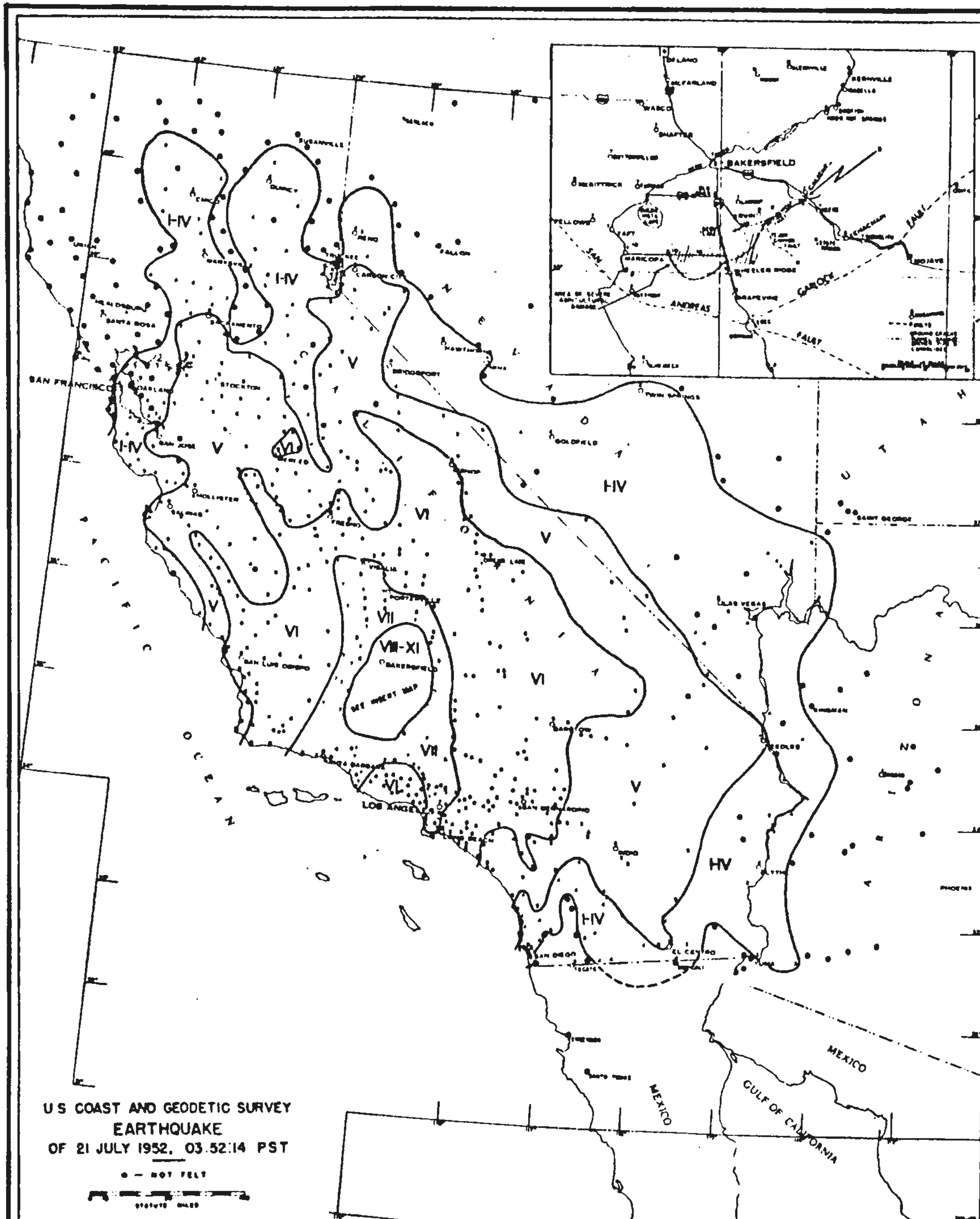


FIG. 9. INTENSITY MAP SHOWING ISOSEISMIC LINES

51. Unfortunately, although scales of seismic intensity are coming into general use, they relate, for the most part, to the effects of earthquakes on structures typical of certain specific locations, so that it is very difficult to make a comparative evaluation of earthquakes occurring in different places.

52. The seismic intensity scale most widely used today is the Modified Mercalli scale (1931), the twelve grades of which are summarized below:

1. Detected only by very sensitive instruments.

2. Felt only by a few persons at rest, particularly on the upper floors of buildings; some hanging objects swing.

3. Felt indoors although not always recognized as an earthquake because of its similarity to the vibrations caused by passing of trucks; standing motor-cars rock slightly; duration of movement can be estimated.

4. Felt indoors by many people, outdoors by a few; standing vehicles rock noticeably.

5. Felt by most persons; at night, some sleepers awakened; tall objects upset, pendulum clocks change rate; direction of movement can be estimated.

6. Felt by all; many frightened and run outdoors, persons walk unsteadily; plaster falls off, causing minor damage.

7. All frightened; they run outdoors, and it is difficult to stand; noticed by drivers of motor vehicles; damage to structures varies according to their quality.

8. Difficulty in steering motor vehicles; fall of chimneys, walls and monuments, panel walls thrown out of frame structures.

9. General panic; some buildings shifted from their foundations, thrown out of plumb and cracked; cracks appear in ground; mud and sand ejected.

10. Most masonry structures destroyed; serious damage to major structures, bridges, dams and dikes; landslides.

11. Railway rails bent severely; underground pipelines out of service.

12. Damage nearly total; large rock masses displaced; objects thrown into the air; lines of sight and level greatly distorted.

One of the most important revisions of the Modified Mercalli scale (MM) is that carried out in 1964 by Medvedev, Sponheuer and Karnik, better known as the MSK-64 scale, the adoption of which has been recommended by UNESCO.

53. This scale retains the twelve grades of the Mercalli scale (MM) but endeavours to define the phenomena corresponding to each grade in a more systematic and quantitative form. For this purpose non-earthquake-resistant structures are first classified into three types, then the phenomena are defined in percentage terms and, finally, the intensity of damage to buildings is classified into five categories. The appropriate combination of these elements gives a scale which

can be used to obtain a more objective classification of earthquake intensity. In view of the importance of this scale because of its recommendation by UNESCO, it is set out in detail below.

MSK-64 scale:

Classification of the scale

I. Types of structures (buildings not antiseismic)

Structure A: Buildings in fieldstone, rural structures, adobe houses, clay houses

B: Ordinary brick buildings, buildings of the large block and prefabricated type, half timbered structures, buildings in natural hewn stone

C: Reinforced buildings, well-built wooden structures

II. Definition of quantity

Single, few: about 5 per cent

Many: about 50 per cent

Most: about 75 per cent

III. Classification of damage to buildings

Grade 1: Slight damage: fine cracks in plaster; fall of small pieces of plaster.

Grade 2: Moderate damage: small cracks in walls; fall of fairly large pieces of plaster; pantiles slip off; cracks in chimneys; parts of chimneys fall down.

Grade 3: Heavy damage: large and deep cracks in walls; fall of chimneys.

Grade 4: Destruction: gaps in walls; parts of buildings may collapse; separate parts of the building lose their cohesion; inner walls and filled-in walls of the frame collapse.

Grade 5: Total damage: total collapse of buildings.

IV. Arrangement of the scale

(a) Persons and surroundings

(b) Structures of all kinds

(c) Nature

Intensity

I. Not noticeable

(a) The intensity of the vibration is below the limit of sensibility; the tremor is detected and recorded by seismographs only.

II. Scarcely noticeable (very slight)

(a) Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.

III. Weak, partially observed only

(a) The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects, somewhat more heavily on upper floors.

IV. Largely observed

(a) The earthquake is felt indoors by many people, outdoors by few. Here and there people awake, but no one is frightened. The vibration is like that due to the passing of a heavily loaded truck. Windows, doors and dishes rattle. Floors and walls creak. Furniture begins to shake. Hanging objects swing slightly. Liquids in open vessels are slightly disturbed. In standing motorcars the shock is noticeable.

V. Awakening

(a) The earthquake is felt indoors by all, outdoors by many. Many sleeping people awake. A few run outdoors. Animals become uneasy. Buildings tremble throughout. Hanging objects swing considerably. Pictures knock against walls or swing out of place. Occasionally pendulum clocks stop. Few unstable objects may be overturned or shifted. Open doors and windows are thrust open and slam back again. Liquids spill in small amounts from well-filled open containers. The sensation of vibration is like that due to a heavy object falling inside the building.

(b) Slight damages of grade 1 in buildings of type A are possible.

(c) Sometimes change in flow of springs.

VI. Frightening

(a) Felt by most indoors and outdoors. Many people in buildings are frightened and run outdoors. A few persons lose their balance. Domestic animals run out of their stalls. In few instances dishes and glassware may break, books fall down. Heavy furniture may possibly move and small steeple bells may ring.

(b) Damage of grade 1 is sustained in single buildings of type B and in many of type A. Damage in few buildings of type A is of grade 2.

(c) In few cases cracks up to widths of 1 cm possible in wet ground; in mountains occasional landslips; changes in flow of springs and in level of well-water are observed.

VII. Damage to buildings

(a) Most people are frightened and run outdoors. Many find it difficult to stand. The vibration is noticed by persons driving motor-cars. Large bells ring.

(b) In many buildings of type C damage of grade 1 is caused; in many buildings of type B damage is of grade 2. Many buildings of type A suffer damage of grade 3, few of grade 4. In single instances landslips of roadway on steep slopes; cracks in roads; joints of pipelines damaged; cracks in stone walls.

(c) Waves are formed on water, and water is made turbid by mud stirred up. Water levels in wells change, and the flow of springs changes. In few cases dry springs have their flow restored and existing springs stop flowing. In isolated instances parts of sandy or gravelly banks slip off.

VIII. Destructions of buildings

(a) Fright and panic; even persons driving motor-cars are disturbed. Here and there branches of trees break off. Even heavy furniture moves and partly overturns. Hanging lamps are in part damaged.

(b) Many buildings of type C suffer damage of grade 2, few of grade 3. Many buildings of type B suffer damage of grade 3 and few of grade 4, and many buildings of type A suffer damage of grade 4 and few of grade 5. Occasional breakage of pipe seams. Memorials and monuments move and twist. Tombstones overturn. Stone walls collapse.

(c) Small landslips in hollows and on banked roads on steep slopes; cracks in ground up to widths of several centimeters. Water in lakes becomes turbid. New reservoirs come into existence. Dry springs have their flow restored and existing springs stop flowing. In many cases change in flow and level of water.

IX. General damage to buildings

(a) General panic; considerable damage to furniture. Animals run to and fro in confusion and cry.

(b) Many buildings of type C suffer damage of grade 3, a few of grade 4. Many buildings of type B show damage of grade 4; a few of grade 5. Many buildings of type A suffer damage of grade 5. Monuments and columns fall. Considerable damage to reservoirs; underground pipes partly broken. In individual cases railway lines are bent and roadways damaged.

(c) On flat land overflow of water, sand and mud is often observed. Ground cracks to widths of up to 10 cm, on slopes and river banks more than 10 cm; furthermore a large number of slight cracks in ground; falls of rock, many landslides and earth flows; large waves on water. Dry springs renew their flow and existing springs dry up.

X. General destruction of buildings

(b) Many buildings of type C suffer damage of grade 4, a few of grade 5. Many buildings of type B show damage of grade 5; most of type A have destruction of grade 5; critical damage to dams and dikes and severe damage to bridges. Railway lines are bent slightly. Underground pipes are broken or bent. Road paving and asphalt show waves.

(c) In ground, cracks up to widths of several decimetres, sometimes up to 1 metre. Broad fissures occur parallel to watercourses. Loose ground slides from steep slopes. From river banks and steep coasts considerable landslides are possible. In coastal areas displacement of sand and mud; change of water level in wells; water from canals, lakes, rivers etc. thrown on land. New lakes appear.

XI. Catastrophe

(b) Severe damage even to well-built buildings, bridges, water dams and railway lines; highways become useless; underground pipes destroyed.

(c) Ground considerably distorted by broad cracks and fissures, as well as by movement in horizontal and vertical directions; numerous landslips and falls of rock.

The intensity of the earthquake must be investigated specially.

XII. Landscape changes

(b) Practically all structures above and below ground are greatly damaged or destroyed.

(c) The surface of the ground is radically changed. Considerable ground cracks with extensive vertical and horizontal movements are observed. Falls of rock and slumping of river banks over wide areas; lakes are dammed; waterfalls appear, and rivers are deflected.

The intensity of the earthquake must be investigated specially.

E. Magnitude and energy of earthquakes

54. Because of the need to be able to classify earthquakes according to the violence of the ground movement, a method of determining their "magnitude" was devised. This is an objective instrumental measurement related to the energy released by the seismic movement.

55. Magnitude is expressed in terms of the Richter scale, which is based on measurements made on a seismogram recorded by a standard seismograph located 100 km from the epicentre. Correction tables exist for calculating magnitude at different distances from the epicentre.

56. Empirical studies using available data on a large number of earthquakes have made it possible to devise the following formula, which establishes a relation between the energy liberated by an earthquake and its magnitude:

$$\log_{10} E = 11.4 + 1.5M,$$

where M is the magnitude measured on the Richter scale and E is the energy released, in ergs.

57. This relation is shown graphically in Fig. 10, which makes it clear how great an increase in released energy is signified by a change of 1 unit in magnitude in the severe earthquake range. From the logarithmic nature of the relation between the magnitude of an earthquake on the Richter scale and the energy liberated, it can be seen that small changes in magnitude beyond 7 correspond to large differences in the energy and destructive power of the earthquake.

58. The Richter scale has no upper limit, but because of physical limitations imposed by the characteristics of the earth's crust it is highly improbable that an earthquake of a magnitude much greater than 9 could occur.

59. By way of illustration, the following table shows some of the earthquakes of the greatest magnitude that have occurred during this century.

Table 1. Major twentieth-century earthquakes

Year	Location	Magnitude
1905	China	8.7
1906	Colombia	8.9
1906	Chile (Valparaíso)	8.6
1911	Russia	8.7
1914	Japan	8.7
1922	Chile (Huasco)	8.4
1928	Chile (Talca)	8.4
1933	Japan	8.9
1938	Alaska	8.7
1939	Chile (Chillán)	8.3
1943	Chile (Illapel)	8.3
1950	India	8.7
1960	Chile (Concepción)	7.8
1960	Chile (Valdivia)	8.5

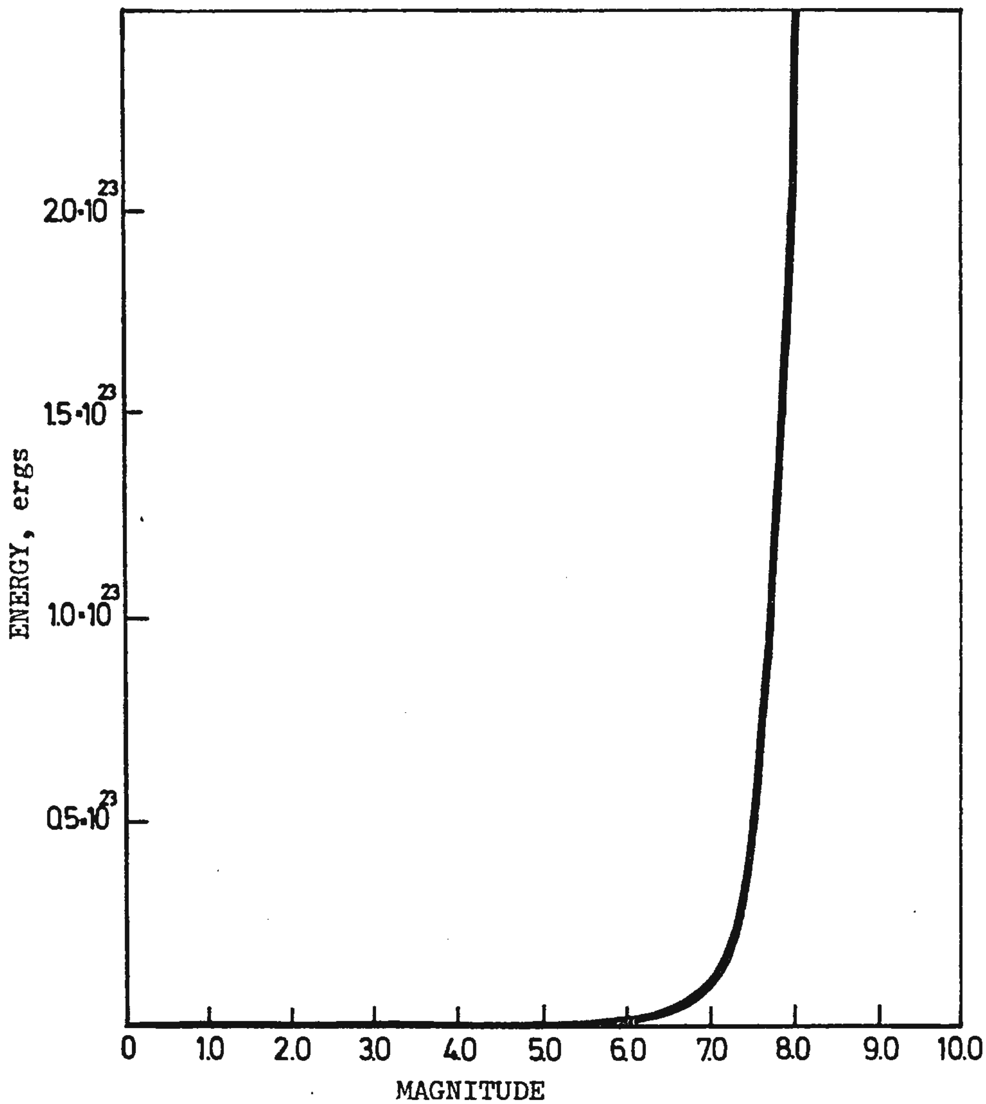


FIG. 10. RELATION BETWEEN MAGNITUDE AND SEISMIC ENERGY

F. Seismic geography

60. In order to be able to predict the risk of earthquakes in different regions, it is necessary to have a detailed knowledge of the pattern of seismic activity throughout the world. It has been noted that earthquakes do not occur with the same frequency in all regions of the earth. For example, areas such as Japan, the Philippines, Indonesia, New Zealand, the western mountain ranges of America, the Balkans, Italy and Asia Minor are noted for their high level of seismic activity, while there are other areas in which earthquakes are unknown.

61. Figure 11 shows the location of the most severe shallow earthquakes which have occurred. It can be seen that they occur along the main faults or zones of contact between the plates which make up the earth's crust.

62. Most earthquakes in oceanic regions take place in remarkably narrow belts associated with the major features of the ocean floor: ridges, trenches and fracture zones. These features mark the boundaries between plates.

63. The areas of the world with the greatest seismic activity are the circum-Pacific belt and the Mediterranean seismic belt, as shown in Fig. 12.

64. The circum-Pacific belt borders the Pacific Ocean, extending along the west coast of the American continent, from Alaska to Chile, then through New Zealand and Japan and along part of the east coast of Asia. This is the most seismically active area of the world; more than 80 per cent of the destructive earthquakes recorded thus far have taken place within it. The highest level of seismicity is observed in Japan, where many medium-magnitude earthquakes (7.0 to 7.7 on the Richter scale) occur, thus considerably raising the risk of earthquakes. The Andean area of South America, on the other hand, shows the highest energy-release index, owing to the occasional occurrence of earthquakes of great magnitude (8.0 to 8.7).

65. The Mediterranean earthquake belt, which extends from Burma through Asia Minor and the Mediterranean Sea to the Azores, is much less active than the circum-Pacific belt, and the earthquakes which occur in this area are essentially shallow, with focal depths of not more than 60 km.

66. Another area of relatively high seismic activity which should be mentioned lies between the Pamir plateau and Lake Baikal in Central Asia, where the Kansu earthquakes in northern China were recorded in 1920 and 1932.

G. Seismic zoning and seismic risk

67. The object of seismic zoning is to divide territories into zones with different levels of seismic risk. In this way, it is possible to establish different criteria for earthquake-resistant design and construction, with an acceptable level of safety in high-seismic-risk areas and more economical designs in areas where destructive earthquakes are highly improbable.

68. Seismic risk is a measure of the probability that the strongest earthquake which can occur in an area over a given period of years will exceed a particular level of magnitude. It is calculated by superposing the effects of all potential seismic sources in the area, weighted with an activity factor whose value depend on local tectonics.

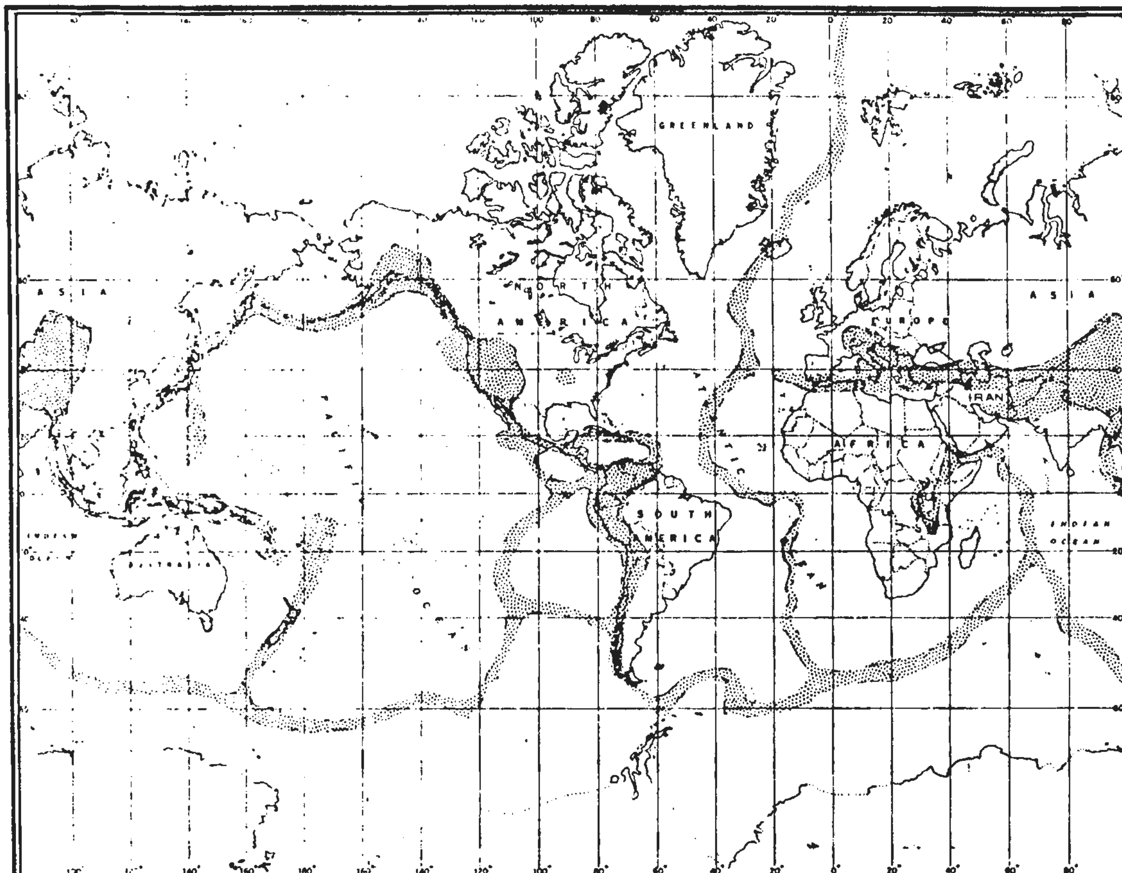


FIG. 11. ZONES OF MAJOR SEISMIC ACTIVITY

Most earthquakes occur along the faults indicated by the shaded areas on the map; these faults are seldom observable on the surface of a continent, except in the case of the San Andreas fault in California and the Anatolian fault in Turkey. The limits of the oceanic plates determined from earthquakes are narrow zones whose width seldom exceeds the possible location error distance. On the other hand, seismicity on the continents is spread over broad belts.

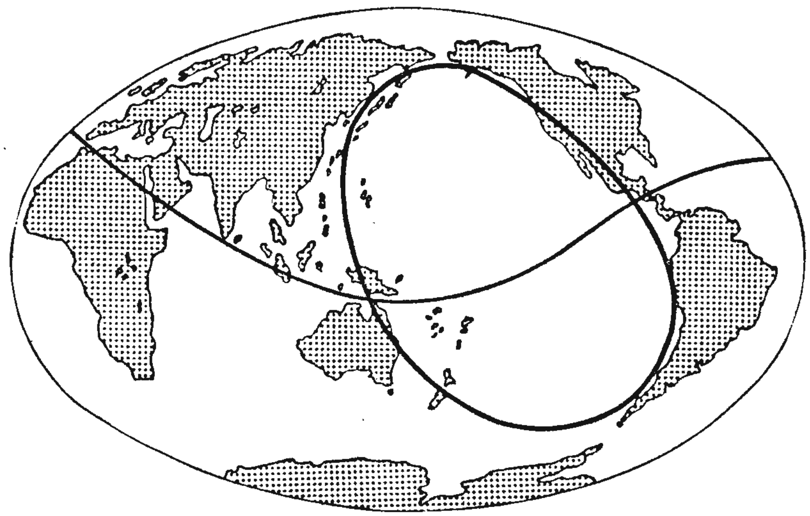


FIG. 12. THE CIRCUM-PACIFIC BELT AND THE MEDITERRANEAN BELT

The illustration shows the belts about which the seismic zones are grouped. The first rings the Pacific Ocean, and the second passes through the marine basins formed by major subsidences, such as the Mediterranean Sea, the Caribbean, the Celebes basin and the Red Sea. From this circle, which crosses the equator at an angle of 23°, radiate two fairly active zones: the first including the faults of eastern Africa and consisting of a belt running from Ethiopia to the Cape of Good Hope, and the second beginning at the Black Sea and passing through all the mountainous regions of the Caucasus, Iran and the Himalayas.

69. The seismic zoning procedure involves compiling past seismological and geological data for a region and correlating them with seismic characteristics such as energy released, intensity and magnitude, so that each area can be classified according to its anticipated seismic activity.

70. In analysing such data, care must be taken to avoid distortions of the information which sometimes arise from the fact that seismicity is more evident in populous areas, among peoples with an ancient culture and a long history, and in areas well provided with instruments for recording seismic phenomena.

71. Local geology plays an important part in seismic zoning because of such factors as tendency towards landslides, depth of the water table, and soil quality, which has a selective effect on seismic waves, amplifying the components of some frequencies and attenuating those of others. As a result of this phenomenon, the effect of earthquakes on tall, slender buildings differs from their effect on low, rigid structures, depending on soil characteristics and distance from the epicentre.

72. Because of different subsoil characteristics, an area zoned as being of uniform seismic potential can have different local intensities. So as to take account of this phenomenon, detailed zoning, or "Microzoning", is carried out in areas of special scientific, economic or residential importance.

73. Seismic microzoning means conducting a soil analysis, sometimes accompanied by measurements obtained from artificial shocks caused by explosions. In very general terms, it has been found that soft soils tend to show higher seismic intensities than firm soils. Consequently, in zones with recent deposits of mud, sand or gravel the seismic intensity will be greater than in outcrops of firm rock, in compact limestone, shale, or granite. Moreover, in soft soils the seismic intensity will be greater if the water table is very shallow. Another factor which must be taken into account in microzoning is the potential danger presented by inclines, geological transmissions and fissure which occur close to rivers, lakes and ravines. During the microzoning process, it must be borne in mind that rigid buildings suffer less damage if their foundations are set in soft soils, while flexible buildings behave better if they are built on firm soils. These considerations can be useful in planning the growth of cities. Seismic zoning has received the most attention in Japan, New Zealand and the Soviet Union.

H. Tsunamis

74. "Tsunami" is a Japanese term used internationally to describe the large waves which have struck the coasts of a number of countries, particularly in the Pacific Ocean (Chile, Japan, Hawaii and Peru), causing extensive damage and great loss of life.

75. Although the biggest tsunamis have occurred following earthquakes, there is no evidence that the occurrences of the two phenomena are closely linked. The origin of tsunamis is uncertain; some sources associate them with sudden tectonic movements beneath the ocean, others with landslides on underwater mountains, and some have even suggested that they are resonance phenomena caused by the low-frequency waves of an earthquake.

76. Tsunamis are very massive and form suddenly. On the open sea they are practically imperceptible, since they have a very long wavelength, (of the order of 80 km) and small amplitude (less than 1 metre); they move at high speeds, which can be expressed by the formula

$$c = \sqrt{gh},$$

where g is the acceleration of gravity and h is the depth of the sea. As a tsunami crosses the Pacific Ocean, this speed is of the order of 700 km/h. When the waves reach shallow water close to coastlines, the speed is reduced rapidly and the waves begin to accumulate, so that large waves strike ports and coastal installations. Sometimes tsunamis are preceded by a drop in the sea level far below that of the lowest tides; however, this cannot be relied on as a warning sign, since the tsunami may reach the coast without any prior drop in sea level.

II. EFFECT OF EARTHQUAKES ON BUILDINGS

A. Inertial forces

77. During an earthquake, ground motion resulting from seismic waves is transmitted to structures through their foundations. The inertia of the building's mass resists the motion applied to its base, thus creating inertial forces which affect the structure in the way indicated in Fig. 13. Vertical inertial forces, generated by the vertical component of the ground motion, are not taken into consideration in current design practice for earthquake-resistant structures, since it is presumed that the vertical-load safety factors used in the design are large enough to provide adequate resistance in the event of an earthquake.

78. Horizontal inertial forces, on the other hand, produce the effect of lateral loads, analogous to wind loads, applied to the structure, and adequate provision must be made in the design to give the structure sufficient lateral resistance to withstand such forces with safety.

B. Seismic response of structures

79. Since structures are not infinitely rigid, they are deformed when subjected to inertial forces and move in a direction opposite to that of the seismic motion applied to the base of the structure. Thus, the inertial forces, whose magnitude is the mass of the structure times its acceleration, are a function of the characteristics of the ground motion and the structural characteristics of the building.

80. The most severe seismic motion to which a structure may be exposed during its useful life cannot be accurately predicted, but from a knowledge of the geology of the region and an analysis of the properties of the soil on the foundation site, it is possible to estimate the most likely characteristics of earthquakes which may affect a given structure.

81. The elastic properties and mass of structures cause them to develop a vibratory motion when they are put in motion by dynamic stresses. This vibration has features in common with the vibration of a violin string, which consists of a fundamental tone and the additional contribution of various harmonics. The vibration of a structure likewise consists of a fundamental mode of vibration and the additional contribution of various modes which vibrate at higher frequencies. In low-rise buildings (less than five storeys high) the seismic response depends primarily on the fundamental mode of vibration; accordingly, the period of vibration of this mode, expressed in seconds, is one of the most representative characteristics of the dynamic response of a building.

82. The fundamental mode of vibration of a building may be determined by any one of several methods developed for the dynamic analysis of structures. For this purpose it is necessary to visualize an ideal model of the structure of the

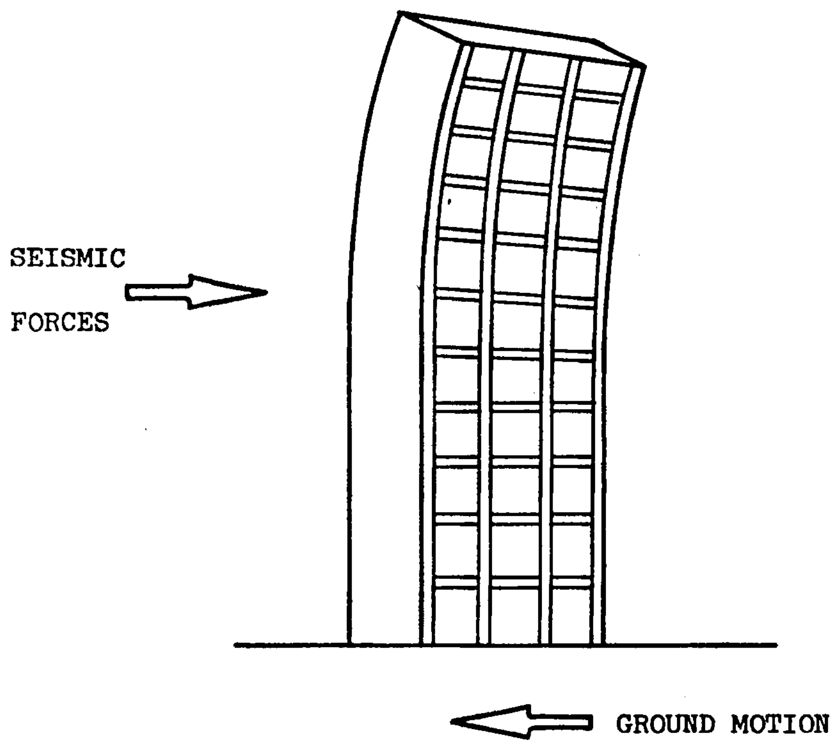


FIG. 13. GRAPHIC ILLUSTRATION OF THE EFFECT OF AN EARTHQUAKE ON A TALL BUILDING

Sudden ground motion is transmitted to the building through its foundations, thus giving rise to inertial forces opposing this motion. These are the seismic forces which act upon the building.

building, similar to the model used in current methods of structural analysis for evaluating the elastic properties and mass of the building.

83. The simplest case that may be cited as an example is that of the ideal model of a structure consisting of a simple portal, like the one shown in Fig. 14, in which all the mass is concentrated in the lintel and the elastic properties have been concentrated in the form of lateral flexibility of the columns.

84. The seismic motion affecting this structure is represented by a horizontal acceleration - as indicated in Fig. 13 - applied at the base of the portal. The equations of dynamic equilibrium make it possible to determine the lateral deformations of the lintel and the shear forces which develop in the posts during an earthquake.

85. Experiments carried out on models and full-scale structures have shown that the stresses and deformations caused by dynamic loads are less than the values estimated analytically by using concepts of structural dynamics. The difference is due to the fact that actual structures can dissipate dynamic energy even at very low levels of deformation; this dissipation of energy helps to reduce the dynamic response.

86. The methods of analysis subsequently had to be refined in order to include appropriate terms for simulating this capacity to dissipate energy. To simplify the analysis, this effect was simulated by introducing viscous damping into the model. In practice this damping effect is expressed as a percentage of the critical damping which is the greatest damping value that allows vibratory movement to develop. Experience has made it possible to estimate the degree of damping in various types of structures, and some values are shown in table 2.

Table 2. Degrees of damping

Type of structure	Damping
Reinforced concrete	5-10 per cent
Metal frame	1-5 per cent
Masonry	8-15 per cent

Because of the interaction between the ground motion and the structure's own motion, the seismic response is quite complex, even in the case of such simple structures as that depicted in Fig. 14. For example, the lateral displacement of the lintel at a time t following the inception of the earthquake is given by the expression

$$\frac{T}{2\pi}\int_0^t \ddot{u}(\tau)\, e^{-n\frac{2\pi}{T}(t-\tau)} \sin\frac{2\pi}{T}(t-\tau)\, d\tau \,,$$

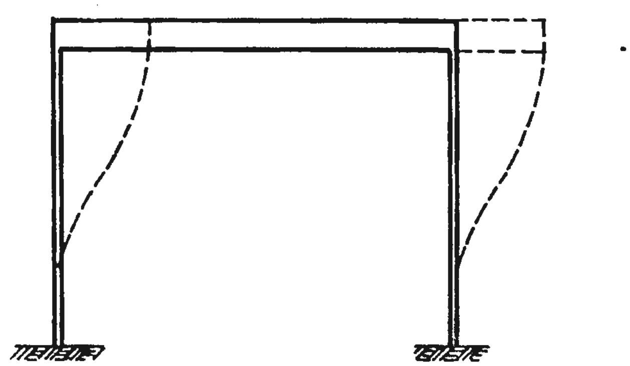


FIG. 14. IDEAL STRUCTURAL MODEL OF A SIMPLE PORTAL

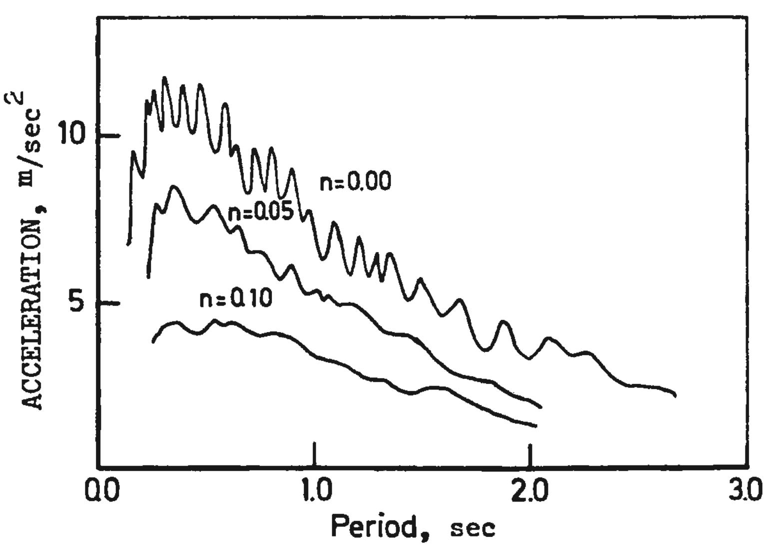


FIG. 15. ACCELERATION SPECTRUM, EL CENTRO, 1940, N-S.
Graphic representation

where $\ddot{u}(t)$ is the ground acceleration, n is the degree of damping and T is the period of vibration of the portal.

The period of vibration may be calculated from the equation

$$T = 0.2 \sqrt{\Delta} ,$$

where is the lateral deformation of the portal (in centimetres) produced by a lateral load applied at the level of the lintel and having a magnitude equal to the weight of the portal.

87. This shows that, in general, less deformable rigid structures will have shorter periods of vibration than flexible structures.

88. From these facts it can be deduced that the analysis of the seismic response of complex structures presents even more serious analytical difficulties, and it is necessary to resort to the techniques of structural dynamics in order to evaluate the periods and modes of vibration.

C. Spectra of seismic response

89. Fortunately the analysis of a structure's seismic response is greatly facilitated by the fact that, in general, it is not necessary to know how the forces and deformations have varied during the earthquake; what is of importance is to be able to determine the maximum values reached by the seismic response during the vibration of the structure. The response spectrum of an earthquake is a graph showing the maximum response of all possible buildings affected by a given seismic event. Such graphs are plotted by using as the abscissa the period of vibration of the structure and as the ordinate the maximum value of the seismic response of the structure for a given value of damping. The maximum response is calculated for a large number of portals with various periods and damping values, in order to cover a sufficiently broad range to include all possible structures. It should be noted that each point on such a graph is obtained from a complete seismic analysis of a structure with specific values for the damping and the period of vibration.

90. The most useful response spectra for purposes of earthquake-resistant design are acceleration spectra, in which the maximum response plotted as the ordinate is the maximum acceleration of the structure. This maximum acceleration is directly proportional to the maximum value of the inertial force applied to the structure. Velocity and displacement spectra may be plotted in a similar manner.

91. Figure 15 shows the acceleration spectrum of the seismic motion recorded in a north-south direction during the earthquake which occurred at El Centro, California, on 18 May 1940.

92. In studies of the response spectra of a large number of earthquakes, the following general characteristics have been noted:

(a) The response spectrum of undamped structures shows wide variations, indicating that the maximum response of such structures is highly sensitive to small errors in the determination of the period of vibration;

(b) The introduction of a small amount of damping substantially reduces the oscillations in the spectrum and lowers the magnitudes of the maximum responses, particularly in structures with short periods of vibration;

(c) In the case of earthquakes recorded relatively near the epicentre, it has been noted that the maximum values of the spectra of mild earthquakes are closer to the short-period range than is the case with strong earthquakes; and

(d) The effect of the foundation soil on earthquake characteristics can be seen from the fact that the response spectra of earthquakes recorded in soft soils exhibit maximum values in the long-period range, while earthquakes recorded in firm soils characteristically have response spectra with the maxima more concentrated in the short-period range. This observation leads to the conclusion that, as a general rule, if the foundation site is firm, rigid structures will have a more unfavourable seismic response than flexible structures, whereas the seismic response of flexible structures on soft foundation sites will be less favourable than that of rigid structures.

93. Figure 16 shows the velocity spectra of two earthquakes recorded under quite different soil conditions. Figure 16 (a) represents the earthquake recorded in the firm soil of El Centro, California, and Fig. 16 (b) represents the 1962 earthquake recorded in the soft soil of Mexico City. These spectra demonstrate clearly long-period waves are the ones most magnified in soft soils and short-period waves are the ones most magnified in firm soils.

94. By definition, the response spectrum of an earthquake specifically describes the characteristics of the structural response to the earthquake in question, and it follows that the spectra of different earthquakes, as a rule, are not identical. Despite the variations, seismic-response spectra exhibit certain common features which may be used to draw up an ideal response spectrum that will eliminate those oscillations in the response spectra which are peculiar to each earthquake and will yield a graph which varies smoothly while preserving the general features.

95. Figure 17 shows an ideal spectrum of accelerations for structural response in the case of earthquakes recorded in firm soils at moderate distances from the epicentre (Housner, 1959). This ideal spectrum was obtained by averaging the spectra of a number of earthquakes recorded in similar circumstances and smoothing out the average graphs thus obtained.

96. Coming back to the case of the simple portal depicted in Fig. 14, the maximum value of the inertial force applied at the lintel is equal to the mass of the lintel times the spectral acceleration, the latter being found on the spectrum in Fig. 17, on the curve for the damping value of the portal, at the abscissa corresponding to its period of vibration. From dynamic-equilibrium considerations, the shear force at the base of the portal is equal to the inertial force, so that its maximum value is given by the equation

$$Q = M \cdot A,$$

where M is the mass of the portal and A is the spectral acceleration. If A is expressed as a fraction of the acceleration of gravity, $A = c \cdot g$, the above equation becomes

$$Q = c \cdot W,$$

where W is the weight of the portal.

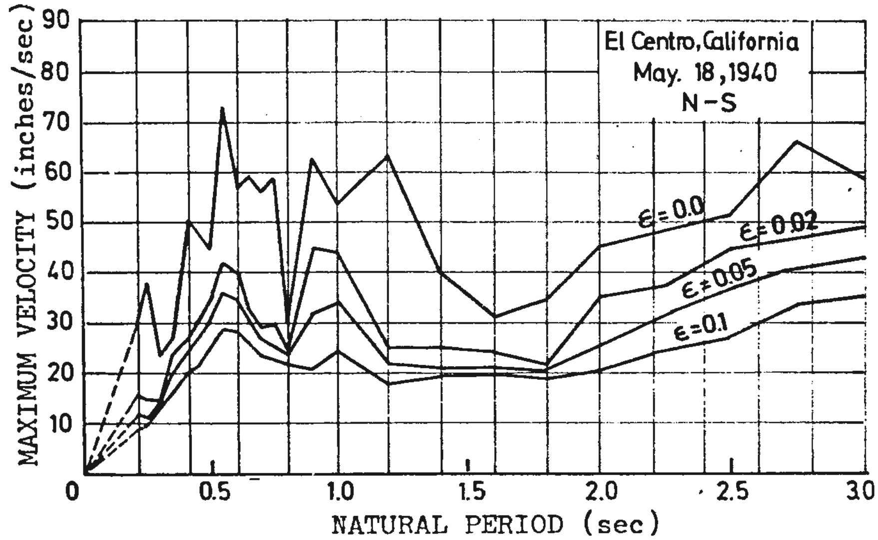


FIG. 16 (a). SPECTRUM OF PSEUDO-VELOCITIES IN THE EL CENTRO EARTHQUAKE

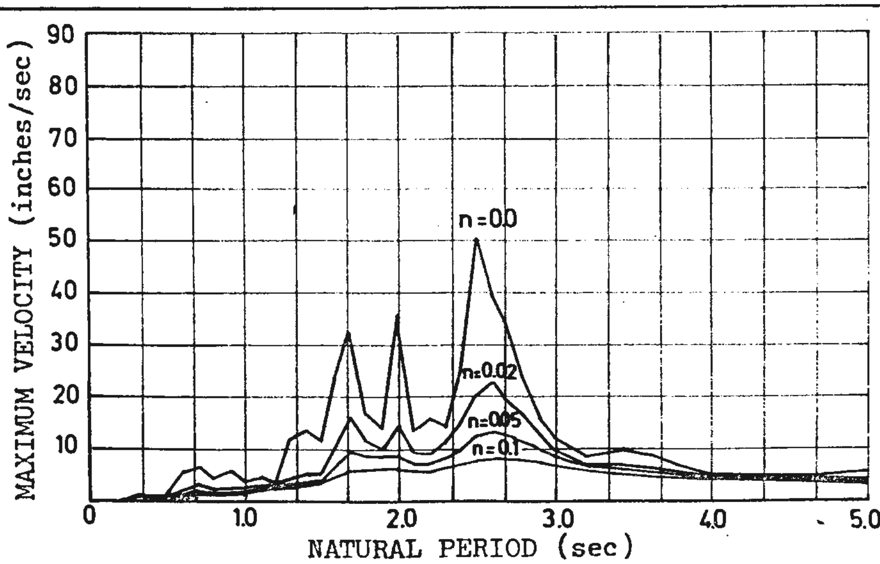


FIG. 16 (b). SPECTRUM OF PSEUDO-VELOCITIES IN THE MEXICO CITY EARTHQUAKE, 1962

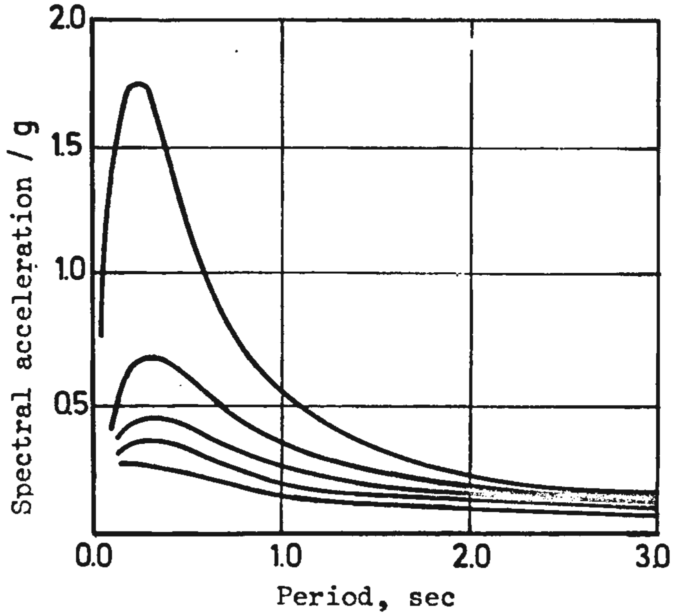


FIG. 17. ACCELERATION SPECTRUM

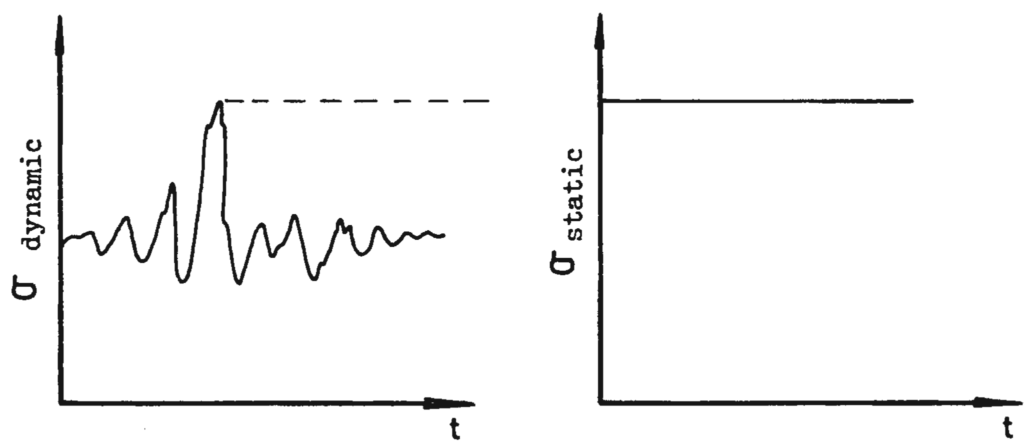


FIG. 18. GRAPHICAL COMPARISON OF A DYNAMIC STRESS WITH A STATIC STRESS EQUIVALENT TO THE MAXIMUM VALUE

97. The shear force at the base, Q, is a value characteristic of the seismic response of structures and is usually expressed as a fraction of the weight in terms of the seismic coefficient c. In general, acceleration spectra make it possible to determine the seismic coefficient directly when the maximum acceleration is expressed as a fraction of gravity.

D. Seismic forces in design

98. The main purpose of earthquake-resistant design is to prevent loss of life and personal injury, to ensure the continuity of essential services and to minimize damage to property in the event of an earthquake.

99. In designing earthquake-resistant structures, it is necessary to specify the required resistance of the structural elements. The determination of adequate resistance is a problem which depends on various factors, such as:

(a) The probability of occurrence of strong earthquakes;

(b) The characteristics of the foundation soil;

(c) The type of building and the quality of materials to be used;

(d) The relationship between the repair costs and the cost of providing greater resistance;

(e) The amount and type of damage that can be tolerated.

100. In view of the great uncertainty associated with the occurrence of destructive earthquakes, it must be recognized that to provide complete protection against earthquake risks is an economic impossibility. Accordingly, in drawing up earthquake-resistant designs for structures, the following criteria are usually accepted with regard to the amount of permissible damage, the violence of the shock and the probability of occurrence of an earthquake during the useful life of the structure:

(a) There should be no damage resulting from mild earthquakes whose probability of occurrence is significant over the useful life of the structure. This implies that the structural response should be within the elastic range;

(b) Certain easily repairable damage in earthquakes of moderate intensity may be acceptable, particularly if it affects only non-structural elements;

(c) Substantial damage, including some which may be irreparable, may be acceptable in earthquakes of great intensity whose probability of occurrence is very low, provided that the risk of any collapse that may endanger human life is avoided. This implies that the structural response may sometimes enter significantly into the inelastic range.

101. Since it is considered acceptable that buildings will sustain some damage during a major earthquake, the seismic response will take place at higher levels than those permissible in ordinary design work, even reaching a range which approaches the breaking stress. This is permissible in earthquake-resistant design because the maximum stress is momentary, as opposed to the essentially permanent or quasi-static nature of typical design stresses. In fact, the maximum response is

attained only for an instant while the earthquake is taking place, and the maximum thereafter may be considerably less. Figure 18 gives a schematic illustration of the typical way in which a dynamic stress affects a structural member as compared with a static stress equivalent to the maximum value.

102. For the reasons given, the design spectrum which specifies the permissible level of stress will be less than the ideal spectrum corresponding to the maximum response. Moreover, where the seismic response takes place at levels near the breaking stress, there will be deformations greater than those corresponding to the elastic limit, and therefore there will be a dissipation of energy which will help to reduce the maximum amplitudes of the response to values below those for purely elastic behaviour.

103. This outline makes it clear that if one wishes to produce a design using a concept of static equivalent stresses, the latter must be less than the values of the maximum response but must guarantee the structure's capacity to withstand the maximum response for an instant, even though this does not necessarily fall within the elastic range.

104. The capacity of a structure and of each of its members to withstand without collapsing momentary stresses corresponding to the maximum response reflects the structure's capacity to absorb energy by means of deformation over a short period of time. This property of structures, which is known as ductility, must be achieved through appropriate design in order to build earthquake-resistant structures.

105. The problem of estimating the probability that an earthquake will occur at a specific place belongs entirely within the field of probabilistic analysis. The probability may be quantified in terms of risk using techniques of probability theory. The factors most commonly used to produce such an estimate are the following:

(a) Analysis based on the historical and seismological background of the area, weighting all potential sources of release of seismic energy so as to construct a seismic map which shows the seismic risk in each area;

(b) Assuming a probability distribution for the occurrence of earthquakes by comparing the area under study with an area of known seismicity and adjusting the distribution to the local level of seismicity.

106. A design spectrum is used for taking all these factors into account in specifying the resistance a structure should have in order to be able to withstand an earthquake. The design spectrum indicates the earthquake resistance which structures should have according to their vibration characteristics (period and damping).

107. Design spectra are based on general information concerning seismic response obtained from response spectra and are used to specify the earthquake resistance that structures should have. Using them, designers prescribe earthquake stresses with reference to a given level of permissible stress (or to the yield point), specifying or implying a capacity to withstand momentary peak stresses in the inelastic range. Unlike response spectra, design spectra do not define the characteristics of earthquakes.

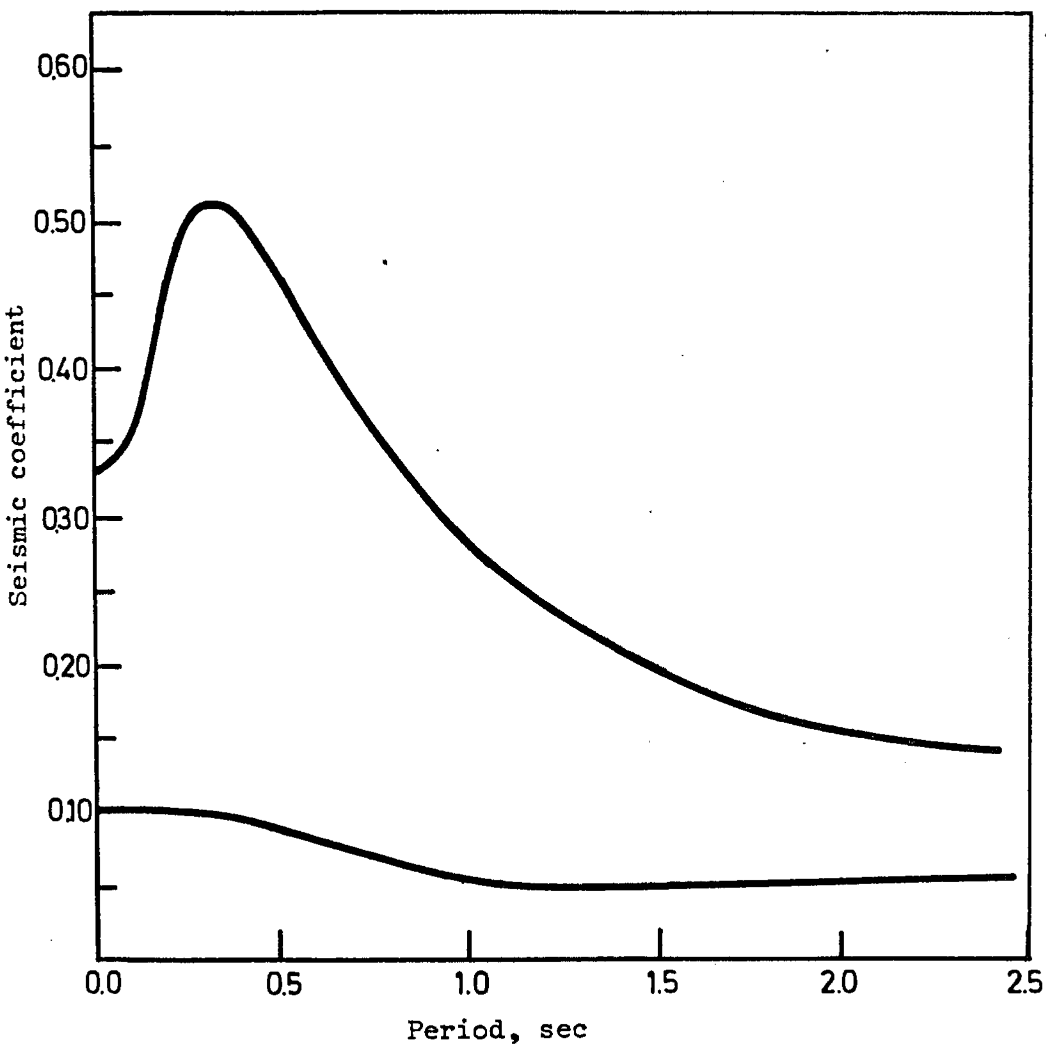


FIG. 19. COMPARISON OF A TYPICAL DESIGN SPECTRUM WITH THE IDEAL RESPONSE SPECTRUM OF FIG. 17 FOR 5% DAMPING

108. Figure 19 compares a typical design spectrum with the ideal response spectrum shown in Fig. 17 for 5 per cent damping. Such design spectra are used in regulations and codes for the design of earthquake-resistant buildings in order to specify the resistance that the structures must have.

III. STRUCTURAL MEMBERS UNDER EARTHQUAKE STRESSES

109. The great majority of conventional structures may be regarded, from the structural point of view, as consisting of assemblies of simple structural members combined in such a way as to form a unit capable of transmitting to the ground all the forces to which the structure may be subjected during its useful lifetime.

110. In this chapter we shall analyse separately the principle elementary structural members used as components in the construction of earthquake-resistant structures, particularly dwellings. For each member we shall study in a general way the stresses to which it may be subjected when an earthquake occurs, analysing in particular those areas in which the most unfavourable combination of forces may arise and which should therefore be considered with special care in the processes of design and construction.

111. Since the magnitude of the stresses to which a structure is subjected during an earthquake depends, among other factors, on its mechanical behaviour, it seems desirable to review first of all some fundamental concepts concerning the mechanical characteristics of materials.

A. Mechanical characteristics of materials

112. Whenever any body is deformed, it generates reaction forces which, for a given system of loads and a given shape of the body, depend both on the material of the body and on the magnitude of the deformations it undergoes.

113. Every material has its own specific way of responding to external excitation; this is called its "mechanical characteristic".

114. Among the various experimental tests that can be used for expressing in quantitative form the mechanical properties of materials, simple compression and tension tests stand out by reason of their simple execution and ease of interpretation.

115. In these tests the specimens are subjected to controlled deformations (strains), and the forces necessary to produce these deformations are systematically measured. The measurements recorded in these tests are plotted on a graph as shown in Fig. 20. The result is called a load-strain curve, and each material has its typical curve.

116. About the middle of the seventeenth century, Robert Hooke discovered that when the results of load-strain tests for specimens which were made of the same material but had different dimensions were plotted, the plotted points formed a single curve if the units used for the co-ordinate axes were unit stress (the load applied to the test specimen divided by its cross section) and unit strain (the strain or deformation of the specimen divided by its length). This is an extremely important fact, since it makes it possible to express the mechanical behaviour of any body made of a specific material on the basis of the results of a single test carried out on a standard specimen.

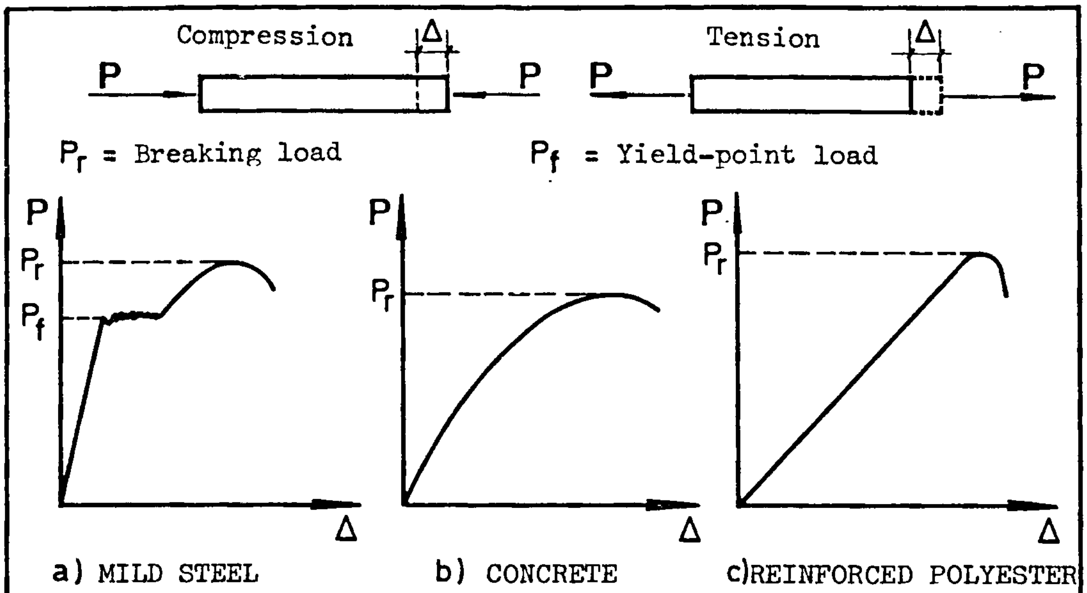


FIG. 20. TYPICAL LOAD-STRAIN CURVES

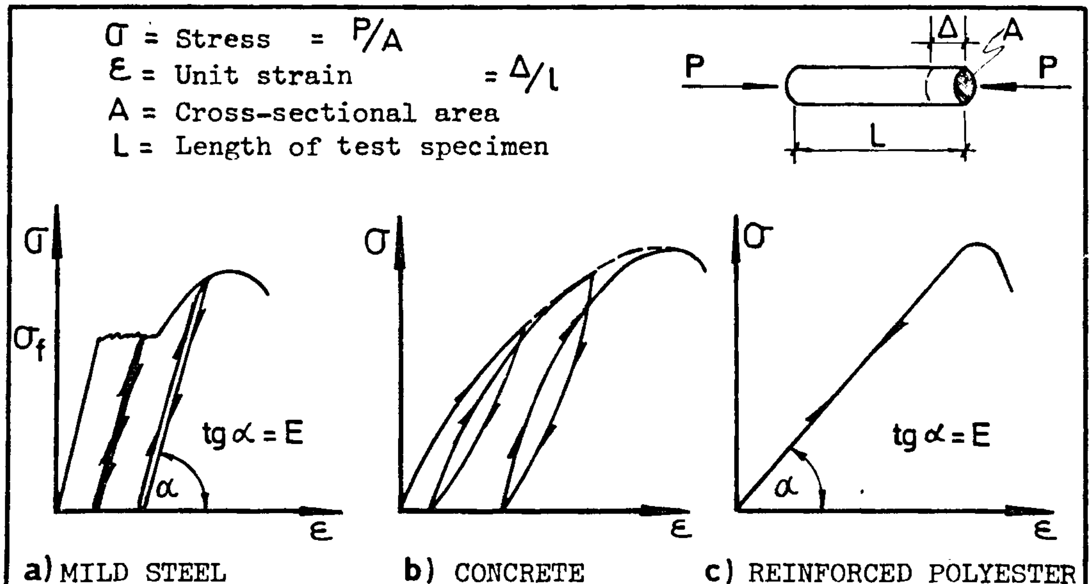


FIG. 21. TYPICAL CURVES OF UNIT STRESS VERSUS UNIT STRAIN FOR LOADING AND UNLOADING PROCESSES

117. Hooke also observed that for loads which are small in comparison with the maximum loads a specimen can resist, the load-strain curves are practically straight lines, that is to say, the strains produced are directly proportional to the stresses producing them. This proportionality between load and strain in a specimen (and hence between stress and strain in a material) is called Hooke's law, which is satisfied in practice by all materials regarded as elastic. In mathematical form, Hooke's law is expressed by the formula

$$\sigma = E \cdot \varepsilon,$$

in which

σ = unit stress = P/A

E = modulus of elasticity (a characteristic uniquely defined for each type of material)

ε = unit deformation = Δ/L

P = load applied to the specimen

A = cross-sectional area of the specimen

Δ = deformation undergone by the specimen

L = length of the specimen

118. If we compare the behaviour of test specimens for successively increased load values, we observe that there are fundamental differences for different types of material, as shown in Fig. 20. In steel, for example, the linear behaviour remains practically perfect until the so-called "yield zone" is reached; in this zone large deformations will be produced without a steady increase in load.

119. In concrete, on the other hand, the relation between load and strain is only very roughly linear, and even this approximate linearity is found only in the range of loads which are small in comparison with the breaking load.

120. In other materials, such as polyester reinforced with fibre glass, the linearity of the load-strain curve remains almost perfect until the specimen is subjected to the breaking load, the maximum load it is capable of resisting before fracture under the test conditions.

121. If we halt the loading of the specimens at any point before fracture takes place and gradually decrease the load, the resulting plot will be the "unloading curve", which is, in general, different from the loading curve. Figure 21 shows typical unloading curves for the three materials illustrated above in Fig. 20. The downturn of these curves after the breaking stress results from the fact that these tests are made by controlling the strain and not the load.

122. Lastly, it must be pointed out that in order to determine the results of load-strain tests and compare them in a systematic, simple and economical manner, it has been necessary to standardize them, basically by defining the dimensions of the specimens and the rate of application of the load.

123. In addition to a review of the concepts involved in Hooke's law, it is important to define and analyse briefly some concepts which relate to certain properties of the mechanical behaviour of materials and are fundamental to the discussion of structural analysis and design recommendations in the later chapters.

(a) Elasticity: As indicated above, all bodies are deformed when they are subjected to any system of loads, and they recover their original shape to a greater or lesser extent when the system of loads ceases to act. The degree of recovery depends on the maximum value of the load and, most importantly, on the material of the body.

In this sense, we define as an "elastic body" any body which completely recovers its original dimensions and shape once the applied loads cease to act.

With reference to the load-strain curve, this means that the unloading curve should be identical with the loading curve and coincide with it. If, in addition, it happens that the two curves, for loading and unloading, are straight lines, we say that the mechanical behaviour of the body is "linear-elastic". Such bodies satisfy Hooke's law both for loading and for unloading.

In reality there is no body which is perfectly linear-elastic; nevertheless, in practice, within certain load limits, the great majority of bodies satisfy Hooke's law with some degree of approximation, and from the practical point of view they may be considered linear-elastic bodies up to those load limits. As examples in this connexion, one may mention the case of reinforced polyester, which behaves elastically for any load, and that of steel, which behaves elastically within the range of loads below the yield point.

Bodies which do not recover fully when they are unloaded - such as lead, clay and practically all materials if they are subjected to loads higher than those which cause yield - are called inelastic bodies.

The limiting stress for which Hooke's law is satisfied by a given material is defined as its "elastic limit", or, more correctly, its "limit of linear behaviour" or "limit of proportionality"; this limit corresponds to the unit stress above which the strain begins to increase more rapidly than the stress and unloading of the body does not bring total recovery from the strain. Such permanent deformations are also known as "plastic deformations".

(b) Stiffness (or rigidity) and flexibility: The greater the applied load required to produce a given deformation, the greater the stiffness (of rigidity) of a body or member is said to be. Analytically, the "stiffness" or a member is expressed by the quotient obtained by dividing the load by the strain it produces; consequently, for members made of a linear-elastic material, stiffness depends only on the shape and size of the member and the mechanical characteristics of the material it is made of.

On the other hand, the greater the deformation produced in a body by the application of a given load, the more flexible the body is said to be. Mathematically, "flexibility" is defined as the reciprocal of stiffness, that is to say, the quotient obtained by dividing the strain by the load which produces it.

(c) Yield: Once the elastic limit is passed, any increase in load produces plastic or permanent deformations which are no longer proportional to the increase in load but take on increasing values for equal increments of load.

The point at which these large permanent deformations without increase in load begin to appear is called the "yield point" and is clearly identifiable in materials such as mild steel, shown in Fig. 21 (a). Other materials, such as concrete and reinforced polyester, have no yield point.

(d) Ultimate strength and breaking load: When any body is subjected to a system of external loads, it will be deformed until it develops the reaction forces necessary to balance the forces imposed upon it. However, this process ceases to be valid at high loads. It will always be possible to increase the external loads to such a point that the deformations begin to increase in an uncontrolled manner. In this case we may say that the "ultimate strength of the body" has been reached. The load at this point is called the "breaking load" and is independent of the stiffness of the member under consideration.

(e) Brittle materials and ductile materials: Materials such as reinforced polyester, which exhibit no significant inelastic deformation before fracture takes place, are called "brittle materials"; those which, on the contrary, undergo considerable inelastic deformation before fracture are called "ductile materials".

(f) Strain energy: Under a system of external loads, all bodies react by deformation. These deformations give rise to internal stresses which, in order to achieve equilibrium, must yield a resultant that is equal and opposite to the resultant of the system of applied loads.

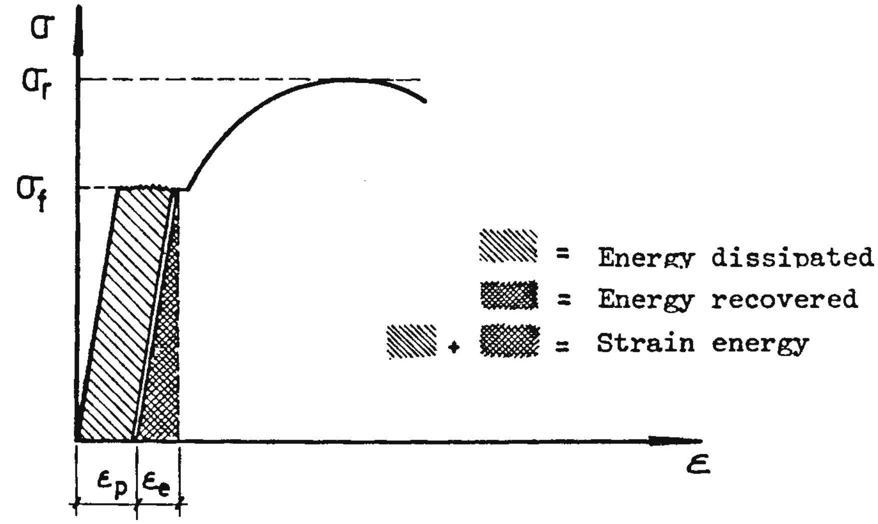
From the energy point of view, the work done by the external loads through the displacement of their points of application must be equal to the sum of the potential energy accumulated in the body and the energy dissipated in the form of heat.

The potential energy accumulated in the body is called the "strain energy". For a unit volume of any body subjected to pure tension or pure compression, it is expressed by the formula

$$W = \int_0^{\varepsilon} \sigma(\varepsilon)\,d\varepsilon.$$

Numerically this energy is equal to the area under the unit stress-unit strain curve. Figure 22 shows the deformation work done during the loading process and the work recovered during the unloading process. The difference between these two amounts of work is equal to the energy dissipated in the form of heat during the process of loading and unloading the body under consideration. For elastic bodies the energy dissipated will be zero, since when such bodies are unloaded, they must return to their initial position along the increasing-load curve.

(g) Elasto-plastic materials: The great majority of real materials exhibit plastic deformation when they are subjected to high stresses. In order to be able to consider these characteristics of yield and hysteresis (the energy dissipated in successive loading and unloading processes) in the analysis of structural members, while keeping the degree of complexity of the calculations within acceptable limits, it is customary to replace the actual stress-strain curve of the material under consideration with straight segments which approximately represent



ε_p = Permanent deformation ε_e = Elastic deformation

FIG. 22. STRAIN ENERGY

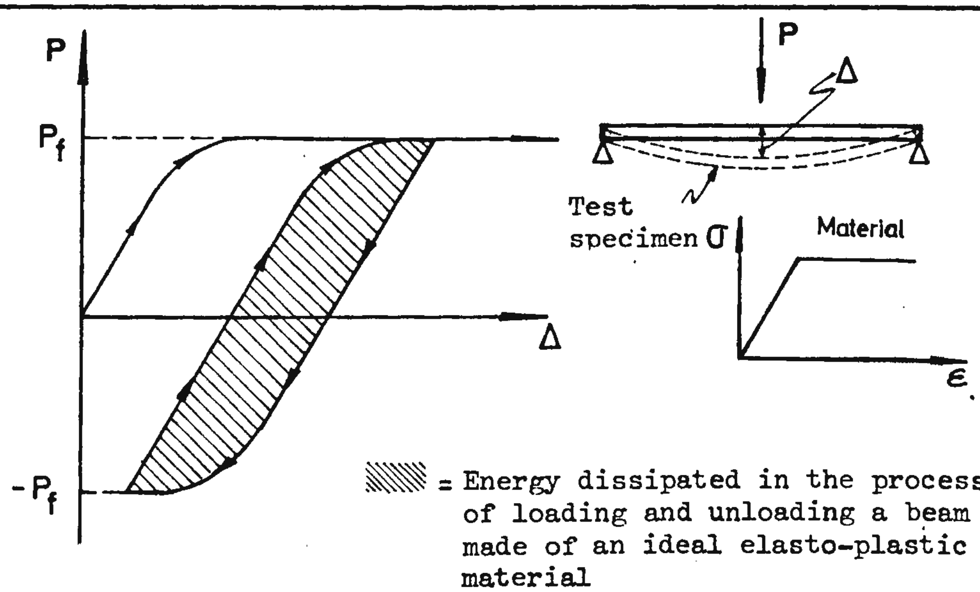


FIG. 23. STRAIN ENERGY: IDEAL ELASTO-PLASTIC MATERIAL

the behaviour of the material. We thus obtain materials with bilinear behaviour, the curves for which are shown in Figs. 23 and 24.

(h) Ductility: The capacity of materials to absorb energy through permanent deformations before failure takes place is known as ductility, a property of great importance in all phenomena in which the body under consideration is subjected to slow alternating stresses with relatively high load values, which occurs, *inter alia*, in the case of earthquake stresses.

Ductility can be expressed by the ratio of the total energy a body can absorb without failure to the maximum energy it is capable of absorbing within the elastic range; it can also be expressed by the "ductility factor", which is the ratio of the maximum possible deformation before failure to the maximum elastic deformation.

(i) Admissible stresses: Members of identical form which are constructed of the same material do not in general behave identically. There are variations, depending on the type of material, and these can be studied statistically. Something similar takes place in the case of loads on members intended for the same purpose. In view of this, and for safety reasons, members must be designed for stresses less than the breaking stress. It is customary to define these design stresses as a fraction of the material's yield point or breaking stress.

These design stresses are called "admissible stresses" and depend both on the material and on the use for which the member is intended.

For structures made of materials whose behaviour is perfectly linear-elastic, design with "admissible stresses" means using a safety factor for the structure (the quotient obtained when the maximum load resisted by the structure without failure is divided by the working load) equal to the quotient obtained by dividing the breaking stress by the admissible stress of the material used in the structure.

On the other hand, in a structure made of materials whose behaviour is non-linear (and this includes, in general, all materials in elevated load ranges), the direct ratio between the safety factor for the structure and the safety factor adopted for the stresses is no longer applicable unless the structure is isostatic. In these cases the structure is designed for fracture, with load values equal to the working loads multiplied by the desired safety factor.

B. Stresses and strains

124. From chapter II it can be concluded that the magnitude of the seismic stresses to which the various members in a structure are subjected depends both on the characteristics of the earthquake and on the characteristics of the structure itself.

125. Generally speaking, a structural member will always be subjected to a dynamic oscillating force which may be exerted in any direction and is usually analysed by decomposition into its component forces along the principal directions of the member.

126. Before analysing the behaviour of structural members under a seismic stress, the main simple stresses to which those members may be subjected will be described briefly.

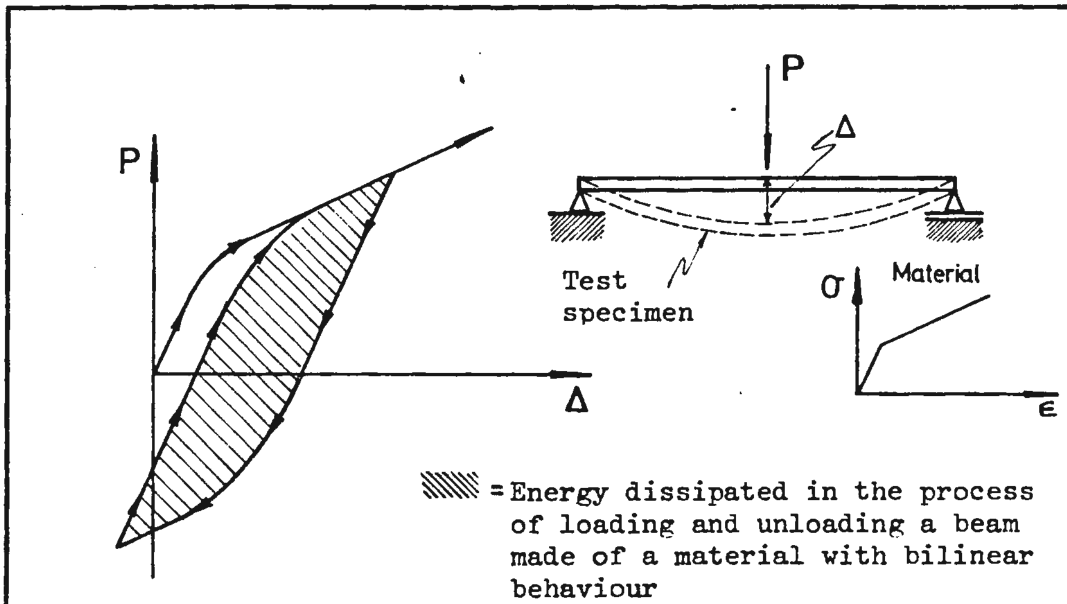


FIG. 24. STRAIN ENERGY: MATERIAL WITH BILINEAR BEHAVIOUR

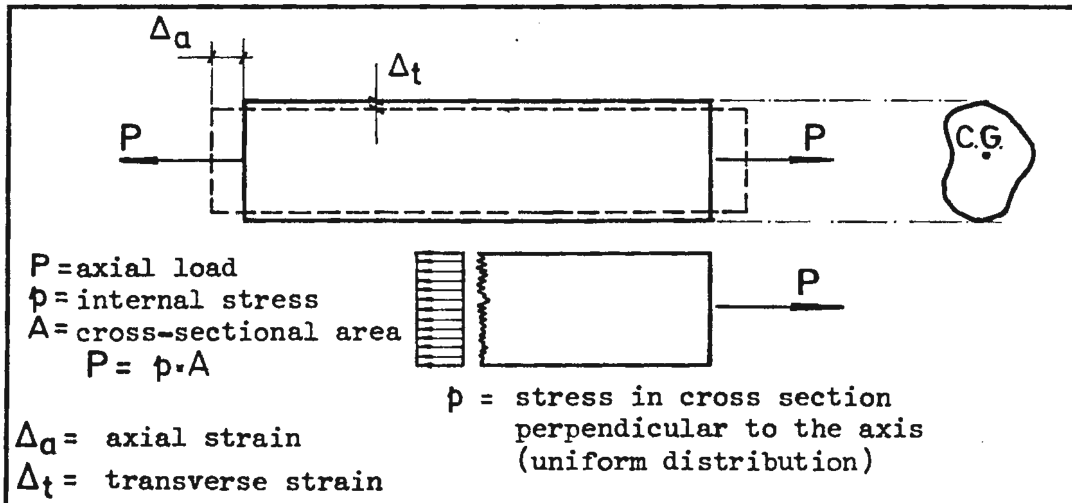


FIG. 25. MEMBER SUBJECTED TO TENSION

(a) Tension: A body is said to be in tension if two divergent and equal forces with the same line of action act on it. If this line of action coincides with the centre of gravity of the cross section of the member, one speaks of "simple tension".

In this case, the distribution of stresses in a cross section perpendicular to the line of action of the forces will be uniform provided that a cross section sufficiently far from the point of application of the external loads is selected and that the cross section itself is homogeneous. Figure 25 shows the case of a prismatic bar and indicates the longitudinal elongations and the transverse contraction which will result from the action of the loads.

(b) Compression: In elementary theory, compression, which is the stress produced on a body when it is acted upon by two equal forces directed towards each other along the same line of action, is the opposite phenomenon to tension; the effect on the compressed body is a decrease in its length along the axis of application of the forces and an increase in its breadth.

However, there is a fundamental difference between the processes of tension and compression in cases where the element in question is "slender", in other words, where its cross section in a plane perpendicular to the action of the forces is substantially smaller than its other cross sections. In such a case, the process of compression may take the form of an unstable phenomenon known as "buckling", in which the member subjected to compression fails not by crushing but by lateral deformation. The maximum load a member is capable of resisting before it buckles is called the "critical load" and may be much less than the maximum crushing load for the material of which the member is made.

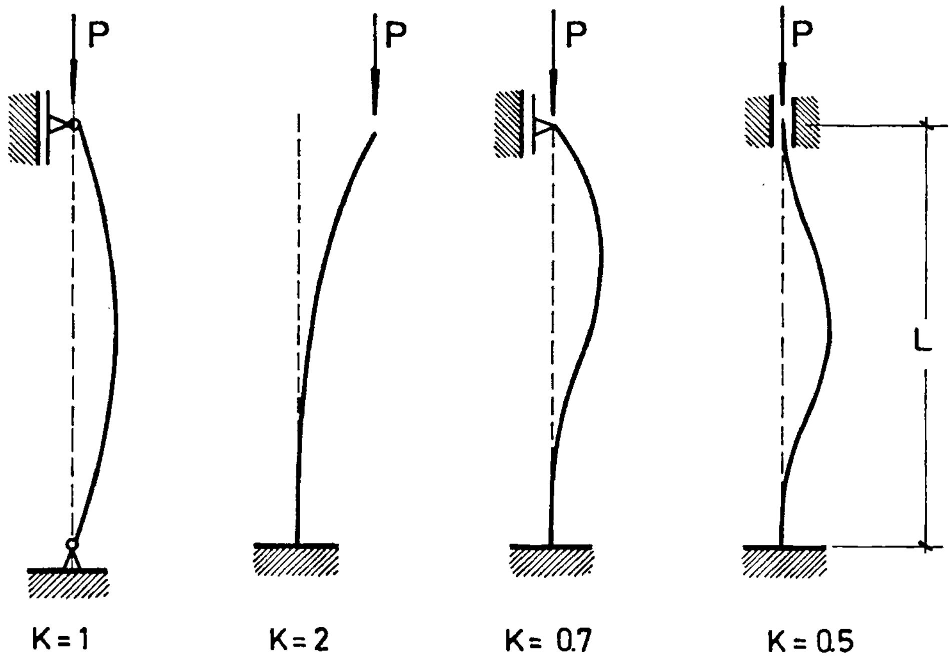
The shape assumed by the member when it fails by buckling (the mode of buckling) depends on its external attachment. The diagrams of Fig. 26 illustrate some of the cases which occur most frequently in practice.

(c) Flexure (bending): Any stress which tends to curve or bend the body on which it acts is called flexure (or bending). The amount of curvature produced in a member under flexure depends on the shape of the section in question, the characteristics of the material of which the member is made and the moment acting through the section. This moment is called the "bending moment". In general the following basic cases may be distinguished:

(i) Pure flexure: Here the stresses acting through any part of the member are only moments. The resultant forces parallel and perpendicular to the section are zero, and consequently, from considerations of static equilibrium, the bending moment must be constant throughout the zone of pure flexure. This also means that in the case of prismatic members the curvature must be constant throughout the zone of pure flexure.

(ii) Simple flexure: In simple flexure the resultant of the stresses transmitted through parallel sections includes not only the moment but also a component parallel to the section, which is called the "shear force". This occurs very frequently in practice, since it is present in most beams.

(iii) Compound flexure: One speaks of "compound flexure" when a normal stress is added to the stresses of simple flexure or pure flexure. This is seen



π = 3.1416

E = Elastic modulus of the material

J = Least moment of inertia of the section

K = Constant for external attachment

L = Length of member

$$P_{crit} = \frac{\pi^2 \cdot E \cdot J}{K \cdot L}$$

FIG. 26. MODES OF BUCKLING

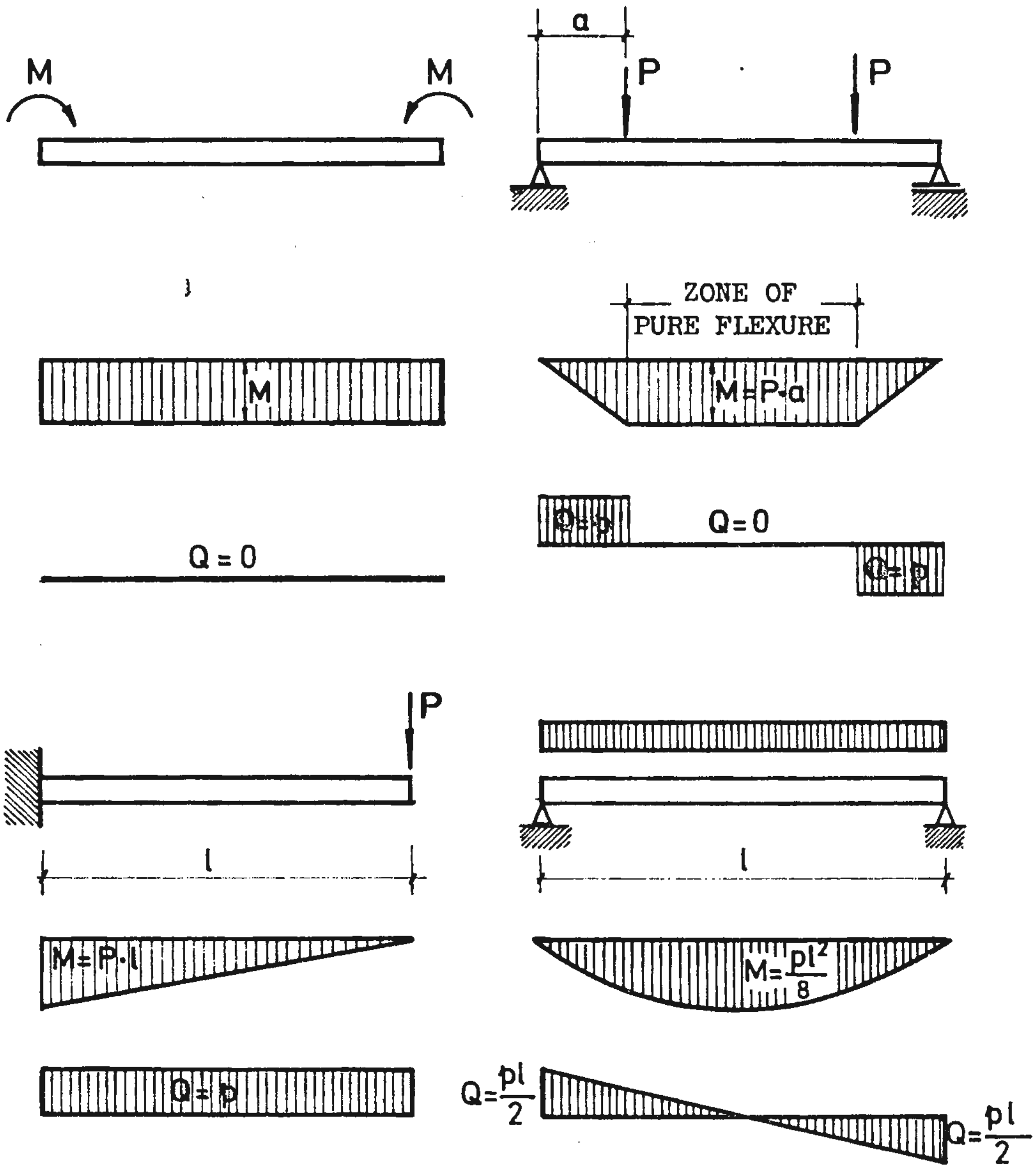


FIG. 27. MEMBERS SUBJECTED TO FLEXURE

commonly in columns which are subjected to eccentric loads or form part of frames. Figure 27 shows some representative examples.

For slender members made of homogeneous material, Navier's hypothesis that the sections remain plane during the process of flexure can be accepted. This is equivalent to saying that for straight members made of a homogeneous material which obeys Hooke's law, the distribution of internal stresses is linear. Such a distribution for the case of a rectangular section is shown in Fig. 28, which also shows how the distribution will change as the stress increases and the section becomes plastic.

(d) Shear force: The force of interaction between two parts of a body along the shearing surface is called the "shear force". It is not physically possible to produce this force in a pure state; it always appears in combination with other forces. However, in the case of materials which obey Hooke's law, it is possible to make a separate estimate of the deformations caused by the shear force. Figure 29 shows some typical simple examples and indicates both the distribution of the shear force along the member and the distribution of shearing stresses in a typical section. It should be noted that, in general, in the design of bolted, nailed or riveted joints between timber or steel members, the decisive limitations are those by the shear force in the members meeting at the joint.

(e) Torsion: If moments produced by couples of forces acting in parallel planes - in other words, moments with parallel representative vectors - act on the ends of a body, the body is said to be subjected to "torsion".

The torsions generated by this stress in the body correspond exclusively to shearing stresses which have the additional property that they have a zero resultant when integrated over any section orthogonal to the axis of the member.

In transmission shafts, torsional stress is usually preponderant, although it is almost always combined with flexure, compression or tension. The diagram in Fig. 30 shows the case of a prismatic shaft with circular cross section subjected to pure torsion.

C. Structural members under seismic stress

127. This chapter will analyse very briefly the main characteristics of the behaviour of simple structural members which, when properly combined, form a structural framework which will withstand both vertical static loads and seismic, wind and other loads.

(a) Simple frames: The simple frame is a very commonly used elementary structure. The loads to which it is subjected are usually much greater than the weight of the frame itself. This means that the problem can be idealized by considering both the weight of the frame itself and the live load acting on the frame to be concentrated at the upper edge of the frame.

As was stated in chapter II, seismic action can be analysed as a static force confined to the horizontal plane which passes through a beam and whose magnitude will depend on the weight of the live load and the weight of the member under consideration, as well as on the elasticity of the frame (the characteristic period).

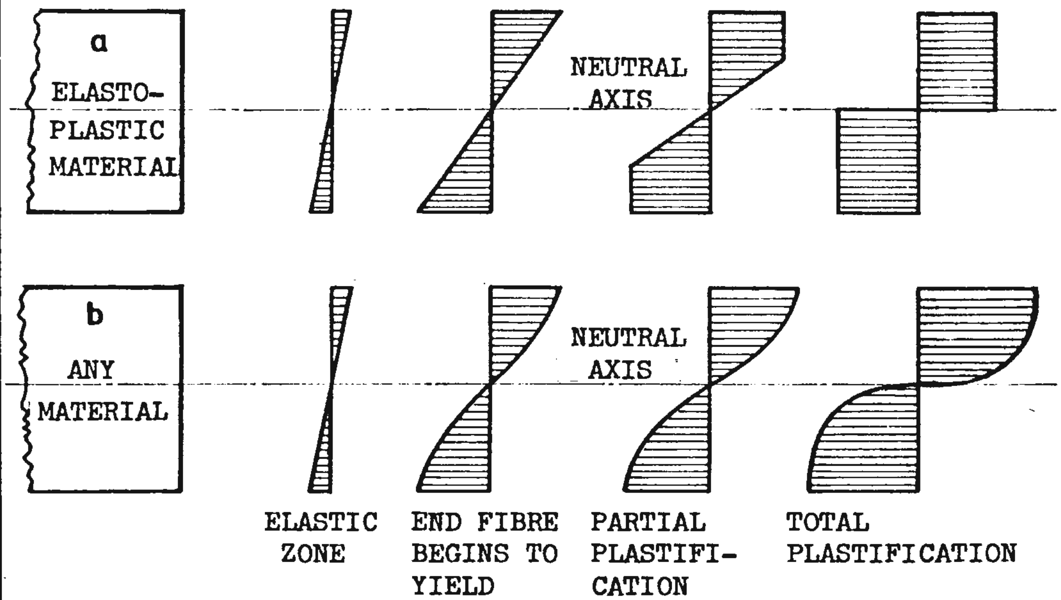


FIG. 28. STRESSES CAUSED BY FLEXURE

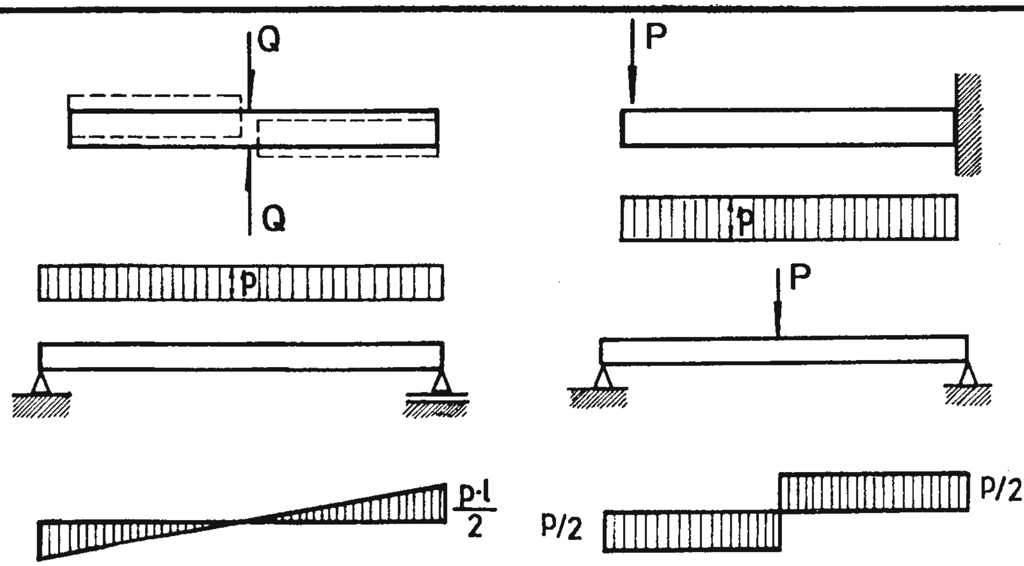


FIG. 29. DIAGRAMS OF SHEAR FORCE

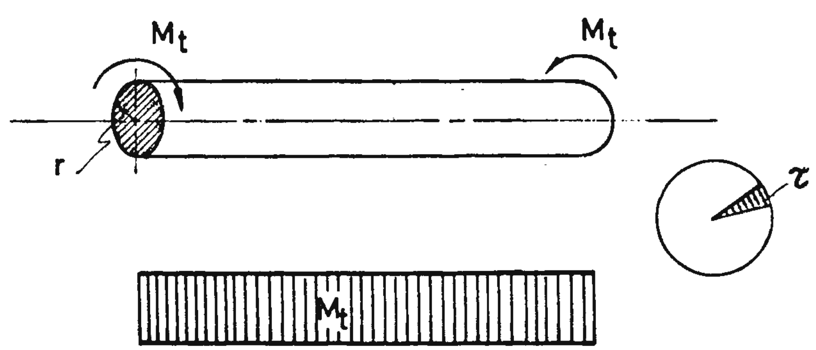
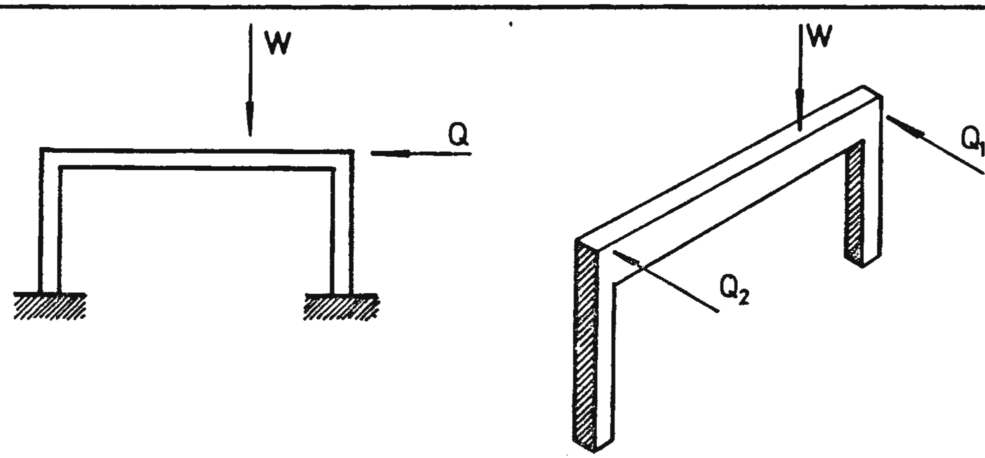


DIAGRAM OF TORSIONAL MOMENT

FIG. 30. SHAFT SUBJECTED TO PURE TORSION



W = load representing the static loads

Q , Q1, Q2 = loads representing seismic action

FIG. 31. DIAGRAM OF SEISMIC ACTION ON A SIMPLE FRAME

Since the direction in which the earthquake will act is not known in advance, the earthquake forces must be analysed separately. First, the earthquake forces acting in the plane of the frame are considered, and then those acting perpendicular to that plane. Any intermediate position can be decomposed into components in those two principal directions.

Figure 31 shows a diagram of the principal seismic loads to which the simple frame may be subjected. The corresponding internal stresses are shown in Fig. 32. It can be seen that the greatest stresses always occur at the ends of the beam and the columns which form the frame. This, together with the fact that such end areas generally present construction problems (concrete joints, reinforcement splices and joints in general), makes it necessary to give special attention to the design and specifications for those areas.

(b) Simple frames with cross bracing: Frames with cross bracing designed to bear tension only or to bear both tension and compression are frequently used in steel structures. These two kinds of bracing are highly dangerous in the event of dynamic stresses. In the first case, if the bracing is not properly arranged, the resistance of the compression brace to dynamic buckling may be of the same order of magnitude as the maximum stress, so that the tension brace will absorb the stress not gradually but as an impact stress, with the corresponding amplification. In the second case, for certain geometrical relations and load values the load is not distributed equally between the two braces. The compression brace receives a much greater fraction of the load, and this gives rise to a serious danger of progressive failure. Accordingly, for large structures only the use of a single brace designed to withstand compression, as is shown in Fig. 33, can be recommended.

At the points of attachment of the brace to the basic structure there will be local stresses of great magnitude which must be considered with special care in the design and construction phases.

(c) Confined-masonry walls: A masonry filling, confined by columns and a tie-beam of reinforced concrete, is used to transmit horizontal earthquakes or wind loads to the ground. Experience has shown that this combination withstands seismic loads well. The main problems which may affect its performance are lack of bonding between the tie-beam and the masonry, insufficient weight and poor confinement of the masonry.

Figure 34 shows a diagram of the internal stresses produced when the tie-beam and the masonry are not well bonded. It can be seen that the upper ends of the columns are subjected to large shear forces, while the masonry is under heavy pressure at those points. When the masonry is not confined, usually in masonry laid after the tie-beam has been concreted, the brittleness of the unconfined brickwork is added to the previous problem.

(d) Reinforced-concrete walls: In a longitudinal direction, reinforced-concrete walls act as high girders; it should be borne in mind that in such cases the deformations caused by shear and due to rotation of the foundations are comparable to deformations caused by flexure and must therefore be taken into account in calculating the relative rigidities of the different elements which make up the structure.

(e) Frames with several bays and several floors: Once the stresses are known, frames with several bays and floors present problems similar to those of the simple frame, and therefore we shall not discuss them in detail.

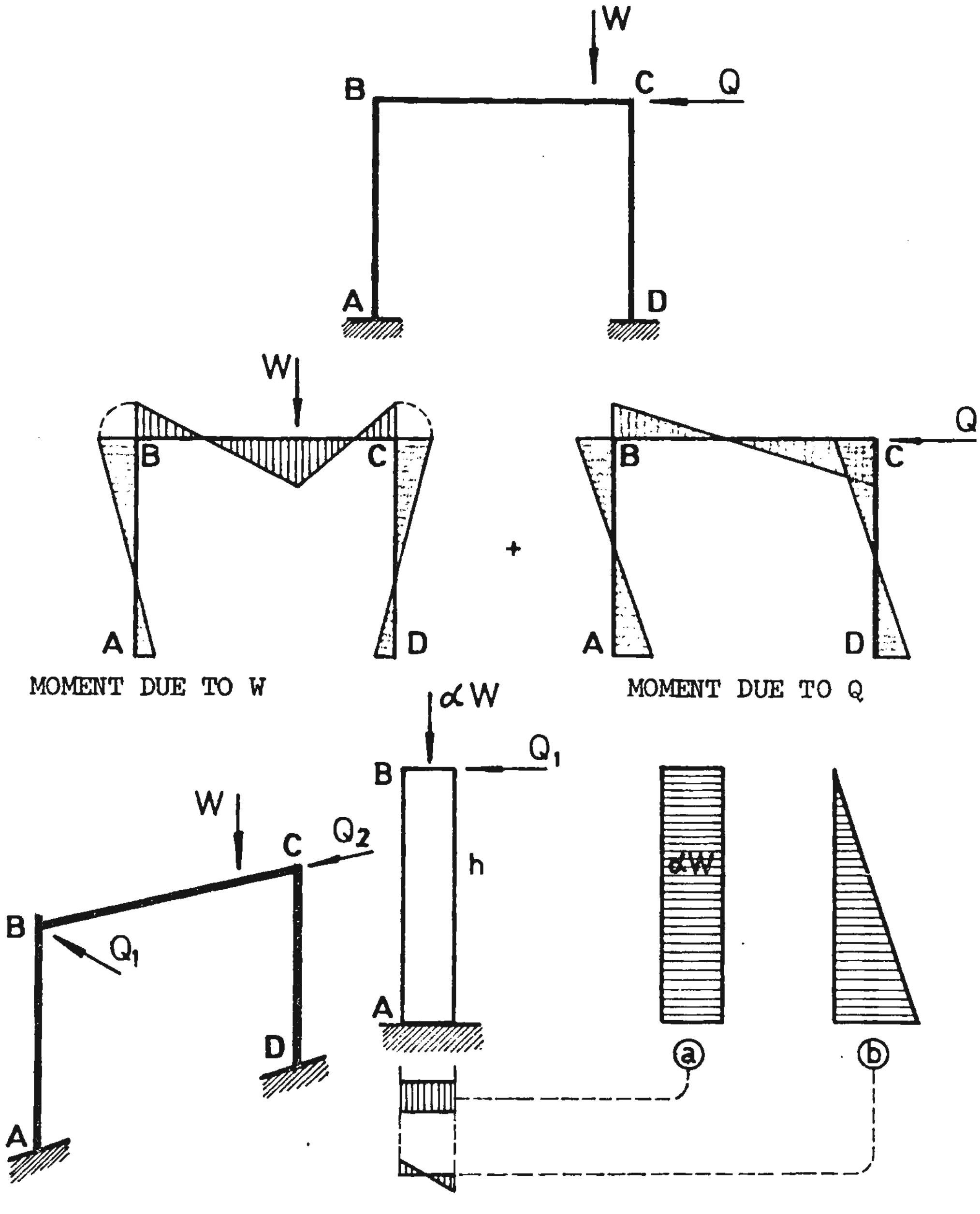
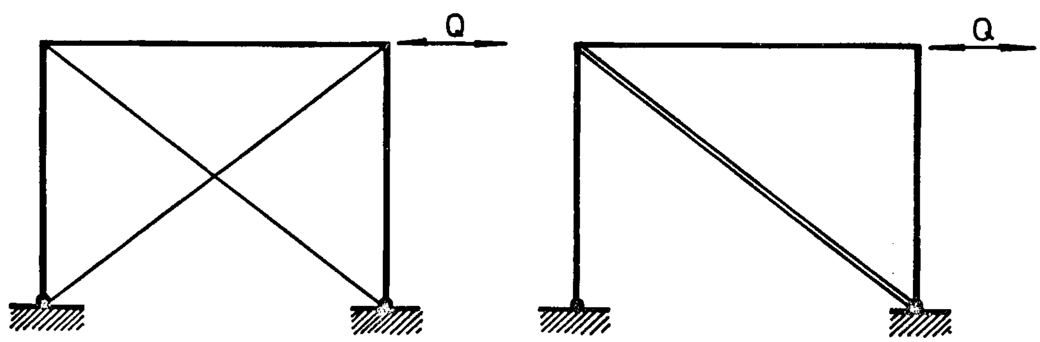


FIG. 32. INTERNAL STRESSES IN SIMPLE FRAMES SUBJECTED TO SEISMIC LOADS



(a) PRESTRESSED BRACES DESIGNED FOR TENSION

(for light structures only)

(b) SINGLE BRACE DESIGNED FOR COMPRESSION

FIG. 33. FRAMES WITH CROSS BRACING

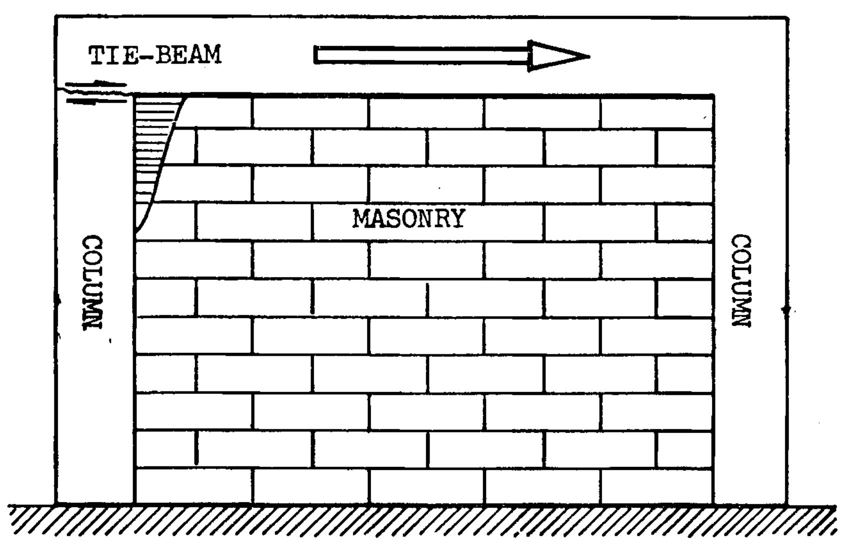


FIG. 34. INTERNAL STRESSES IN CONFINED-MASONRY WALL WITHOUT BONDING BETWEEN TIE-BEAM AND MASONRY

IV. STRUCTURES SUBJECTED TO HORIZONTAL STRESS

128. The structure of a building is composed of a set of structural members so arranged as to form a resistant framework, which is indispensable if the building is to withstand the stresses to which it will be subjected during its useful lifetime.

129. In chapter III we analysed simple structural members and how they behave when subjected to horizontal forces such as those which generally occur in a structure under seismic stress. It should be pointed out that this analysis was based upon the assumption that the structural members involved were planar. With such members the whole problem is confined to the plane of symmetry of the member - which, in turn, is the plane of symmetry of each one of its constituent parts - and all the forces that act upon the member also lie in that plane, or at least are arranged symmetrically with respect to it.

130. That being so, the structural member - for example, a multi-storey bent - will become deformed but will remain in the same plane of symmetry, since the displacements will be parallel to that plane and the rotations will take place about axes that are perpendicular to that plane, and consequently the moments in each section will produce flexure but no torsion.

131. These structural members - each of which has its own plane of symmetry - combine to form the structure of the building, which constitutes a special assembly. Nevertheless, under the conditions that usually occur in practice, it may be assumed in most cases that, even when subjected to seismic loads, the various members forming the structure will remain in their plane during the deformation process and can therefore be analysed as plane structural members without giving rise to serious error.

132. The deformation of individual members depends upon their rigidity and the magnitude of the load to which they are subjected, and these magnitudes are closely related to the concepts of centre of rigidity and centre of mass of the assembly.

133. The centre of rigidity is the centre of gravity of the rigidities of the structural members in the ground plan of the building.

134. Rigidity represents the resistance of the member to deformation. The centre of rigidity is therefore the point at which the reaction of the structure as a whole is concentrated.

135. The centre of mass is the centre of gravity of the masses of the various structural members and non-structural elements making up the building. Generally speaking, the mass of a building is more or less uniformly distributed and its centre of mass will coincide with, or be very close to, the geometrical centre of the ground plan. The seismic force acting on the building during an earthquake will act at the centre of mass.

136. The relative position of the centre of mass (action) and the centre of rigidity (reaction) will depend on the distribution of the seismic structural members (see Fig. 39 below).

137. These various structural members, appropriately distributed and oriented in the building's ground plan, may or may not be joined to one another at the level of each floor by highly rigid diaphragms that make is possible to distribute the horizontal forces appropriately among the resistant members; these two cases give rise to two types of structure which are fundamentally different when considered from the standpoint of horizontal stress (Fig. 35).

A. Buildings with non-deformable floors

138. In these cases the seismic shear force applied at the level of each floor will be transmitted by the rigid diaphragm (floor slab) to the seismic members (walls or bents) in such a way that each seismic member is deformed in a manner compatible with the condition that the diaphragm remains rigid.

139. In the more general case, each storey will undergo a displacement along each principal axis and a rotation. This deformation will affect each of the seismic members. This behaviour of the structure is much simpler, from the technical standpoint in the particular case of a symmetrical floor plan, when the shock follows the direction of one of the principal axes. In that case there will only be a displacement along this principal axis.

140. On the other hand, in the general case the movement is like that of a structure with an asymmetrical floor plan.

(a) Symmetrical floor plans

These are floor plans in which the members resistant to horizontal forces are arranged symmetrically (so far as rigidity and position are concerned) with respect to their principal axes. As the mass of a building is generally distributed more or less uniformly in the floor plan, a symmetrical floor plan ensures that the centre of mass and the centre of rigidity will coincide.

In such cases, therefore, only displacements will occur when the structure is subjected to seismic forces. The displacement will occur along the axis which coincides with the direction of the horizontal seismic force, or, if the seismic force does not act along a principal axis, the displacement will have components along both horizontal axes.

If the principle of superposition is considered valid, the most unfavourable situation will occur when the seismic shear force follows the direction of one of the principal axes, since in such cases the displacement will occur in only one direction, with the total shear force concentrated in that direction.

With a rigid diaphragm which keeps the deformations equal, the total shear force acting on the floor will be distributed among the seismic members in proportion to their rigidities.

Naturally, this analysis must be conducted in both principal directions.

(b) Asymmetrical floor plans

In this case, as the centre of mass does not coincide with the centre of rigidity, there will be not only displacements along the principal axes of the floor

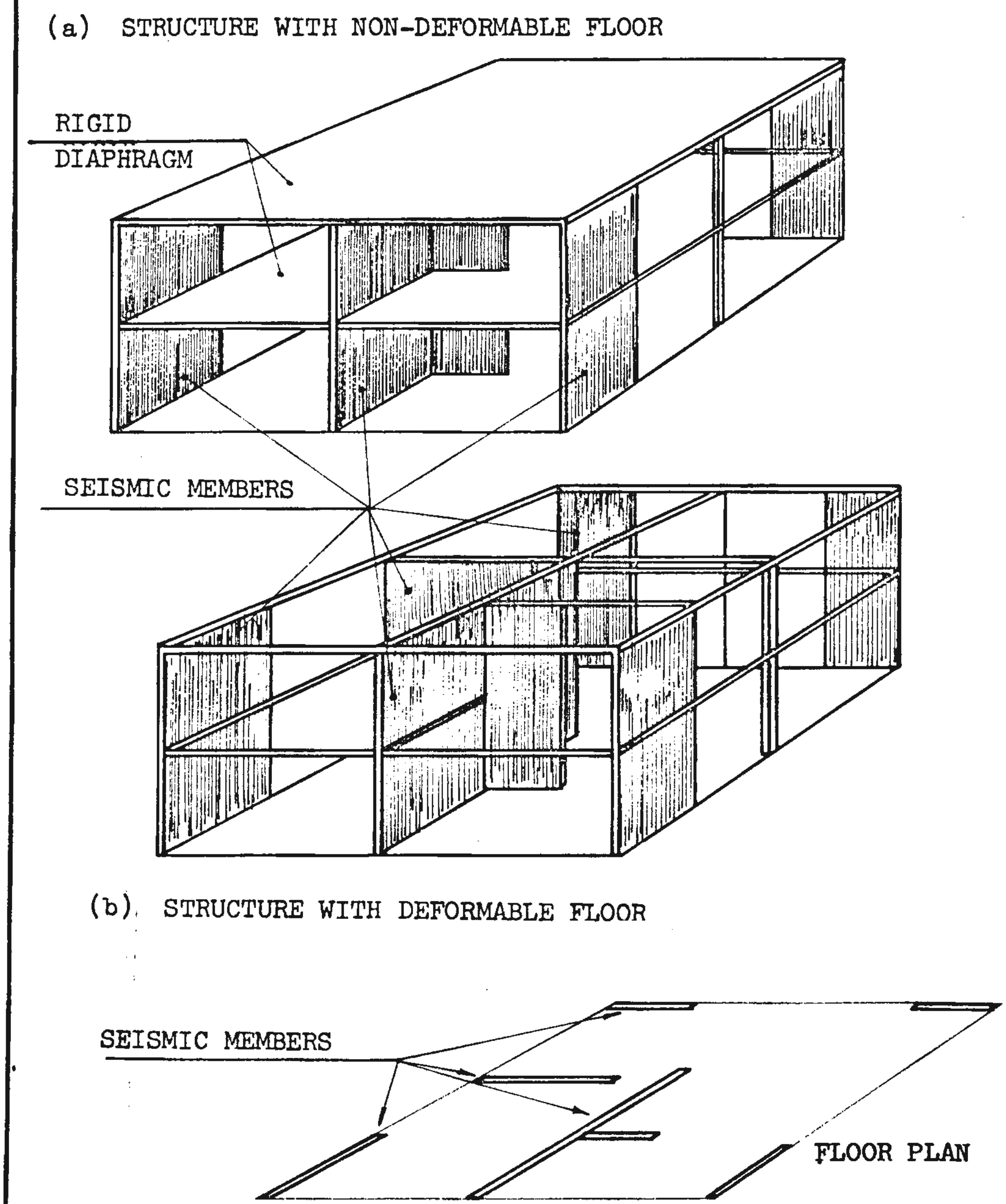


FIG. 35. PERSPECTIVE DRAWING OF TWO STRUCTURES WITH THE SAME FLOOR PLAN, THE FIRST WITH A NON-DEFORMABLE FLOOR, THE SECOND WITH A DEFORMABLE FLOOR

plan but also a torsion which will increase in proportion to the eccentricity of the centre of mass with respect to the centre of rigidity.

141. In addition, owing to the dynamic characteristics of the seismic stress, this static torsion will cause a rotational vibration which will increase the effect. Few studies of the phenomenon have been made as yet, but in most building regulations it has been deemed advisable to take these effects into account by appropriately amplifying the static eccentricity.

142. Thus, thd distribution of the shear force among the seismic members in each principal direction is affected by the presence of this torsion in the horizontal plane. Since the principle of superposition is assumed to be valid, the torsional moment (the shear force multiplied by the distance between the centre of mass and the centre of rigidity) introduces new forces, appropriately amplified, in each of the seismic members, and these new forces will modify the forces obtained from the distribution directly proportional to the rigidities of the members. The new forces which introduce the torsional moment in the horizontal members are directly proportional to the rigidities of the members and to their distances from the centre of rigidity.

143. It should be pointed out that these elements are generally characterized by considerable rigidity in one principal direction, very little rigidity in the other and negligible torsional rigidity.

B. Structures with deformable floors

144. The floor is not a rigid diaphragm, and consequently there is no condition keeping the deformations compatible. Each seismic member will bear that part of the force which acts directly upon it. In such a case there is no possibility that the more resistant members will help the less resistant members by bearing part of their load, as happens in structures with non-deformable floors. This means that, on the whole, deformable floor structures are less safe. This type of structure is not recommended for earthquake-resistant construction, except in the case of smaller buildings in which there is no reason to expect very large shear forces.

V. THE FOUNDATION SOIL

145. The foundation soil of buildings must be regarded as one of their fundamental structural members. The deformations and resistance of the soil should be dealt with as part of the design in the same way as beams or walls. Since it is generally not possible to change the soil conditions, the structure and foundations should be suited to the specific characteristics of the soil at the building site.

146. Knowledge of the foundation soil is essential to correct earthquake-resistant design. In the case of a design in which only static loads are considered, it may be more economical to compensate for an insufficient knowledge of soil properties by providing a greater margin of safety in the design. However, for a proper earthquake-resistant design it is necessary to know the mechanical properties of the foundation soil, since in some cases a soil which behaves well under static loads will present serious problems when subjected to seismic loads, and it will not always be possible to remedy those problems by using a larger safety factor for static loads.

147. A detailed study of the mechanical properties of soils is beyond the scope of this work and belongs to the field of soil mechanics, which is a specialized branch of civil engineering. In this chapter we shall only make some useful recommendations directly applicable to one-storey dwellings and, at the same time, propose some general criteria enabling those responsible for construction work to establish whether the foundation soil might raise special problems requiring the services of a specialist in soil mechanics.

148. Conditions relating to foundation soils will be dealt with under two headings: (a) a study of the seismic properties of the subsoil itself, which are not significantly affected by dwellings and their foundations, and (b) interaction between the soil and the foundations of dwellings, which will have a definite influence on the design of the latter.

149. The first group of problems will include, for example, the influence of the subsoil on the characteristics of seismic movements, landslides and loss of soil resistance. The second group of problems includes soil deformations caused by loads transmitted to the soil by foundations and the settling of foundations under static and seismic loads.

A. Exploration of the foundation soil

150. The first step in the exploration of the foundation soil consists in determining its geological history, which makes it possible to obtain an approximate idea of the general characteristics of the stratigraphy of the land and the properties of the soils composing it.

151. Since low-cost one-storey dwellings do not place great loads on the soil, the depth to which detailed exploration must be carried is relatively shallow. However, for major projects it might be advisable to explore the soil to considerable depths if it is desired, for example, to study the effects of the subsoil on the characteristics of seismic movements. Since such a study goes beyond the scope of the project itself, the methods of exploration for such cases will not be dealt with here.

152. The simplest method of exploring the conditions of the soil at shallow depths consists in digging pits or ditches, in the walls of which the soil stratigraphy may be observed, and, if necessary, samples for the required tests may be obtained. The pits must be deep enough to include any soil which might have a significant effect on the deformation of foundations. This depth is more than 2 metres for low-cost one-storey dwellings. The width of the pits or ditches must be at least 1.5 m in order to provide adequate space for a person inside to obtain samples.

153. It is not possible to lay down norms of general validity for the spacing of exploration pits or ditches, since it depends not only on the type of building planned but also on the uniformity of the subsoil stratigraphy. It is suggested that on land intended for a large number of dwellings, a preliminary exploration by means of pits spaced 50 m apart should be carried out, after which it would be possible to determine the location of additional pits in order to complete the final exploration.

154. The exploration pits or ditches should provide the following information:

(a) Description and classification of the various types of soils and the depths at which they lie;

(b) The depth of the water table; and

(c) Other observations which may be of interest, such as the difficulties encountered during excavation, the stability of the pit walls, water seepage, and the like.

B. Influence of the subsoil on seismic movements

155. Soils rest on bedrock, through which seismic waves are transmitted to them. During an earthquake the behaviour of the subsoil with respect to the bedrock is similar to that of a structure with respect to the soil. The thickness of the various soil strata and their elastic properties determine the type of seismic movement which will be produced at the soil surface by a given movement on the surface of the bedrock. The soil characteristics thus modify the intensity of the seismic movement and also modify its frequency content or response spectra.

156. From observations of the effects of earthquakes at comparable distances but in places with different types of terrain it can be concluded that the intensity may vary by as much as several degrees on the modified Mercalli scale owing to the influence of the subsoil. Intensities are greater in soft terrains than in firm terrains. Figure 36 sums up the observations made after the 1949 Puget Sound earthquake in the state of Washington, USA, when there were recorded differences of up to four degrees of intensity between rocky terrains and soft soil at the same distance from the epicentre.

157. The characteristics of the subsoil also have an important effect on the frequency content of the surface movement. Figure 37 shows two response spectra based on recordings obtained from different places during an earthquake in Japan in 1963. Since the reaction of a structure to an earthquake depends on the value of its characteristic period in accordance with the response spectrum, it may happen that in soft terrains with a response spectrum giving high acceleration values for high periods, more rigid structures (those with short characteristic periods) suffer less damage than more flexible structures. The opposite conclusion may be drawn for buildings with foundations on firm soil or bedrock.

158. In a city it is possible to determine for typical stratigraphic profiles the shape of the response spectrum and, in particular, the characteristic periods of the structures that would be subjected to the greatest stresses during an earthquake.

C. Soil stability during an earthquake

159. The stability of slopes and of foundations supported by soil is determined by soil resistance to shear. During an earthquake there will be increases in the shear forces acting on the soil, and this will inevitably produce deformations of slopes and of foundations resting on the soil. The magnitude of the deformations will, of course, depend on the magnitude of the earthquake and the stress-strain

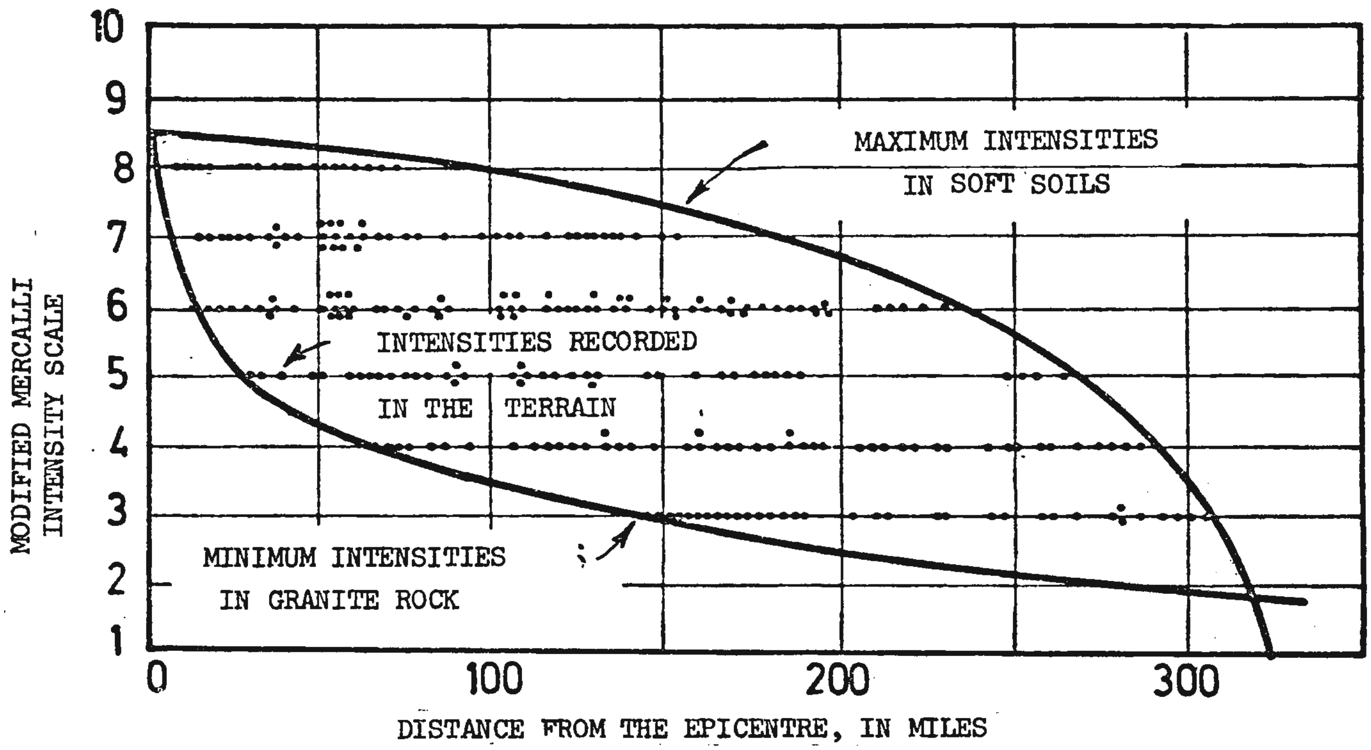


FIG. 36. SEISMIC INTENSITIES BASED ON TERRAIN STUDIES AT VARIOUS DISTANCES FROM THE EPICENTRE. PUGET SOUND EARTHQUAKE (APRIL 1949). (NEUMAN)

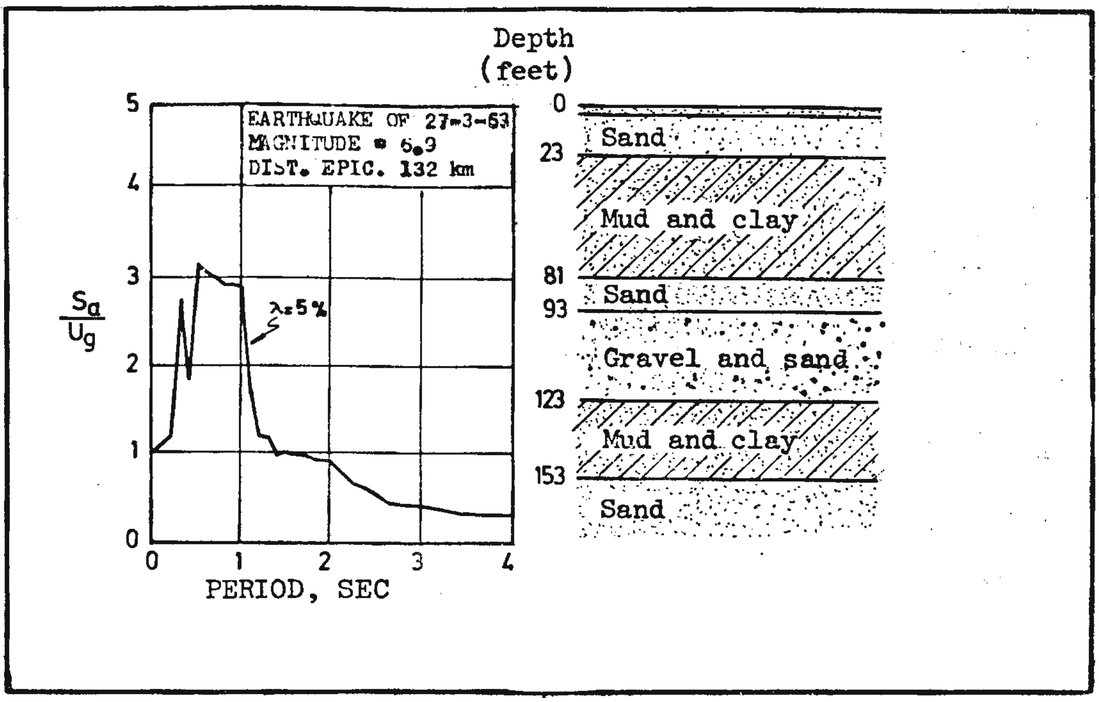


FIG. 37 (a)

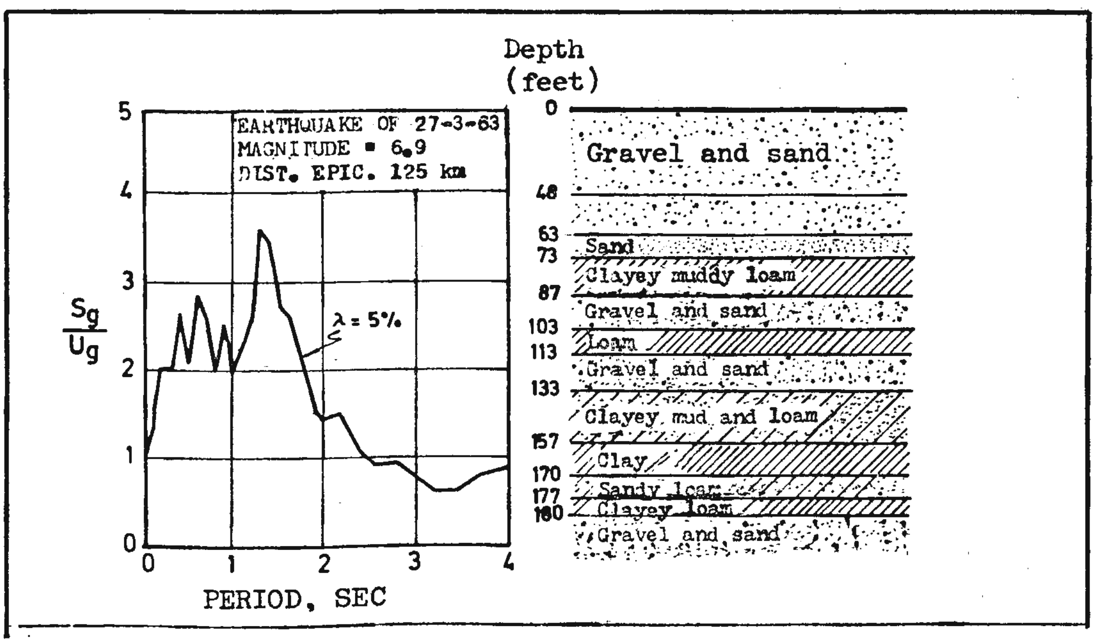


FIG. 37 (b)

COMPARISON OF RESPONSE SPECTRA FOR TWO SITES AT EQUAL DISTANCES FROM THE EPICENTRE BUT WITH DIFFERENT SOIL CHARACTERISTICS (HISADA, NAKIGAWA, IZUMI) properties of the soil, but they will be limited if the resistance of the soil to shearing stress is not altered by the seismic phenomenon. In some soils, however, the earthquake may result in a substantial reduction in resistance to shearing stress, making the resistance less than the static-load stresses existing in the soil. In such a case the deformations of the mass of the soil will be very large, and disaster will result. It is therefore convenient for purposes of analysis to separate the effects of the earthquake on the soil into two distinct effects, one being to produce deformations and the other to reduce soil resistance.

160. The problem of soil deformations during an earthquake will generally arise for loose unsaturated granular soils which may be compacted as a result of the earthquake. In analysing this problem it is important to investigate the geological age of the soil and the number of earthquakes to which it has been subjected, which might have already produced sufficient compaction.

161. Of greater importance is the problem of the loss of resistance which some soils undergo as a result of an earthquake. The soils most commonly affected are loose saturated sands and highly sensitive muds or clayey muds. When the phenomenon affects slopes consisting of such soils, the slope fails in a landslide, and the sand or mud begins to flow rapidly, covering great distances and destroying everything in its path. In 1965 the community of El Cobre, Chile, was destroyed as a result of the failure of a "tailings dam"("tranque de relave") consisting mainly of fine sands and mud in a very loose state. For the construction of dwellings, the type of soil making up any natural or artificial slopes which may exist near by must be carefully investigated. Wherever there are high mountains, there also exists the possibility of avalanches, in which large masses of rock and ice falling from great heights are transformed into a practically fluid mass moving great distances after the fall. A recent example is the avalanche which buried Yungay and Ranrahirca during the May 1970 earthquake in Peru.

162. At Niigata, Japan, in 1964 subsoil of the loose saturated sand underwent a considerable loss of resistance during an earthquake; as a result, many buildings were damaged, severely undermined and, in extreme cases, completely toppled. In addition, some underground storage tanks floated to the soil surface, indicating fluidity of the sand.

163. The phenomenon of loss of resistance in saturated granular soils is called "liquefaction"; it results from the transfer of intergranular forces to the water which fills the pores. The resistance of these soils depends on the pressure transmitted by the contact between the particles. Therefore, when part of the intergranular pressure is transferred to the water, the soil loses a corresponding portion of its resistance. When the pore pressure induced by the earthquake passes away, concentrated currents of water rising through crevices are generated and carry sand up to the soil surface, building it up into mounds resembling small volcanoes, which are characteristic signs of such phenomena.

D. Recommendations for the design of foundations

164. Seismic problems associated with soils (see B and C) are essentially independent of the type of foundation with which dwellings are designed, unlike problems which directly determine the design of the foundations; concerning the latter, specific recommendations for low-cost one-storey houses and general criteria applicable to multi-storey buildings will be given at this point.

165. For low-cost one-storey dwellings, the only type of foundation which can be built at a reasonable cost compared with that of the dwelling is a shallow foundation, either continuous or isolated. If the soil conditions necessitate deep foundations, serious consideration should be given to the possible choice of another building site.

166. The minimum depth of the shallow foundations must be determined by a number of conditions. In the first place, it must be such that the base of the foundation lies beneath the surface layer of soil, which undergoes cycles of humidity and dryness and variations in volume owing to seasonal changes. In addition, the depth must be greater than that of the vegetation layer, in which the soil is weakened by the presence of roots and decaying vegetable matter. These depths depend to a large extent on the local climatic and vegetation conditions and generally do not exceed one metre. In very cold regions the minimum depth of the foundation depends on the depth of the freezing process during the winter.

167. In some semi-arid regions there are soils called "expansive clays", which, as their name indicates, undergo considerable expansion when they come into contact with water. Seasonal fluctuations in the water table affect the soil to depths of several metres.

168. The loads which act on foundations are transmitted to the soil and produce deformations in it. This deformation increases with the load, and if the latter is sufficiently large, the deformations are so great that the result is regarded as a fracture of the soil. The load transmitted through the foundation to the soil is expressed in terms of the "contact pressure", the average pressure calculated as the total load divided by the area of the foundation.

169. The horizontal dimensions of the foundation must be such that the contact pressure is sufficiently low according to the following two criteria:

(a) The foundation must have a safety factor for soil fractures of at least 3; and

(b) The deformations of the foundation must be compatible with the structure and the use for which it is intended.

170. In general, a detailed study of the mechanical properties of the soil is required to determine the load which produces a fracture of the soil and to estimate the deformations.

171. It is recommended that continuous foundations under the walls of low-cost one-storey houses should have a width at least twice the width of the wall, and in any case not less than 35 cm. In such conditions, the contact pressures under the foundations are very low, probably of the order of 0.3 kg/cm2 or less. Under these loads only extremely soft soils will develop excessive deformations. Granular soils will always be sufficiently firm for these low loads, except for the previously mentioned problem of their stability during earthquakes, which in any case will not be affected by the magnitude of the contact pressure. In the case of fine cohesive soils, problems will arise only if the soils are highly organic and have a water table close to the surface.

VI. METHODS OF SEISMIC ANALYSIS

172. The purpose of seismic analysis is to determine stresses and deformations in order to dimension and design a structure capable of resisting the effects of seismic loads. The design is carried out with the aid of a design spectrum of the kind described in chapter II, which specifies the resistance the structure should have. This design spectrum is constructed in such a way that when combined with a predetermined level of allowable stresses, it can take into account the uncertainties characteristic of seismic loads.

173. The methods of analysis most often used in the design of earthquake-resistant buildings can be divided into two types, static and dynamic, depending on what considerations are taken into account in the design. These methods are used together with a series of simplifying hypotheses such as the following:

(a) The weight of the building is assumed to be concentrated at the level of each floor;

(b) The vertical component of the seismic motion is disregarded; and

(c) The horizontal components of the seismic motion are assumed to act independently in two directions at right angles to each other.

A. Static method

174. In this method the effect of the earthquake is considered to be a combination of lateral forces applied at each floor level. These forces are distributed in a way that does not depend on the vibrational properties of the structure and are all assumed to act in the same direction. This static method is used as follows:

1. Base shear force

175. The base shear force is one of the most significant parameters of earthquake-resistant design and is usually expressed as a fraction of the total weight of the building in terms of the seismic coefficient in the form described in chapter II. Thus, the seismic coefficient serves to reflect the complexity of earthquake behaviour and should depend on factors such as:

(a) Vibrational properties of the structure, characteristic period of damping, energy-absorption capacity;

(b) Characteristics of the foundation soil: consistency, compacting, homogeneity (softness or hardness), damping properties;

(c) Additional safety factor depending on the use or size of the structure; and

(d) Seismic-activity coefficient reflecting the probability of earthquakes at the site.

In general the seismic coefficient can be read directly from the design spectrum.

2. Distribution of lateral forces

176. The base shear force obtained from the design spectrum can be subdivided into lateral forces applied at each floor level with a distribution determined by assuming certain accelerations at the various levels. The most common hypothesis is that acceleration increases proportionately to height, which implies a distribution of lateral forces proportional to the weight concentrated at each floor and to its height above the ground. In the case of high-rise buildings greater importance is usually attached to accelerations at high floors in order to take account of the "whiplash effect" observed at those levels.

3. Distribution of the shear force at each level

177. The shear force for each level in a building is distributed among the resistant members at that level in accordance with their rigidity. In buildings whose resistant members are not distributed symetrically, the line of action of the shear force does not coincide with the line of the resultant of the reactions of the structural members; in such a case the building is said to have eccentricity in the horizontal plane, which gives rise to additional stresses as a result of the torsional moment. The torsional effect due to accidental asymmetrical distribution of masses and rigidities is normally added to this. The stresses due to this torsional moment are distributed among the resistant members in accordance with their rigidities.

178. It should be noted that this analysis implies the existence of a floor slab which acts as a rigid diaphragm and distributes the forces among the members.

B. Dynamic method

179. In the dynamic method the vibrational properties of the structure are considered more precisely than in the static method, not only taking into account their effect on the seismic coefficient but also taking the vibrational properties into account in the distribution of stresses in the structure according to the modes of vibration of each resistant member.

180. In the most usual formulation of the dynamic method of calculation the structure is assumed to have a reasonable number of degrees of freedom of deformation (usually equal to the number of floors), the elastic and inertial properties referred to those degrees of freedom are determined by using matrices of rigidities and masses, respectively, and the vibrational properties of the structure are assessed. The vibrational properties of the structure are stated in terms of the normal modes and frequencies of vibration, obtained by using any of the procedures developed for solving eigenvalue problems. This analysis makes it possible to subdivide the complex problem of the vibration of a structure into an independent analysis of each of its normal modes as if they were simple systems of the type described in chapter II.

1. Base shear force

181. The seismic coefficient for each mode is obtained from the design spectrum in such a way that the base shear force can be calculated separately for each mode of vibration. The base shear force for the structure is obtained by superposition of the contributions of each mode, assuming that the design shear forces for each mode correspond to the maximum values. The most usual ways of superposing the contributions of the various modes are the following:

(a) The sum of the absolute values of the base shear forces obtained for each mode; this superposition corresponds to a maximum-possible-shear hypothesis, and hence it gives values of the base shear force that are usually excessive for design purposes. This is considered a very conservative method of superposition;

(b) The square root of the sum of the squares of the base shear forces for each mode, which corresponds approximately to a probably-maximum-shear hypothesis; and

(c) A combination of the first two methods.

2. Stresses in members

182. The resistance of each member is specified on the basis of a superposition of the stresses found for that member in each mode of vibration; this superposition is done by any of the above-mentioned methods.

183. The effect of horizontal torsion can be dealt with independently of the dynamic calculation, in the same way as in the static method, which greatly simplifies the problem. In the case of very irregular buildings, it may be necessary to deal with the problem of horizontal torsion as a part of the dynamic analysis by adding more degrees of freedom; this implies a considerable increase in the amount of calculation and can normally be done only with the aid of a computer.

C. Scope and limitations

184. Of the above-mentioned procedures, the static method has the advantage that it is very simple and can be applied by most structural engineers. For this reason, it is recommended by almost all regulations for earthquake-resistant design.

185. Both methods, the static and the dynamic, establish the resistance of structures on the basis of a design spectrum which implies a certain degree of damping and a specific level of plasticity and, above all, takes into account the experience accumulated from the observation of building behaviour during earthquakes. That is why in designing structures that are very special by reason of their configuration, location or size, an engineer should determine whether or not conditions of the design spectrum are applicable; if they are not, he should carry out an analysis more detailed and more exhaustive than the simple application of methodologies based only on partial experience which cannot cover every special case.

D. Method of exact dynamic analysis

186. In earthquake-resistant design for very special structures, the analysis can be carried out by integrating the equations for the vibration to obtain the response of the structure as a function of time. This analysis can be done in such a way as to include the inelastic behaviour of structural members, although in practice this requires computers.

187. If this analysis is carried out for various seismic stresses typical of the site where the structure is to be built, an envelope of the expected maximum stresses in each member can be obtained. As stated above, these maximum stresses do not correspond directly to the design values but make it possible to estimate the level of stress to which the different members of the structure will be subjected, and thus to design the members properly.

VII. DESIGN RECOMMENDATIONS

A. Basic features of earthquake engineering

188. The purpose of earthquake-resistant structural design is to provide the structure with features which will enable it to respond satisfactorily to seismic loads. These features amy be related to five major objectives, which are listed below in order of importance:

(a) To ensure reasonable safety from collapse following a very severe earthquake;

(b) To keep material damage caused by moderate earthquakes within reasonable limits. Although it can be accepted that substantial damage may result from earthquakes with a low probability of occurrence, such damage is unacceptable in the case of moderate tremors, which are more likely to occur;

(c) For places where many people are usually present, to provide buildings with such deformability features as will enable occupants to remain calm even in the event of strong shocks;

(d) To avoid personal injury;

(e) To avoid damage to neighbouring buildings.

189. Such objectives cannot easily be attained simply by observing certain rules or recommendations. These can apply only to the situations which occur most frequently in practice, and only to their more general aspects, whereas each specific case always presents unique features which will require the designer to use his judgement in adapting those recommendations to the case in question, using all his knowledge and experience to arrive at the most suitable solution.

190. Furthermore, since earthquake-resistant building design involves a great many variables, the significance of which is sometimes difficult to measure, it is not possible to make specific recommendations, and we must be content to establish guidelines for the designer. Specifically, earthquake engineering involves the ductility of the structure, rigidity, damping and other structural factors whose influence on the structure's response to earthquakes is difficult to assess.

191. For these reasons, before formulating specific recommendations applicable to certain specific types of buildings, it is necessary to establish the general criteria which should be taken into account in order to attain each of the objectives mentioned earlier.

Safety from collapse

192. This would appear to be simply a question of resistance. However, in view of the fact that earthquakes occur in random fashion and of the complicated

nature of a building's response to earthquakes, it can be appreciated that the problem is much more complex. To ensure a building's safety from collapse, a dynamic analysis of the situation must be made.

193. Under static loads, safety from collapse can be ensured by the load-bearing capacity of the structure. However, under dynamic loads, such as those caused by an earthquake, safety from collapse must be ensured on the basis of the energy which must be imparted to the structure in order to make it fail. In such instances, consideration must be given to the structure's capacity to absorb energy rather than to its resistance. Accordingly, the primary factor to be considered in the earthquake-resistant design is the ductility of the structure.

194. Although there are as yet no clearly defined methods for determining the ductility of a structure, it is useful to clarify the concept, so that at least a relative appreciation of its importance can be attained and criteria can be established for the selection of a structural scheme. The greater the energy required to cause a structure to fail, the greater is its ductility.

195. Where the resistances of two structures are equal, the more brittle one will collapse under less strain; in other words, it collapses after absorbing less energy. This means that ductile structures have a better chance of resisting earthquakes of unexpectedly high intensity than brittle structures have.

196. The ductility of a structure depends on the type of material used and also on the structural characteristics of the assembly. Accordingly, structures which are built with ductile materials and have a rigid frame are preferable to others whose horizontal resistance is provided by shear-resistant walls; even worse are structures of the inverted-pendulum type (such as elevated tanks), in which case there is even less ductility.

197. Another factor closely related to safety from collapse is hyperstaticity of the structure. Apart from those cases in which, owing to the particular conditions involved, an isostatic (statically determinate) structure must be used (for example, where the possibility of differential settlement exists), it can be stated with certainty that, in general, hyperstatic (statically indeterminate) structures have advantages which make them much safer under seismic loads. Where such a load forces the structure into the range of plastic yield of the material, the hyperstaticity of the structure causes the formation of plastic hinges that can absorb considerable energy without depriving the structure of its stability, so that it becomes a mechanism. In other words, the plastic response of the material and the redundancy of hyperstatic structures protects such structures in various ways against unexpected stress of totally unpredictable intensity, such as occurs in the case of seismic stress.

198. To sum up, safety from collapse - the most important of the five objectives - must be sought not only by ensuring that the structure has sufficient static resistance but also, and in particular, by selecting materials and a structural scheme which will produce a ductile structure, preferably one that is hyperstatic.

199. However, this conclusion should not be interpreted as meaning that safety from collapse is what primarily determines which materials should be used. This is not so, because it is possible, with the most commonly used construction materials - steel, concrete and masonry - to obtain the desired ductility and hyperstaticity

simply by using a suitable design. A poor design, on the other hand, may result in a brittle structure which cannot withstand the effects of an intense earthquake.

200. The ductility of metal structures can be ensured by properly designing their joints, so that under stresses above the normal values, they will yield plastically, permitting rotation by causing the formation of plastic hinges, with the result that considerable energy is absorbed but the structure does not reach the breaking point. Another way to achieve the same result is to avoid elastic instability and, more particularly, plastic instability.

201. It is possible to build ductile structures with reinforced concrete if care is taken in the design to provide the joints with sufficient abutments that can adequately confine the concrete, thus permitting it to deform plastically without breaking. It is also important for this purpose to ensure that the tension edges of the structure are adequately reinforced and that there are sufficient stirrups to ensure that the concrete is properly confined along the compression edges. The use of only a few is not recommended for cases where a structure which will be subjected to intense earthquakes must be protected against collapse. Moreover, it is advisable to adopt a higher safety factor in the design of reinforced concrete subjected to stresses which cause brittle fracture, such as shear, where it has not been taken up with steel, and bonding, particularly where it serves as the basis for the splices of the principal reinforcements.

202. Masonry can also be used to build ductile structures if properly designed. Firstly, it is necessary to use bricks of the proper size and form. A parallelepipedal brick which is not too thick (i.e., one which will require many tiers per unit of height) responds well. The confinement of walls by reinforced-concrete or steel members (confined masonry) gives the assembly satisfactory ductility. The replacement of this confinement with vertical and horizontal reinforcements, resulting in "reinforced masonry", permits stresses which also can have good results, particularly when the masonry is carefully built by construction methods that ensure perfect linearity of the reinforcements.

Protection against material damage

203. It is known that every building has, in addition to its structure, other very important elements which permit the building to function in accordance with its purposes. These include partitions, windows, ducts, equipment, furniture and the like, which, although they do not play a part in the building's resistance, should remain intact during an earthquake.

204. The structural design should be such that these elements do not suffer damage under the action of moderate earthquakes. In order to establish what characteristics the design must have if it is to satisfy this requirement, the necessary relationships between these non-structural elements and the building's structure should be explained.

205. The non-structural elements generally have no stability of their own. They are usually supported by the structure and must therefore be able to move with the building without being destroyed. However, the way in which they are connected to the structure may interfere with the latter's functioning, causing structural members to become rigid or placing them under stress, a situation which can be serious in some cases, particularly when the connexions are installed without

detailed study or supervision. There have been many instances in which buildings were damaged because non-structural elements interfered with the structure itself.

206. This shows why it is necessary to have a design which takes into account the integrity of the non-structural elements when the structure is subjected to moderate earthquakes, in order to attain the following two objectives: to prevent the vibrations of the structure from breaking the non-structural elements and to prevent the latter from interfering uncontrollably with the functioning of the structure.

207. These difficulties can be overcome by making an adequate study of the anchorages and separation joints between the non-structural elements and the structure, in accordance with the quality and type of materials used, and by selecting the most suitable materials in relation to the elasticity of the structure.

208. It is a mistake to select brittle materials for partitions or other secondary components in elastic buildings and to use them without sufficient study, as is the case with ordinary masonry in rigid-frame structures without shearing walls.

209. Assuming that the material has been selected with due regard to the elasticity of the structure, it will be necessary to design the separation joints and anchorages with the dual purpose of ensuring the component's stability and at the same time avoiding any appreciable interference with the functioning of the structure. The separation joint must be of adequate size and the placement of anchorages or guides must be carefully studied.

210. It is useful at this point to describe the solution commonly adopted when affixing masonry partitions - whether structural or not - in buildings which are low, and therefore rigid, as in the case of dwellings. The solution consists in simply anchoring the masonry in the structural members of reinforced concrete which surround it (beams and columns). This solution is satisfactory if an effective anchorage can be made by using specially placed reinforcements or some other similar arrangement. It is unsuitable for elastic buildings, in which these fragile partitions should remain floating and should be supported by the structure without interfering with it. The same holds for furniture and large windows. The latter should be kept floating through the use of carefully-designed devices, not only to avoid destruction of the windows but also to prevent the possible consequences of their destruction.

Restriction of deformability to prevent panic

211. It is unquestionably true that flexible buildings under seismic stress react in a manner which has a very unfavourable psychological impact on the occupants. Rigid buildings, on the other hand, do not present this problem.

212. When one is faced with the alternative of using either a rigid or a flexible structure, the decision depends on a number of factors, one of which is the comfort of the occupants. However, except in the case of buildings which will house large numbers of people, this decision generally is determined by other reasons of an economic and technical nature. Depending on the local conditions (geology of the site and foundation terrain), technical or economic factors may make either

rigid construction or flexible construction more advisable. On rocky ground, a flexible structure is generally preferable, whereas a rigid structure is more desirable on deformable ground with a low load-bearing capacity.

213. Greater rigidity generally results in higher cost; this is partly compensated by the lower cost of anchorages for non-structural elements, which are simpler in such cases. Rigid structures are also more comfortable for the occupants, and movable objects suffer less damage in such structures when earthquakes occur.

214. On the other hand, flexibility means lower cost, owing to the lower seismic coefficients, however, this cost is increased by the higher cost of more complicated anchoring systems for partitions or other secondary components. Moreover, the damage caused as a result of panic and the destruction of movable property may be substantial.

215. As can be seen, many factors are involved here and it is not easy to establish well-defined criteria. The designer, taking into account the characteristics of each case, must make a decision by weighing on the one hand the technical and economic factors and on the other hand the comfort of the occupants.

Protection against personal injury

216. Experience shows the importance of avoiding injury to persons inside and outside the building from falling accessories or ornamental elements (cornices, facing and the like). No earthquake-resistant design is satisfactory unless it has also taken into account these components, which, although apparently secondary, have been responsible for substantial loss of life in severe earthquakes. Here again, reference must be made to anchorages and the attachment of structural members, both in the interior and on building façades, as discussed earlier. During intense earthquakes, façade coverings and the glass of large windows frequently come loose when insufficient attention has been paid to affixing these components.

217. Even if a satisfactory solution to these problems is provided, it is also recommended that in tall buildings, in which such damage tends to occur, a sufficiently wide marquee slanting towards the interior should be installed, affording satisfactory protection to passers-by.

Protection against damage to neighbouring buildings

218. The main cause of this type of damage is collision between neighbouring buildings. To ensure protection against this type of damage, it is necessary to determine accurately how much space should be left between buildings. This separation should be determined on the basis of the deformations which both structures will undergo; the values will depend on the type of structure selected.

219. In the case of elastic buildings with rigid frames, if properly designed, the deformations may be three or four times those for the elastic limit of their materials. In such instances, therefore, the separation must be calculated on the basis of the sum of the elastic displacements multiplied by four, plus such deformations as may result from the possible rotation or displacement of the foundations.

220. Accordingly, very tall elastic buildings should preferably be completely isolated from all other structures of similar height; if proximity is unavoidable, the calculated separation distance should be observed, bearing in mind each of the different possible types of displacement which might occur.

221. It is often necessary to build low buildings at short distances from one another. In such cases, where the deformations are smaller, it is possible to reduce the calculated spacing value even further by introducing shock-absorbing components in the joint (or separation) which will reduce this type of damage to a minimum. The use of soft wood or other products easily obtained on the market has yielded good results.

222. These basic conditions which earthquake engineering must satisfy in order to meet the five objectives described above should be pursued from the standpoint of the three factors which must be taken into account in any design: the architectural design, the structural design and the materials used.

223. It should be emphasized that these are not three independent factors. In modern construction, these three aspects of building design are inseparable. Studies of buildings damaged by severe earthquakes, where the damage was attributable to defects in the project, frequently reveal that these defects in structures resulted from poor architectural design or improper choice of materials. In other cases, an excellent reinforced-concrete project became a defective project when the structure was built in steel.

224. The architectural scheme will depend on the materials selected, and there will be an ideal structural scheme for that combination. Conversely, a structural form imposed by local conditions and by the materials to be used will suggest the architectural scheme most suited to the programme's requirements.

225. It can be concluded from the foregoing that the design of a building is an interdisciplinary task in which the architect and the civil engineer must co-operate very closely in order to arrive at the most suitable architectural and structural scheme. They must constantly bear in mind the requirements of the programme, the local ecological conditions and the materials available, without losing sight of the five basic objectives of earthquake engineering.

B. Structural configuration and detail

226. As stated in the preceding paragraph, the structural configuration should always be considered from the three points of view of architecture, structure and construction (construction materials and techniques).

227. However, the present analysis of structural configuration and detail will place greater emphasis on the first two aspects, leaving aside for the moment any detailed consideration of the effect of construction materials and techniques on the selection of a structural scheme.

228. In general, one may be sure that a building has good resistance to earthquakes if it has: (a) a vertical structure capable of withstanding horizontal forces; (b) rigid horizontal diaphragms at each floor which will transmit the forces

resulting from seismic shocks to the vertical structure (where there are no rigid horizontal diaphragms, this fact must be taken into account in the structural design); and (c) foundations specially designed to receive and transmit the shear forces generated between the foundation soil and the building.

1. The structure

229. Uniformity and structural symmetry. A basic condition which must be met by the structural configuration of an earthquake-resistant building is that of structural symmetry and uniformity. The architectural and structural scheme of the building must be studied so that the execution of the programme conforms as closely as possible to this essential requirement.

230. The greater the height of the building, the more rigorously this requirement must be observed; in buildings more than 20 stories high it becomes an absolute necessity. Although this requirement is not related to the architectural scheme as such, the latter must provide for as uniform and symmetrical a structural configuration as possible.

231. The objective of structural symmetry is to reduce the static torsion of the building, which has been the cause of major failures in many cases. It should also be pointed out that static eccentricity alone does not determine the effect of the horizontal torsion, since this is augmented by the effect of dynamic torsion, which is generally a good deal greater than static torsion and is influenced to some extent by the value of the latter. Although this phenomenon is not yet fully understood, it is unquestionably advisable, in any event, to reduce static eccentricity as much as possible.

232. Torsional rigidity. The dynamic amplification of torsion mentioned in the preceding paragraph, represents a major threat to buildings and must be reduced as much as possible by selecting a structural configuration which possesses the required torsional rigidity. Accordingly, the members which are to withstand the horizontal shocks not only must have a certain symmetry and possess the necessary resistance but also must be placed on an appropriate perimeter and so arranged relative to one another that high torsional rigidity can be achieved. Figure 38 clarifies the concepts of symmetry and torsional rigidity by illustrating four simple cases.

233. Structural symmetry should not be understood in absolute terms. In the case of low-rise buildings it is sufficient to produce a design with the centres of mass and rigidity as close to each other as possible, and this can generally be achieved by balancing the rigidities of the horizontal resistant members and selecting appropriate dimensions, structural type or materials for them (see fig. 39).

234. This freedom of procedure means that in some cases there is no well-defined solution, particularly when members of different structural types, e.g., portals and walls, have to be balanced; matters are even worse when members made of different materials are involved (concrete walls and masonry walls; concrete portals or walls and steel portals).

235. It is recommended that these structural combinations should be avoided, in so far as possible, and when their use is unavoidable, the most unfavourable extreme situations should be considered.

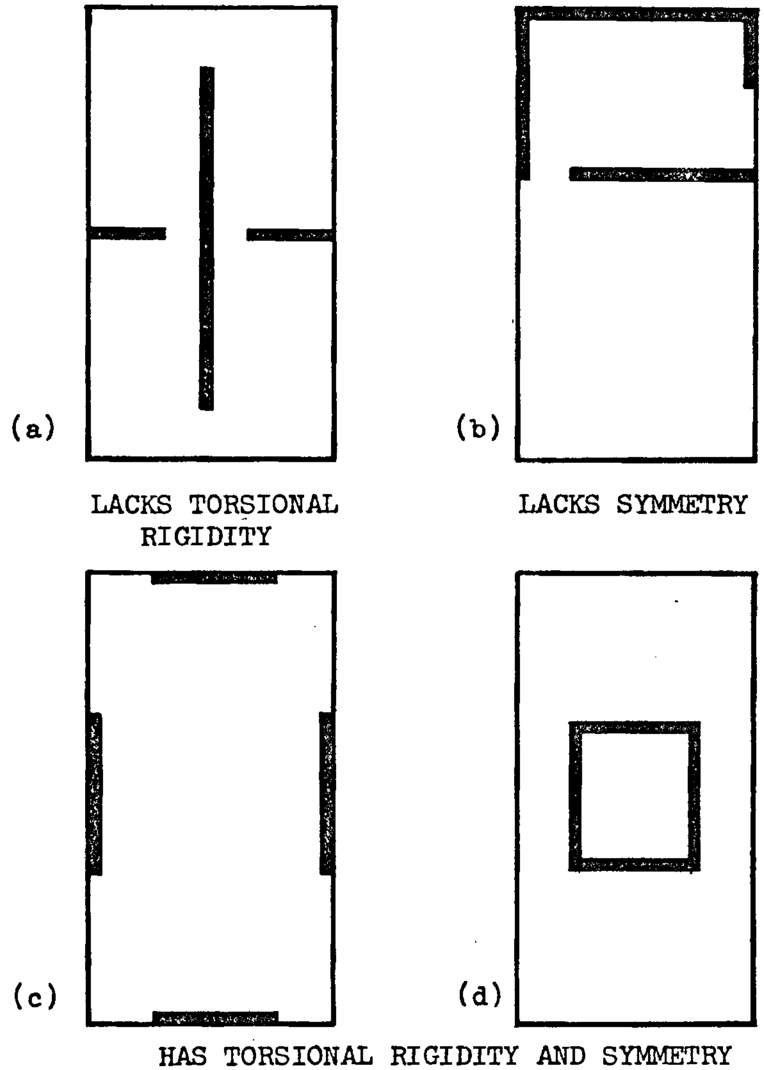
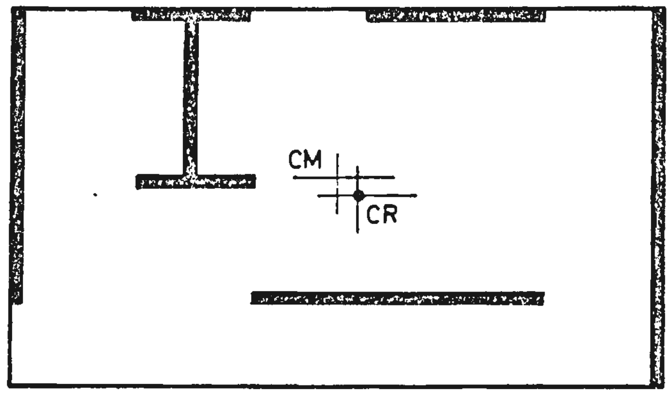
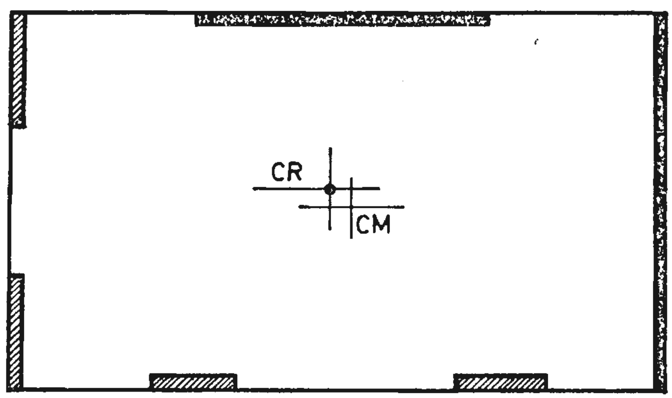


FIG. 38. TORSIONAL RIGIDITY AND SYMMETRY

The distribution of the walls lacks torsional rigidity in plan diagram (a) and lacks symmetry in (b); both defects have frequently caused serious damage and even destruction of buildings under the impact of an earthquake. Plan diagrams (c) and (d) show a satisfactory distribution of walls which provides both symmetry and torsional rigidity.



(a) ACCEPTABLE NON-SYMMETRICAL DISTRIBUTION
(Members made of the same material)



(b) ACCEPTABLE NON-SYMMETRICAL DISTRIBUTION
(Members made of different materials)

FIG. 39. SYMMETRICAL DISTRIBUTION IN GROUND PLANS

The two plan views show a non-symmetrical geometrical distribution of the earthquake-resistant members in the horizontal plane, but the placement of the centres of rigidity (CR) in relation to the centres of mass (CM) is acceptable. In case (a) this was achieved through proper distribution of members made of the same material, namely, masonry walls. In case (b) the situation is similar, the same acceptable result being achieved through the use of members made of different materials and having different rigidities such as walls of masonry and reinforced concrete.

236. Structural scheme. Given the local conditions, such as the programme and the architectural scheme, the characteristics of the building's ground plan and elevation, and the geological, soil-mechanics and materials conditions, the designer should proceed to select the structural scheme, bearing in mind the need for symmetry and torsional rigidity. The structural scheme is selected by determining the proper placement for the various earthquake-resistant members and choosing appropriate structural types.

237. Since many factors enter into this process, it is difficult to make specific recommendations, the more so since conditions vary from one case to another. In choosing between elastic and rigid structure, it is essential to take local conditions and user needs into account. In many cases these factors are sufficient to determine the material selected, and that in turn will affect the decision as to the most appropriate structural type - portals, walls or braces - for the horizontal earthquake-resistant members. For example, from the standpoint of conditions at the foundation site, a rockly terrain may justify the use of portals; sandy soil may indicate the use of reinforced-concrete walls or masonry walls in concrete structures or the use of bracing in steel-frame structures.

238. It is necessary to take the architectural scheme into consideration in order to ensure proper placement of the earthquake-resistant members and design the rest of the structure while endeavouring at the same time to make the configuration simple and regular.

239. For example, the use of continuous beams of widely differing lengths gives rise to a concentration of shear forces and moments in certain zones which may reduce the capacity of the structure to resist earthquakes. The higher the building, the more important this problem becomes. The recommendation of structural regularity is less applicable to low buildings.

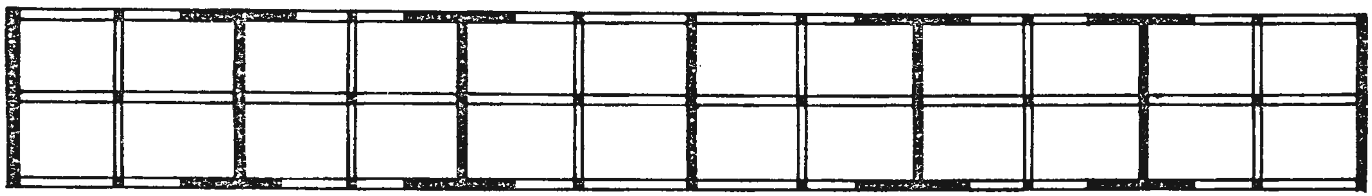
240. Consideration of the architectural scheme also makes it possible to study materials and the mechanics of the attachment of non-structural elements in relation to the characteristics of the structural design selected.

241. Taking into account the general characteristics of the ground plan and elevation is important in determining whether the project should be treated as one structural unit or broken down into a number of units. The following paragraphs analyse certain cases which occur frequently.

242. Buildings with a very elongated ground plan. It is often desirable, for reasons of dynamic structural design and temperature, to divide the ground plan into independent sections, particularly where the horizontal resisting members are quite far apart in the transverse direction. The same solution should be adopted where the transverse members are placed asymmetrically in the assembly.

243. It may be appropriate, depending on the circumstances, to provide two resisting members instead of one at the separation joint. The separation at the joint will be determined from a calculation of the elastic and plastic deformations, as indicated at the beginning of this chapter (Figs. 40 and 41).

CASE I SYMMETRICAL DISTRIBUTION OF EARTHQUAKE-RESISTANT MEMBERS



SOLUTION FOR CASE I

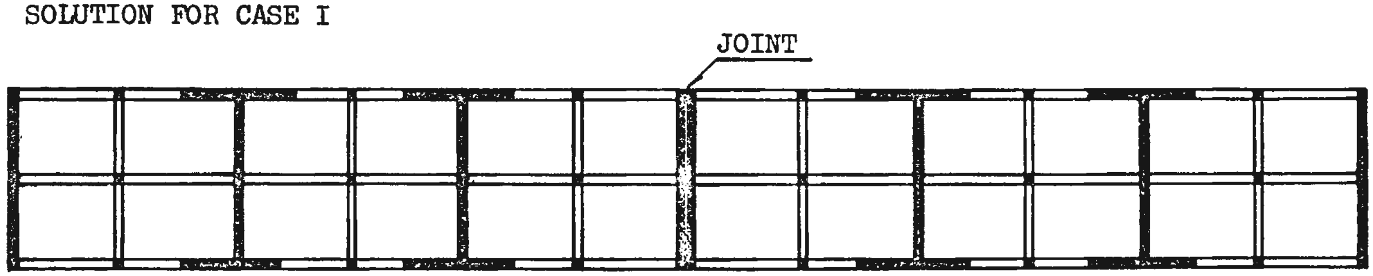
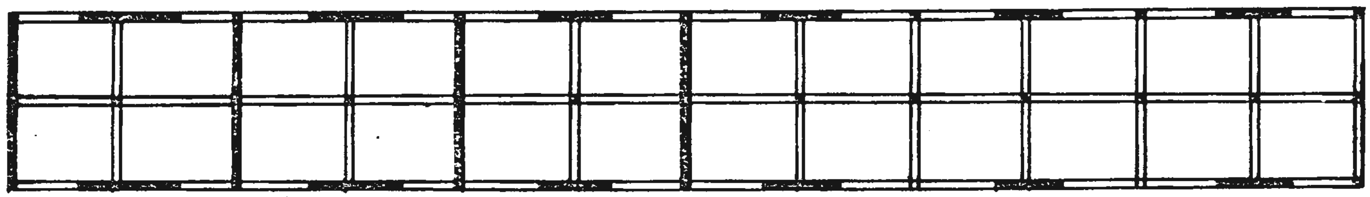


FIG. 40. SYMMETRICAL DISTRIBUTION OF EARTHQUAKE-RESISTANT MEMBERS

The diagram shows a structure with symmetrical distribution of the earthquake-resistant members. When the structure is split into two separate blocks, it is advisable to provide for a second earthquake-resistant member in the middle in order to maintain symmetry.

CASE II ASYMMETRICAL DISTRIBUTION OF EARTHQUAKE-RESISTANT MEMBERS



SOLUTION FOR CASE II

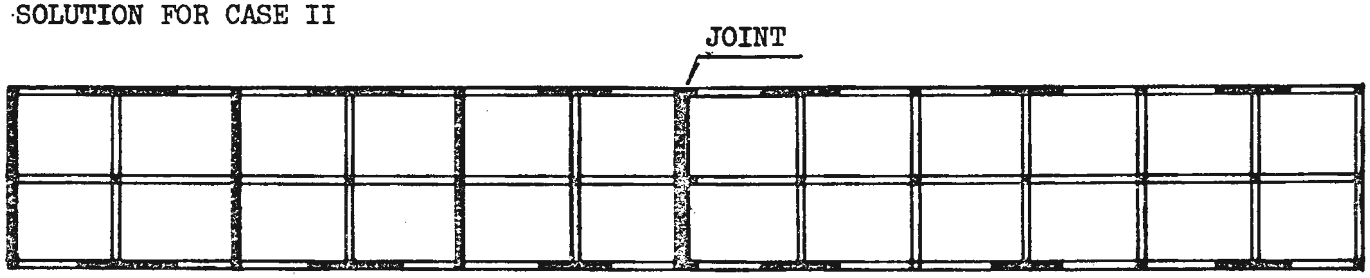


FIG. 41. ASYMMETRICAL DISTRIBUTION OF EARTHQUAKE-RESISTANT MEMBERS

This diagram shows a ground plan with asymmetrical distribution of the structural members in the horizontal plane. In determining the placement of a joint to divide the ground plan into two smaller ones, it was specified that the structural members on the two sides of the joint should be different, so that each section would have a symmetrical distribution of its earthquake-resistant members.

244. Buildings whose parts are of substantially different heights. In this case it is recommended that two parts of a building which differ substantially in height should be structurally independent because of the great differences in rigidity between the two parts. The separation joints which must be incorporated into the design should make allowance for plastic and elastic deformations calculated separately for each of the parts (see Fig. 42).

245. Buildings with ground plans of irregular shape. Many buildings have ground plans made up of sections with different orientations, so that the ground-plan shape resembles an L, a T, a U, etc. In such cases it is advisable to design an independent structure for each section, since each will have a different rigidity for a particular earthquake direction. In buildings with small ground plans it may be preferable to maintain structural unity but make provision in the design for the effects of the irregular shape (Fig. 43).

2. Rigid diaphragms

246. It is essential to have a rigid diaphragm at each floor in any earthquake-resistant building; its omission can be considered acceptable only in the case of one- or two-storey constructions. Its function, apart from the static one, is to distribute the force of the shock throughout the storey among the various earthquake-resistant members by virtue of its high rigidity in the horizontal plane.

247. In designing the diaphragm the designer must never lose sight of this fundamental function and must make sure that the diaphragm's rigidity in the horizontal plane is indeed high. Where this is not possible, the diaphragm's flexibility will have to be taken into account in determining the seismic-force distribution in the horizontal plane. The cases which occur most frequently in practice in this connexion are mentioned below.

248. Regular ground plans. Regular ground plans do not present any special problems, and ordinary reinforced-concrete floor structures provide the best solution for these cases. In the case of buildings with steel frames and in cases where the use of reinforced-concrete floor structures is not suitable, horizontal triangulation of the floor structures may be the best solution.

249. Irregular ground plans. Where buildings with an irregular ground plan are designed as a single unit, care must be taken with the design of the floor-structure diaphragms, which will be affected by large concentrations of stresses at the re-entrant corners of the ground plan when subjected to seismic shocks. Figure 44 illustrates the areas in which stresses are concentrated in different types of structures. In the case of reinforced-concrete slabs the inclusion of diagonal reinforcements, as shown in Figure 44, is recommended.

250. Hatchways. The presence of hatchways in the diaphragm produces a concentration of stresses of the same kind that occurs in buildings with irregular ground plans. Where they are small in size, as is usually the case, no special provision need be made for them; it is sufficient simply to provide perimetral reinforcement. However, where they occupy a substantial part of the ground plan, they narrow the floor structure, which impairs its functioning as a rigid diaphragm, so that it is frequently advisable to divide the building into independent sections on each side of the narrowed portion (Fig. 45).

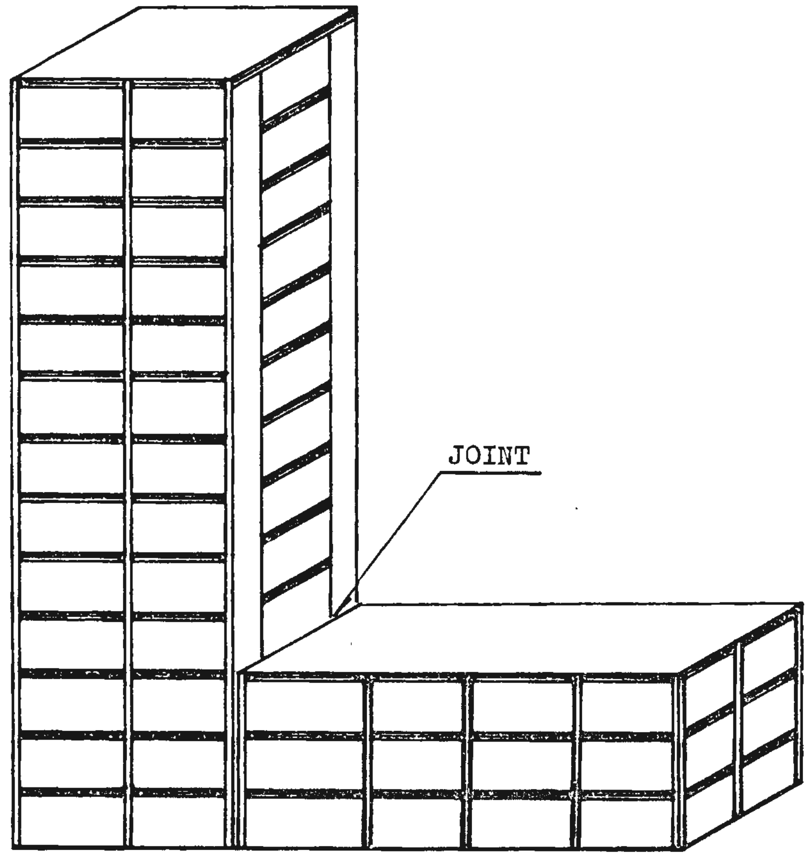


FIG. 42. SEPARATION JOINT BETWEEN PARTS OF A BUILDING WHICH HAVE DIFFERENT HEIGHTS

The illustration shows a building consisting of two parts of substantially different heights. In order to avoid problems arising out of the differences in rigidity, **a separation** joint has been specified, so that each part can function as a structurally independent unit.

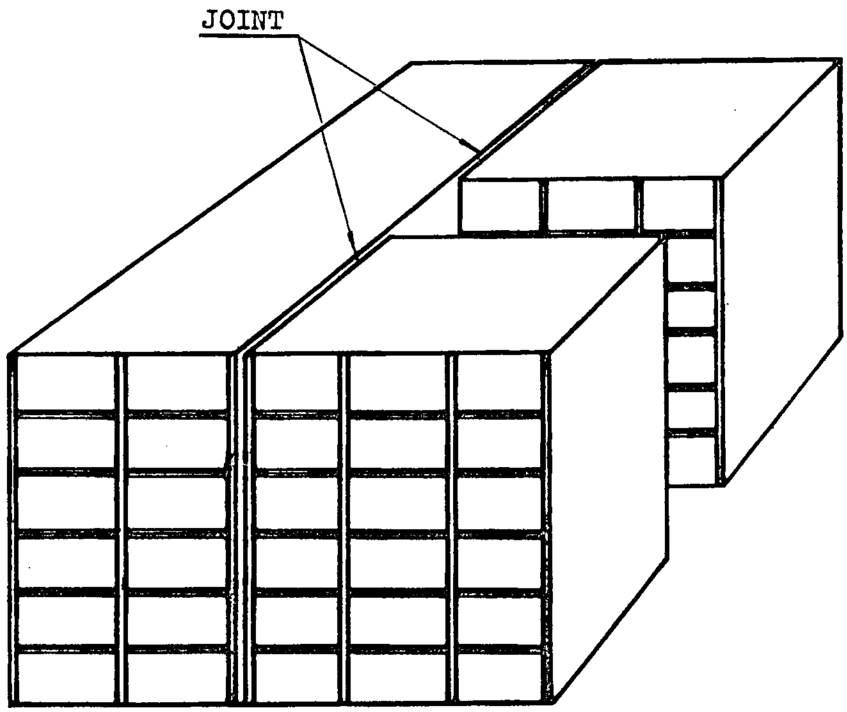


FIG. 43. JOINTS TO ALLOW FOR THE SHAPE OF A BUILDING'S GROUND PLAN

In this U-shaped building the problem of its shape has been eliminated by using joints to make each of the wings structurally independent.

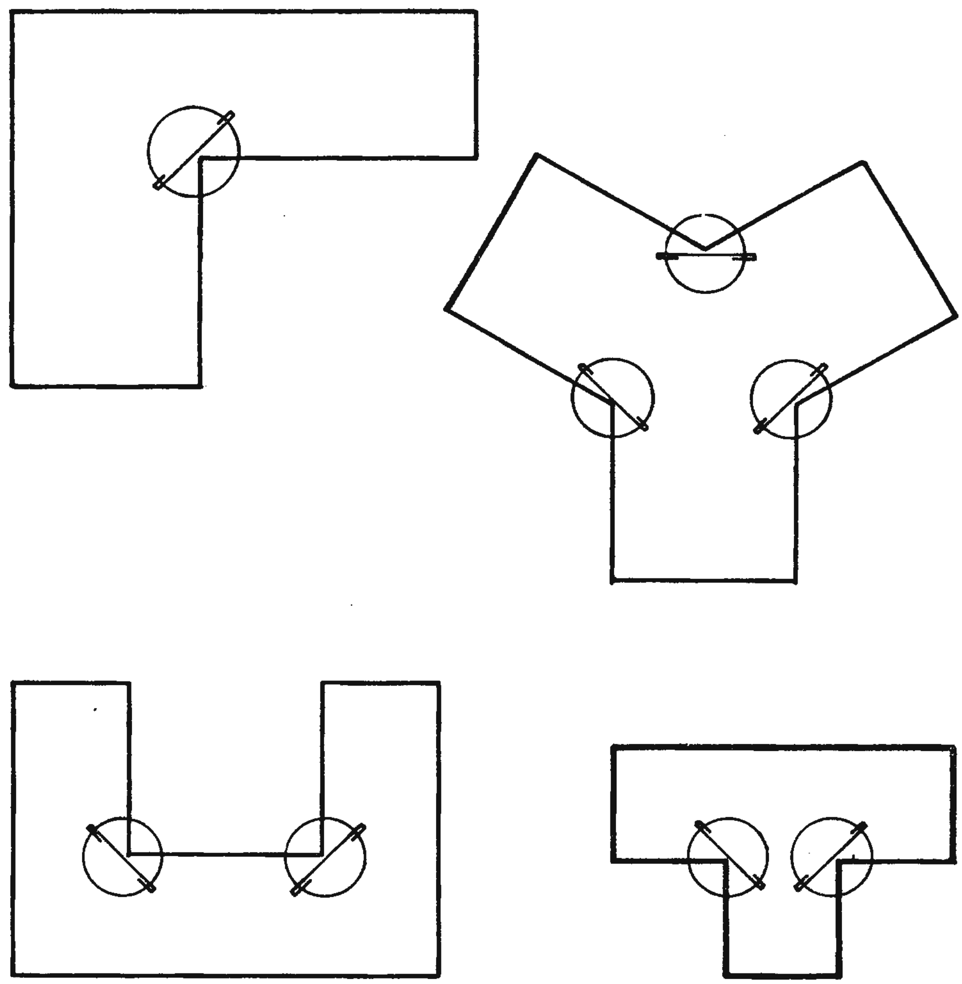


FIG. 44. REINFORCEMENTS IN IRREGULAR GROUND PLANS

In irregular ground plans in which it is not possible to make the sections independent by using joints, it should be borne in mind when designing floor diaphragms that stresses will be concentrated at the re-entrant corners and must be carried by diagonal reinforcements, as shown in the illustration.

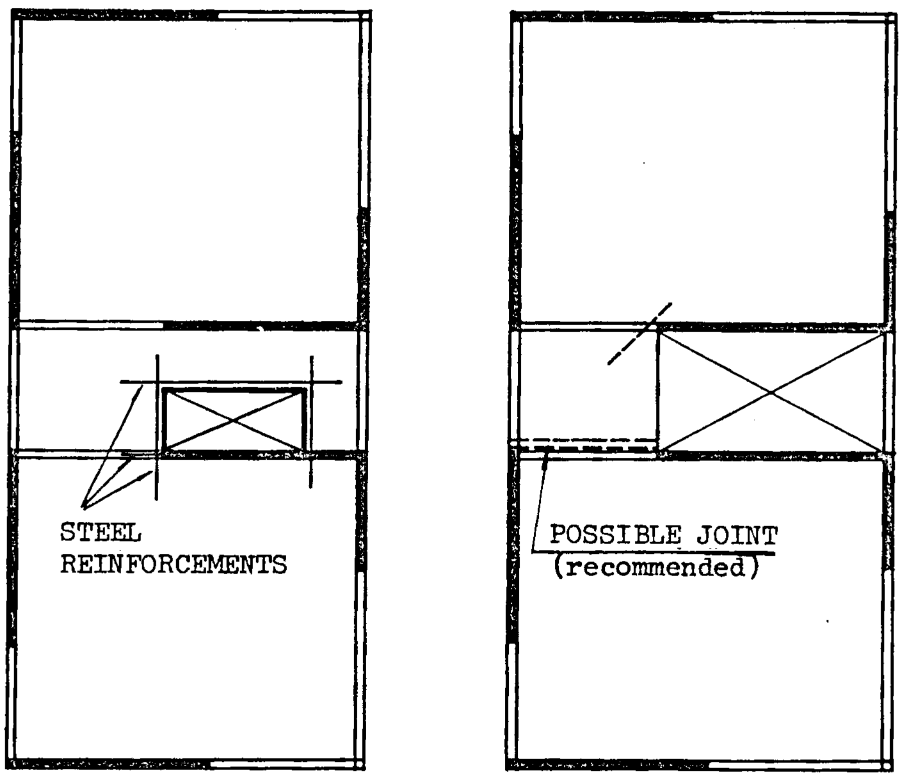


FIG. 45. REINFORCEMENTS AND JOINTS AT HATCHWAYS

The presence of hatchways of a certain size in a diaphragm gives rise to stress concentrations which necessitate the use of additional reinforcements or the inclusion of joints to make the sections on the two sides of the narrowed part of the floor independent.

3. Foundations

251. During an earthquake the foundation of a building is subjected to sharply varying stresses, frequently of an alternating nature, in some of its structural members. Accordingly, the foundations of earthquake-resistant buildings must be carefully designed, taking into account certain recommendations relating to these features of their functioning.

252. In view of the great importance of foundations in the structural response of a building subjected to earthquake stresses, the following major recommendations on structural design must be borne in mind:

(a) Foundations should preferably be designed as continuous in order to avoid relative horizontal displacements;

(b) Where circumstances require the use of separate foundations, they should be joined to each other and to the rest of the structure by means of foundation beams. Such foundation beams may or may not be designed to withstand seismic moments. Where simple ties are used, they should be designed to take compression and tension forces of the order of one tenth of the maximum vertical load supported by the uprights of the footings being joined;

(c) It is recommended that parts of building foundations which rest on soils of different types or are sunk to different depths should be designed as separate units. In such cases there should also be structural independence in the superstructure;

(d) It is recommended that if different parts of a building are to be structurally independent because of the shape of their ground plan, their foundations should also be independent;

(e) In specifying the quality of concrete, it should be borne in mind that earthquakes produce alternating stresses in foundations and it is therefore of greater importance that the concrete used in foundations should be capable of withstanding tension and shear;

(f) The design of water pipes and drains requires special care in the case of buildings in earthquake zones. Improper design of such installations can cause breakage of pipes during an earthquake, so that water escapes to filter down into the foundation soil, where it may change the soil structure and partially alter its bearing capacity. Proper design of such installations and correct placement of pipes in relation to foundations will prevent these problems;

(g) For the reasons given in the preceding subparagraph, it is important in designing the foundations of a building to know the ground-water level and its usual variations. The ground-water level should be at least one metre below the foundation level. If this requirement is not met under existing conditions, a drain may be installed to lower the ground-water level to the required depth.

C. Effect of materials and construction techniques on structural design

253. It is difficult to draw up design recommendations which will cover in general all the materials most commonly used in construction work.

254. At the same time, each material is covered by specific building regulations, which are not always sound from the viewpoint of earthquake resistance. Much of the damage caused by earthquakes is due to inadequate building techniques or regulations.

1. Design recommendations for structures of reinforced concrete and confined masonry

255. The reasons given above justify treating each material individually from the point of view of design on the one hand and that of construction on the other.

(a) In view of the monolithic and generally rigid nature of a structure, it is essential to link the various foundation units by designing them, in so far as possible, to be continuous, particularly on sites which have poor soil consistency and are therefore subject to possible differential settling;

(b) For economic reasons, it is recommended that good interlocking should be ensured between masonry walls and reinforced-concrete confining members, so as to transmit to the walls all or part of the earthquake force. Where it is not desirable for the masonry walls to provide earthquake resistance, they should be made completely independent of the concrete structure by means of appropriate anchorages. This approach is usually more costly and will be used only where the chosen structural scheme so requires;

(c) In so far as possible, floors should be constructed as rigid diaphragms of reinforced concrete which distribute the force of the earthquake among the earthquake-resistant members in accordance with their relative rigidities. Where pre-fabricated members are used, it is necessary to study the distribution of the shear force in the horizontal plane;

(d) Care should be taken in the design and distribution of the reinforcements in reinforced-concrete members to ensure adequate ductility of the members. At joints between beams and columns the concrete should be confined by stirrups and lacings in order to ensure the ductility of the structure; and

(e) It is recommended that designs should be based on shearing walls, avoiding eccentricities and discontinuities in height which might cause sharp changes in rigidity.

2. Design recommendations for steel-frame structures

256. In this type of structure the horizontal forces are usually carried by diagonal braces.

(a) The use of braces which are resistant only to tension is not recommended except in very light-weight structures. The use of diagonal cross-bracing which resists only tension has yielded very poor results when the structures were subjected to earthquake shocks. Diagonals should preferably be designed to withstand compression;

(b) It is recommended that the tolerance for structural deformations should be specified in the design, in order to prevent the destruction of movable or secondary elements;

(c) Proper care should be taken in selecting materials to be used in the construction of filler elements, so that they are matched to the elastic properties of the structure, and the anchorage elements holding the filler to the structure must be carefully designed;

(d) Anchorages must be carefully designed to provide ductile performance, if possible. For example, the anchorage bolts must have sufficient free length to take advantage of their ductile behaviour.

3. Design recommendations for timber structures

257. Horizontal forces are usually borne by bracing or by diaphragms formed by plates or diagonal boards.

(a) It is recommended that a rational distribution of the bracing which is intended to bear the earthquake forces should be designed. The use of diagonal lining boards constitutes an acceptable alternative to bracing. The diaphragm thus created will effectively withstand earthquake stresses;

(b) Heavy fillers should not be put in the partitions, ncr on the roofs. The use of light-weight roofing is recommended;

(c) The design should make provision for protecting the wood from rotting, particularly in areas near the foundation, where it is most exposed to moisture;

(d) The design of the foundation anchorage of timber members should not be overlooked; and

(e) The design should take account of the fact that the structure is highly deformable, owing to the low rigidity at the knots. For this reason it is recommended that foundations should be rigid.

4. Design recommendations for adobe structures

258. Although adobe is not a recommended material for housing construction in earthquake regions, adobe walls may be capable of withstanding earthquake forces if the recommendations listed below are followed:

(a) Buildings higher than one storey should not be planned; the total wall height should not exceed 2.40 metres;

(b) Walls should run at right angles to each other. In view of the characteristics of the material, only right-angle placement of walls provides the required safety;

(c) Resisting walls should be supported on wall foundations extending above ground level in order to protect the adobe from the moisture in the adjacent soil;

(d) The structure should have a light-weight roof;

(e) Since the walls of a dwelling which run in one direction will serve as buttresses for those running at right angles to them, no section wall should exceed 4 metres. In the case of longer sections, it will be necessary to provide intermediate buttressing. Such buttresses should project at least 0.9 metres;

(f) The adobe bricks used should be of the necessary size to provide solid interlocking at crossings and corners; and

(g) Provision should be made for a tie-beam at the top of walls to ensure proper bonding of the last tiers of bricks, since such bonding is generally not very effective unless the wall is under a load.

5. Superposition of structures made of different materials

259. The superposition of light timber structures over masonry or concrete (two-storey houses) does not cause any problems, provided that the design is carefully thought out and special care is taken with anchorages. Such superposition over adobe structures should be avoided.

VIII. RECOMMENDATIONS FOR THE EXECUTION OF BUILDING PROJECTS

260. As shown in the previous chapter, where a proper earthquake-resistant design has been used, much of the earthquake damage suffered by buildings can be attributed to unsuitable construction techniques or practices, which reduce the resistance of the structure. Conversely, on examining structures which have withstood major earthquakes, it is often found that such structures were not designed with earthquake effects in mind but that although their structural design was normal, they stood up well to those effects mainly because they were built with good materials and excellent techniques, which considerably improved their capacity to resist earthquakes.

261. For that reason, recommendations for the execution of building projects are given in this chapter in order to emphasize some aspects which sometimes seem to be of little importance but which have in many cases meant the difference between safety and collapse in structures which were based on a proper earthquake-resistant design. The aspects of building which affect the earthquake resistance of a structure are: (a) materials; (b) labour; (c) tools, instruments and equipment; (d) building procedures or methods; and (e) site conditions. These aspects can be analysed with reference to various parts of a construction and to different types of buildings.

262. The standards of quality for building materials are not uniform and in some cases deviate from the norm by as much as 20 per cent of the arithmetic mean value, particularly when they are prepared at the site. There are frequently variations in (a) the grading and cleanness of the dry substances; (b) the proportions of water and cement in concrete; (c) the dimensions and resistance of clay bricks; (d) the proportioning and mixing of materials for adobe bricks; and (e) the resistance of cement components (bricks, blocks etc.) which have not yet been fully cured. These problems are more acute in the developing countries because of a lack of proper control and insufficient mechanization in the building-materials industry and in the erection of structures.

263. Labour affects the result through the workers' skill and knowledge. The builder's degree of confidence in the competence of the work force must be taken into account when drawing up work instructions. It is to be expected that less-skilled workers will not be "earthquake-conscious" at the outset. Awareness of the earthquake problem is acquired intellectually, since it is a complex process and any individual's living experience of it is necessarily very short. For this reason, instructions must be very carefully given in countries in which earthquakes are likely to occur. Supervisory groups must always take care to clarify the aims of the earthquake-resistance provisions in the building specifications and make sure that they are complied with. This background must be clearly passed on to the construction workers, so that they will reproduce the designer's idea exactly, without leaving any aspects open to free interpretation in the building process.

264. Other factors, such as the proper choice of mechanical equipment and tools, the determination of the construction method used and a knowledge of site conditions, are important parts of the function of construction management.

265. In the analysis below on building materials and recommendations on the execution of building projects, priority is given to problems relating to housing units of one or two storeys, since this is the category that generally receives inadequate attention and inspection, particularly when built in remote areas or when such housing forms part of small projects.

A. Building materials

266. As stated earlier, one of the basic factors for a housing unit to be earthquake-resistant is that the materials used should satisfy minimum standards of quality and resistance. Such quality is guaranteed by the manufacturer in the case of some industrially produced materials, such as cement, steel and bricks, provided that they are used according to certain established rules. For materials made at the site, such as adobe or concrete, it is not enough to have components of good quality; the way in which they are mixed and worked can be even more decisive for the final result. That is why this chapter will discuss such materials at some length and will give minimum instructions concerning their preparation and use in order to avoid faults which can cause a house of earthquake-resistant design to lose that characteristic when it is built.

1. Materials with a soil base

267. Soil is a cheap building material because it is easily obtained and used, and it can have good insulation and resistance characteristics for some structures when properly used. Soils are generally classified according to grain size as coarse or fine. Coarse soils (gravel, grit, and coarse, medium and fine sand) lack cohesion and have considerable internal friction and no plasticity.

268. Fine soils (clay and silt) on the other hand, are cohesive and have little or no internal friction. The essential difference between clays and silt is that silt has little or no plasticity and is generaly an inert material. In contrast, clay is a highly active material in the presence of water and is subject to variations in volume with moisture content. It has plasticity and serves as a binder for the other soil particles in materials such as adobe and rammed soil. In nature, the components of soils are not generally found in isolation but are always mixed together in various proportions.

269. The main defect of soil as a building material is that its resistance is considerably affected by changes in moisture content. The influence of water is decisive for its resistance behaviour. Consequently, in order to avoid the action of water, various methods have had to be used in order to make soil suitable as a building material for walls. Some of the processes are as follows:

(a) Walls are protected by high wall foundations, waterproof stuccos, eaves, etc., which prevent rain from wetting them directly;

(b) Soil material is stabilized with salts, cement, bitumen, resins, etc., which make it independent of the action of water and generally improve its resistance and insulation properties;

(c) The material is stabilized by physical and chemical means. The most common is heat. When soil is heated to an appropriate temperature, its properties change and it becomes a hard, inert and stable mass: this is the basic material for making bricks and tiles of baked clay.

270. The following paragraphs describe some of the building features of adobe, or sun-dried brick, which is widely used for rural housing in vast areas of the world.

271. Adobe is an unburned-soil brick prepared in wooden moulds, with a base of soil and water (mud), to which straw is generally added. The moulded adobe is left to dry in the sun and is then set in place, using a suitable mortar in the joints. This material is always prepared at the site. It is useful, therefore, to describe some details of the adobe-making process, since the resistance of adobe depends to a great extent on its preparation.

272. Not all soils are suitable for making adobe, and therefore a preliminary study of the raw material must be made before beginning the brick-making process.

273. The most suitable soils are those which contain no gravel (i.e. particles over 5 mm in size must be removed) and contain less than 50 per cent fine soils (clay and silt) by weight. Soils with more than 50 per cent of fines are not recommended because of the large amount of shrinkage that occurs when the adobe dries out. Soils which are excessively poor in fine clayey materials (less than 10 to 15 per cent) are also unacceptable, since they have no binding properties.

274. Soils which are unsuitable because of excess or insufficient fines can be improved by mixing with others until the proper proportions are obtained. Such a mixture must be carefully made, using completely dry soils in order to obtain a homogeneous mass.

275. In order to absorb the stresses caused by the contraction of the adobe when it dries out, which may cause cracks, it is customary to add wheat straw or any other suitable vegetable fibre to the mixture.

276. The straw should be cut into lengths of 10 to 20 cm and be evenly distributed throughout the mud mass. The best proportion of straw is 35-45 per cent of the adobe by volume. The volume of straw is measured by squeezing it with the hand. Where it is not possible to determine the required quantity in this way, a good handful of straw should be added for each adobe brick to be made.

277. The optimum quantity of water is difficult to establish. In general, good adobe can be obtained by adding not more than 30 per cent water in proportion to the weight of the dry soil. In any case, the amount of water used should be the minimum allowing the mud to be placed in the mould without pressing and at the same time allowing it to be easily turned out of the mould. Any excess water will produce a less resistant and poorly shaped brick.

278. Once the proper mixture of materials for the adobe (soil, straw, water) has been obtained, it should be left to stand for at least three days, so that the water will be completely and evenly distributed throughout the mass. The bricks should be moulded on a flat surface free from grass or other substances which might be detrimental to the face that remains in contact with the earth. They should be be prepared in simple bottomless rectangular moulds made of dry wood; as far as possible, the moulds should be brushed clean and their inner faces oiled. The

size of the forms or moulds will depend on the thickness of the wall to be built. An effort should be made to make the length twice the width plus the joint. For a wall thickness of 30 cm and a 2 cm joint, for example, the brick would be 30 cm by 62 cm. In no case should the brick be less than 8 cm thick. When the soil is placed in the mould, care should be taken to ensure that the corners are well filled and that the mixture is sufficiently pressed in. The upper surface should be carefully smoothed, after which the mould should be raised with care; the fresh adobe is then readv for drying. The mould should be lifted up immediately after use in order to remove any material which may have stuck to the walls. After this the moulding process is repeated.

279. Newly made adobe bricks should be left resting on their larger face as moulded, and they should not be moved for two or three days, until they can be handled without losing their shape. Once this has been achieved, the bricks will be placed on one of their lateral faces (on edge), so that air can circulate and dry the two larger faces evenly. This avoids the tendency of adobe bricks to bend and reduces the danger of cracking. Within one or two weeks, if the weather is dry and hot, the adobe bricks can be stored in high stacks near the construction site.

280. If heavy rain falls during the drying period, the adobe bricks must be protected from the water. In no case should adobe bricks be used on a construction job if they are not completely dry.

2. Materials for masonry

281. Masonry is made of solid components of more or less regular shapes (stones, bricks, blocks, etc.) of natural or artificial origin, held together with mortar. Materials of natural origin include rocks used either in the shape in which they were found or after mechanical processing in order to give them the requisite shape. Components of artificial origin are produced by the more or less activated transformation of natural materials or a combination of various materials. Such components are joined together with mortar, which fills the spaces between the pieces and holds them together. In this way, resistant waterproof walls are obtained.

(a) Stone: Stone cannot in general be regarded as an economical material, but it can be used in some regions where plentiful quarries or cheap labour or both make it competitive in price with other masonry materials. In such cases stone can replace bricks or blocks;

(b) Bricks and blocks: Bricks are solid artificial components of regular shape made of ceramic or agglomerate materials. Ceramic bricks are made from clay burned in suitable ovens. The most commonly used system is at the same time the most primitive one. It goes back to the time of the Chaldeans and consists of piling unburned bricks into truncated-pyramid-shaped stacks of 100,000 to 500,000 units. Charcoal is placed between the bricks, and the whole mass is covered with beaten clay. The charcoal is set afire, and the bricks can be removed when the burning is completed. The system is not very economical and produces bricks of irregular quality. Good quality ceramic brick is obtained by machine-moulding the raw brick and burning it in an industrial oven. Agglomerate bricks are produced by mixing sand, stones, slag, etc. with binder such as cement or lime.

The most commonly used are the hollow mortar block, the sand-lime brick and the cement mortar brick. The hollow block is a brick which has one or more hollow spaces and has walls made of a mortar of sand, grit and cement. Hollow blocks are made in various sizes and shapes, depending on the building purpose for which they are designed, but in general they are of the following sizes: length 0.40 m, height 0.20 m and breadth 0.20 m, 0.15 m or 0.10 m, depending on the thickness of the wall. Sand-lime bricks are obtained by moulding and pressing a mixture of finely ground siliceous sand and quicklime or Portland cement, which is then set in an autoclave. The result is a light and resistant brick. Mortar bricks are made with a mixture of sand and cement and are left to set in a damp atmosphere.

(c) Mortar: This is a mixture of sand and a hydraulic binder (cement, lime or plaster) formed into a mass with a certain amount of water. The presence of an inert material (sand) helps to reduce the binder shrinkage effect and prevents cracking as the mortar sets. Mortar is used to join bricks or blocks and serves as a coating and finishing material for walls. It must be of good quality in order to obtain good working consistency and adequate yield. The resistance of the joint may be affected by various factors: the type and quantity of cement, the "give" or plasticity of the mortar, the surface texture of the area to be joined, the rate at which water is absorbed by the block or brick, the water-retention capacity of the mortar and, in every case, the quality of the labour. Walls subjected to severe cold or severe structural stresses require stronger and more durable mortars than walls subjected to ordinary stresses. The cement mortar proportions recommended in table 3 are indicated for the types of structure referred to. Mortar should be prepared on a suitable mixing-board, unless a mechanical mixer is available.

Table 3. Proportions in Portland-cement mortars

Quality	Use	Cement-to-sand ratio (by volume)	Proportion per m^3 of mortar		
			Cement (kg)	Sand (litres)	Water (litres)
Very poor	Coarse masonry, not subjected to stress (enclosures, etc.)	1:5.5	277	1,000	180
Ordinary	Ordinary masonry, not required to be waterproof	1:3.5	340	950	190
Standard	Masonry in external walls or walls less than 0.20 m thick	1:3	396	900	200
Superior	Masonry which must be waterproof	1:2	566	875	225

3. Portland-cement concrete

282. Concrete is an artificial material made of sand, stone (gravel or ballast), Portland cement, which is the binder, and water. The characteristics of fresh concrete (a mass which can be moulded easily) combined with the strength, economy and plastic possibilities of hardened concrete have made this material one of the most widely used in modern times in every kind of construction work.

283. However, despite its good resistance to compression, concrete has a precarious and unreliable resistance to tension. This weakness has been remedied by reinforcing the concrete with steel rods which absorb the tensile stresses. As the two materials, concrete and steel, adhere well to each other and have similar coefficients of expansion within normal temperature ranges, they function together as a single material, known as "reinforced concrete". In addition, the concrete protects the steel from oxidation.

284. The correct calculation and distribution of the steel reinforcement in a reinforced-concrete structure depends on a number of more or less complex factors, and therefore a specialist must always be consulted. Otherwise the structure will be uneconomical and, in most cases, totally unsafe because the reinforcement has been badly placed.

285. Concrete is composed essentially of a binder and inert aggregates which serve to reduce its cost and restrict shrinkage during hardening to tolerable limits. In addition, concrete requires a certain amount of water to make the binder harden. The conditions which these elements must satisfy are given below.

(a) Binder: The binder most commonly used in making concrete is Portland cement, a product obtained by pulverizing a mixture of clinker and gypsum. Clinker is an agglutinated mass produced by calcining a mixture of claystone and limestone and heating it to the point of incipient fusion. There are various kinds of Portland cement, with different characteristics. The most commonly used are the following:

(i) Ordinary Portland cement, which corresponds to grade I of the American Society for Testing Materials (ASTM). It is the most common and economical kind;

(ii) High-early-strength Portland cement, a more finely ground Portland cement whose initial strength is higher than that of ordinary Portland cement, as its name indicates. It is used in delicate work which must be fitted out or delivered within a short period;

(iii) Waterproof cement, a special kind of cement used in construction which is to be subjected to conditions of severe humidity.

Storage of cement: Cement should be stored in closed storehouses protected from the damp and on boards.

The quality of cement is impaired when it is stored for a long time or stored outdoors. From the granules formed in those conditions it is possible to determine whether or not the cement is usable: if they crumble easily between the fingers, the cement may be used; otherwise the cement should be rejected unless laboratory analysis provides information on whether and under what conditions it may be used.

(b) Aggregates: Inert aggregates namely, gravel and sand, make up most of the volume of concrete. The principal characteristics of the aggregates which have an important bearing on the quality of the concrete are the following:

(i) Petrographical quality. This affects the final result according to the resistance of the granular material, its porosity, its chemical inertness with respect to cement in the presence of water, the roughness of its surface, etc.;

(ii) Shape and origin. River pebbles are round and smooth. It is thus possible to obtain a coarse aggregate with a lower percentage of voids and a smaller surface area of the particles. Concrete made with pebbles is more workable, flows more easily in moulds and surrounds the reinforcement better. Because of these qualities the use of pebbles as a coarse aggregate is recommended for reinforced-concrete structures;

(iii) Artificial aggregate or ballast obtained from grinding up rock. It has sharp edges and a rough surface. Concrete made with it is less workable but has a better resistance to tension and wear, and it is therefore recommended for unreinforced pavement;

(iv) Grading. Proper granulometric distribution in the aggregates is one of the basic factors in the quality of concrete. In general, "continuous" grading, in which there are particles of all sizes, is preferred. When the granulometric composition of the material is too homogeneous, good-quality concrete probably will not be obtained unless it is proportioned by special processes.

(v) Impurities. The aggregates should not contain clay or silt particles of a finer grain size than 0.074 mm (ASTM No. 200 screen) because such particles disrupt the formation of the crystallization patterns which appear when the cement sets (sand should not contain more than 5 per cent impurities by weight, and gravel not more than 1 per cent. However, when a small proportion of cement is used, such impurities are not so harmful; on the contrary, in such cases it is recommended that the aggregate should contain a percentage of fines close to the admissible limit.

The form of the clay in the aggregates also affects the resistance of the concrete. Clay is more harmful when it is adhering to the particles because it then prevents the cement from adhering to their surface.

Acid organic substances are also harmful impurities because they neutralize the basic action of the cement with the water. The use of aggregates containing them should be avoided. However, their acidity can be neutralized by means of milk of lime or by prolonged airing of the aggregates.

Lastly, there are other elements which cause delayed disintegration in concrete: sulphides and sulphates, magnesium salts, etc. These salts cause efflorescence and surface flaking of the mortar.

(c) Water: The water used in the preparation of concrete should not contain dissolved or suspended chemical elements that react with cement. Water which is used for irrigation or which passes through industrial or mining installations should not be used in concrete. It may be said that in general the water best suited to the preparation of concrete is drinking water. It is worth noting that in some cases sea water may be used, since its salt content falls within the limits acceptable under the standards, but in any event, it is advisable to consult a specialist before making concrete in such circumstances.

(d) Proportioning: The proportioning of concrete is the proper determination of the amounts of aggregates, cement and water which must be mixed to obtain concrete of the quality prescribed by the designer. It is recommended that aggregates should be analysed by a materials-testing laboratory and the mix chosen according to the analysis results whenever structures of some importance are involved. Where this is not possible, an attempt should be made to use materials which meet the requirements already established, particularly as concerns grading and impurity content. Table 4 gives five common types of mix and indicates the most common use for each of them. These proportions are given as an example and expressed by volume. The correct proportions should be given by weight, taking into account the specific characteristics of the aggregates in each case. In these types of mix it has been assumed that the materials available have the following characteristics:

Cement: Ordinary Portland cement (type I ASTM);

Water: Drinking water;

Fine aggregate: River or pit sand in which 100 per cent of the grains pass through the 3/8" (9.51 mm) ASTM screen, with continuous grading, impurity content within acceptable limits, and a dry compact porosity of approximately 27 per cent, with a loose specific gravity of 1.6 kg/dm^3;

Coarse aggregate: Gravel in which 100 per cent of the grains are in the range between the 2" (50 mm) ASTM screen and the 3/8" (9.51 mm) ASTM screen, with continuous grading, impurity content within acceptable limits and a porosity of approximately 30 per cent, with a loose specific gravity of 1.8 kg/dm^3.

(e) Preparation: Concrete should always be prepared in a concrete mixer, into which, once it is running, part of the water is first poured, then the cement and then the sand. The machine is allowed to rotate for half a minute, and then the gravel and the remainder of the water are added. The whole mixture should be rotated for a minimum of one minute.

In the case of mixers which have a lifting hopper, the materials are placed in the hopper in such a way as to fall into the rotating drum in the order mentioned above, except for the water, which will be added from the automatic proportioning tank when the hopper empties into the mixer.

It is very important to pay attention to the mixing time, calculated from the moment when the last of the material is introduced to the moment when the concrete begins to be poured out. The time depends on the type and size of mixer, but in general it should not be less than one minute. However, the mixing time should not be too long, because that may result in the separation of the aggregate, which may even be worn away or ground up by friction if it is soft.

Table 4. Typical concrete mixes

Requirements and uses	Mix No.	Cement/m^3 (42.5-kg. sacks) (kg)	Sand (litres)	Gravel (litres)	Water (litres)
Very strong (slabs, beams etc.) or highly waterproof (pavement, tanks, etc.)	1	8 340	425	735	155
Good strength. If machine-made, can be used for slabs and beams	2	7 297	430	740	155
Columns, tie-beams, reinforced wall foundations	3	6 255	435	750	155
Unreinforced concrete walls; simple foundations	4	5 212	440	755	150
Foundations of one or two-storey houses; foundation beams or slabs	5	4 170	445	760	150
Foundations of one-storey houses; foundation beams or slabs; filler material. Advisable to prepare it in a mixer	6	3-1/3 142	450	765	150

In small jobs, when it is impossible to obtain a mechanical mixer, concrete may be mixed with a shovel if certain precautions are taken. First a clean and solid mixing area should be prepared. A well-washed concrete floor, a wooden floor or metal plates which do not allow the cement paste to leak away may be used for this purpose. In no circumstances should concrete be prepared on a soil surface.

The proper amount of sand.for the mix being used is spread on the mixing surface. The cement is spread over it and the two dry materials are mixed until the mass has a uniform colour. Then the gravel is added and the three dry materials are again stirred until a uniform mixture is obtained.

Finally the water is added while stirring the mass until it is seen to be perfectly homogenous.

It should be noted that concrete prepared by hand, no matter how carefully, will have less than 60 per cent of the strength obtained by preparing concrete in a mixer. It is therefore recommended that where concrete is prepared manually, it should be enriched with 30 per cent more cement. In both processes described it should be borne in mind that the quantity of concrete prepared should not exceed the amount which can be used within an hour after the water has been added. Any concrete which has not been used within that period should be discarded.

(f) Reinforcement: The steel rods used to reinforce concrete are of uniform quality, checked by the factory or by an official authority or laboratory, and therefore it is usually unnecessary to conduct special tests before using them.

The precautions which must be taken when placing the reinforcement in the form and before placing the concrete are simple. The first is that the steel rods should be completely free of any foreign matter adhering to them, such as soil, grease, paint, rust, etc.; to clean them, it is sufficient to rub them vigorously with sacking.

Special care should be taken to check that the rods have the shape, size and arrangement indicated in the plans. It is advisable to have the reinforcement checked by a competent professional before the concrete is placed. Immediately before placing the concrete, the position of the reinforcement in the formwork should be corrected so that the prescribed protective distance of 2 cm between the surface of the formwork and the rods is maintained.

(g) Placing: When the concrete is removed from the mixer, it should be poured directly into the formwork. It should be placed with the minimum possible excess water and be well compacted so as to have more strength. The best way of "compacting" the concrete properly is by using a vibrator, but that requires expensive machinery and strong, watertight formwork. Where these are not available, a more rudimentary but nevertheless effective way of compacting the concrete is to ram it ("pun" it) vigorously by means of a round steel bar ("punner") with a diameter of approximately 16 mm and a closed hook at the end, while at the same time beating the sides of the formwork with a wooden mallet. This operation cannot be omitted, since without it the concrete will not be properly compacted.

(h) Removing the formwork ("striking"): The formwork should not be removed from a concrete member before the concrete has hardened sufficiently. In this way, when the formwork is removed, there will not be any flaking, deformation, sagging or sign of damage that could be caused by premature striking of the formwork or supports. Damage produced in that way may not be apparent to the eye, but it is always serious and irreparable and will be felt when an unusual stress, such as those produced by an earthquake, is placed on the damaged member.

(i) Curing: The purpose of curing is to keep the concrete sufficiently wet so that the setting and hardening processes take place normally. Proper curing makes the concrete harder, more durable and more waterproof and, in general, gives it greater structural strength, all of which tends to improve the final structure.

Curing should be started as soon as the concrete begins to harden, by placing wet sand and sacking on it, by frequent watering and, if possible, as in the case of slabs or pavements, by permanent flooding or by covering with polythene. The curing should be continuous; any interruption will cause irreparable damage which will not be remedied by subsequently renewing and prolonging the wetting process.

What has been said here about concrete also holds true for any other substance with a cement base, such as plasters, hollow blocks, etc.; they must be kept damp for at least the first week, while ensuring that they do not receive direct sunlight.

Table 5. Minimum periods for striking

Member	Minimum periods, in days	
	Ordinary cement	High-early-strength cement
Columns		
Non-supporting	3	2
Supporting	8	4
Slabs with spans		
Less than 3 m	8	4
Between 3 and 6 m	15	7
Over 6 m	21	10
Beams and arches		
Sides or end walls	3	2
Bottom or base:		
Under 6 m	21	10
Over 6 m	28	15

4. Timber

286. Timber is one of the most useful and most widely used building materials, since every part of a house can be built with it.

(a) Mechanical properties: The strength of timber involves the combination of properties which make it capable of resisting the stresses of different kinds to which it may be subjected. The relationship between the resistances to different stresses (compression, bending, shear, etc.) is not the same for all kinds of wood. Therefore it cannot be said that one wood is stronger than another, since strength depends on the type of stress involved. For example, it cannot be said that oak is stronger or weaker than rauli beech without specifying what kind of stress is involved, since oak resists bending better than rauli but rauli resists compression at right angles to the grain better than oak.

Timber treated with creosote, tannin, zinc chloride, etc., is in general weaker than untreated timber. Dry timber is much stronger (by about 75 per cent) than wet or green timber. It is therefore important to know the water content of a sample of timber subjected to testing.

It should also be borne in mind that the resistance of a piece of timber under any kind of permanent load is roughly half its resistance to loads which act over a short period.

(b) Safety factor and working stress of timber. The safety factor which should be used to determine the working stress of timber varies according to the kind of stress and the type of load. The following safety factors are considered adequate for "moving" loads: 10 for tension, 5 for compression in the direction of the grain, 4 for compression at right angles to the grain, 6 for bending at right angles to the grain, 2 for the modulus of elasticity for bending at right angles to the grain, and 4 for shear. For "fixed" loads, as in the case of housing units, these factors may be reduced by 33 per cent, which corresponds to an increase of 50 per cent in the working stress. These factors are for sound timber; they are too low for timber which has defects or large knots.

(c) Choice of timber: Timber for the building of housing units should be chosen from among those sufficiently resistant kinds which are the most abundant and hence the cheapest, so long as they can be obtained in the necessary lengths. If the timber is to be either partially or totally buried in the soil, the main characteristic to be taken into account is durability. Timber which will remain exposed should be free of defects (such as warp or knots), be easy to work and have a good appearance. For flooring, the main consideration is resistance to wear, and in some cases the appearance of the grain, colour, etc. should also be considered.

(d) Preservation of timber: The working life of timber may be prolonged through adequate seasoning; however, the best system is to inject it with certain substances which, without substantially damaging it, will destroy fungi, worms and other organisms that weaken timber.

The substances most commonly used to treat timber are creosote, zinc chloride, copper sulphate and bichloride of mercury (corrosive sublimate). There are three methods of injecting these substances. The first, a pressure process, requires industrial equipment - pumps, compressors etc. The second uses atmospheric pressure alone: one process is to immerse the timber for a period varying from one to six hours in a hot bath and then quickly place it in a cold bath. This change causes the air and moisture inside the timber to contract and thus allows the preservative to enter.

The third process is the simplest of all: it consists of painting the timber with one or more coats of the chosen preservative. This is the case, for example, of the coat of tar usually applied to timbers which will be in contact with the soil.

(e) Termites: Termites are very dangerous wood-eaters, but they can be completely controlled. They penetrate into the timber structures of houses through cracks in the masonry or timbers which are in contact with the ground, or through holes which they themselves bore through the soil or timber. However, in order to survive, termites must be in permanent contact with soil and moisture. If that contact is broken, they die and cease to damage the building. The first problem, therefore, is to prevent the entry of termites. This is done by ensuring that the timber does not remain in contact with the ground or near it. In addition, the upper part of foundations or masonry foundations should be protected - with sheet metal, for example - to prevent termites from entering through the cracks. The sheet metal should be continuous, and its joints should be welded. Another method of protecting the timber structure against termites is to use timber which has been impregnated with a suitable preservative, as explained above.

B. Typical failures due to faulty execution

287. A study of the structural failures occurring in buildings as a result of earthquakes reveals that in most cases one of the main causes, if not the basic cause, of the failures has been faulty execution. The most important faults of execution, classified according to structural members and construction systems, are indicated below.

1. Substructures

288. Apart from substructure failures attributable to the soil, it is important to list some defects which can contribute to poor performance in foundations and wall foundations.

(a) Foundations

When excavations prove to be wider than specified, it is necessary to provide a suitable back filling, which may consist of lean concrete or tamped earth. Carelessness in making these fillings causes settlement of the soils adjacent to the building and fracture of the piping. Poor protection of the excavation edges makes it possible for soil to fall into the concrete, thus causing discontinuities and lowered resistance.

Sometimes there are layout errors which change the location of the loads provided for in the design. These failures are aggravated by attempts to correct them by bending the reinforcements; if the defect is corrected, substantial expenditure will be required for partial demolition, the results of which are always uncertain.

Carrying out excavation works at various levels requires special instructions from the designer. It is important to note that many errors are committed in the building process when due consideration is not given to the proper sequence of operations and/or the precautions which must be taken when work is done below a foundation level.

The most frequent failures result from excavations for the placement of sewers, especially when they are very deep and close to the foundations and are carelessly filled up afterwards.

In the construction of foundations on steeply sloping ground, benched foundations should be laid after consultation with the designer of the building. It is recommended that the installation ducts passing through the foundation should be put in place first. Subsequent perforations and patches are especially dangerous.

The foundations of low-cost dwellings should always be made of cement concrete. Since its mechanical strength must be greater than that of the foundation soil, 140-170 kg of cement are usually used to manufacture each cubic metre of ready-mixed concrete, including as displacement material up to 300 litres of large stones (20-25 cm in diameter). The reinforced foundation should contain over 270 kg of cement per cubic metre in order to afford suitable protection for the steel. In no case can the use of stone foundations set in mud be accepted.

(b) Wall foundations

The same problems may arise in the construction of wall foundations as in that of foundations; in addition, the following points should be noted:

(a) All possible precautions should be taken to ensure that the surface at the junction with the foundation is clean;

(b) The forms must be carefully removed once the concrete has aged sufficiently, in order to avoid breaks in the mass of the foundation;

(c) The surface of the upper face of the wall foundations must be plane and horizontal, so as to provide adequate support for the type of wall the design calls for it to carry;

(d) The fixed anchorage components in the wall foundations and the foundations must meet the specified tolerances;

(e) The quality of the concrete in the wall foundations must be such that it prevents the rise of moisture up the walls by capillary action.

2. Brick and block masonry

289. In general, clay brick masonry for earthquake-resistant construction must satisfy the requirements common to all good masonry. The main requirement is to make a homogeneous whole which is as ductile as possible and free from internal stresses due to the contraction of the mortar. The main masonry failures attributable to faulty execution are the following:

(a) Excessive sagging (more than one thirtieth of the thickness);

(b) Segregation of the cementing mortar as a result of poor choice of cement, lack of cement or use of an excessive amount of water in the mixture;

(c) Unsuitable brick laying techniques, causing a loss of adhesion between the bricks and the mortar;

(d) Incomplete filling between bricks, careless arrangement of vertical joints;

(e) Use of high and fragile bricks (hollow bricks with very thin walls) or poorly baked bricks;

(f) Carelessness in curing of the cementing mortars;

(g) Improper arrangement of the bond and placement of the bricks;

(h) Failure to meet the specifications for the fastenings and separations between the masonry and the main structure;

(i) In the case of confined masonry, poor execution of the bond between the masonry and the reinforced-concrete columns. The length of the bonding key between the masonry and the column should not exceed one fourth of a brick;

(j) Practically the same requirements indicated in the case of clay brick masonry may be applied to buildings constructed with cement mortar blocks. In simple concrete masonry, secondary elements are made according to the same construction principles as those followed in the case of the concrete in a reinforced member;

(k) One of the most frequent failures observed after an earthquake in masonry confined by means of reinforced-concrete columns and tie-beams is failure by shear at the upper ends of the columns confining the masonry bracing walls. This type of failure, analysed in chapter III, occurs because the walls were built after the tie-beam was concreted and were therefore poorly or not at all anchored to it. Owing to this failure, the shear force of the floor, instead of being transmitted directly to the wall, is conveyed through the tie-beam to the column, and from the latter by compression to the wall, so that the column fails if its resistance to shear is insufficient (Fig. 34).

It is advisable to erect the masonry wall first and then to concrete the columns and the tie-beam in order to form a suitable anchorage between the masonry and the reinforced-concrete structure (Fig. 46).

3. Reinforced-concrete structures

290. The main failures found in reinforced concrete structures that are attributable to faulty execution relate especially to the quality of the concrete, the location of the reinforcements, and defective reinforcement anchoring, and in particular to the column-beam joints, flexible joints, etc., problems which are more fully described below:

(a) Quality of the concrete: In almost all observed cases the poor quality of the concrete in reinforced-concrete structures is due basically to failure to observe the minimum standards referred to under heading 3 of this chapter.

(i) Incorrect proportioning;

(ii) Poor quality of the preparation, especially insufficient mixing, which causes segregation and stone nests;

(iii) Aggregates with excessive impurities or improper grading;

(iv) Excessively high water/cement ratio;

(v) Excessive maximum size of the aggregates in relation to the size of the members.

(b) Construction joints: Poor execution of the concrete joints is the main and most frequent cause of destruction of reinforced-concrete members during earthquakes. In these joints there occurs a discontinuity which reduces by a high percentage the adhesion resistance of the concrete section. A further loss of adhesion results from the accumulation of gravel caused by a lack of mortar due to segregation and from the accumulation of dust and rubbish.

The study of buildings damaged by earthquakes has shown that defective concrete joints have contributed significantly to causing the failure. In that

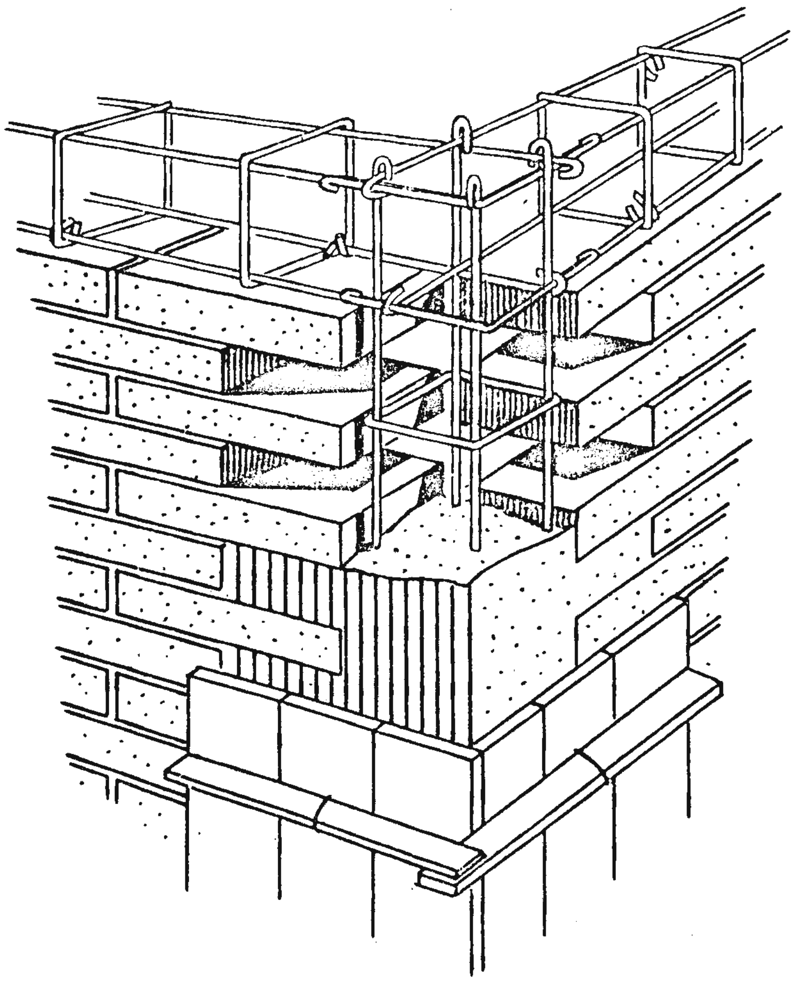


FIG. 46. TIE-BEAMS FOR CONFINEMENT OF WALLS

The tie-beam must be concreted on the wall. The illustration shows the reinforcements already in place and ready to receive the concrete once the side forms are in place.

connexion, it is recommended that construction joints should be located at the points specified by the designer following an analysis of the stresses to which the members in question will be subjected. It is also recommended that the joint surfaces should be kept clean, removing sawdust, dust and loose material and the coating of cement slurry on the face of the concrete which may cover the joint. For better results, it is advisable to provide a cement mortar coating as a base for the new concrete, after thoroughly watering the joint.

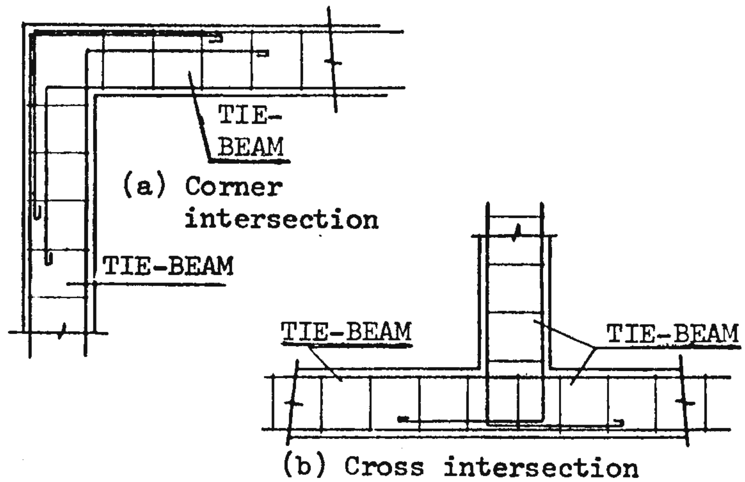
(c) <u>Positioning the reinforcements</u>: Proper positioning of the reinforcements is essential to proper performance of a structure, especially in case of an earthquake, and most particularly when dealing with longitudinal reinforcements and stirrups. In many cases the reinforcements are displaced during the process of casting and shifted to places different from those provided for in the design. The resistance capacity of a member is lowered when the steel is displaced towards the inside, while oxidation problems may arise when it is displaced towards the outside. In extreme cases, reinforcements may move from the tension side to the compression side.

The most frequent problems with stirrups arise at the upper ends of the columns, where they are sometimes spaced farther apart than in the rest of the column. It is recommended that great care should be taken to keep the stirrup spacing as close as specified by the designer, up to a joint higher than the junction of the column with the beam or tie-beam. In no case should wider spacing be allowed between stirrups at the upper end of the column. It should be noted that in case of an earthquake the column is subjected at that point to the maximum combination of stresses. Furthermore, it is very important to have proper confinement of the concrete in that area in order to ensure adequate ductility.

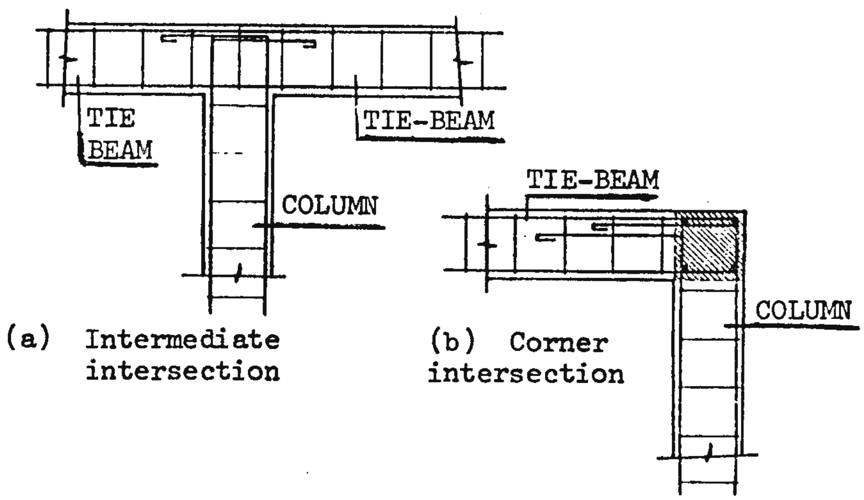
(d) <u>Defective anchoring of the reinforcements</u>. <u>Column-beam junction</u>: Some of the main problems of execution relate to the shape, size and location of the anchors, the bending of bars in order to position them correctly, the location of the overlapping of bars and the size of the concrete embedment.

Inspection of reinforced-concrete structures after an earthquake reveals many failures due to defective anchoring of the reinforcements rather than to failures of the steel itself, since the reason why the steel no longer took the stress was always that the defective anchoring allowed it to slide before failing, especially at column-beam junctions, at which such a failure occurs quite frequently. The failure of the column-beam junction is due to the lack of ductility of the junction resulting from poor confinement of the concrete in the area considered. Ductility, which is a basic factor in the absorption of energy by the structure during an earthquake, is obtained by using good-quality concrete, not less than 220 kg cm^2, and by ensuring a sufficient confinement of the concrete by means of a minimum cross reinforcement and anchor lengths in accordance with the characteristics and quality of the materials used. Figure 47 shows the arrangement of reinforcements between tie-beams or other beams or between such members and columns. Anchor lengths are based on the norms in force, and those specified by the designer must be maintained. The anchor length depends on the quality of both the steel and the concrete used.

In order to make the steel and the concrete function properly in combination, it is recommended that high-resistance steel should not be used in combination with relatively low-resistance concrete, or for small buildings in areas where



(A) TIE-BEAM-TO-TIE-BEAM INTERSECTION (GROUND PLAN)



(B) COLUMN-TO-TIE-BEAM INTERSECTION (FRONT VIEW)

FIG. 47. ILLUSTRATION OF ANCHORS AT TIE-BEAM-TO-TIE-BEAM AND COLUMN-TO-BEAM INTERSECTIONS

it is difficult to obtain average-to-high-resistance concrete (220 kg/cm^2), without strict supervision.

Steel splices must not coincide at the same cross section, and an accumulation of splices must not be positioned just above a concrete joint, as is often the case in vertical members. This situation becomes more complicated when hooks are used and the sections are narrow.

Curves in reinforcement bars cause thrusts on the concrete when the bar is subjected to tension or compression. Such a shape is considered in the design only when there is radial reinforcement to absorb the stress. These curves are designed to correct the position of the reinforcements. The case is more serious when the concrete embedments are relatively thin.

(e) Supports, anchors or fastenings: A constant problem of execution consists in ensuring that the conditions of support or embedding of structures or of parts fixed to them correspond to those specified in the design. This is the case with stairway supports, anchors of partition panels or door and window frames. Important problems arise in connexion with expansion joints and possible contacts between neighbouring buildings; the study of structures damaged by earthquakes has revealed many cases of structures adjoining stairways which act as a prop, thus causing rigidity in a part of the structure and contributing to its failure. The fastenings of some ornaments, facings and roofing elements, the anchors of the roof and, in general, of all installations involve such problems.

It frequently occurs that these joints are not given much attention because they involve non-structural elements; however, in addition to the possibility of causing panic among the occupants, they are more than heavy enough to cause a serious accident.

4. Fittings

291. In the installation of fittings the structural members must not be damaged. Structural members are often indiscriminately pierced in order to install piping. The location and method of installation must be well established in advance, and it is recommended that the fitting ducts should not pass through columns, walls and slabs.

All the specifications for the insulation of piping must be complied with in order to avoid the possible effects of corrosive electro-chemical processes and the effects of humidity and high temperatures.

5. Roofing

292. Trusses are commonly used for roof structures. Frequent use is also made of reinforced-concrete cover plates or slabs. The fundamental importance of the fastenings which fix these components to the basic structure has already been indicated. In constructing timber or metal trusses, special care should be given to the joints (welding, nailing, connectors, adhesives, etc.). The main errors observed are attributable both to poor jointing systems and to lack of alignment between the axes of the members.

293. The roof structures of low-cost dwellings are frequently built of timber left over from the construction process, and their construction is often somewhat careless. It should be noted that such timber roofs may be exposed to severe temperature and humidity changes, resulting in substantial deterioration of the material.

6. Construction defects in adobe houses

294. It is of some interest to list the main construction faults affecting the stability of adobe houses since this material is widely used in rural areas in the developing countries.

(a) Poor adobe-making technique;

(b) Use of insufficiently dried adobe;

(c) Incomplete covering in the horizontal layer between adobe blocks;

(d) Incomplete fill of the vertical joints between adobe blocks;

(e) Poor geometrical quality of the walls (undulations and sagging);

(f) Poor interlocking at the wall intersections;

(g) Lack of adjustment in the wall arrangement and introduction of adobe pieces to correct the arrangement;

(h) Walls built too rapidly (the maximum vertical rate should be 1 metre per day);

(i) Plastering without taking the necessary precautions to ensure adhesion of the coating to the wall;

(j) Failure during construction of the wall to insert the wooden plugs for fastening frames, partitions and ornaments;

(k) The courses must be laid along the whole contour of the walls, avoiding construction by complete or isolated panels, in order to prevent possible differential settlement cause by the loads or by different degrees of dryness;

(l) Lack of continuity in the timber tie-beams because of careless execution of the joints at the corners and intersections (splicing, use of double-headed nails) and/or the splicing of the longitudinal parts;

(m) Poor execution of the wall coverings designed to protect the adobe blocks.

C. Repairs to structures

295. The repairing of structures damaged by earthquakes involves a number of special considerations not involved in the designing of new buildings. The first question to consider is whether the repairs are economically justifiable, since they might well be impracticable in the light of actual architectural limitations and structural conditions.

296. In designing repairs to strengthen a weakened structure, a comprehensive approach to the problem should be taken, so that the repair work will actually remedy the causes of the problem rather than serve as a mere palliative. The repair process should comprise the following stages:

(a) General examination of the structure;

(b) Detailed examination of the damaged areas;

(c) Redesigning of the structure;

(d) Explanation of the damage;

(e) Conclusions;

(f) Repair design;

(g) Execution;

(h) Inspection of the results.

297. The initial examination should be a general one made by competent specialists. The purpose of such an examination is to determine the likely alternatives and to report on the general functioning of the structure. The subsequent detailed investigation may be carried out by the same or other specialists and should provide information regarding the quality of the materials, their arrangement in the building, the distribution of cracks, deformations etc. It is important to have information regarding the quality of the foundation, soil, peculiarities of the building, interference of neighbouring structures or changes in use which may have contributed to the damage that has occurred.

298. The best techniques known should be employed to make as thorough a check as possible of the structural design. An attempt should be made to assess as realistically as possible the stress phenomena in the existing structure, taking into account its complexities and the characteristics resulting from the method of its construction. By comparing the results of the new calculations with the actual history of the structure, it is possible to find an explanation for the damage.

299. It is often impossible to detect the initial point of the failure, which does not always coincide with the most spectacular damage. The conclusions include the basic concept of reinforcing the building, which may involve: (a) replacing damaged members; (b) strengthening existing members; (c) adding new earthquake-resistant members.

300. On the other hand, fundamental changes in the structure or in its use may be made. Change in the structure may involve the construction of new walls or new supports for members with a long clear span. Change in use may entail redistributing loads or reducing stresses in certain areas, which can be brought about by altering the function of specific spaces and adding new members.

1. Execution of repairs to reinforced-concrete buildings

301. In repairing buildings made of reinforced concrete, the following scheme should be followed:

(a) Unload the affected members;

(b) Remove the damaged materials;

(c) If necessary, install new reinforcements;

(d) Carefully clean the areas where old and new concrete will come into contact;

(e) Put the formwork into position;

(f) Pour the concrete under pressure;

(g) Allow the concrete to cure and then remove the formwork.

302. The above scheme for replacing concrete is followed in cases where there is serious damage. When cracks are less than 1 mm wide, it is possible to fill them with epoxide resins.

303. For good results it is essential to obtain good adhesion between old and new concrete. This necessitates carefully cleaning the contact surface with compressed air, a sand-blast and/or a steel brush. The purpose is to free the surface of loose particles, dust or other foreign bodies. To prepare a surface which has been in contact with formwork or the exterior surfaces of a member which has been in contact with the air, the outer layer of carbonated mortar is removed completely. In no circumstances is it sufficient merely to roughen the surface here and there with a chisel.

304. It is difficult to obtain good bonding with the horizontal surfaces of existing members. It may be obtained by single-face or double-face bevelling, so that during concreting good contact will be made with the existing piece, thus ensuring that no air or laitance becomes trapped on that surface. These steps are supplemented by revibrating the concrete, mixing it with an expander additive, using the maximum possible amount of coarse aggregate and using formwork in which the concrete can be poured under pressure. If the necessary equipment is available, the use of pre-packed concrete (Prepakt) is recommended.

2. Repairs on brick masonry

305. When dealing with brick masonry walls damaged or cracked by an earthquake, it is necessary to distinguish the case of resistant walls confined by columns and tie-beams from that of non-resistant walls.

306. In the case of resistant walls it is essential to determine whether the damage involves the joint between the wall and the reinforced-concrete structure, with a fissure or crack along the contact surface. If this is the case, the wall cannot be repaired. It must be demolished and replaced with a concrete wall which will function properly in combination with the other members.

307. If the failure does not involve the joint between the wall and the structure, cracks can be filled with cement mortar to which some expander additive or epoxide resin has been added.

308. Damaged non-resistant walls can be repaired or rebuilt without much difficulty if care is taken to preserve the independence of nearby structural members so as not to alter their rigidity.

3. Repairs to adobe dwellings

309. An adobe wall which is cracked, subsiding or partially destroyed must be demolished and rebuilt. Patching will not suffice. In general, this opportunity should be taken to lower the height of walls where excessive and to rebuild two-storey buildings as single-storey buildings.

310. Below are a number of suggestions for repairing lintel cracks or separations which occur at wall joints. The lintel crack is one of the most common failures and generally starts at the corners of doors or windows (Fig. 48). Possible causes of cracking are: (a) weak lintel; (b) inadequate embedding of the lintel; (c) excessive load of adobe on the lintel; (d) excessive width of the span; (e) a structural member of the roof resting on the centre of the span; (f) lack of a tie-beam.

311. Repairs consist of replacing the lintel, reducing the load of adobe, narrowing the span, installing a tie-beam and rearranging the roof load.

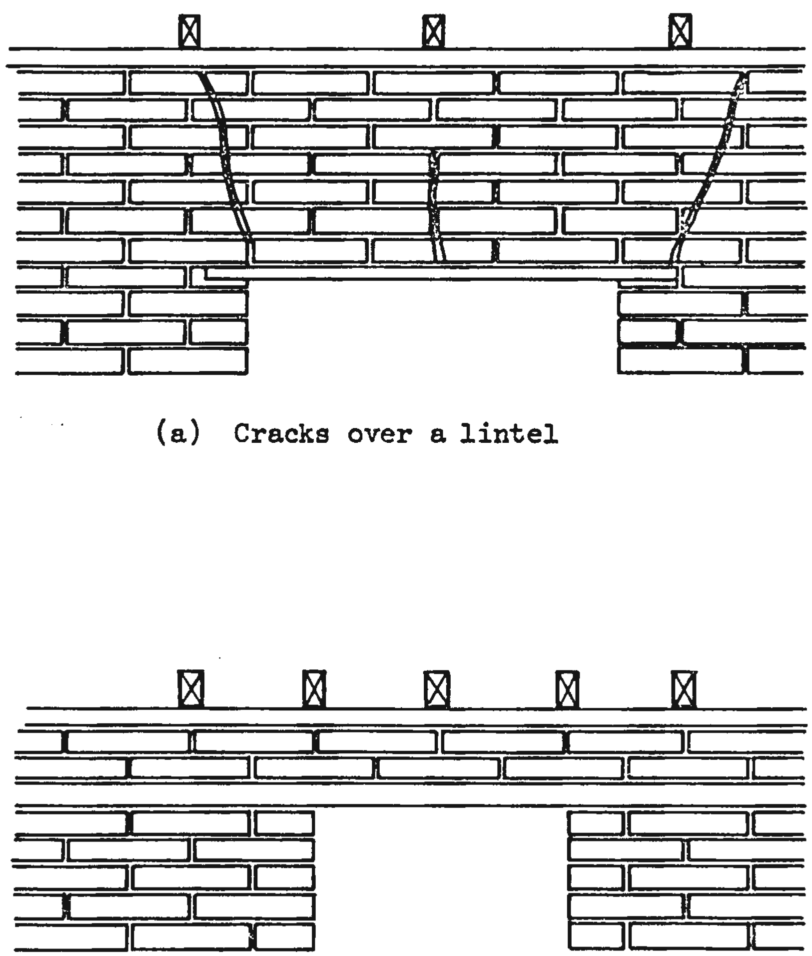
Repairing of cracks: For these, steel rods 6-10 mm in diameter with 10 x 10 cm plugs 5 cm thick are used. The rods are placed every 50 cm, in alternate directions, as shown in Fig. 49.

Separation at wall intersections: Vertical cracks may appear at the corner of wall intersections as a result of separation or collision of the walls during an earthquake. If there is no significant subsidence (1 cm for each metre of height, up to 3 cm), the following methods may be used:

(a) Install a tie-beam;

(b) Reinforce the entire area with timber or steel ties and two steel rods.

312. The procedure is the following: using nuts and washers, fasten two rods 16 to 25 mm in diameter to 2-1/2 inch oak ties or 1/2-inch steel plate. The wall remains in its current position, with no attempt being made to restore it to its original position. The crack is filled with clay and straw.



(a) Cracks over a lintel

(b) Repaired structure

FIG. 48. REPAIRING A LINTEL IN AN ADOBE HOUSE

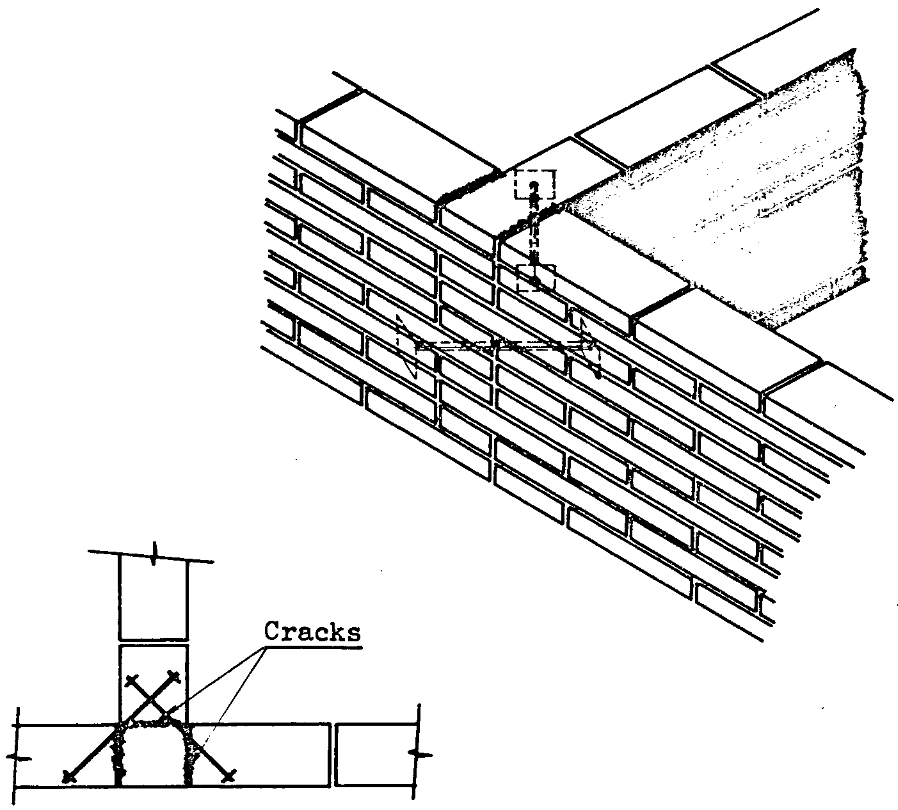


FIG. 49. REPAIRING A CRACK AT THE INTERSECTION OF ADOBE WALLS

BIBLIOGRAPHY

The following is a list of recommended reference works arranged in the approximate order in which their subject matter is dealt with in the body of this text.

Asimicidad en viviendas económicas, cinva-(OEA), Bogotá, 1959.

El sismo de mayo de 1970 en el Perú, report prepared for the United Nations, Multilith, Santiago, 1970.

Conference on Seismic Design for Nuclear Power Plants, Arias, A., "A Measure of Earthquake Intensity", MIT, Cambridge, Mass. April 1969.

"Plate tectonics and continental drift", D. P. McKenzie, Endeavour (ICI), Vol. XXIX No. 106, London, January 1970.

Elementary Seismology, Richter, C. F., Freeman and Co., 1958.

Scale of Seismic Intensity, Medvedev, S. V., Sponheuer, W., 4th World Conference on Earthquake Engineering, Santiago, Chile, January 1969.

Earthquake Engineering, Wiegel, R. L., editor, Prentice-Hall, 1970.

An Introduction to the Mechanics of Solids, S. H. Crandall, N. C. Dahl, McGraw-Hill, New York, 1959.

Strength of Materials, S. Timoshenko, D. H. Young, Van Nostrand, Princeton, 1962.

Strength of Materials, J. P. Den Hartog, McGraw-Hill, New York, 1949.

Dynamics and Basis for Foundations, Barkan, D. D., McGraw-Hill, New York, 1962.

Soil Mechanics in Engineering Practice, Terzaghi K., Peck R., John Wiley, New York, 1968, 2nd edition.

Shear Resistance of Sands During Rapid Loading, R. V. Whitman and K. A. Healy, Report No. 9 on Response of Soils to Dynamic Loading, MIT, 1962.

Analysis of Soil Liquefaction; Nilgata Earthquake, H. B. Seed and I. M. Idriss, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 93-SM3, 1967.

International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, Proceedings, University of New Mexico, 1967.

Vibration Problems in Engineering, S. Timoshenko, D. H. Young, Van Nostrand, Princeton, 1955.

Mechanical Vibrations, J. P. Den Hartog, McGraw-Hill, New York, 1956.

Vibrations, Theoretical Methods, Y. Chen, Addison-Wesley, Mass. 1966.

Shock and Vibration Concepts in Engineering Design, C. E. Crede, Prentice-Hall, Englewood Cliffs, 1965.

An Introduction to the Dynamics of Framed Structures, G. L. Rogers, John Wiley, 1959.

Design Essentials in Earthquake Resistant Building, Architectural Institute of Japan, Tokyo, 1970.

Dieseño sísmico de edificios, E. Rosenblueth y L. Esteva, Edic. Ingeniería, México, 1969.

Construcciones antisísmicas, José Creixell M., Editorial Continental, S. A., México, 1969.

Cálculo antisísmico de edificios, Norma Chilena Inditecnor NCh433, Instituto Nacional de Investigaciones Tecnológicas y Normalización, Santiago, Chile.

Annex I

PROVISIONAL REGULATIONS FOR EARTHQUAKE-RESISTANT STRUCTURES

VENEZUELA, 1967

CONTENTS

PROVISIONAL REGULATIONS FOR EARTHQUAKE-RESISTANT STRUCTURES

Article 1

Validity:

These Provisional Regulations shall supersede article 7, "Action of Earthquake Movements", of Part II, "Loads and Live Loads", of the Building Design Regulations - 1955.

It is pointed out, in particular, that Part III, "Walls and Partitions", of the Building Design Regulations - 1955 remains in full effect and shall be strictly observed in zones 3 and 2 of the new Seismic Map in all respects that do not clash with these Provisional Regulations.

Article 2

Scope:

These Regulations shall apply to all structures designed or constructed by the Buildings Administration of the Ministry of Public Works.

The requirements contained in these Regulations shall not apply to the design of other engineering works.

Article 3

Terminology and symbols:

Base level:

The term "base level" shall mean the level of a building's ground floor.

If the building has cellars and if the bearing walls are not confined in their entire height or perimeter, the base level shall be that of the storey immediately below the level at which they are confined.

On sloping ground the base level shall be that of the lowest storey of the building.

Shear force at the base:

The shear force at the base is the horizontal shear force produced by the seismic movement at the base level.

Seismic coefficient C:

The coefficient C is defined as the ratio of the horizontal shear force T acting on the building's base level to the weight P of the building above the said level (table 1).

b_i = horizontal dimension of the i-th storey in a direction perpendicular to T_i.

e_i = distance between the centre of rigidities of the i-th storey and the line of action of T_i.

h_i = height of the i-th storey i above the base level.

h_n = total height of the building.

i = ordinal number of the storey counted from the base. For the base level, i = 0.

n = total number of storeys.

C = seismic coefficient (table 1).

C_p = seismic coefficient for elements that are not part of a structure (table 2).

F_i = horizontal force applied to the i-th storey.

F_p = horizontal force acting upon elements that are not part of a structure.

M_i = moment with respect to the i-th level of the forces P_i acting above it.

M_{ti} = torsional moment caused by the forces T_i at the i-th level.

$M_{ov.i}$ = overturning moment at the i-th level.

P = total weight of the building at the base level.

P_i = weight of the storey at the i-th level.

P_p = weight of the elements which do not form part of a structure.

T = shear force at the base.

T_i = shear force at the i-th level.

α_i = coefficient of reduction for obtaining the overturning moment.

Article 4

General requirement:

All new buildings of not more than 20 storeys or not more than 60 metres in height shall be analysed, and the calculations for them made, according to the static method explained below. In the said method, seismic stresses shall be treated as horizontal loads applied at the level of each storey, all acting in the same direction, which may be any direction in the horizontal plane.

In every case, the analysis and calculations of the structures shall be made in two orthogonal or approximately orthogonal directions, unless there exists a direction of application of the horizontal forces which will give rise to more unfavourable stresses.

For buildings of more than 20 storeys or more than 60 metres in height, it shall be required that, in addition to the static method, methods of dynamic analysis duly approved by the competent authority should be applied to the study of the seismic action. The shear forces finally used may not be less than 80 per cent of those obtained by the static analysis method.

Buildings intended for use as dwellings (houses and country homes) of not more than two storeys and not more than 8 metres in height shall be checked for seismic effects; approximate methods of analysis may be used for this purpose.

Article 5

Classification of buildings:

For the purposes of the application of these Regulations, buildings shall be classified according to their intended use and to the type of structure.

Use for which a building is intended:

(1) Governmental, municipal, public-utility or public-service buildings (post and telegraph offices, broadcasting offices, pumping stations, power stations and telephone exchanges, fire stations and police stations, etc.); buildings that are important in the event of a disaster (hospitals, first-aid posts, etc.); buildings with valuable contents (museums, libraries, etc.); schools, stadiums, theatres, places of worship, transport stations and, in general, all places characterized by the fact that they often contain large numbers of people.

(2) Buildings for public or private use which are not usually filled with large numbers of people (dwellings, offices, banks, hotels, restaurants); industrial plants and installations, material or equipment warehouses and buildings whose collapse might endanger buildings in this group or in group 1.

(3) All isolated buildings which cannot be classified in either of the above groups (stables, storehouses, sheds, etc.) and whose collapse would not damage buildings in the first two groups.

Types of structure:

(I) Structures whose deformations under seismic stress, in the direction under consideration, are due essentially to flexure of their structural members, in other words, structures that are sufficiently ductile to prevent brittle-fracture failures; for example, systems consisting mainly of portals and therefore resistant to lateral forces.

(II) Structures whose capacity to withstand seismic forces in the direction under consideration consists essentially in resistance to shear or axial force and which are therefore not sufficiently ductile to prevent brittle-fracture failures. This group includes, inter alia, all systems that are resistant to horizontal forces, and consist mainly of concrete or brick walls.

Both type I and type II must have, in the direction under consideration, two or more lines of resistance which consist of walls, portals, etc. and in which the horizontal members constituted by the floor structures and roofs are sufficiently rigid and resistant to distribute the seismic forces effectively among the vertical members.

(III) Structures supported by a single column or having only one line of resistance normal to the direction under consideration, or structures whose floor structures and roofs lack the necessary rigidity and resistance to distribute the seismic forces effectively among the various vertical members.

Article 6

Seismic coefficient for static calculations:

It will be assumed that the distribution of horizontal accelerations is linear, with a value of zero at the base of the structure and a maximum value at its top, so that the ratio $\frac{T}{P}$ will be equal to the coefficient C, the values of which are given below.

The value of P will be the sum of the relevant partial loads measured as follows:*

(1) For the roofs, the total dead load.

(2) For the floor structures, the total dead load plus 25 per cent of the design live load. This does not apply to warehouses and storehouses or, in general, to any building with a permanent live load, in which cases the full live load will be taken into consideration.

(3) For chimneys, walls or other similar elements, the total weight of the structure.

(4) For storage tanks, the weight of the structure plus the weight of its contents. For buildings situated in seismic zone 3 the static analysis shall be based on the coefficients given in table 1. The coefficients will be multiplied by 0.5 for all buildings situated in seismic zone 2 and by 0.25 for all buildings in zone 1.

No seismic calculation is necessary for buildings belonging to group 3 according to their intended use, nor for any building situated in zone 0 on the map.

---

* Particular care must be taken when estimating the load P, in order to obtain a realistic value.

Table 1

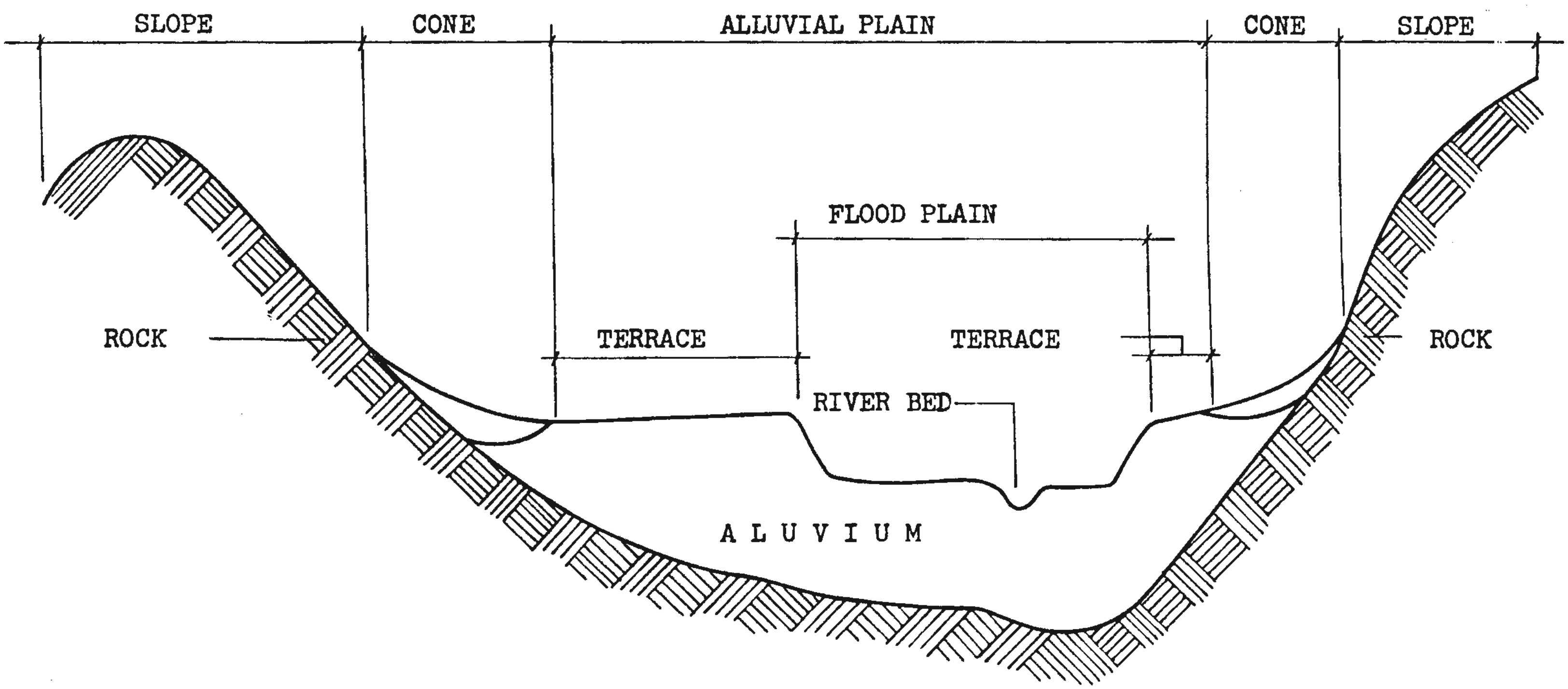
Type of structure	Values of C: Alluvial deposits (1)		Rock (2)	
	Use for which building is intended		Use for which building is intended	
	Group 1	Group 2	Group 1	Group 2
I	0.075	0.06	0.06	0.045
II	0.09	0.07	0.11	0.085
III	0.15	0.12	0.13	0.10

(1) Alluvium: In general and for the purposes of these Regulations, "alluvium" shall mean any detritus resulting from the action of rivers of recent times, such as sediment in river beds, alluvial plains, flood plains, lake plains, cones at the foot of mountains, estuaries and deltas.

(2) Rock: For the purposes of these Regulations, "rock" shall mean any natural conglomeration *in situ* of mineral particles joined together by permanent cohesive forces of such magnitude that gentle mechanical means, such as agitation in water or in chemical substances, cannot destroy them.

The diagram in Fig. 1 gives an idea of the situations that most commonly occur in practice.

The classification of the foundation soil for the purposes of table 1 shall be established in the soils survey and recorded in the relevant report.


SLOPE
CONE
ALLUVIAL PLAIN
CONE
SLOPE
FLOOD PLAIN
ROCK
TERRACE
TERRACE
ROCK
RIVER BED
A L U V I U M

ANNEX I - FIG. 1

Article 7

Vertical distribution of seismic forces in the static method:

The horizontal forces applied at the level of each storey shall be calculated according to the following formula:

$$F_i = CP \frac{P_i h_i}{\sum_{i=1}^{i=n} P_i h_i}$$

In buildings with cellars below the base level, where the cellar is entirely confined by the ground, it may be assumed that the seismic forces at the level of each cellar floor are zero and that therefore the shear force is constant and equal to T.

Elements that are not an integral part of the structure of the building or of the subsidiary related structures shall be designed to withstand the horizontal force calculated by applying the above rule, it being assumed that the said elements form a single entity with the structure, or the horizontal force calculated by applying the coefficients set out in table 2, whichever is greater.

Horizontal elements such as marquees, balconies, eaves, etc., shall be designed to withstand 1.3 times the stress resulting from the dead load and the design liveload.

Table 2

Values of C_p for seismic zone 3[1/]

$(F_p = C_p P_p)$

Structure in question	C_p	Direction of the force
Bearing walls and any other type of wall or partition	0.20	Perpendicular to the surface of the wall
Balustrades, attics and any vertical parapet when they are in cantilever	1.0	Perpendicular to the surface of the wall
Exterior and interior ornamentation	1.0	Horizontal in any direction
Towers, storage tanks and their contents, chimneys and enclosed balconies on buildings	0.25	Horizontal in any direction

1/ For seismic zones 2 and 1 the values in table 2 shall be multiplied by 0.5 and 0.25 respectively.

Roofs and floor structures which act as diaphragms shall be designed for the stress calculated by applying the coefficient $C_p = 0.10$ or the stress calculated by applying the appropriate coefficient C_p according to the formula $C = \frac{T}{P}$, whichever is greater.

Article 8

Horizontal distribution of shear forces:

In buildings with floor structures and roofs consisting of slabs or plates, whether solid or ribbed, or of other members having equal horizontal rigidity, the shear force T_i acting at the i-th level shall be distributed among the various vertical members in proportion to their respective rigidities.

Care must be taken to ensure that the said slabs or plates have the necessary rigidity and resistance to achieve such distribution. If their deformation is appreciable, their flexibility must be taken into account in the distribution of the shear force.

Prefabricated reinforced-concrete or precompressed-concrete members forming the floor structures or roofs may be regarded as capable of effecting a distribution in accordance with the rigidities of the vertical members, provided that effective joints are made between the various members and provided it has been ascertained that the deformations of the horizontal diaphragm so constituted are negligible.

In floor structures and roofs without slabs or other members having equal horizontal rigidity, the shear force shall be distributed among the vertical members in proportion to the vertical load acting upon them.

Article 9

Overturning moment

The axial loads of the vertical members and of the foundations will be modified owing to the action of the overturning moment M_{ov}, which shall be determined according to the formula

where $$M_{ov.i} = \alpha_i M_i ,$$

$$\alpha_i = 0.8 + 0.2 \frac{h_i}{h_n}$$

The reduction coefficient α_i shall be assumed equal to unity in buildings of not more than four storeys or not more than 12 metres in height.

Article 10

Torsion:

To the shear forces for the vertical members at each level, calculated according to the rules in these Regulations, shall be added those obtained by applying a torsional moment calculated according to the following formula (Fig. 2):

$$M_{ti} = T_i \ (1.5 \ e_i \pm 0.05 \ b_i)$$

The sign resulting in the more unfavourable condition shall be chosen for each structural member.

Article 11

Limitation of horizontal displacements:

The maximum relative displacement between two points in a building which are situated on one vertical line shall not exceed 0.2 per cent of the distance between them.

An exception shall be made in the case of roofs and those floor structures which do not normally bear a live load.

It is recommended that precautions should be taken to ensure that the filler elements, window frames, etc. are not damaged by deformations of the structure.

Article 12

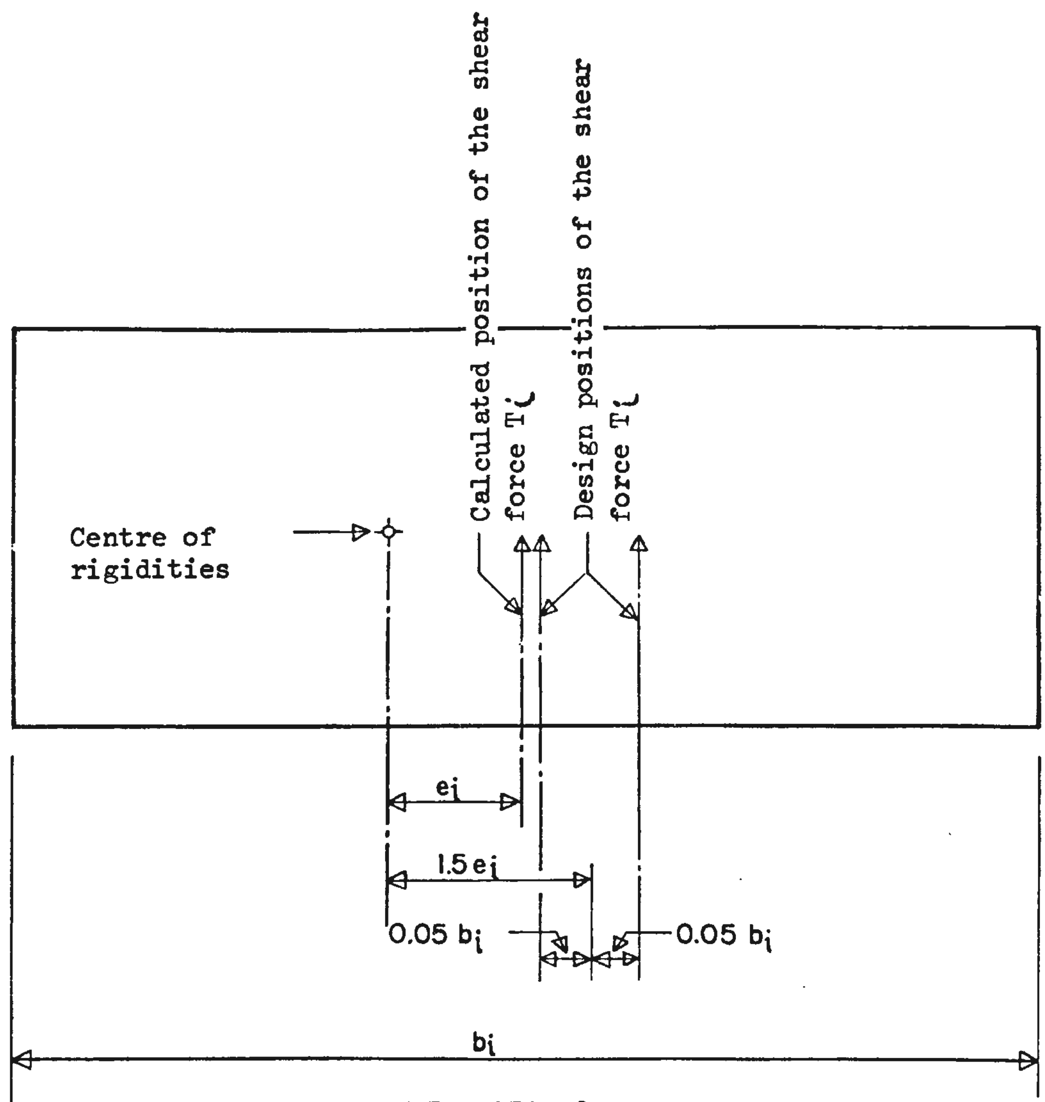
Separations at the boundaries of structures and at expansion joints:

Every construction must be separated from neighbouring constructions by a distance of not less than 2.5 centimetres for the first 6 metres of height. An additional separation of 0.4 centimetres shall be allowed for each additional metre over and above that height.

The additional separation may be reduced, provided that special precautions are taken to prevent damage caused by the seismic shock.

The same criterion shall apply to expansion joints.

The joint-covers placed at such separations must allow for differential movement and yield plastically or break before fracture of the members which they cover.



Calculated position of the shear force T_i
Design positions of the shear force T_i
Centre of rigidities
e_i
$1.5 e_i$
$0.05\ b_i$
$0.05\ b_i$
b_i

ANNEX I - FIG. 2

Article 13

General requirements and recommendations:

Reinforced-concrete or heavy-brick (adoboncito) walls shall be considered integral parts of the structure, both for the calculation of rigidities and deformations and for the distribution of seismic forces, and their dimensions shall be determined according to the stresses placed upon them. If it is not deemed advisable to have such members act in combination, separations compatible with the deformation of the structure shall be left between them and the structure.

All of the above-mentioned members which contribute to the structure's rigidity shall be drawn on the structural plans and may not be eliminated or altered without written authorization from the competent authority.

If the deformations due to flexure in the walls and to axial load in the columns are considerable, they shall be taken into account in the distribution of forces.

Any building which has an irregular floor plan or is composed of various sections may be designed as a single structure, provided the horizontal members are calculated in such a manner that the building behaves as a single unit in so far as the effects of seismic stresses are concerned. Particular care shall be taken in the design and calculation of the joints between the various sections.

Roof structures shall be properly braced and anchored so that they can effectively transmit their own seismic forces to the supporting structure.

Portals designed to absorb seismic forces shall be clearly defined as such, so that the rigidities determined by the calculation will actually be the true rigidities.

Particular care shall be taken in designing and calculating the main and emergency staircases, particularly if they are orthopolygonal.

It is recommended that in columns and beams the spacing between binders or stirrups in the areas adjacent to the nodes should be reduced to half of the distance calculated for the rest of the member. In columns, this area shall include one fifth of the height of each storey. In beams, this area shall extend to a distance at least twice the height of the beam.

Particular care shall be taken in evaluating the shearing and flexural stresses due to the combined action of vertical and seismic loads, particularly at the edges or ends of slabs and beams.

The placement of swimming pools in buildings is not recommended. If a pool is so placed, a dynamic calculation must be carried out regardless of the building's height or the number of storeys.

Appropriate reinforcements shall be provided to absorb the stresses produced by the inversion of forces which results from vertical seismic shocks.

Article 14

States of loading and safety factors:

When the combined action of vertical and seismic loads is considered, the normal values of the safety factor both for steel and for concrete may be increased by 33 per cent.

This increase shall not apply in the case of marquees, balconies, corbels, etc.

Simultaneous action of wind and earthquake need not be considered.

Article 15

Repairs:

Buildings that suffered severe structural damage in the earthquake of 29 July 1967 shall be repaired in accordance with plans approved by the competent authority. Such plans must ensure that the structure will be at least as strong as it was originally, provided that the said authority deems the original strength satisfactory by present standards.

Article 16

Instruments:

All buildings with more than 10,000 square metres of floor space or more than 45 metres in height shall contain at least two accelerographs capable of precisely recording strong movements. The plans must allow for space to install these instruments.

The Ministry of Public Works shall be responsible for the monitoring and maintenance of the instruments.

<u>Annex II</u>

PHOTOGRAPHIC ILLUSTRATIONS

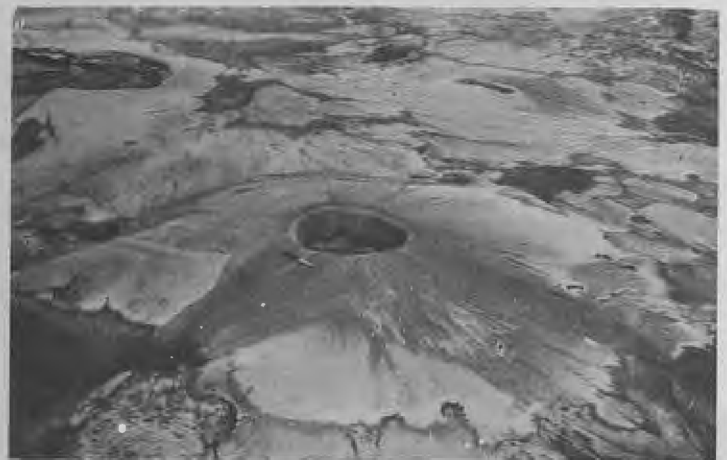
This annex contains photographs taken by the authors in the course of visits to earthquake-affected areas. They illustrate a number of typical failures and conditions often found in structures which have suffered the effects of earthquakes.

Chimbote, Peru
Earthquake of 31 May 1970

The landslip caused a slide towards the sea. Note the relative displacement between the piping anchored to the Sogesa pier and the powerhouse.

Puerto Casma, Peru, earthquake of 31 May 1970

The general subsidence of the ground caused flooding through emergence of the water table to the surface. Condition of the town two months after the earthquake.



Chimbote, Peru, earthquake of 31 May 1970

Classic evidence of liquefaction of sand in a hydraulic fill in the harbour of Chimbote.

San Antonio, Chile, earthquake of 8 July 1971

Wharf. Densification of the fill has caused it to settle, which is shown by the rupture and lowering of the pavement.

San Antonio, Chile
Earthquake of 8 July 1971

Wharf.

(a) Settlement of fills
(b) Rupture of the pavement.

Llo-Lleo, Chile
Earthquake of 8 July 1971

Maipo Bridge

(a) Displacement of the spans on the supports
(b) No anchors.

Pan-American Highway, Northern Chile section, earthquake of 8 July 1971

Llay-Llay overpass

Fracture of piers and collapse of the railing

Pan-American Highway, Northern
Chile section
Earthquake of 8 July 1971

Llay-Llay overpass

(a) Detail of a pier
(b) Inadequate stirrups at a node
of the portal.

Pan-American Highway, Northern Chile section, earthquake of 8 July 1971

Pullalli Bridge

(a) Partial collapse of spans which were simply supported
(b) Supports and anchors badly designed.

Pan-American Highway, Northern Chile section, earthquake of 8 July 1971

Pullalli Bridge

Detail of the fallen spans

Huaraz, Peru
Earthquake of 31 May 1970

Adobe dwelling

(a) Typical corner failure
(b) Inadequate size of the adobe blocks.

Putaendo, Chile, earthquake of 8 July 1971

New adobe house

(a) No tie-beam
(b) Lintels inadequately encastered
(c) Excessive height-to-thickness ratio of the walls
(d) Inadequate structural configuration.

Putaendo, Chile, earthquake of 8 July 1971

New adobe dwelling

(a) Tie-beam present
(b) Concrete wall foundation
(c) Light roof, well braced, with adequate eaves
(d) Building well distributed in ground plan from the structural point of view
(e) Good performance.

Huaraz, Peru
Earthquake of 31 May 1970

View of part of the city

(a) Confined-masonry building with no important failures despite its inadequate structural configuration and construction
(b) Unconfined-masonry building has collapsed
(c) Adobe houses destroyed.

Llay-Llay, Chile, earthquake of 8 July 1971

Parish secondary school

(a) Prefabricated-concrete panels with asbestos-cement roof
(b) No roof-wall anchors
(c) No wall bracing.

Llay-Llay, Chile, earthquake of 8 July 1971

House of light concrete panels with a timber frame

(a) Good performance during earthquake.

Puchuncaví, Chile, earthquake of 8 July 1971

Police post

(a) Framework of light steel sections with concrete fill
(b) Good performance during earthquake.

Huaraz, Peru, earthquake of 31 May 1970

Huaraz Cathedral

(a) Masonry without reinforced-concrete confinement
(b) Total destruction of the towers.

Chimbote, Peru
Earthquake of 31 May 1970

Buenos Aires quarter

(a) Masonry without reinforced-concrete confinement
(b) Total destruction of all the houses.

Chimbote, Peru, earthquake of 31 May 1970

Buenos Aires quarter

(a) Interior view
(b) The staircase acted as a prop.

Casma, Peru
Earthquake of 31 May 1970

Regional Hospital

Total destruction of non-structural partitions

Quinteros, Chile
Earthquake of 8 July 1971

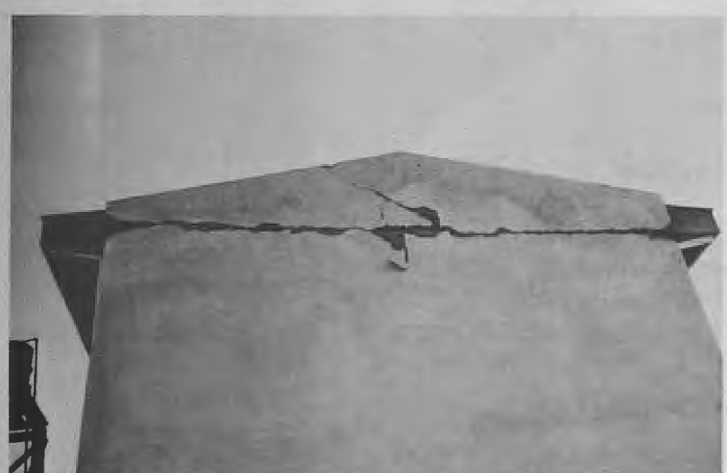
One-storey house

(a) Total failure of the chimney at roof level
(b) No reinforced-concrete confinement.

Illapel, Chile, earthquake of 8 July 1971

House built of cement mortar blocks

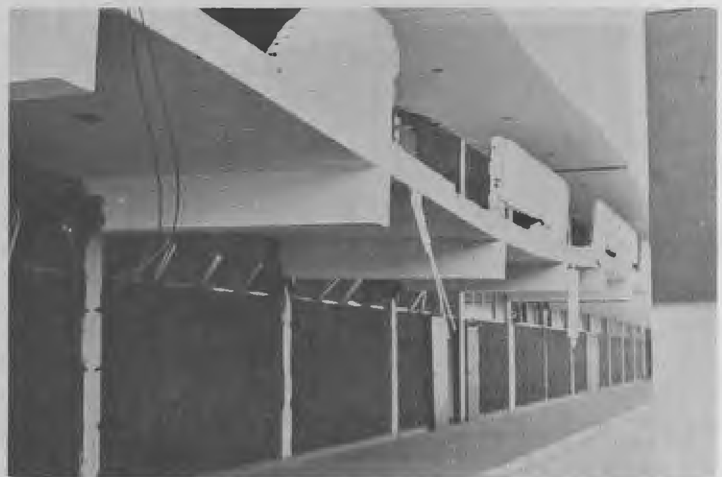
(a) Failure of second-storey walls by shear
(b) Inadequate reinforced-concrete confinement.



Papudo, Chile, earthquake of 8 July 1971

Confined-masonry house

(a) Failure of the masonry end-wall at the level of the tie-beam
(b) Lack of reinforced-concrete confinement.



Chimbote, Peru, earthquake of 31 May 1970

Regional College

(a) Destruction of the columns made rigid by the walls
(b) The free column (between doors) has only plaster failure
(c) Note the sag of the superstructure in the area of the damaged columns

Chimbote, Peru
Earthquake of 31 May 1970

Regional College

(a) Detail of column which failed by shear
(b) Less ductility because of lack of confinement. Very long distance between stirrups. Very small number of stirrups
(c) Intermediate stirrup broken.

Huarmey, Peru, earthquake of 31 May 1970

Municipal market

(a) Columns held rigid between walls have been destroyed
(b) Free columns not severely damaged.

Viña del Mar, Chile, earthquake of 8 July 1971

Tiers of the Sausalito Stadium

(a) Inadequate reinforcement against shear
(b) Large stress values because the columns were held rigid.

Chimbote, Peru
Earthquake of 31 May 1970

Regional centre

(a) No stirrups
(b) Failure by shear.

Huarmey, Peru, earthquake of 31 May 1970

Eaves of a shop

(a) Reinforcement badly placed; no reinforcement in the area subjected to tension
(b) Total collapse of the eaves.

Casma, Peru, earthquake of 31 May 1970

Water tank of the Regional College

(a) Column anchors badly constructed
(b) Total collapse.

Chimbote, Peru, earthquake of 31 May 1970

Beam-column joint

Inadequate anchors.

Casma, Peru, earthquake of 31 May 1970

Regional College

(a) Low-strength concrete
(b) Beams with inadequately anchored reinforcement
(c) Total collapse.

Casma, Peru, earthquake of 31 May 1970

Regional College

(a) No stirrups
(b) Lack of confinement of the concrete.

Part Two

HURRICANES

INTRODUCTION

313. Even though the actual construction of houses has not kept pace with increasing demand, it is a fact that the total volume of housing construction is growing in all parts of the world, including areas periodically affected by hurricanes. Increased population density increases the risk of the loss of life and damage to structures when natural disaster strikes. It is, therefore, particularly important to adopt special precautions and adhere to minimal standards when building housing and other low-cost structures in areas subject to hurricanes.

314. This part of the document identifies hurricane areas and explains the behaviour of buildings under the dynamic forces of strong winds. It also reviews methods and makes recommendations for structural design resistant to hurricanes.

315. In a similar fashion as the scales of intensities adopted for earthquakes (see part one), a damage scale of intensities for hurricanes based on subjective observations is being proposed for the first time. This scale is considered particularly important for those areas still lacking hurricane building codes.

316. The present publication also contains two sets of information on wind loads that are expected to be very practical for design work. One is a listing of shape factors given in the annex as a reference from the South Florida Building Code. The other is a number of tables on velocity pressures and pressure coefficients taking into account the angle of wind and the shape of buildings.

317. Part two, on hurricanes, contains six chapters and one annex. Chapter IX deals with the nature of winds, high velocity winds, frequency and geographical occurrence. Chapter X refers to wind effects on buildings and tidal effects. Chapter XI is devoted to general methods of structural design. Chapter XII is mostly concentrated on velocity pressures for the purpose of structural design. Chapter XIII deals with safety factors and shows construction details of hurricane-resistant structures. Chapter XIV contains a summary of recommendations and examples of structural design calculations for small buildings.

318. Since the purpose of this study is to improve low-cost housing construction, the text and information have been carefully selected and considerable weight has been assigned to economic considerations.

This part of the study has been prepared for the United Nations by Herbert Saffir, Consulting Engineer, Coral Gables, Florida, United States of America.

IX. WIND LOADS, ESPECIALLY HIGH VELOCITY WINDS

A. Definition of wind: importance in building design; hurricane winds

319. Winds are by definition air in motion relative to the surface of the earth, and they are caused primarily by differences of atmospheric pressure and temperature.

320. Obviously, one of the prime functions of a building is to provide shelter from the weather and from wind; therefore, wind loads constitute one of the series of loadings and forces which an architect and structural engineer must consider in their design. (Other major loads consist of the dead load of the structure, the live load due to floor loadings and other superimposed loads, impact loads, earth pressure loads, temperature loads, and earthquake loads.) Generally, wind loads are considered to be secondary loads and do not have the importance of dead load and floor live loads in the structural design of buildings. Higher working stresses are permitted for wind loads, and, in many cases, wind loads are ignored entirely for smaller buildings.

321. However, experience gathered after almost a dozen hurricanes shows that they are not phenomena of secondary importance. Loadings caused by hurricanes on buildings and other structures are not secondary loads and although equivalent static loads are generally used for wind design, the turbulence and gusting of hurricane winds are certainly not static loadings. Until a structural engineer has spent several hours personally enduring a hurricane, with the high noise level of the winds approaching the noise of a jet aircraft, and with solid objects, both large and small, flying through the air, he is not really aware of the destructive forces that a hurricane contains. There is a tremendous difference between the effects of windstorms used for design loads in Chicago, or London or Paris, and the effects of hurricane wind loads on structures in Miami, or Nassau, or Guam, during a hurricane. In terms of design loads, the equivalent loads caused by a 210 kilometre per hour (kmph) hurricane will be 69 per cent greater than the effects of a 160 kmph hurricane. The hurricane winds will search and find every flaw of construction in a building.

B. High velocity winds caused by hurricanes

322. Hurricanes are defined as storms of tropical origin, with rotating cyclonic winds of 120 kmph or higher. Tropical storms are defined as storms of tropical origin, with closed circulation winds of 65 kmph or greater. These may develop into hurricanes. Hurricane is the name given this type of storm in the north Atlantic area. The same type of storm in the Pacific Ocean is termed a typhoon, and in the Indian Ocean, a tropical cyclone. Hurricanes are not to be confused with tornadoes, inverted cloud cores consisting of a violently rotating column of air. These are local in area and may develop wind velocities of from 160 kmph to 480 kmph.

323. At the eye of this closed storm, the surface pressure is very low, but rises rapidly toward the periphery. Winds are of high velocity, because of this large pressure gradient, and they blow counterclockwise in the northern hemisphere and clockwise in the southern hemisphere.

324. Riehl breaks down the life span of a hurricane (or typhoon) in his authoritative Tropical Meteorology, 1/ into the following four stages, with an over-all life of from a few hours to two weeks:

Formative stage, in which a pre-existing disturbance can develop into a closed circulation storm with a well-defined eye within 12 hours;

Immature stage. Some storms may die within this period even though winds attain hurricane force; other storms may travel long distances with only a small area of storm involved;

Mature stage, in which maximum winds no longer increase, but the circulation expands and the radius of winds of hurricane force may reach 320 kmph. Symmetry of the storm is lost;

Decaying stage. As a hurricane recurves from the tropics and enters the westerlies, the size usually decreases. However, not all hurricanes recurve; some die over the tropical oceans.

325. The maximum velocity for hurricane winds is unknown, but gusts in extremely severe hurricanes have reached more than 320 kmph. (Camille, 1969; Florida Keys, 1935.) The United States Weather Bureau recorded winds of 290 kmph in Puerto Rico on 13 September 1928. 2/

326. In considering housing design in hurricane regions, wind loadings must be considered of more than just secondary importance. If the structure is to be built to withstand hurricane loads, more consideration must be given to design for wind than is generally the case in areas where hurricanes do not occur. In general, (except for special wind sensitive structures), structural engineers convert the kinetic energies of winds into equivalent static loadings for structural design purposes; these methods of design will be discussed in detail later.

C. Regions where hurricanes occur; probabilities of occurrence

327. Hurricanes develop in the following areas:

The southern portions of the north Atlantic Ocean, including the Caribbean Sea and the Gulf of Mexico; the south-west portion of the north Pacific Ocean; the north Pacific Ocean off the west coast of Mexico; the north Indian Ocean in the Bay of Bengal; the north Indian Ocean in the Arabian Sea; the south Indian Ocean from Madagascar eastward; the south Indian Ocean - north-west Australia and the south Pacific Ocean. 3/

1/ H. Riehl, Tropical Meteorology (New York, McGraw Hill, 1954).

2/ United States of America, National Weather Service, N.O.A.A., various velocity data and reports on past hurricanes.

3/ Gordon E. Dunn and Banner Miller, Atlantic Hurricanes (Baton Rouge, Louisiana State University Press, 1960).

328. In the decaying stages, the storm can recurve from the tropics and enter more northerly areas, such as Japan, or the New England areas of the United States.

329. The average yearly occurrence of tropical storms on a global basis is plotted up as shown in figure 50: (adapted from tables given by Dunn and Miller). 3/

330. Gray shows the climatology of tropical-storm and hurricane formation by 5-degree latitude-longitude grid areas, as modified in figure 51. 4/

331. In figures 52 and 53, the passage of tropical storms across areas of the north Atlantic and north Pacific, are indicated, across 5-degree latitude-longitude grid areas. These are adapted from Thom's studies. 5/ 6/ Figure 54 shows the passage of tropical storms across the Bay of Bengal in the Indian Ocean, also adapted from Thom's studies.

332. It should be borne in mind that many tropical storms went undetected in the past. However, since the launching of Tiros I, the first meteorological satellite, in April 1960, it has been possible to monitor weather conditions on a regular basis throughout the world. Before the use of meteorological satellites, many off-shore tropical storms, away from shipping lanes, were never recorded.

4/ William Gray, "Global view of the Origin of Tropical Disturbances and Storms", Atmospheric Science Paper 114, Colorado State University, October 1967.

5/ H. C. S. Thom, private communication on engineering, climatology of wind speed with special reference to the Pacific Area.

6/ H. C. S. Thom, Toward a Universal Climatological Extreme Wind Distribution, International Research Seminar on Wind Effects on Buildings and Structures, Ottawa, Canada, September 1968.

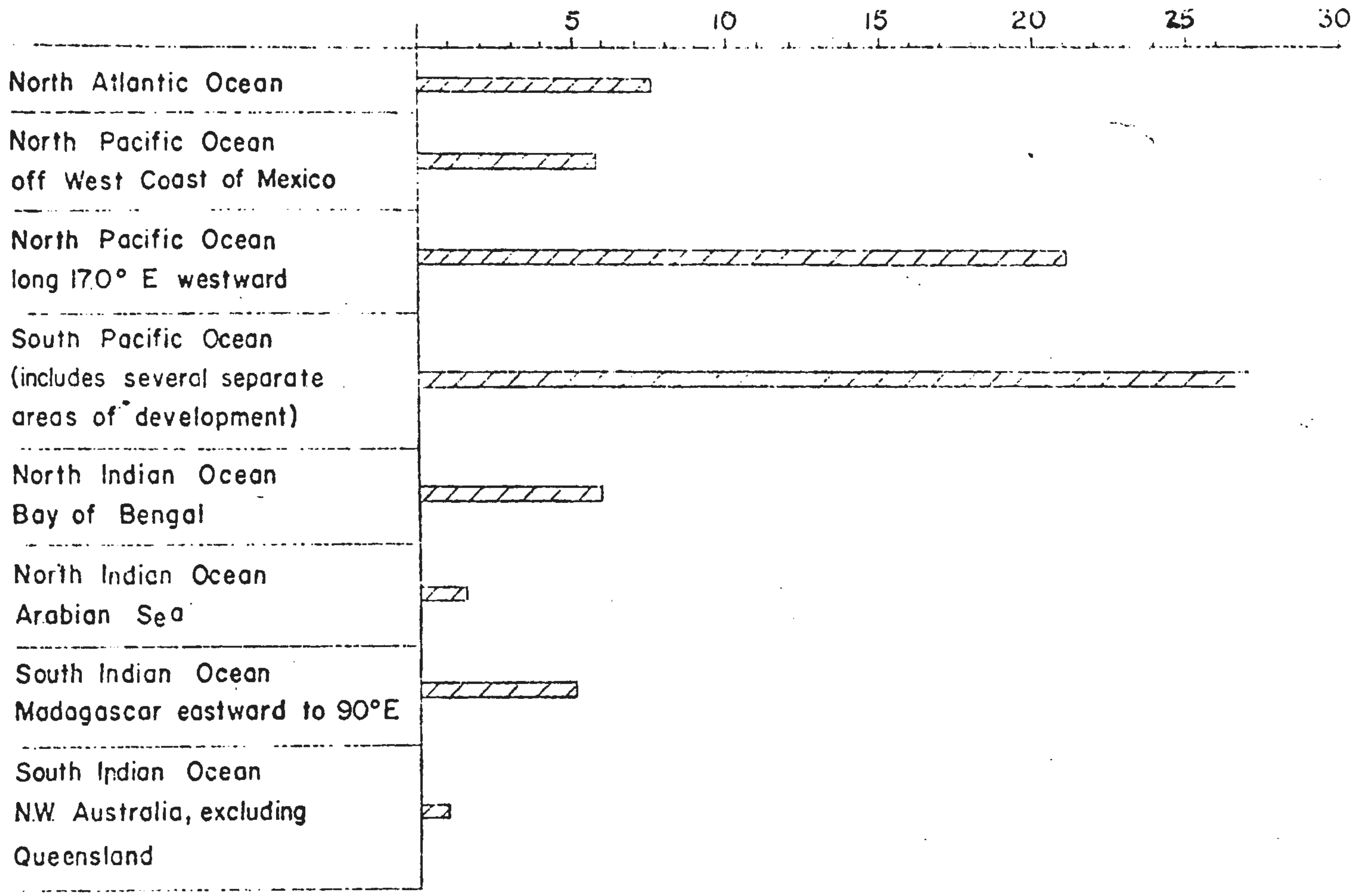


Fig. 50. Average frequency of occurrence of tropical storms per year

FORMATION OF TROPICAL STORMS AND HURRICANES — GLOBAL FREQUENCY
(modified from Gray (3))

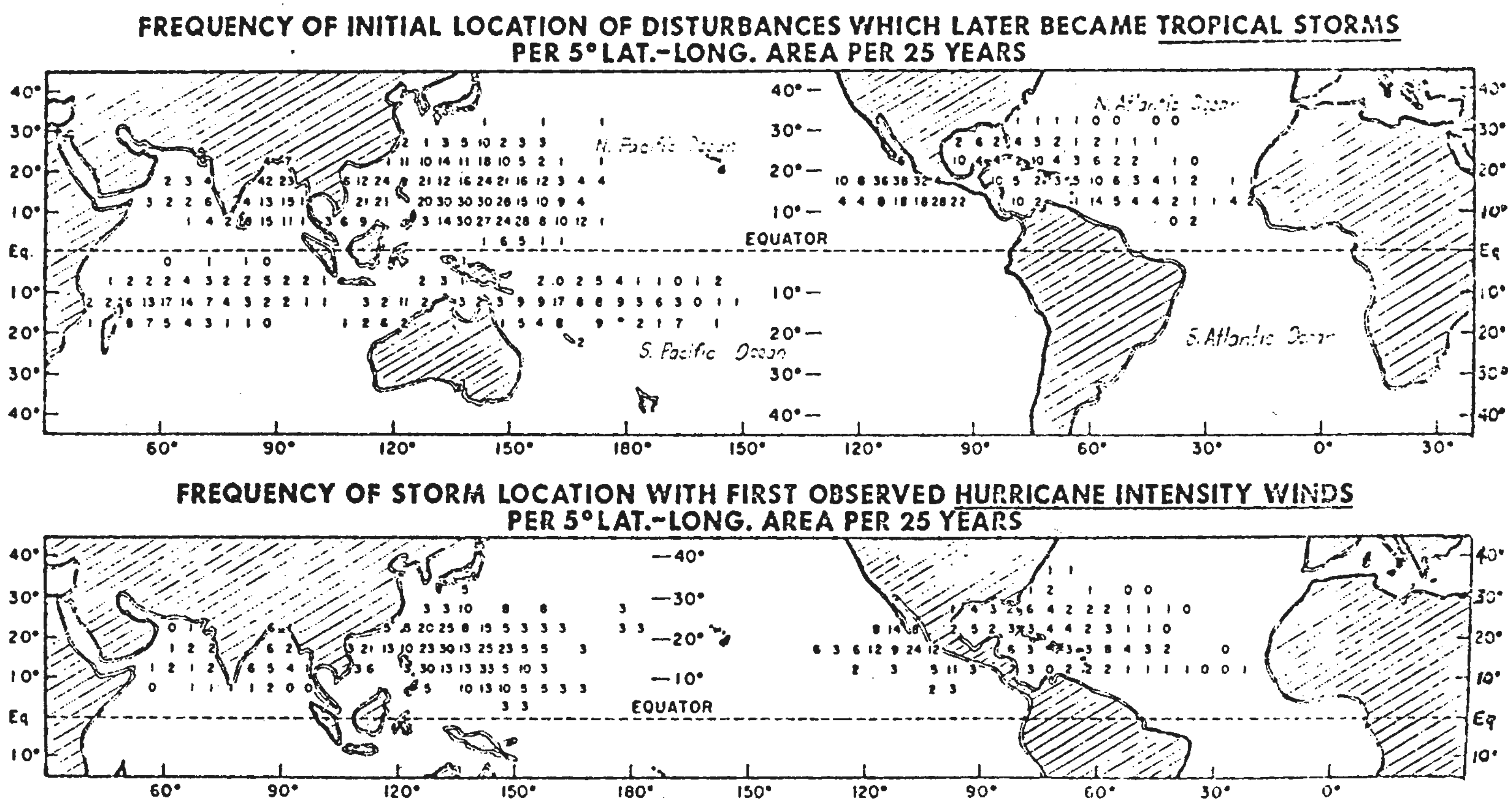


Fig. 51. Formation of tropical storms and hurricanes — global frequency

Source: Modified from Gray (see foot-note 4).

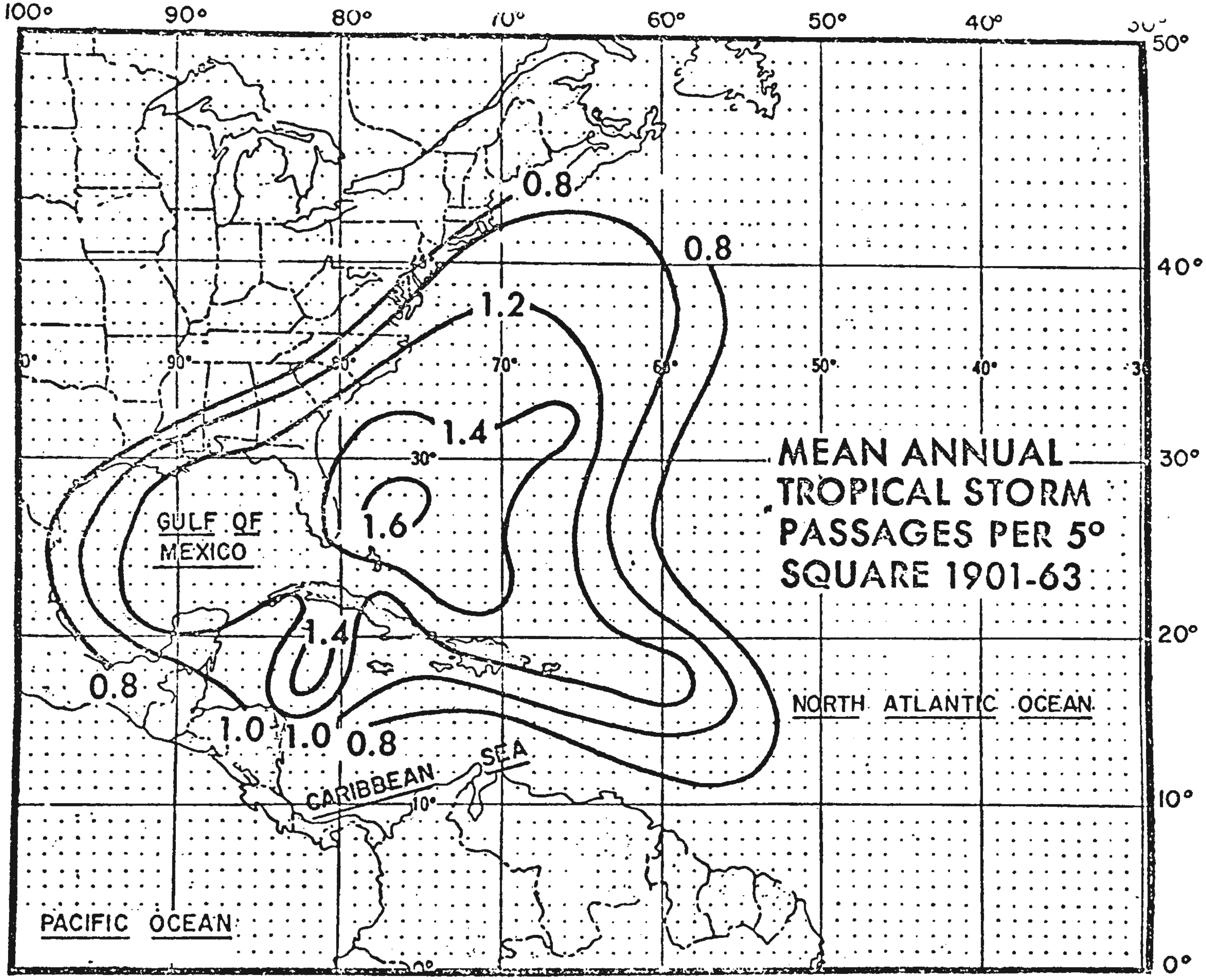


Fig. 52. Mean annual tropical storm passages per 5° square 1901-1963, Caribbean Sea

Source: After Thom (see foot-note 5).

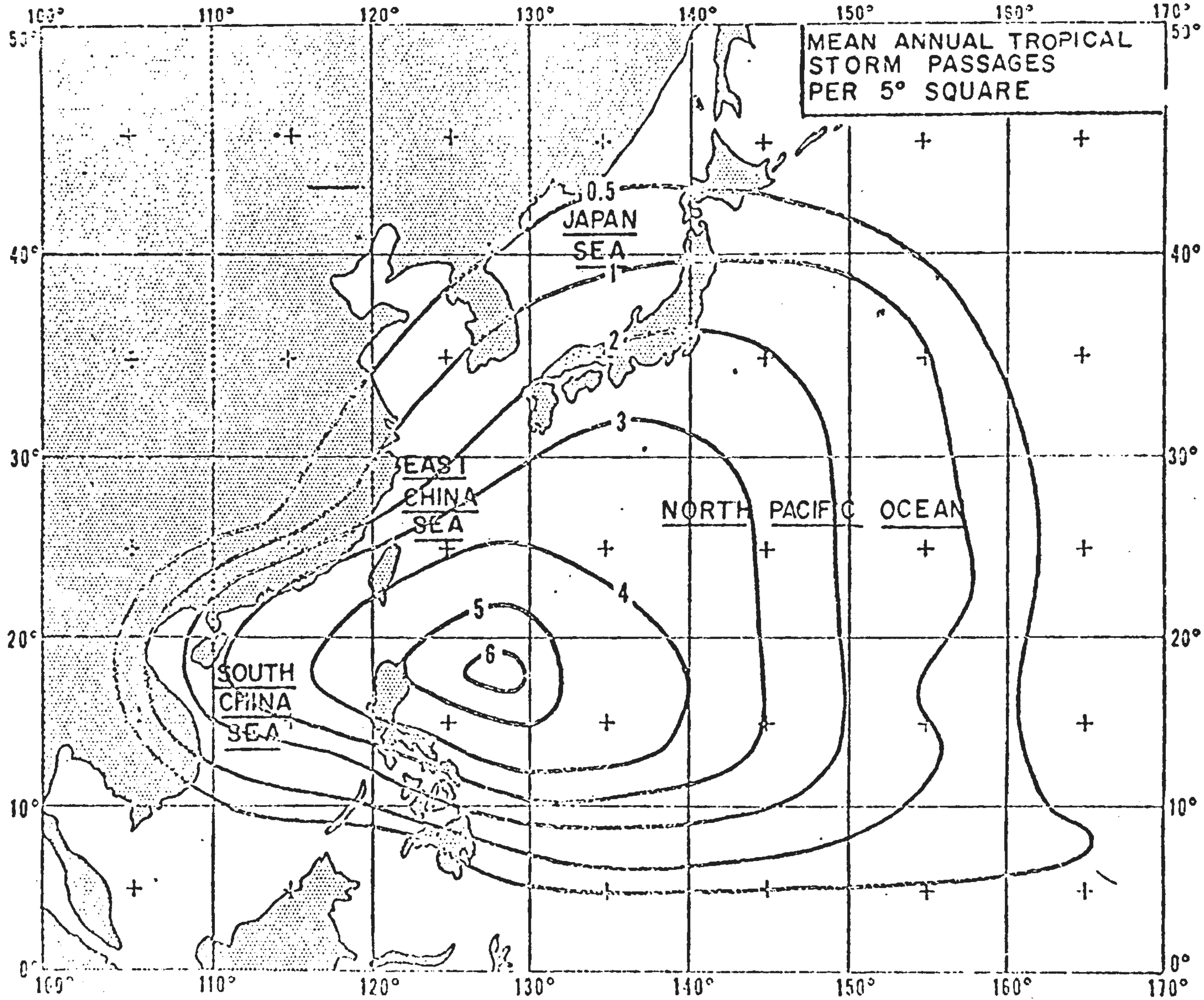


Fig. 53. Mean annual tropical storm passages per 5° square, North Pacific Ocean

Source: After Thom (see foot-note 4).

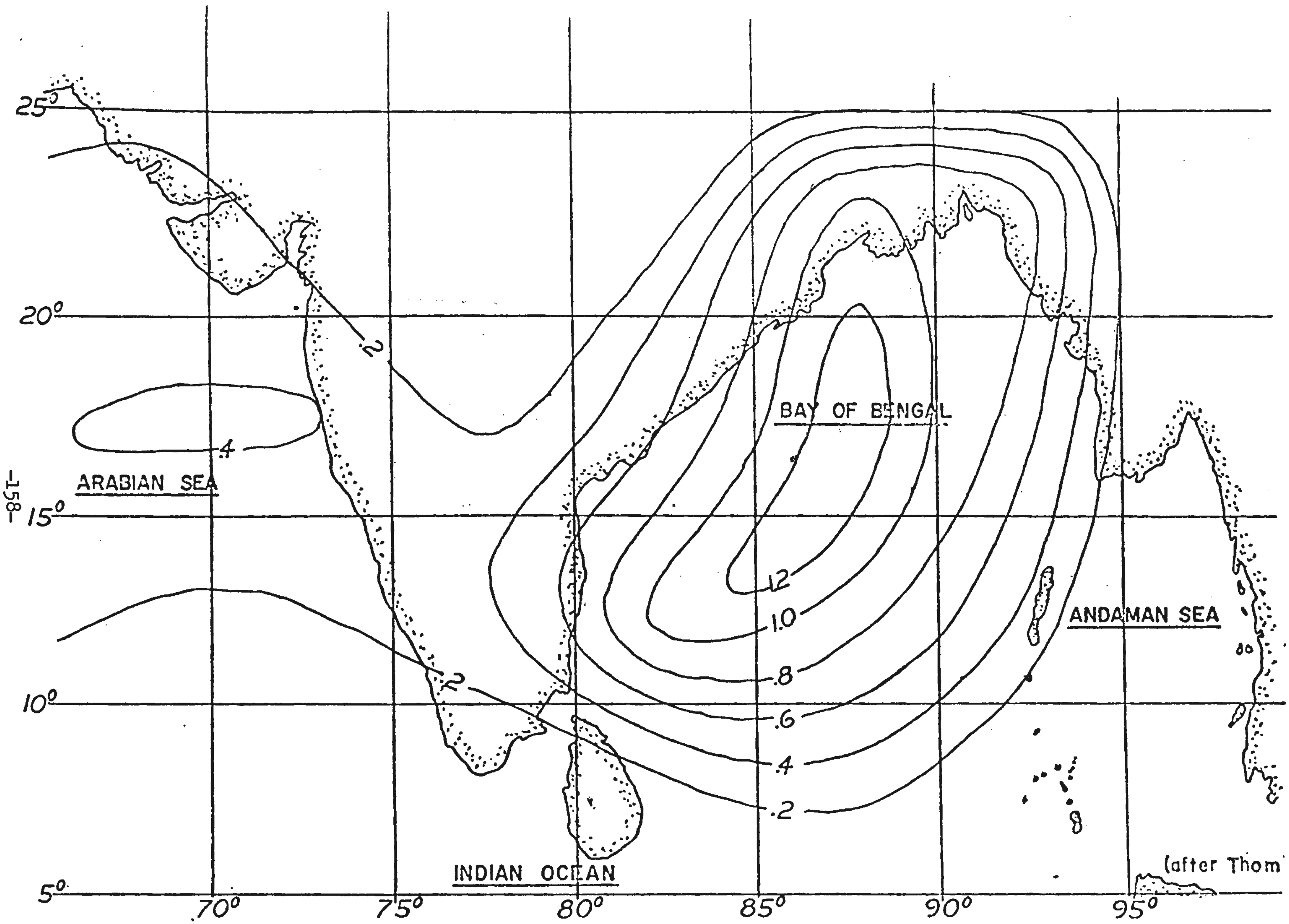


Fig. 54. Mean annual tropical storm passages per 5° square, Indian Ocean

X. EFFECTS OF HURRICANES ON BUILDINGS AND OTHER STRUCTURES

A. Effects due to hurricane winds in general

333. The effects of hurricane winds on structures have been documented over the past 20 years, especially in the south Florida coastal areas and areas exposed along the Gulf of Mexico in the southern United States. 7/

334. Hurricanes in the south Florida area prior to about 1950 caused considerable structural damage. In the September 1926 hurricane - where fastest-mile velocities may not have exceeded 190 kmph - extensive structural damage occurred in the Miami area. Deformation of the steel superstructure in several high rise office buildings in the downtown area was so extensive that the "high rise" portion of these buildings had to be dismantled and removed. Smaller building damage was wide-spread in this storm and many roof structures were lost completely. 8/ The design criteria and codes in force at that time in the south Florida area did not take cognizance of hurricane requirements; damage was so extensive that it resulted in a near economic catastrophe for the south Florida area.

335. Subsequent storms reaching the south Florida area repeated their catastrophic blows in 1928, 1935, 1945, 1947, 1948, 1950 and 1960. These severe hurricanes caused considerable structural damage in south Florida. There was no repetition of the high-rise steel frame failures of 1926; however, there were many structural failures, especially failures where structural roofs were blown off, and failures of windows and doors.

B. Proposed scale of intensities based on subjective observations

336. In order to provide some means of comparing effects of hurricanes of various intensities in different parts of the globe, the following damage scale is proposed as a guide in coastal areas subjected to hurricanes that do not have a hurricane building code; no scales of this type exist for wind storms or hurricanes, although similar scales of intensities are used for earthquakes.

GRADE 1. 120 mph to 140 kmph velocity, in 2- or 3-second gusts, at 9.20 m elevation. Some damage to shrubbery, trees, foliage. No real damage to building structures. Some damage to poorly constructed signs.

7/ Herbert S. Saffir, "The effects on structures of winds of hurricane force", American Society of Civil Engineers, Proceedings, vol. 79 (1953), separate No. 206; _______, "Lessons learned from Hurricane Donna", Civil Engineering, March 1961; _______, "Hurricane Betsy in South Florida", Civil Engineering, August 1966; _______, "Hurricane exposes structural flaws", Civil Engineering, February 1971; _______, "Hurricane Camille - data on storm and structural damage", paper presented at the Third International Research Conference on Wind Effects on Buildings and Structures, Tokyo, 1971.

8/ Herbert S. Saffir, "The effects on structures of winds of hurricane force", American Society of Civil Engineers, Proceedings, vol. 79 (1953), separate No. 206.

GRADE 2. 151 to 180 kmph velocity in 2- or 3- second gusts, at 9.20 m elevation. Considerable damage to shrubbery and tree foliage. Some trees blown down. Extensive damage to poorly constructed signs. Some roofing material damage to buildings; some window and door damage; no major damage to building structures.

GRADE 3. 181 to 210 kmph velocity, in 2- or 3- second gusts, at 9.20 m elevation. Extensive damage to shrubbery and trees. Foliage off trees; large trees blown down. Practically all poorly constructed signs blown down. Some roofing material damage; some window and door damage; some structural damage to small residences and utility buildings. Minor amount of curtainwall failures.

GRADE 4. 211 to 240 kmph velocity, in 2- or 3- second gusts, at 9.20 m elevation. Shrubs and trees down. All signs down. Extensive roofing material damage; extensive window and door damage; complete failure of roof structures on many small residences. Some curtainwall failures.

GRADE 5. 241 kmph and higher velocity, in 2- or 3- second gusts at 9.20 m elevation. Shrubs and trees down. Roofing damage considerable. All signs down. Very severe and extensive window and door damage. Complete failure of roof structures on many small residences and industrial buildings. Extensive curtainwall failure and sidewall failure on industrial buildings. Extensive glass failure. Some complete building failures; small buildings overturned and blown over or away.

C. Tidal effects on buildings, due to high water and tidal surge

337. A storm surge along the coast, during a hurricane, will have considerable effect on structures erected on or along the coast line. Factors involved in causing storm surge are:

(a) The effect of the low barometric pressure on the sea surface, during the hurricane;

(b) The rise in water along the shore produced by onshore hurricane winds;

(c) The normal astronomical tide.

338. The maximum effect of the low barometric pressure may only be 0.90 m or 1.20 m at the most; effects of the onshore winds and the normal tides may be much higher.

339. In Hurricane Camille, along the Mississippi coast in 1969, the highest tidal surge (based on debris elevations) was recorded as about 7.50 m above mean Gulf of Mexico levels. Much of the coastal area was at elevations of 4.50 m above Gulf level, and only some portions of the coast, in the Pass Christian area, where ground elevations were only 2.00 m or 2.40 m above water, felt the worst effects of the tidal surge. 9/

9/ H. Saffir, "Hurricane Camille Data on Storm and Structural Damage", paper presented at Third International Research Conference on Wind Effects on Buildings and Structures, Tokyo, Japan, 1971.

340. In Hurricane Celia, which hit the Texas coast near Corpus Christi in August 1970 with gust velocities of at least 250 kmph, south of the eye of the hurricane in Corpus Christi Bay, debris elevations observed after passage of the storm were about only 1.20 m to 1.55 m above normal water level; north of the passage of the eye, high water mark elevations were 3.00 m above normal water level. This is an indication of the effect of the rise in water produced by onshore hurricane winds.

341. In the tropical cyclone (same as hurricane, typhoon) that reached East Pakistan in November 1970, a high tidal surge combined with low land elevations was responsible for much of the tremendous loss of life in the offshore island areas. There, apparently all three factors indicated above - including high astronomical tides - combined to cause a high tidal surge.

D. Behaviour of buildings under hurricane winds

342. In considering the behaviour of buildings under wind, the stability of the structure as a whole and the strength of the component parts of the building are taken into account. The structure must be restrained from overturning or sliding when subjected to lateral forces.

343. The component parts of the building such as the sidewalls, or windows, or roof anchorages, must also be strong enough to withstand the forces of the wind. The failure of component parts can render the remainder of the building unsafe, even if the building frame is still in good condition.

344. In studying wind forces on buildings, it should be understood that all wind forces are dynamic, in the sense that they are effects of a moving fluid. However, in order to simplify design, the loads are assumed to be equivalent static loads. The dynamic characteristics of the wind may be neglected, except for certain types of structures such as free-standing towers, vertical smoke-stacks, suspension bridges, and other wind-sensitive structures.

345. To obtain equivalent static loads for use in structural engineering, four items are considered:

(a) The kinetic energy that the fluid (air) contains by virtue of its movement. This is the velocity-pressure relationship and depends only on the speed of the fluid and its density. It may be expressed as $0.00256V^2$, where V is the speed in miles per hour, for air at ground level.

(b) The shape and dimensions of the building (the obstruction in this moving fluid) influences the resulting equivalent static pressure. For example, a cylindrical object placed in a moving fluid will have less total force or drag exerted on it than on a rectangular object. For a fully enclosed building, the shape factor on the windward face (depending on the dimensional ratios of the building and assuming no interior leakage) may be 0.9. For a flat sign with no appreciable depth, this shape factor for the entire sign may be 1.4. Wind tunnel tests on various building shapes and full scale observations are required for the evaluation of these factors.

(c) The height-velocity relationship. This describes the effect of surface roughness on the rate of increase of mean velocity with height. Data available indicates that wind velocities above ground will increase by a ratio of the height above ground related to the height at 9.15 m raised to an exponential power. Experimental data obtained at various observational towers show that this exponential power may vary, for example, from $\frac{1}{4.5}$ for readings taken in wooded areas to $\frac{1}{7}$ for flat, open, exposed country.

(d) Gusts. A gust is a basic wind characteristic. It is a sudden brief increase in the speed, set up by some surface obstacle, by eddies, or by thrusts of more rapidly moving air from the free atmosphere.

Because of the uncertainty of the structure of gusts and the lack of data indicating their effect on structures, tables of design loads using a gust factor of 1.00 are provided here. This is consistent with low-cost housing.

346. However, for structures of more importance than low-rise housing, parts and portions of these more important structures such as windows, girts, purlins, curtainwall sections etc., should use the minimum design loads given in these tables multiplied by a gust factor of 1.25. This is a realistic figure that takes into account the effect of wind over the entire building and the fact that gusts will affect only small portions of the building at one time. (For wind sensitive structures subject to instability due to flutter or vibration, a more rigorous dynamic approach should be made to the action of gusts.)

(e) Finally, combining these factors, we can arrive at an equivalent static design load by the following equation:

$$P = (0.00256) \times (\text{fastest kilometre velocity at height in question})^2 \times (\text{gust factor}) \times (\text{shape factor for building or component part}).$$

P is in kg/sq m, V is in kmph.

The fastest kilometre velocity is based on a coefficient equal to:

$$\frac{(H)^{1/7}}{(H_{30})}, \text{ or } \frac{(H)^{1/4.5}}{(H_{30})}$$

H is the height in metres above grade. The fastest kilometre speed at 9.20 m elevation is multiplied by this coefficient.

A gust factor of 1.00, except for small areas of cladding, which case, use a gust factor of 1.25, where the tributary area is less than 20 sq m, only for buildings of importance.

E. Behaviour of small one-storey residential buildings under hurricane winds

347. Figures 55, 56, 57, 58 and 59 show the direction of wind forces acting on sidewalls and gable and flat roof structures. Note the upward direction and magnitude of arrows on the roof, especially near the leading corner, indicating very high uplift forces with the wind blowing at the 45° angle shown.

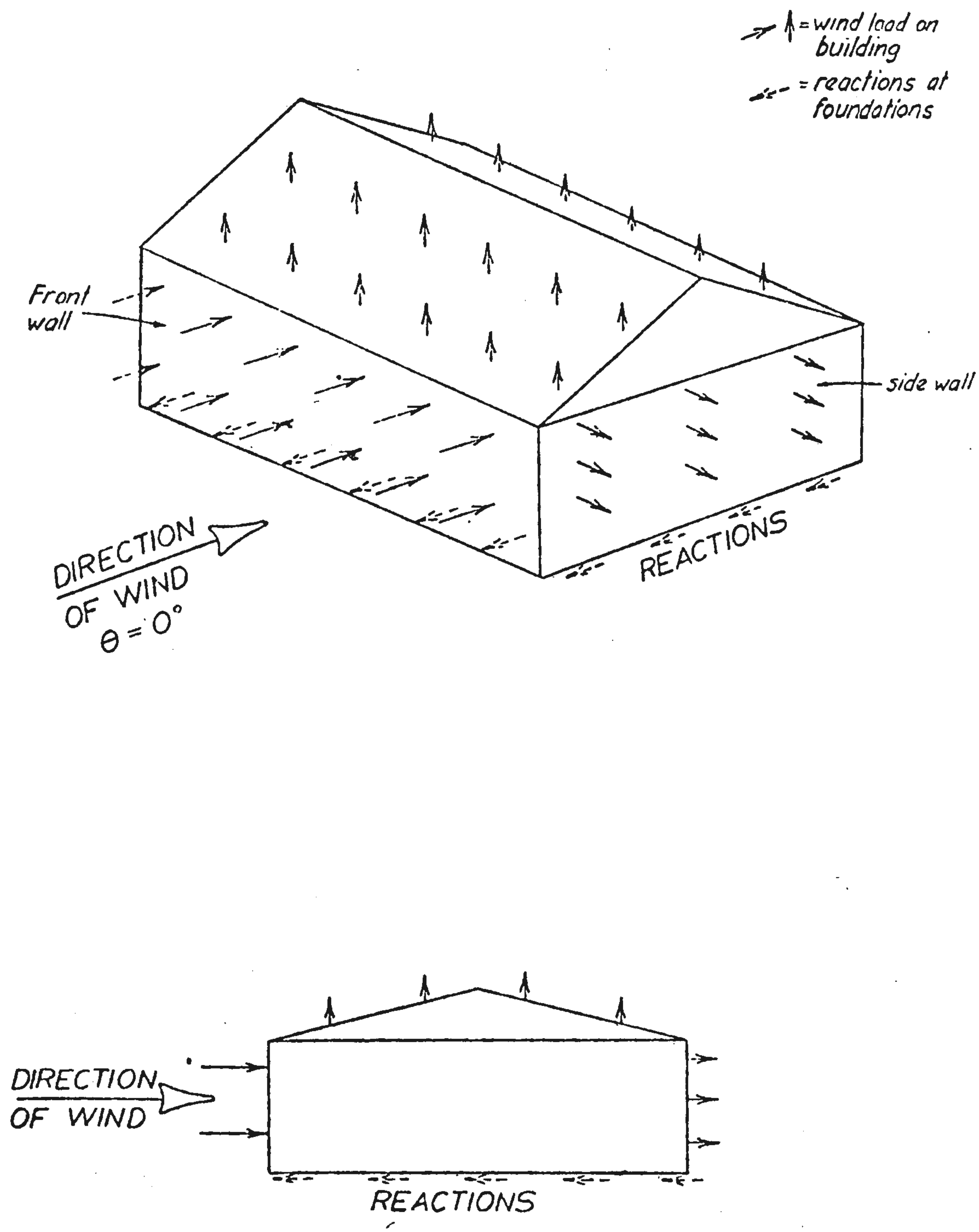


Fig. 55. Schematic wind-load action on a gable-roofed building

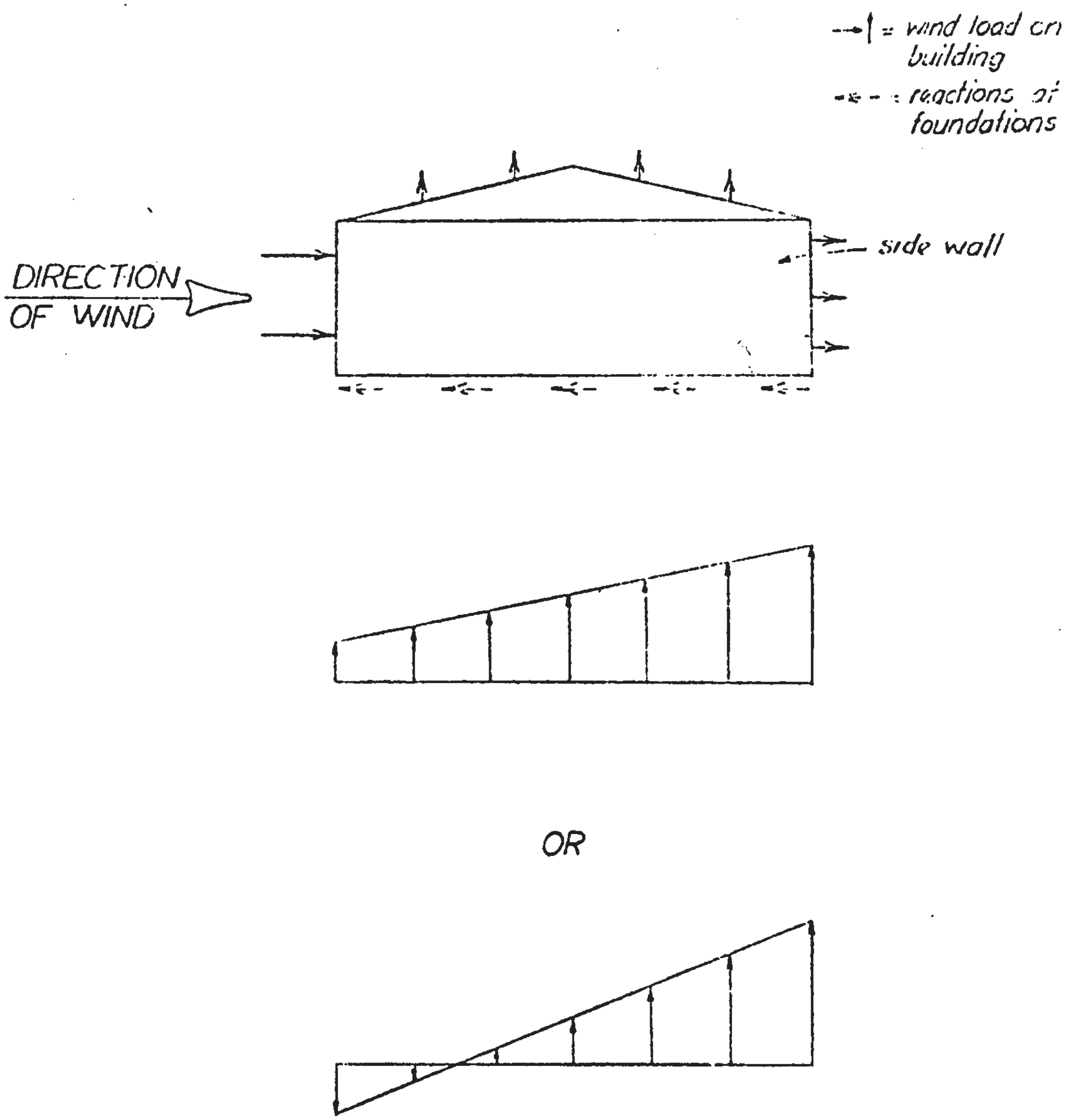


Fig. 56. Stress diagrams at foundation under sidewall

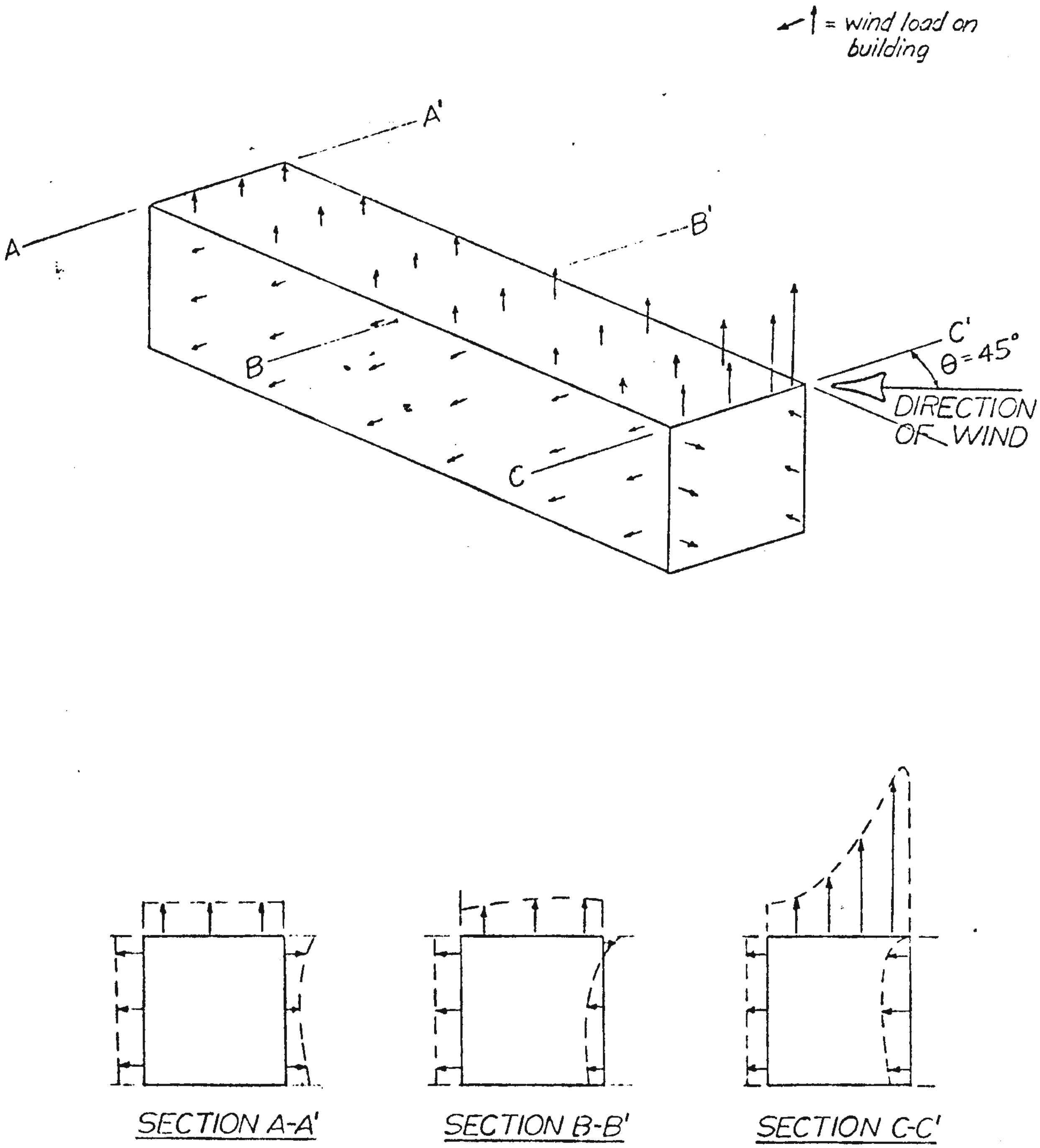
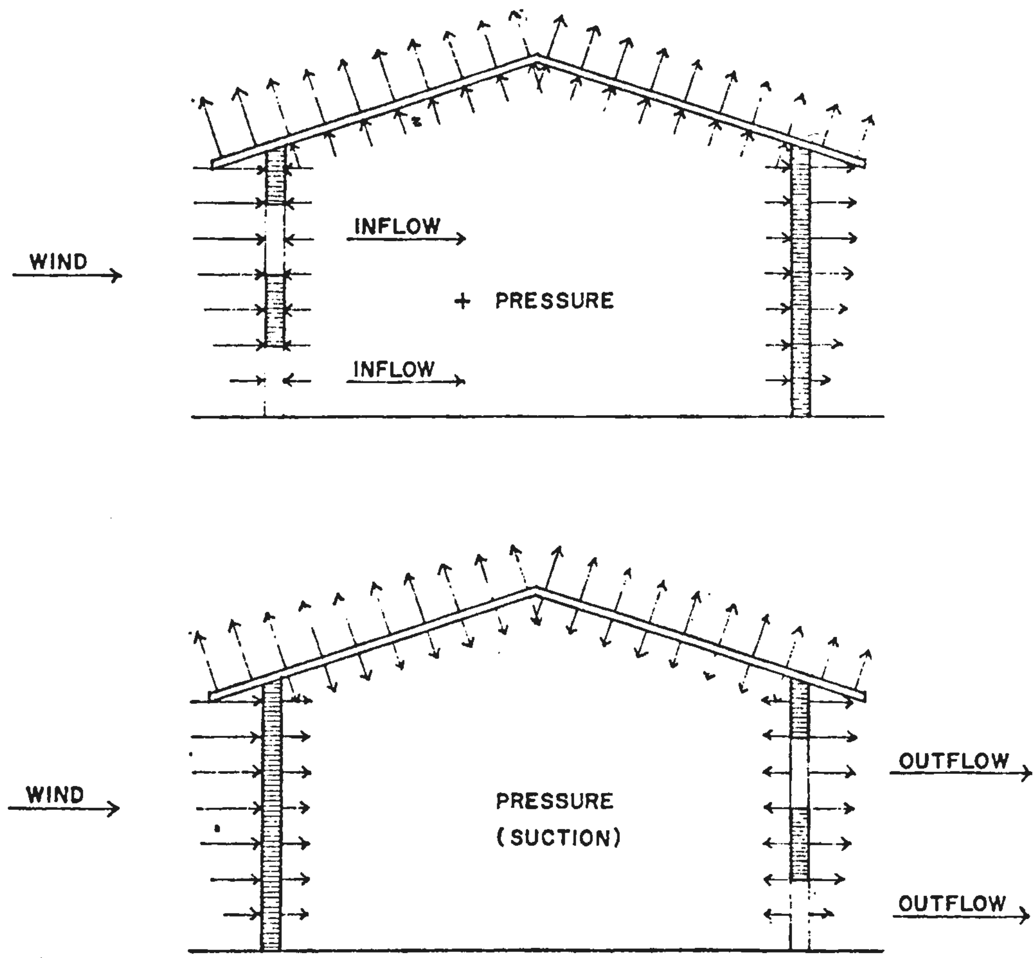


Fig. 57 Schematic wind-load action on a flat-roofed building

DESIGN WIND LOADS MAY BE OBTAINED BY ADDING VECTOR SUM OF FORCES ACTING ON ELEMENTS.
FORCE = VELOCITY PRESSURE X PRESSURE COEFFICIENT.



NOTE: INTERNAL PRESSURES DEPEND ON LOCATION OF OPENINGS, WINDWARD OR LEEWARD WALLS.

Fig. 58. Schematic wind-load action on a building

NATURE OF WIND PRESSURES ON A SINGLE STOREY HOUSE

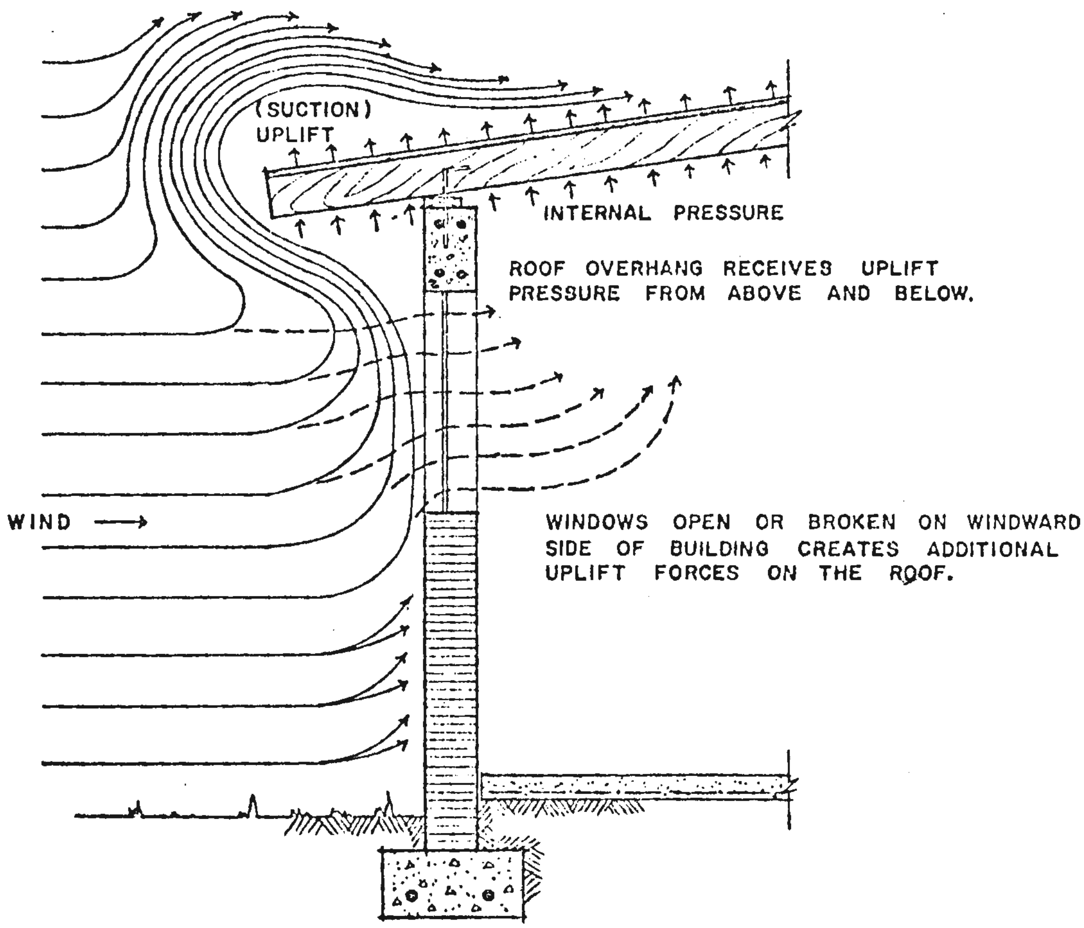


Fig. 59. Nature of wind pressures on a single-storey house

XI. GENERALLY USED METHODS OF STRUCTURAL DESIGN FOR BUILDINGS TO WITHSTAND HURRICANE DAMAGE

A. Establishment of criteria for building design

348. Structural design must consider the dead load of the structure, live load, impact load, wind load, earth pressure loads, temperature loads and earthquake loads. However, only wind loads will be dealt with here. In general, all loads are applied to the structure in more or less similar fashion, with the resulting stress analysis and structural design proceeding along similar lines; however, for loadings other than the primary dead and live loads, increased working stresses are permitted because of the infrequent occurrence of the secondary loads.

B. Commonly used building materials and design standards

349. Generally, standard design methods are used and adopted on a nation-wide basis for different construction materials. This is true where different local and regional building codes generally use the same engineering design standards for various construction materials. Design standards specify allowable working stresses based on the importance of the component being designed and based on yield strength or the ultimate capacity of the member and structure.

350. Some of the generally used construction material design standards are as listed below.

Concrete (reinforced and pre-stressed)

American Concrete Institute Building Code Standard, 318-71.

The Structural Use of Reinforced Concrete in Buildings, British Code of Practice, CP 114.

The Structural Use of Prestressed Concrete, British Code of Practice, CP 115.

The Structural Use of Prestressed Concrete, British Code of Practice, CP 116.

Structural steel

Specification for Design, Fabrication and Erection of Structural Steel for Building, American Institute of Steel Construction, United States of America, 1970.

The Use of Structural Steel in Building, British Code of Practice, CP 449.

Aluminium

Specifications for Structures of Aluminum Alloys, Aluminum Association, United States of America.

Timber

Timber Construction Standards, American Institute of Timber Construction, United States of America.

The Structural Use of Timber, British Code of Practice, CP 112.

C. Building codes

351. Some of the building codes in use that take into account hurricane force winds on buildings and structures are listed below. These codes range from "specification" type codes which specify and detail every operation, to "performance" type codes which give the desired end product or loading the structure must withstand, but do not tell how it is to be accomplished.

352. The Uniform Building Code of the International Conference of Building Officials, United States of America. This is a regional "model" code in use in many areas of the United States. On a map of the United States that includes a table of wind loads (chap. 23), the code sets forth wind pressures for areas such as California, where a minimum basic design wind pressure of 10 psf is used, to south Florida and some regions on the Gulf of Mexico, where a maximum basic design wind pressure of 50 psf is used, increasing to a 100 psf maximum for increasing heights. Slight mention is made of shape factors for wind loads and design. As far as this writer knows, this code - although calling for high wind loadings in hurricane-prone areas - is not in use in those hurricane-prone areas, and is used mainly on the west coast of the United States where wind speeds are low.

353. The National Building Code of the American Insurance Association, United States of America. This is a minimal type code which has been prepared by various insurance companies for use or adoption by municipalities. It gives minimum requirements for severe wind conditions, both by specifying design wind loads and by construction specifications. Although not sufficient for maximum hurricane conditions, in the writer's opinion, it is fair for moderately severe windstorm exposures.

354. The South Florida Building Code, Dade County and Miami, and several other counties in southern Florida in the United States of America. This is a code which has been based on the Uniform Building Code of the International Conference of Building Officials, mentioned above, but has diverged to some extent to provide more stringent requirements for hurricane resistant construction. Wind loadings are fairly realistic, and a basic design velocity of 190 kmph at 9.20 m elevation is used. The chapter on wind loads includes shape factors and at the time of its preparation it pioneered the use of shape factors for use with wind loadings, in United States building codes.

355. The Bahamas Building Code is now under preparation for the Bahama Islands, where probabilities of tropical storm passage exceed probabilities in the

United States. It is based in part on the South Florida Code and in turn on the Uniform Building Code, but it has important variations as far as wind loadings and specification requirements are concerned.

356. The Japanese Building Code. Standards have been established for design loads, working stresses etc., for use throughout Japan; both seismic and wind load provisions are given in these standards. Wind pressures are given with increased intensity with height and either stepped wind loads or velocity pressures based on $q = \sqrt{H}$ may be used, with design wind loads equal to a wind pressure coefficient multiplied by the velocity pressure ($P = cq$). Fairly extensive use of shape factors is provided. Provisions are made for the reduction of wind loads in areas away from the sea, or in areas where typhoon probabilities are lower.

357. Southern Standard Building Code of the Southern Building Code Congress. This is a regional model building code developed for use in the southern states of the United States of America, including areas bordering on the Gulf of Mexico. It is more of a performance type code than a specification type code. The code gives wind loadings for coastal regions subject to hurricanes, defined as those areas lying within 200 kms of the coast. In the writer's opinion, these loadings are not high enough for hurricanes with 190 kmph wind speeds at 9.20 m elevation. Some detailed construction specifications are given for hurricane requirements, but these are not strong enough for adequate resistance to a severe hurricane.

358. There are other very good building codes and design standards in use throughout the world; this writer feels, however, that to provide hurricane-resistant design, more is required than merely extending design loads to cover hurricane wind loadings; attention also must be given to details of design and construction.

359. The building code itself is a legal document for the public safety and welfare. Various nationally accepted design standards covering methods of design may be incorporated into the building code, and local factors such as the climate or the existence of high velocity winds must be specified in the building code itself.

360. Note that a building code is generally a minimum standard and will not cover every eventuality; also, without enforcement of the building code provisions, the adoption of the code is valueless.

361. Generally, secondary materials used for cladding (the "skin" of the building), such as windows, glass, composition board and cement-asbestos materials are not standardized. It is necessary to adopt some basic standard or design criterion based on the strength of the material, with consideration given to a safety factor based on yield or ultimate strengths, and with some consideration given to deformation or deflection if these factors are important to the structural integrity of the building.

D. Foundation design

362. The basic requirements of foundation design are covered in standards such as the American Concrete Institute Standard, 318-71, mentioned above, various foundation requirements for buildings given in building codes, standards such as the American Standard Building Code Requirements for Excavation and Foundations etc.

363. In general, for construction in hurricane-prone areas, consideration must be given to (a) the possibility of scour under foundations in exposed areas on the shoreline and (b) the possibility of uplift or overturning of buildings. The foundation must be considered as a means of resisting the wind loads on the structure, and wind forces must be carried through the foundation into the ground. Without proper consideration for tying the structure down and carrying these wind loads into the ground, the design for hurricane-resistant construction is not complete.

XII. RECOMMENDATIONS FOR HURRICANE RESISTANT DESIGN AND CONSTRUCTION: WIND LOADINGS

A. Probability of high wind occurrence

364. The most important item to take into account in establishing wind loadings is a consideration of records taken in a given area. Most of the climatological records pertaining to hurricanes throughout the world are very limited; much of the older data has no engineering value.

365. Charts of fastest-mile speeds have been established for the United States based on available climatological data, and have been prorated down to heights of 9.20 m above ground. Extraordinary conditions such as tornadoes have not been considered in developing these standards.

366. As an example of these velocities, several hurricane-prone localities in the United States are listed below with the highest velocities recorded at 9.20 m above ground. The figures do not include wind gusts.

(Kilometres per hour)*

Area	25-year mean recurrence interval	50-year mean recurrence interval	100-year mean recurrence interval
South Florida, south of Miami	160	180	210
Louisiana, south of New Orleans	130	160	180
Coast of Mississippi, on Gulf of Mexico	130	160	180

* Based on extrapolations of wind records by statistical analysis, these are probable values and there is a 2 per cent probability that these winds will be exceeded within the mean recurrence interval used.

367. In setting up design criteria in geographical areas subject to hurricanes, it is necessary to establish a minimum recurrence time interval for high winds, either by using existing climatological data with sufficient analysis, or by estimating past storm velocities and adopting an estimated figure for high wind occurrence within the periods of recurrence used.

B. Legislative requirements

368. By legislation, the climatological data for the area should be incorporated into the building code requirements or standards used in the area. A design wind

velocity, based on either a 25-year, 50-year, or 100-year (or greater) mean recurrence interval must be adopted. Buildings must, then, be designed to withstand the loads from these winds, in conformance with (a) approved methods of applying these loads to buildings or structures, and (b) recognized design standards for building materials in use in that geographical area or country. As indicated below, longer recurrence intervals must be used for important buildings.

369. In addition to the climatological data adopted by legislation, and the design standards which may be in use already, it may be necessary to adopt by legislation specific design details and special provisions for hurricane resistant construction. Such additional data could be adopted with a prefatory statement such as: "There is hereby adopted by the _____, for the purpose of supplementing existing building construction laws, sections and details given as follows: ....."

C. Importance of the structure

370. In establishing a mean recurrence interval for wind velocities, the following points must be considered:

(a) The importance of the structure;

(b) The anticipated life of the structure;

(c) The risk to human life and property in case of failure;

(d) The possible post-disaster uses for the structure;

(e) The use of the structure for shelter during hurricanes.

371. For important permanent structures such as power plants, a 100-year or greater mean recurrence interval should be used. Most permanent structures should use a 50-year mean recurrence interval. For structures where there is no danger involved to human life, a 25-year mean recurrence interval may be used.

D. Velocity pressures

372. Based on surrounding terrain in the immediate geographical area, velocity pressures to be used in structural design computations are given in tables 6 and 7. Table 6 is based on the exposure of towns, cities, suburban areas, wooded areas and rolling terrain located away from the coastline. Table 7 is based on coastal areas, open country, open flat country areas and grass land. These tables have been computed for different basic wind speeds, at 9.20 m elevation, varying from 130 kmph to 210 kmph and for heights of from 9.20 m to 240 m above grade. A gust factor of 1.0 is included (no increase). A velocity must be picked for the geographical area in question; the choice of velocity used will be based on the mean recurrence interval used.

E. Gust effects

373. Velocity pressures given in the two tables use a gust factor of 1.0. No higher gust factor than 1.0 has been used because of the fact that the

Table 6. Velocity pressures in pounds per square foot for towns, cities, suburban areas, wooded areas and rolling terrain (away from coast line)

Height (feet above ground of portion of building being designed)	For basic wind speed (miles per hour) of: 80	90	100	110	120	130
30 or less	8	10	13	15	18	22
50	10	13	16	19	23	27
100	14	18	22	26	31	37
150	17	21	26	32	38	44
200	19	24	30	36	43	50
250	21	27	33	40	47	55
300	23	29	36	43	51	60
350	24	31	38	46	55	64
400	26	33	40	49	58	68
450	27	34	42	51	61	72
500	29	36	45	54	64	75
550	30	38	47	56	67	79
600	31	39	48	59	70	82
650	32	41	50	61	72	85
700	33	42	52	63	74	87
750	34	43	54	65	77	90
800	35	45	55	67	79	93

NOTES: Velocity pressures are based on a power law exponent of 1/4.5, related to height at 30 feet.

Velocity pressures include a gust factor of 1.0. For buildings of importance, or for buildings essential to post-disaster use, multiply above figures by a gust factor of 1.25 for design of cladding, windows, girts, purlins etc. having a tributary area of less than 200 sq ft.

Equivalent metric measurements:

1 pound/sq ft = 4.8824 kg/sq m
1 foot = .3048 m
1 sq ft = .0929 sq m
1 mile/hour = 1.6093 km/hour

Table 7. Velocity pressures in pounds per square foot for flat, open country; open flat coastal areas and grassland

Height (feet above ground of portion of building being designed)	For basic wind speed (miles per hour) of: 80	90	100	110	120	130
30 or less	16	21	26	31	37	43
50	19	24	30	36	43	50
100	23	29	36	44	52	61
150	26	33	40	49	58	68
200	28	36	44	53	63	74
250	30	38	47	57	67	79
300	32	40	49	60	71	83
350	33	42	51	62	74	87
400	34	43	54	65	77	90
450	35	45	55	67	80	93
500	37	46	57	69	82	96
550	38	47	59	71	84	99
600	38	49	60	73	87	102
650	39	50	61	74	88	104
700	40	51	63	76	91	106
750	41	52	64	77	92	108
800	42	53	65	79	94	110

NOTES: Velocity pressures are based on a power law exponent of 1/7, related to height at 30 feet.

Velocity pressures include a gust factor of 1.0. For buildings of importance, or for buildings essential to post-disaster use, multiply above figures by a gust factor of 1.25 for design of cladding, windows, girts, purlins etc. having a tributary area of less than 200 sq ft.

Equivalent measurements in the metric system:

1 pound/sq ft = 4.8824 kg/sq m
1 foot = .3048 m
1 sq ft = .0929 sq m
1 mile/hour = 1.6093 km/hour

recommendations are directed towards low-cost housing; it is felt that gust factors can be omitted for this type of construction, based on economics and building life. However, for buildings of importance or for buildings essential to post-disaster use, or for buildings used as public hurricane shelters, all figures in the tables should be multiplied by a gust factor of 1.25, for the design of cladding, windows, girts, purlins etc. with a tributary area of less than 20 sq m. (Areas larger than this will not be affected by gusts, except in those specially wind-sensitive structures for which special dynamic analyses should be made.)

F. Pressure coefficients (shape factors)

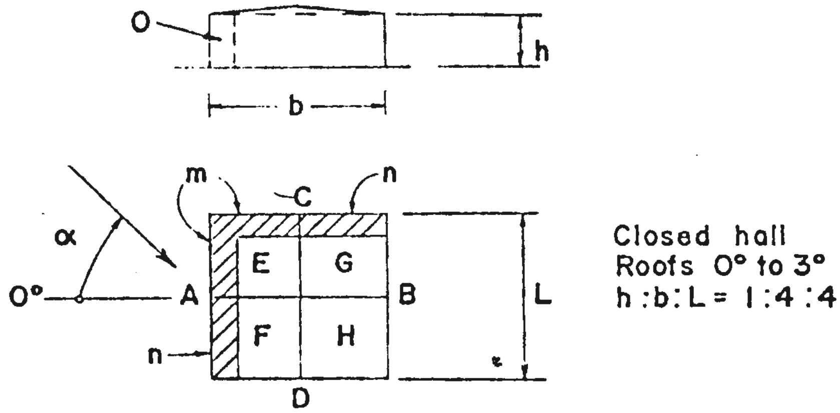
374. The velocity pressures given in tables 6 and 7 must be multiplied by the pressure coefficients for various building shapes. Pressure coefficients define pressures acting normally at localized positions on the surface of the building. In calculating wind loads on buildings and structures, the pressure difference between opposite faces should be considered. The total wind load on the building or structure is obtained by taking the sum of the resultant forces on its elements.

375. Basically, the wind load used in the design is equivalent to the velocity pressure given in the velocity pressure tables, multiplied by the shape factor for the surface in question, multiplied by the projected area of the structure on a plane normal to the wind direction.

P = design load = velocity pressure x pressure coefficient x area

376. The following 10 figures showing pressure coefficients apply to common shapes of rectangular buildings. Positive coefficients indicate positive pressure towards the face of the areas; negative pressures indicate pressures away from the surface areas. These have been adapted from the Swiss Engineers and Architects Report, 1951. 10/

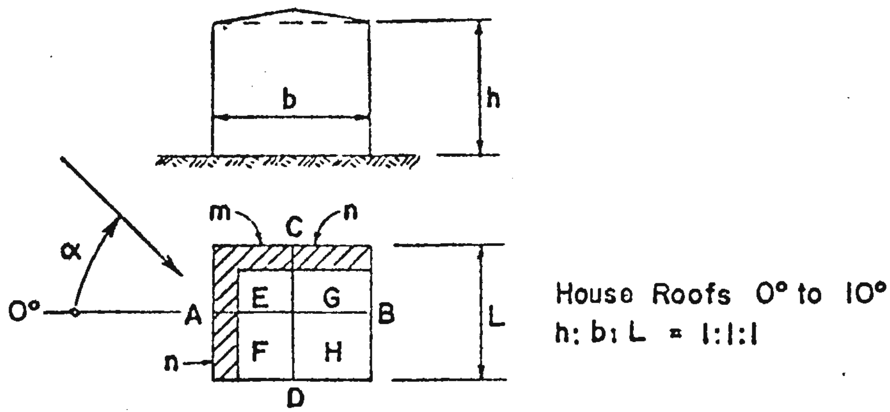
10/ Swiss Architects and Engineers Shape Factor Tables (1951).



α	External Pressure Coefficient, C_{pe}								Internal Pressure Coefficient, C_{pi}			
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in A	B	C
0°	0.9	−0.3	−0.4	−0.4	−0.8	−0.8	−0.3	−0.3	± 0.2	0.8	−0.2	−0.3
15°	0.8	−0.3	−0.1	−0.5	−0.7	−0.8	−0.2	−0.3	± 0.2	0.7	−0.3	−0.2
45°	0.5	−0.4	0.5	−0.4	−0.9	−0.6	−0.6	−0.3	± 0.2	0.4	−0.4	0.4
15°	For o, side C, C_{pe} = −0.8											
45°	For m, C_{pe} = −2.0; n, C_{pe} = −1.0											

\+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

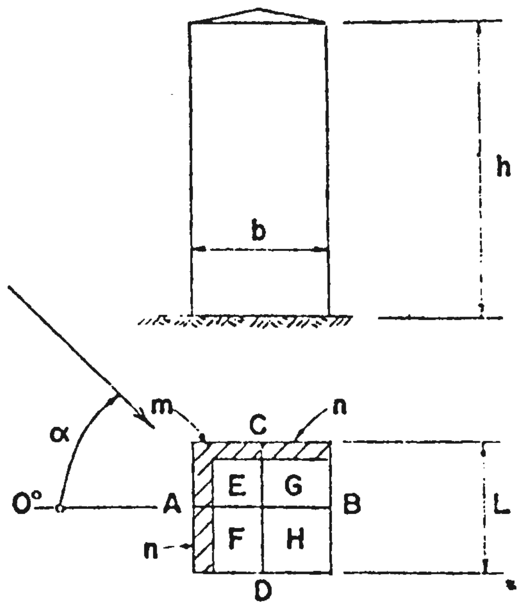
Fig. 60. Pressure coefficients for low, closed hall with roof slope of 0° to 3°



α	External Pressure Coefficient, C_{pe}								Internal Pressure Coefficient, C_{pi}			
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in A	Openings Mainly in B	Openings Mainly in C
0°	0.9	−0.5	−0.6	−0.6	−0.7	−0.7	−0.5	−0.5	± 0.2	0.8	−0.4	−0.5
15°	0.8	−0.5	−0.7	−0.5	−0.7	−0.6	−0.5	−0.6	± 0.2	0.7	−0.4	−0.6
45°	0.5	−0.5	0.5	−0.5	−0.8	−0.5	−0.5	−0.4	± 0.2	0.4	−0.4	0.4
45°	For m, C_{pe} = −1.2; for n, C_{pe} = −0.8											

\+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

Fig. 61. Pressure coefficients for medium-height building with roof slope of 0° to 10°

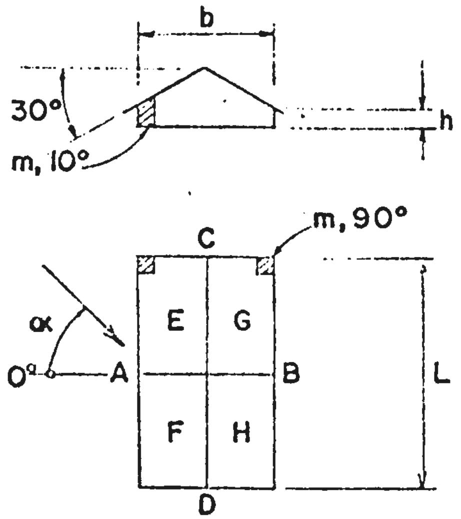


Tall buildings closed
Roofs 0° to 15°
h : b : L = 2.5 : 1 : 1

α	External Pressure Coefficient, Cpe								Internal Pressure Coefficient, Cpi			
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in: A	B	C
0°	0.9	−0.6	−0.7	−0.7	−0.8	−0.8	−0.8	−0.8	± 0.2	0.8	−0.5	−0.6
15°	0.8	−0.5	−0.9	−0.6	−0.8	−0.8	−0.7	−0.7	± 0.2	0.7	−0.5	−0.8
45°	0.5	−0.5	0.5	−0.5	−0.8	−0.7	−0.7	−0.5	± 0.2	0.4	−0.4	0.4
45°	For m, Cpe = −1.0 ; n, Cpe = −0.8											

\+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

Fig. 62. Pressure coefficients for tall building with roof slopes of 0° to 15°

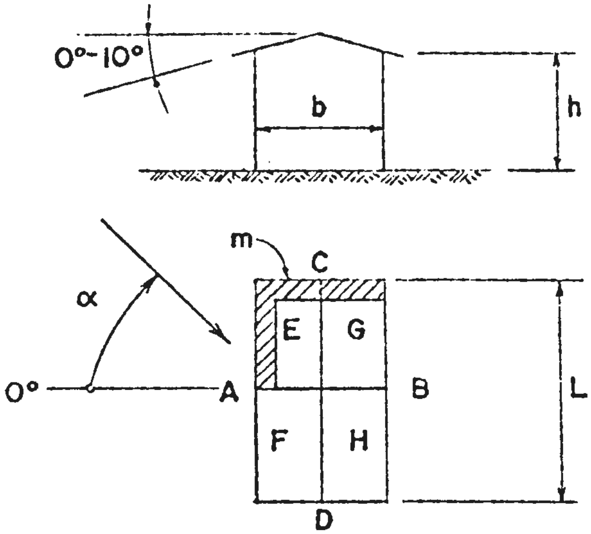


Closed hall
h : b : L = 1 : 8 : 16

α	External Pressure Coefficient, Cpe								Internal Pressure Coefficient, Cpi			
									Openings distributed uniformly	Openings Mainly in		
	A	B	C	D	E	F	G	H		A	B	C
0°	0.8	−0.5	−0.5	−0.5	0.2	0.2	−0.6	−0.6	±0.2	0.7	−0.4	−0.4
45°	0.5	−0.5	0.4	−0.3	0.1	−0.1	−0.8	−0.5	±0.2	0.4	−0.4	0.3
90°	−0.3	−0.3	0.9	−0.3	−0.5	−0.1	−0.5	−0.1	±0.2	−0.2	−0.2	0.8
10° and 90°	For m, Cpe = −1.0											

\+ = inward (toward wall or roof section)
− = outward (away from wall or roof section).
All external pressure coefficients are simultaneous for each α
Only one internal pressure coefficient occurs at same time.

Fig. 63. Pressure coefficients for low, closed hall with roof slope of 30°

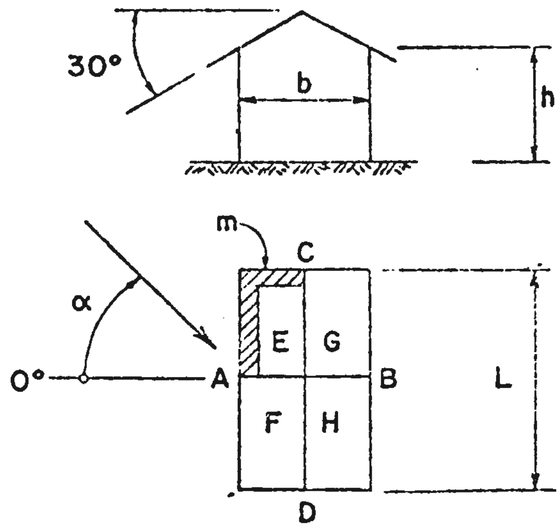


House
h:b:L = 2.5:2:5

α	External Pressure Coefficient, C_{pe}								Internal Pressure Coefficient, C_{pi}			
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in		
										A	B	C
0°	0.9	−0.5	−0.7	−0.7	−0.6	−0.6	−0.5	−0.5	± 0.2	0.8	−0.4	−0.6
45°	0.6	−0.5	0.4	−0.5	−0.9	−0.7	−0.6	−0.7	± 0.2	0.5	−0.4	0.3
90°	−0.5	−0.5	0.9	−0.4	−0.8	−0.2	−0.8	−0.2	± 0.2	−0.4	−0.4	0.8
45°	For m, C_{pe} = −1.5											

\+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

Fig. 64. Pressure coefficients for house with roof slopes of 0° to 10°

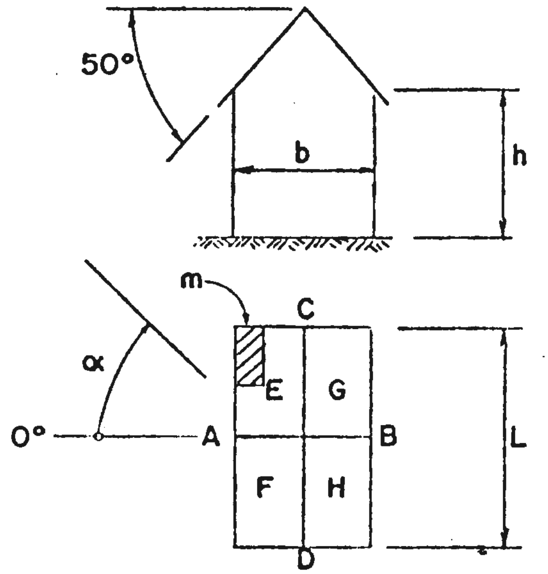


House
h : b : L = 2.5 : 2 : 5

α	External Pressure Coefficient, C_{pe}								Internal Pressure Coefficient, C_{pi}			
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in A	B	C
0°	0.9	−0.5	−0.7	−0.7	−0.6	−0.6	−0.5	−0.5	± 0.2	0.8	−0.4	−0.6
45°	0.6	−0.5	0.4	−0.4	−0.4	−0.5	−0.6	−0.7	± 0.2	0.5	−0.4	0.3
90°	−0.5	−0.5	0.9	−0.4	−0.7	−0.2	−0.7	−0.2	± 0.2	−0.4	−0.4	0.8
45°	For m, $C_{pe} = -1.2$											

+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

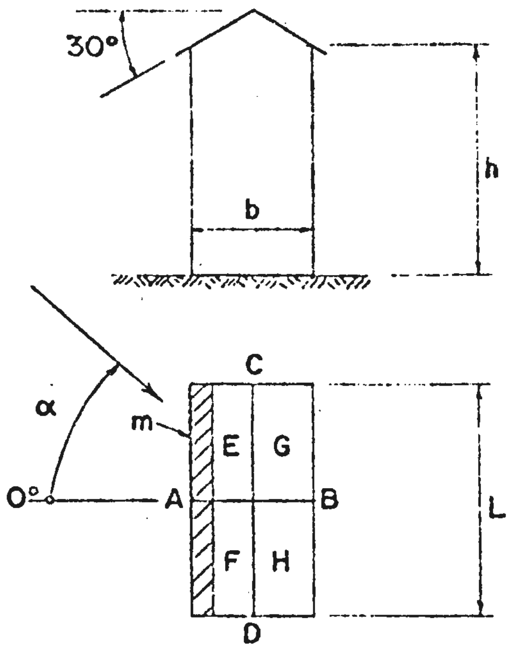
Fig. 65. Pressure coefficients for house with roof slope of 30°



α	External Pressure Coefficient, C_{pe}								Internal Pressure Coefficient, C_{pi}			
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly In A	Openings Mainly In B	Openings Mainly In C
0°	0.9	−0.5	−0.8	−0.8	0.3	0.3	−0.6	−0.6	± 0.2	0.8	−0.4	−0.7
45°	0.6	−0.5	0.4	−0.4	0.3	−0.1	−0.5	−0.6	± 0.2	0.5	−0.4	0.3
90°	− 0.5	− 0.5	0.9	− 0.4	− 0.8	−0.2	−0.8	−0.2	± 0.2	−0.4	−0.4	0.8
75°	For m, C_{pe} = −1.2											

\+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

Fig. 66. Pressure coefficients for house with roof slope of 50°

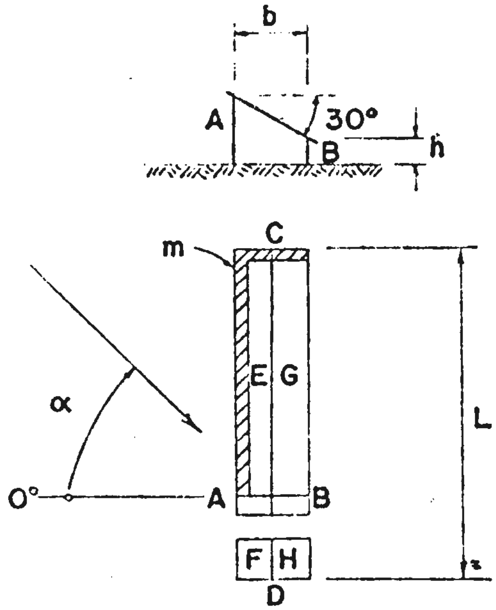


Tall buildings closed,
h : b : L = 2:1:2

α	External Pressure Coefficient, Cpe								Internal Pressure Coefficient, Cpi			
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in A	B	C
0°	0.9	− 0.5	−0.8	− 0.8	− 1.0	−1.0	− 0.5	−0.5	± 0.2	0.8	− 0.4	−0.7
45°	0.6	−0.5	0.4	−0.4	−0.3	−0.4	−0.5	− 0.6	± 0.2	0.5	− 0.4	0.3
90°	− 0.6	− 0.6	0.9	−0.4	− 0.7	−0.5	− 0.7	−0.5	± 0.2	−0.5	− 0.5	0.8
0°	For m, Cpe = − 1.2											

\+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficient are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

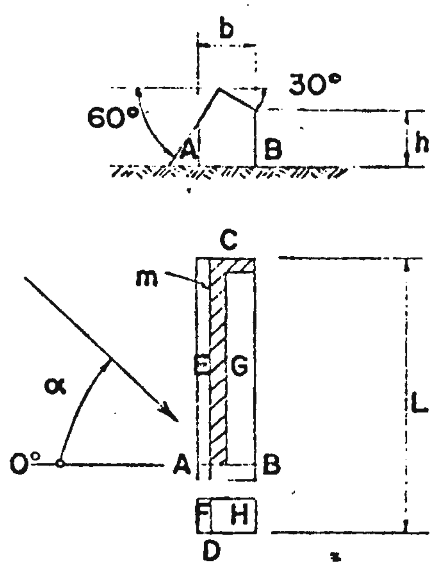
Fig. 67. Pressure coefficients for tall, closed building with roof slope of 30°



α	External Pressure Coefficient, C_{pe}								Internal Pressure Coefficient, C_{pi}				
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in A	B	C	Roof EF
0°	0.9	- 0.5	- 0.6	- 0.6	- 0.5	- 0.5	- 0.5	- 0.5	± 0.2	0.8	- 0.4	- 0.5	- 0.4
45°	0.5	- 0.6	0.4	- 0.4	- 1.2	- 0.7	- 1.1	- 0.7	± 0.2	0.4	- 0.5	0.3	- 0.8
90°	- 0.4	- 0.3	0.9	- 0.2	- 0.3	0	- 0.3	0	± 0.2	- 0.2	- 0.1	0.8	0
180°	- 0.4	0.8	- 0.7	- 0.7	0.1	0.1	0.2	0.2	± 0.2	- 0.3	0.7	- 0.6	0
45°	For m, C_{pe} = -1.4												

+ = inward (toward wall or roof section)
− = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

Fig. 68. Pressure coefficients for shed roof with slope of 30°.



Peaked roof
h : b : L = 1:1:5

α	External Pressure Coefficient, C_{pe}								Internal Pressure Coefficient, C_{pi}				
	A	B	C	D	E	F	G	H	Openings distributed uniformly	Openings Mainly in A	Openings Mainly in B	Openings Mainly in C	Openings Mainly in Roof EF
0°	0.9	-0.5	-0.6	-0.6	0.6	0.6	-0.5	-0.5	± 0.2	0.8	-0.4	-0.5	0.5
45°	0.5	-0.8	0.4	-0.5	0.2	-0.1	-1.0	-0.8	± 0.2	0.4	-0.7	0.3	0
90°	-0.4	-0.4	0.9	-0.3	-0.4	0	-0.4	0	± 0.2	-0.1	-0.1	0.8	-0.1
180°	-0.5	0.9	-0.6	-0.6	-0.5	-0.5	-0.1	-0.1	± 0.2	-0.4	0.8	-0.5	-0.4
45°	For m, C_{pe} = -1.3												

+ = inward (toward wall or roof section)
— = outward (away from wall or roof section)
All external pressure coefficients are simultaneous for each α.
Only one internal pressure coefficient occurs at same time.

Fig. 69. Pressure coefficients for building with peaked roof

XIII. RECOMMENDATIONS FOR HURRICANE RESISTANT DESIGN AND CONSTRUCTION: STRUCTURAL DESIGN CONSIDERATIONS

A. Stress and safety factors

377. Working stresses for construction materials usually depend on the design standard for the material being used, that is, steel, concrete, timber etc. These standards incorporate safety factors. For instance, much structural steel design is on the basis of about a 1.65 safety factor against yield; if the allowable working stress is increased by 1.33 for wind, this means the safety factor is reduced to about 1.24 for wind load.

378. Specific safety factors, for example, for aluminium structural design, are as follows, using United States of America aluminium standards for structural design:

Tension members

Factor of safety on tensile strength	1.95
Factor of safety on yield strength	1.65

Columns

Factor of safety on buckling strength	1.95
Factor of safety on yield strength for short columns	1.65

Beams

Factor of safety on tensile strength	1.95
Factor of safety on tensile yield strength	1.65
Factor of safety on buckling strength	1.65
Factor of safety on shear buckling of webs	1.20

379. If the allowable working stresses for aluminium are increased by 1.33 for wind loads, safety factors above are decreased by 1/1.33.

380. For reinforced concrete design, the new United States design standard, ACI 318-71, for ultimate design, calls for an ultimate strength load factor of $U = 1.4D + 1.7L$. This is the basic load factor for ultimate structural design in concrete. For wind loading, an ultimate load factor of $U = 0.9D + 1.3W$ is used, with the provision that the strength of the member cannot be less than the basic load factor. Obviously, with a load factor at the ultimate of $0.9D + 1.3W$, there is very little safety factor for wind, except that already provided by the basic load factor. If local climatology calls for a 160 kmph design wind, a 190 kmph wind will increase equivalent static loadings to 1.44 times the static loading for a 160 kmph wind, exceeding the load factor coefficient for wind design for concrete.

381. However, actual collapse loads may be greater and a complex statically indeterminate structure probably would not fail if overloaded, especially since portions of the entire structure may not simultaneously undergo the same wind loadings.

382. In considering safety factors, failures can very well occur in smaller components such as cladding, windows, roof anchors, light gauge girts and purlins etc. In these smaller components, the safety factors used in design standards for concrete, structural steel, structural aluminium, timber etc., may be not be sufficient. Safety factors for smaller components must be realistic.

B. General height-width ratios

383. Under many building codes, a building with a height to width ratio of about 2:1 generally does not require analysis for wind loadings. For hurricanes, however - since these cause extraordinarily high wind loads - all buildings should be analysed for wind, or at least checked by comparison or by inspection against known standards for wind loadings. This is especially true for prefabricated buildings and construction materials such as cement-asbestos panels and glass panels.

C. Provisions for high water during hurricanes

384. For resisting high water or high tidal surges, embankments raised to sufficiently high elevations above sea-level will be the first means of protection. Other means of protecting structures against high water are to design pole-type structures with timber piles or poles, or concrete piles or columns embedded into the ground, and with the first floor of the structure at a sufficiently high elevation to be above high water.

D. Specific recommendations for various materials

385. Specific detailed recommendations are given, by engineering construction details, in figures 70 through 76, for hurricane-resistant design and construction. These show important details for concrete block structures with wood roof trusses, steel joists, and timber structures. Tie down details in these figures are of the utmost importance. Sizes of beams and footings should be varied to fit specific designs. Tie beam action and diaphragm action of floors and roofs are important to resist racking loads on the exterior walls.

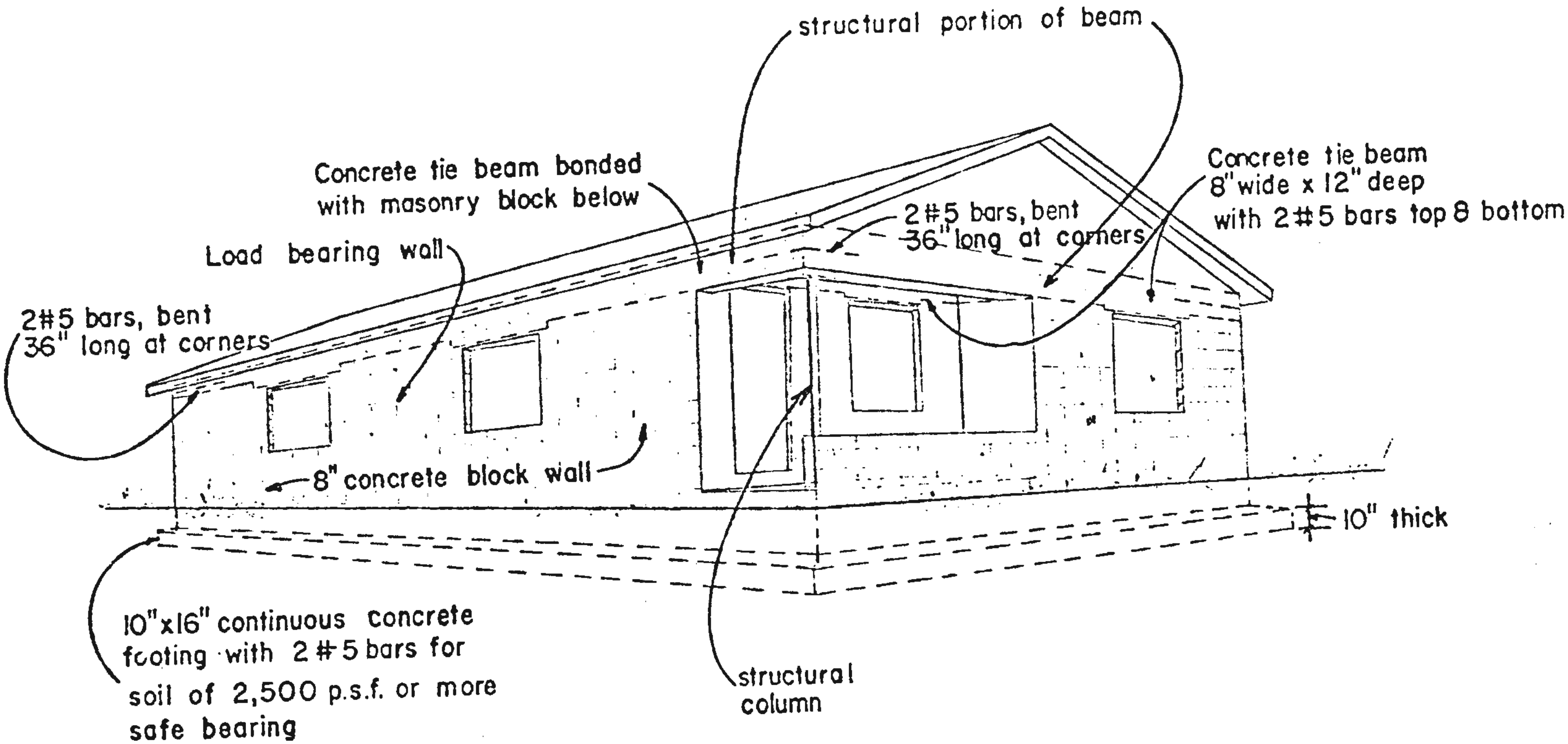


Fig. 70. Construction details of a one-storey house

Note: Equivalent metric measurements:

1" = 2.54 cm.
1 p.s.f. = 4.8 8 kgs/sq. m.
#5 bar = 16 mm bar

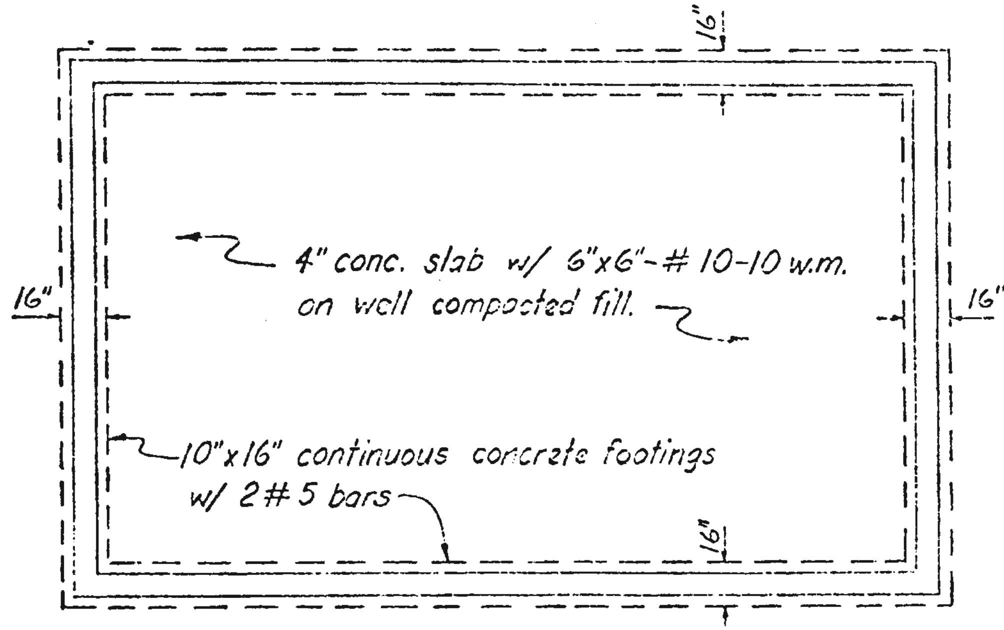


Fig. 71. Foundation plan for a one-storey house

Note: Equivalent metric measurements:

1" = 2.54 cm
#5 bar = 16 mm bar

(1) Roof trusses spaced 18" or 24" anchored to conc. tie beam with 1/8" x 1" strap iron embedded into the conc. a min. of 7" and nailed to the rafter with not less than 16 d. galvanized nails.

(2) Plywood 1/2" min. thickness continuous over two or more spans, staggered with face grain perpendicular to supports, nailed to rafters with 8 d. nails spaced 6" edges and 12" o.c. at intermediate supports.

(3) 30# roofing felt nailed to plywood by 12 gage wire ring-shanked nails applied through tin caps not less than 1 5/8" diameter and not more than 2", not less in thickness than 32 gage sheet metal, spaced 12" o.c.

(4) Coal tar applied in a quantity not less than 25# per square per ply.

(5) 90# roofing felt on top of hot tar (275°-350°F).

(6) Gravel on top of flood coat of tar (150# per square).

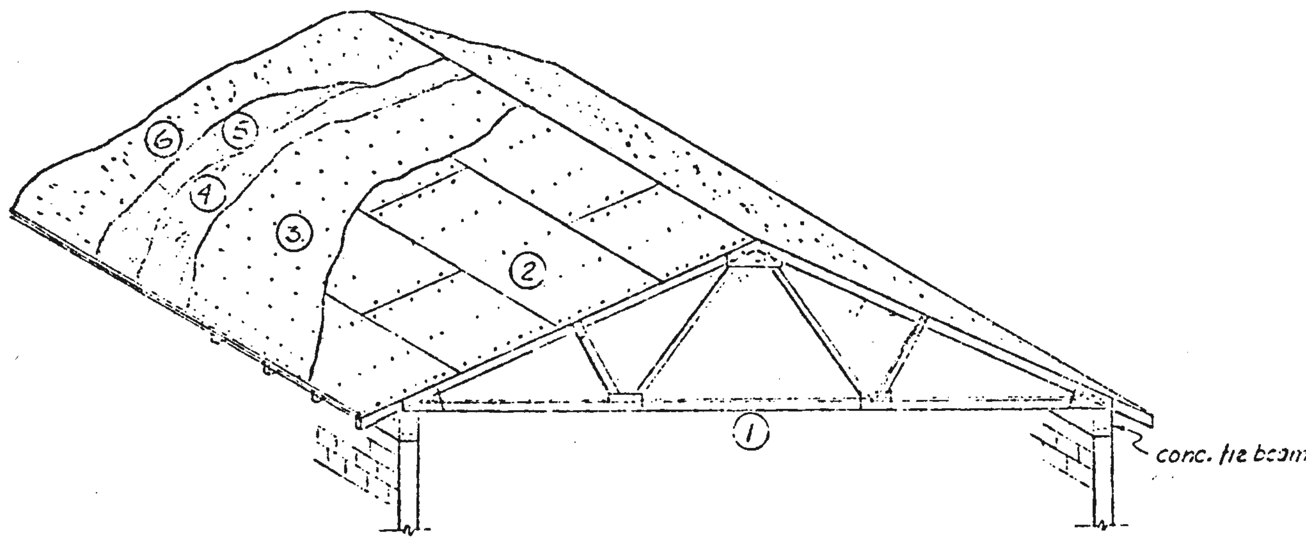


Fig. 72. Roof construction detail

Note: Equivalent metric measurements:

1" = 2.54 cm
1# = .465 kg
275-350 F = 135-176 C

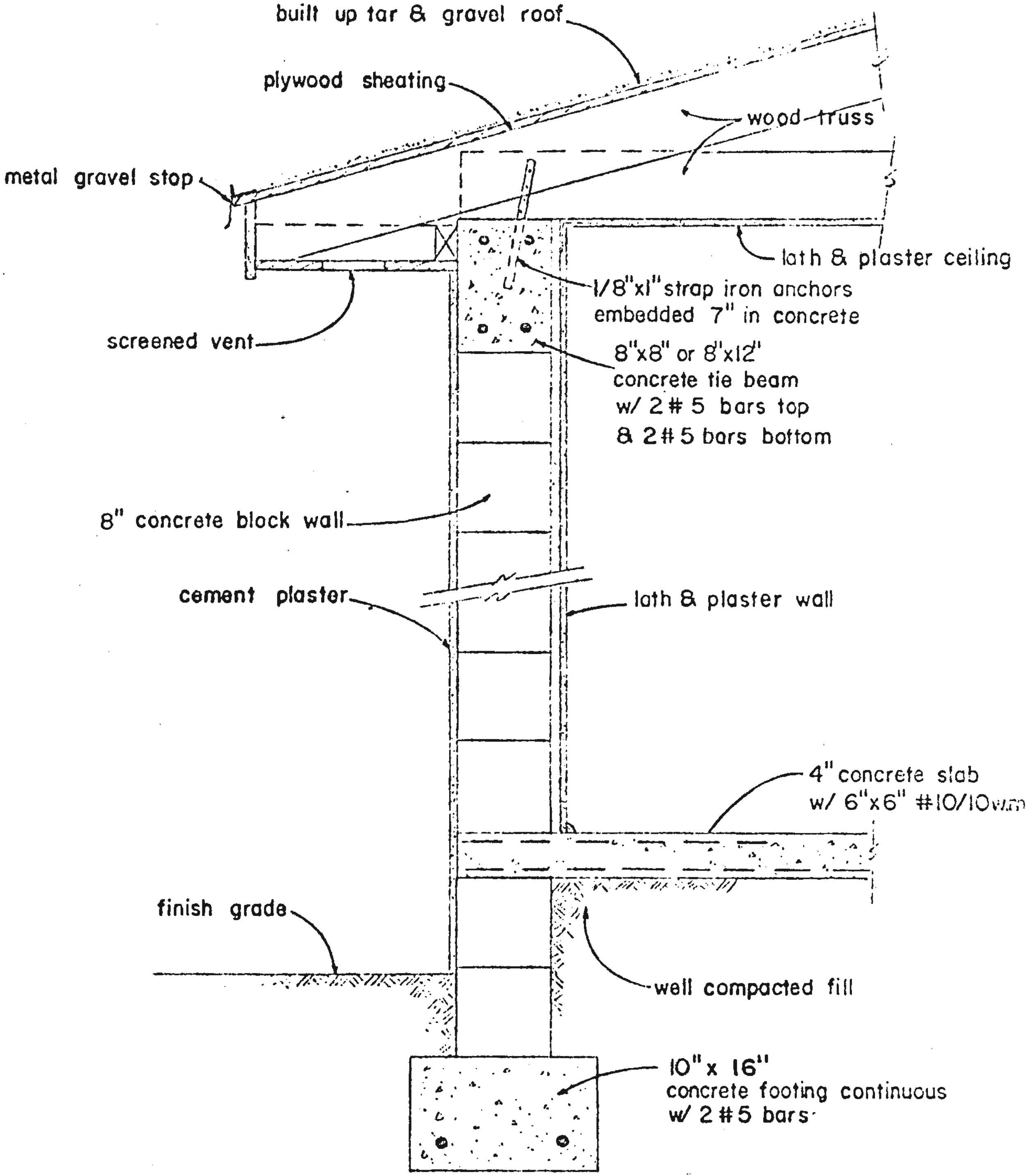


Fig. 73. Typical block wall section - residential structure

Note: 1" = 2.54 cm
#5 bar = 16 mm bar

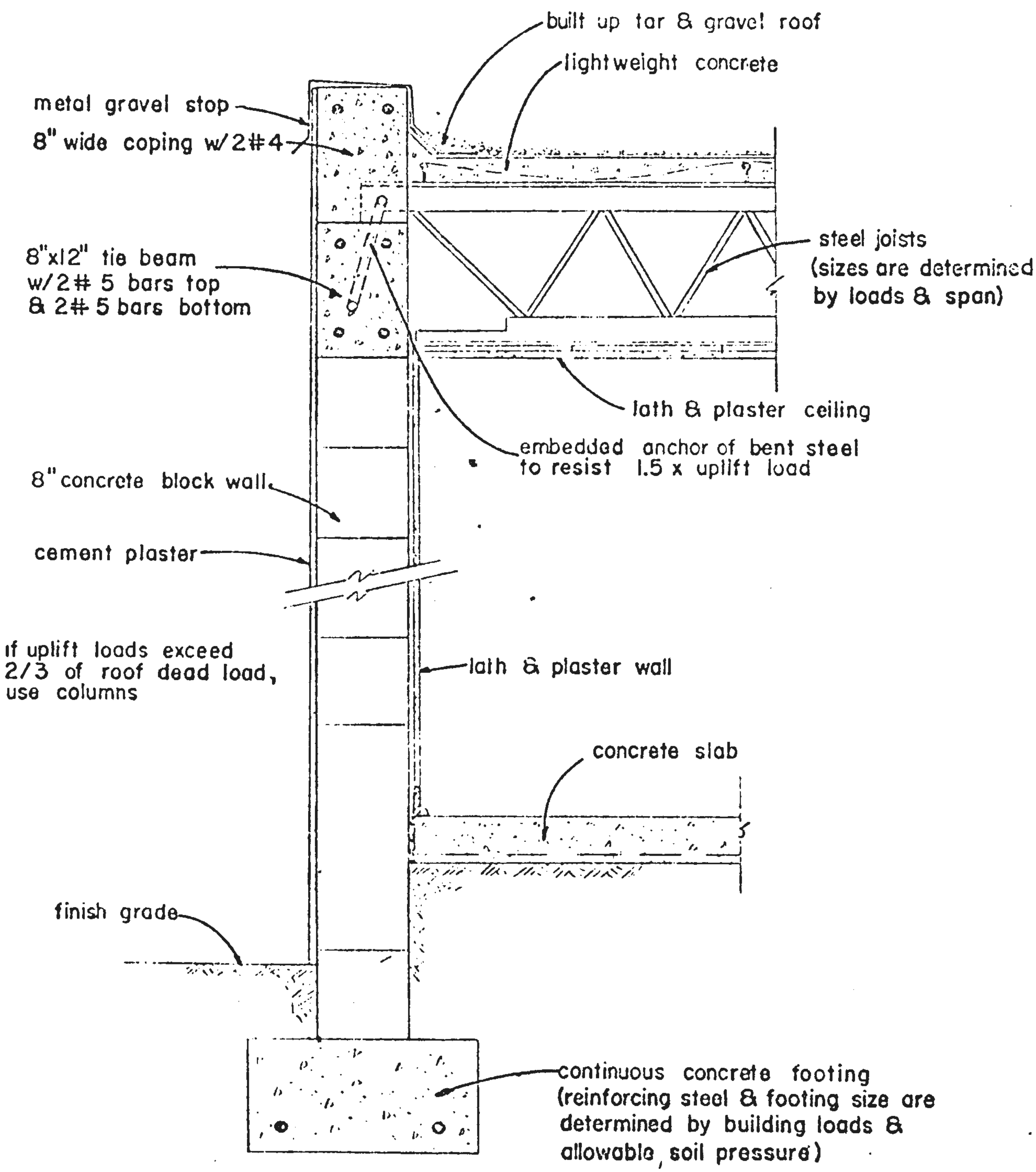
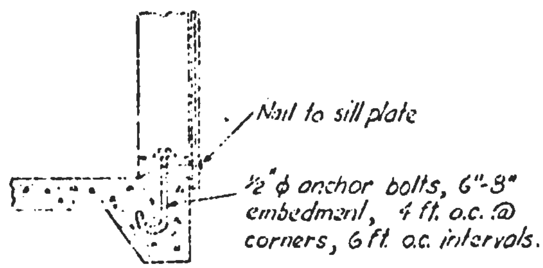


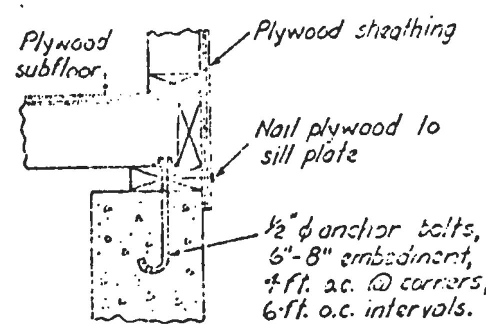
Fig. 74. Typical block wall section - commercial structure

Note: Equivalent metric measurements:

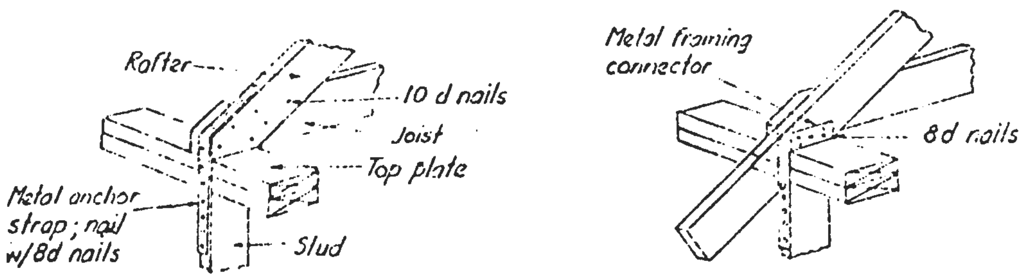
1" = 2.54 cm
#4 bar = 12 mm bar
#5 bar = 16 mm bar



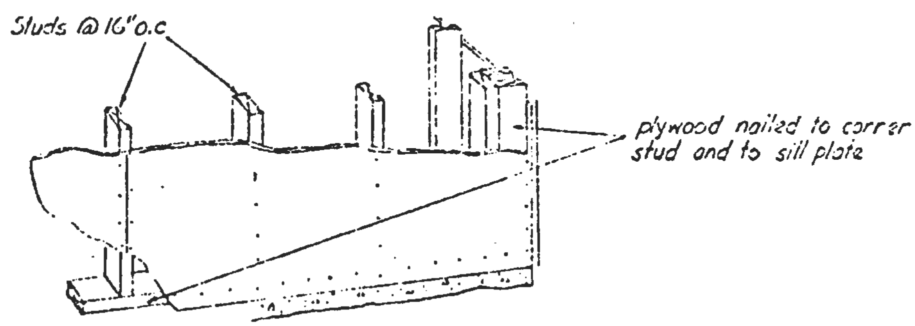
WALL-TO-SLAB CONNECTION



WALL-TO-FOOTING CONNECTION



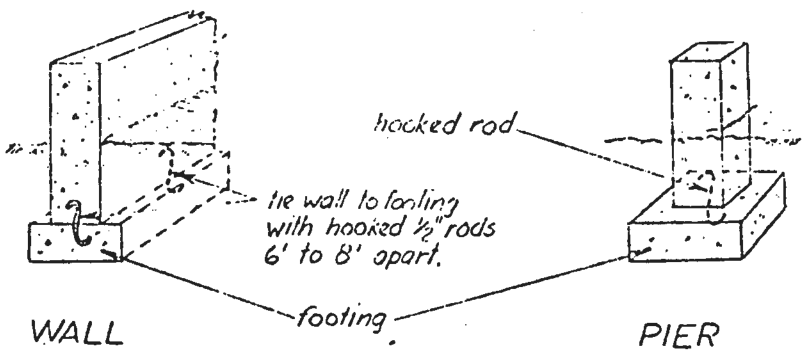
JOIST-STUD CONNECTIONS



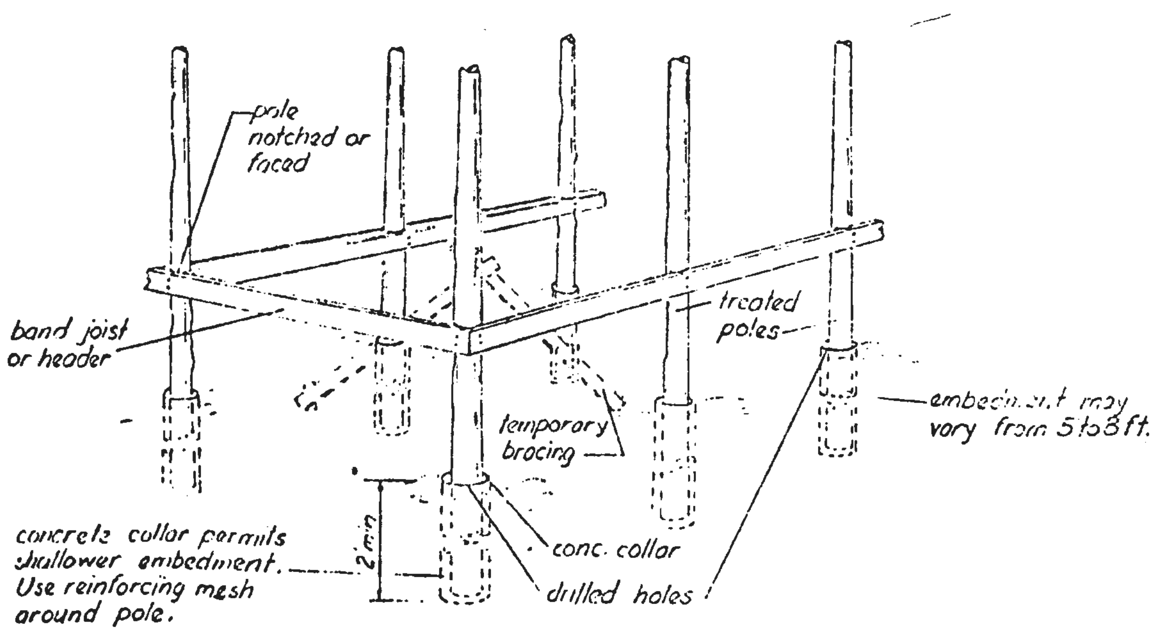
Corner detail

Fig. 75. Details for structural connexion of members

Note: 1" = 2.54 cm
1 foot = 30.48 cm



Anchoring footings to concrete wall or pier on solid ground



Foundation detail for pole type structure

Fig. 76. Details of foundations

Note: 1" = 2.54 cm
1' = 1 ft. = 30.48 cm

XIV. SUMMARY OF RECOMMENDATIONS FOR WIND LOADINGS AND STRUCTURAL DESIGN CONSIDERATIONS

386. The following rules and recommendations in summary form are furnished:

(a) Based on past climatology of the area, adopt a design storm, taking into account probabilities of recurrence over a period of years and the type of structures and occupancies being considered.

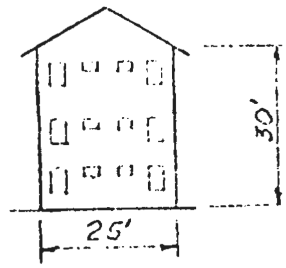
(b) Adopt a building code or set of standards that incorporate high velocity wind loadings set up specifically for hurricane winds or typhoon winds. If the code in force does not consider hurricane winds, either adopt a new code or modify the existing code in conformance with detailed requirements given here.

(c) For coastal areas, based on past experience or estimates, establish a map showing possible hurricane tidal surges for several design storm periods (50 years, 100 years, 150 years, 200 years etc.). For construction in these areas subject to tidal surge, additional consideration or safeguards must be provided (dikes, levees, escape roads, fill, pole-type construction etc.).

387. These three general rules will cover hurricane-resistant design and construction. Detailed recommendations, of course, will depend on the variable factors considered, such as the type and usage of building, whether it will be used for a shelter, accuracy of climatological data etc., always bearing in mind that probabilities are generally estimates, and may be exceeded.

TYPICAL EXAMPLE NO. 1

A multiple dwelling as shown:



In this case, h = 30' b = 25' L = 60'

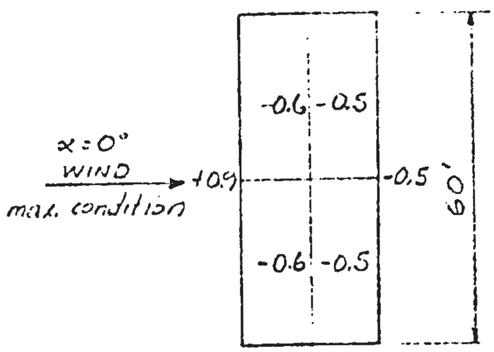
Figure 65 meets the dimensions
h:b:L = 2.5:2:5

Assume the building is at the coast, therefore use table 7

Use 100 mph as design velocity and gust factor = 1.00, therefore from table 4, velocity pressure p = 26 p.s.f.

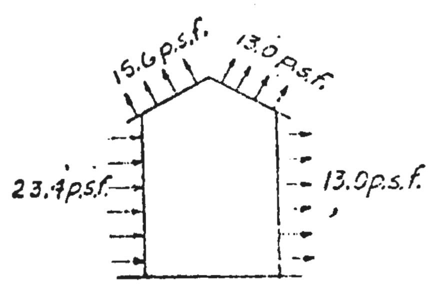
From figure 65 shape factors are obtained

Openings are uniformly distributed
Use ± 0.2 for Cpi (internal pressure coefficient)



For overall stability, using velocity pressure of 26 p.s.f.

26 p.s.f. × (0.9 + 0.5) = 36.4 p.s.f. (23.4 + 13.0)
26 p.s.f. × (0.6) = 15.6 p.s.f.
26 p.s.f. × (0.5) = 13.0 p.s.f.



Design pressures

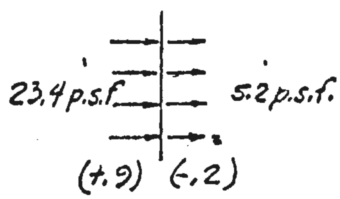
Fig. 77. Structural analysis of a multiple dwelling

Note: 1' = 1 ft. = 30.48 cm
1 p.s.f. = 4.8824 kgs/sq. m

HOUSING CONSTRUCTION IN HURRICANE PRONE AREAS:
TYPICAL EXAMPLE NO. 1
continued

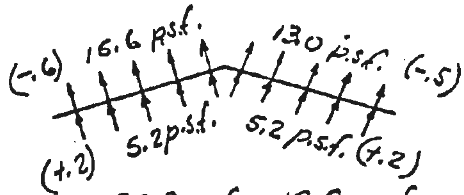
For walls, windows, roof, as components.
Use previous figures with ± 0.2 added & 1.00 gust factor.

The front wall governs 23.4 p.s.f.



Total on wall: 23.4 + 5.2 = 28.6 p.s.f.

Roof



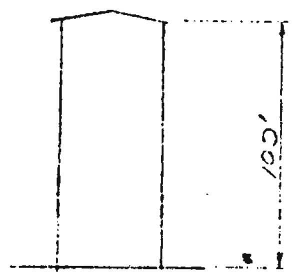
total: 20.8 p.s.f. 18.2 p.s.f.

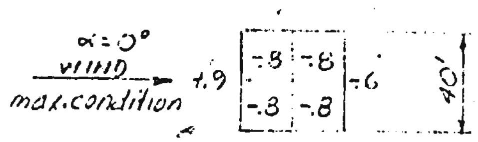
Fig. 77 (continued)

Note: 1 p.s.f. = 4.8824 kgs/sq. m

TYPICAL EXAMPLE NO. 2

A 100' high power plant tower





In this case h = 100' b = 40' L = 40'

Fig. 10 meets the dimensions

h:b:L = 2:5:1:1

Assume the building is at the coast, therefore use table 7

Use 130 mph as design velocity and gust factor = 1.25, therefore from table 7 velocity pressure p =

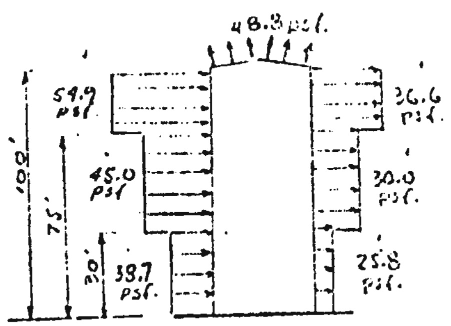
30' - 43 p.s.f.
50' - 50 p.s.f.
100' - 61 p.s.f.

From figure 62 shape factors are obtained

Openings are uniformly distributed
Use ± 0.2 for C_{pi} (internal pressure coefficient)

For over-all stability, using above velocity pressures:

43 p.s.f. x (0.9) + (0.6)= 38.7 + 25.8 p.s.f.
50 p.s.f. x (0.9 + 0.6)= 45.0 + 30.0 p.s.f.
61 p.s.f. x (0.9 + 0.6)= 54.9 + 36.6 p.s.f.
61 p.s.f. x (.8) = 48.8 p.s.f.



Design pressures

Fig. 78. Structural analysis for a 100-foot tower

Note: 1' = 1 ft. = 30.48 cm.
1 p.s.f. = 4.8824 kg/sq. m

TYPICAL EXAMPLE NO. 2

continued

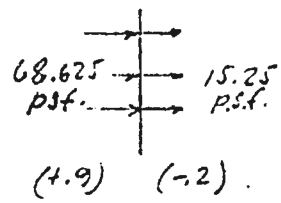
For walls, windows, roof, as components
Use previous figures with ±0.2 added & 1.25 gust factor

The front top wall governs.

61×1.25 = 76.25 p.s.f.

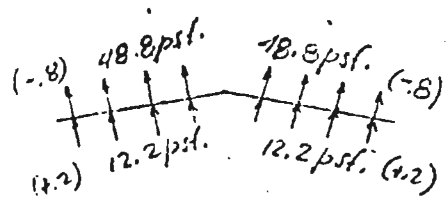
76.25 × .9 = 68.625 psf.

76.25 × .2 = 15.25 psf.



Total on wall: 68.63 + 15.25 = 83.88 p.s.f.

Roof:



(on roof, computed for tributary areas of over 200 sq. ft., gust factor = 1.00)

total: 61 psf.

Above component figures are for height range 75' to 100'. Wall loadings can be reduced at lower heights.

Fig. 78. (continued)

Note: 1 p.s.f. = 4.8824 kg/sq. m

Annex

WIND REQUIREMENTS FOR STRUCTURAL DESIGN IN THE BUILDING CODE IN EFFECT IN SOUTH FLORIDA, UNITED STATES OF AMERICA

Section 2306 on wind requirements and shape factors of the South Florida Building Code is reproduced below.

2306

2306.1 GENERAL: (a) Building and structures and every portion thereof shall be designed and constructed to resist the forces due to wind pressure. The wind velocity shall be taken as not less than 120 MPH at a height of 30 feet above the ground, except as may be otherwise set forth herein.

(b) Such forces shall be applied in any direction, with all possible combinations based on height and shape factors, but in no case shall any roof be designed for less than 30 pounds per square foot live load. The said live load shall not be considered to act simultaneously with the wind load.

(c) Systems shall be designed and constructed to transfer wind forces to the ground.

(d) No allowance shall be made for the shielding effect of buildings or other structures.

(e) The minimum unit wind pressures to be used in design shall be obtained by multiplying the velocity pressures set forth in Table 23-B of Sub-section 2306.2 by the Shape Factors as described in Sub-section 2306.3.

(f) The Building Official may accept a design based on other nationally recognized and accepted data, the validity of which is shown by wind tunnel and/or satisfactory test data, and may require evidence to support the values for wind pressure used in the design of structures not specifically included in this Section.

(g) Structural members, providing stability for the building or structure, shall be designed to resist the forces set forth in Table 23-B multiplied by the shape factors set forth in Paragraph 2306.3(a).

(h) Building components such as, but not limited to, purlins, girts, wall panels and sheathing, transferring wind loads to the structural frame, shall be designed to resist the forces set forth in Table 23-B multiplied by the shape factors set forth in Paragraph 2306.3(b).

2306.2 VELOCITY PRESSURES: (a) Velocity pressures, in pounds per square foot, based on height above ground, in feet, shall be taken as not less than those in Table 23-B.

TABLE 23-B

HEIGHT ABOVE GROUND (In Feet)	MINIMUM VELOCITY PRESSURE In pounds per square foot
0 to 5	22
5 to 15	27
15 to 25	33
25 to 35	37
35 to 55	41
55 to 75	46
75 to 100	50
100 to 150	55
150 to 250	63
250 to 350	71
350 to 550	80
550 to 750	89
750 to 1000	97
over 1000	100

(b) Velocity pressures are based on the formula

$$P = 0.00256 \times V^2 \times \left(\frac{H}{30}\right)^{2/7} \quad \text{where:}$$

V = 120 MPH; and
H = the height above grade (in feet) of the pressure being computed.

(c) Velocity pressure for heights above 1000 feet may be taken as that for 100 feet.

2306.3 SHAPE FACTORS: (a) Shape factors for the stability of a building or structure shall be taken as ("Plus" signifies pressures inward or downward and "minus" signifies pressures outward or upward).

(1) For Vertical Surfaces:

(aa) Rectangular Prismatic Structures 1.3
(sum of + 0.8 windward and - 0.5 leeward)
(bb) Cylinders 0.7
(cc) Flat surfaces with no appreciable depth such as signs and fences 1.4
(dd) Partially Open Surfaces:

Per cent Solid	Shape Factor (times gross area)
10	0.35
20	0.55
40	0.80
60	1.00
80	1.20
100	1.30

(2) For Horizontal Surfaces (Including Surfaces with less than 10° inclination to the horizontal).

	Windward* 1/3 of surface	Leeward** 2/3 of surface
(aa) Enclosed Buildings:	-1.0	-0.75
(bb) Buildings with one ore more sides open	-1.5	-1.25
(cc) Overhangs and eaves	-1.5 (all cases)	

* The direction from which the wind is coming.

** The direction towards which the wind is going.

(3) FOR INCLINED SURFACES:

Angles from the Horizontal	Normal to Windward Surface	Normal to Leeward Surface
(aa) Above 70° to 90°	+0.80	-0.50
Above 60° to 70°	+0.70	-0.50
Above 50° to 60°	+0.50	-0.50
Above 40° to 50°	+0.20	-0.50
Above 30° to 40°	-0.20	-0.50
Above 20° to 30°	-0.40	-0.50
10° to 20°	-0.70	-0.50

(bb) Overhangs and Eaves . . . . -1.50 (all cases)

(cc) For buildings with one or more sides open, add -1.0 to the negative factors for inclined surfaces.

(dd) For gable roofs a factor of -0.6 shall be used when the wind is assumed to blow parallel with the roof ridge.

(ee) The wind pressure on a curved roof due to wind blowing at right angles to the axis of the roof shall be computed on the basis that the curved portion is divided into not less than five equal segments. The pressure on each segment, whether positive or negative, shall be determined by the use of shape factors in Sub-paragraph (aa) above, appropriate to the slope of the chords of the segments.

(ff) In multi-span or saw-tooth roofs where the span heights and slopes are approximately the same and where there is a sheltering effect from the windward span, the external pressures and forces on the intermediate spans may be approximately reduced.

(b) Shape factors for building components transferring wind loads to the structural frame shall be taken as:

(1) VERTICAL SURFACE SHAPE FACTORS

	Pressure Inward	Pressure Outward
(aa) Exterior walls of enclosed buildings, including fixed lites of glass, glazing and all supporting members	+1.1	-1.1
(bb) Operative doors and windows, including all constituent parts	+1.1	-0.55
(cc) Exterior walls of buildings with one or more sides open	+1.1	-1.5

(2) Horizontal Surface Shape Factors as set forth in Paragraph 2306.3(a) (2).

(3) Inclined Surface Shape Factors as set forth in Paragraph 2306.3(a) (3).

2306.4 OVERTURNING MOMENT AND UPLIFT: (a) Computations for overturning and uplift shall be based on the building as a whole using the shape factors set forth in Paragraph 2306.3(a).

(b) Overturning stability of any building or structure taken as a whole shall be provided and shall be not less than 150 percent of wind load overturning moment.

(c) Uplift stability of any building structure or part thereof or isolated component thereof shall be provided and shall be not less than 150 per cent of the wind load uplift thereon.

(d) Stability may be provided by dead loads, anchors, attachments, the weight of earth superimposed over footings or anchors, the withdrawal resistance of piles or the resisting moment of vertical members embedded in the ground.

2306.5 STRESSES: (a) For members carrying wind stresses only, and for combined stresses due to wind and other loads, the allowable stresses and the allowable loads on connexions may be increased 33 1/3 percent from the maximums set forth in this Code for the materials used except as follows:

(1) Such increased stresses shall not apply to foundations except as provided in Section 2310.

(2) Such increased stresses shall not apply to towers, cantilevered projections or metal sheathing where vibrations or fluttering action could be anticipated.

(3) Glass areas shall not be increased from those set forth in Table 35-E.

(4) Such increased stresses shall not apply to glazing materials other than glass.

(b) In no case shall the cross-section properties be less than required for dead load plus live load without wind load.

2306.6 SCREEN ENCLOSURES: The wind loads on screen surfaces shall not be less than set forth in Paragraph 4403.4(c). Design shall be based on such loads applied horizontally inward and outward to the walls with a shape factor of 1.3, and applied vertically upward and downward on the roof with a shape factor of 0.7.

كيفية الحصول على منشورات الامم المتحدة

يمكن الحصول على منشورات الامم المتحدة من المكتبات ودور التوزيع في جميع انحاء العالم • استعلم عنها من المكتبة التي تتعامل معها أو اكتب الى : الامم المتحدة ،قسم البيع في نيويورك او في جنيف •

如何购取联合国出版物

联合国出版物在全世界各地的书店和经售处均有发售。请向书店询问或写信到纽约或日内瓦的联合国销售组。

HOW TO OBTAIN UNITED NATIONS PUBLICATIONS

United Nations publications may be obtained from bookstores and distributors throughout the world. Consult your bookstore or write to: United Nations, Sales Section, New York or Geneva.

COMMENT SE PROCURER LES PUBLICATIONS DES NATIONS UNIES

Les publications des Nations Unies sont en vente dans les librairies et les agences dépositaires du monde entier. Informez-vous auprès de votre libraire ou adressez-vous à : Nations Unies, Section des ventes, New York ou Genève.

КАК ПОЛУЧИТЬ ИЗДАНИЯ ОРГАНИЗАЦИИ ОБЪЕДИНЕННЫХ НАЦИЙ

Издания Организации Объединенных Наций можно купить в книжных магазинах и агентствах во всех районах мира. Наводите справки об изданиях в вашем книжном магазине или пишите по адресу: Организация Объединенных Наций, Секция по продаже изданий, Нью-Йорк или Женева.

COMO CONSEGUIR PUBLICACIONES DE LAS NACIONES UNIDAS

Las publicaciones de las Naciones Unidas están en venta en librerías y casas distribuidoras en todas partes del mundo. Consulte a su librero o diríjase a: Naciones Unidas, Sección de Ventas, Nueva York o Ginebra.

Litho in United Nations, New York
05988—August 1975—3,400

Price: $U.S. 9.00
(or equivalent in other currencies)

United Nations publication
Sales No. E.75.IV.7