1 Introduction: Concepts of Seismic Design

1.1 SEISMIC DESIGN AND SEISMIC PERFORMANCE: A REVIEW

Design philosophy is a somewhat grandiose term that we use for the fundamental basis of design. It covers reasons underlying our choice of design loads, and forces, our analytical techniques and design procedures, our preferences for particular structural configuration and materials, and our aims for economic optimization. The importance of a rational design philosophy becomes paramount when seismic considerations dominate design. This is because we typically accept higher risks of damage under seismic design forces than under other comparable extreme loads, such as maximum live load or wind forces. For example, modern building codes typically specify an intensity of design earthquakes corresponding to a return period of 100 to 500 years for ordinary structures, such as office buildings. The corresponding design forces are generally too high to be resisted within the elastic range of material response, and it is common to design for strengths which are a fraction, perhaps as low as 15 to 25%, of that corresponding to elastic response, and to expect the structures to survive an earthquake by large inelastic deformations and energy dissipation corresponding to material distress. The consequence is that the full strength of the building can be developed while resisting forces resulting from very much smaller earthquakes, which occur much more frequently than the design-level earthquake. The annual probability of developing the full strength of the building in seismic response can thus be as high as 1 to 3%. This compares with accepted annual probabilities for achieving ultimate capacity under gravity loads of perhaps 0.01%. It follows that the consequences resulting from the lack of a rational seismic design philosophy are likely to be severe.

The incorporation of seismic design procedures in building design was first adopted in a general sense in the 1920s and 1930s, when the importance of inertial loadings of buildings began to be appreciated. In the absence of reliable measurements of ground accelerations and as a consequence of the lack of detailed knowledge of the dynamic response of structures, the magnitude of seismic inertia forces could not be estimated with any reliability. Typically, design for lateral forces corresponding to about 10% of the building weight was adopted. Since elastic design to permissible stress levels was invariably used, actual building strengths for lateral forces were generally somewhat larger.

By the 1960s accelerograms giving detailed information on the ground acceleration occurring in earthquakes were becoming more generally available. The advent of strength design philosophies, and development of sophisticated computer-based analytical procedures, facilitated a much closer examination of the seismic response of multi-degree-of-freedom structures. It quickly became apparent that in many cases, seismic design to existing lateral force levels specified in codes was inadequate to ensure that the structural strength provided was not exceeded by the demands of strong ground shaking. At the same time, observations of building responses in actual earthquakes indicated that this lack of strength did not always result in failure, or even necessarily in severe damage. Provided that the structural strength could be maintained without excessive degradation as inelastic deformations developed, the structures could survive the earthquake, and frequently could be repaired economically. However, when inelastic deformation resulted in severe reduction in strength, as, for example, often occurred in conjunction with shear failure of concrete or masonry elements, severe damage or collapse was common.

With increased awareness that excessive strength is not essential or even necessarily desirable, the emphasis in design has shifted from the resistance of large seismic forces to the "evasion" of these forces. Inelastic structural response has emerged from the obscurity of hypotheses, and become an essential reality in the assessment of structural design for earthquake forces. The reality that all inelastic modes of deformation are not equally viable has become accepted. As noted above, some lead to failure and others provide ductility, which can be considered the essential attribute of maintaining strength while the structure is subjected to reversals of inelastic deformations under seismic response.

More recently, then, it has become accepted that seismic design should encourage structural forms that are more likely to possess ductility than those that do not. Generally, this relates to aspects of structural regularity and careful choice of the locations, often termed plastic hinges, where inelastic deformations may occur. In conjunction with the careful selection of structural configuration, required strengths for undesirable inelastic deformation modes are deliberately amplified in comparison with those for desired inelastic modes. Thus for concrete and masonry structures, the shear strength provided must exceed the actual flexural strength to ensure that inelastic shear deformations, associated with large deterioration of stiffness and strength, which could lead to failure, cannot occur. These simple concepts, namely (1) selection of a suitable structural configuration for inelastic response, (2) selection of suitable and appropriately detailed locations (plastic hinges) for inelastic deformations to be concentrated, and (3) insurance, through suitable strength differentials that inelastic deformation does not occur at undesirable locations or by undesirable structural

Fig. 1.1 Soft-story sway mechanism, 1990 Philippine earthquake. (Courtesy of EQE Engineering Inc.)

modes—are the bases for the capacity design philosophy, which is developed further in this chapter, and described and implemented in detail in subsequent chapters.

Despite the increased awareness and understanding of factors influencing the seismic behavior of structures, significant disparity between earthquake engineering theory, as reported, for example, in recent proceedings of the World Conferences on Earthquake Engineering [1968–88], and its application in design and construction still prevails in many countries. The damage in, and even collapse of, many relatively modern buildings in seismically active regions, shown in Figs. 1.1 to 1.7, underscores this disparity.

Figure 1.1 illustrates one of the most common causes of failure in earthquakes, the "soft story mechanism." Where one level, typically the lowest, is weaker than upper levels, a column sway mechanism can develop with high local ductility demand. In taller buildings than that depicted in Fig. 1.1, this often results from a functional desire to open the lowest level to the maximum extent possible for retail shopping or parking requirements.

Figure 1.2, also from the July 1990 Philippine's earthquake, shows a confinement failure at the base of a first-story column. Under ductile response to earthquakes, high compression strains should be expected from the combined effects of axial force and bending moment. Unless adequate, closely spaced, well-detailed transverse reinforcement is placed in the potential plastic hinge region, spalling of concrete followed by instability of the compression reinforcement will follow. In the example of Fig. 1.2, there is

Fig. 1.2 Confinement failure of column base of 10-story building. (Courtesy of EQE Engineering, Inc.)

clearly inadequate transverse reinforcement to confine the core concrete and restrain the bundled flexural reinforcement against buckling. It must be recognized that even with a weak beam/strong column design philosophy which seeks to dissipate seismic energy primarily in well-confined beam plastic hinges, a column plastic hinge must still form at the base of the column. Many structures have collapsed as a result of inadequate confinement of this hinge.

The shear failure of a column of a building in the 1985 Chilean earthquake, shown in Fig. 1.3, demonstrates the consequences of ignoring the stiffening effects of so-called nonstructural partial height masonry or concrete infill built hard up against the column. The column is stiffened in comparison with other columns at the same level, which may not have adjacent infill (e.g., interior columns) attracting high shears to the shorter columns, often with

Fig. 1.3 Influence of partial height infill increasing column shear force (1985 Chilean earthquake). (Courtesy of *Earthquake Spectra* and the Earthquake Engineering Research Institute.)

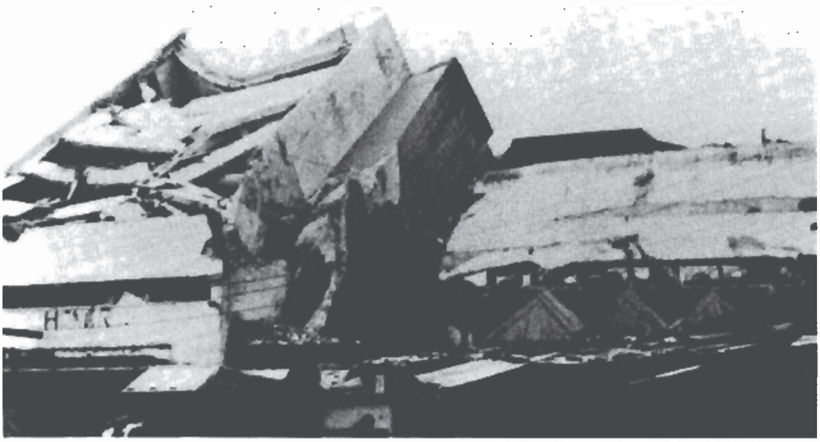

Fig. 1.4 Failure of structural wall resulting from inadequate flexural and shear strength (1990 Philippine earthquake). (Courtesy of EQE Engineering Inc.)

Fig. 1.5 Failure of coupling beams between shear walls (1964 Alaskan earthquake). (Courtesy of the American Iron and Steel Institute.)

disastrous effects. This common structural defect can easily be avoided by providing adequate separation between the column and infill for the column to deform freely during seismic response without restraint from the infill.

Unless adequately designed for the levels of flexural ductility, and shear force expected under strong ground shaking, flexural or shear failures may develop in structural walls forming the primary lateral force resistance of buildings. An example from the 1990 Philippine earthquake is shown in Fig. 1.4, where failure has occurred at the level corresponding to a significant reduction in the stiffness and strength of the lateral force resisting system—a common location for concentration of damage.

Spandrel beams coupling structural walls are often subjected to high ductility demands and high shear forces as a consequence of their short length. It is very difficult to avoid excessive strength degradation in such elements, as shown in the failure of the McKinley Building during the 1964 Alaskan earthquake, depicted in Fig. 1.5, unless special detailing measures are adopted involving diagonal reinforcement in the spandrel beams.

Figure 1.6 shows another common failure resulting from "nonstructural" masonry infills in a reinforced concrete frame. The stiffening effect of the infill attracts higher seismic forces to the infilled frame, resulting in shear

Fig. 1.6 Failure of lower level of masonry-infilled reinforced concrete frame (1990 Philippine earthquake). (Courtesy of EQE Engineering Inc.)

failure of the infill, followed by damage or failure to the columns. As with the partial height infill of Fig. 1.3, the effect of the nonstructural infill is to modify the lateral force resistance in a way not anticipated by the design.

The final example in Fig. 1.7 shows the failure of a beam–column connection in a reinforced concrete frame. The joint was not intended to become the weak link between the four components. Such elements are usually subjected to very high shear forces during seismic activity, and if inadequately reinforced this will result in excessive loss in strength and stiffness of the frame, and even collapse.

While there is something new to be learned from each earthquake, it may be said that the majority of structural lessons should have been learned. Patterns in observed earthquake damage have been identified and reported [B10, J2, M15, P20, S4, S10, U2, W4] for some time. Yet many conceptual, design, and construction mistakes, that are responsible for structural damage

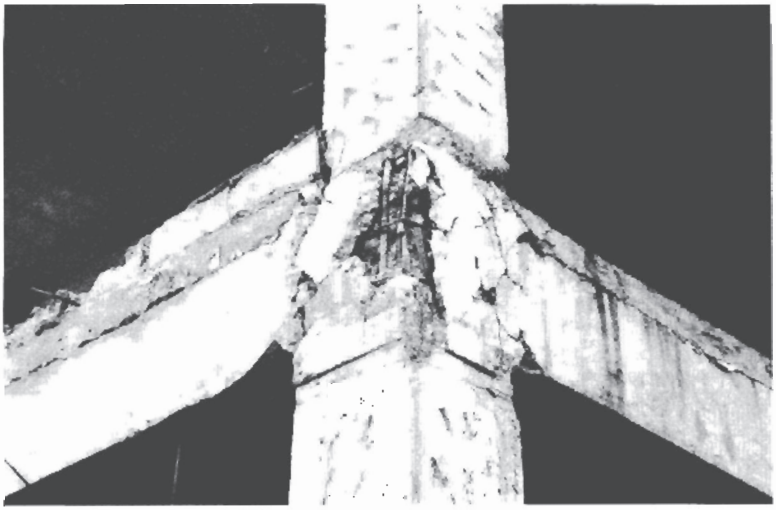

Fig. 1.7 Beam–column connection failure (1990 Philippine earthquake). (Courtesy of T. Minami.)

in buildings are being repeated. Many of these originate from traditional building configurations and construction practices, the abandonment of which societies or the building industry of the locality are reluctant to accept. There is still widespread lack of appreciation of the predictable and quantifiable effects of earthquakes on buildings and the impact of seismic phenomena on the philosophy of structural design.

Well-established techniques, used to determine the safe resistance of structures with respect to various static loads, including wind forces, cannot simply be extended and applied to conditions that arise during earthquakes. Although many designers prefer to assess earthquake-induced structural actions in terms of static equivalent loads or forces, it must be appreciated that actual seismic response is dynamic and is related primarily to imposed deformation rather than forces. To accommodate large seismically induced deformations, most structures need to be ductile. Thus in the design of structures for earthquake resistance, it is preferable to consider forces generated by earthquake-induced displacements rather than traditional loads. Because the magnitudes of the largest seismic-displacement-generated forces in a ductile structure will depend on its capacity or strength, the evaluation of the latter is of importance.

1.1.1 Seismic Design Limit States

It is customary to consider various levels of protection, each of which emphasizes a different aspect to be considered by the designer. Broadly, these relate to preservation of functionality, different degrees of efforts to minimize damage that may be caused by a significant seismic event, and the prevention of loss of life.

The degree to which levels of protection can be afforded will depend on the willingness of society to make sacrifices and on economic constraints within which society must exist. While regions of seismicity are now reasonably well defined, the prediction of a seismic event within the projected lifespan of a building is extremely crude. Nevertheless, estimates must be made for potential seismic hazards in affected regions in an attempt to optimize between the degree of protection sought and its cost. These aspects are discussed further in Chapter 2.

(a) Serviceability Limit State Relatively frequent earthquakes inducing comparatively minor intensity of ground shaking should not interfere with functionality, such as the normal operation of a building or the plant it contains. This means that no damage needing repair should occur to the structure or to nonstructural components, including contents. The appropriate design effort will need to concentrate on the control and limitation of displacements that could occur during the anticipated earthquake, and to ensure adequate strengths in all components of the structure to resist the earthquake-induced forces while remaining essentially elastic. Reinforced concrete and masonry structures may develop considerable cracking at the serviceability limit state, but no significant yielding of reinforcement, resulting in large cracks, nor crushing of concrete or masonry should result. The frequency with which the occurrence of an earthquake corresponding to the serviceability limit state may be anticipated will depend on the importance of preserving functionality of the building. Thus, for office buildings, the serviceability limit state may be chosen to correspond to a level of shaking likely to occur, on average, once every 50 years (i.e., a 50-year-return-period earthquake). For a hospital, fire station, or telecommunications center, which require a high degree of protection to preserve functionality during an emergency, an earthquake with a much longer return period will be appropriate.

(b) Damage Control Limit State For ground shaking of intensity greater than that corresponding to the serviceability limit state, some damage may occur. Yielding of reinforcement may result in wide cracks that require repair measures, such as injection grouting, to avoid later corrosion problems. Also, crushing or spalling of concrete may occur, necessitating replacement of unsound concrete. A second limit state may be defined which marks the boundary between economically repairable damage and damage that is irreparable or which cannot be repaired economically. Ground shaking of intensity likely to induce response corresponding to the damage control limit state should have a low probability of occurrence during the expected life of the building. It is expected that after an earthquake causes this or lesser intensity of ground shaking, the building can be successfully repaired and reinstated to full service.

(c) ***Survival Limit State*** In the development of modern seismic design strategies, very strong emphasis is placed on the criterion that loss of life should be prevented even during the strongest ground shaking feasible for the site. For this reason, particular attention must be given to those aspects of structural behavior that are relevant to this single most important design consideration: survival. For most buildings, extensive damage to both the structure and building contents, resulting from such severe but rare events, will have to be accepted. In some cases this damage will be irreparable, but collapse must not occur. Unless structures are proportioned to possess exceptionally large strength with respect to lateral forces, usually involving significant cost increase, inelastic deformations during large seismic events are to be expected. Therefore, the designer will need to concentrate on structural qualities which will ensure that for the expected duration of an earthquake, relatively large displacements can be accommodated without significant loss in lateral force resistance, and that integrity of the structure to support gravity loads is maintained. In this book, particular emphasis is placed on these aspects of structural response.

It must be appreciated that the boundaries between different intensities of ground shaking, requiring each of the foregoing three levels of protection to be provided cannot be defined precisely. A much larger degree of uncertainty is involved in the recommendations of building codes to determine the intensities of lateral seismic design forces than for any other kind of loading to which a building might be exposed. The capacity design process, which is developed in this book, aims to accommodate this uncertainty. To achieve this, structural systems must be conceived which are tolerant to the crudeness in seismological predictions.

1.1.2 Structural Properties

The specific structural properties that need to be considered in conjunction with the three levels of seismic protection described in the preceding section are described below.

(a) ***Stiffness*** If deformations under the action of lateral forces are to be reliably quantified and subsequently controlled, designers must make a realistic estimate of the relevant property—stiffness. This quantity relates loads or forces to the ensuing structural deformations. Familiar relationships are readily established from first principles of structural mechanics, using geometric properties of members and the modulus of elasticity for the material. In reinforced concrete and masonry structures these relationships are, however, not quite as simple as an introductory text on the subject may

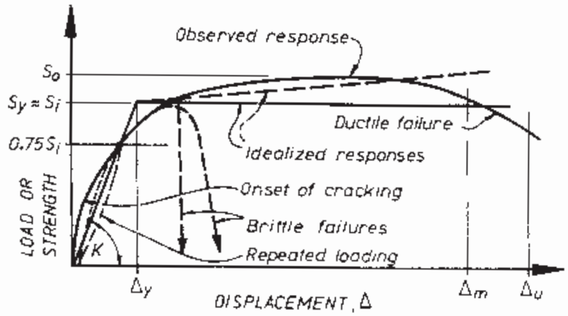

Fig. 1.8 Typical load–displacement relationship for a reinforced concrete element.

suggest. If serviceability criteria are to be satisfied with a reasonable degree of confidence, the extent and influence of cracking in members and the contribution of concrete or masonry in tension must be considered, in conjunction with the traditionally considered aspects of section and element geometry, and material properties.

A typical nonlinear relationship between induced forces or loads and displacements, describing the response of a reinforced concrete component subjected to monotonically increasing displacements, is shown in Fig. 1.8. For purposes of routine design computations, one of the two bilinear approximations may be used, where S_y defines the yield or ideal strength S_i of the member. The slope of the idealized linear elastic response, $K = S_y/\Delta_y$ is used to quantify stiffness. This should be based on the effective secant stiffness to the real load–displacement curve at a load of about $0.75S_y$, as shown in Fig. 1.8, as it is effective stiffness at close to yield strength that will be of concern when estimating response for the serviceability limit state. Under cyclic loading at high "elastic" response levels, the initial curved load–displacement characteristic will modify to close to the linear relationship of the idealized response. An early task within the design process will be the checking of typical interstory deflections (drift), using realistic stiffness values to satisfy local requirements for serviceability [Section 1.1.1(*a*)].

(b) Strength If a concrete or masonry structure is to be protected against damage during a selected or specified seismic event, inelastic excursions during its dynamic response should be prevented. This means that the structure must have adequate strength to resist internal actions generated during the elastic dynamic response of the structure. Therefore, the appropriate technique for the evaluation of earthquake-induced actions is an elastic analysis, based on stiffness properties described in the preceding section. These seismic actions, combined with those due to other loads on the structure, such as gravity, will lead, perhaps with minor modifications, to the proportioning of structural members. Thereby the designer can provide the desired strength, shown as S_i in Fig. 1.8, in terms of resistance to lateral forces envisaged.

(c) Ductility To minimize major damage and to ensure the survival of buildings with moderate resistance with respect to lateral forces, structures must be capable of sustaining a high proportion of their initial strength when a major earthquake imposes large deformations. These deformations may be well beyond the elastic limit. This ability of the structure or its components, or of the materials used to offer resistance in the inelastic domain of response, is described by the general term *ductility*. It includes the ability to sustain large deformations, and a capacity to absorb energy by hysteretic behavior, as discussed in Section 2.3.3. For this reason it is the single most important property sought by the designer of buildings located in regions of significant seismicity.

The limit to ductility, as shown for example in Fig. 1.8 by the displacement of Δ_u, typically corresponds to a specified limit to strength degradation. Although attaining this limit is sometimes termed *failure*, significant additional inelastic deformations may still be possible without structural collapse. Hence a ductile failure must be contrasted with brittle failure, represented in Fig. 1.8 by the dashed curves. Brittle failure implies near-complete loss of resistance, often complete disintegration, and the absence of adequate warning. For obvious reasons, brittle failure, which may be said to be the overwhelming cause for the collapse of buildings in earthquakes, and the consequent loss of lives, must be avoided. More precise definitions for the essential characteristics of ductility are given in Section 3.5.

Ductility is defined by the ratio of the total imposed displacements Δ at any instant to that at the onset of yield Δ_y. Using the idealizations of Fig. 1.8, this is

$$\mu = \Delta/\Delta_y > 1 \tag{1.1}$$

The displacements Δ_y and Δ in Eq. (1.1) and Fig. 1.8 may represent strain, curvature, rotation, or deflection. The ductility developed when failure is imminent is, from Fig. 1.8, $\mu_u = \Delta_u/\Delta_y$. Ductility is the structural property that will need to be relied on in most buildings if satisfactory behavior under damage control and survival limit state is to be achieved. An important consideration in the determination of the required seismic resistance will be that the estimated maximum ductility demand during shaking, $\mu_m = \Delta_m/\Delta_y$ (Fig. 1.8), does not exceed the ductility potential μ_u.

The roles of both stiffness and strength, as well as their quantification, are well established. The sources, development, quantification, and utilization of ductility, to serve best the designer's intent, are generally less well understood. For this reason many aspects of ductile structural response are examined in considerable detail in this book.

Structures which, because of their nature or importance to society, are to be designed to respond elastically, even during the largest expected seismic event, may be readily designed with the well-established tools of structural mechanics. Hence these structures will receive only superficial mention in the remainder of the book.

Ductility in structural members can be developed only if the constituent material itself is ductile. Thus it is relatively easy to achieve the desired ductility if resistance is to be provided by steel in tension. However, precautions need to be taken when steel is subject to compression, to ensure that premature buckling does not interfere with the development of the desired large inelastic strains in compression.

Concrete and masonry are inherently brittle materials. Although their tensile strength cannot be relied on as a primary source of resistance, they are eminently suited to carry compression stresses. However, the maximum strains developed in compression are rather limited unless special precautions are taken. The primary aim of the detailing of composite structures consisting of concrete or masonry and steel is to combine these materials in such a way as to produce ductile members, which are capable of meeting the inelastic deformation demands imposed by severe earthquakes.

In the context of reinforced concrete and masonry structures, *detailing* refers to the preparation of placing drawings, reinforcing bar configurations, and bar lists that are used for fabrication and placement of reinforcement in structures. But detailing also incorporates a design process by which the designer ensures that each part of the structure can perform safely under service load conditions and also when specially selected critical regions are to accommodate large inelastic deformations. Particularly, it covers such aspects as the choice of bar sizes, the distribution of bars, curtailment and splice details of flexural reinforcement, and the size, spacing, configuration, and anchorage of transverse reinforcement, intended to provide shear strength and ductility to critical regions. Detailing based on an understanding of and a feeling for structural behavior, on the appreciation of changing demands of economy, and on the limitations of construction practices is at least as important as other attributes in the art of structural design [P1].

1.2 ESSENTIALS OF STRUCTURAL SYSTEMS FOR SEISMIC RESISTANCE

All structural systems are not created equal when response to earthquake-induced forces is of concern. Aspects of structural configuration, symmetry, mass distribution, and vertical regularity must be considered, and the importance of strength, stiffness, and ductility in relation to acceptable response appreciated. The first task of the designer will be to select a structural system most conducive to satisfactory seismic performance within the constraints dictated by architectural requirements. Where possible, architect and structural engineer should discuss alternative structural configurations at the earliest stage of concept development to ensure that undesirable geometry is not locked-in to the system before structural design begins.

Irregularities, often unavoidable, contribute to the complexity of structural behavior. When not recognized, they may result in unexpected damage and even collapse. There are many sources of structural irregularities. Drastic changes in geometry, interruptions in load paths, discontinuities in both strength and stiffness, disruptions in critical regions by openings, unusual proportions of members, reentrant corners, lack of redundancy, and interference with intended or assumed structural deformations are only a few of the possibilities. The recognition of many of these irregularities and of conceptions for remedial measures for the avoidance or mitigation of their undesired effects rely on sound understanding of structural behavior. Awareness to search for undesired structural features and design experience are invaluable attributes. The relative importance of some irregularities may be quantified. In this respect some codes provide limited guidance. Examples for estimating the criticality of vertical and horizontal irregularities in framed buildings are given in Section 4.2.6. Before the more detailed discussion of these aspects later in this chapter, it is, however, necessary to review some general aspects of seismic forces and structural systems.

1.2.1 Structural Systems for Seismic Forces

The primary purpose of all structures used for building is to support gravity loads. However, buildings may also be subjected to lateral forces due to wind or earthquakes. The taller a building, the more significant the effects of lateral forces will be. It is assumed here that seismic criteria rather than wind or blast forces govern the design for lateral resistance of buildings. Three types of structures, most commonly used for buildings, are considered in this book.

(a) Structural Frame Systems Structures of multistory reinforced concrete buildings often consist of frames. Beams, supporting floors, and columns are continuous and meet at nodes, often called "rigid" joints. Such frames can readily carry gravity loads while providing adequate resistance to horizontal forces, acting in any direction [B4]. Chapter 4 deals with the design of reinforced concrete ductile frames.

(b) Structural Wall Systems When functional requirements permit it, resistance to lateral forces may be assigned entirely to structural walls, using reinforced concrete or masonry [B4]. Gravity load effects on such walls are seldom significant and they do not control the design. Usually, there are also other elements within such a building, which are assigned to carry only gravity loads. Their contribution to lateral force resistance, if any, is often neglected. In Chapter 5 we present various structural aspects of buildings in which the resistance to lateral forces is assigned entirely to structural walls. The special features of reinforced masonry, particularly suited for the construction of walls that resist both gravity loads and lateral forces, are presented in Chapter 7.

(c) Dual Systems Finally, dual building systems are studied briefly in Chapter 6. In these, reinforced concrete frames interacting with reinforced concrete or masonry walls together provide the necessary resistance to lateral forces, while each system carries its appropriate share of the gravity load. These types of structures are variously known as dual, hybrid, or wall–frame structures.

The selection of structural systems for buildings is influenced primarily by the intended function, architectural considerations, internal traffic flow, height and aspect ratio, and to a lesser extent, the intensity of loading. The selection of a building's configuration, one of the most important aspects of the overall design [A4], may impose severe limitations on the structure in its role to provide seismic protection. Because the intent is to present design concepts and principles, rather than a set of solutions, various alternatives within each of these three groups of distinct structural systems, listed above, will not be considered. Some structural forms are, however, deliberately omitted. For example, construction consisting of flat slabs supported by columns is considered to be unsuitable on its own to provide satisfactory performance under seismic actions because of excessive lateral displacements and the difficulty to providing the adequate and dependable shear transfer between columns and slabs, necessary to sustain lateral forces, in addition to gravity loads.

Sufficient information for both the design and detailing of components of the principal structural systems is provided in subsequent chapters to allow easy adaptation of any principle to other structural forms, for example those using precast concrete components, which may occur in buildings. Valuable information in this respect may be obtained from a report by ACI-ASCE Committee 442 on the response of concrete buildings to lateral forces [A14].

1.2.2 Gross Seismic Response

(a) Response in Elevation: The Building as a Vertical Cantilever When subjected to lateral forces only, a building will act as a vertical cantilever. The resulting total horizontal force and the overturning moment will be transmitted at the level of the foundations. Once the lateral forces, such as may act at each level of the building, are known, the story shear forces, as well as the magnitude of overturning moments at any level, shown in Fig. 1.9, can readily be derived from usual equilibrium relationships. For example, in Fig. 1.9(a), the sum V_j of all floor forces acting on the shaded portion of the building

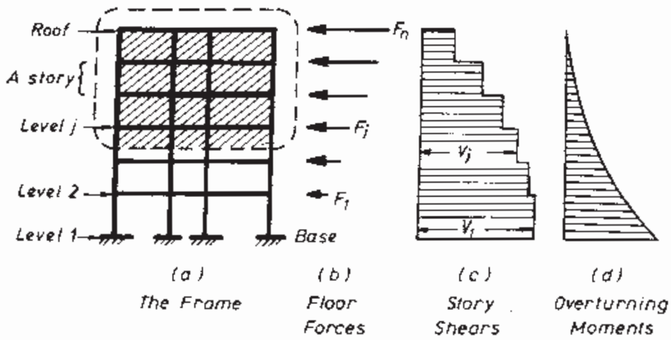

Fig. 1.9 Effects of lateral forces on a building.

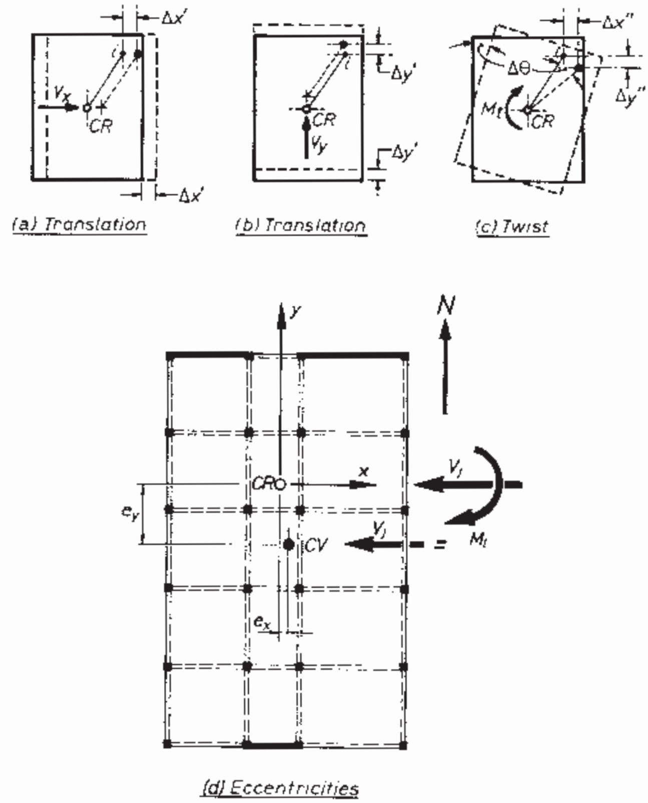

Fig. 1.10 Relative floor displacement.

must be resisted by shear and axial forces and bending moments in the vertical elements in the third story.

In the description of multistory buildings in this book, the following terminology is used. All structures are assumed to be founded at the base or level 1. The position of a floor will be identified by its level above the base. Roof level is identical with the top level. The space or vertical distance between adjacent levels is defined as a story. Thus the first story is between levels 1 and 2, and the top story is that below roof level (Fig. 1.9).

(b) Response in Plan: Centers of Mass and Rigidity The structural system may consist of a number of frames, as shown in Fig. 1.9(a), or walls, or a combination of these, as described in Section 1.2.1 [Fig. 1.10(d)]. The position of the resultant force V_j in the horizontal plane will depend on the plan distribution of vertical elements, and it must also be considered. As a consequence, two important concepts must be defined. These will enable the effects of building configurations on the response of structural systems to lateral forces to be better appreciated. The evaluation of the effects of lateral forces, such as shown in Fig. 1.9(a), on the structural systems described in Section 1.2.1 is given in Chapters 4 through 6.

(i) Center of Mass: During an earthquake, acceleration-induced inertia forces will be generated at each floor level, where the mass of an entire story may be assumed to be concentrated. Hence the location of a force at a particular level will be determined by the center of the accelerated mass at that level. In regular buildings, such as shown in Fig. 1.10(d), the positions of the centers of floor masses will differ very little from level to level. However, irregular mass distribution over the height of a building may result in variations in centers of masses, which will need to be evaluated. The summation of all the floor forces, F_j in Fig. 1.9(a), above a given story, with due allowance for the in-plane position of each, will then locate the position of the resultant force V_j within that story. For example, the position of the shear force V_j within the third story is determined by point CV in Fig. 1.10(d), where this shear force is shown to act in the east–west direction. Depending on the direction of an earthquake-induced acceleration at any instant, the force V_j passing through this point may act in any direction. For a building of the type shown in Fig. 1.10(d), it is sufficient, however, to consider seismic attacks only along the two principal axes of the plan.

(ii) Center of Rigidity: If, as a result of lateral forces, one floor of the building in Fig. 1.9 translates horizontally as a rigid body relative to the floor below, as shown in Fig. 1.10(a), a constant interstory displacement $\Delta x'$ will be imposed on all frames and walls in that story. Therefore, the induced forces in these elastic frames and walls, in the relevant east–west planes, will be proportional to the respective stiffnesses. The resultant total force, $V_j = V_x$, induced by the translational displacements $\Delta x'$, will pass through the center

of rigidity (CR) in Fig. 1.10(d). Similarly, a relative floor translation to the north, shown as $\Delta y'$ in Fig. 1.10(b), will induce corresponding forces in each of the four frames [Fig. 1.10(d)], the resultant of which, V_y, will also pass through point CR. This point, defined as the center of rigidity or center of stiffness, locates the position of a story shear force V_j, which will cause only relative floor translations.

The position of the center of rigidity may be different in each story. It is relevant to story shear forces applied in any direction in a horizontal plane. Such a force may be resolved into components, such as V_x and V_y shown in Fig. 1.10(a) and (b), which will cause simultaneous story translations $\Delta x'$ and $\Delta y'$, respectively.

Since the story shear force V_j in Fig. 1.10(d) acts through point CV rather than the center of rigidity CR, it will cause floor rotation as well as relative floor translation. For convenience, V_j may be replaced by an equal force acting through CR, thus inducing pure translation, and a moment $M_t = e_y V_j$ about CR, leading to rigid floor rotation, as shown in Fig. 1.10(c). The angular rotation $\Delta\theta$ is termed *story twist*. It will cause additional interstory displacements $\Delta x''$ and $\Delta y''$ in lateral force resisting elements in both principal directions, x and y. The displacements due to story twist are proportional to the distance of the element from the center of rotation, [i.e., the center of rigidity (CR)].

Displacements due to story twist, when combined with those resulting from floor translations, can result in total element interstory displacements that may be difficult to accommodate. For this reason the designer should attempt to minimize the magnitude of story torsion M_t. This may be achieved by a deliberate assignment of stiffnesses to lateral force-resisting components, such as frames or walls, in such a way as to minimize the distance between the center of rigidity (CR) and the line of action of the story shear force (CV). To achieve this in terms of floor forces, the distance between the center of rigidity and the center of mass should be minimized.

1.2.3 Influence of Building Configuration on Seismic Response

An aspect of seismic design of equal if not greater importance than structural analysis is the choice of building configuration [A4]. By observing the following fundamental principles, relevant to seismic response, more suitable structural systems may be adopted.

1. Simple, regular plans are preferable. Building with articulated plans such as T and L shapes should be avoided or be subdivided into simpler forms (Fig. 1.11).
2. Symmetry in plan should be provided where possible. Gross lack of symmetry may lead to significant torsional response, the reliable prediction of which is often difficult. Much greater damage due to earth-

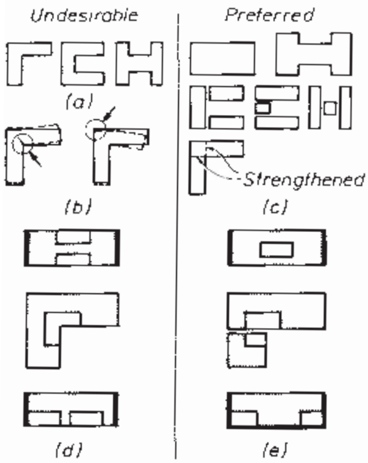

Fig. 1.11 Plan configurations in buildings.

quakes has been observed in buildings situated at street corners, where structural symmetry is more difficult to achieve, than in those along streets, where a more simple rectangular and often symmetrical structural plan could be utilized.

3. An integrated foundation system should tie together all vertical structural elements in both principal directions. Foundations resting partly on rock and partly on soils should preferably be avoided.
4. Lateral-force-resisting systems within one building, with significantly different stiffnesses such as structural walls and frames, should be arranged in such a way that at every level symmetry in lateral stiffness is not grossly violated. Thereby undesirable torsional effects will be minimized.
5. Regularity should prevail in elevation, in both the geometry and the variation of story stiffnesses.

The principles described above are examined in more detail in the following sections.

(a) Role of the Floor Diaphragm Simple and preferably symmetrical building plans hold the promise of more efficient and predictable seismic response of each of the structural components. A prerequisite for the desirable interaction within a building of all lateral-force-resisting vertical components of the structural system is an effective and relatively rigid interconnection of these components at suitable levels. This is usually achieved with the use of floor systems, which generally possess large in-plane stiffness. Vertical elements will thus contribute to the total lateral force resistance, in proportion to their own stiffness. With large in-plane stiffness, floors can act as diaphragms. Hence a close to linear relationship between the horizontal displacements of the various lateral-force-resisting vertical structural elements will exist at every level. From rigid-body translations and rotations, shown in Fig. 1.10, the relative displacements of vertical elements can readily be derived. This is shown for frames in Appendix A.

Another function of a floor system, acting as a diaphragm, is to transmit inertia forces generated by earthquake accelerations of the floor mass at a given level to all horizontal-force-resisting elements. At certain levels, particularly in lower storys, significant horizontal forces from one element, such as a frame, may need to be transferred to another, usually stiffer element, such as a wall. These actions may generate significant shear forces and bending moments within a diaphragm. In squat rectangular diaphragms, the resulting stresses will be generally insignificant. However, this may not be the case when long or articulated floor plans, such as shown in Fig. 1.11(a) have to be used. The correlation between horizontal displacements of vertical elements [Fig. 1.11(b)] will be more difficult to establish in such cases. Reentrant corners, inviting stress concentrations, may suffer premature damage. When such configurations are necessary, it is preferable to provide structural separations. This may lead to a number of simple, compact, and independent plans, as shown in Fig. 1.11(c). Gaps separating adjacent structures must be large enough to ensure that even during a major seismic event, no hammering of adjacent structures will occur due to out-of-phase relative motions of the independent substructures. Inelastic deflections, resulting from ductile dynamic response, must be allowed for.

Diaphragm action may be jeopardized if openings, necessary for vertical traffic within a multistory building or other purposes, significantly reduce the ability of the diaphragm to resist in-plane flexure of shear, as seen in examples in Fig. 1.11(d). The relative importance of openings may be estimated readily from a simple evaluation of the flow of forces within the diaphragm, necessary to satisfy equilibrium criteria. Preferred locations for such openings are suggested in Fig. 1.11(e).

As a general rule, diaphragms should be designed to respond elastically, as they are not suitable to dissipate energy through the formation of plastic regions. Using capacity design principles, to be examined subsequently, it is relatively easy to estimate the magnitudes of the largest forces that might be introduced to diaphragms. These are usually found to be easily accommodated. Other aspects of diaphragms, including flexibility, are discussed in Section 6.5.3.

(b) Amelioration of Torsional Effects It was emphasized in Section 1.2.2 that to avoid excessive displacements in lateral-force-resisting components that are located in adverse positions within the building plan, torsional effects should be minimized. This is achieved by reducing the distance between the center of mass (CM), where horizontal seismic floor forces are applied, and the center of rigidity (CR) (Fig. 1.10). A number of examples for both

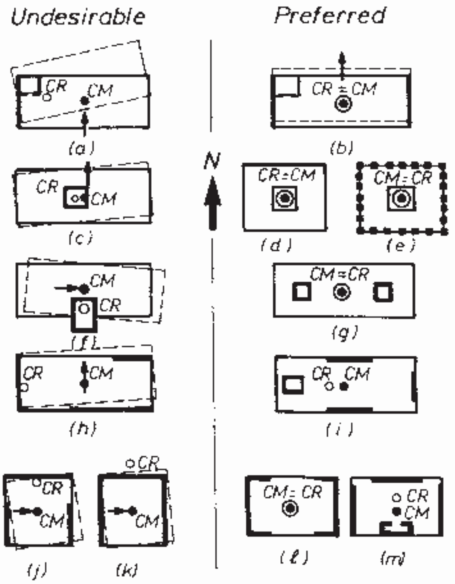

Fig. 1.12 Mass and lateral stiffness relationship with floor plans. (The grid of frames in each plan, required primarily for gravity loads, is not shown.)

undesirable positioning of major lateral-force-resisting elements, consisting of structural walls and frames, and for the purpose of comparison, preferred locations, are given in Fig. 1.12. For the sake of clarity the positioning of frames required solely for gravity load resistance within each floor plan is generally not shown. While the primary role of the frames in these examples will be the support of gravity load, it must be appreciated that frames will also contribute to both lateral force resistance and torsional stiffness.

Figure 1.12(*a*) shows that because of the location of a stiff wall at the west end of a building, very large displacements, as a result of floor translations and rotations (Fig. 1.10), will occur at the east end. As a consequence, members of a frame located at the east end may be subjected to excessive inelastic deformations (ductility). Excessive ductility demands at such a location may cause significant degradation of the stiffness of a frame. This will lead to further shift of the center of rigidity and consequently to an amplification of torsional effects. A much improved solution, shown in Fig. 1.12(*b*), where the service core has been made nonstructural and a structural wall added at the east end will ensure that the centers of mass and stiffness virtually coincide. Hence only dominant floor translations, imposing similar ductility demands on all lateral force resisting frames or walls, are to be expected.

Analysis may show that in some buildings torsional effects [Fig. 1.12(*c*)] may be negligible. However, as a result of normal variations in material properties and section geometry, and also due to the effects of torsional components of ground motion, torsion may arise also in theoretically perfectly symmetrical buildings. Hence codes require that allowance be made in all buildings for so-called "accidental" torsional effects.

Although a reinforced concrete or masonry core, such as shown in Fig. 1.12(c), may exhibit good torsional strength, its torsional stiffness, particularly after the onset of diagonal cracking, may be too small to prevent excessive deformations at the east and west ends of the building. Similar twists may lead, however, to acceptable displacements at the perimeter of square plans with relatively large cores, seen in Fig. 1.12(d). Closely placed columns, interconnected by relatively stiff beams around the perimeter of such buildings [Fig. 1.12(e)], can provide excellent control of torsional response. The eccentrically placed service core, shown in Fig. 1.12(f), may lead to excessive torsional effects under seismic attack in the east–west direction unless perimeter lateral force resisting elements are present to limit torsional displacements.

The advantages of the arrangement, shown in Fig. 1.12(g), in terms of response to horizontal forces are obvious. While the locations of the walls in Fig. 1.12(h), to resist lateral forces, it satisfactory, the large eccentricity of the center of mass with respect to the center of rigidity will result in large torsion when lateral forces are applied in the north–south direction. The placing of at least one stiff element at or close to each of the four sides of the buildings, as shown in Fig. 1.12(i), provides a particularly desirable structural arrangement. Further examples, showing wall arrangements with large eccentricities and preferred alternative solutions, are given in Fig. 1.12(j) to (m). Although large eccentricities are indicated in the examples of Fig. 1.12(j) and (k), both stiffness and the strength of these walls may well be adequate to accommodate torsional effects.

The examples of Fig. 1.12 apply to structures where walls provide the primary lateral load resistance. The principles also apply to framed systems, although it is less common for excessive torsional effects to develop in frame structures.

(c) Vertical Configurations A selection of undesirable and preferred configurations is illustrated in Fig. 1.13. Tall and slender buildings [Fig. 1.13(a)] may require large foundations to enable large overturning moments to be transmitted in a stable manner. When subjected to seismic accelerations, concentration of masses at the top of a building [Fig. 1.13(b)] will similarly impose heavy demands on both the lower stories and the foundations of the structure. In comparison, the advantages of building elevations as shown in Fig. 1.13(c) and (d) are obvious.

An abrupt change in elevation, such as shown in Fig. 1.13(e), also called a *setback*, may result in the concentration of structural actions at and near the level of discontinuity. The magnitudes of such actions, developed during the dynamic response of the building, are difficult to predict without sophisticated analytical methods. The separation into two simple, regular structural systems, with adequate separation [Fig. 1.13(f)] between them, is a prefer-

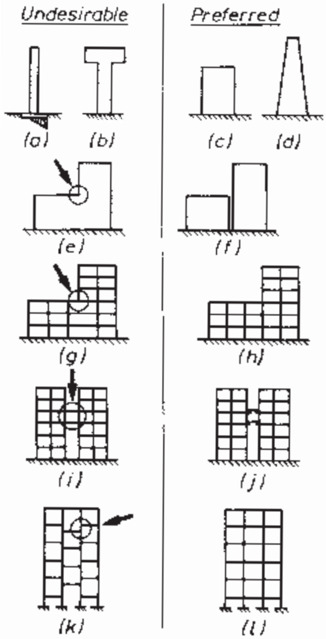

Fig. 1.13 Vertical configurations.

able alternative. Irregularities within the framing system, such as a drastic interference with the natural flow of gravity loads and that of lateral-force-induced column loads at the center of the frame in Fig. 1.13(g), must be avoided.

Although two adjacent buildings may appear to be identical, there is no assurance that their response to ground shaking will be in phase. Hence any connections (bridging) between the two that may be desired [Fig. 1.13(i)] should be such as to prevent horizontal force transfer between the two structures [Fig. 1.13(j)].

Staggered floor arrangements, as seen in Fig. 1.13(k), may invalidate the rigid interconnection of all vertical lateral-force-resisting units, the importance of which was emphasized in Section 1.2.3(a). Horizontal inertia forces, developed during dynamic response, may impose severe demands, particularly on the short interior columns. While such frames [Fig. 1.13(k)] may be readily analyzed for horizontal static forces, results of analyses of their inelastic dynamic response to realistic ground shaking should be treated with suspicion.

Major deviations from a continuous variation with height of both stiffness and strength are likely to invite poor and often dangerous structural response. Because of the abrupt changes of story stiffnesses, suggested in Fig. 1.14(a) and (b), the dynamic response of the corresponding structures [Fig. 1.14(e) and (f)] may be dominated by the flexible stories. Reduced story

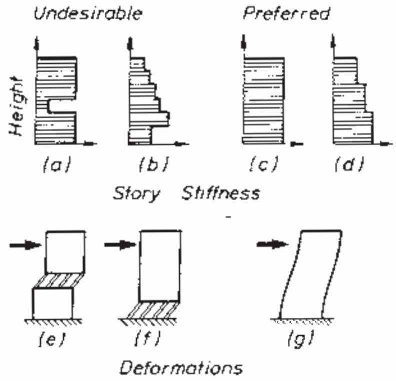

Fig. 1.14 Interacting frames and walls.

stiffness is likely to be accompanied by reduced strength, and this may result in the concentration of extremely large inelastic deformations [Fig. 1.14(*e*) and (*f*)] in such a story. This feature accounts for the majority of collapsed buildings during recent earthquakes. Constant or gradually reducing story stiffness and strength with height [Fig. 1.14(*c*), (*d*), and (*g*)] reduce the likelihood of concentrations of plastic deformations during severe seismic events beyond the capacities of affected members.

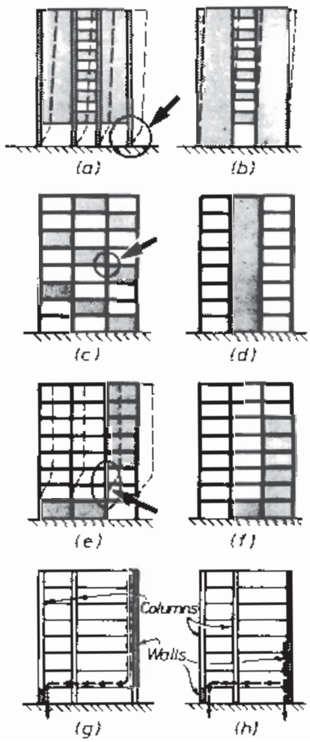

Fig. 1.15 Variation of story stiffness with height.

Examples of vertical irregularities in buildings using structural walls as primary lateral-force-resisting elements are shown in Fig. 1.15, together with suggested improvements. When a large open space is to be provided in the first story, designers are often tempted to terminate structural walls, which may extend over the full height of the building, at level 2 [Fig. 1.15(*a*)]. Unless other parallel walls, perhaps at the boundaries of the floor plan, are provided, a so-called *soft story* will develop. This is likely to impose ductility demands on columns which may well be beyond their ductility capacity. A continuation of the walls, interconnected by coupling beams at each floor, down to the foundations, shown in Fig. 1.15(*b*), will, on the other hand, result in one of the most desirable structural configurations. This system is examined in some detail in Section 5.6.

Staggered wall panels, shown in Fig. 1.15(*c*), may provide a stiff load path for lateral earthquake forces. However, the transmission of these forces at corners will make detailing of reinforcement, required for adequate ductility, extremely difficult. The assembly of all panels into one single cantilever [Fig. 1.15(*d*)] with or without interacting frames will, however, result in an excellent lateral force resisting system. The structural system of Fig. 1.15(*d*) is discussed in Chapter 6.

An interruption of walls over one or more intermediate stories [Fig. 1.15(*e*)] will invite concentrations of drift in those stories, as suggested in Fig. 1.14(*a*) and (*e*). Discontinuities of the type shown in Fig. 1.15(*f*), are, on the other hand, acceptable, as strength and stiffness distribution with height is compatible with the expected forces and displacements.

The side view of the structure shown in Fig. 1.15(*a*) may be as shown in Fig. 1.15(*g*). It shows that a major portion of the accumulated earthquake forces from upper levels, resulting in large shear at level 2 of the wall extending above that level, will need to be transferred to a very stiff short wall at the opposite side of the building. The arrows in Fig. 1.15(*g*) indicate the gross deviation of the path of internal forces leading to the foundation, which may impose excessive demands in both torsion in the first story and actions within the floor diaphragm. Both of these undesired effects will be alleviated if the tall wall terminates in the foundations [Fig. 1.15(*b*)], while sharing the base shear with a short wall, as shown in Fig. 1.15(*h*).

Another source of major damage, particularly in columns, repeatedly observed in earthquakes, is the interference with the natural deformations of members by rigid nonstructural elements, such as infill walls. As Fig. 1.16 shows, the top edge of a brick wall will reduce the effective length of one of the columns, thereby increasing its stiffness in terms of lateral forces. Since seismic forces are attracted in proportion to element stiffness, the column may thus attract larger horizontal shear forces than it would be capable of resisting. Moreover, a relatively brittle flexural failure may occur at a location (midheight) where no provision for the appropriate detailing of plastic regions would have been made. The unexpected failure of such major gravity-load-carrying elements may lead to the collapse of the entire building. Therefore, a very important task of the designer is to ensure, both in the

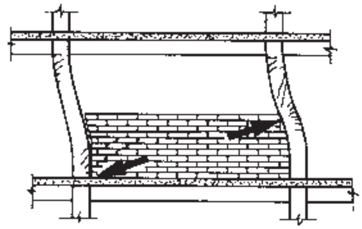

Fig. 1.16 Unintended interference with structural deformations.

design and during construction, that intended deformations, including those of primary lateral-force-resisting components in the inelastic range of seismic response, can take place without interference.

A wide range of irregularities, together with considerations of numerous other very important issues, relevant to the overall planning of buildings and the selection of suitable structural forms within common architectural constraints are examined in depth by Arnold and Reitherman [A4]. In the context of seismic design the observance of principles relevant to configurations is at least as important as those of structural analysis, the art in detailing for ductilities of critical regions, and the assurance of high quality in workmanship during construction.

1.2.4 Structural Classification in Terms of Design Ductility Level

It is possible to satisfy the performance criteria of the damage control and survival limit states of Section 1.1.1 by any one of three distinct design approaches, related to the level of ductility permitted of the structure. A qualitative illustration of these approaches is shown in Fig. 1.17, where

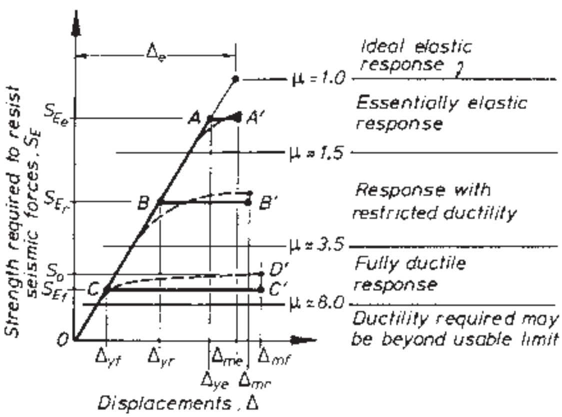

Fig. 1.17 Relationship between strength and ductility.

strength S_E, required to resist earthquake-induced forces, and structural displacements Δ at the development at different levels of strength are related to each other.

(a) Elastic Response Because of their great importance, certain buildings will need to possess adequate strength to ensure that they remain essentially elastic. Other structures, perhaps of lesser importance, may nevertheless possess a level of inherent strength such that elastic response is assured. The analysis and design of both categories of structures can be carried out with conventional procedures. Although the determination of the required resistance at each critical section will usually be based on the principles of strength design, implying that a plastic state is achieved at these sections, it is unlikely that inelastic deformations of any significance will be developed in the structure when the intensity of lateral design forces is attained. This is because specified (nominal) material strength properties and various strength reduction factors, considered in Sections 1.3.3 and 1.3.4, are used when members are being proportioned. The extra protection thus provided ensures that, at most, only insignificant inelastic deformations during an earthquake are expected. Hence the need for special detailing of potential plastic regions does not arise. Even with detailing practices established for structures in reinforced concrete and masonry, subjected only to gravity loads and wind forces, a certain amount of ductility can always be developed. As no special features arise in the design of these structures, even when subjected to design earthquake forces of the highest intensity, no further attention will be given them in this book.

The idealized response of such a structure is shown in Fig. 1.17 by the bilinear strength–displacement path OAA'. The maximum displacement Δ_{me} is very close to the displacement of the ideal elastic structure Δ_e and the displacement of the real structure Δ_{ye} at the onset of yielding.

(b) Ductile Response Most ordinary buildings are designed to resist lateral seismic forces which are smaller than those that would be developed in an elastically responding structure, implying, as Fig. 1.17 shows, that inelastic deformations and hence ductility will be required of the structure. Depending on the force level adopted for strength design, the level of ductility required may vary from insignificant, requiring no special detailing, to considerable, requiring most careful consideration of detailing. It is convenient to divide ductile responding structures into the two subcategories discussed below.

(i) Fully Ductile Structures: These are designed to possess the maximum ductility potential that can reasonably be achieved at carefully identified and detailed inelastic regions. A full consideration must be given to the effects of

dynamic response, using simplified design procedures, to ensure that nonductile modes or undesirable location of inelastic deformation cannot occur.

The idealized bilinear response of this type of structure is shown in Fig. 1.17 by the path OCC'. The magnitude of ductility implied is $\mu_f = \Delta_{mf}/\Delta_{yf}$, where Δ_{mf} and Δ_{yf} are the maximum expected and yield displacements, respectively, and S_{Ef} is the required strength of the fully ductile system. Details of these relationships are also shown in Fig. 1.8. A more realistic strength–displacement path is the curve OD', which shows that the strength S_o, developed at maximum displacement Δ_{mf}, may in fact be larger than the required strength S_{Ef}, used for design purposes. The principal aim of this book is to describe design procedures and detailing techniques for such fully ductile structures.

(ii) Structures with Restricted Ductility: Certain structures inherently possess significant strength with respect to lateral forces as a consequence, for example, of the presence of large areas of structural walls. It may well be that very little strength, if any, in addition to that obtained for the resistance of gravity loads and wind forces would need to be provided to achieve seismic resistance corresponding with elastic response. In other buildings, because of less than ideal structural configurations, it may be difficult to develop large ductilities, which would allow the use of low-intensity seismic design forces. Instead, it may be possible to provide greater resistance to lateral forces with relative ease, to reduce ductility demands. These are structures with *restricted ductility*, sometimes termed *limited ductility*.

An example of the response of a structure with restricted ductility is shown in Fig. 1.17 by the curve OBB'. It shows that the displacement ductility demand is $1 < \mu_r = \Delta_{mr}/\Delta_{yr} < \mu_f$. A more realistic response is shown by the dashed curve. Because requirements for the seismic resistance of such structures are seldom critical, the use of less onerous and simpler design procedures, combined with simpler detailing requirements for components, are recommended by some codes [A1, X3].

It should be appreciated that precise limits cannot be set for structures with full and reduced ductility. The transition from one system to the other will be gradual. Figure 1.17 shows approximate values of ductility factors $\mu = \Delta_m/\Delta_y$ which may be used as guides for the limits of the categories discussed. Although displacement ductilities in excess of 8 can be developed in some well-detailed reinforced concrete structures, the associated maximum displacements Δ_{mf} are likely to be beyond limits set by other design criteria, such as structural stability. Elastically responding structures, implying no or negligible ductility demands, represent the other limit. This suggests that a procedure, incorporating transitions in its requirements for fully ductile to those of elastically responding structures, may be used when designing structures with reduced ductility. This is examined in Chapter 8.

1.3 DEFINITION OF DESIGN QUANTITIES

1.3.1 Design Loads and Forces

In this book the following distinction between loads and forces is made:

1. Loads results from the effect of gravity. Dead loads, live loads, and snow loads are typical examples.
2. Wind, earthquakes, or restraints against deformations, such as shrinkage or creep, lead to forces. Although the term *load* is often also used to describe these forces, it will be avoided herein.

Most codes give characteristic values for design loads or forces, when these cannot be derived simply from the weights of materials to be sustained or from first principles.

(a) Dead Loads (D) Dead loads result from the weight of the structure and all other permanently attached materials. Average or typical values for superimposed materials and loads are readily obtained from manuals. The characteristic feature of dead loads is that they are permanent. Codes prescribe certain variable loads, such as movable partition walls, to be considered as permanent. For the sake of safety, dead loads are customarily overestimated. This fact should be considered in seismic design, whenever gravity load effects enhance strength when combined with effects of seismic forces, such as may occur when estimating moment capacity of columns.

(b) Live Loads (L) Live loads result from expected usage. They may be movable and their intensity may vary. Maximum intensities specified by codes are based on probablistic estimates. In most cases they are simulated by uniformly distributed loads placed over the entire area of the floor. However, for certain areas with special use, point loads may also be specified.

The probability of an area being subjected to the maximum specified intensity of live load diminishes as the size of the loaded floor area increases. Floors used for offices are typical examples. While an element of floor slab must be designed to sustain the full intensity of the live load, a beam or a column, receiving live load from a considerable tributary floor area A, may be assumed to receive live loads with smaller intensity. A typical code [X8] recommendation is that

$$L_r = rL \tag{1.2}$$

where

$$r = 0.3 + 3/\sqrt{A} \leq 1.0 \tag{1.3}$$

and L = code specified live load, usually expressed in kPa (or psf)
L_r = reduced live load, assumed to be uniformly distributed over the area tributary to a beam, or to a column extending over the height of one of more stories
r = live-load reduction factor
A = total tributary area in m^2, not to be taken less than 20 m^2

Only the symbol L is used in this book, whenever reference to live load is made, but this will imply that reduced live load L_r will be substituted where appropriate. Equation (1.3) is appropriate for floor slabs of office buildings, but not for storage areas, which have a high probability of being subjected to close to their full design live load over extensive areas.

Dead and live loads need also be considered when an estimate is made for the equivalent mass of a building. For the purpose of estimating seismic acceleration-induced horizontal inertia forces, it is sufficient to assume that the mass of the floor system, including finishes, partitions, beams, and columns one-half story above and below a floor, and with a fraction of the live load acting over the entire floor of a story, is concentrated at the floor level of the structural model (i.e., at the center of the mass) [Section 1.2.2(*b*)]. In assessing inertia forces some codes ignore some types of live loads [X10], while others [X8] specify that a certain fraction, typically one-third of the code-specified intensity, be converted into an equivalent mass.

(c) Earthquake Forces (E) In Chapter 2 the various techniques of simulating the effects of earthquakes on structures for buildings are described. The design quantities and procedures used in this book are based on effects resulting from the application of equivalent static horizontal earthquake forces, the determination of which is given in Section 2.4.3. This technique is preferred for its simplicity, its reliability for most common regular building structures, and because most designers are familiar with its use. It will be seen, however, that the method of capacity design for ductile structures, the principal subject of this book, does not depend on the technique with which design earthquake effects have been derived.

(d) Wind Forces (W) It was shown in Section 1.2.4 that the design intensity of the horizontal earthquake forces, duly adjusted for potential ductility capacity, may be a small fraction of that which would be generated in the elastic structure by the motions of the design earthquake. Thus it may well be, particularly in the case of tall or rather flexible buildings, that code-specified wind, rather than earthquake forces, when combined with appropriate gravity loads, will control strength requirements for many or all components of the structure. While ductility requirements do not arise, or are negligible, in the case when wind forces dominate structural strength, they are of paramount importance if the satisfactory response of the building during a strong ground motion is to be assured. The intensity of horizontal forces, corresponding with the true elastic response of the building to design seismic excitation, may in fact be many times that of design wind force. For this reason the application of capacity design procedures is still relevant to the majority of multistory buildings, even though wind rather than earthquake forces may control strength requirements.

(e) Other Forces Other force effects, including those due to shrinkage, creep, and temperature, must be considered in conventional strength design. Although these have significance when considering the strength design of buildings during elastic response, they have little, if any relevance to structures responding with full or reduced ductility. This is because this category of force effects occurs as a result of small imposed displacements on the structure. The magnitude of forces induced in responding to these effects depends on the incremental stiffness. When fully ductile, or reduced ductile response is assured, the incremental stiffness for imposed displacements is negligible. The structural effect is no longer that of significant increase in force, but of insignificant decrease in ductility. For example, strains induced by thermal or shrinkage effects, when compensated for creep relaxation, will rarely exceed 0.0002. This is less than 7% of the dependable unconfined compression strain capacity of concrete and masonry, and a much smaller proportional of the compression strain capacity when confinement, as discussed in Section 3.2.2, is provided. Although the elastic compression force corresponding to a strain of 0.0002 may be as high as 20% of the compression strength, the significance for a ductile system decreases as ultimate strain capacity increases. As a consequence, it is unnecessary to consider strain-induced forces in conjunction with the ductile response of building systems, and such forces will be ignored in the following.

1.3.2 Design Combinations of Load and Force Effects

Design bending moments, shears, and axial forces are effects developed as a consequence of loads and forces in appropriate combinations. To this end an appropriate model of the real structure must be established which lends itself to rational analyses. Appropriate analytical models are discussed in the relevant chapters on structural types: in particular, Chapters 4 to 6. Loads and forces may be superimposed for structural analysis, or if the structure can reasonably be represented by elastic response, load effects may be superimposed.

Critical combinations of load and force effects to be considered for member design are based on the limit state for strength. The required strength, S_u, defined in Section 1.3.3(*a*), to be provided at any section of a member is thus

$$S_u = \gamma_D S_D + \gamma_L S_L + \gamma_E S_E \tag{1.4}$$

TABLE 1.1 Commonly Used Load Factors[a]

Country	γ_D	γ_L	γ_E
United States [A1]	1.40	1.70	—
	1.05	1.28	1.40[b]
	0.90	—	1.43[b]
New Zealand[X8]	1.40	1.70	—
	1.00	1.30	1.00[b]
	0.90	—	1.00[b]
	1.00[c]	1.00[c]	1.00[c]
	0.90[c]	—	1.00[c]

[a]Applicable also to effects due to reduced live load L_r [Eq. (1.1)].
[b]The intensities of specified earthquake forces in the United States and New Zealand, when factored, are similar.
[c]These factors are applicable only when actions due to earthquake effects are derived from capacity design considerations.

where S = denotes strength in general
D, L, E = denotes the causative load or force (i.e., dead load, live load, etc.)
$\gamma_D, \gamma_L, \gamma_E$ = specified load factors relevant to dead load, live load, and earthquake forces, respectively

Relevant codes specify values for load factors to be used for different load combinations or combinations of load effects. Some typical values, relevant to building structures studied in this book, are listed in Table 1.1. Load factors are intended to ensure adequate safety against increase in service loads beyond intensities specified, so that failure is extremely unlikely. Load factors also help to ensure that deformations at service load are not excessive.

Although load factors for dead and live loads are very similar in the United States and New Zealand, it will be seen that a substantial discrepancy exits in the treatment of the factors applicable to earthquake forces. In the United States, factors of approximately 1.40 are adopted, while in New Zealand, the appropriate factor is unity.

The original intention of load factors, when first implemented in strength design of structural elements in the 1960s, was to avoid the development of the resistance capacity of elements under maximum loads likely to occur during the building's economic life. With a seismic design philosophy based on ductility this approach is inappropriate, since development of strength, or resistance capacity, is expected under the design-level ground shaking. Applying load factors to a force level that has already been reduced from the level corresponding to elastic response merely implies a reduction to the expected ductility requirement. Unfortunately, this obscures the true level of ductility required. As a consequence, examples of seismic design developed later in the book will be based on load factors of unity for seismic forces.

Thus typical combinations [X8] of bending moments M for a beam, leading to the determination of its required flexural strength, would be

$$M_u = 1.4M_D + 1.7M_L \tag{1.5a}$$

or

$$M_u = 1.0M_D + 1.3M_L + M_E \tag{1.5b}$$

The design axial load P on a column would be obtained, for example, from

$$P_u = P_D + 1.3P_L + P_E \tag{1.6a}$$

or

$$P_u = 0.9P_D - P_E \tag{1.6b}$$

The latter combination is often critical when the seismic and gravity axial forces counteract each other. For example, a column, under compression due to dead load, may be subjected to axial tension due to earthquake forces.

Where gravity load effects are to be combined with effects resulting from the ductile response of the structure, with overstrength being developed at plastic hinges, as defined in Section 1.3.3(d) and Eq. (1.10), little if any reserve strength is necessary. Hence, where using capacity design procedures to satisfy the limit state for survival, the following combinations of actions [X3] may be used:

$$S_u = S_D + S_L + S_{E_o} \tag{1.7a}$$

and

$$S_u = 0.9S_D + S_{E_o} \tag{1.7b}$$

where S_{E_o} denotes an action derived from considerations of earthquake-induced overstrengths of relevant plastic regions, examined in detail in Section 1.3.3(d) and (f) and Chapters 4 and 5.

1.3.3 Strength Definitions and Relationships

Definitions of the strengths of a structure or its members, made in subsequent sections, correspond in general with the intent of most codes. In conformance with general usage, the term *strength* will be used to express the resistance of a structure, or a member, or a particular section. In terms of design practice, strength, however, is not an absolute. Material strengths and section dimensions are not known precisely but vary between probable limits. Choices of these properties should be made dependent on the purpose of application of the computed strength. The meaning of strengths developed at different levels and their relationships as used in this book are given in the following paragraphs.

(a) Required Strength (S_u) The strength demand arising from the application of prescribed loads and forces, in accordance with Section 1.3.2, defines the required strength, S_u. The principal aim of the design is to provide *resistance*, also termed *design strength* [A1] or *dependable strength* [X3], to meet this demand.

(b) Ideal Strength (S_i) The ideal or nominal strength of a section of a member, S_i, the most commonly used term, is based on established theory predicting a prescribed limit state with respect to failure of that section. It is derived from the dimensions, reinforcing content, and details of the section designed, and code-specified nominal material strength properties. The definition of nominal material strengths differs from country to country. In some cases it is a specified minimum strength, which suppliers guarantee to exceed; in others a characteristic strength is adopted, typically corresponding to the lower 5 percentile limit of measured strengths. A summary of established procedures for the determination of the ideal strengths of sections, subjected to different kinds of actions, is reviewed in Chapter 3. The ideal strength to be provided is related to the required strength by

$$\phi S_i \geq S_u \tag{1.8}$$

where ϕ is a strength reduction factor, typical values of which are given in Section 3.4.1. The designer will aim to proportion member sections so that the relationship $S_i \geq S_u/\phi$ is satisfied. Because of the necessity to round off various quantities in practice, the equality $S_i = S_u/\phi$ will seldom be achieved. Because the design philosophy, pursued in this book, relies on the hierarchy of capacities (i.e., strengths provided in various members), it is important to remember that as a general rule, the ideal strength S_i is not the optimum strength desired, but it is the nominal strength that will be provided in the construction. It will be seen that often the ideal strength of a section may well be in excess of that which is required (i.e., $S_i > S_u/\phi$).

(c) Probable Strength (S_p) The probable strength, S_p, takes into account the fact that material strengths, which can be utilized in a member, are generally greater than nominal strengths specified by codes. The probable strength of materials can be established from routine testing, normally conducted during construction. Alternatively, it may be based on previous experience with the relevant materials. The probable strength, or mean resistance, can be related to the ideal strength by

$$S_p = \phi_p S_i \tag{1.9}$$

where ϕ_p is the probable strength factor allowing for materials being stronger than specified, and is thus greater than 1.

Probable strengths are often used when the strength of existing structures are estimated or when time-history dynamic analysis, to predict the likely behavior of a structure when it is exposed to a selected earthquake record (Section 2.4.1), is undertaken. Developments to adopt probable strength as a basis for design, replacing the ideal strength, were known to the authors when preparing this book. Strength reduction factors would then relate dependable strength to probable rather than ideal strength.

(d) Overstrength (S_o) The overstrength of a section, S_o, takes into account all possible factors that may contribute to strength exceeding the nominal or ideal value. These include steel strength greater than the specified yield strength, additional strength enhancement of steel due to strain hardening at large deformations, concrete of masonry strength at a given age of the structure being higher than specified, unaccounted-for compression strength enhancement of the concrete due to its confinement, and strain rate effects. The overstrength of a section can be related to the ideal strength of the same section by

$$S_o = \lambda_o S_i \tag{1.10}$$

where λ_o is the overstrength factor due to strength enhancement of the constituent materials. This is an important property that must be accounted for in the design when large ductility demands are imposed on the structure, since brittle elements must possess strengths exceeding the maximum feasible strength of ductile elements. Typical values of λ_o for both reinforcing steel and concrete are given in Sections 3.2.4(e). Similar strength enhancement in confined concrete is given by Eq. (3.10) and in members subjected to moment and axial compression by Eq. (3.28).

(e) Relationships Between Strengths Because strengths to be considered in design are most conveniently expressed in terms of the ideal strength S_i of a section, as constructed, the following simple relationships exist:

$$S_i \geq S_u/\phi \tag{1.8a}$$

$$S_p \geq \phi_p S_i \geq \phi_p S_u/\phi \tag{1.9a}$$

$$S_o \geq \lambda_o S_i \geq \lambda_o S_u/\phi \tag{1.10a}$$

For example, with typical values of $\phi = 0.9$ and $\lambda_o = 1.25$, $S_o \geq 1.39 S_u$, i.e., the overstrength of the section designed "exactly" to match the required strength would be 39% larger than that required by Eq. (1.4).

(f) Flexural Overstrength Factor ϕ_o To quantify the hierarchy of strength in the design of ductile structures, it is convenient to express the overstrength of a member in flexure $S_o = M_o$ at a specific section, such as a node point of the analytical model, in terms of the required flexural strength $S_E = M_E$ at the same section, derived by an elastic analysis for earthquake forces alone. The ratio so formed,

$$\phi_o = S_o/S_E = M_o/M_E \tag{1.11}$$

is defined as the flexural overstrength factor. When the two factors ϕ_o and λ_o, given by Eq. (1.10), are compared, it should be noted that apart from the overstrengths of materials, the following additional sources of flexural overstrength are also included in Eq. (1.11):

1. The strength reduction factor ϕ [Eq. (1.8)] used to relate ideal to required strength
2. More severe strength requirements, if any, due to gravity loads and wind forces
3. Changes in design moments due to any redistribution of these, which the designer may have undertaken (see Section 4.3)
4. Deviations from the optimum ideal strength due to the choice of the amount of reinforcement as dictated by practicality (availability of bar sizes and numbers)

Moreover, the overstrength factor, λ_o, is relevant to the critical section of a potential plastic hinge which may be located anywhere along a member, while the flexural overstrength factor, ϕ_o, expresses strengths ratios at node points. Where the critical section coincides with a node point, as may be the case of a cantilever member, such as a structural wall resisting seismic forces, the relationships from Eqs. (1.8), (1.10), and (1.11) reveal that $M_o = \lambda_o M_i = \phi_o M_E$ and hence that

$$\phi_o = \frac{M_o}{M_E} = \frac{\lambda_o M_i}{M_E} \gtreqless \frac{\lambda_o(M_E/\phi)}{M_E} = \frac{\lambda_o}{\phi} \tag{1.12}$$

An equality means that the dependable strength ϕM_i provided, for example, at the base of a cantilever wall is exactly that (M_E) required to resist seismic forces. With typical values of $\lambda_o = 1.25$ and $\phi = 0.9$, the flexural overstrength factor in this case becomes $\phi_o = 1.39$. Values of ϕ_o larger than λ_o/ϕ indicate that the dependable flexural strength of the base section of the wall is in excess of required strength (i.e., the overturning moment M_E resulting from design earthquake forces only). When $\phi_o < \lambda_o/\phi$, a deficiency of required strength is indicated, the sources of which should be identified.

Any of, or the combined sources (2) to (4) of overstrength listed above, as well as computational errors, may be causes of ϕ_o being more or less than the ratio λ_o/ϕ. The flexural overstrength factor ϕ_o is thus a very convenient parameter in the application of capacity design procedures. It is a useful indicator, evaluated as the design of members progresses, to measure first, the extent to which gravity loads or earthquakes forces dominate strength requirements, and second, the relative magnitudes of any over- or under-design by choice or as a result of an error made.

(g) System Overstrength Factor ψ_o The flexural overstrength factor ϕ_o measures the flexural overstrength in terms of the required strength for *earthquake forces alone* at one node point of the structural model. In certain situations it is equally important to compare the sum of the overstrengths of a number of interrelated members with the total demand made on the same members by the specified earthquake forces alone. For example, the sum of the flexural overstrengths of all column sections of a framed building at the bottom and the top of a story may be compared with the total story moment demand due to the total story shear force, such as V_j in Fig. 1.9.

The system or overall overstrength factor, ψ_o, may then be defined:

$$\psi_o = \frac{\Sigma S_o}{\Sigma S_E} = \frac{\Sigma \phi_o S_E}{\Sigma S_E} \tag{1.13}$$

As an example, consider a multistory building in which the entire seismic resistance in a particular direction is provided by n reinforced concrete structural walls. The relevant material overstrength and strength reduction factors in this example are $\lambda_o = 1.4$ and $\phi = 0.9$. The flexural overstrength factor, relevant to the base moments of each of these walls, may well be more or less than $\phi_o = \lambda_o/\phi = 1.56$ [Eq. (1.12)]. Deviations would indicate that some walls have been designed for larger or smaller moments than required for seismic resistance (i.e., $M_i < M_E/\phi$). However, if the value of the system overstrength factor

$$\psi_o = \sum_i^n M_o / \sum_i^n M_E = \sum_i^n (\phi_o M_E) / \sum_i^n M_E$$

is less than 1.56, this indicates that the strength requirement for the structure as a whole, to resist seismic design forces, has been violated. On the other hand, a value much larger than 1.56 in this example will warn the designer that for reasons which should be identified, strength well in excess of that required to resist the specified earthquake forces has been provided. This is of importance when the design of the foundation structure, examined in Chapter 9, is to be considered.

1.3.4 Strength Reduction Factors

Strength reduction factors ϕ, introduced in Section 1.3.3(b), are provided in codes [A1, X3] to allow approximations in the calculations and variations in material strengths, workmanship, and dimensions. In addition, consideration has been given to the seriousness and consequences of failure of a member in respect to the whole structure and the degree of warning involved in the mode of failure [P1].

Thus the overall safety factor for a structure, subjected to dead and live loads only, from Eqs. (1.4) and (1.8) may be expressed by the ratio

$$\frac{S_i}{S_D + S_L} \geq \frac{S_u}{\phi(S_D + S_L)} = \frac{\gamma_D S_D + \gamma_L S_L}{\phi(S_D + S_L)} \tag{1.14}$$

For example, with $L = D$ and $\gamma_D = 1.4$, $\gamma_L = 1.7$ and $\phi = 0.9$, the overall safety factor with respect to the ideal strength S_i to be reached, is 1.72. Commonly used values of strength reduction factors are given in Section 3.4.1.

In some recent codes, strength reduction factors, applicable to a specific action such as flexure or shear, have been replaced by resistance factors with values specified for each of the constituent materials, such as concrete and different types of steels [X5]. Values of load and load combinations factors have been adjusted accordingly. With the use of resistance factors, relationships between ideal, required, probable and overstrength are similar to those described in the previous sections. However, instead of using the equations given here, these relationships have to be established for each case using elementary first principles.

1.4 PHILOSOPHY OF CAPACITY DESIGN

1.4.1 Main Features

Procedures for the application of capacity design to ductile structures, which may be subjected to large earthquakes, have been developed primarily in New Zealand over the last 20 years [P1, P3, P4, P6, P17], where they have been used extensively [X3, X8]. With some modification the philosophy has also been adopted in other countries [A5, C11, X5]. However, for specific situations, the application of capacity design principles was already implied in earlier editions of some codes [X4, X10]. The seismic design strategy adopted in this book is based on this philosophy. It is a rational, deterministic, and relatively simple approach.

In the capacity design of structures for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations. The critical regions of these members, often termed *plastic hinges*, are detailed for inelastic flexural action, and shear failure is inhibited by a suitable strength differential. All other structural elements are then protected against actions that could cause failure, by providing them with strength greater than that corresponding to development of maximum feasible strength in the potential plastic hinge regions.

It must be recognized that in an element subjected to full or reduced ductility demands, the strength developed is considerably less than that corresponding to elastic response, as shown, for example, in Fig. 1.17. It follows that it is the actual strength, not the nominal or ideal strengths, that will be developed, and at maximum displacement, overstrength S_o response is expected. Nonductile elements, resisting actions originating from plastic hinges, must thus be designed for strength based on the overstrength S_o rather than the code-specified strength S_u, which is used for determining required dependable strengths of hinge regions. This "capacity" design procedure ensures that the chosen means of energy dissipation can be maintained.

The following features characterize the procedure:

1. Potential plastic hinge regions within the structure are clearly defined. These are designed to have dependable flexural strengths as close as practicable to the required strength S_u. Subsequently, these regions are carefully detailed to ensure that estimated ductility demands in these regions can be reliably accommodated. This is achieved primarily by close-spaced and well-anchored transverse reinforcement.
2. Undesirable modes of inelastic deformation, such as may originate from shear or anchorage failures and instability, within members containing plastic hinges, are inhibited by ensuring that the strengths of these modes exceeds the capacity of the plastic hinges at overstrength.
3. Potentially brittle regions, or those components not suited for stable energy dissipation, are protected by ensuring that their strength exceeds the demands originating from the overstrength of the plastic hinges. Therefore, these regions are designed to remain elastic irrespective of the intensity of the ground shaking or the magnitudes of inelastic deformations that may occur. This approach enables traditional or conventional detailing of these elements, such as used for structures designed to resist only gravity loads and wind forces, to be employed during construction.

An example illustrating the application of these concepts to a simple structure is given in Section 1.4.4.

As discussed in Chapter 2, the intensity of design earthquake forces and structural actions which result from these are rather crude estimates, irrespective of the degree of sophistication on which analyses may be based. Provided that the intended lateral force resistance of the structure is assured, approximations in both analysis and design can be used, within reason, without affecting in any way the seismic performance of the structure. The area of greatest uncertainty of response of capacity-designed structures is the level of inelastic deformations that might occur under strong ground motion. However, the high quality of the detailing of potential plastic regions, the subject addressed in a considerable part of this book, will ensure that significant variations in ductility demands from the expected value can be accommodated without loss of resistance to lateral forces. Hence capacity-designed ductile structures are extremely tolerant with respect to imposed seismic deformations.

It is emphasized that capacity design is not an analysis technique but a powerful design tool. It enables the designer to "tell the structure what to do" and to desensitize it to the characteristics of the earthquake, which are, after all, unknown. Subsequent judicious detailing of all potential plastic regions will enable the structure to fulfill the designer's intentions.

1.4.2 Illustrative Analogy

To highlight the simple concepts of capacity design philosophy, the chain shown in Fig. 1.18 will be considered. Using the adage that the strength of a chain is the strength of its weakest link, a very ductile link may be used to achieve adequate ductility for the entire chain. The ideal or nominal tensile strength [Section 1.3.3(*a*)] of this ductile steel link is P_i, but the actual strength is subject to the normal uncertainties of material strength and strain hardening effects at high strains. The other links are presumed to be brittle. Note that if they were designed to have the same nominal strength as the ductile link, the randomness of strength variation between all links, including the ductile link, would imply a high probability that failure would occur in a brittle link and the chain would have no ductility. Failure of all other links

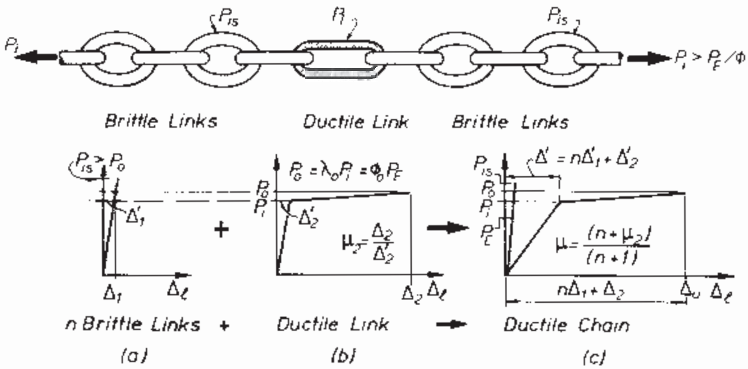

Fig. 1.18 Principle of strength limitation illustrated with ductile chain.

can, however, be prevented if their strength is in excess of the maximum feasible strength of the weak link, corresponding to the level of ductility envisaged. Using the terminology defined in Section 1.3.3, the dependable strength ϕP_{is} of the strong links should therefore not be less than the overstrength P_o of the ductile link $\lambda_o P_i$. As ductility demands on the strong links do not arise, they may be brittle (cast iron).

The chain is to be designed to carry an earthquake-induced tensile force $P_u = P_E$. Hence the ideal strength of the weak link needs to be $P_i \geq P_E/\phi$ [Eq. (1.8)]. Having chosen an appropriate ductile link, its overstrength can be readily established (i.e., $P_o = \lambda_o P_i = \phi_o P_E$) [Eqs. (1.10) and (1.11)], which becomes the design force P_{us}, and hence required strength, for the strong and brittle links. Therefore, the ideal or nominal strength of the strong link needs to be

$$P_{is} > P_{us}/\phi_s = P_o/\phi_s = \phi_o P_E/\phi_s$$

where quantities with subscript s refer to the strong links. For example, when $\phi = 0.9$, $\lambda_o = 1.3$, and $\phi_s = 1.0$, we find that $P_i > 1.11 P_E$ and $P_{is} \geq 1.3$ $(1.11 P_E)/1.0 = 1.44 P_E$.

The example of Fig. 1.18 may also be used to draw attention to an important relationship between the ductility potential of the entire chain and the corresponding ductility demand of the single ductile link. Linear and bilinear force–elongation relationships, as shown in Fig. 1.18(b) and (c), are assumed for all links. Inelastic elongations can develop in the ductile link only. As Fig. 1.18 shows, elongations at the onset of yielding of the brittle and ductile links are Δ'_1 and Δ'_2, respectively. Subsequent and significant yielding of the weak link will increase its elongation from Δ'_2 and Δ_2, while its resistance increases from $P_y = P_i$ to P_o due to strain hardening. The weak link will thus exhibit a ductility of $\mu_2 = \Delta_2/\Delta'_2$. As Fig. 1.18($c$) shows, the total elongation of the chain, comprising the weak link and n strong links, at the onset of yielding in the weak link will be $\Delta' = n\Delta'_1 + \Delta'_2$. At the development of the overstrength of the chain (i.e., that of the weak link), the elongation of the strong link will increase only slightly from Δ'_1 to Δ_1. Thus at ultimate the elongation of the entire chain becomes $\Delta_u = n\Delta_1 + \Delta_2$. As Fig. 1.18 shows, the ductility of the chain is then

$$\mu = \Delta_u/\Delta' = (n\Delta_1 + \Delta_2)/(n\Delta'_1 + \Delta'_2)$$

With the approximation that $\Delta_1 \simeq \Delta'_1 \simeq \Delta'_2 = \Delta_y$, it is found that the relationship between the ductility of the chain μ and that of weak link is μ_2 is

$$\mu = (n + \mu_2)/(n + 1)$$

If the example chain of Fig. 1.18 consists of eight strong links and the maximum elongation of the weak link Δ_2 is to be limited to 10 times its elongation at yield, Δ'_2 (i.e., $\mu_2 = 10$), we find that the ductility of the chain is limited to $\mu = (8 + 10)/(8 + 1) = 2$. Conversely, if the chain is expected to develop a ductility of $\mu = 3$, the ductility demand on the weak link will increase to $\mu_2 = 19$. The example was used to illustrate the very large differences in the magnitudes of overall ductilities and local ductilities that may occur in certain types of structures. In some structures the overall ductility to be considered in the design will need to be limited to ensure that ductility demands at a critical locality do not become excessive. Specific ductility relationships are reviewed in Section 3.4.

1.4.3 Capacity Design of Structures

The principles outlined for the example chain in Fig. 1.18 can be extended to encompass the more complex design of a large structure (e.g., a multistory building). The procedure uses the following major steps:

1. A kinematically admissible plastic mechanism is chosen.
2. The mechanism chosen should be such that the necessary overall displacement ductility can be developed with the smallest inelastic rotation demands in the plastic hinges (Fig. 1.19*a*).
3. Once a suitable plastic mechanism is selected, the regions for energy dissipation (i.e., plastic hinges) are determined with a relatively high degree of precision.
4. Parts of a structure intended to remain elastic in all events are designed so that under maximum feasible actions corresponding to overstrength in the plastic hinges, no inelastic deformations should occur in those regions. Therefore, it is immaterial whether the failure of regions, intended to remain elastic, would be ductile or brittle (Fig. 1.8). The actions originating from plastic hinges are those associated with the overstrength [Section 1.3.3(*d*)] of these regions. The required strength of all other regions [Section 1.3.3(*b*)] is then in excess of the strength demand corresponding to the overstrength of relevant plastic hinges.

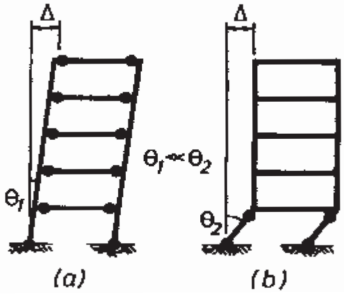

Fig. 1.19 Comparison of energy-dissipating mechanisms.

5. A clear distinction is made with respect to the nature and quality of detailing for potentially plastic regions and those which are to remain elastic in all events.

A comparison of the two example frames in Fig. 1.19 shows that for the same maximum displacement Δ at roof level, plastic hinge rotations θ_1 in case (a) are much smaller than those in case (b), θ_2. Therefore, the overall ductility demand, in terms of the large deflection Δ, is much more readily achieved when plastic hinges develop in all the beams instead of only in the first-story column. The column hinge mechanism, shown in Fig. 1.19(b), also referred to as a soft-story, may impose plastic hinge rotations, which even with good detailing of the affected regions, would be difficult to accommodate. This mechanism accounts for numerous collapses of framed buildings in recent earthquakes. In the case of the example given in Fig. 1.19, the primary aim of capacity design will be to prohibit formation of a soft story and, as a corollary, to ensure that only the mechanism shown in Fig. 1.19(a), can develop.

A capacity design approach is likely to assure predictable and satisfactory inelastic response under conditions for which even sophisticated dynamic analyses techniques can yield no more than crude estimates. This is because the capacity-designed structure cannot develop undesirable hinge mechanisms or modes of inelastic deformation, and is, as a consequence, insensitive to the earthquake characteristics, except insofar as the magnitude of inelastic flexural deformations are concerned. When combined with appropriate detailing for ductility, capacity design will enable optimum energy dissipation by rationally selected plastic mechanisms to be achieved. Moreover, as stated earlier, structures so designed will be extremely tolerant with respect to the magnitudes of ductility demands that future large earthquakes might impose.

1.4.4 Illustrative Example

The structure of Fig. 1.20(a) consists of two reinforced concrete portal frames connected by a monolithic slab. The slab supports computer equipment that applies a gravity load of W to each portal as shown in Fig. 1.20(b). The computer equipment is located at level 2 of a large light industrial two-story building whose design strength, with respect to lateral forces, is governed by wind forces. As a consequence of the required strength for wind forces, the building as a whole can sustain lateral accelerations of at least $0.6g$ elastically (g is the acceleration due to gravity). However, the computer equipment, which must remain functional even after a major earthquake, cannot sustain lateral accelerations greater than $0.35g$. As a consequence, the equipment is supported by the structure of Fig. 1.20(a), which itself is isolated from the rest of the building by adequate structural separation. The design requirements are that the beams should remain elastic and that the

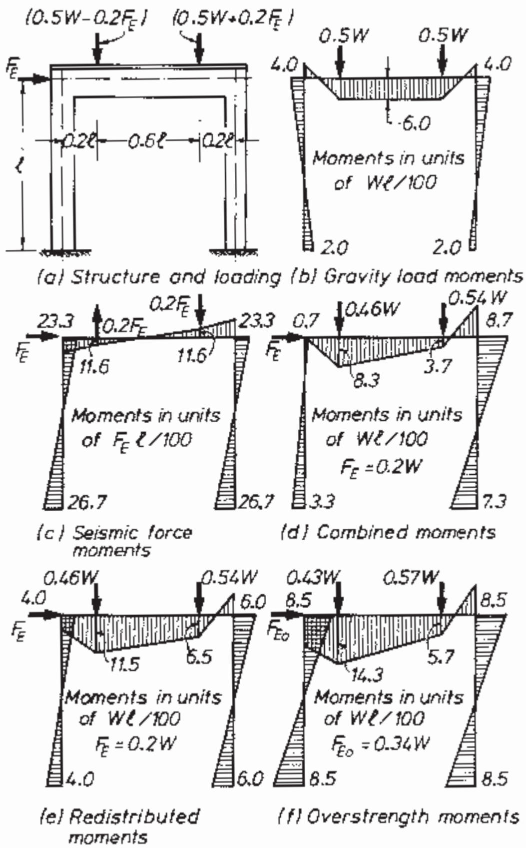

Fig. 1.20 Dimensions and moments of example portal frame.

columns should provide the required ductility by plastic hinging. Columns are to be symmetrically reinforced over the full height.

Elastic response for the platform under the design earthquake is estimated to be $1.0g$, and a dependable ductility capacity of $\mu_\Delta = 5$ is assessed to be reasonable. Correspondingly, the design seismic force at level 2 is based on $0.2g$ lateral acceleration. Design combinations of loads and forces are

$$U \geq 1.4D + 1.7L$$
$$U \geq D + 1.3L + E$$

(i) Gravity Load: The computer is permanent equipment of accurately known weight, so is considered as superimposed dead load. The weight of the platform and the beam and live load are small in comparison with the weight of the equipment, and hence for design purposes the total gravity load W,

corresponding to $U = D + 1.3L$, can be simulated as shown in Fig. 1.20(b), where the induced moments are also shown.

(ii) Seismic Forces: The center of mass of the computer is some distance above the platform; hence vertical seismic forces of $\pm 0.2F_E$ as well as the lateral force F_E are induced, as shown in Fig. 1.20(a) and (c). Moments induced by this system of forces are shown in Fig. 1.20(c).

(iii) Combined Actions: Figure 1.20(d) shows the moments under W and F_E, where the seismic moments are based on the ductile design level of $F = 0.2W$ (i.e., $0.2g$ lateral acceleration) and W is based on $D + 1.3L$. It will be noted that the bending moments generated in the two columns are very different. If column strength is based on the computed moments for the right-hand column, and if the influence of variable axial loads between the two columns is considered to be small enough to ignore, the dependable shear strength of each column will be $V = (0.073\ Wl + 0.087Wl)/l = 0.16W$, since both columns will have the same reinforcement. (The seismic forces could, of course, act from right to left, making the left-hand column critical.)

The moment pattern of Fig. 1.20(d) corresponds to the elastic distribution of moments applicable at the yield displacement Δ_y. Since the design ductility factor is $\mu_\Delta = 5$, the structure is expected to deform to five times this displacement. Under the large displacements, the left-hand column could also develop its strength at top and bottom, and both columns could be expected to develop overstrength moments. Thus if a strength reduction factor of $\phi = 0.9$ was used and $\lambda_o = 1.25$ is applicable at overstrength, the maximum horizontal force F_o that could be sustained would be, from Eqs. (1.8) and (1.10),

$$F_o = (1/0.9)1.25(2 \times 0.160)W = 0.444W$$

corresponding to a lateral acceleration of $0.444g$. This exceeds the permissible limit of $0.35g$.

(iv) Redistribution of Design Forces: A solution is to redistribute the moments in Fig. 1.20(d) so as to reduce the shear developed in the right column and increase the shear carried by the left column, as shown in Fig. 1.20(e). Here the maximum moments at the top and bottom of the right column have been decreased to $0.060Wl$, and those of the left column have been increased to $0.040Wl$. The total lateral force carried at this stage by the portal, which is the sum of the shear forces in the two columns, is $0.2W$. It would be possible to make all column end moments equal at $0.05Wl$, but that would make the column top moment significantly less than that required for gravity load alone ($U \geq 1.4D$; $M_u \geq 1.4 \times 0.04Wl = 0.056Wl$). Also, to reduce elastic design moments by 43% might be considered excessive.

(v) Evaluation of Overstrengths: After selection of suitable reinforcement, the dependable column moment strengths are found to be, for both columns,

$$\phi M_i = 0.061Wl$$

The dependable lateral strength of the frame is thus

$$\phi F_i = (4 \times 0.061)Wl/l = 0.244W = 1.22F_E$$

The column overstrength moment capacity is, from Eq. (1.10), $M_o = \lambda_o M_i = 1.25 \times 0.0061Wl/\phi = 0.085Wl$, and the system overstrength factor is, from Eq. (1.13) and Fig. 1.20(f) and (c), $\psi_o = (4 \times 0.085)/[2 \times (0.233 + 0.267) \times 0.2] = 1.70$. Thus the maximum feasible strength will correspond to $1.7 \times 0.2g = 0.34g$, which is less than the design limit of $0.35g$. Moments at overstrength response are shown in Fig. 1.20(f).

To ensure ductile response ($\mu_\Delta \approx 1.0/0.244 = 4.1$ required), shear failure must be avoided. Consequently, the required shear strength of each column must be based on flexural overstrength, that is,

$$V_u = 2 \times 0.085Wl/l = 0.17W = 0.5\psi_o F_E$$

The beam supporting the computer platform must be designed for a maximum required positive moment capacity of $0.143Wl$ to ensure that the platform remains elastic. Note that this is 1.72 times the initial value given in Fig. 1.20(d). In a more realistic example, however, it might be necessary to design for even larger flexural strength because of higher mode response, corresponding to the computer equipment "bouncing" on the vertically flexible beam.