Chapter 10

# Seismic Design of Reinforced Concrete Structures

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Abstract: This chapter covers various aspects of seismic design of reinforced concrete structures with an emphasis on design for regions of high seismicity. Because the requirement for greater ductility in earthquake-resistant buildings represents the principal departure from the conventional design for gravity and wind loading, the major part of the discussion in this chapter will be devoted to considerations associated with providing ductility in members and structures. The discussion in this chapter will be confined to monolithically cast reinforced-concrete buildings. The concepts of seismic demand and capacity are introduced and elaborated on. Specific provisions for design of seismic resistant reinforced concrete members and systems are presented in detail. Appropriate seismic detailing considerations are discussed. Finally, a numerical example is presented where these principles are applied. Provisions of ACI-318/95 and IBC-2000 codes are identified and commented on throughout the chapter.

# 10.1 INTRODUCTION

## 10.1.1 The Basic Problem

The problem of designing earthquake-resistant reinforced concrete buildings, like the design of structures (whether of concrete, steel, or other material) for other loading conditions, is basically one of defining the anticipated forces and/or deformations in a preliminary design and providing for these by proper proportioning and detailing of members and their connections. Designing a structure to resist the expected loading(s) is generally aimed at satisfying established or prescribed safety and serviceability criteria. This is the general approach to engineering design. The process thus consists of determining the expected *demands* and providing the necessary *capacity* to meet these demands for a specific structure. Adjustments to the preliminary design may likely be indicated on the basis of results of the analysis-design-evaluation sequence characterizing the iterative process that eventually converges to the final design. Successful experience with similar structures should increase the efficiency of the design process.

In earthquake-resistant design, the problem is complicated somewhat by the greater uncertainty surrounding the estimation of the appropriate design loads as well as the capacities of structural elements and connections. However, information accumulated during the last three decades from analytical and experimental studies, as well as evaluations of structural behavior during recent earthquakes, has provided a strong basis for dealing with this particular problem in a more rational manner. As with other developing fields of knowledge, refinements in design approach can be expected as more information is accumulated on earthquakes and on the response of particular structural configurations to earthquake-type loadings.

As in design for other loading conditions, attention in design is generally focused on those areas in a structure which analysis and experience indicate are or will likely be subjected to the most severe demands. Special emphasis is placed on those regions whose failure can affect the integrity and stability of a significant portion of the structure.

## 10.1.2 Design for Inertial Effects

Earthquake-resistant design of buildings is intended primarily to provide for the inertial effects associated with the waves of distortion that characterize dynamic response to ground shaking. These effects account for most of the damage resulting from earthquakes. In a few cases, significant damage has resulted from conditions where inertial effects in the structure were negligible. Examples of these latter cases occurred in the excessive tilting of several multistory buildings in Niigata, Japan, during the earthquake of June 16, 1964, as a result of the liquefaction of the sand on which the buildings were founded, and the loss of a number of residences due to large landslides in the Turnagain Heights area in Anchorage, Alaska, during the March 28, 1964 earthquake. Both of the above effects, which result from ground motions due to the passage of seismic waves, are usually referred to as secondary effects. They are distinguished from so-called primary effects, which are due directly to the causative process, such as faulting (or volcanic action, in the case of earthquakes of volcanic origin).

## 10.1.3 Estimates of Demand

Estimates of force and deformation demands in critical regions of structures have been based on dynamic analyses—first, of simple systems, and second, on inelastic analyses of more complex structural configurations. The latter approach has allowed estimation of force and deformation demands in local regions of specific structural models. Dynamic inelastic analyses of models of representative structures have been used to generate information on the variation of demand with major structural as well as ground-motion parameters. Such an effort involves consideration of the practical range of values of the principal structural parameters as well as the expected range of variation of the ground-motion parameters. Structural parameters include the structure fundamental period, principal member yield levels, and force—displacement characteristics; input motions of reasonable duration and varying intensity and frequency characteristics normally have to be considered.

A major source of uncertainty in the process of estimating demands is the characterization of the design earthquake in terms of intensity, frequency characteristics, and duration of large-amplitude pulses. Estimates of the intensity of ground shaking that can be expected at particular sites have generally been based on historical records. Variations in frequency characteristics and duration can be included in an analysis by considering an ensemble of representative input motions.

Useful information on demands has also been obtained from tests on specimens subjected to simulated earthquake motions using shaking tables and, the pseudo-dynamic method of testing. The latter method is a combination of the so-called quasi-static, or slowly reversed, loading test and the dynamic shaking-table test. In this method, the specimen is subjected to essentially statically applied increments of deformation at discrete points, the magnitudes of which are calculated on the basis of predetermined earthquake input and the measured stiffness and estimated damping of the structure. Each increment of load after the initial increment is based on the measured stiffness of the structure during its response to the imposed loading of the preceding increment.

### 10.1.4 Estimates of Capacity

Proportioning and detailing of critical regions in earthquake-resistant structures have mainly been based on results of tests on laboratory specimens tested by the quasi-static method, i.e., under slowly reversed cycles of loading. Data from shaking-table tests and from pseudo-dynamic tests have also contributed to the general understanding of structural behavior under earthquake-type loading. Design and detailing practice, as it has evolved over the last two or three decades, has also benefited from observations of the performance of structures subjected to actual destructive earthquakes.

Earthquake-resistant design has tended to be viewed as a special field of study, not only because many engineers do not have to be concerned with it, but also because it involves additional requirements not normally dealt with in designing for wind. Thus, while it is generally sufficient to provide adequate stiffness and strength in designing buildings for wind, in the case of earthquake-resistant design, a third basic requirement, that of ductility or inelastic deformation capacity, must be considered. This third requirement arises because it is generally uneconomical to design most buildings to respond elastically to moderate-to-strong earthquakes. To survive such earthquakes, codes require that structures possess adequate ductility to allow them to dissipate most of the energy from the ground motions through inelastic deformations. However, deformations in the seismic force resisting system must be controlled to protect elements of the structure that are not part of the lateral force resisting system. The fact is that many elements of the structure that are not intended as a part of the lateral force resisting system and are not detailed for ductility will participate in the lateral force resistant mechanism and can become severely damaged as a result. In the case of wind, structures are generally expected to respond to the design wind within their "elastic" range of stresses. When wind loading governs the design (drift or strength), the structure still should comply with the appropriate seismic detailing requirements. This is required in order to provide a ductile system to resist earthquake forces. Figure 10-1 attempts to depict the interrelationships between the various considerations involved in earthquake-resistant design.

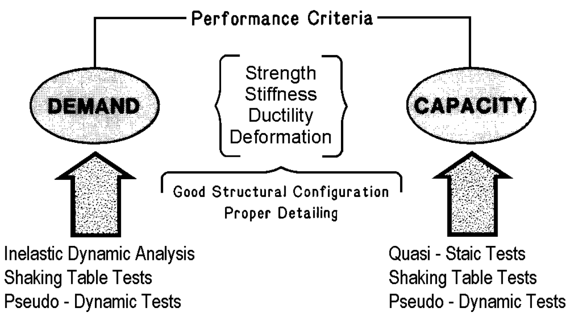



*Figure 10- 1.* Components of and considerations in earthquake-resistant building design

### 10.1.5 The Need for a Good Design Concept and Proper Detailing

Because of the appreciable forces and deformations that can be expected in critical regions of structures subjected to strong ground motions and a basic uncertainty concerning the intensity and character of the ground motions at a particular site, a good design concept is essential at the start. A good design concept implies a structure with a configuration that behaves well under earthquake excitation and designed in a manner that allows it to respond to strong ground motions according to a predetermined pattern or sequence of yielding. The need to start with a sound structural configuration that minimizes "incidental" and often substantial increases in member forces resulting from torsion due to asymmetry or force concentrations associated with discontinuities cannot be overemphasized. Although this idea may not be met with favor by some architects, clear (mainly economic) benefits can be derived from structural configurations emphasizing symmetry, regularity, and the avoidance of severe discontinuities in mass, geometry, stiffness, or strength. A direct path for the lateral (inertial) forces from the superstructure to an appropriately designed foundation is very desirable. On numerous occasions, failure to take account of the increase in forces and deformations in certain elements due to torsion or discontinuities has led to severe structural distress and even collapse. The provision of relative strengths in the various types of elements making up a structure with the aim of controlling the sequence of yielding in such elements has been recognized as desirable from the standpoint of structural safety as well as minimizing post-earthquake repair work.

An important characteristic of a good design concept and one intimately tied to the idea of ductility is structural redundancy. Since yielding at critically stressed regions and subsequent redistribution of forces to less stressed regions is central to the ductile performance of a structure, good practice suggests providing as much redundancy as possible in a structure. In monolithically cast reinforced concrete structures, redundancy is normally achieved by continuity between moment-resisting elements. In addition to continuity, redundancy or the provision of multiple load paths may also be accomplished by using several types of lateral-load-resisting systems in a building so that a "backup system" can absorb some of the load from a primary lateral-load-resisting system in the event of a partial loss of capacity in the latter.

Just as important as a good design concept is the proper detailing of members and their connections to achieve the requisite strength and ductility. Such detailing should aim at preventing nonductile failures, such as those associated with shear and with bond anchorage. In addition, a deliberate effort should be made to securely tie all parts of a structure that are intended to act as a unit together. Because dynamic response to strong earthquakes, characterized by repeated and reversed cycles of large-amplitude deformations in critical elements, tends to concentrate deformation demands in highly stressed portions of yielding members, the importance of proper detailing of potential hinging regions should command as much attention as the development of a good design concept. As with most designs but more so in design for earthquake resistance, where the relatively large repeated deformations tend to "seek and expose," in a manner of speaking, weaknesses in a structure—the proper field implementation of engineering drawings ultimately determines how well a structure performs under the design loading.

Experience and observation have shown that properly designed, detailed, and constructed reinforced-concrete buildings can provide the necessary strength, stiffness, and inelastic deformation capacity to perform satisfactorily under severe earthquake loading.

### 10.1.6 Accent on Design for Strong Earthquakes

The focus in the following discussion will be on the design of buildings for moderate-to-strong earthquake motions. These cases correspond roughly to buildings located in seismic zones 2, 3 and 4 as defined in the Uniform Building Code (UBC-97).[10-1] By emphasizing design for strong ground motions, it is hoped that the reader will gain an appreciation of the special considerations involved in this most important loading case. Adjustments for buildings located in regions of lesser seismic risk will generally involve relaxation of some of the requirements associated with highly seismic areas.

Because the requirement for greater ductility in earthquake-resistant buildings represents the principal departure from the conventional design for gravity and wind loading, the major part of the discussion in this chapter will be devoted to considerations associated with providing ductility in members and structures.

The discussion in this chapter will be confined to monolithically cast reinforced-concrete buildings.

## 10.2 DUCTILITY IN EARTHQUAKE-RESISTANT DESIGN

### 10.2.1 Design Objective

In general, the design of economical earthquake resistant structures should aim at providing the appropriate dynamic and structural characteristics so that acceptable levels of response result under the design earthquake. The magnitude of the maximum acceptable deformation will vary depending upon the type of structure and/or its function.

In some structures, such as slender, free-standing towers or smokestacks or suspension-type buildings consisting of a centrally located corewall from which floor slabs are suspended by means of peripheral hangers, the stability of the structure is dependent on the stiffness and integrity of the single major element making up the structure. For such cases, significant yielding in the principal element cannot be tolerated and the design has to be based on an essentially elastic response.

For most buildings, however, and particularly those consisting of rigidly connected frame members and other multiply redundant structures, economy is achieved by allowing yielding to take place in some critically stressed elements under moderate-to-strong earthquakes. This means designing a building for force levels significantly lower than would be required to ensure a linearly elastic response. Analysis and experience have shown that structures having adequate structural redundancy can be designed safely to withstand strong ground motions even if yielding is allowed to take place in some elements. As a consequence of allowing inelastic deformations to take place under strong earthquakes in structures designed to such reduced force levels, an additional requirement has resulted and this is the need to insure that yielding elements be capable of sustaining adequate inelastic deformations without significant loss of strength, i.e., they must possess sufficient ductility. Thus, where the strength (or yield level) of a structure is less than that which would insure a linearly elastic response, sufficient ductility has to be built in.

### 10.2.2 Ductility vs. Yield Level

As a general observation, it can be stated that for a given earthquake intensity and structure period, the ductility demand increases as the strength or yield level of a structure decreases. To illustrate this point, consider two vertical cantilever walls having the same initial fundamental period. For the same mass and mass distribution, this would imply the same stiffness properties. This is shown in Figure 10-2, where idealized force-deformation curves for the two structures are marked (1) and (2). Analyses(10-2, 10-3) have shown that the maximum lateral displacements of structures with the same initial fundamental period and reasonable properties are approximately the same when subjected to the same input motion. This phenomenon is largely attributable to the reduction in local accelerations, and hence displacements, associated with reductions in stiffness due to yielding in critically stressed portions of a structure. Since in a vertical cantilever the rotation at the base determines to a large extent the displacements of points above the base, the same observation concerning approximate equality of maximum lateral displacements can be made with respect to maximum rotations in the hinging region at the bases of the walls. This can be seen in Figure 10-3, from Reference 10-3, which shows results of dynamic analysis of isolated structural walls having the same fundamental period ($T_1$ = 1.4 sec) but different yield levels $M_y$. The structures were subjected to the first 10 sec of the east—west component of the 1940 El Centro record with intensity normalized to 1.5 times that of the north—south component of the same record. It is seen in Figure 10-3a that, except for the structure with a very low yield level ($M_y$ = 500,000 in.-kips), the maximum displacements for the different structures are about the same. The corresponding ductility demands, expressed as the ratio of the maximum hinge rotations, $\theta_{max}$ to the corresponding rotations at first yield, $\theta_y$, are shown in Figure 10-3b. The increase in ductility demand with decreasing yield level is apparent in the figure.

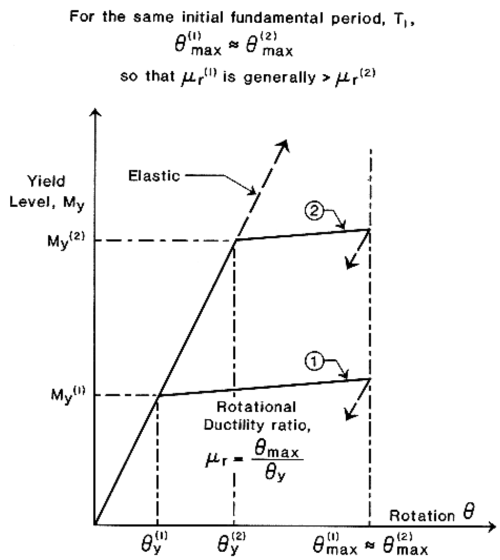



*Figure 10-2.* Decrease in ductility ratio demand with increase in yield level or strength of a structure.

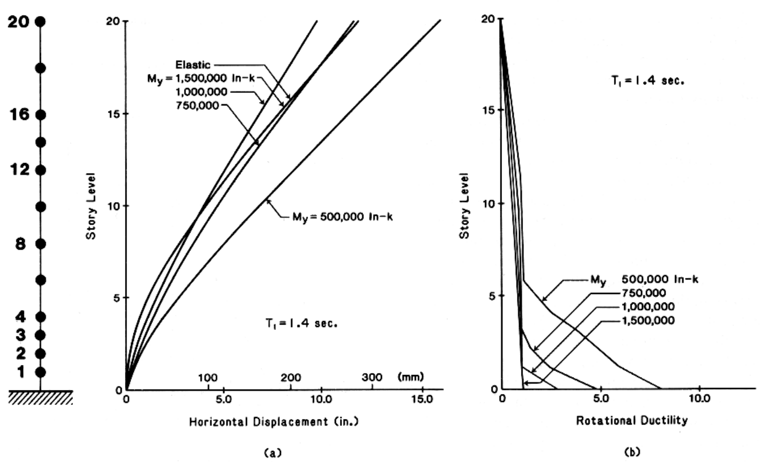



*Figure 10-3.* Effect of yield level on ductility demand. Note approximately equal maximum displacements for structures with reasonable yield levels. (From Ref. 10-3.)

A plot showing the variation of rotational ductility demand at the base of an isolated structural wall with both the flexural yield level and the initial fundamental period is shown in Figure 10-4.(10-4) The results shown in Figure 10-4 were obtained from dynamic inelastic analysis of models representing 20-story isolated structural walls subjected to six input motions of 10-sec duration having different frequency characteristics and an intensity normalized to 1.5 times that of the north—south component of the 1940 El Centro record. Again, note the increase in ductility demand with decreasing yield level; also the decrease in ductility demand with increasing fundamental period of the structure.

The above-noted relationship between strength or yield level and ductility is the basis for code provisions requiring greater strength (by specifying higher design lateral forces) for materials or systems that are deemed to have less available ductility.

### 10.2.3 Some Remarks about Ductility

One should note the distinction between inelastic deformation demand expressed as a *ductility ratio,* $\mu$ (as it usually is) on one hand, and in terms of absolute rotation on the other. An observation made with respect to one quantity may not apply to the other. As an example, Figure 10-5, from Reference 10-3,

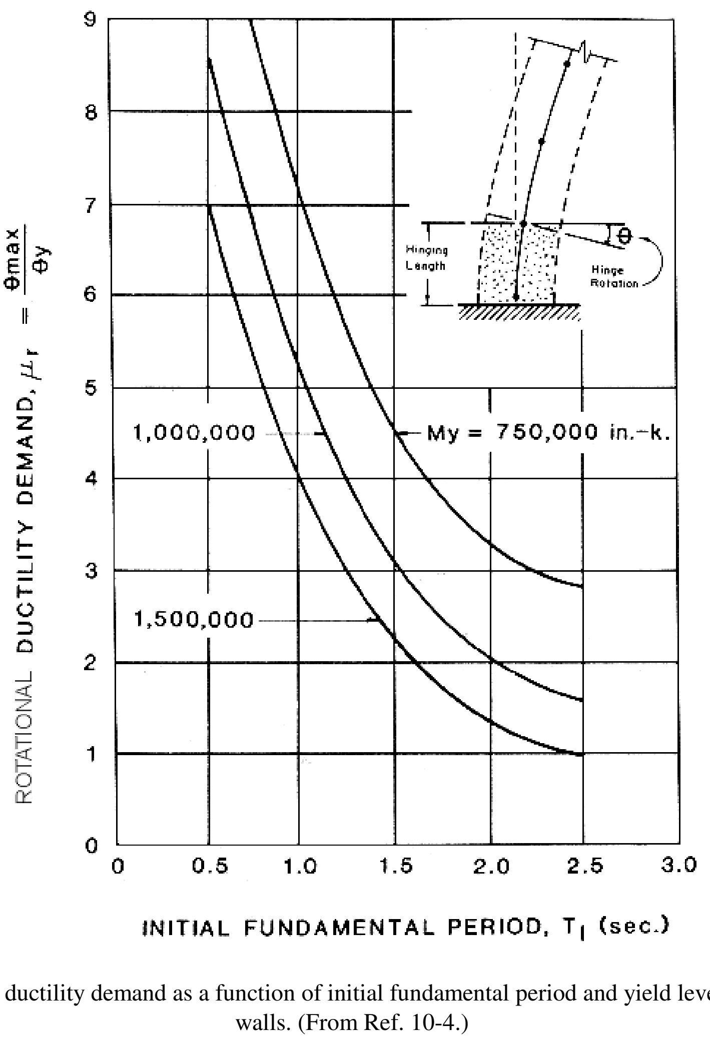



*Figure 10-4.* Rotational ductility demand as a function of initial fundamental period and yield level of 20-story structural walls. (From Ref. 10-4.)

shows results of dynamic analysis of two isolated structural walls having the same yield level *($M_y$ = 500,000 in.-kips)* but different stiffnesses, as reflected in the lower initial fundamental period $T_1$ of the stiffer structure. Both structures were subjected to the E—W component of the 1940 El Centro record. Even though the maximum rotation for the flexible structure (with $T_1$ = 2.0 sec) is 3.3 times that of the stiff structure, the ductility ratio for the stiff structure is 1.5 times that of the flexible structure. The latter result is, of course, partly due to the lower yield rotation of the stiffer structure.

$M_y$ = 500,000 in-kips

| | Yield Rotation $\theta_y$(rad.) | Max. Rotation $\theta_{max.}$(rad.) |
|---|---|---|
| 1 | .00014 | .00135 |
| 2 | .00070 | .00448 |

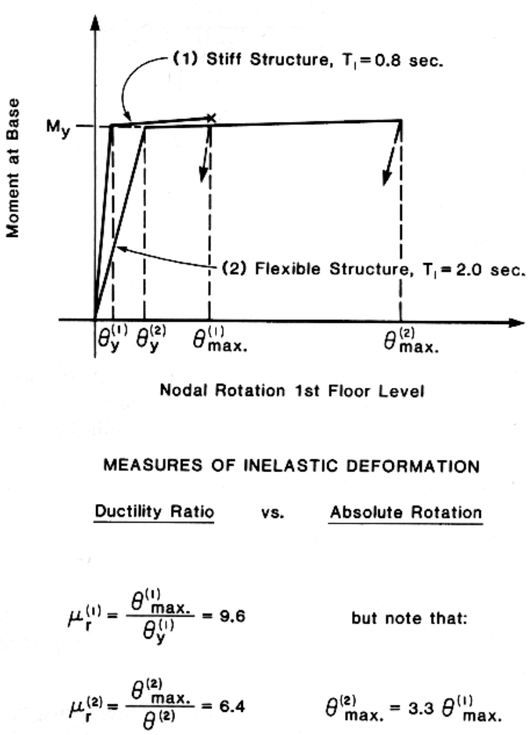



*Figure 10-5.* Rotational ductility ratio versus maximum absolute rotation as measures of inelastic deformation.

The term "curvature ductility" is also a commonly used term which is defined as rotation per unit length. This is discussed in detail later in this Chapter.

Another important distinction worth noting with respect to ductility is the difference between displacement ductility and rotational ductility. The term *displacement ductility* refers to the ratio of the maximum horizontal (or transverse) displacement of a structure to the corresponding displacement at first yield. In a rigid frame or even a single cantilever structure responding inelastically to earthquake excitation, the lateral displacement of the structure is achieved by flexural yielding at local critically stressed regions. Because of this, it is reasonable to expect—and results of analyses bear this out[10-2, 10-3, 10-5]—that rotational ductilities at these critical regions are generally higher than the associated displacement ductility. Thus, overall displacement ductility ratios of 3 to 6 may imply local rotational ductility demands of 6 to 12 or more in the critically stressed regions of a structure.

### 10.2.4 Results of a Recent Study on Cantilever Walls

In a recent study by Priestley and Kowalsky [10-6] on isolated cantilever walls, it has been shown that the yield curvature is not directly proportional to the yield moment; this is in contrast to that shown in Figure 10-2 which in their opinions leads to significant errors. In fact, they have shown that yield curvature is a function of the wall length alone, for a given steel yield stress as indicated in Figure 10-6. The strength and stiffness of the wall vary proportionally as the strength of the section is changed by varying the amount of flexural reinforcement and/or the level of axial load. This implies that the yield curvature, not the section stiffness, should be considered the fundamental section property. Since wall yield curvature is inversely proportional to wall length, structures containing walls of different length cannot be designed such that they yield simultaneously. In addition, it is stated that wall design should be proportioned to the square of wall length, $L^2$, rather than the current design assumption, which is based on $L^3$ .

It should be noted that the above findings apply to cantilever walls only. Further research in this area in various aspects is currently underway at several institutions.

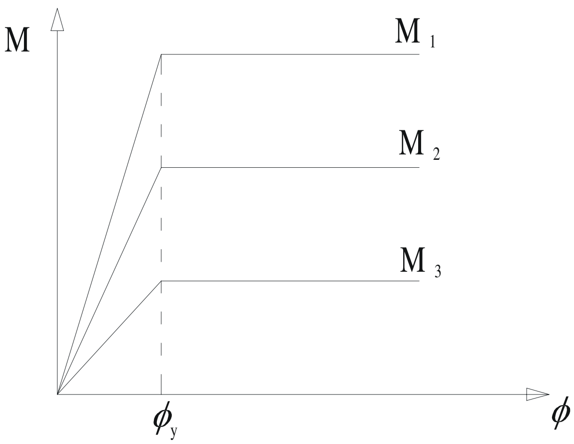



*Figure 10-6.* Influence of strength on moment-curvature relationship (From Ref. 10-6).

## 10.3 BEHAVIOR OF CONCRETE MEMBERS UNDER EARTHQUAKE-TYPE LOADING

### 10.3.1 General Objectives of Member Design

A general objective in the design of reinforced concrete members is to so proportion such elements that they not only possess adequate stiffness and strength but so that the strength is, to the extent possible, governed by flexure rather than by shear or bond/anchorage. Code design requirements are framed with the intent of allowing members to develop their flexural or axial load capacity before shear or bond/anchorage failure occurs. This desirable feature in conventional reinforced concrete design becomes imperative in design for earthquake motions where significant ductility is required.

In certain members, such as conventionally reinforced short walls—with height-to-width ratios of 2 to 3 or less—the very nature of the principal resisting mechanism would make a shear-type failure difficult to avoid. Diagonal reinforcement, in conjunction with horizontal and vertical reinforcement, has been shown to improve the performance of such members [(10-7)].

### 10.3.2 Types of Loading Used in Experiments

The bulk of information on behavior of reinforced-concrete members under load has ‘generally been obtained from tests of full-size or near-full-size specimens. The loadings used in these tests fall under four broad categories, namely:

1. *Static monotonic loading*—where load in one direction only is applied in increments until failure or excessive deformation occurs. Data which form the basis for the design of reinforced concrete members under gravity and wind loading have been obtained mainly from this type of test. Results of this test can serve as bases for comparison with results obtained from other types of test that are more representative of earthquake loading.

2. *Slowly reversed cyclic (“quasistatic”) loading*—where the specimen is subjected to (force or deformation) loading cycles of predetermined amplitude. In most cases, the load amplitude is progressively increased until failure occurs. This is shown schematically in Figure 10-7a. As mentioned earlier, much of the data upon which current design procedures for earthquake resistance are based have been obtained from tests of this type. In a few cases, a loading program patterned after analytically determined dynamic response[(10-8)] has been used. The latter, which is depicted in Figure 10-7b, is usually characterized by large-amplitude load cycles early in the test, which can produce early deterioration of the strength of a specimen.[(10-9)] In both of the above cases, the load application points are fixed so that the moments and shears are always in phase—a condition, incidentally, that does not always occur in dynamic response.

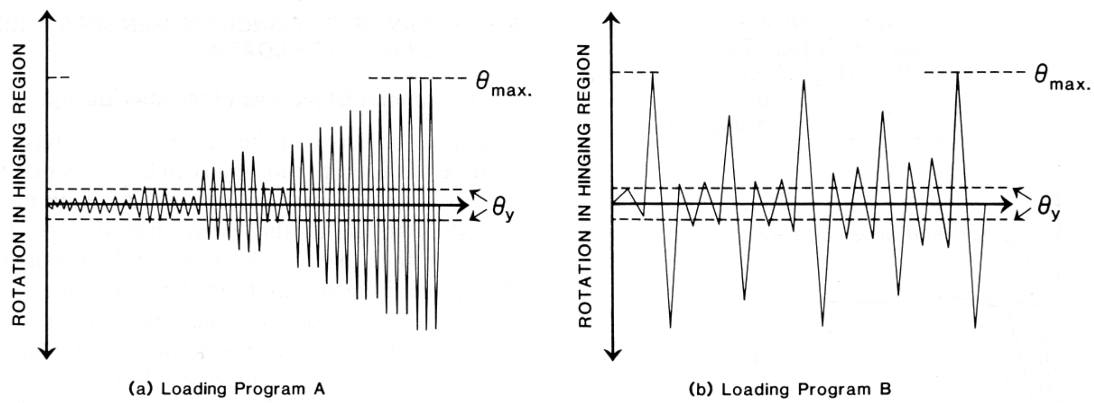



*Figure 10-7.* Two types of loading program used in quasi-static tests.

This type of test provides the reversing character of the loading that distinguishes dynamic response from response to unidirectional static loading. In addition, the relatively slow application of the load allows close observation of the specimen as the test progresses. However, questions concerning the effects of the sequence of loading as well as the phase relationship between moment and shear associated with this type of test as it is normally conducted need to be explored further.

3. *Pseudo-dynamic tests.* In this type of test, the specimen base is fixed to the test floor while time-varying displacements determined by an on-line computer are applied to selected points on the structure. By coupling loading rams with a computer that carries out an incremental dynamic analysis of the specimen response to a preselected input motion, using measured stiffness data from the preceding loading increment and prescribed data on specimen mass and damping, a more realistic distribution of horizontal displacements in the test structure is achieved. The relatively slow rate at which the loading is imposed allows convenient inspection of the condition of the structure during the progress of the test.

This type of test, which has been used mainly for testing structures, rather than members or structural elements, requires a fairly large reaction block to take the thrust from the many loading rams normally used.

4. *Dynamic tests using shaking tables (earthquake simulators).* The most realistic test conditions are achieved in this setup, where a specimen is subjected to a properly scaled input motion while fastened to a test bed impelled by computer-controlled actuators. Most current earthquake simulators are capable of imparting controlled motions in one horizontal direction and in the vertical direction.

The relatively rapid rate at which the loading is imposed in a typical dynamic test generally does not allow close inspection of the specimen while the test is in progress, although photographic records can be viewed after the test. Most currently available earthquake simulators are limited in their capacity to small-scale models of multistory structures or near-full-scale models of segments of a structure of two or three stories. The difficulty of viewing the progress of damage in a specimen as the loading is applied and the limited capacity of available (and costly) earthquake simulators has tended to favor the recently developed pseudo-dynamic test as a basic research tool for testing structural systems.

The effect of progressively increasing lateral displacements on actual structures has been studied in a few isolated cases by means of forced-vibration testing. These tests have usually been carried out on buildings or portions of buildings intended for demolition.

### 10.3.3 Effects of Different Variables on the Ductility of Reinforced Concrete Members

Figure 10-8 shows typical stress—strain curves of concrete having different compressive strengths. The steeper downward slope beyond the point of maximum stress of curves corresponding to the higher strength concrete is worth noting. The greater ductility of the lower-strength concrete is apparent in the figure. Typical stress-strain curves for the commonly available grades of reinforcing steel, with nominal yield strengths of 60 ksi and 40 ksi, are shown in Figure 10-9. Note in the figure that the ultimate stress is significantly higher than the yield stress. Since strains well into the strain-hardening range can occur in hinging regions of flexural members, stresses in excess of the nominal yield stress (normally used in conventional design as the limiting stress in steel) can develop in the reinforcement at these locations.

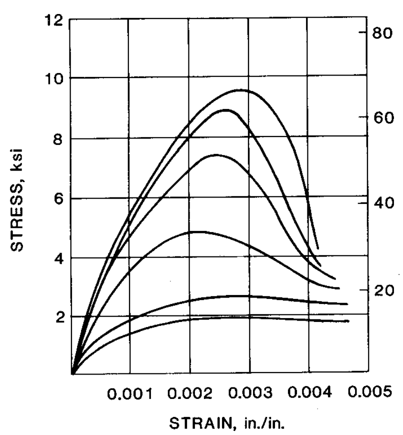



*Figure 10-8.* Typical stress-strain curves for concrete of varying compressive strengths.

***Rate of Loading*** An increase in the strain rate of loading is generally accompanied by an increase in the strength of concrete or the yield stress of steel. The greater rate of loading associated with earthquake response, as compared with static loading, results in a slight increase in the strength of reinforced concrete members, due primarily to the increase in the yield strength of the reinforcement. The calculation of the strength of reinforced concrete members in earthquake-resistant structures on the basis of material properties obtained by static tests (i.e., normal strain rates of loading) is thus reasonable and conservative.

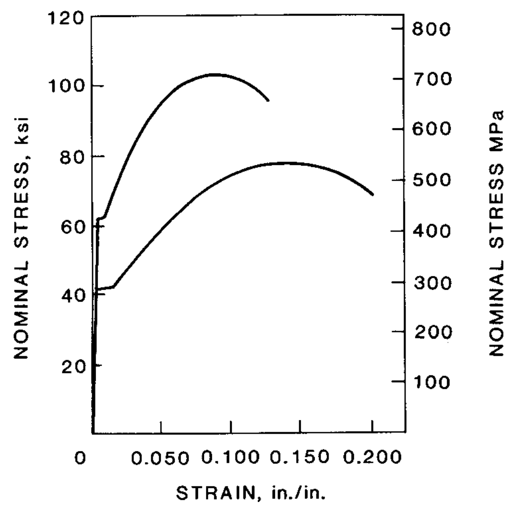



*Figure 10-9.* Typical stress-strain curves for ordinary reinforcing steel.

***Confinement Reinforcement*** The American Concrete Institute *Building Code Requirements for Reinforced Concrete,* ACI 318-95[10-10] (hereafter referred to as the ACI Code), specifies a maximum usable compressive strain in concrete, $\varepsilon_{cu}$ of 0.003. Lateral confinement, whether from active forces such as transverse compressive loads, or passive restraints from other framing members or lateral reinforcement, tends to increase the value of $\varepsilon_{cu}$. Tests have shown that $\varepsilon_{cu}$, can range from 0.0025 for unconfined concrete to about 0.01 for concrete confined by lateral reinforcement subjected to predominantly axial (concentric) load. Under eccentric loading, values of $\varepsilon_{cu}$ for confined concrete of 0.05 and more have been observed.[10-11, 10-12,10-13]

Effective lateral confinement of concrete increases its compressive strength and deformation capacity in the longitudinal direction, whether such longitudinal stress represents a purely axial load or the compressive component of a bending couple.

In reinforced concrete members, the confinement commonly takes the form of lateral ties or spiral reinforcement covered by a thin shell of concrete. The passive confining effect of the lateral reinforcement is not mobilized until the concrete undergoes sufficient lateral expansion under the action of compressive forces in the longitudinal direction. At this stage, the outer shell of concrete usually has reached its useful load limit and starts to spall. Because of this, the net increase in strength of the section due to the confined core may not amount to much in view of the loss in capacity of the spalled concrete cover. In many cases, the total strength of the confined core may be slightly less than that of the original section. The increase in ductility due to effective confining reinforcement, however, is significant.

The confining action of rectangular hoops mainly involves reactive forces at the corners, with only minor restraint provided along the straight unsupported sides. Because of this, rectangular hoops are generally not as effective as circular spiral reinforcement in confining the concrete core of members subjected to compressive loads. However, confinement in rectangular sections can be improved using additional transverse ties. Square spirals, because of their continuity, are slightly better than separate rectangular hoops.

The stress—strain characteristics of concrete, as represented by the maximum usable compressive strain $\varepsilon_{cu}$ is important in designing for ductility of reinforced concrete members. However, other factors also influence the ductility of a section: factors which may increase or diminish the effect of confinement on the ductility of concrete. Note the distinction between the ductility of concrete as affected by confinement and the ductility of a reinforced concrete section (i.e., sectional ductility) as influenced by the ductility of the concrete as well as other factors.

***Sectional Ductility*** A convenient measure of the ductility of a section subjected to flexure or combined flexure and axial load is the ratio $\mu$ of the ultimate curvature attainable without significant loss of strength, $\phi_u$ , to the curvature corresponding to first yield of the tension reinforcement, $\phi_y$. Thus

$$\text{Sectional ductility, } \mu = \frac{\phi_u}{\phi_y}$$

Figure 10-10, which shows the strains and resultant forces on a typical reinforced concrete section under flexure, corresponds to the condition when the maximum usable compressive strain in concrete, $\varepsilon_{cu}$ is reached. The corresponding curvature is denoted as the

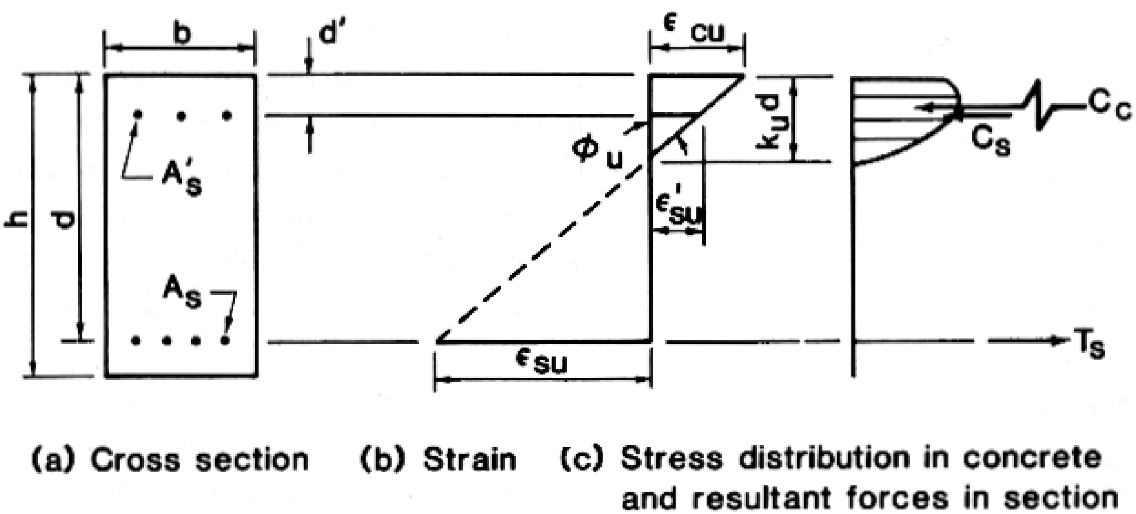



*Figure 10-10.* Strains and stresses in a typical reinforced concrete section under flexure at ultimate condition.

ultimate curvature, $\phi_u$. It will be seen in the figure that

$$\phi_u = \frac{\varepsilon_{cu}}{k_u d}$$

where $k_u d$ is the distance from the extreme compression fiber to the neutral axis.

The variables affecting sectional ductility may be classified under three groups, namely: (i) material variables, such as the maximum usable compressive strain in concrete, particularly as this is affected by confinement, and grade of reinforcement; (ii) geometric variables, such as the amount of tension and compression reinforcement, and the shape of the section; (iii) loading variables, such as the level of the axial load and accompanying shear.

As is apparent from the above expression for ultimate curvature, factors that tend to increase $\varepsilon_{cu}$ or decrease $k_u d$ tend to increase sectional ductility. As mentioned earlier, a major factor affecting the value of $\varepsilon_{cu}$ is lateral confinement. Tests have also indicated that $\varepsilon_{cu}$ increases as the distance to the neutral axis decreases, that is, as the strain gradient across the section increases[10-14, 10-15] and as the moment gradient along the span of the member increases or as the shear span decreases.[10-16, 10-17] (For a given maximum moment, the moment gradient increases as the distance from the point of zero moment to the section considered decreases.)

The presence of compressive reinforcement and the use of concrete with a high compressive strength,[a] as well as the use of flanged sections, tend to reduce the required depth of the compressive block, $k_u d$, and hence to increase the ultimate curvature $\phi_u$. In addition, the compressive reinforcement also helps confine the concrete compression zone and, in combination with adequate transverse reinforcement, allows the spread of the inelastic action in a hinging region over a longer length than would otherwise occur, thus improving the ductility of the member.[10-19] On the other hand, compressive axial loads and large amounts of tensile reinforcement, especially tensile reinforcement with a high yield stress, tend to increase the required $k_u d$ and thus decrease the ultimate curvature $\phi_u$.

Figure 10-11 shows axial-load—moment-strength interaction curves for a reinforced-concrete section subjected to a compressive axial load and bending about the horizontal axis. Both confined and unconfined conditions are assumed. The interaction curve provides a convenient way of displaying the combinations of bending moment *M* and axial load *P* which a given section can carry. A point on the interaction curve is obtained by calculating the forces *M* and *P* associated with an assumed linear strain distribution across the section, account being taken of the appropriate stress—strain relationships for concrete and steel. For an ultimate load curve, the concrete strain at the extreme compressive fiber, $\varepsilon_c$ is assumed to be at the maximum usable strain, $\varepsilon_{cu}$ while the strain in the tensile reinforcement, $\varepsilon_s$, varies. A loading combination represented by a point on or inside the interaction curve can be safely resisted by the section. The balance point in the interaction curve corresponds to the condition in which the tensile reinforcement is stressed to its yield point at the same time that the extreme concrete fiber reaches its useful limit of compressive strain. Points on the interaction curve above the balance point represent conditions in which the strain in the tensile reinforcement is less than its yield strain $\varepsilon_y$, so that the strength of the section in this range is governed by failure of the concrete compressive zone. For those points on the curve below the balance point, $\varepsilon_s > \varepsilon_y$. Hence, the strength of the section in this range is governed by rupture of the tensile reinforcement.

Figure 10-11 also shows the variation of the ultimate curvature $\phi_u$ (in units of $1/h$) with the axial load *P*. It is important to note the greater ultimate curvature (being a measure of sectional ductility) associated with values of *P* less than that corresponding to the balance condition, for both unconfined and confined cases. The significant increase in ultimate curvature

[a] The lower ductility of the higher-strength ($f'_c$ >5000 psi ), however, has been shown to result in a decrease in sectional ductility, particularly for sections with low reinforcement indexes. [10-18]

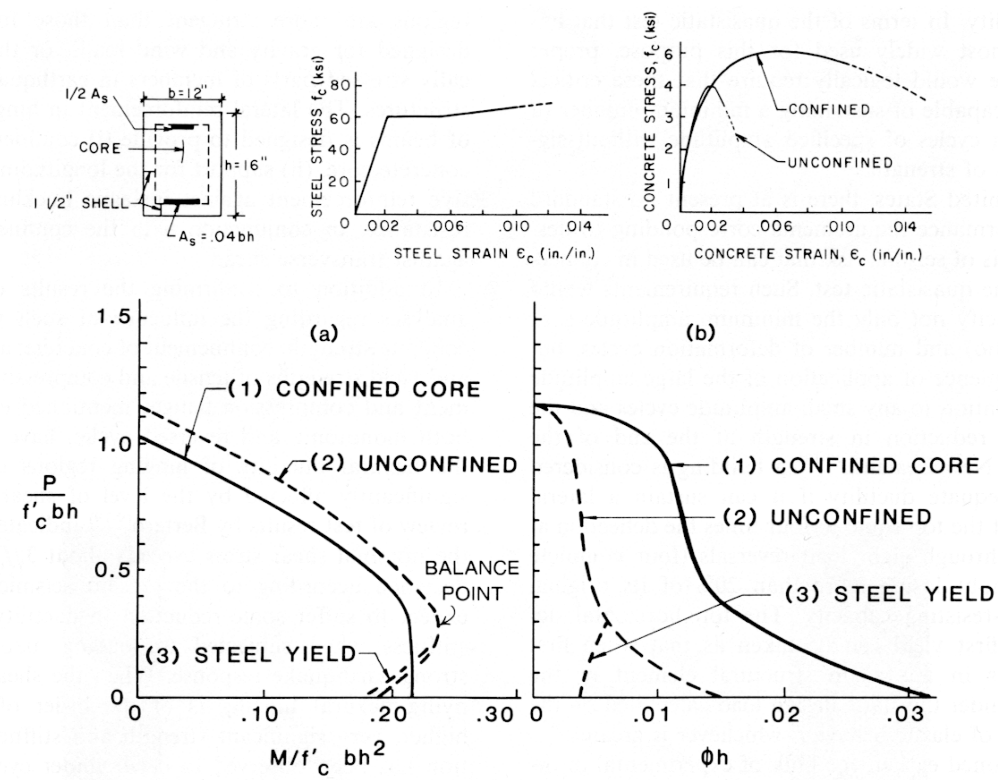



*Figure 10-11.* Axial load-moment interaction and load-curvature curves for a typical reinforced concrete section with unconfined and confined cores.

resulting from confinement is also worth noting in Figure 10-11b.

In the preceding, the flexural deformation capacity of the hinging region in members was examined in terms of the curvature at a section, $\phi$, and hence the sectional or curvature ductility. Using this simple model, it was possible to arrive at important conclusions concerning the effects of various parameters on the ductility of reinforced concrete members. In the hinging region of members, however, the curvature can vary widely in value over the length of the "plastic hinge." Because of this, the total rotation over the plastic hinge, $\theta$, provides a more meaningful measure of the inelastic flexural deformation in the hinging regions of members and one that can be related directly to experimental measurements. (One can, of course, speak of average curvature over the hinging region, i.e., total rotation divided by length of the plastic hinge.)

***Shear*** The level of shear present can have a major effect on the ductility of flexural hinging regions. To study the effect of this variable, controlled tests of laboratory specimens have been conducted. This will be discussed further in the following section.

### 10.3.4 Some Results of Experimental and Analytical Studies on the Behavior of Reinforced Concrete Members under Earthquake-Type Loading and Related Code Provisions

Experimental studies of the behavior of structural elements under earthquake-type loading have been concerned mainly with identifying and/or quantifying the effects of variables that influence the ability of critically stressed regions in such specimens to perform properly. Proper performance means primarily possessing adequate ductility. In terms of the quasistatic test that has been the most widely used for this purpose, proper performance would logically require that these critical regions be capable of sustaining a minimum number of deformation cycles of specified amplitude without significant loss of strength.

In the United States, there is at present no standard set of performance requirements corresponding to designated areas of seismic risk that can be used in connection with the quasi-static test. Such requirements would have to specify not only the minimum amplitude (i.e., ductility ratio) and number of deformation cycles, but also the sequence of application of the large-amplitude cycles in relation to any small-amplitude cycles and the permissible reduction in strength at the end of the loading.

As mentioned earlier, the bulk of experimental information on the behavior of elements under earthquake-type loading has been obtained by quasi-static tests using loading cycles of progressively increasing amplitude, such as is shown schematically in Figure 10-7a. Adequacy with respect to ductility for regions of high seismicity has usually been inferred when displacement ductility ratios of anywhere from 4 to 6 or greater were achieved without appreciable loss of strength. In New Zealand,[10-20] moment resisting frames are designed for a maximum ductility, $\mu$, of 6 and shear walls are designed for a maximum ductility of between 2.5 to 5. Adequate ductile capacity is considered to be present if all primary that are required to resist earthquake-induced forces are accordingly designed and detailed.

In the following, some results of tests and analyses of typical reinforced-concrete members will be briefly reviewed. Where appropriate, related code provisions, mainly those in Chapter 21 of the ACI Code[10-10] are also discussed.

***Beams*** Under earthquake loading, beams will generally be most critically stressed at and near their intersections with the supporting columns. An exception may be where a heavy concentrated load is carried at some intermediate point on the span. As a result, the focus of attention in the design of beams is on these critical regions where plastic hinging can take place.

At potential hinging regions, the need to develop and maintain the strength and ductility of the member through a number of cycles of reversed inelastic deformation calls for special attention in design. This special attention relates mainly to the lateral reinforcement, which takes the form of closed hoops or spirals. As might be expected, the requirements governing the design of lateral reinforcement for potential hinging regions are more stringent than those for members designed for gravity and wind loads, or the less critically stressed parts of members in earthquake-resistant structures. The lateral reinforcement in hinging regions of beams is designed to provide (i) confinement of the concrete core, (ii) support for the longitudinal compressive reinforcement against inelastic buckling, and (iii) resistance, in conjunction with the confined concrete, against transverse shear.

In addition to confirming the results of sectional analyses regarding the influence of such variables as concrete strength, confinement of concrete, and amounts and yield strengths of tensile and compressive reinforcement and compression flanges mentioned earlier, tests, both monotonic and reversed cyclic, have shown that the flexural ductility of hinging regions in beams is significantly affected by the level of shear present. A review of test results by Bertero[10-21] indicates that when the nominal shear stress exceeds about $3\sqrt{f_c'}$ , members designed according to the present seismic codes can expect to suffer some reduction in ductility as well as stiffness when subjected to loading associated with strong earthquake response. When the shear accompanying flexural hinging is of the order of $5\sqrt{f_c'}$ or higher, very significant strength and stiffness degradation has been observed to occur under cyclic reversed loading.

The behavior of a segment at the support region of a typical reinforced-concrete beam subjected to reversed cycles of inelastic deformation in the presence of high shear[10-22, 10-23) is shown schematically in Figure 10-12. In Figure 10-12a, yielding of the top longitudinal steel under a downward movement of the beam end causes flexure—shear cracks to form at the top. A reversal of the load and subsequent yielding of the bottom longitudinal steel is also accompanied by cracking at the bottom of the beam (see Figure 10-l2b). If the area of the bottom steel is at least equal to that of the top steel, the top cracks remain open during the early stages of the load reversal until the top steel yields in compression, allowing the top crack to close and the concrete to carry some compression. Otherwise, as in the more typical case where the top steel has greater area than the bottom steel, the top steel does not yield in compression (and we assume it does not buckle), so that the top crack remains open during the reversal of the load (directed upward). Even in the former case, complete closure of the crack at the top may be prevented by loose particles of concrete that may fall into the open cracks. With a crack traversing the entire depth of the beam, the resisting flexural couple consists of the forces in the tensile and compressive steel areas, while the shear along the through-depth crack is resisted primarily by dowel action of the longitudinal steel. With subsequent reversals of the load and progressive deterioration of the concrete in the hinging region (Figure 10-12c), the through-depth crack widens. The resulting increase in total length of the member due to the opening of through-depth cracks under repeated load reversals is sometimes referred to as *growth* of the member.

Where the shear accompanying the moment is high, sliding along the through-depth crack(s) can occur. This sliding shear displacement, which is resisted mainly by dowel action of the longitudinal reinforcement, is reflected in a *pinching* of the associated load—deflection curve near the origin, as indicated in Figure 10-13. Since the area under the load—deflection curve is a measure of the energy-dissipation capacity of the member, the pinching in this curve due to sliding shear represents a degradation not only of the strength but also the energy-dissipation capacity of the hinging region. Where the longitudinal steel is not adequately restrained by lateral reinforcement, inelastic buckling of the compressive reinforcement followed by a rapid loss of flexural strength can occur.

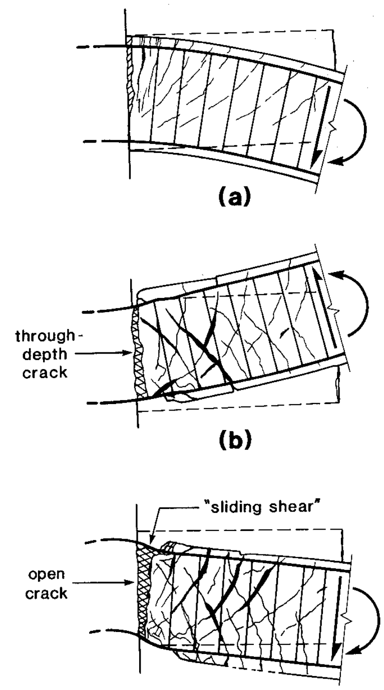



*Figure 10-12.* Plastic hinging in beam under high shear. (Adapted from Ref. 10-31.)

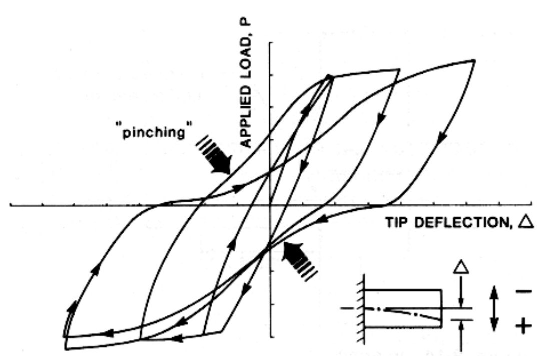



*Figure 10-13.* Pinching in load-displacement hysteresis loop due to mainly to sliding shear

Because of the significant effect that shear can have on the ductility of hinging regions, it has been suggested(10-24) that when two or more load reversals at a displacement ductility of 4 or more are expected, the nominal shear stress in critical regions reinforced according to normal

U.S. code requirements for earthquake-resistant design should be limited to 6 $\sqrt{f_c'}$. Results of tests reported in Reference 10-24 have shown that the use of crossing diagonal or inclined web reinforcement, in combination with vertical ties, as shown in Figure 10-14, can effectively minimize the degradation of stiffness associated with sliding shear. Relatively stable hysteretic force—displacement loops, with minimal or no pinching, were observed. Tests reported in Reference 10-25 also indicate the effectiveness of intermediate longitudinal shear reinforcement, shown in Figure 10-15, in reducing pinching of the force—displacement loops of specimens subjected to moderate levels of shear stresses, i.e., between $3\sqrt{f_c'}$ and $6\sqrt{f_c'}$.

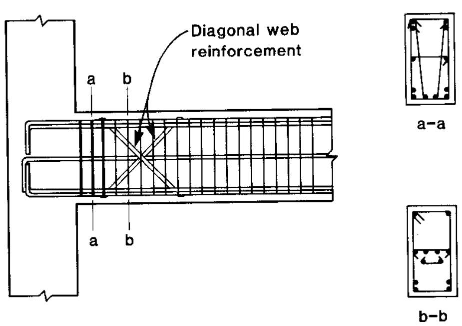



*Figure 10-14.* Crossing diagonal web reinforcement in combination with vertical web steel for hinging regions under high shear. (Adapted from Ref. 10-24)

As mentioned earlier, a major objective in the design of reinforced concrete members is to have the strength controlled by flexure rather than shear or other less ductile failure mechanisms. To insure that beams develop their full strength in flexure before failing in shear, ACI Chapter 21 requires that the design for shear in beams be based not on the factored shears obtained from a lateral-load analysis but rather on the shears corresponding to the maximum *probable flexural strength, $M_{pr}$*, that can be developed at the beam ends. Such a probable flexural strength is calculated by assuming the stress in the tensile reinforcement to be equal to $1.25f_y$ and using a strength reduction factor $\phi$ equal to 1.0 (instead of 0.9). This is illustrated in Figure 10-16 for the case of uniformly distributed beam. The use of the factor 1.25 to be applied to $f_y$ is intended to take account of the likelihood of the actual yield stress in the steel being greater (tests indicate it to be commonly 10 to 25% greater) than the specified nominal yield stress, and also in recognition of the strong possibility of strain hardening developing in the reinforcement when plastic hinging occurs at the beam ends.

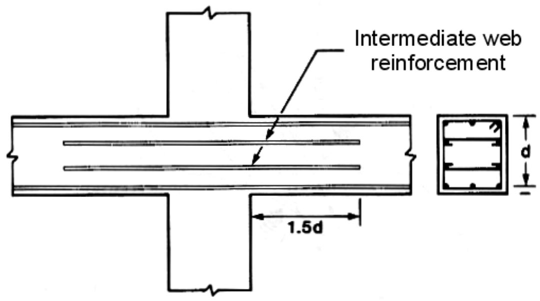
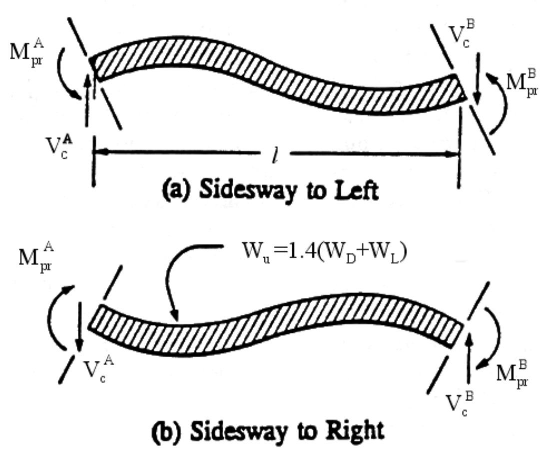



*Figure 10-15.* Intermediate longitudinal web reinforcement for hinging regions under moderate levels of shear.




$$V_c^A = \frac{M_{pr}^A + M_{pr}^B}{l} + \frac{W_u l}{2}$$

$$V_c^B = \frac{M_{pr}^A + M_{pr}^B}{l} - \frac{W_u l}{2}$$

$M_{pr}$ based on $f_s = 1.25 f_y$ and $\phi = 1.0$

*Figure 10-16.* Loading cases for shear design of beams uniformly distributed gravity loads

ACI Chapter 21 requires that when the earthquake-induced shear force calculated on the basis of the maximum probable flexural strength at the beam ends is equal to or more than one-half the total design shear, the contribution of the concrete in resisting shear, $V_c$, be neglected if the factored axial compressive force including earthquake effects is less than $A_g f_c' /20$, where $A_g$ is the gross area of the member cross-section. In the 1995 New Zealand Code,[10-26] the concrete contribution is to be entirely neglected and web reinforcement provided to carry the total shear force in plastic-hinging regions. It should be pointed out that the New Zealand seismic design code appears to be generally more conservative than comparable U.S. codes. This will be discussed further in subsequent sections.

***Columns*** The current approach to the design of earthquake-resistant reinforced concrete rigid (i.e., moment-resisting) frames is to have most of the significant inelastic action or plastic hinging occur in the beams rather than in the columns. This is referred to as the "strong column-weak beam" concept and is intended to help insure the stability of the frame while undergoing large lateral displacements under earthquake excitation. Plastic hinging at both ends of most of the columns in a story can precipitate a story-sidesway mechanism leading to collapse of the structure at and above the story.

ACI Chapter 21 requires that the sum of the flexural strengths of the columns meeting at a joint, under the most unfavorable axial load, be at least equal to 1.2 times the sum of the design flexural strengths of the girders in the same plane framing into the joint. The most unfavorable axial load is the factored axial force resulting in the lowest corresponding flexural strength in the column and which is consistent with the direction of the lateral forces considered. Where this requirement is satisfied, closely spaced transverse reinforcement need be provided only over a short distance near the ends of the columns where potential hinging can occur. Otherwise, closely spaced transverse reinforcement is required over the full height of the columns.

The requirements associated with the strong column-weak beam concept, however, do not insure that plastic hinging will not occur in the columns. As pointed out in Reference 10-5, a bending-moment distribution among frame members such as is shown in Figure 10-17, characterized by points of inflection located away from the mid-height of columns, is not uncommon. This condition, which has been observed even under static lateral loading, occurs when the flexural mode of deformation (as contrasted with the shear—beam component of deformation) in tall frame structures becomes significant and may also arise as a result of higher-mode response under dynamic loading. As Figure 10-17 shows, a major portion of the girder moments at a joint is resisted (assuming the columns remain elastic) by one column segment, rather than being shared about equally (as when the points of inflection are located at mid-height of the columns) by the column sections above and below a joint. In extreme cases, such as might result from substantial differences in the stiffnesses of adjoining column segments in a column stack, the point of contraflexure can be outside the column height. In such cases, the moment resisted by a column segment may exceed the sum of the girder moments. In recognition of this, and the likelihood of the hinging region spreading over a longer length than would normally occur, most building codes require confinement reinforcement to be provided over the full height of the column.

Tests on beam-column specimens incorporating slabs,[10-27, 10-28] as in normal monolithic construction, have shown that slabs significantly increase the effective flexural strength of the beams and hence reduce the column-to-beam flexural strength ratio, if the beam strength is based on the bare beam section. Reference 10-27 recommends consideration of the slab reinforcement over a width equal to at least the width of the beam on each side of the member when calculating the flexural strength of the beam.

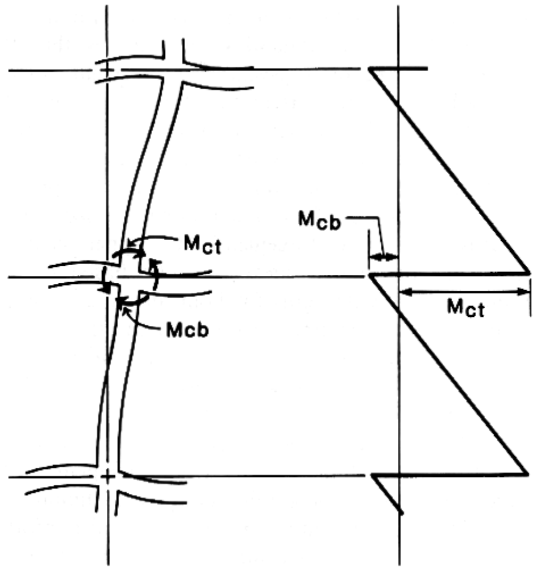



*Figure 10-17.* Distribution of bending moments in columns at a joint when the point of inflection is located away from mid-height.

Another phenomenon that may lead to plastic hinging in the columns occurs in two-way (three-dimensional rigid) frames subjected to ground motions along a direction inclined with respect to the principal axes of the structure. In such cases, the resultant moment from girders lying in perpendicular planes framing into a column will generally be greater than that corresponding to either girder considered separately.(10-5) ( except for certain categories of structures and those with certain irregularities, codes allow consideration of design earthquake loads along each principal axes of a structure separately, as non-concurrent loadings.) Furthermore, the biaxial moment capacity of a reinforced-concrete column under skew bending will generally be less than the larger uniaxial moment capacity. Tests reported in Reference 10-28 indicate that where bi-directional loading occurs in rectangular columns, the decrease in strength of the column due to spalling of concrete cover, and bond deterioration along the column longitudinal bars at and near the corner can be large enough to shift the hinging from the beams to the columns. Thus, under concurrent bi-directional loading, columns in two-way frames designed according to the strong column-weak beam concept mentioned above can either yield before the framing girders or start yielding immediately following yielding of the girders.

It is worth noting that the 1985 report of ACI-ASCE Committee 352 on beam-column joints in monolithic reinforced concrete structures(10-29) recommends a minimum overstrength factor of 1.4, instead of the 1.2 given in ACI 318-95, for the flexural strength of columns relative to that of beams meeting at a joint when the beam strength is based only on the bare beam section (excluding slab). A design procedure *(capacity design),* based on the work of Paulay,(10-13,10-30) that attempts to minimize the possibility of yielding in the columns of a typical frame due to the factors described in the preceding paragraph has been adopted in New Zealand.(10-26) The avowed purpose of capacity design is to limit inelastic action, as well as the formation of plastic hinges, to selected elements of the primary lateral-force-resisting system. In the case of frames, the ideal location for plastic hinges would be the beams and the bases of the first or lowest story columns. Other elements, such as columns, are intended to remain essentially elastic under the design earthquake by designing them with sufficient overstrength relative to the yielding members. Thus elements intended to remain elastic are designed to have strengths in the plastic hinges. For all elements, and particularly regions designed to develop plastic hinges, undesirable modes of failure, such as shear or bond/anchorage failures, are precluded by proper design/detailing. The general philosophy of capacity design is no different from that underlying the current approach to earthquake-resistant design found in ACI Chapter 21, UBC-97 and IBC-2000. The principle difference lies in the details of implementation and particularly in the recommended overstrength factors. For example, the procedure prescribes overstrength factors of 1.5 or greater(10-13,10-32) for determining the flexural strength of columns relative to beams. This compares with the 1.2 factor specified in ACI Chapter 21. In capacity design, the flexural strength of T or inverted-L beams is to be determined by considering the slab reinforcement over the specified width (depending upon column location) beyond the column faces as effective in resisting negative moments. It is clear from the above that the New Zealand capacity design requirements call for greater relative column strength than is currently required in U.S. practice. A similar approach has also been adopted in the Canadian Concrete Code of Practice, CSA Standard A23.3-94.[10-33] Reference 10-13 gives detailed recommendations, including worked out examples, relating to the application of capacity design to both frames and structural wall systems.

To safeguard against strength degradation due to hinging in the columns of a frame, codes generally require lateral reinforcement for both confinement and shear in regions of potential plastic hinging. As in potential hinging regions of beams, the closely spaced transverse reinforcement in critically stressed regions of columns is intended to provide confinement for the concrete core, lateral support of the longitudinal column reinforcement against buckling and resistance (in conjunction with the confined core) against transverse shear. The transverse reinforcement can take the form of spirals, circular hoops, or rectangular hoops, the last with crossties as needed.

Early tests[10-34] of reinforced concrete columns subjected to large shear reversals had indicated the need to provide adequate transverse reinforcement not only to confine the concrete but also to carry most, if not all, of the shear in the hinging regions of columns. The beneficial effect of axial load—a maximum axial load of one-half the balance load was used in the tests—in delaying the degradation of shear strength in the hinging region was also noted in these tests. An increase in column strength due to improved confinement by longitudinal reinforcement uniformly distributed along the periphery of the column section was noted in tests reported in Reference 10-35. Tests cited in Reference 10-32 have indicated that under high axial load, the plastic hinging region in columns with confinement reinforcement provided over the usually assumed hinging length (i.e., the longer section dimension in rectangular columns or the diameter in circular columns) tends to spread beyond the confined region. To prevent flexural failure in the less heavily confined regions of columns, the New Zealand Code[10-20] requires that confining steel be extended to 2 to 3 times the usual assumed plastic-hinge length when the axial load exceeds $0.25\phi f_c' A_g$, where $\phi$ = 0.85 and $A_g$ is the gross area of the column section.

The basic intent of the ACI Code provisions relating to confinement reinforcement in potential hinging regions of columns is to preserve the axial-load-carrying capacity of the column after spalling of the cover concrete has occurred. This is similar to the intent underlying the column design provisions for gravity and wind loading. The amount of confinement reinforcement required by these provisions is independent of the level of axial load. Design for shear is to be based on the largest nominal moment strengths at the column ends consistent with the factored design axial compressive load. Some investigators,[10-5] however, have suggested that an approach that recognizes the potential for hinging in critically stressed regions of columns should aim primarily at achieving a minimum ductility in these regions. Studies by Park and associates, based on sectional analyses[10-32] as well as tests,[10-36, 10-37] indicate that although the ACI Code provisions based on maintaining the load-carrying capacity of a column after spalling of the cover concrete has occurred are conservative for low axial loads, they can be unconservative for high axial loads, with particular regard to attaining adequate ductility. Results of these studies indicate the desirability of varying the confinement requirements for the hinging regions in columns according to the magnitude of the axial load, more confinement being called for in the case of high axial loads.

ACI Chapter 21 limits the spacing of confinement reinforcement to 1/4 the minimum member dimension or 4 in., with no limitation related to the longitudinal bar diameter. The New Zealand Code requires that the maximum spacing of transverse reinforcement in the potential plastic hinge regions not exceed the least of 1/4 the minimum column dimension or 6 times the diameter of the longitudinal reinforcement. The second limitation is intended to relate the maximum allowable spacing to the need to prevent premature buckling of the longitudinal reinforcement. In terms of shear reinforcement, ACI Chapter 21 requires that the design shear force be based on the maximum flexural strength, $M_{pr}$ , at each end of the column associated with the range of factored axial loads. However, at each column end, the moments to be used in calculating the design shear will be limited by the probable moment strengths of the beams (the negative moment strength on one side and the positive moment strength on the other side of a joint) framing into the column. The larger amount of transverse reinforcement required for either confinement or shear is to be used.

One should note the significant economy, particularly with respect to volume of lateral reinforcement, to be derived from the use of spirally reinforced columns.[10-32] The saving in the required amount of lateral reinforcement, relative to a tied column of the same nominal capacity, which has also been observed in designs for gravity and wind loading, acquires greater importance in earthquake-resistant design in view of the superior ductile performance of the spirally reinforced column. Figure 10-18b, from Reference 10-38, shows one of the spirally reinforced columns in the first story of the Olive View Hospital building in California following the February 9, 1971 San Fernando earthquake. A tied corner column in the first story of the same building is shown in Figure 10-18c. The upper floors in the four-story building, which were stiffened by shear walls that were discontinued below the second-floor level, shifted approximately 2 ft. horizontally relative to the base of the first-story columns, as indicated in Figure 10-18a.

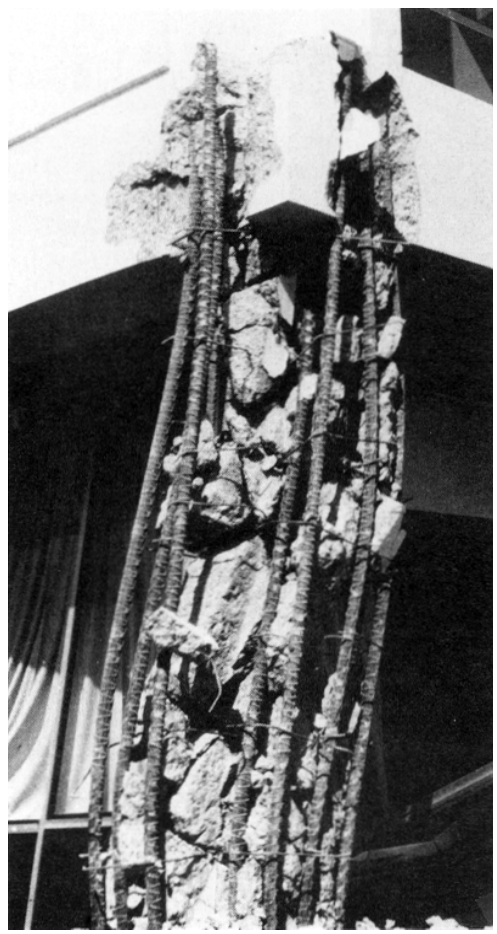
***Beam—Column Joints*** Beam-column joints are critical elements in frame structures. These elements can be subjected to high shear and bond-slip deformations under earthquake loading. Beam-column joints have to be designed so that the connected elements can perform properly. This requires that the joints be proportioned and detailed to allow the columns and beams framing into them to develop and maintain their strength as well as stiffness while undergoing large inelastic deformations. A loss in strength or stiffness in a frame resulting from deterioration in the joints can lead to a substantial increase in lateral displacements of the frame, including possible instability due to P-delta effects.

The design of beam-column joints is primarily aimed at (i) preserving the integrity of the joint so that the strength and deformation capacity of the connected beams and columns can be developed and substantially maintained, and (ii) preventing significant degradation of the joint stiffness due to cracking of the joint and loss of bond between concrete and the longitudinal column and beam reinforcement or anchorage failure of beam reinforcement. Of major concern here is the disruption of the joint core as a result of high shear reversals. As in the hinging regions of beams and columns, measures aimed at insuring proper performance of beam-column joints have focused on providing adequate confinement as well as shear resistance to the joint.

The forces acting on a typical interior beam-column joint in a frame undergoing lateral displacement are shown in Figure 10-19a. It is worth noting in Figure 10-19a that each of the longitudinal beam and column bars is subjected to a pull on one side and a push on the other side of the joint. This combination of forces tends to push the bars through the joint, a condition that leads to slippage of the bars and even a complete pull through in some test specimens. Slippage resulting from bond degradation under repeated yielding of the beam reinforcement is reflected in a reduction in the beam-end fixity and thus increased beam rotations at the column faces. This loss in beam stiffness can lead to increased lateral displacements of the frame and potential instability.

(a)

(b)



(c)

*Figure 10-18.* Damage to columns of the 4-story Olive View Hospital building during the February 9, 1971 San Fernando, California, earthquake. (From Ref. 10-38.) (a) A wing of the building showing approximately 2 ft drift in its first story. (b) Spirally reinforced concrete column in first story. (c) Tied rectangular corner column in first story.

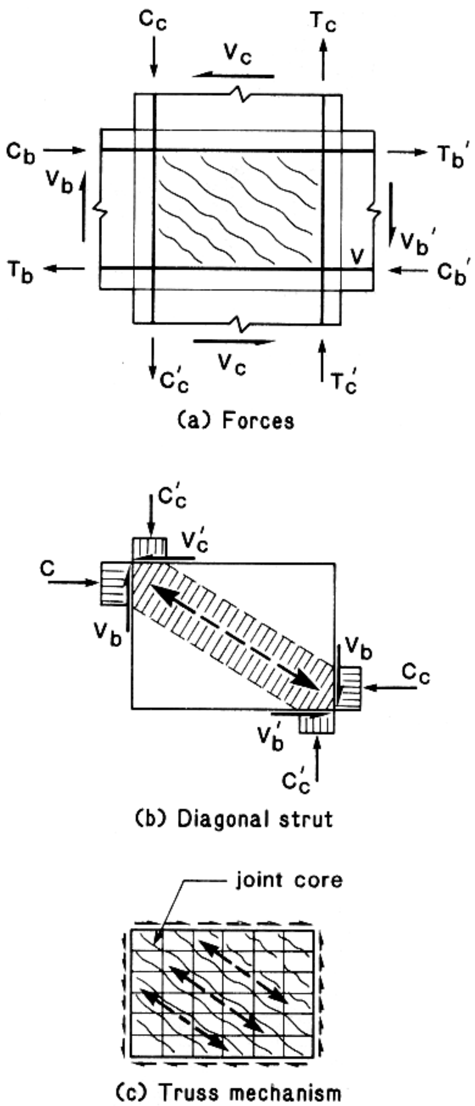



*Figure 10-19.* Forces and postulated shear-resisting mechanisms in a typical interior beam-column joint. (Adapted from Ref. 10-32.) (a) Forces acting on beam-column joint. (b) Diagonal strut mechanism. (c) Truss mechanism.

Two basic mechanisms have been postulated as contributing to the shear resistance of beam—column joints. These are the diagonal strut and the joint truss (or diagonal compression field) mechanisms, shown in Figure 10-19b and c, respectively. After several cycles of inelastic deformation in the beams framing into a joint, the effectiveness of the diagonal strut mechanism tends to diminish as through-depth cracks start to open between the faces of the column and the framing beams and as yielding in the beam bars penetrates into the joint core. The joint truss mechanism develops as a result of the interaction between confining horizontal and vertical reinforcement and a diagonal compression field acting on the elements of the confined concrete core between diagonal cracks. Ideally, truss action to resist horizontal and vertical shears would require both horizontal confining steel and intermediate vertical column bars (between column corner bars). Tests cited in Reference 10-39 indicate that where no intermediate vertical bars are provided, the performance of the joint is worse than where such bars are provided.

Tests of beam-column joints[(10-27,10-40,10-41)] in which the framing beams were subjected to large inelastic displacement cycles have indicated that the presence of transverse beams (perpendicular to the plane of the loaded beams) considerably improves joint behavior. Results reported in Reference 10-27 show that the effect of an increase in joint lateral reinforcement becomes more pronounced in the absence of transverse beams. However, the same tests indicated that slippage of column reinforcement through the joint occurred with or without transverse beams. The use of smaller-diameter longitudinal bars has been suggested [(10-39)] as a means of minimizing bar slippage. Another suggestion has been to force the plastic hinge in the beam to form away from the column face, thus preventing high longitudinal steel strains from developing in the immediate vicinity of the joint. This can be accomplished by suitably strengthening the segment of beam close to the column (usually a distance equal to the total depth of the beam) using appropriate details. Some of the details proposed include a combination of heavy vertical reinforcement with cross-ties (see Figure 10-14), intermediate longitudinal shear reinforcement (see Figure 10-15),[(10-42)] and supplementary flexural reinforcement and haunches, as shown in Figure 10-20.[(10-32)]

The current approach to beam—column joint design in the United States, as contained in ACI Chapter 21, is based on providing sufficient horizontal joint cross-sectional area that is adequately confined to resist the shear stresses in the joint. The approach is based mainly on results of a study by Meinheit and Jirsa[10-41] and subsequent studies by Jirsa and associates. The parametric study reported in Reference 10-41 identified the horizontal cross-sectional area of the joint as the most significant variable affecting the shear strength of beam—column connections. Although recognizing the role of the diagonal strut and joint truss mechanisms, the current approach defines the shear strength of a joint simply in terms of its horizontal cross-sectional area. The approach presumes the provision of confinement reinforcement in the joint. In the ACI Chapter 21 method, shear resistance calculated as a function of the horizontal cross-sectional area at mid-height of the joint is compared with the total horizontal shear across the same mid-height section. Figure 10-21 shows the forces involved in calculating the shear at mid-height of a typical joint. Note that the stress in the yielded longitudinal beam bars is to be taken equal to 1.25 times the specified nominal yield strength $f_y$ of the reinforcement.

The ACI-ASCE Committee 352 Recommendations[10-29] have added a requirement relating to the uniform distribution of the longitudinal column reinforcement around the perimeter of the column core, with a maximum spacing between perimeter bars of 8 in. or one-third the column diameter or the cross-section dimension. The lateral confinement, whether from steel hoops or beams, and the distributed vertical column reinforcement, in conjunction with the confined concrete core, provide the necessary elements for the development of an effective truss mechanism to resist the horizontal and vertical shears acting on a beam—column joint. Results of recent tests on bi-directionally loaded beam—column joint specimens[10-28] confirm the strong correlation between joint shear strength and the horizontal cross-sectional area noted by Meinheit and Jirsa.[10-41]

Some investigators[10-13, 10-32, 10-39] have suggested that the ACI Chapter 21 approach does not fully reflect the effect of the different variables influencing the mechanisms of resistance operating in a beam-column joint and have proposed alternative expressions based on idealizations of the strut and joint truss mechanisms.

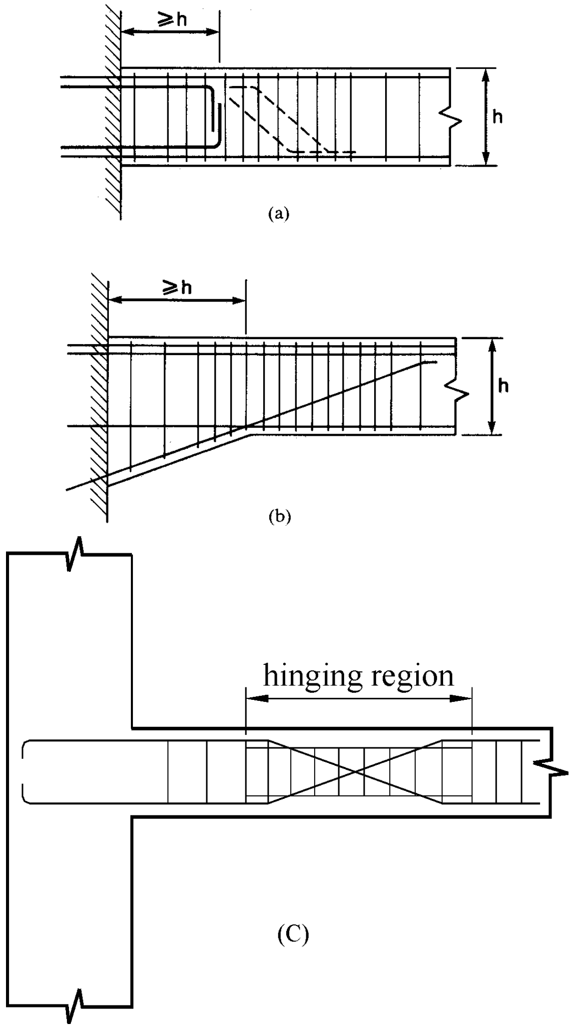



*Figure 10-20.* Proposed details for forcing beam hinging away from column face[10-26]. See also Fig. 10-15. (a) Supplementary flexural reinforcement. (b) Haunch. (c) Special reinforcement detail.

To limit slippage of beam bars through interior beam-column joints, the ACI-ASCE Committee 352 Recommendations call for a minimum column dimension equal to 20 times the diameter of beam bars passing through the joint. For exterior joints, where beam bars terminate in the joint, the maximum size of beam bar allowed is a No. 11 bar.

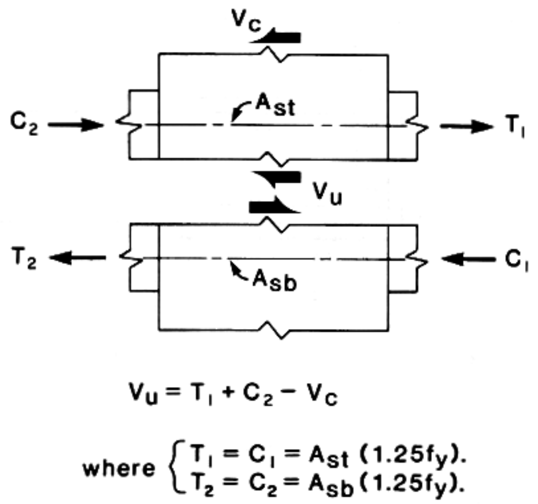



*Figure 10-21.* Shear force at mid-height of beam-column joint- ACI Chapter 21 design practice.

When the depth of an exterior column is not sufficient to accommodate the required development length for beam bars, a beam stub at the far (exterior) side of the column,(10-32) such as is shown in Figure 10-22, can be used. Embedding the $90^{\circ}$ beam bar hooks outside of the heavily stressed joint region reduces the stiffness degradation due to slippage and improves the overall performance of the connection.

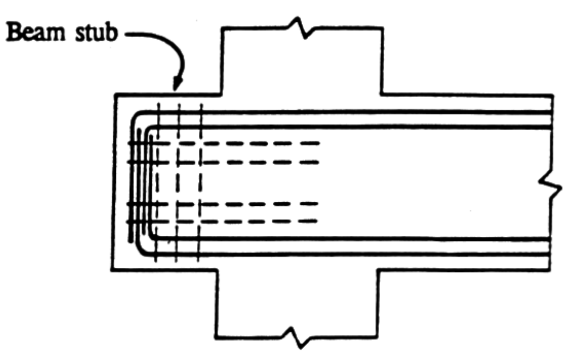



*Figure 10-22.* Exterior beam stub for anchoring beam bars

***Slab—Column Connections*** By omitting consideration of the reinforced concrete flat plate in its provisions governing the design of structures in high-seismic-risk areas, ACI Chapter 21 essentially excludes the use of such a system as part of a ductile frame resisting seismic loads in such areas. Two-way slabs without beams, i.e., flat plates, are, however, allowed in areas of moderate seismic risk.

The flat plate structure is an economical and widely used form of construction in non-seismic areas, especially for multistory residential construction. Its weakest feature, as is well known, is its vulnerability to a punching shear failure at the slab-column junctions. The collapse of a number of buildings using such a system during the 1964 Anchorage, Alaska and the 1967 Caracas, Venezuela earthquakes, as well as several buildings using waffle slabs during the September 1985 Mexican earthquake,(10-43, 10-44) clearly dramatized this vulnerability. Although a flat plate may be designed to carry vertical loads only, with structural walls taking the lateral loads, significant shears may still be induced at the slab-column junctions as the structure displaces laterally during earthquake response.

Tests on slab—column connections subjected to reversed cyclic loading(10-45, 10-46) indicate that the ductility of flat-slab—column connections can be significantly increased through the use of stirrups enclosing bands of flexural slab reinforcement passing through the columns. Such shear-reinforced bands essentially function as shallow beams connecting the columns.

***Structural Walls*** Reinforced concrete structural walls (commonly referred to as shear walls), when properly designed, represent economical and effective lateral stiffening elements that can be used to reduce potentially damaging interstory displacements in multistory structures during strong earthquakes. The structural wall, like the vertical steel truss in steel buildings, has had a long history of use for stiffening buildings laterally against wind forces. The effectiveness of properly designed structural walls in reducing earthquake damage in multistory buildings has been well demonstrated in a number of recent earthquakes.

In earthquake-resistant design, the appreciable lateral stiffness of structural walls can be particularly well utilized in combination with properly proportioned coupling beams in coupled wall systems. Such systems allow considerable inelastic energy dissipation to take place in the coupling beams (which are relatively easy to repair) at critical levels, sometimes even before yielding occurs at the bases of the walls.

Attention in the following discussion will be focused on slender structural walls, i.e., walls with a height-to-width ratio greater than about 2.0, such as are used in multistory buildings. These walls generally behave like vertical cantilever beams. Short or squat walls, on the other hand, resist horizontal forces in their plane by a predominantly truss-type mechanism, with the concrete providing the diagonal compressive strut(s) and the steel reinforcement the equilibrating vertical and horizontal ties. Tests on low-rise walls subjected to slowly reversed horizontal loading[10-47] indicate that for walls with height-to-width ratios of about 1.0 , horizontal and vertical reinforcement are equally effective. As the height-to-width ratio of a wall becomes smaller, the vertical reinforcement becomes more effective in resisting shear than the horizontal steel.[10-48]

In the following discussion, it will be assumed that the isolated structural wall is loaded by a resultant horizontal force acting at some distance above the base. Under such a loading, flexural hinging will occur at the base of the wall. Where the wall is designed and loaded so that it yields in flexure at the base, as might be expected under strong earthquakes, its behavior becomes a function primarily of the magnitude of the shear force that accompanies such flexural hinging as well as the reinforcement details used in the hinging region near the base. Thus, if the horizontal force acts high above the base (long shear arm), it will take a lesser magnitude of the force to produce flexural hinging at the base than when the point of application of the load is close to the base (short shear arm). For the same value of the base yield moment, the moment-to-shear ratio in the former case is high and the magnitude of the applied force (or shear) is low, while in the latter case the moment-to-shear ratio is low and the applied shear is high. In both cases, the magnitude of the applied shear is limited by the flexural yield strength at the base of the wall.

In this connection, it is of interest to note that dynamic inelastic analyses of isolated walls[10-4] covering a wide range of structural and ground motion parameters have indicated that the maximum calculated shear at the base of walls can be from 1.5 to 3.5 times greater than the shear necessary to produce flexural yielding at the base, when such shear is distributed in a triangular manner over the height of the wall, as is prescribed for design in most codes. This is shown in Figure 10-23, which gives the ratio of the calculated maximum dynamic shear, $V^{dyn}_{max}$, to the resultant of the triangularly distributed shear necessary to produce flexural yielding at the base, $V_T$, as a function of the fundamental period $T_1$ and the available rotational ductility $\mu^a_r$ . The input accelerograms used in the analyses had different frequency characteristics and were normalized with respect to intensity so that their spectrum intensity (i.e., the area under the corresponding 5%-damped velocity response spectrum, between periods 0.1 and 3.0 sec) was 1.5 times that of the N-S component of the 1940 El Centro record. The results shown in Figure 10-23 indicate that a resultant shear force equal to the calculated maximum dynamic shear need not be applied as high as two-thirds the height of the wall above the base to produce yielding at the base. Figure 10-24, also from Reference 10-4, shows the distance (expressed as the ratio $M_y / V^{dyn}_{\max}$ ) from the base at which the resultant dynamic force would have to act to produce yielding at the base, as a function of the fundamental period and the available rotational ductility of the wall. The ordinate on the right side of the figure gives the distance above the base as a fraction of the wall height. Note that for all cases, the resultant dynamic force lies below the approximate two-thirds point associated with the triangular loading specified in codes.

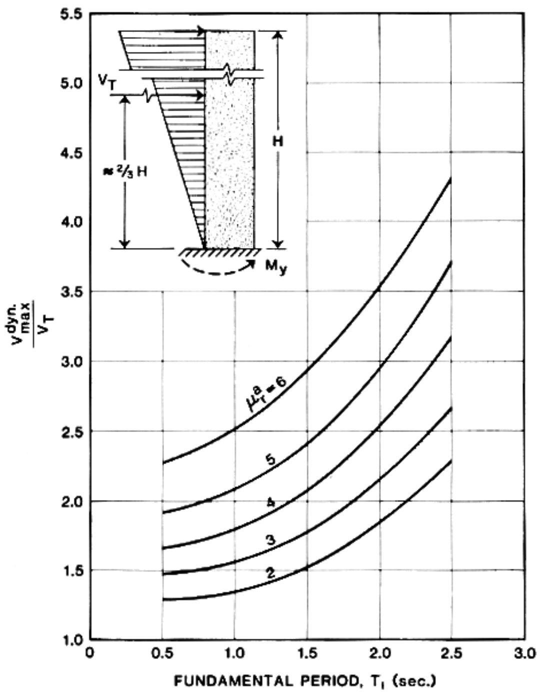



*Figure 10-23.* Ratio $V_{\max}^{dyn}$ /$V_T$ as a function of $T_1$ and $\mu_r^a$ . -20 story isolated structural walls. (From Ref. 10-4.)

These analytical results suggest not only that under strong earthquakes the maximum dynamic shear can be substantially greater than that associated with the lateral loads used to design the flexural strength of the base of the wall, but also, as a corollary, that the moment-to-shear ratio obtained under dynamic conditions is significantly less than that implied by the code-specified distribution of design lateral loads. These results are important because unlike beams in frames, where the design shear can be based on the maximum probable flexural strengths at the ends of the member as required by statics (see Figure 10-16), in cantilever walls it is not possible to determine a similar design shear as a function of the flexural strength at the base of the wall using statics alone, unless an assumption is made concerning the height of the applied resultant horizontal force. In the capacity design method adopted in New Zealand as applied to structural walls,[10-13,10-49] the design base shear at the base of a wall is obtained by multiplying the shear at the base corresponding to the code-specified forces by a flexural overstrength factor and a "dynamic shear magnification factor". The flexural overstrength factor in this case represents the ratio of flexural overstrength (accounting for upward deviations from the nominal strength of materials and other factors) to the moment due to the code-specified forces, with a typical value of about 1.39 or higher. Recommended values for the dynamic shear magnification factor range from 1.0 for a one-story high wall to a maximum of 1.8 for walls 6-stories or more in height.

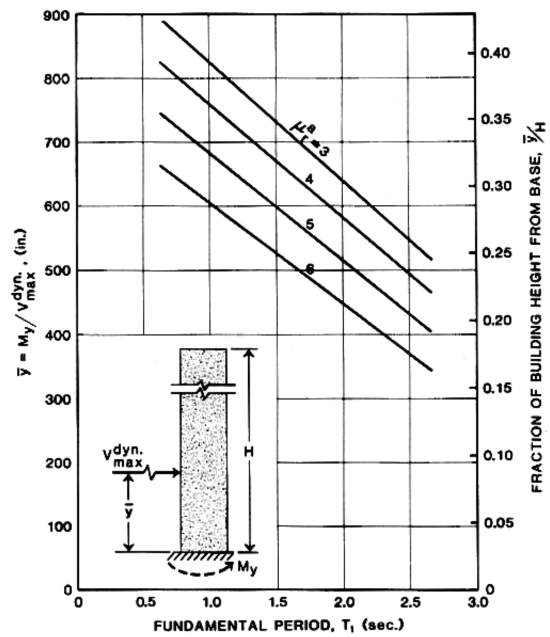



*Figure 10-24.* Ratio Y = $M_y/V_{\max}^{dyn}$ *as a function of $T_1$* and $\mu_r^a$ - 20 story isolated structural walls. (From Ref. 10-4.)

Tests on isolated structural walls[10-50,10-51] have shown that the hinging region, i.e., the region where most of the inelastic deformation occurs, extends a distance above the base roughly equal to the width of the wall. The ductility of the hinging region at the base of a wall, like the hinging region in beams and columns, is heavily dependent on the reinforcing details used to prevent early disruption of critically stressed areas within the region. As observed in beams and columns, tests of structural walls have confirmed the

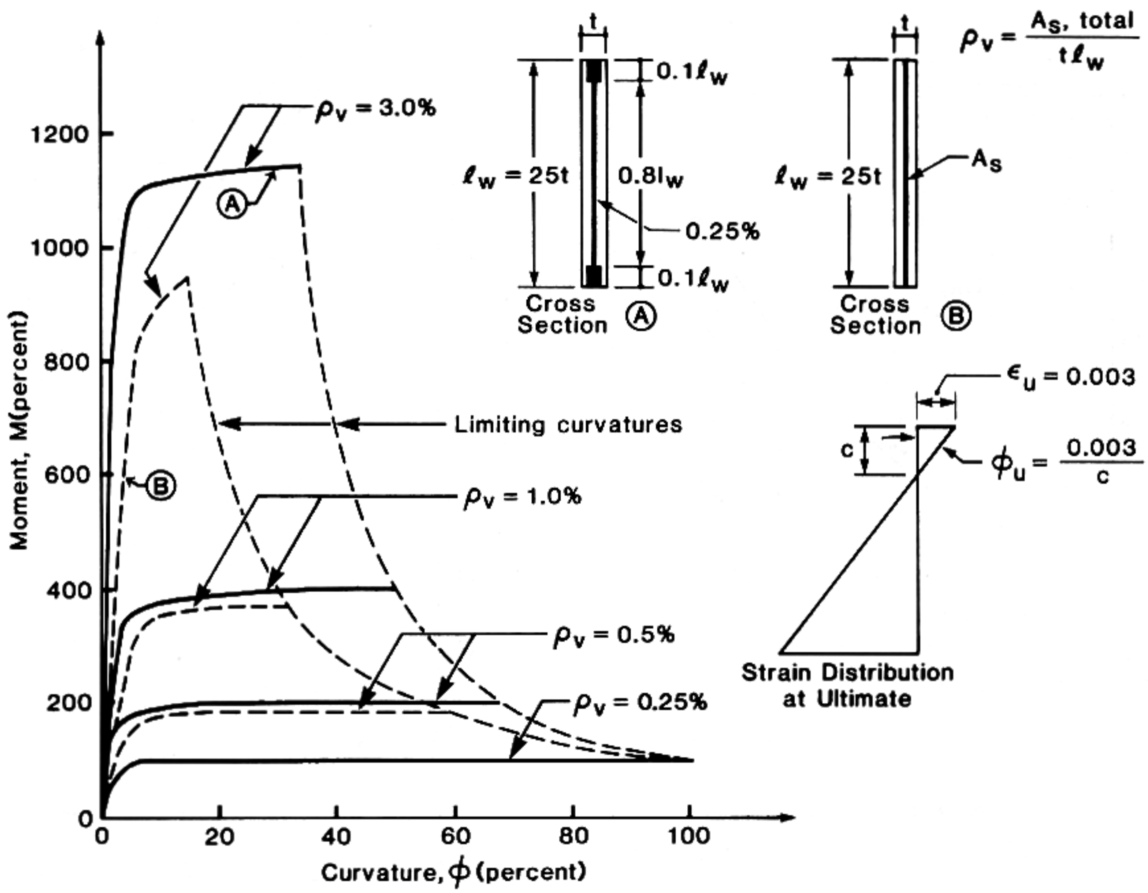



*Figure 10-25.* Moment-curvature curves for statically loaded rectangular walls as a function of reinforcement distribution.(10-52)

effectiveness of adequate confinement in maintaining the strength of the hinging region through cycles of reversed inelastic deformation. The adverse effects of high shears, acting simultaneously with the yield moment, on the deformation capacity of the hinging region of walls has also been noted in tests.

Early tests of slender structural walls under static monotonic loading(10-52) have indicated that the concentration of well-confined longitudinal reinforcement at the ends of the wall section can significantly increase the ductility of the wall. This is shown in Figure 10-25 from Reference 10-52. This improvement in behavior resulting from a concentration of well-confined longitudinal reinforcement at the ends of a wall section has also been observed in tests of isolated walls under cyclic reversed loading.(10-50, 10-51) Plain rectangular walls, not having relatively stiff confined boundary elements, are prone to lateral buckling of the compression edge under large horizontal displacements.(10-50, 10-52)

Figure 10-26 shows a sketch of the region at the base of a wall with boundary elements after a few cycles of lateral loading. Several modes of failure have been observed in the laboratory. Failure of the section can occur in flexure by rupture of the longitudinal reinforcement or by a combination of crushing and sliding in a weakened compression flange. Alternatively, failure, i.e., loss of lateral-load-resisting capacity, can occur by sliding along a near-horizontal plane near the base (in rectangular-section walls especially) or by crushing of the web concrete at the junction of the diagonal struts and the compression flange (in walls with thin webs and/or heavy boundary elements).

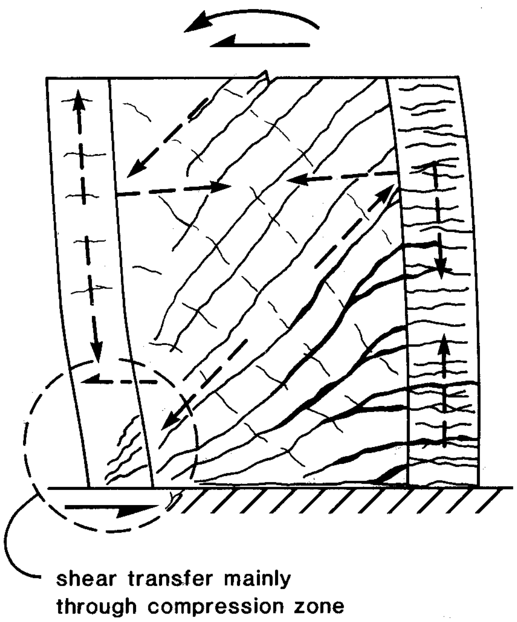



*Figure 10-26.* Moment-curvature curves for statically loaded rectangular walls as a function of reinforcement distribution.(10-54)

Since walls are generally designed to be under-reinforced, crushing in the usual sense associated with monotonic loading does not occur. However, when the flanges are inadequately confined, i.e., with the longitudinal and lateral reinforcement spaced far apart, concrete fragments within the cores of the flanges that had cracked in flexure under earlier cycles of loading can be lost in subsequent loading cycles. The longitudinal bars can buckle under compression and when subsequently stretched on reversal of the loading can rupture in low-cycle fatigue. It is also worth noting that because of the Bauschinger effect (i.e., the early yielding, reflected in the rounding of the stress—strain curve of steel, that occurs during load reversals in the inelastic range and the consequent reduction in the tangent modulus of the steel reinforcement at relatively low compressive stresses), the compression steel in members subjected to reversed cycles of inelastic loading tends to buckle earlier than in comparable monotonically loaded specimens.

As in beams and columns, degradation of strength and ductility of the hinging region of walls is strongly influenced by the magnitude of the shear that accompanies flexural yielding. High shears ( $> 6\sqrt{f_c'}$ ), when acting on a web area traversed by crisscrossing diagonal cracks, can precipitate failure of the wall by crushing of the diagonal web struts or a combined compression—sliding failure of the compression flange near the base. Shear in the hinging region is resisted by several mechanisms, namely, shear-friction along a near-horizontal plane across the width of the wall, dowel action of the tensile reinforcement and to a major extent (as in beams) by shear across the compression flange. After several cycles of load reversals and for moderate moment-to-shear ratios, the flexural cracks become wide enough to reduce the amount of shear carried by shear friction. As suggested by Figure 10-26, the truss action that develops in the hinging region involves a horizontal (shear) component of the diagonal strut that acts on the segment of the compression flange close to the base. If the compression flange is relatively slender and inadequately confined, the loss of core concrete under load reversals results in a loss of stiffness of this segment of the compression flange. The loss of stiffness and strength in the compression flange or its inability to support the combined horizontal (shear) component of the diagonal strut and the flexural compressive force can lead to failure of the wall.

Thus confinement of the flanges of walls, and especially those in the hinging region, is necessary not only to increase the compressive strain capacity of the core concrete but also to delay inelastic bar buckling and, together with the longitudinal reinforcement, prevent loss of the core concrete during load reversals (the so-called "basketing effect"). By maintaining the strength and stiffness of the flanges, confinement reinforcement improves the shear transfer capacity of the hinging region through the so-called "dowel action" of the compression flange, in addition to serving as shear reinforcement. As in beams, the diagonal tension cracking that occurs in walls and the associated truss action that develops induces tensile stresses in the horizontal web reinforcement. This suggests the need for proper anchorage of the horizontal reinforcement in the flanges.

Where high shears are involved, properly anchored crossing diagonal reinforcement in the hinging regions of walls, just as in beams, provides an efficient means of resisting shear and particularly the tendency toward sliding along cracked and weakened planes.

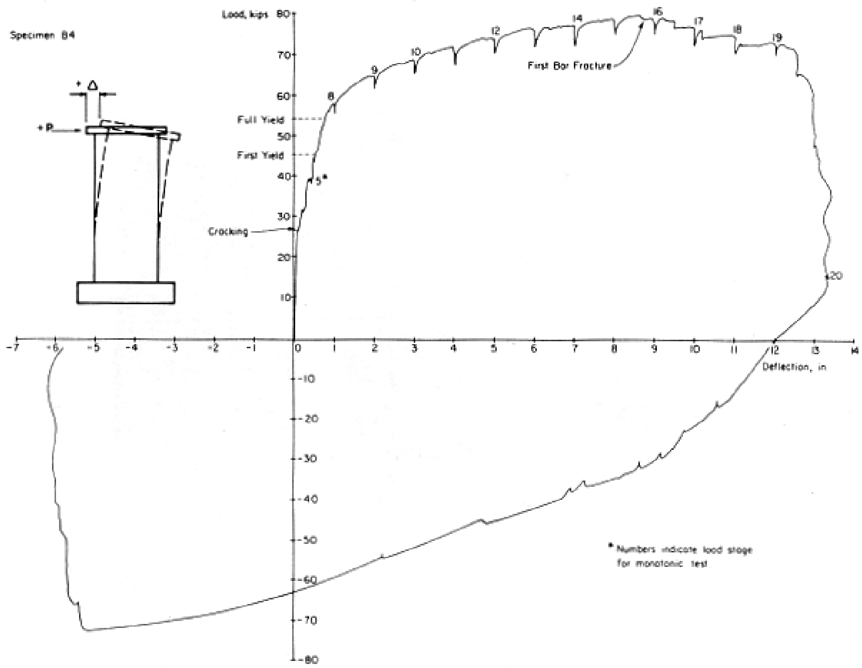
A series of tests of isolated structural wall specimens at the Portland Cement Association(10-50, 10-51) have provided some indication of the effect of several important variables on the behavior of walls subjected to slowly reversed cycles of inelastic deformations. Some results of this investigation have already been mentioned in the preceding. Three different wall cross-sections were considered in the study, namely, plain rectangular sections, barbell sections with heavy flanges (columns) at the ends, and flanged sections with the flanges having about the same thickness as the web. In the following, results for some of the parameters considered will be presented briefly.

1. *Monotonic vs. reversed cyclic loading.* In an initial set of two nominally identical specimens designed to explore the effect of load reversals, a 15% decrease in flexural strength was observed for a specimen loaded by cycles of progressively increasing amplitude of displacement when compared with a specimen that was loaded monotonically. Figures 10-27a and 10-28a show the corresponding load—deflection curves for the specimens. A comparison of these figures shows not only a reduction in strength but also that the maximum deflection of the wall subjected to reversed loading was only 8 in., compared to about 12 in. for the monotonically load specimen, indicating a reduction in deflection capacity of about 30%. Figure 10-28b, when compared with Figure 10-27b, shows the more severe cracking that results from load reversals.

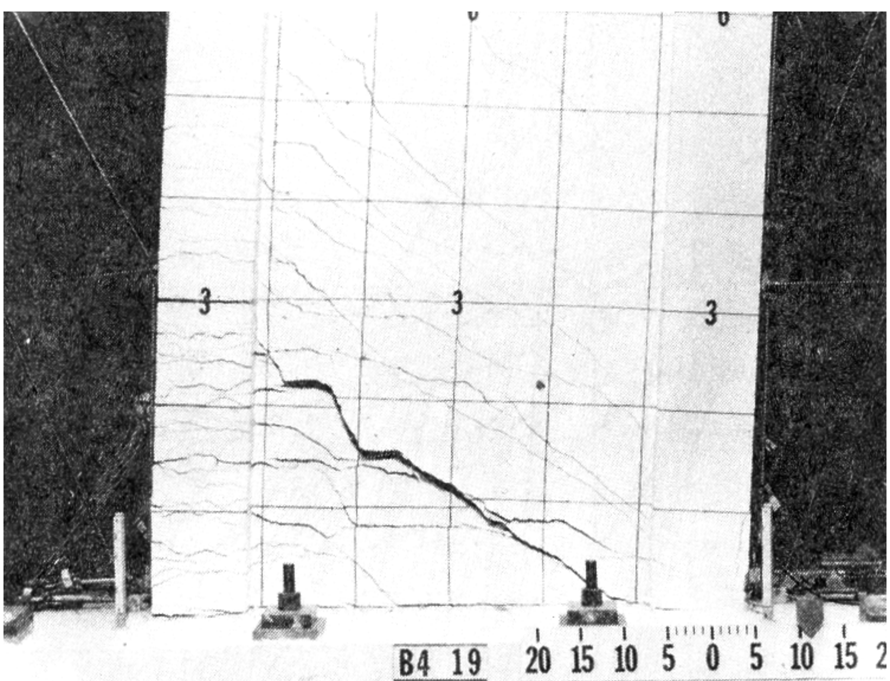
2. *Level of shear stress.* Figure 10-29 shows a plot of the variation of the maximum rotational ductility with the maximum nominal shear stress in isolated structural wall specimens reported in References 10-50 and 10-51. The decrease in rotational ductility with increasing values of the maximum shear stress will be noted. The maximum rotation used in determining ductility was taken as that for the last cycle in which at least 80% of the previous maximum observed load was sustained throughout the cycle. The yield rotation was defined as the rotation associated with the yielding of all of the tensile reinforcement in one of the boundary elements.

The presence of axial loads—of the order of 10% of the compressive strength of the walls—increased the ductility of specimens subjected to high shears. In Figure 10-29, the specimens subjected to axial loads are denoted by open symbols. The principal effect of the axial load was to reduce the shear distortions and hence increase the shear stiffness of the hinging region. It may be of interest to note that for walls loaded monotonically,(10-52) axial compressive stress was observed to increase moment capacity and reduce ultimate curvature, results consistent with analytical results from sectional analysis.

3. *Section shape.* As mentioned earlier, the use of wall sections having stiff and well-confined flanges or boundary elements, as against plain rectangular walls, not only allows development of substantial flexural capacity (in addition to being less susceptible to lateral buckling), but also improves the shear resistance and ductility of the wall. In walls with relatively stiff and well-confined boundary elements, some amount of web crushing can occur without necessarily limiting the flexural capacity of the wall. Corley et al.(10-53) point out that trying to avoid shear failure in walls, particularly walls with stiff and well-confined boundary elements, may be a questionable design objective.




(a)




(b)

*Figure 10-27.* (a) Load-deflection curve of monotonically loaded specimen. (b) view of specimen at +12 in. top deflection.(10-53)

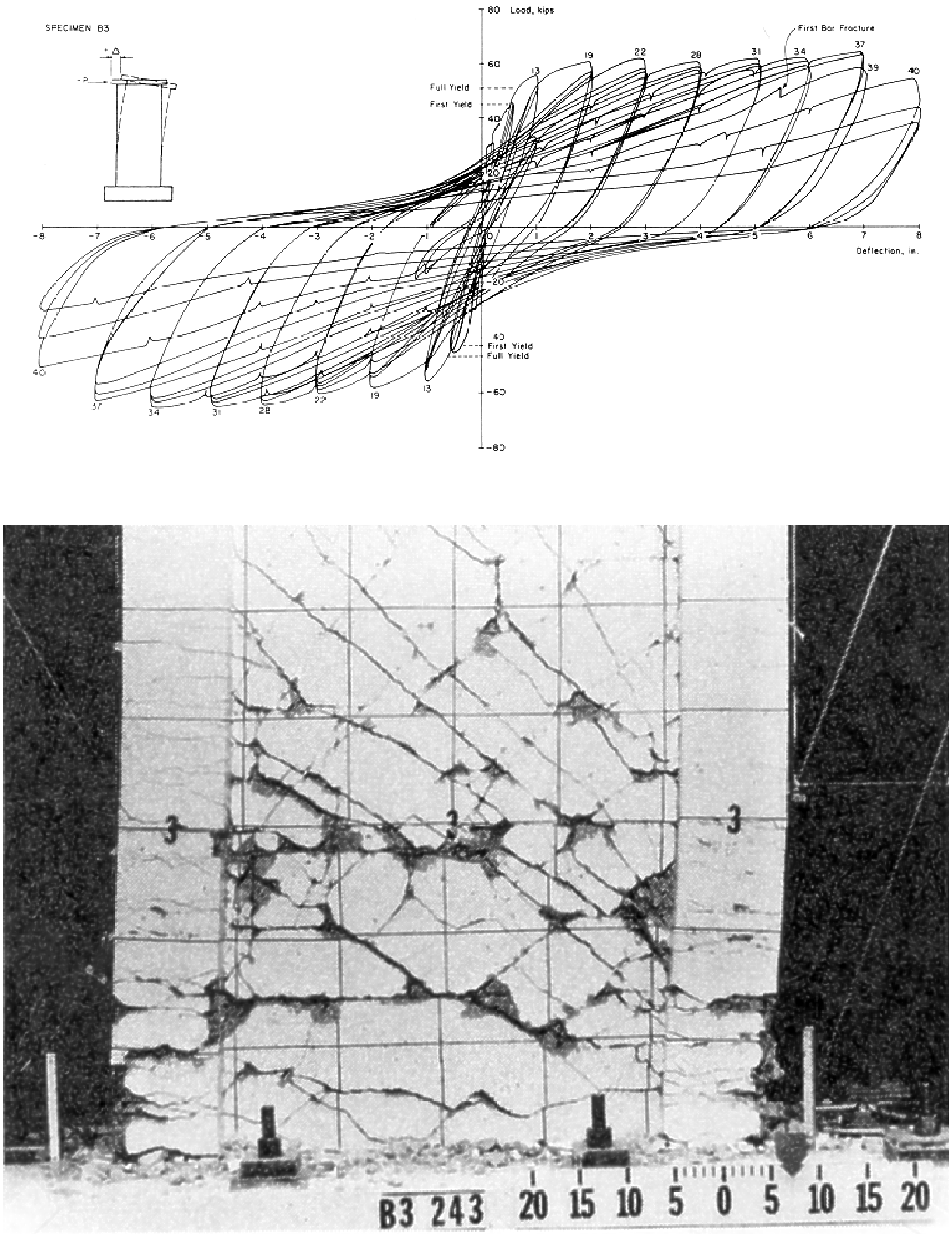



*Figure 10-28.* (a) Load-deflection curve of specimen subjected to load cycles of progressively increasing amplitude. (b) View of specimen at +8 in. top deflection. (10-53)

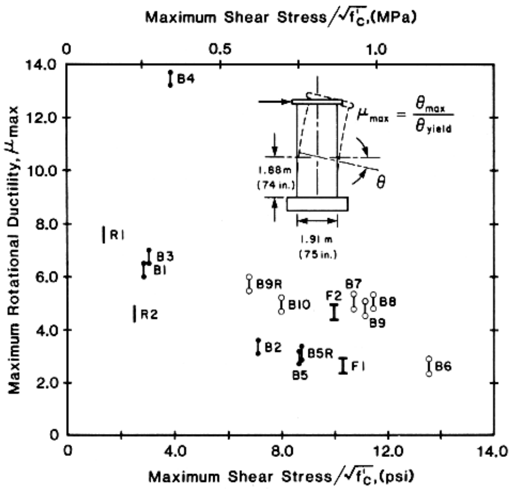



*Figure 10-29.* Variation of rotational ductility with maximum average shear stress in PCA isolated wall tests[10-51].

Thus, although ACI Chapter 21 limits the maximum average shear stress in walls to $10\sqrt{f'_c}$ (a value based on monotonic tests) with the intent of preventing web crushing, web crushing occurred in some specimens subjected to shear stresses only slightly greater than $7\sqrt{f'_c}$. However, those specimens where web-crushing failure occurred were able to develop deformations well beyond the yield deformation prior to loss of capacity.

4. *Sequence of large-amplitude load cycles.* Dynamic inelastic analyses of isolated walls[10-8] have indicated that in a majority of cases, the maximum or a near-maximum response to earthquakes occurs early, with perhaps only one elastic response cycle preceding it. This contrasts with the loading program commonly used in quasi-static tests, which consists of load cycles of progressively increasing amplitude. To examine the effect of imposing large-amplitude load cycles early in the test, two nominally identical isolated wall specimens were tested. One specimen was subjected to load cycles of progressively increasing amplitude, as were most of the specimens in this series. Figure 10-30a indicates that specimen B7 was able to sustain a rotational ductility of slightly greater than *5* through three repeated loading cycles. The second specimen (B9) was tested using a modified loading program similar to that shown in Figure 10-7b, in which the maximum load amplitude was imposed on the specimen after only one elastic load cycle. The maximum load amplitude corresponded to a rotational ductility of *5.* As indicated in Figure 10-30b, the specimen failed before completing the second load cycle. Although results from this pair of specimens cannot be considered conclusive, they suggest that tests using load cycles of progressively increasing amplitude may overestimate the ductility that can be developed under what may be considered more realistic earthquake response conditions. The results do tend to confirm the reasonable expectation that an extensively cracked and "softened" specimen subjected to several previous load cycles of lesser amplitude can better accommodate large reversed lateral deflections than a virtually uncracked specimen that is loaded to near-capacity early in the test. From this standpoint, the greater severity of the modified loading program, compared to the commonly used progressively increasing-amplitude loading program, appears obvious.

5. *Reinforcement detailing.* On the basis of the tests on isolated walls reported in References 10-50 and 10-51, Oesterle et al.[10-54] proposed the following detailing requirements for the hinging regions of walls:

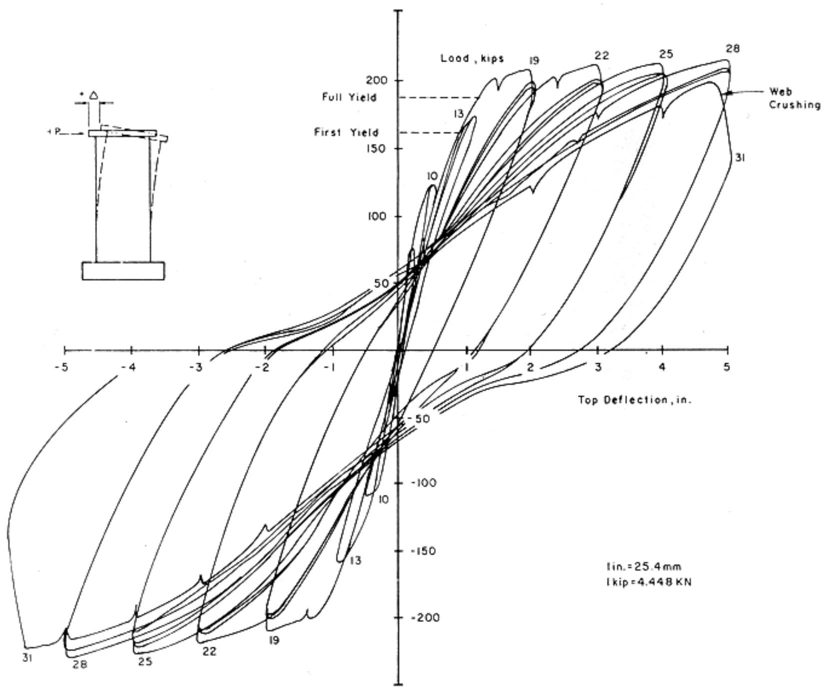
- The maximum spacing of transverse reinforcement in boundary elements should be $5d_b$, where $d_b$ is the diameter of the longitudinal reinforcement.
- Transverse reinforcement in the boundary element should be designed for a shear

$$V_{nb} = M_{nb}/1.5\, l_b\,,$$

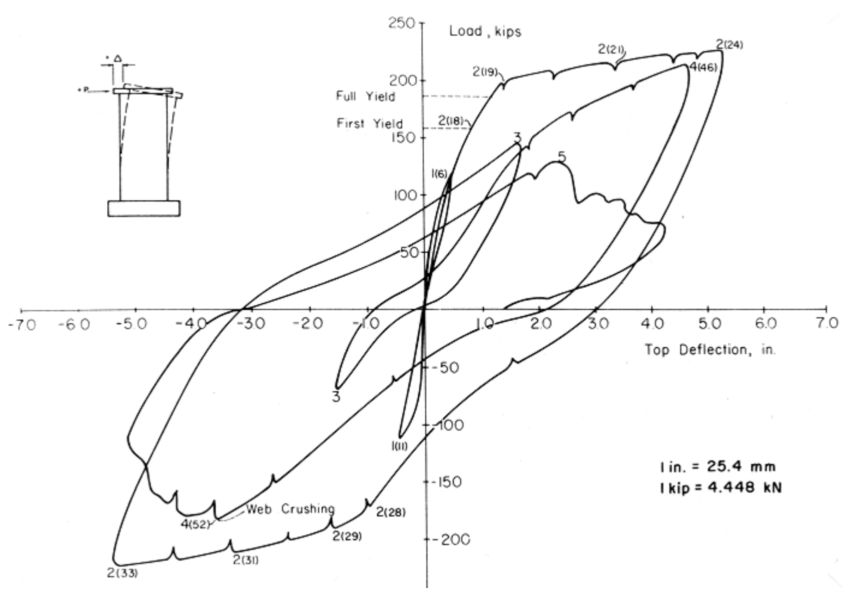
where

$M_{nb}$ = nominal moment strength of boundary element

$l_b$ =width of boundary element (in the plane of the wall)




(a)




(b)

*Figure 10-30.* Comparison of behavior of isolated walls subjected to different loading histories. (10-53)
(a) specimen subjected to progressively increasing load amplitudes (see Fig. 10-7a). (b) Specimen subjected to loading history characterized by large-amplitude cycles early in loading (see Fig. 10-7b).

• No lap splices should be used for cross-ties in segments of boundary elements within the hinging region.

• A recommendation on anchoring horizontal web reinforcement in the boundary elements, such as is shown in Figure 10-31a, has been adopted by ACI Chapter 21. For levels of shear in the range of $5\sqrt{f_c'}$ to $10\sqrt{f_c'}$, the study indicates that alternate 90° and 135° hooks, as shown in Figure 10-31b, can be used.

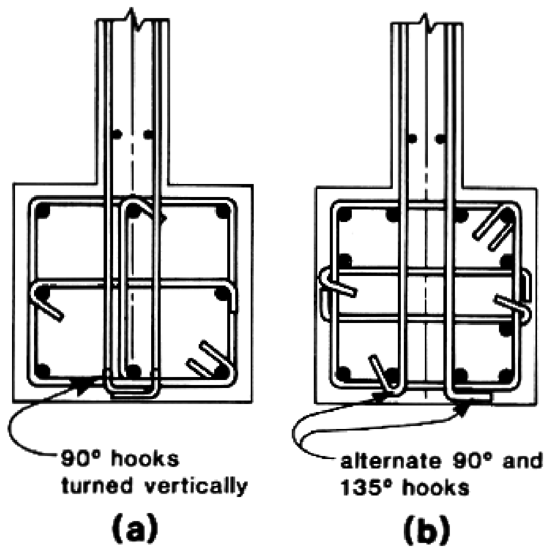



*Figure 10-31.* Alternative details for anchorage of horizontal web reinforcement in boundary elements.(10-54) (a) detail for walls subjected to low –to-moderate stress levels. (b) Detail for walls subjected to high shear stress levels.

The specimens tested in this series had special confinement reinforcement only over a length near the base equal to the width of the wall, i.e., the approximate length of the hinging region. Strain readings as well as observations of the general condition of the walls after failure showed that significant inelasticity and damage were generally confined to the hinging region. In view of this, it has been suggested that special confinement reinforcement for boundary elements need be provided only over the lengths of potential hinging regions. These are most likely to occur at the base and at points along the height of the wall where discontinuities, associated with abrupt and significant changes in geometry, strength, or stiffness, occur.

***Coupled Walls*** As mentioned earlier, a desirable characteristic in an earthquake-resistant structure is the ability to respond to strong ground motion by progressively mobilizing the energy-dissipative capacities of an ascending hierarchy of elements making up the structure.

In terms of their importance to the general stability and safety of a building, the components of a structure may be grouped into primary and secondary elements. *Primary elements* are those upon the integrity of which depend the stability and safety of the entire structure or a major part of it. In this category fall most of the vertical or near-vertical elements supporting gravity loads, such as columns and structural walls, as well as long-span horizontal elements. *Secondary elements* are those components whose failure would affect only limited areas or portions of a structure.

The strong column-weak beam design concept discussed earlier in relation to moment-resisting frames is an example of an attempt to control the sequence of yielding in a structure. The "capacity design" approach adopted in New Zealand which, by using even greater conservatism in the design of columns relative to beams, seeks to insure that no yielding occurs in the columns (except at their bases)—is yet another effort to achieve a controlled response in relation to inelastic action. By deliberately building in greater flexural strength in the primary elements (the columns), these design approaches force yielding and inelastic energy dissipation to take place in the secondary elements (the beams).

When properly proportioned, the coupled-wall system can be viewed as a further extension of the above design concept. By combining the considerable lateral stiffness of structural walls with properly proportioned coupling beams that can provide most of the energy-dissipative mechanism during response to strong ground motions, a better-performing structural system is obtained. The stiffness of the structural wall makes it a desirable primary element from the standpoint of damage control (by restricting interstory distortions), while the more conveniently repairable coupling beams provide the energy-dissipating secondary elements. Figure 10-32a shows a two-wall coupled-wall system and the forces acting at the base and on a typical coupling beam. The total overturning moment at the base of the coupled wall = $M_1 + M_2 + TL$. A typical distribution of the elastic shear force in the coupling beams along the height of the structure due to a statically applied lateral load is shown in Figure 10-32b. Note that the accumulated shears at each end of the coupling beams, summed over the height of the structure, are each equal to the axial force (T) at the base of the corresponding wall. The height to the most critically stressed coupling beam tends to move downward as the coupling-beam stiffness (i.e., the degree of coupling between the two walls) increases.

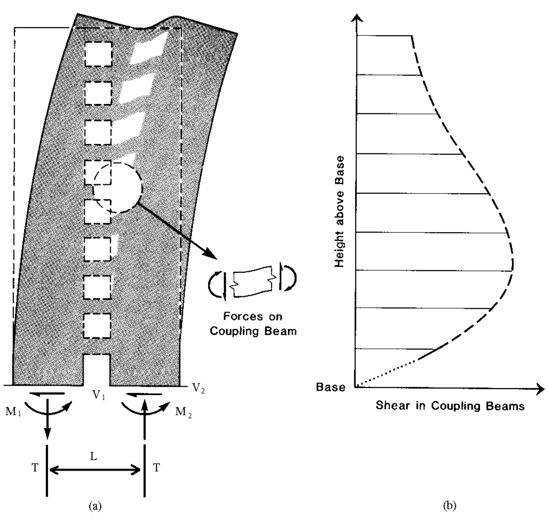



*Figure 10-32.* Laterally loaded coupled wall system. (a) Forces on walls at base. (b) Typical distribution of shears in coupling beams over height of structure.

In a properly designed earthquake-resistant coupled-wall system, the critically stressed coupling beams should yield first—before the bases of the walls. In addition, they must be capable of dissipating a significant amount of energy through inelastic action. These requirements call for fairly stiff and strong beams. Furthermore, the desire for greater lateral-load-resisting efficiency in the system would favor stiff and strong coupling beams. However, the beams should not be so stiff or strong flexurally that they induce appreciable tension in the walls, since a net tension would reduce not only the yield moment but also the shear resistance of the wall (recall that a moderate amount of compression improves the shear resistance and ductility of isolated walls). This in turn can lead to early flexural yielding and shear-related inelastic action at the base of the tension wall. Dynamic inelastic analyses of coupled-wall systems[(10-56)] have shown, and tests on coupled-wall systems under cyclic reversed loading[(10-57)] have indicated, that when the coupling beams have appreciable stiffness and strength, so that significant net tension is induced in the "tension wall", a major part of the total base shear is resisted by the "compression wall" (i.e., the wall subjected to axial compression for the direction of loading considered), a situation not unlike that which occurs in a beam.

The design of a coupled-wall system would then involve adjusting the wall-to-coupling beam strength and stiffness ratios so as to strike a balance between these conflicting requirements. A basis for choosing an appropriate beam-to-wall strength ratio, developed from dynamic inelastic response data on coupled-wall systems, is indicated in Reference 10-58. The Canadian Code for Concrete, CSA Standard A23.3-94[(10-33)], recommends that in order to classify as a fully effective coupled wall system, the ratio $\frac{TL}{M_1 + M_2 + TL}$ must be greater than 2/3. Those with lower ratios are classified as partially coupled wall system in which the coupled wall system are to be designed for higher seismic design forces (14% greater) due to their lower amount of energy dissipation capacity due to reduced coupling action. Once the appropriate relative strengths and stiffness have been established, details to insure adequate ductility in potential hinging regions can be addressed.

Because of the relatively large shears that develop in deep coupling beams and the likelihood of sliding shear failures under reversed loading, the use of diagonal reinforcement in such elements has been suggested (see Figure 10-33). Tests by Paulay and Binney[10-59] on diagonally reinforced coupling beams having span-to-depth ratios in the range of 1 to 1½ have shown that this arrangement of reinforcement is very effective in resisting reversed cycles of high shear. The specimens exhibited very stable force—deflection hysteresis loops with significantly higher cumulative ductility than comparable conventionally reinforced beams. Tests by Barney et al.[10-60] on diagonally reinforced beams with span-to-depth ratios in the range of 2.5 to 5.0 also indicated that diagonal reinforcement can be effective even for these larger span-to-depth ratios.

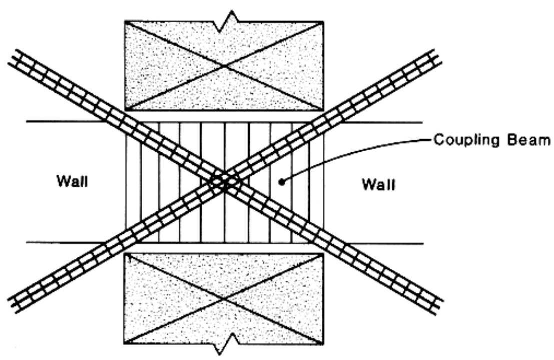



*Figure 10-33.* Diagonally reinforced coupling beam. (Adapted from Ref. 10-59.)

In the diagonally reinforced couplings beams reported in Reference 10-60, no significant flexural reinforcement was used. The diagonal bars are designed to resist both shear and bending and assumed to function at their yield stress in both tension and compression. To prevent early buckling of the diagonal bars, Paulay and Binney recommend the use of closely spaced ties or spiral binding to confine the concrete within each bundle of diagonal bars. A minimum amount of “basketing reinforcement,” consisting of two layers of small-diameter horizontal and vertical bars, is recommended. The grid should provide a reinforcement ratio of at least 0.0025 in each direction, with a maximum spacing of 12 in. between bars.

## 10.4 CODE PROVISIONS FOR EARTHQUAKE-RESISTANT DESIGN

### 10.4.1 Performance Criteria

In recent years, the performance criteria reflected in some building code provisions such as IBC-2000[10-61] have become more explicit than before. Although these provisions explicitly require design for only a single level of ground motion, it is expected that buildings designed and constructed in accordance with these requirements will generally be able to meet a number of performance criteria, when subjected to earthquake ground motions of differing severity. The major framework of the performance criteria is discussed in the report by the Structural Association of California Vision 2000 (SEAOC, 1995).[10-62] In this report, four performance levels are defined and each performance level is expressed as the desired maximum level of damage to a building when subjected to a specific seismic ground motion. Categories of performance are defined as follows:

1. fully operational
2. operational
3. life-safe
4. near collapse

For each of the performance levels, there is a range of damage that corresponds to the building’s functional status following a specified earthquake design level. These earthquake design levels represent a range of earthquake excitation that have defined probabilities of occurrence over the life of the building. SEAOC Vision 2000 performance level definition includes descriptions of structural and non-structural damage, egress systems and overall building state. Also included in the performance level descriptions is the level of both transient and permanent drift in the structure. Drift is defined as the ratio of interstory deflection to the story height.

The fully operational level represents the least level of damage to the building. Except for very low levels of ground motion, it is generally not practical to design buildings to meet this performance level.

Operational performance level is one in which overall building damage is light. Negligible damage to vertical load carrying elements as well as light damage to the lateral load carrying element is expected. The lateral load carrying system retains almost all of its original stiffness and strength, with minor cracking in the elements of the structure is expected. Transient drift are less than 0.5% and there is inappreciable permanent drift. Building occupancy continues unhampered.

Life-safe performance level guidelines include descriptions of damage to contents, as well as structural and non-structural elements. Overall, the building damage is described as moderate. Lateral stiffness has been reduced as well as the capacity for additional loads, while some margin against building collapse remains. Some cracking and crushing of concrete due to flexure and shear is expected. Vertical load carrying elements have substantial capacity to resist gravity loads. Falling debris is limited to minor events. Levels of transient drift are to be below 1.5% and permanent drift is less than 0.5%.

Near collapse performance includes severe overall damage to the building, moderate to heavy damage of the vertical load carrying elements and negligible stiffness and strength in the lateral load carrying elements. Collapse is prevented although egress may be inhibited. Permissible levels of transient and permanent drift are less than 2.5%. Repair of a building following this level of performance may not be practical, resulting in a permanent loss of building occupancy.

In the IBC-2000 provisions, the expected performance of buildings under the various earthquakes that can affect them are controlled by assignment of each building to one of the three seismic use groups. These seismic use groups are categorized based on the type of occupancy and importance of the building. For example, buildings such as hospitals, power plants and fire stations are considered as essential facilities also known as post-disaster buildings and are assigned as seismic use group III. These provisions specify progressively more conservative strength, drift control, system selection, and detailing requirements for buildings contained in the three groups, in order to attain minimum levels of earthquake performance suitable to the individual occupancies.

### 10.4.2 Code-Specified Design Lateral Forces

The availability of dynamic analysis programs (see References 10-63 to 10-68) has made possible the analytical estimation of earthquake-induced forces and deformations in reasonably realistic models of most structures. However, except perhaps for the relatively simple analysis by modal superposition using response spectra, such dynamic analyses, which can range from a linearly elastic time-history analysis for a single earthquake record to nonlinear analyses using a representative ensemble of accelerograms, are costly and may be economically justifiable as a design tool only for a few large and important structures. At present, when dynamic time-history analyses of a particular building are undertaken for the purpose of design, linear elastic response is generally assumed. Nonlinear (inelastic) time-history analyses are carried out mainly in research work. However, non-linear pushover static analysis can be used as a design tool to evaluate the performance of the structure in the post-yield range of response. Pushover analysis is used to develop the capacity curve, illustrated generally as a base shear versus top story displacement curve. The pushover test shows the sequence of element cracking and yielding as a function of the top story displacement and the base shear. Also, it exposes the elements within the structure subjected to the greatest amount of inelastic deformation. The force displacement relationship shows the strength of the structure and the maximum base shear that can be developed. Pushover analysis, which is relatively a new technology, should be carried out with caution. For example, when the response of a structure is dominated by modes other than the first mode, the results may not represent the actual behavior.

For the design of most buildings, reliance will usually have to be placed on the simplified prescriptions found in most codes[10-1] Although necessarily approximate in character-in view of the need for simplicity and ease of application-the provisions of such codes and the philosophy behind them gain in reliability as design guides with continued application and modification to reflect the latest research findings and lessons derived from observations of structural behavior during earthquakes. Code provisions must, however, be viewed in the proper perspective, that is, as minimum requirements covering a broad class of structures of more or less conventional configuration. Unusual structures must still be designed with special care and may call for procedures beyond those normally required by codes.

The basic form of modern code provisions on earthquake-resistant design has evolved from rather simplified concepts of the dynamic behavior of structures and has been greatly influenced by observations of the performance of structures subjected to actual earthquakes.[10-69] It has been noted, for instance, that many structures built in the 1930s and designed on the basis of more or less arbitrarily chosen lateral forces have successfully withstood severe earthquakes. The satisfactory performance of such structures has been attributed to one or more of the following[10-70, 10-71]: (i) yielding in critical sections of members (yielding not only may have increased the period of vibration of such structures to values beyond the damaging range of the ground motions, but may have allowed them to dissipate a sizable portion of the input energy from an earthquake); (ii) the greater actual strength of such structures resulting from so-called nonstructural elements which are generally ignored in analysis, and the significant energy-dissipation capacity that cracking in such elements represented; and (iii) the reduced response of the structure due to yielding of the foundation.

The distribution of the code-specified design lateral forces along the height of a structure is generally similar to that indicated by the envelope of maximum horizontal forces obtained by elastic dynamic analysis. These forces are considered *service loads,* i.e., to be resisted within a structure's elastic range of stresses. However, the magnitudes of these code forces are substantially smaller than those which would be developed in a structure subjected to an earthquake of moderate-to-strong intensity, such as that recorded at El Centro in 1940, if the structure were to respond elastically to such ground excitation. Thus, buildings designed under the present codes would be expected to undergo fairly large deformations (four to six times the lateral displacements resulting from the code-specified forces) when subjected to an earthquake with the intensity of the 1940 El Centro.[10-2] These large deformations will be accompanied by yielding in many members of the structure, and, in fact, such is the intent of the codes. The acceptance of the fact that it is economically unwarranted to design buildings to resist major earthquakes elastically, and the recognition of the capacity of structures possessing adequate strength and ductility to withstand major earthquakes by responding inelastically to them, lies behind the relatively low forces specified by the codes. These reduced forces are coupled with detailing requirements designed to insure adequate inelastic deformation capacity, i.e., ductility. The capacity of an indeterminate structure to deform in a ductile manner, that is to deform well beyond the yield limit without significant loss of strength, allows such a structure to dissipate a major portion of the energy from an earthquake without serious damage.

### 10.4.3 Principal Earthquake-Design Provisions of ASCE 7-95, IBC-2000, UBC-97, and ACI Chapter 21 Relating to Reinforced Concrete

The principal steps involved in the design of earthquake-resistant cast-in-place reinforced concrete buildings, with particular reference to the application of the provisions of nationally accepted model codes or standards, will be discussed below. The minimum design loads specified in ASCE 7-95, *Minimum design Loads for Buildings and Other Structures*[10-72] and the design and detailing provisions contained in Chapter 21 of ACI 318-95, *Building Code Requirements for Reinforced Concrete,*[10-10] will be used as bases for the discussion. Emphasis will be placed on those provisions relating to the proportioning and detailing of reinforced concrete elements, the subject of the determination of earthquake design forces having been treated in Chapters 4 and 5. Where appropriate, reference will be made to differences between the provisions of these model codes and those of related codes. Among the more important of these is the IBC-2000[10-61] which is primarily a descendant of ATC 3-06[10-73] and the latest edition of the *Recommended Lateral Force Requirements* of the Structural Engineers Association of California (SEAOC-96).[10-74]

The ASCE 7-95 provisions relating to earthquake design loads are basically similar to those found in the 1997 Edition of the Uniform Building Code (UBC-97)[10-1]. The current UBC-97 earthquake design load requirements are based on the 1996 SEAOC Recommendations (SEAOC-96). Except for minor modifications, the design and detailing requirements for reinforced concrete members found in UBC-97 (SEAOC-96) and IBC-2000 are essentially those of ACI Chapter 21.

Although the various code-formulating bodies in the United States tend to differ in what they consider the most appropriate form in which to cast specific provisions and in their judgment of the adequacy of certain design requirements, there has been a tendency for the different codes and model codes to gradually take certain common general features. And while many questions await answers, it can generally be said that the main features of the earthquake-resistant design provisions in most current regional and national codes have good basis in theoretical and experimental studies as well as field observations. As such, they should provide reasonable assurance of attainment of the stated objectives of earthquake-resistant design. The continual refinement and updating of provisions in the major codes to reflect the latest findings of research and field observations[10-75] should inspire increasing confidence in the soundness of their recommendations.

The following discussion will focus on the provisions of ASCE 7-95 and ACI Chapter 21, with occasional references to parallel provisions of IBC-2000 and UBC-97 (SEAOC-96).

The design earthquake forces specified in ASCE 7-95 is intended as equivalent static loads. As its title indicates, ASCE 7-95 is primarily a load standard, defining minimum loads for structures but otherwise leaving out material and member detailing requirements. ACI Chapter 21 on the other hand, does not specify the manner in which earthquake loads are to be determined, but sets down the requirements by which to proportion and detail monolithic cast-in-place reinforced concrete members in structures that are expected to undergo inelastic deformations during earthquakes.

***Principal Design Steps*** Design of a reinforced concrete building in accordance with the equivalent static force procedure found in current U.S. seismic codes involves the following principal steps:

1. Determination of design "earthquake" forces:

- Calculation of base shear corresponding to the computed or estimated fundamental period of vibration of the structure. (A preliminary design of the structure is assumed here.)
- Distribution of the base shear over the height of the building.

2. Analysis of the structure under the (static) lateral forces calculated in step (1), as well as under gravity and wind loads, to obtain member design forces and story drift ratios. The lateral load analysis, of course, can be carried out most conveniently by using a computer program for analysis.

For certain class of structures having plan or vertical irregularities, or structure over 240 feet in height, most building codes require dynamic analysis to be performed. In this case, ASCE 7-95 and IBC-2000 require that the design parameters including story shears, moments, drifts and deflections determined from dynamic analysis to be adjusted. Where the design value for base shear obtained from dynamic analysis ($V_t$) is less than the calculated base shear ($V$) determined using the step 1 above, these design parameters is to be increased by a factor of $V/V_t$.

3. Designing members and joints for the most unfavorable combination of gravity and lateral loads. The emphasis here is on the design and detailing of members and their connections to insure their ductile behavior.

The above steps are to be carried out in each principal (plan) direction of the building. Most building codes allow the design of a structure in each principal direction independently of the other direction on the assumption that the design lateral forces act non-concurrently in each principal direction. However, for certain building categories which may be sensitive to torsional oscillations or characterized by significant irregularities and for columns forming part of two or more intersecting lateral-force-resisting systems, orthogonal effects need to be considered. For these cases, the codes consider the orthogonal effects requirement satisfied if the design is based on the more severe combination of 100 percent of the prescribed seismic forces in one direction plus 30 percent of the forces in the perpendicular direction.

Changes in section dimensions of some members may be indicated in the design phase under step (3) above. However, unless the required changes in dimensions are such as to materially affect the overall distribution of forces in the structure, a reanalysis of the structure using the new member dimensions need not be undertaken. Uncertainties in the actual magnitude and distribution of the seismic forces as well as the effects of yielding in redistributing forces in the structure would make such refinement unwarranted. It is, however, most important to design and detail the reinforcement in members and their connections to insure their ductile behavior and thus allow the structure to sustain without collapse the severe distortions that may occur during a major earthquake. The code provisions intended to insure adequate ductility in structural elements represent the major difference between the design requirements for conventional, non-earthquake-resistant structures and those located in regions of high earthquake risk.

***Load Factors, Strength Reduction Factors, and Loading Combinations Used as Bases for Design*** Codes generally require that the strength or load-resisting capacity of a structure and its component elements be at least equal to or greater than the forces due to any of a number of loading combinations that may reasonably be expected to act on it during its life. In the United States, concrete structures are commonly designed using the ultimate-strength[b] method. In this approach, structures are proportioned so that their (ultimate) capacity is equal to or greater than the required (ultimate) strength. The required strength is based on the most critical combination of factored loads, that is, specified service loads multiplied by appropriate *load factors.* The capacity of an element, on the other hand, is obtained by applying a *strength-reduction factor* $\phi$ to the nominal resistance of the element as determined by code-prescribed expressions or procedures or from basic mechanics.

Load factors are intended to take account of the variability in the magnitude of the specified

[b] Since ACI 318-71, the term "ultimate" has been dropped, so that what used to be referred to as "ultimate-strength design" is now simply called "strength design."

loads, lower load factors being used for types of loads that are less likely to vary significantly from the specified values. To allow for the lesser likelihood of certain types of loads occurring simultaneously, reduced load factors are specified for some loads when considered in combination with other loads.

ACI 318-95 requires that structures, their components, and their foundations be designed to have strengths not less than the most severe of the following combinations of loads:

$$U = \begin{cases} 1.4D + 1.7L \\ 0.75[1.4D + 1.7L \pm (1.7W \text{ or } 1.87E)] \\ 0.9D \pm (1.3W \text{ or } 1.43E) \\ 1.4D + 1.7L + (1.7H \text{ or } 1.4F) \\ 0.9D + (1.7H \text{ or } 1.4F) \\ 0.75(1.4D + 1.7L + 1.4T) \end{cases} \quad (10\text{-}1)$$

where

$U$ = required strength to resist the factored loads
D = dead load
L = live load
W = wind load
E = earthquake load
F = load due to fluids with and maximum heights well-defined pressures
H = load due to soil pressure
T = load due to effects of temperature, shrinkage, expansion of shrinkage compensating concrete, creep, differential settlement, or combinations thereof.

ASCE 7-95 specifies slightly different load factors for some load combinations, as follows:

$$U = \begin{cases} 1.4\,D \\ 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \\ 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8\,W) \\ 1.2\,D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) \\ 1.2\,D + 1.0\,E + 0.5\,L + 0.2\,S \\ 0.9\,D + (1.3\,W \text{ or } 1.0\,E) \end{cases} \quad (10\text{-}2)$$

where
$L_r$ = roof live load
S = snow load
R = rain load

For garages, places of public assembly, and all areas where the live load is greater than 100 lb/ft$^2$, the load factor on *L* in the third, fourth, and fifth combinations in Equation 10-2 is to be taken equal to 1.0.

For the design of earthquake-resistant structures, UBC-97 uses basically the same load combinations specified by ASCE 7-95 as shown in Equation 10-2.

IBC-2000 requires that the load combinations to be the same as those specified by ASCE 7-95 as shown in Equation 10-2. However, the effect of seismic load, E, is defined as follows:

$$E = \rho\, Q_E + 0.2\, S_{DS}\, D$$
$$E = \rho\, Q_E - 0.2\, S_{DS}\, D \quad (10\text{-}3)$$

where
E = the effect of horizontal and vertical earthquake-induced forces,
$S_{DS}$ = the design spectral response acceleration at short periods
D = the effect of dead load
ρ = the reliability factor
$Q_E$ = the effect of horizontal seismic forces

To consider the extent of structural redundancy inherent in the lateral-force-resisting system, the reliability factor, ρ, is introduced for buildings located in areas of moderate to high seismicity. This is basically a penalty factor for buildings in which the lateral resistance is limited to only few members in the structure. The maximum value of ρ is limited to 1.5.

The factor 0.2 $S_{DS}$ in Equation (10-3) is placed on the dead load to account for the effects of vertical acceleration.

For situations where failure of an isolated, individual, brittle element can result in the loss of a complete lateral-force-resisting system or in instability and collapse, IBC-2000 has a specific requirement to determine the seismic design forces. These elements are referred to as collector elements. Columns supporting discontinuous lateral-load-resisting elements such as walls also fall under this category. The seismic loads are as follows:

$$E = \Omega_o Q_E + 0.2 S_{DS} D$$
$$E = \Omega_o Q_E - 0.2 S_{DS} D \qquad (10\text{-}4)$$

where $\Omega_o$ is the system overstrength factor which is defined as the ratio of the ultimate lateral force the structure is capable of resisting to the design strength. The value of $\Omega_o$ varies between 2 to 3 depending on the type of lateral force resisting system.

As mentioned earlier, the capacity of a structural element is calculated by applying a strength reduction factor $\phi$ to the nominal strength of the element. The factor $\phi$ is intended to take account of variations in material strength and uncertainties in the estimation of the nominal member strength, the nature of the expected failure mode, and the importance of a member to the overall safety of the structure. For conventional reinforced concrete structures, ACI 318-95 specifies the following values of the strength reduction factor $\phi$:

0.90 for flexure, with or without axial tension

0.90 for axial tension

0.75 for spirally reinforced members subjected to axial compression, with or without flexure

0.70 for other reinforced members (tied columns) subjected to axial compression, with or without flexure (an increase in the $\phi$ value for members subjected to combined axial load and flexure is allowed as the loading condition approaches the case of pure flexure)

0.85 for shear and torsion
0.70 for bearing on concrete

ACI Chapter 21 specifies the following exception to the above values of the strength-reduction factor as given in the main body of the ACI Code:

For structural members other than joints, a value $\phi = 0.60$ is to be used for shear when the nominal shear strength of a member is less than the shear corresponding to the development of the nominal flexural strength of the member. For shear in joints, $\phi = 0.85$.

The above exception applies mainly to low-rise walls or portions of walls between openings.

***Code Provisions Designed to Insure Ductility in Reinforced Concrete Members***

The principal provisions of ACI Chapter 21 will be discussed below. As indicated earlier, the requirements for proportioning and detailing reinforced concrete members found in UBC-97 (SEAOC-96) and IBC-2000 are essentially those of ACI Chapter 21. Modifications to the ACI Chapter 21 provisions found in UBC-97 and IBC-2000 will be referred to where appropriate.

Special provisions governing the design of earthquake-resistant structures first appeared in the 1971 edition of the ACI Code. The provisions Chapter 21 supplement or supersede those in the earlier chapters of the code and deal with the design of ductile moment-resisting space frames and shear walls of cast-in-place reinforced concrete.

ACI 318-95 does not specify the magnitude of the earthquake forces to be used in design. The Commentary to Chapter 21 states that the provisions are intended to result in structures capable of sustaining a series of oscillations in the inelastic range without critical loss in strength. It is generally accepted that the intensity of shaking envisioned by the provisions of the first seven sections of ACI Chapter 21 correspond to those of UBC seismic zones 3 and 4. In the 1983 edition of the ACI Code, a section (Section A.9; now section 21.8) was added to cover the design of frames located in areas of moderate seismic risk, roughly corresponding to UBC seismic zone 2. For structures located in areas of low seismic risk (corresponding to UBC seismic zones 0 and 1) and designed for the specified earthquake forces, very little inelastic deformation may be expected. In these cases, the ductility provided by designing to the provisions contained in the first 20 Chapters of the code will generally be sufficient.

A major objective of the design provisions in ACI Chapter 21, as well as in the earlier chapters of the code, is to have the strength of a structure governed by a ductile type of flexural failure mechanism. Stated another way, the provisions are aimed at preventing the brittle or abrupt types of failure associated with inadequately reinforced and over-reinforced members failing in flexure, as well as with shear (i.e., diagonal tension) and anchorage or bond failures. The main difference between Chapter 21 and the earlier chapters of the ACI Code lies in the greater range of deformation, with yielding actually expected at critical locations, and hence the greater ductility required in designs for resistance to major earthquakes. The need for greater ductility follows from the design philosophy that uses reduced forces in proportioning members and provides for the inelastic deformations that are expected under severe earthquakes by special ductility requirements.

A provision unique to earthquake-resistant design of frames is the so-called strong column-weak beam requirement. As discussed in Section 10.3.4 under "Beam—Column Joints," this requirement calls for the sum of the flexural strengths of columns meeting at a frame joint to be at least 1.2 times that of the beams framing into the joint. This is intended to force yielding in such frames to occur in the beams rather than in the columns and thus preclude possible instability due to plastic hinges forming in the columns. As pointed out earlier, this requirement may not guarantee non-development of plastic hinges in the columns. The strong column-weak beam requirement often results in column sizes that are larger than would otherwise be required, particularly in the upper floors of multistory buildings with appreciable beam spans.

1. *Limitations on material strengths.* ACI Chapter 21 requires a minimum specified concrete strength $f_c'$ of 3000 lb/in.$^2$ and a maximum specified yield strength of reinforcement, $f_y$ of 60,000 lb/in.$^2$. These limits are imposed with a view to restricting the unfavorable effects that material properties beyond these limits can have on the sectional ductility of members. ACI Chapter 21 requires that reinforcement for resisting flexure and axial forces in frame members and wall boundary elements be ASTM 706 grade 60 low-alloy steel intended for applications where welding or bending, or both, are important. However, ASTM 615 billet steel bars of grade 40 or 60 may be used provided the following two conditions are satisfied:

$$(\text{actual } f_y) \leq (\text{specified } f_y) \pm 18{,}000 \text{ lb/in.}^2$$

$$\frac{\text{actual ultimate tensile stress}}{\text{actual } f_y} \geq 1.25$$

The first requirement helps to limit the increase in magnitude of the actual shears that can develop in a flexural member beyond that computed on the basis of the specified yield stress when plastic hinges form at the ends of a beam. The second requirement is intended to insure reinforcement with a sufficiently long yield plateau.

In the "strong column-weak beam" frame intended by the code, the relationship between the moment capacities of columns and beams may be upset if the beams turn out to have much greater moment capacity than intended by the designer. Thus, the substitution of 60-ksi steel of the same area for specified 40-ksi steel in beams can be detrimental. The shear strength of beams and columns, which is generally based on the condition of plastic hinges forming (i.e., $M_y$ acting) at the member ends, may become inadequate if the actual moment capacities at the member ends are greater than intended as a result of the steel having a substantially greater yield strength than specified.

2. *Flexural members (beams).* These include members having a clear span greater than four times the effective depth that are subject to a factored axial compressive force not exceeding $A_g f_c'/10$, where $A_g$ is the gross cross-sectional area. Significant provisions relating to flexural members of structures in regions of high seismic risk are discussed below.

(a) Limitations on section dimensions
width/depth ≥ 0.3

$$width \begin{cases} \geq 10 \text{ in.} \\ \leq \text{width of supporting } column + \\ 1.5 \times (\text{depth of beam}) \end{cases}$$

(b) Limitations on flexural reinforcement ratio (see also Figure 10-34):

$$\rho_{\min} = \begin{cases} 200/f_y \\ \text{two continuous bars at both top} \\ \text{and bottom of member} \\ \dfrac{3\sqrt{f_c'}}{f_y} \end{cases}$$

$\rho_{\max} = 0.025$

The minimum steel required can be waived if the area of tensile reinforcement at every section is at least one-third greater than required by analysis.

(c) Moment capacity requirements:
At beam ends
$M_y^+ \geq 0.5 M_y^-$
At any section in beam span
$M_y^+$ or $M_y^- \geq 0.25$ ($M_y^{max}$ at beam ends)

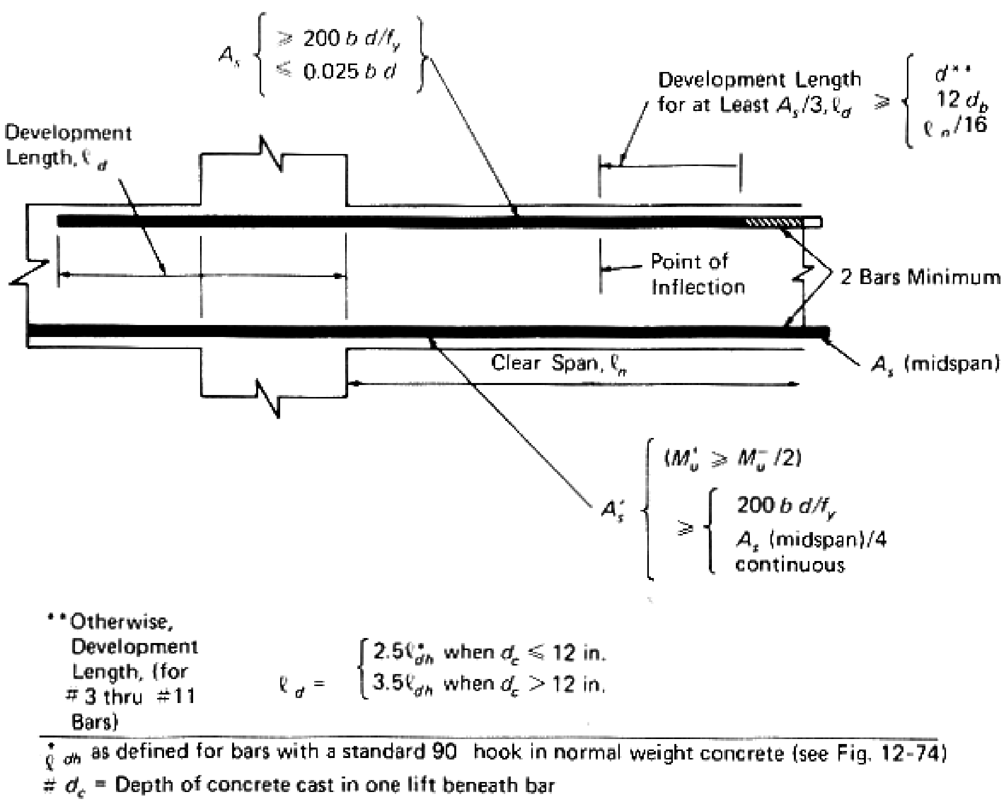



*Figure 10-34.* Longitudinal reinforcement requirements for flexural members

(d) Restrictions on lap splices: Lap splices shall not be used

(1) within joints,
(2) within $2h$ from face of support, where $h$ is total depth of beam,
(3) at locations of potential plastic hinging. Lap splices, where used, are to be confined by hoops or spiral reinforcement with a maximum spacing or pitch of d/4 or 4 in.

(e) Restrictions on welding of longitudinal reinforcement: Welded splices and mechanical connectors may be used provided:

(1) they are used only on alternate bars in each layer at any section;
(2) the distance between splices of adjacent bars is ≥ 24 in.
(3) Except as noted above, welding of reinforcement required to resist load combinations including earthquake effects is not permitted. Also, the welding of stirrups, ties, inserts, or other similar elements to longitudinal bars is prohibited

(f) Development length requirements for longitudinal bars in tension:

(1) For bar sizes 3 through 11 with a standard 90° hook (as shown in Figure 10-35) in normal weight concrete, the development length

$$l_{dh} \geq \begin{cases} \dfrac{f_y d_b}{65\sqrt{f_c{}'}} \\ 8d_b \\ 6\,in. \end{cases}$$

($d_b$ is bar diameter).

(2) When bars are embedded in lightweight-aggregate concrete, the development length is to be at least equal to the greater of $10d_b$, 7.5 in. or 1.25 times the values indicated above.
(3) The 90° hook shall be located within the confined core of a column or boundary element.
(4) For straight bars of sizes 3 through 11, the development length,

$l_d \geq 2.5$ x ($l_{dh}$ for bars with 90° hooks) , when the depth of concrete cast in one lift beneath the bar is ≤ 12 in., or $l_d \geq 3.5$ × ($l_{dh}$ for bars with 90° hooks) if the above mentioned depth is > 12 in.

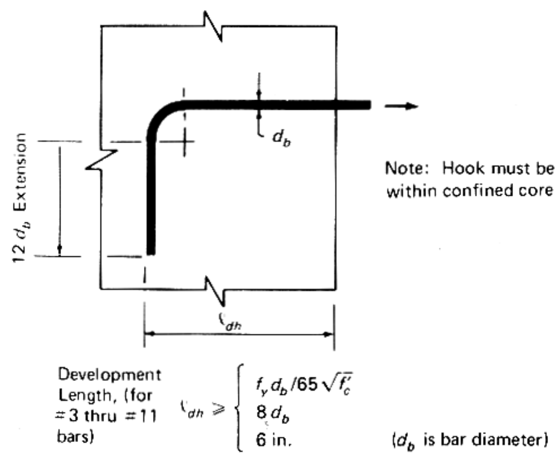



*Figure 10-35.* Development length for beam bars with 90° hooks.

(5) If a bar is not anchored by means of a 90° hook within the confined column core, the portion of the required straight development length not located within the confined core shall be increased by a factor of 1.6.
(6) When epoxy-coated bars are used, the development lengths calculated above to be increased by a factor of 1.2. However, for straight bars, with covers less than $3d_b$ or clear spacing less than $6d_b$, a factor of 1.5 to be used.

(g) Transverse reinforcement requirements for confinement and shear: Transverse reinforcement in beams must satisfy requirements associated with their dual function as confinement reinforcement and shear reinforcement (see Figure 10-36).

(1) Confinement reinforcement in the form of hoops is required:

(i) over a distance 2d from faces of support (where *d* is the effective depth of the member);

(ii) over distances 2d on both sides of sections within the span where flexural yielding may occur due to earthquake loading.

(2) Hoop spacing:

(iii) First hoop at 2 in. from face of support.

(iv) Maximum spacing

$$\le \begin{cases} d/4 \\ 8\times(\textit{diameter of smallest longitudinal bar}) \\ 24\times(\textit{diameter of hoop bars}) \\ 12\textit{in.} \end{cases}$$

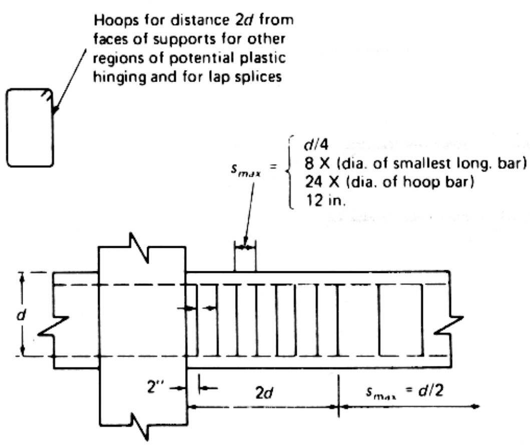



*Figure 10-36.* Transverse reinforcement limitations for flexural members. Minimum bar size- #3

(3) Lateral support for perimeter longitudinal bars where hoops are required: Every corner and alternate longitudinal bar shall be supported by the corner of a hoop with an included angle 135°, with no longitudinal bar farther than 6 in. along the tie from such a laterally supported bar. Where the longitudinal perimeter bars are arranged in a circle, a circular hoop may be used.

(4) Where hoops are not required, stirrups with seismic hooks at both ends with a spacing of not more than d/2 to be provided throughout the length of the member.

(5) Shear reinforcement—to be provided so as to preclude shear failure prior to development of plastic hinges at beam ends. Design shears for determining shear reinforcement are to be based on a condition where plastic hinges occur at beam ends due to the combined effects of lateral displacement and factored gravity loads (see Figure 10-16). The *probable flexural strength,* $M_{pr}$ associated with a plastic hinge is to be computed using a strength reduction factor $\phi$ = 1.0 and assuming a stress in the tensile reinforcement $f_s = 1.25f_y$.

(6) In determining the required shear reinforcement, the contribution of the concrete, $V_c$, is to be neglected if the shear associated with the probable flexural strengths at the beam ends is equal to or greater than one-half the total design shear and the factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.

(7) The transverse reinforcement provided must satisfy the requirements for confinement or shear, whichever is more stringent.

**Discussion:**

(a) Limitations on section dimensions: These limitations have been guided by experience with test specimens subjected to cyclic inelastic loading.

(b) Flexural reinforcement limitations: Because the ductility of a member decreases with increasing tensile reinforcement ratio, ACI Chapter 21 limits the maximum reinforcement ratio to 0.025. The use of a limiting ratio based on the "balanced condition" as given in the earlier chapters of the code, while applicable to members loaded monotonically, fails to describe conditions in flexural members subjected to reversals of inelastic deformation. The limiting ratio of 0.025 is based mainly on considerations of steel congestion and also on limiting shear stresses in beams of typical proportions. From a practical standpoint, low steel ratios should be used whenever possible. The requirements of at least two continuous bars top and bottom, refers to construction rather than behavioral requirements.

The selection of the size, number, and arrangement of flexural reinforcement should be made with full consideration of construction requirements. This is particularly important in relation to beam-column connections, where construction difficulties can arise as a result of reinforcement congestion. The preparation of large-scale drawings of the connections, showing all beam, column, and joint reinforcements, will help eliminate unanticipated problems in the field. Such large-scale drawings will pay dividends in terms of lower bid prices and a smooth-running construction job. Reference 10-76 provides further recommendations on reinforcement detailing.

(c) Positive moment capacity at beam ends: To allow for the possibility of the positive moment at the end of a beam due to earthquake-induced lateral displacements exceeding the negative moment due to the gravity loads, the code requires a minimum positive moment capacity at beam ends equal to 50% of the corresponding negative moment capacity.

(d) Lap splices: Lap splices of flexural reinforcement are not allowed in regions of potential plastic hinging since such splices are not considered to be reliable under reversed inelastic cycles of deformation. Hoops are mandatory for confinement of lap splices at any location because of the likelihood of loss of the concrete cover.

(e) Welded splices and mechanical connectors: Welded splices and mechanical connectors are to conform to the requirements given in Chapter 12 of the ACI 318-95. A major requirement is that the splices develop at least 125% of the specified yield strength of the bar.

As mentioned earlier, the welding of stirrups, ties, inserts, or other similar elements to longitudinal bars is not permitted.

(f) Development length: The expression for $l_{dh}$ given above already includes the coefficients 0.7 (for concrete cover) and 0.80 (for ties) that are normally applied to the basic development length, $l_{db}$. This is so because ACI Chapter 21 requires that hooks be embedded in the confined core of a column or boundary element. The expression for $l_{dh}$ also includes a factor of about 1.4, representing an increase over the development length required for conventional structures, to provide for the effect of load reversals.

Except in very large columns, it is usually not possible to develop the yield strength of a reinforcing bar from the framing beam within the width of a column unless a hook is used. Where beam reinforcement can extend through a column, its capacity is developed by embedment in the column and within the compression zone of the beam on the far side of the connection (see Figure 10-34). Where no beam is present on the opposite side of a column, such as in exterior columns, the flexural reinforcement in a framing beam has to be developed within the confined region of the column. This is usually done by means of a standard 90° hook plus whatever extension is necessary to develop the bar, the development length being measured from the near face of the column, as indicated in Figure 10-35. The use of a beam stub at the far (exterior) side of a column may also be considered (see Figure 10-22). ACI Chapter 21 makes no provision for the use of size 14 and 18 bars because of lack of sufficient information on the behavior at anchorage locations of such bars when subjected to load reversals simulating earthquake effects.

(g) Transverse reinforcement: Because the ductile behavior of earthquake-resistant frames designed to current codes is premised on the ability of the beams to develop plastic hinges with adequate rotational capacity, it is essential to insure that shear failure does not occur before the flexural capacity of the beams has been developed. Transverse reinforcement is required for two related functions: (i) to provide sufficient shear strength so that the full flexural capacity of the member can be developed, and (ii) to insure adequate rotation capacity in plastic-hinging regions by confining the concrete in the compression zones and by providing lateral support to the compression steel. To be equally effective with respect to both functions under load reversals, the transverse reinforcement should be placed perpendicular to the longitudinal reinforcement.

Shear reinforcement in the form of stirrups or stirrup ties is to be designed for the shear due to factored gravity loads and the shear corresponding to plastic hinges forming at both ends of a beam. Plastic end moments associated with lateral displacement in either direction should be considered (Figure 10-16). It is important to note that the required shear strength in beams (as in columns) is determined by the flexural strength of the frame member (as well as the factored loads acting on the member), rather than by the factored shear force calculated from a lateral load analysis. The use of the factor 1.25 on $f_y$ for calculating the probable moment strength is intended to allow for the actual steel strength exceeding the specified minimum and also recognizes that the strain in reinforcement of sections undergoing large rotations can enter the strain-hardening range.

To allow for load combinations not accounted for in design, a minimum amount of web reinforcement is required throughout the length of all flexural members. Within regions of potential hinging, stirrup ties or hoops are required. A hoop may be made of two pieces of reinforcement: a stirrup having 135° hooks with 6-diameter extensions anchored in the confined core and a crosstie to close the hoop (see Figure 10-37). Consecutive ties are to have their 90° hooks on opposite sides of the flexural member.

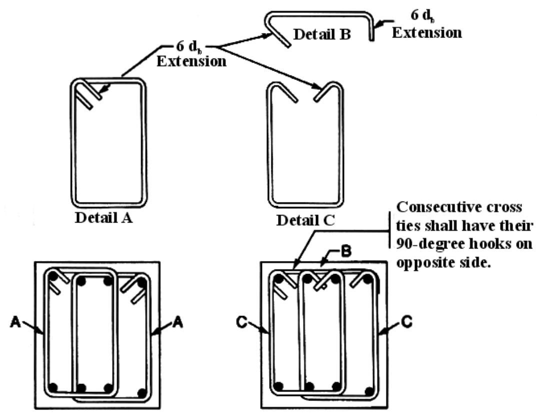



*Figure 10-37.* Single and two-piece hoops

3. *Frame members subjected to axial load and bending.* ACI Chapter 21 makes the distinction between columns or beam—columns and flexural members on the basis of the magnitude of the factored axial load acting on the member. Thus, if the factored axial load does not exceed $A_g f'_c/10$, the member falls under the category of flexural members, the principal design requirements for which were discussed in the preceding section. When the factored axial force on a member exceeds $A_g f'_c/10$, the member is considered a beam—column. Major requirements governing the design of such members in structures located in areas of high seismic risk are given below.

(a) Limitations on section dimensions: shortest cross-sectional dimension ≥ 12 in. (measured on line passing through geometric centroid);

$$\frac{\text{shortest dimension}}{\text{perpendicular dimension}} \geq 0.4$$

(b) Limitations on longitudinal reinforcement:

$$\rho_{min} = 0.01, \qquad \rho_{max} = 0.06$$

(c) Flexural strength of columns relative to beams framing into a joint (the so-called "strong column-weak beam" provision):

$$\sum M_e \geq \frac{6}{5}\sum M_g \tag{10-5}$$

where

$\sum M_e$ = sum of the design flexural strengths of the columns framing into joint. Column flexural strength to be calculated for the factored axial force, consistent with the direction of the lateral loading considered, that results in the lowest flexural strength

$\sum M_g$ =sum of design flexural strengths of beams framing into joint

The lateral strength and stiffness of columns not satisfying the above requirement are to be ignored in determining the lateral strength and stiffness of the structure. Such columns have to be designed in accordance with the provisions governing members not proportioned to resist earthquake-induced forces, as contained in the ACI section 21.7. However, as the commentary to the Code cautions, any negative effect on the building behavior of such non-conforming columns should not be ignored. The potential increase in the base shear or of torsional effects due to the stiffness of such columns should be allowed for.

(d) Restriction on use of lap splices: Lap splices are to be used only within the middle half of the column height and are to be designed as tension splices.

(e) Welded splices or mechanical connectors for longitudinal reinforcement: Welded splices or mechanical connectors may be used at any section of a column, provided that:
  (1) they are used only on alternate longitudinal bars at a section;
  (2) the distance between splices along the longitudinal axis of the reinforcement is ≥ 24 in.

(f) Transverse reinforcement for confinement and shear: As in beams, transverse reinforcement in columns must provide confinement to the concrete core and lateral support for the longitudinal bars as well as shear resistance. In columns, however, the transverse reinforcement must all be in the form of closed hoops or continuous spiral reinforcement. Sufficient reinforcement should be provided to satisfy the requirements for confinement or shear, whichever is larger.
  (1) Confinement requirements (see Figure 10-38):

– Volumetric ratio of spiral or circular hoop reinforcement:

$$\rho_s \geq \begin{cases} 0.12\dfrac{f_c'}{f_{yh}} \\ 0.45\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f_c'}{f_{yh}} \end{cases} \tag{10-6}$$

$f_{yh}$ = specified yield strength of transverse reinforcement, in lb/in.$^2$

$A_{ch}$ = core area of column section, measured to the outside of transverse reinforcement, in in.$^2$

– Rectangular hoop reinforcement, total cross-sectional area, within spacings:

$$A_{sh} \geq \begin{cases} 0.09\, sh_c \dfrac{f'_c}{f_{yh}} \\ 0.3 sh_c\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f_c'}{f_{yh}} \end{cases} \tag{10-7}$$

where

$h_c$ = cross-sectional dimension of column core, measured center-to-center of confining reinforcement

$s$ = spacing of transverse reinforcement

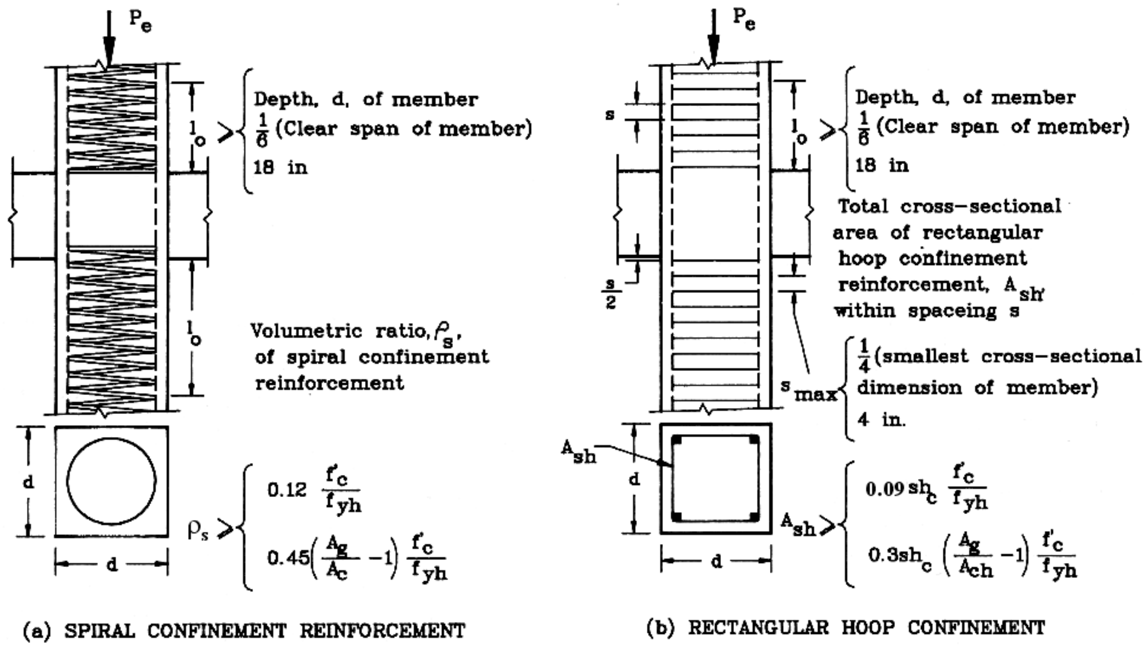



*Figure 10-38.* Confinement requirements for column ends.

measured along axis of member, in in.

$s_{max}$ = min { ¼(smallest cross-sectional dimension of member),4 in. }

maximum permissible spacing in plane of cross-section between legs of overlapping hoops or cross ties is 14 in.

(2) Confinement reinforcement is to be provided over a length $l_0$ from each joint face or over distances $l_0$ on both sides of any section where flexural yielding may occur in connection with lateral displacements of the frame, where

$$l_0 \geq \begin{cases} \text{depth } d \text{ of member} \\ 1/6(\text{clear span of member}) \\ 18\,in. \end{cases}$$

UBC-97 further requires that confinement reinforcement be provided at any section of a column where the nominal axial strength, $\phi P_n$ is less than the sum of the shears corresponding to the probable flexural strengths of the beams (i.e., based on $f_s = 1.25 f_y$ and $\phi = 1.0$) framing into the column above the level considered.

(3) over segments of a column not provided with transverse reinforcement in accordance with Eqs. (10-6) and (10-7) and the related requirements described above, spiral or hoop reinforcement is to be provided, with spacing not exceeding 6 × (diameter of longitudinal column bars) or 6 in., whichever is less.

(4) Transverse reinforcement for shear in columns is to be based on the shear associated with the maximum probable moment strength, $M_{pr}$, at the column ends (using $f_s = 1.25\ f_y$ and $\phi = 1.0$) corresponding to the range of factored axial forces acting on the column. The calculated end moments of columns meeting at a joint need not exceed the sum of the probable moment strengths of the girders framing into the joint. However, in no case should the design

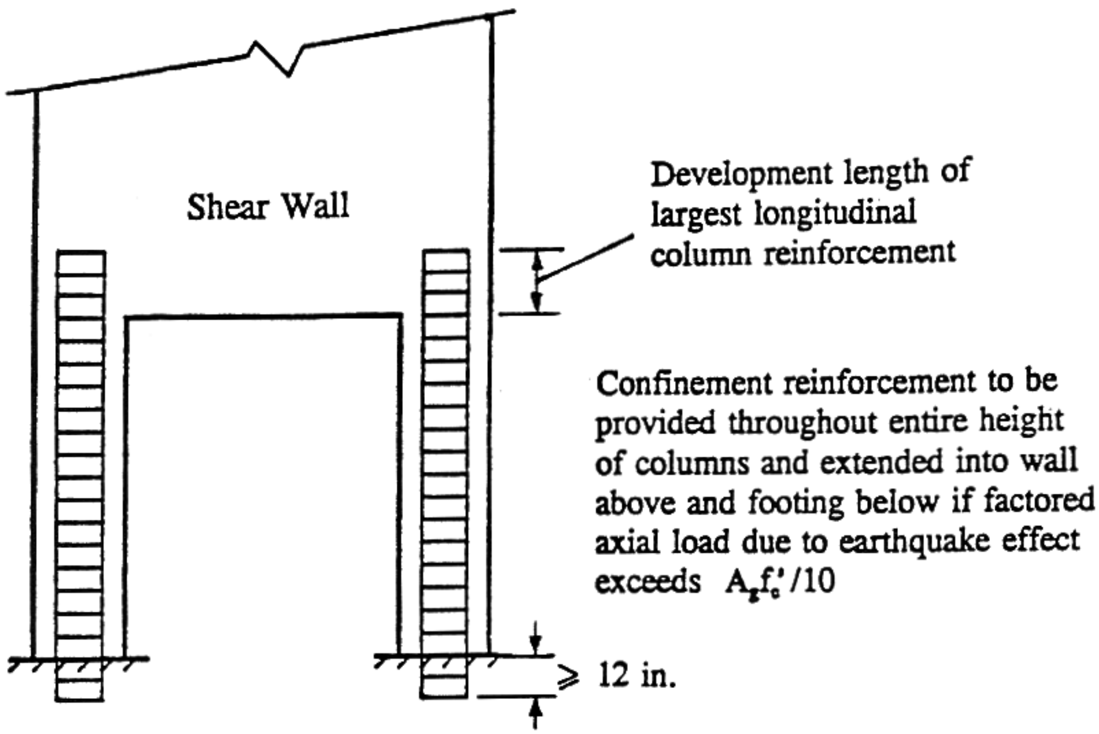



*Figure 10-39.* Columns supporting discontinued wall.

shear be less than the factored shear determined by analysis of the structure.

(g) Column supporting discontinued walls: Columns supporting discontinued shear walls or similar stiff elements are to be provided with transverse reinforcement over their full height below the discontinuity (see Figure 10-39) when the axial compressive force due to earthquake effects exceeds $A_g f_c'/10$.

The transverse reinforcement in columns supporting discontinued walls be extended above the discontinuity by at least the development length of the largest vertical bar and below the base by the same amount where the column rests on a wall. Where the column terminates in a footing or mat, the transverse reinforcement is to be extended below the top of the footing or mat a distance of at least 12 in.

**Discussion:**

(b) Reinforcement ratio limitation: ACI Chapter 21 specifies a reduced upper limit for the reinforcement ratio in columns from the 8% of Chapter 10 of the code to 6%. However, construction considerations will in most cases place the practical upper limit on the reinforcement ratio $\rho$ near 4%. Convenience in detailing and placing reinforcement in beam-column connections makes it desirable to keep the column reinforcement low.

The minimum reinforcement ratio is intended to provide for the effects of time-dependent deformations in concrete under axial loads as well as maintain a sizable difference between cracking and yield moments.

$$(M_{pr}^{ct} + M_{pr}^{cb}) \geq \frac{6}{5}(M_{pr}^{bl} + M_{pr}^{br})$$

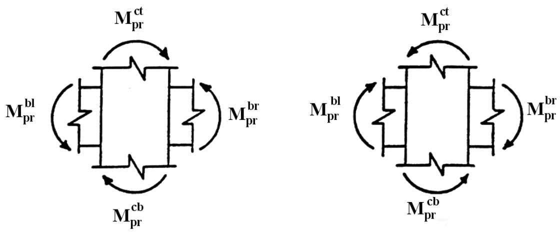



*Figure 10-40.* Strong column-weak beam frame requirements.

(c) Relative column-to-beam flexural strength requirement: To insure the stability of a frame and maintain its vertical-load-carrying capacity while undergoing large lateral displacements, ACI Chapter 21 requires that inelastic deformations be generally restricted to the beams. This is the intent of Equation 10-5 (see Figure 10-40). As mentioned, formation of plastic hinges at both ends of most columns in a story can precipitate a sidesway mechanism leading to collapse of the story and the structure above it. Also, as pointed out in Section 10.3.4 under "Beam—Column Joints," compliance with this provision does not insure that plastic hinging will not occur in the columns.

If Equation 10-5 is not satisfied at a joint, columns supporting reactions from such a joint are to be provided with transverse reinforcement over their full height. Columns not satisfying Equation 10-5 are to be ignored in calculating the strength and stiffness of the structure. However, since such columns contribute to the stiffness of the structure before they suffer severe loss of strength due to plastic hinging, they should not be ignored if neglecting them results in unconservative estimates of design forces. This may occur in determining the design base shear or in calculating the effects of torsion in a structure. Columns not satisfying Equation 10-5 should satisfy the minimum requirements for members not proportioned to resist earthquake-induced forces, discussed under item 6 below.

(f) Transverse reinforcement for confinement and shear: Sufficient transverse reinforcement in the form of rectangular hoops or spirals should be provided to satisfy the larger requirement for either confinement or shear.

Circular spirals represent the most efficient form of confinement reinforcement. The extension of such spirals into the beam—column joint, however, may cause some construction difficulties.

Rectangular hoops, when used in place of spirals, are less effective with respect to confinement of the concrete core. Their effectiveness may be increased, however, with the use of supplementary cross-ties. The cross-ties have to be of the same size and spacing as the hoops and have to engage a peripheral longitudinal bar at each end. Consecutive cross-ties are to be alternated end for end along the longitudinal reinforcement and are to be spaced no further than 14 in. in the plane of the column cross-section (see Figure 10-41). The requirement of having the cross-ties engage a longitudinal bar at each end would almost preclude placing them before the longitudinal bars are threaded through.

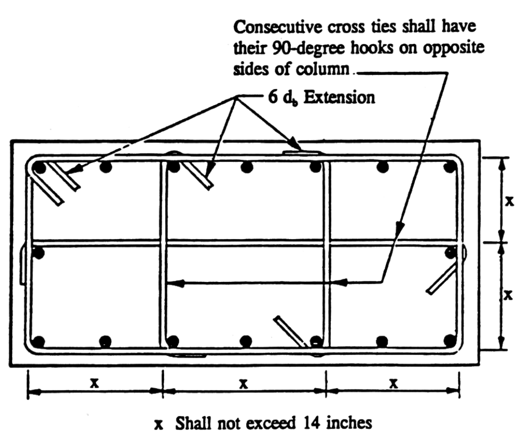



*Figure 10-41.* Rectangular transverse reinforcement in columns.

$$V_t = V_b = \frac{M_{pr}^t + M_{pr}^b}{h}$$

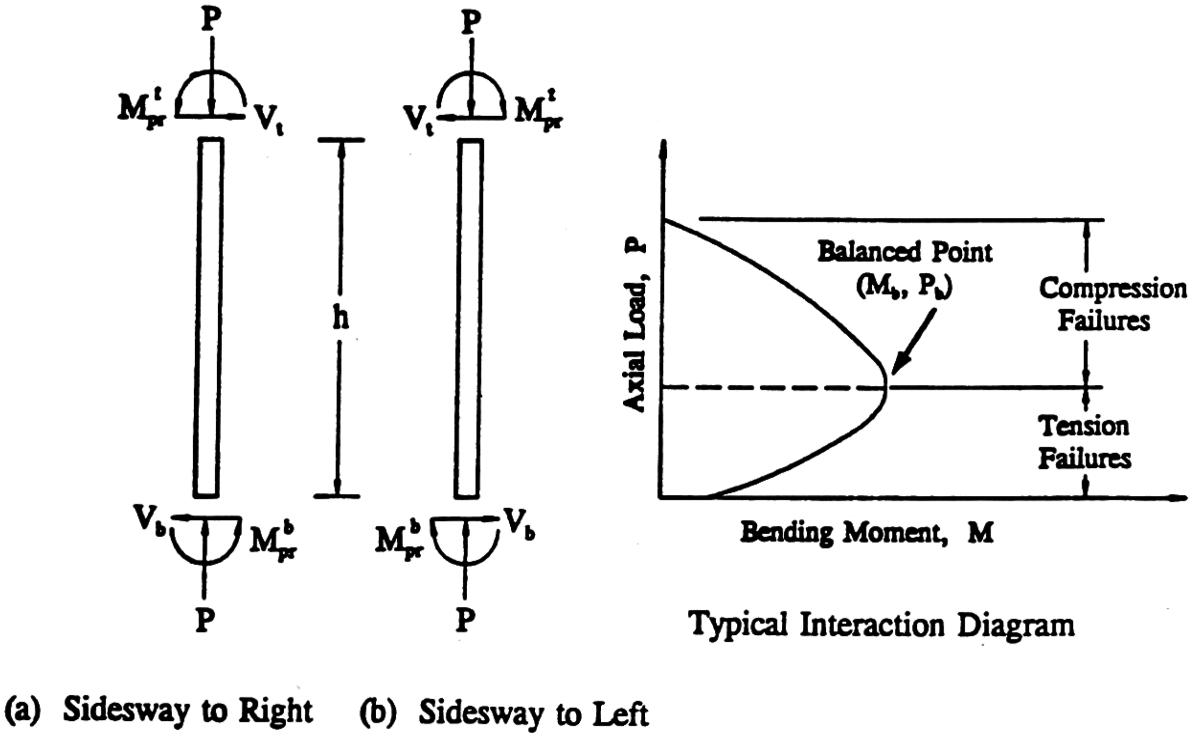



*Figure 10-42*. Loading cases for design of shear reinforcement for columns.

In addition to confinement requirements, the transverse reinforcement in columns must resist the maximum shear associated with the formation of plastic hinges at the column ends. Although the strong column-weak beam provision governing relative moment strengths of beams and columns meeting at a joint is intended to have most of the inelastic deformation occur in the beams of a frame, the code recognizes that hinging can occur in the columns. Thus, the shear reinforcement in columns is to be based on the shear corresponding to the development of the probable moment strengths at the ends of the columns, i.e., using $f_s = 1.25\ f_y$ and $\phi = 1.0$. The values of these end moments —obtained from the *P-M* interaction diagram for the particular column section considered—are to be the maximum consistent with the range of possible factored axial forces on the column. Moments associated with lateral displacements of the frame in both directions, as indicated in Figure 10-42, should be considered. The axial load corresponding to the maximum moment capacity should then be used in computing the permissible shear in concrete, $V_c$.

*(g)* Columns supporting discontinued walls: Columns supporting discontinued shear walls tend to be subjected to large shears and compressive forces, and can be expected to develop large inelastic deformations during strong earthquakes; hence the requirement for transverse reinforcement throughout the height of such columns according to equations (10-6) and (10-7) if the factored axial force exceeds $A_g\ f_c'/10$

*4. Beam-column connections.*

In conventional reinforced-concrete buildings, the beam-column connections usually are not designed by the structural engineer. Detailing of reinforcement within the joints is normally relegated to a draftsman or detailer. In earthquake resistant frames, however, the design of beam-column connections requires as much attention as the design of the members themselves, since the integrity of the frame may well depend on the proper performance of such connections. Because of the congestion that may result from too many bars converging within the limited space of the joint, the requirements for the beam—column connections have to be considered when proportioning the columns of a frame. To minimize placement difficulties, an effort should be made to keep the amount of longitudinal reinforcement in the frame members on the low side of the permissible range.

The provisions of ACI Chapter 21 dealing with beam-column joints relate mainly to:

(a) Transverse reinforcement for confinement: Minimum confinement reinforcement, as required for potential hinging regions in columns and defined by Equations 10-6 and 10-7, must be provided in beam-column joints. For joints confined on all four sides by framing beams, a 50% reduction in the required amount of confinement reinforcement is allowed, the required amount to be placed within the depth of the shallowest framing member. In this case, the reinforcement spacing is not to exceed one-quarter of the minimum member dimension nor 6 in. (instead of 4 in. for non-confined joints). A framing beam is considered to provide confinement to a joint if it has a width equal to at least three-quarters of the width of the column into which it frames.

(b) Transverse reinforcement for shear: The horizontal shear force in a joint is to be calculated by assuming the stress in the tensile reinforcement of framing beams equal to $1.25f_y$ (see Figure 10-21). The shear strength of the connection is to be computed (for normal-weight concrete) as

$$\phi V_c = \begin{cases} \phi 20\sqrt{f_c{}'}A_j \\ \text{for joints confined on all four sides} \\ \phi 15\sqrt{f_c{}'}A_j \\ \text{for joints confined on three sides or} \\ \text{on two opposite sides} \\ \phi 12\sqrt{f_c{}'}A_j \\ \text{for all other cases} \end{cases}$$

where

$\phi = 0.85$ (for shear)

$A_j$ = effective (horizontal) cross-sectional area of joint in a plane parallel to the beam reinforcement generating the shear forces (see Figure 10-43)

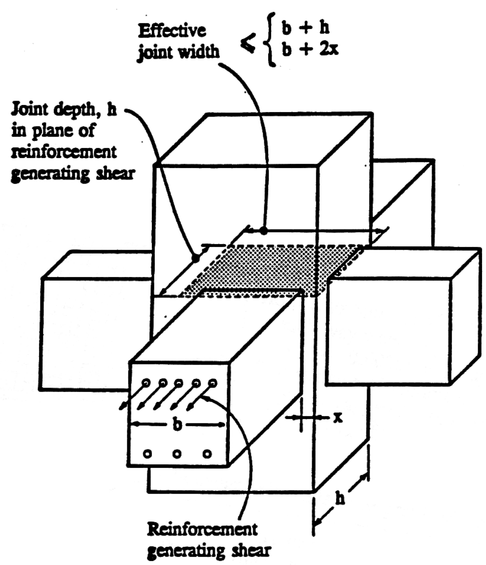



*Figure 10-43.* Beam-column panel zone.

As illustrated in Fig. 10-43, the effective area, $A_j$, is the product of the joint depth and the effective width of the joint. The joint depth is taken as the overall depth of the column (parallel to the direction of the shear considered), while the effective width of the joint is to be taken equal to the width of the column if the beam and the column are of the same width, or, where the column is wider than the framing beam, is not to exceed the smaller of:

– beam width plus the joint depth, and
– beam width plus twice the least column projection beyond the beam side, i.e. the distance x in Fig. 10-43.

For lightweight concrete, $V_c$ is to be taken as three-fourths the value given above for normal-weight concrete.

(c) Anchorage of longitudinal beam reinforcement terminated in a column must be extended to the far face of the confined column core and anchored in accordance with the requirements given earlier for development lengths of longitudinal bars in tension and according to the relevant ACI Chapter 12 requirements for bars in compression.

Where longitudinal beam bars extend through a joint ACI Chapter 21 requires that the column depth in the direction of loading be not less than 20 times the diameter of the largest longitudinal beam bar. For lightweight concrete, the dimension shall be not less than 26 times the bar diameter.

**Discussion:**

(a) Transverse reinforcement for confinement: The transverse reinforcement in a beam-column connection helps maintain the vertical-load-carrying capacity of the joint even after spalling of the outer shell. It also helps resist the shear force transmitted by the framing members and improves the bond between steel and concrete within the joint.

The minimum amount of transverse reinforcement, as given by Equations 10-6 and 10-7, must be provided through the joint regardless of the magnitude of the calculated shear force in the joint. The 50% reduction in the amount of confinement reinforcement allowed for joints having beams framing into all four sides recognizes the beneficial confining effect provided by these members.

(b) Results of tests reported in Reference 10-41 indicate that the shear strength of joints is not too sensitive to the amount of transverse (shear) reinforcement. Based on these results, ACI Chapter 21 defines the shear strength of beam-column connections as a function only of the cross-sectional area of the joint, $(A_j)$ and $f'_c$ (see Section 10.3.4 under "Beam-Column Joints").

When the design shear in the joint exceeds the shear strength of the concrete, the designer may either increase the column size or increase the depth of the beams. The former will increase the shear capacity of the joint section, while the latter will tend to reduce the required amount of flexural reinforcement in the beams, with accompanying decrease in the shear transmitted to the joint. Yet another alternative is to keep the longitudinal beam bars from yielding at the faces of the columns by detailing the beams so that plastic hinging occurs away from the column faces.

(c) The anchorage or development-length requirements for longitudinal beam reinforcement in tension have been discussed earlier under flexural members. Note that lap splicing of main flexural reinforcement is not permitted within the joint.

5. *Shear Walls.* When properly proportioned so that they possess adequate lateral stiffness to reduce inter-story distortions due to earthquake-induced motions, shear walls or structural walls reduce the likelihood of damage to the non-structural elements of a building. When used with rigid frames, walls form a system that combines the gravity-load-carrying efficiency of the rigid frame with the lateral-load-resisting efficiency of the structural wall. In the form of coupled walls linked by appropriately proportioned coupling beams (see Section 10.3.4 under "Coupled Walls"), alone or in combination with rigid frames, structural walls provide a laterally stiff structural system that allows significant energy dissipation to take place

in the more easily repairable coupling beams.

Observations of the comparative performance of rigid-frame buildings and buildings stiffened by structural walls during earthquakes[(10-77)] have pointed to the consistently better performance of the latter. The performance of buildings stiffened by properly designed structural walls has been better with respect to both life safety and damage control. The need to insure that critical facilities remain operational after a major tremor and the need to reduce economic losses from structural and nonstructural damage, in addition to the primary requirement of life safety (i.e., no collapse), has focused attention on the desirability of introducing greater lateral stiffness in earthquake-resistant multistory buildings. Where acceleration-sensitive equipment is to be housed in a structure, the greater horizontal accelerations that may be expected in laterally stiffer structures should be allowed or provided for.

The principal provisions of ACI Chapter 21 relating to structural walls and diaphragms are as follows (see Figure 10-44):

(a) Walls (and diaphragms) are to be provided with shear reinforcement in two orthogonal directions in the plane of the wall. The minimum reinforcement ratio for both longitudinal and transverse directions is

$$\rho_v = \frac{A_{sv}}{A_{cv}} = \rho_n \geq 0.0025$$

where the reinforcement is to be continuous and distributed uniformly across the shear area, and

$A_{cv}$ = net area of concrete section, i.e., product of thickness and width of wall section
$A_{sv}$ = projection on $A_{cv}$ of area of shear reinforcement crossing the plane of $A_{cv}$

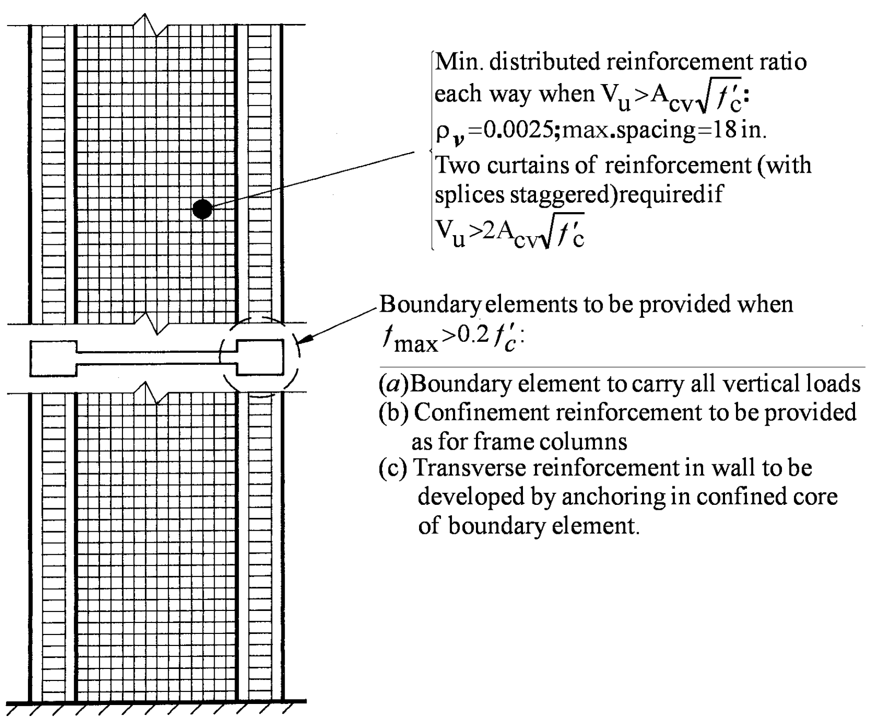



*Figure 10-44.* Structural wall design requirements.

$\rho_n$ = reinforcement ratio corresponding to plane perpendicular to plane of $A_{cv}$

The maximum spacing of reinforcement is 18 in. At least two curtains of reinforcement, each having bars running in the longitudinal and transverse directions, are to be provided if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f_c'}$. If the (factored) design shear force does not exceed $A_{cv}\sqrt{f_c'}$, the shear reinforcement may be proportioned in accordance with the minimum reinforcement provisions of ACI Chapter 14.

(b) Boundary elements: Boundary elements are to be provided, both along the vertical boundaries of walls and around the edges of openings, if any, when the maximum extreme-fiber stress in the wall due to factored forces including earthquake effects exceeds $0.2\sqrt{f_c'}$. The boundary members may be discontinued when the calculated compressive stress becomes less than $0.15\sqrt{f_c'}$. Boundary elements need not be provided if the entire wall is reinforced in accordance with the provisions governing transverse reinforcement for members subjected to axial load and bending, as given by Equations 10-6 and 10-7.

Boundary elements of structural walls are to be designed to carry all the factored vertical loads on the wall, including self-weight and gravity loads tributary to the wall, as well as the vertical forces required to resist the overturning moment due to factored earthquake loads. Such boundary elements are to be provided with confinement reinforcement in accordance with Equations 10-6 and 10-7.

Welded splices and mechanical connections of longitudinal reinforcement of boundary elements are allowed provided that:

1) they are used only on alternate longitudinal bars at a section;
2) the distance between splices along the longitudinal axis of the reinforcement is ≥ 24 in.

The requirements for boundary elements in UBC-97 and IBC-2000 provisions which are essentially similar are much more elaborate and detailed in comparison with ACI-95. In these two provisions , the determination of boundary zones may be based on the level of axial, shear, and flexural wall capacity as well as wall geometry. Alternatively, if such conditions are not met, it may be based on the limitations on wall curvature ductility determined based on inelastic displacement at the top of the wall. Using such a procedure, the analysis should be based on cracked shear area and moment of inertia properties and considering the response modification effects of possible non-linear behavior of building. The requirements of boundary elements using these provisions are discussed in detail under item (f) below.

(c) Shear strength of walls (and diaphragms): For walls with a height-to-width ratio $h_w/l_w \geq 2.0$, the shear strength is to be determined using the expression:

$$\phi V_n = \phi A_{cv}\left(2\sqrt{f_c'} + \rho_n f_y\right)$$

where

$\phi = 0.60$, unless the nominal shear strength provided exceeds the shear corresponding to development of nominal flexural capacity of the wall

$A_{cv}$= net area as defined earlier

$h_w$ = height of entire wall or of segment of wall considered

$l_w$= width of wall (or segment of wall) in direction of shear force

For walls with $h_w/l_w < 2.0$, the shear may be determined from

$$\phi V_n = \phi A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right)$$

where the coefficient $\alpha_c$ varies linearly from a value of 3.0 for $h_w/l_w = 1.5$ to 2.0 for $h_w/l_w$

= 2.0. Where the ratio $h_w/l_w < 2.0$ , $\rho_v$ can not be less than $\rho_n$.

Where a wall is divided into several segments by openings, the value of the ratio $h_w/l_w$ to be used in calculating $V_n$ for any segment is not to be less than the corresponding ratio for the entire wall.

The nominal shear strength $V_n$ of all wall segments or piers resisting a common lateral force is not to exceed $8A_{cv}\sqrt{f_c'}$ where $A_{cv}$ is the total cross-sectional area of the walls. The nominal shear strength of any individual segment of wall or pier is not to exceed $10A_{cp}\sqrt{f_c'}$ where $A_{cp}$ is the cross-sectional area of the pier considered.

(d) Development length and splices: All continuous reinforcement is to be anchored or spliced in accordance with provisions governing reinforcement in tension, as discussed for flexural members.

Where boundary elements are present, the transverse reinforcement in walls is to be anchored within the confined core of the boundary element to develop the yield stress in tension of the transverse reinforcement. For shear walls without boundary elements, the transverse reinforcement terminating at the edges of the walls are to be provided with standard hooks engaging the edge (vertical) reinforcement. Otherwise the edge reinforcement is to be enclosed in U-stirrups having the same size and spacing as, and spliced to, the transverse reinforcement. An exception to this requirement is when $V_u$ in the plane of the wall is less than $A_{cv}\sqrt{f_c'}$ .

(e) Coupling beams: UBC-97 and IBC-2000 provide similar guidelines for coupling beams in coupled wall structures. For coupling beams with $l_n/d \geq 4$, where $l_n$ = clear length of coupling beam and d = effective depth of the beam, conventional reinforcement in the form of top and bottom reinforcement can be used. However, for coupling beams with $l_n/d < 4$ , and factored shear stress exceeding $4\sqrt{f_c'}$ , reinforcement in the form of two intersecting groups of symmetrical diagonal bars to be provided. The design shear stress in coupling beams should be limited to $10\phi\sqrt{f_c'}$ where $\phi$ = 0.85.

(f) Provisions of IBC-2000 and UBC-97 related to structural walls: These provisions treat shear walls as regular members subjected to combined flexure and axial load. Since the proportions of such walls are generally such that they function as regular vertical cantilever beams, the strains across the depth of such members (in the plane of the wall) are to be assumed to vary linearly, just as in regular flexural members, i.e., the nonlinear strain distribution associated with deep beams does not apply. The effective flange width to be assumed in designing I-, L-, C- or T-shaped shear wall sections, i.e., sections formed by intersecting connected walls, measured from the face of the web, shall not be greater than (a) one-half the distance to the adjacent shear wall web, or (b) 15 percent of the total wall height for the flange in compression or 30 percent of the total wall height for the flange in tension, not to exceed the total projection of the flange.

Walls or portions of walls subject to an axial load $P_u > 0.35\ P_0$ shall not be considered as contributing to the earthquake resistance of a structure. This follows from the significantly reduced rotational ductility of sections subjected to high compressive loads (see Fig. 10-11(b)).

When the shear $V_u$ in the plane of the wall exceeds $A_{cv}\sqrt{f_c'}$, the need to develop the yield strength in tension of the transverse reinforcement is expressed in the requirement to have horizontal reinforcement terminating at the edges of shear walls, with or without boundary elements, anchored using standard hooks engaging the (vertical) edge reinforcement or alternatively, having the vertical edge reinforcement enclosed in “U” stirrups of the same size and spacing as, and spliced to, the horizontal reinforcement.

Shear Wall Boundary Zones - The detailing requirements for boundary zones, to be described subsequently, need not be satisfied in walls or portions of walls where

$$P_u \le \begin{cases} 0.10 A_g f_c' & \text{for geometrically symmetrical wall sections} \\ 0.05 A_g f_c' & \text{otherwise} \end{cases}$$

and either

$$\frac{M_u}{V_u l_u} \le 1.0 \qquad \text{or} \qquad V_u \le 3 l_w h_w \sqrt{f_c'}$$

where $l_w$ is the length of the entire wall in the direction of the shear force, and $h_w$ is the height of the wall.

Shear walls or portions of shear walls not meeting the above conditions and having $P_u < 0.35\ P_o$ (so that they can be considered as contributing to the earthquake resistance of the structure) are to be provided with boundary zones at each end having a length varying linearly from $0.25l_w$ for $P_u = 0.35P_o$ to $0.15l_w$ for $P_u = 0.15P_o$, with a minimum of $0.15l_w$ and are to be detailed as will be described.

Alternatively, the requirements of boundary zones not meeting the above conditions may be based on the determination of the compressive strain levels at wall edges using cracked section properties. Boundary zone detailing, however, is to be provided over the portions of the wall where compressive strains exceed 0.003. It is important to note that compressive strains are not allowed to exceed 0.015.

For shear walls in which the flexural limit state response is governed by yielding at the base of the wall, the total curvature demand ($\phi_t$) can be obtained from:

$$\phi_t = \frac{\Delta_i}{(h_w - l_p/2) l_p} + \phi_y$$

where

$\Delta_i$ = inelastic deflection at the top of the wall $= (\Delta_t - \Delta_y)$

$\Delta_t$ = total deflection at the top of the wall equal $\Delta_M$, using cracked section properties, or may be taken as $2\Delta_M$, using gross section properties.

$\Delta_y$ = displacement at the top of wall corresponding to yielding of the tension reinforcement at critical section, or may be taken as $(M'_n/M_E)\ \Delta_E$, where $M_E$ equals unfactored moment at critical section when top of wall is displaced $\Delta_E$. $M'_n$ is nominal flexural strength of critical section at $P'_u$.

$h_w$ = height of the wall

$l_p$ = height of the plastic hinge above critical section and which shall be established on the basis of substantiated test data or may be alternatively taken at $0.5l_w$

$\phi_y$ = yield curvature which may be estimated at $0.003/l_w$

If $\phi_t$ is less than or equal to $0.003/c'_u$, boundary zone details as defined below are not required. $c'_u$ is the neutral axis depth at $P'_u$ and $M'_n$. If $\phi_t$ exceeds $0.003/c'_u$, the compressive strains may be assumed to vary linearly over the depth $c'_u$, and have maximum value equal to the product of $c'_u$ and $\phi_t$.

The use of the above procedure is further discussed with the aid of the design example at the end of this Chapter.

Shear wall boundary zone detailing requirements. When required as discussed above, the boundary zones in shear walls are to be detailed in accordance with the following requirements:

(1) *Dimensional requirements:*

(a) The minimum section dimension of the boundary zone shall be $l_w/16$.

(b) Boundary zones shall extend above the elevation where they are required a distance equal to the development length of the largest vertical bar in the boundary zone. Extensions of the boundary zone lateral reinforcement below its base shall conform to the same requirements as for columns terminating on a mat or footing. However, the transverse boundary zone reinforcement need not extend above the base of the boundary zone a distance greater than the larger of $l_w$ or $M_u/4V_u$.

(c) Boundary zones shall have a minimum length of 18 inches (measured along the length) at each end of the wall or portion of wall.

(d) In I-, L-, C- or T-section walls, the boundary zone at each end shall include the effective flange width and shall extend at least 12 in. into the web.

(2) *Confinement Reinforcement:*

(a) All vertical reinforcement within the boundary zone shall be confined by hoops or cross-ties having a steel cross-sectional area

$$A_{sh} > 0.09\, h f_c' / f_{yh}$$

(b) Hoops and cross-ties shall have a vertical spacing,

$$s_{\max} < \begin{cases} 6 \text{ in.} \\ 6 \times (\text{diameter of largest vertical bar} \\ \text{within boundary zone}) \end{cases}$$

(c) The length-to-width ratio of the hoops shall not exceed 3; and all adjacent hoops shall be overlapping.

(d) Cross-ties or legs of overlapping hoops shall not be spaced farther apart than 12 in. along the wall.

(e) Alternate vertical bars shall be confined by the corner of a hoop or cross-tie.

(3) *Horizontal reinforcement:*

(a) All horizontal reinforcement terminating within a boundary zone shall be anchored as described earlier, i.e., when $V_u > A_{cv}\sqrt{f_c'}$, horizontal reinforcement are to be provided with standard hooks or be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal bars.

(b) Horizontal reinforcement shall not be lap spliced within the boundary zone.

*(4) Vertical reinforcement:*

(a) Vertical reinforcement shall be provided to satisfy all tension and compression requirements indicated by analysis. (Note again that, in contrast to earlier editions of the code, there is no longer the stipulation of rather arbitrary forces that "boundary elements", and hence the vertical steel reinforcement in these, are to be designed for.)

(b) Area of vertical reinforcement,

$$A_v > \begin{cases} 0.005 \times (\text{area of boundary zone}) \\ \text{Two No. 5 bars at each edge of} \\ \text{the boundary zone} \end{cases}$$

(c) Lap splices of vertical reinforcement within the boundary zone shall be confined by hoops and crossties. The spacing of hoops and crossties confining lap-spliced vertical reinforcement shall not exceed 4 in.

**Discussion:**

(a) The use of two curtains of reinforcement in walls subjected to significant shear (i.e., > $2A_{cv}\, f_c'$) serves to reduce fragmentation and premature deterioration of the concrete under load reversals into the inelastic range. Distributing the reinforcement uniformly across the height and width of the wall helps control the width of inclined cracks.

(b) ACI Chapter 21 allows calculation of the shear strength of structural walls using a coefficient $\alpha_c = 2.0$. However, advantage can be taken of the greater observed shear strength of walls with low height-to-width ratios $h_w/l_w$ by using an $\alpha_c$ value of up to 3.0 for walls with $h_w/l_w = 1.5$ or less.

The upper bound on the average nominal shear stress that may be developed in any individual segment of wall ($10\sqrt{f_c'}$) is intended to limit the degree of shear redistribution among several connected wall segments. A wall segment refers to a part of a wall bounded by openings or by an opening and an edge.

It is important to note that ACI Chapter 21 requires the use of a strength-reduction factor $\phi$ for shear of 0.6 for all members (except joints) where the nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. In the case of beams, the design shears are obtained by assuming plastic end moments corresponding to a tensile steel stress of $1.25f_y$ (see Figure 10-16). Similarly, for a column the design shears are determined not by applying load factors to shears obtained from a lateral load analysis, but from consideration of the maximum probable moment strengths at the column ends consistent with the axial force on the column. This approach to shear design is intended to insure that even when flexural hinging occurs at member ends due to earthquake-induced deformations, no shear failure would develop. Under the above conditions, ACI Chapter 21 allows the use of the normal strength-reduction factor for shear of 0.85. When design shears are not based on the condition of flexural strength being developed at member ends, the code requires the use of a lower shear strength-reduction factor to achieve the same result, that is, prevention of premature shear failure.

As pointed out earlier, in the case of multistory structural walls, a condition similar to that used for the shear design of beams and columns is not so readily established. This is so primarily because the magnitude of the shear at the base of a (vertical cantilever) wall, or at any level above, is influenced significantly by the forces and deformations beyond the particular level considered. Unlike the flexural behavior of beams and columns in a frame, which can be considered as close-coupled systems (i.e., with the forces in the members determined by the forces and displacements within and at the ends of the member), the state of flexural deformation at any section of a structural wall (a far-coupled system) is influenced significantly by the displacements of points far removed from the section considered. Results of dynamic inelastic analyses of isolated structural walls under earthquake excitation[10-3] also indicate that the base shear in such walls is strongly influenced by the higher modes of response.

A distribution of static lateral forces along the height of the wall essentially corresponding to the fundamental mode response, such as is assumed by most codes,[10-1] will produce flexural yielding at the base if the section at the base is designed for such a set of forces. Other distributions of lateral forces, with a resultant acting closer to the base of the wall, can produce yielding at the base only if the magnitude of the resultant horizontal force, and hence the base shear, is increased. Results of the study of isolated walls referred to above,[10-3] which would also apply to frame—shear-wall systems in which the frame is flexible relative to the wall, in fact indicate that for a wide range of wall properties and input motion characteristics, the resultant of the dynamic horizontal forces producing yielding at the base of the wall generally occurs well below the two-thirds-of-total-height level associated with the fundamental-mode response (see Figure 10-24). This would imply significantly larger base shears than those due to lateral forces distributed according to the fundamental mode response. The study of isolated walls mentioned above indicates ratios of maximum dynamic shears to “fundamental-mode shears” (i.e., shears associated with horizontal forces distributed according to the fundamental-mode response, as used in codes) ranging from 1.3 to 4.0, the value of the ratio increases with increasing fundamental period (see Figure 10-23).

(c) Since multistory structural walls behave essentially as vertical cantilever beams, the horizontal transverse reinforcement is called upon to act as web reinforcement. As such, these bars have to be fully anchored in the boundary elements, using standard 90° hooks whenever possible.

(d) ACI Chapter 21 uses an extreme-fiber compressive stress of $0.2f_c'$, calculated using a linearly elastic model based on gross sections of structural members and factored forces, as indicative of significant compression. Structural walls subjected to compressive stresses exceeding this value are generally required to have boundary elements.

Figure 10-45 illustrates the condition assumed as basis for requiring that boundary elements of walls be designed for all the gravity loads *(W)* as well as the vertical forces associated with overturning of the wall due to earthquake forces *(H)*. This requirement assumes that the boundary element alone may have to carry all the vertical (compressive) forces at the critical wall section when the maximum horizontal earthquake force acts on the wall. Under load reversals, such a loading condition imposes severe demands on the concrete in the boundary elements; hence the requirement for confinement reinforcement similar to those for frame members subjected to axial load and bending. Diaphragms of reinforced concrete, such as floor slabs, that are called upon to transmit horizontal forces through bending and shear in their own plane, are treated in much the same manner as structural walls.

6. *Frame members not forming part of lateral-force-resisting system*. Frame members that are not relied on to resist earthquake-induced forces need not satisfy the stringent requirements governing lateral-load-resisting elements. These relate particularly to the transverse reinforcement requirements for confinement and shear. Non-lateral-load-resisting elements, whose primary function is the transmission of vertical loads to the foundation, need comply only with the reinforcement requirements of ACI Chapter 21, in addition to those found in the main body of the code.

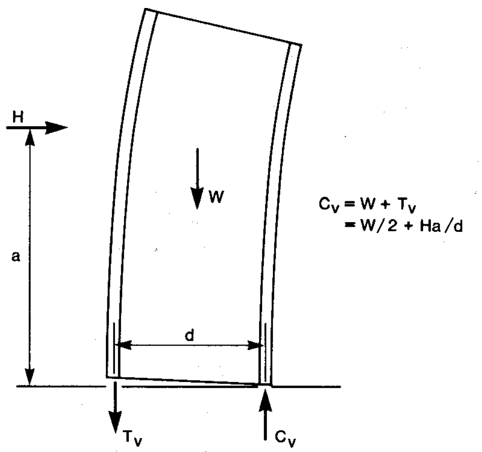



*Figure 10-45*. Loading condition assumed for design of boundary elements of structural walls.

The 1994 Northridge earthquake caused the collapse or partial collapse of at least two parking structures that could be attributed primarily to the failure of interior columns designed to gravity loads only. Following the experience, the requirements for frame members not proportioned to resist forces induced by earthquake motions have been extensively rewritten for the ACI 95 code. A flow chart is provided in Figure 10-46 for ease in understanding the new provisions. The requirements are as follows:

A special requirement for non-lateral-load-resisting elements is that they be checked for adequacy with respect to a lateral displacement representing the expected actual displacement of the structure under the design earthquake. For the purpose of this check, ACI Chapter 21 uses a value of twice the displacement calculated under the factored lateral loads, or 2×1.7 = 3.4 times the displacement due to the code-specified loads. This effect is combined with the effects of dead or dead and live load whichever is critical. If $M_u$ and $V_u$ for an element of gravity system are less than the corresponding nominal values, that element is going to remain elastic under the design earthquake displacements. If such an element is a beam ($P_u \leq A_g f_c'/10$), it must conform to section 2 described earlier for minimum longitudinal reinforcement requirements. In addition, stirrups spaced at no more than $d/2$ must be provided throughout the length of the member. If such an element is a column, it must conform to some of the requirements listed under sections 2 and 3 for longitudinal and shear reinforcement. In addition, similar requirements for cross-ties under section 3(f), discussion, must be met. Also ties at a maximum spacing of $s_o$ must not exceed six times the smallest longitudinal bar diameter, nor 6 in. Further, if $P_u > 0.35\ P_o$, the amount of transverse reinforcement provided must be no less than one-half that required by 3(f).

If $M_u$ and $V_u$ for an element of gravity system exceeds the corresponding nominal values, then it is likely to become inelastic under the design earthquake displacements. Also if deformation compatibility is not checked, this condition will be assumed to be the case. In that case, the structural material must satisfy the requirements described in section 1 and splices of reinforcement must satisfy section 2(e). If such an element is a beam ($P_u \leq A_g f_c'/10$), it must conform to sections 2(b), and 2(g)- (5) and (6). In addition, the stirrups at no more than d/2 must be provided throughout the length of the member. If it is a column, it must be provided with full ductile detailing in accordance with section 3(f), 3(g), and 4(a) as well as sections 2(g)-(5) and (6).

7. *Frames in regions of moderate seismic risk.* Although ACI Chapter 21 does not define "moderate seismic risk" in terms of a commonly accepted quantitative measure, it assumes that the probable ground-motion intensity in such regions would be a fraction of that expected in a high-seismic-risk zone, to which the major part of Chapter 21 is addressed. By the above description, an area of moderate seismic risk would correspond roughly to zone 2 as defined in UBC-97[10-1] and ASCE 7-95.[10-72] For regions of moderate seismic risk, the provisions for the design of structural walls given in the main body of the ACI Code are considered sufficient to provide the necessary ductility. The requirements in ACI Chapter 21 for structures in moderate-risk areas relate mainly to frames and are contained in the last section, section 21.8.

The same axial compressive force ($A_g f_c'/10$) used to distinguish flexural members from columns in high-seismic-risk areas also applies in regions of moderate seismicity.

(a) Shear design of beams, columns, or two-way slabs resisting earthquake effects: The magnitude of the design shear is not to be less than either of the following:

(1) The sum of the shear associated with the development of the nominal moment strength at each restrained end and that due to factored gravity loads, if any, acting on the member. This is similar to the corresponding requirement for high-risk zones and illustrated in Figure 10-16, except that the stress in the flexural tensile reinforcement is taken as $f_y$ rather than $1.25f_y$.

(2) The maximum factored shear corresponding to the design gravity and earthquake forces, but with the earthquake forces taken as twice the value normally specified by codes. Thus, if the critical load combination consists of dead load *(D)* + live load *(L)* + earthquake effects *(E),* then the design shear is to be computed from

$$U = 0.75[1.4D + 1.7L + 2(1.87E)]$$

(b) Detailing requirements for beams: The positive moment strength at the face of a joint must be at least one-third the negative moment capacity at the same section. (This compares with one-half for high-seismic-risk areas.) The moment strength—positive or negative—at any section is to be no less than one-fifth the maximum moment strength at either end of a member. Stirrup spacing requirements are identical to those for beams in high-seismic-risk areas.

However, closed hoops are not required within regions of potential plastic hinging. It should be noted that lateral reinforcement for flexural framing members subjected to stress reversals at supports to consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement as required according to chapter 7 of ACI 318-95.

(c) Detailing requirements for columns: The same region of potential plastic hinging ($l_o$) as at the ends of columns in a region of high seismicity is defined at each end of a column. The spacing of ties within the region of potential plastic hinging must not exceed the smallest of 8 times the diameter of the smallest longitudinal bar enclosed; 24 times the diameter of the tie bar; or One-half the smallest cross-sectional dimension of the column, and 12 in. Outside the region of potential plastic hinging, the spacing must not exceed twice the above value. The first tie must be located at no more than half the above spacing from the joint face.

(e) Detailing requirements for two-way slabs without beams: As mentioned earlier, requirements for flat plates in ACI Chapter 21 appear only in the section relating to areas of moderate seismic risk. This suggests that ACI Chapter 21 considers the use of flat plates as acceptable components of the lateral-load-resisting system only for areas of moderate seismicity.

Specific requirements relating to flat-plate and flat-slab reinforcement for frames in moderate-risk zones are given in ACI Chapter 21 and illustrated in the corresponding Commentary.

## 10.5 DESIGN EXAMPLES — REPRESENTATIVE ELEMENTS OF A 12-STORY FRAME - SHEAR WALL BUILDING

### 10.5.1 Preliminaries

A significant part of the damage observed in engineered buildings during earthquakes has resulted from the effects of major structural discontinuities that were inadequately provided for. The message here is clear. Unless proper provision is made for the effects of major discontinuities in geometry, mass, stiffness, or strength, it would be prudent on the part of the engineer to avoid such conditions, which are associated with force concentrations and large ductility demands in localized areas of the structure. Where such discontinuities are unavoidable or desirable from the architectural standpoint, an analysis to obtain estimates of the forces associated with the discontinuity is recommended. IBC-2000[10-61] provides guidelines for estimating design forces in structures with various types of vertical and plan irregularities.

In addition to discontinuities, major asymmetry, with particular regard to the disposition in plan of the lateral-load-resisting elements, should be avoided whenever possible. Such asymmetry, which can result in a significant eccentricity between the center of stiffness and the center of mass (and hence of the resultant inertial force), can produce appreciable torsional forces in the structure. Torsional effects can be critical for corner columns or end walls, i.e., elements located far from the center of stiffness.

Another important point to consider in the preliminary design of a structure relates to the effectiveness of the various lateral-load-resisting components, particularly where these differ significantly in deformation capacity. Efficient use of structural components would suggest that the useful range of deformation of

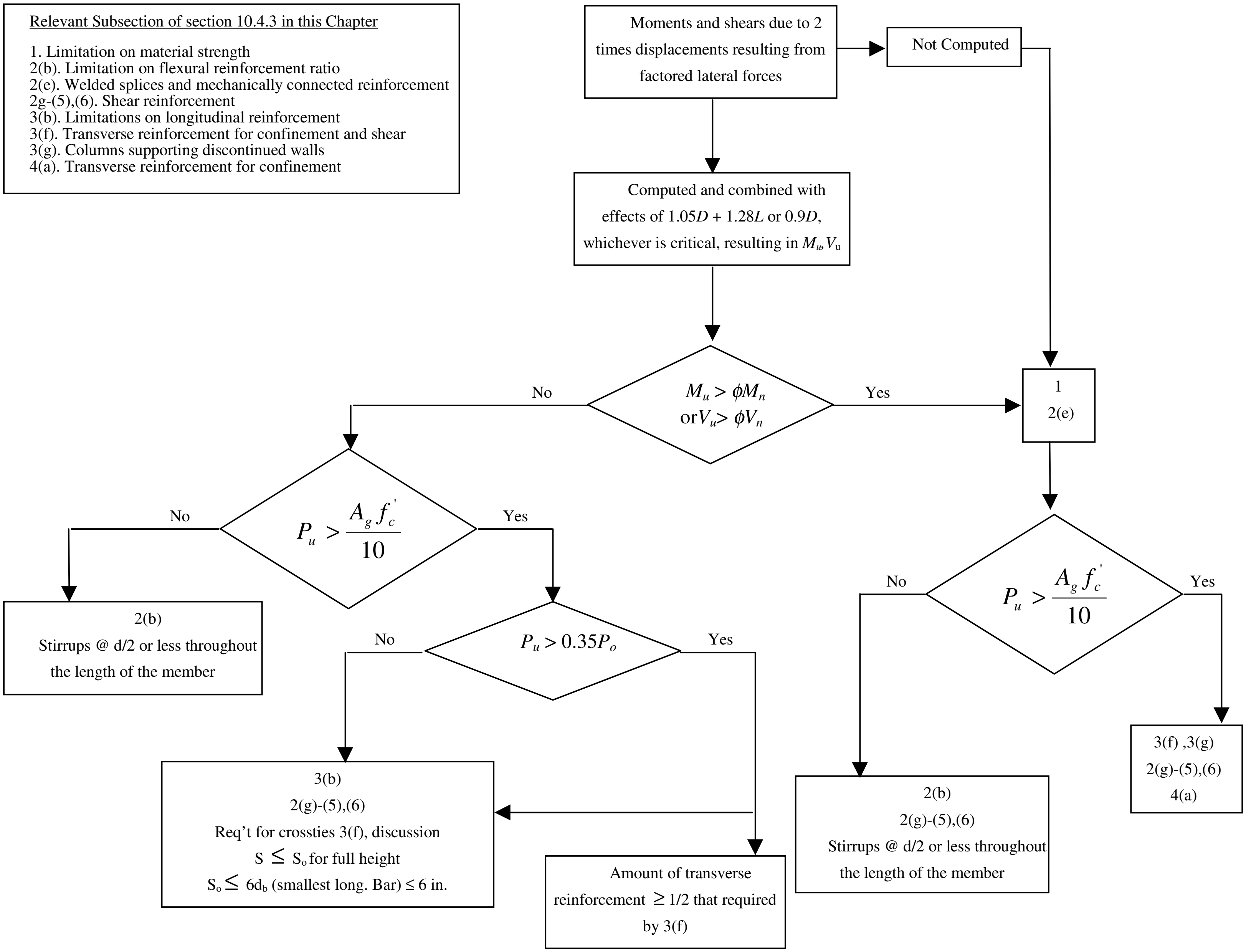



*Figure 10-46.* Requirements for frame members not proportioned to resist forces induced by earthquake motions.

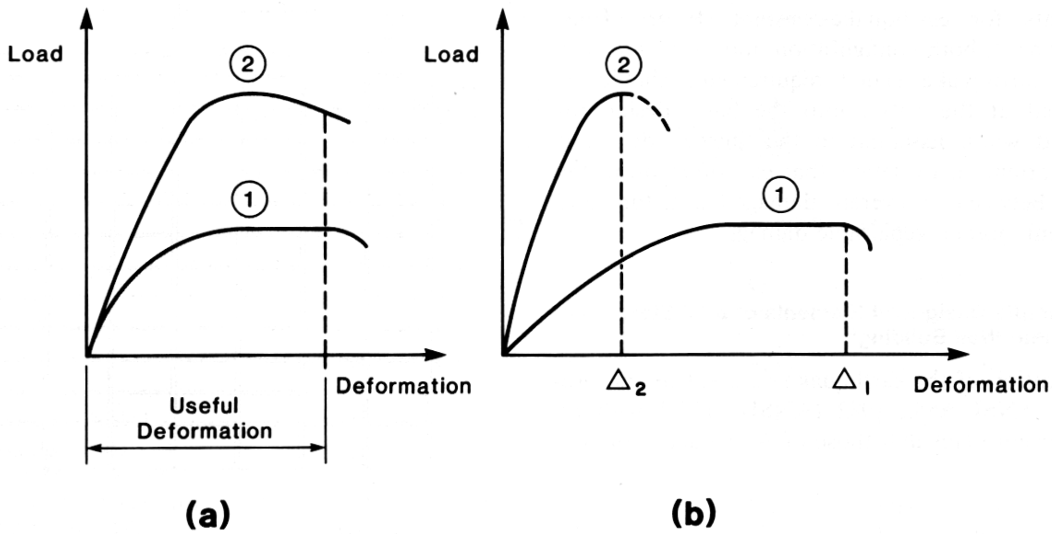



*Figure 10-47.* Relative deformation capacity in lateral-load-resisting elements in structure

the principal lateral-load-resisting elements in a structure be of about the same magnitude whenever practicable. This is illustrated in Figure 10-47a, which shows load—deformation curves of representative elements (1) and (2) in a structure. Such a design allows all the resisting elements to participate in carrying the induced forces over the entire range of deformation. In Figure 10-47b, the resisting elements (1) and (2) not only possess different initial stiffnesses but, more importantly, exhibit different ductilities (not *ductility ratios)* or deformation capacities. In such a case, which is typical of a frame—shear-wall structure, the design should be aimed at insuring that the maximum probable deformation or lateral displacement under dynamic conditions does not exceed the deformation capacity $\Delta_2$ of element (2); or, if the maximum expected deformation could exceed $\Delta_2$, then element (1) should be so designed that it can support the additional load that may come upon it when element (2) loses a considerable part of its load-carrying capacity. It is worth noting that, generally, the lateral displacements associated with full mobilization of the ductility of rigid (open) frames are such that significant nonstructural damage can be expected. For this reason, the building codes limit the amount of deformation that can be tolerated in the structure.

The need to tie together all the elements making up a structure or a portion of it that is intended to act as a unit cannot be overemphasized. This applies to the superstructure as well as foundation elements. Where a structure is divided into different parts by expansion joints, as when the various parts differ considerably in height, plan size, shape, or orientation, a sufficient gap should be provided between adjacent parts to prevent their pounding against each other. To avoid pounding between adjacent buildings or parts of the same building when vibrating out of phase with each other, a gap equal to the square root of the sum of the squares (SRSS) of the maximum lateral deflections (considering the deflection amplification factors specified in building codes) of the two structures under the design (code-specified) lateral forces, or the SRSS of the maximum deflections of the two structures as indicated by a dynamic analysis, would be desirable.

A good basis for the preliminary design of an earthquake-resistant building is a structure proportioned to satisfy the requirements for gravity and wind loads. The planning and layout of the structure, however, must be undertaken with due consideration of the special requirements for earthquake-resistant design. Thus, modifications in both configuration and proportions to anticipate earthquake-related requirements should be incorporated at the outset into the basic design for gravity and wind. Essential to the finished design is particular attention to details that can often mean the difference between a severely damaged structure and one with only minor, repairable damage.

### 10.5.2 Example Designs of Elements of a 12-Story Frame-Shear Wall Building

The application of the earthquake-resistant design provisions of IBC-2000 with respect to design loads and those of ACI 318-95[10-10] relating to proportioning and detailing of members will be illustrated for representative elements of a 12-story frame—shear wall building located in seismic zone 4. The use of the seismic design load provisions in IBC-2000, is based on the fact that it represents the more advanced version, in the sense of incorporating the latest revisions reflecting current thinking in the earthquake engineering profession.

The typical framing plan and section of the structure considered are shown in Figure 10-48a[c] and b, respectively. The columns and structural walls have constant cross-sections throughout the height of the building. The floor beams and slabs also have the same dimensions at all floor levels. Although the dimensions of the structural elements in this example are within the practical range, the structure itself is hypothetical and has been chosen mainly for illustrative purposes. Other pertinent design data are as follows:

Service loads — vertical:

- Live load:
  Basic, 50 lb/ft$^2$.
  Additional average uniform load to allow for heavier basic load on corridors, 25 lb/ft$^2$.
  Total average live load, 75 lb/ft$^2$.
  Roof live load = 20 lb/ft$^2$
- Superimposed dead load:
  Average for partitions 20 lb/ft$^2$.
  Ceiling and mechanical 10 lb/ft$^2$.
  Total average superimposed dead load, 30 lb/ft$^2$.

Material properties:

- Concrete:
  $f_c' = 4000$ lb/in.$^2$ $w_c = 145$ lb/ft$^3$.
- Reinforcement:
  $f_y = 60$ ksi.

***Determination of design lateral forces***

On the basis of the given data and the dimensions shown in Figure 10-48, the weights that may be considered lumped at a floor level (including that of all elements located between two imaginary parallel planes passing through mid-height of the columns above and below the floor considered) and the roof were estimated and are listed in Tables 10-1 and 10-2. The calculation of base shear *V*, as explained in Chapter 5, for the transverse and longitudinal direction is shown at the bottom of Tables 10-1 and 10-2. For this example, it is assumed that the building is located in Southern California with values of $S_s$ and $S_1$ of 1.5 and 0.6 respectively. The site is assumed to be class B (Rock) and the corresponding values of $F_a$ and $F_b$ are 1.0. On this basis, the design spectral response acceleration parameters $S_{DS}$ and $S_{MI}$ are 1.0 and 0.4 respectively. At this level of design parameters, the building is classified as Seismic Group D according to IBC-2000. The building consist of moment resisting frame in the longitudinal direction, and dual system consisting of wall and moment resisting frame in the transverse direction. Accordingly, the response modification factor, R, to be used is 8.0 in both directions.

[c] Reproduced, with modifications, from Reference 10-78, with permission from Van Nostrand Reinhold Company.

***Calculation of the undamped (elastic) natural periods of vibration of the structure in the transverse direction (N-S)***

As shown in Figure 10-49 using the story weights listed in Table 10-1 and member stiffnesses based on gross concrete sections, yielded a value for the fundamental period of 1.17 seconds. The mode shapes and the corresponding periods of the first five modes of vibration of the structure in the transverse direction are shown in Figure 10-49. The fundamental period in the longitudinal (E-W) direction was 1.73 seconds. The mode shapes were calculated using the Computer Program ETABS(10-66), based on three dimensional analysis. In the computer model, the floors were assumed to be rigid. Rigid end offsets were assumed at the end of the members to reflect the actual behavior of the structure. The portions of the slab on each side of the beams were considered in the analysis based on the ACI 318-95 provisions. The structure was assumed to be fixed at the base. The two interior walls were modeled as panel elements with end piers (26x26 in.). The corresponding values of the fundamental period determined based on the approximate formula given in IBC-2000 were 0.85 and 1.27 seconds in the N-S and the E-W directions respectively. However, these values can be increased by 20% provided that they do not exceed those determined from analysis. On this basis, the value of T used to calculate the base shears were 1.02 and 1.52 seconds in the N-S and the E-W directions respectively.

The lateral seismic design forces acting at the floor levels, resulting from the distribution of the base shear in each principal direction are also listed in Tables 10-1 and 10-2.

For comparison, the wind forces and story shears corresponding to a basic wind speed of 85 mi/h and Exposure B ( urban and suburban areas), computed as prescribed in ASCE 7-95, are shown for each direction in Tables 10-1 and 10-2.

Lateral load analysis of the structure along each principal direction, under the respective seismic and wind loads, based on three dimensional analysis were carried out assuming no torsional effects.

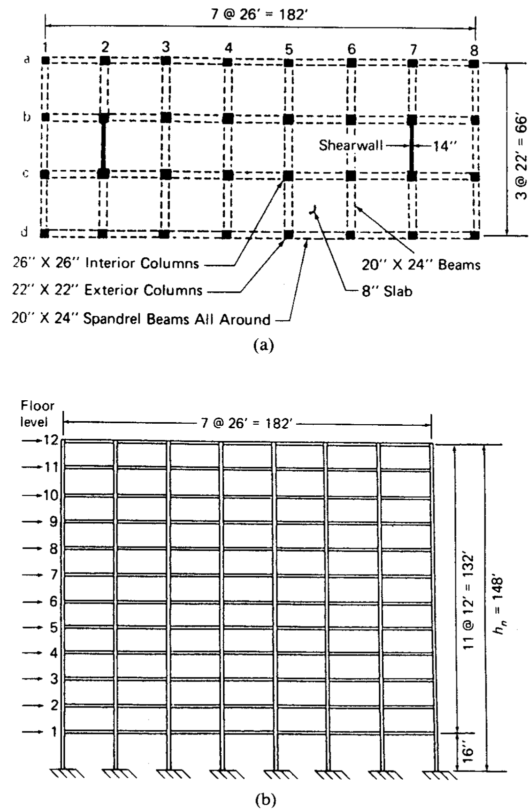



*Figure 10-48.* Structure considered in design example. (a) Typical floor framing plan. (b) Longitudinal section

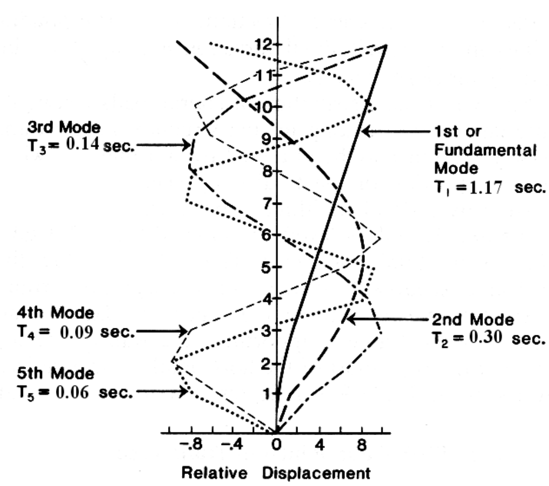



*Figure 10-49.* Undamped natural modes and periods of vibration of structure in transverse direction

*Table 10-1.* Design Lateral Forces in Transverse (Short) Direction (Corresponding to Entire Structure).

| Floor Level | Height, $h_x$, ft | $h_x^k$ k=1.26 | story weight, $w_x$, kips | Seismic forces | | | | Wind forces | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | $w_x h_x^k$ ft-kips x$10^3$ | $C_{vx}$ | Lateral force,$F_x$kips | Story shear $\Sigma F_x$, kips | wind pressure lbs/ft$^2$ | lateral force $H_x$, kips | Story shear $\Sigma H_x$, kips |
| Roof | 148 | 543 | 2100 | 1140 | 0.162 | 208.8 | 208.8 | 21.1 | 23.0 | 23.0 |
| 11 | 136 | 488 | 2200 | 1073 | 0.152 | 196.0 | 404.8 | 20.9 | 45.6 | 68.9 |
| 10 | 124 | 434 | 2200 | 955 | 0.135 | 174.0 | 578.8 | 20.5 | 44.8 | 113.4 |
| 9 | 112 | 382 | 2200 | 840 | 0.120 | 154.7 | 733.5 | 20.2 | 44.1 | 157.5 |
| 8 | 100 | 331 | 2200 | 728 | 0.103 | 132.8 | 866.3 | 19.8 | 43.2 | 200.7 |
| 7 | 88 | 282 | 2200 | 620 | 0.088 | 113.4 | 979.7 | 19.4 | 42.4 | 243.1 |
| 6 | 76 | 234 | 2200 | 515 | 0.073 | 94.1 | 1073.8 | 18.9 | 41.3 | 284.4 |
| 5 | 64 | 189 | 2200 | 415 | 0.059 | 76.1 | 1149.9 | 18.4 | 40.2 | 324.6 |
| 4 | 52 | 145 | 2200 | 320 | 0.045 | 58.0 | 1207.9 | 17.8 | 38.9 | 363.5 |
| 3 | 40 | 104 | 2200 | 230 | 0.033 | 42.5 | 1250.4 | 17.1 | 37.3 | 400.8 |
| 2 | 28 | 67 | 2200 | 147 | 0.021 | 27.1 | 1277.5 | 16.2 | 35.4 | 436.2 |
| 1 | 16 | 33 | 2200 | 72 | 0.010 | 12.9 | 1290.4 | 14.9 | 38.0 | 474.2 |
| Total | | - | 26,300 | 7055 | - | 1290.4 | - | - | 474.2 | - |

Calculation of Design Base Shear in Transverse (Short) Direction

Base shear, V= $C_S$ W where 0.1 $S_{D1}$ I < $C_S = \dfrac{S_{DS}}{R/I} < \dfrac{S_{D1}}{T(R/I)}$

$S_{DS} = 2/3\ S_{MS}$, where $S_{MS} = F_a S_S = 1.0 \times 1.5 = 1.5$ and $S_{D1} = 2/3\ S_{MI}$
where $S_{MI} = F_v S_1 = 1.0 \times 0.6 = 0.6$; $S_{DS} = 1.0$, $S_{D1} = 0.4$; R=8; I=1.0;$T=C_T h_n^{3/4} = 0.02 \times (148)^{3/4}$ =0.849 sec; T can be increased by a factor of 1.2 but should be less than the calculated value (i.e. 1.17 sec). $\therefore T = 0.849 \times 1.2 = 1.018 < 1.17$

$$0.1 \times 0.4 < C_S = \frac{1.0}{8/1} < \frac{0.4}{1.018(8/1)}$$

$0.04 < C_S = 0.125 < 0.0491$ $\therefore$ use $C_S = 0.0491$
<u>V = 0.0491 x 26,300 = 1290.4 kips</u>

*Table 10-2.* Design Lateral Forces in Longitudinal Direction (Corresponding to Entire Structure).

| Floor Level | Height, $h_x$, ft | $h_x^k$ k=1.51 | story weight, $w_x$, kips | Seismic forces | | | | Wind forces | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | $w_x h_x^k$ ft-kips x$10^3$ | $C_{vx}$ | Lateral force, $F_x$, kips | Story shear $\Sigma F_x$, kips | wind pressure lbs/ft$^2$ | lateral force $H_x$, kips | Story shear $\Sigma H_x$, kips |
| Roof | 148 | 1893 | 2100 | 3975 | 0.178 | 154.5 | 154.5 | 17.2 | 6.8 | 6.8 |
| 11 | 136 | 1666 | 2200 | 3665 | 0.164 | 142.4 | 296.9 | 17.0 | 13.5 | 20.3 |
| 10 | 124 | 1449 | 2200 | 3188 | 0.142 | 123.3 | 420.2 | 16.6 | 13.1 | 33.4 |
| 9 | 112 | 1243 | 2200 | 2734 | 0.122 | 105.9 | 526.1 | 16.3 | 12.9 | 46.3 |
| 8 | 100 | 1047 | 2200 | 2304 | 0.103 | 89.4 | 615.5 | 15.9 | 12.6 | 58.9 |
| 7 | 88 | 863 | 2200 | 1899 | 0.085 | 73.8 | 689.3 | 15.5 | 12.3 | 71.2 |
| 6 | 76 | 692 | 2200 | 1522 | 0.068 | 59.0 | 748.3 | 15.0 | 12.0 | 83.2 |
| 5 | 64 | 534 | 2200 | 1174 | 0.052 | 45.1 | 793.4 | 14.5 | 11.5 | 94.7 |
| 4 | 52 | 390 | 2200 | 858 | 0.038 | 33.0 | 826.4 | 13.9 | 11.0 | 105.7 |
| 3 | 40 | 263 | 2200 | 578 | 0.026 | 22.6 | 849.0 | 13.2 | 10.5 | 116.2 |
| 2 | 28 | 153 | 2200 | 337 | 0.015 | 13.0 | 862.0 | 12.3 | 9.7 | 125.9 |
| 1 | 16 | 66 | 2200 | 145 | 0.006 | 5.2 | 867.2 | 11.0 | 10.2 | 136.1 |
| Total | | - | 26,300 | 22,379 | - | 867.2 | - | - | 136.1 | - |

In longitudinal direction, $C_t$ (for reinforced concrete moment resisting frames) = 0.03;
$T = C_t\,(h_n)^{3/4} = (0.03)\,(148) = 1.27$; T can be increased by a factor of 1.2,
$\therefore\; T = 1.2 \times 1.27 = 1.524 < 1.73$

$$0.1 \times 0.4 < C_S = \frac{1.0}{8/1} < \frac{0.4}{1.524(8/1)}$$

$0.04 < C_S = 0.125 < 0.0329 \therefore$ use $C_S = 0.0329$
<u>V = 0.033 × 26,300 = 867.2 kips</u>

(a) Lateral displacements due to seismic and wind effects: The lateral displacements due to both seismic and wind forces listed in Tables 10-1 and 10-2 are shown in Figure 10-50 . Although the seismic forces used to obtain the curves of Figure 10-50 are approximate, the results shown still serve to draw the distinction between wind and seismic forces, that is, the fact that the former are external forces the magnitudes of which are proportional to the exposed surface, while the latter represent inertial forces depending primarily on the mass and stiffness properties of the structure. Thus, while the ratio of the total wind force in the transverse direction to that in the longitudinal direction (see Tables 10-1 and 10-2) is about 3.5, the corresponding ratio for the seismic forces is only 1.5. As a result of this and the smaller lateral stiffness of the structure in the longitudinal direction, the displacement due to seismic forces in the longitudinal direction is significantly greater than that in the transverse direction. By comparison, the displacements due to wind are about the same for both directions. The typical deflected shapes associated with predominantly cantilever or flexure structures (as in the transverse direction) and shear (open-frame) buildings (as in the longitudinal direction) are evident in Figure 10-50. The average deflection indices, that is, the ratios of the lateral displacement at the top to the total height of the structure, are 1/5220 for wind and 1/730 for seismic

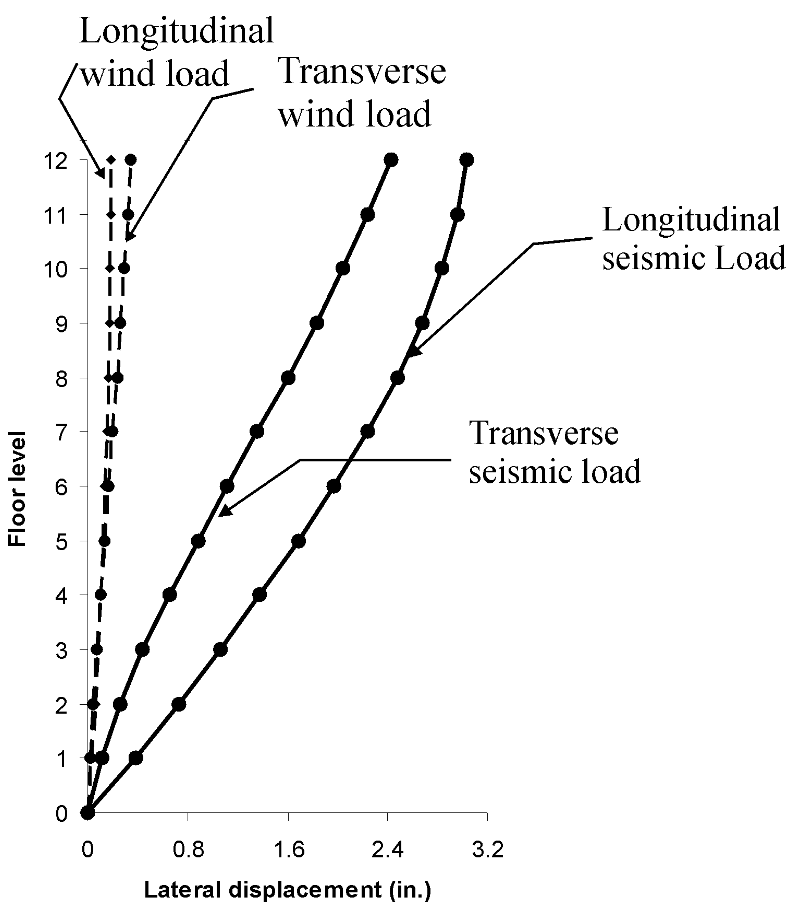



*Figure 10-50.* Lateral displacements under seismic and wind loads.

loads in the transverse direction. The corresponding values in the longitudinal direction are 1/9350 for wind and 1/590 for seismic loads. It should be noted that the analysis for wind was based on uncracked sections whereas that for seismic was based on cracked sections. The use of cracked section moment of inertia is a requirement by IBC-2000 for calculation of drift due to earthquake loading. However, under wind loading, the stresses within the structure in this particular example are within the elastic range as can also be observed from the amount of lateral deflections. As a result, the amount of cracking within the members is expected to be insignificant. However, for the case of seismic loading, the members are expected to deform well into inelastic range of response under the design base shear. To consider the effects of cracked sections due to seismic loads, the moments of inertia of beams, columns and walls were assumed to be 0.5, 0.7 and 0.5 of the gross concrete sections respectively.

(b) Drift requirements: IBC-2000 requires that the design story drift shall not exceed the allowable limits. In calculating the drift limits, the effect of accidental torsion was considered in the analysis. On this basis, the mass at each floor level was assumed to displace from the calculated center of mass a distance equal to 5% of the building dimension in each direction. Table 10-3 shows the calculated displacements and the corresponding story drifts in both E-W and N-S directions. To determine the actual story drift, the calculated drifts were amplified using the $C_d$ factor of 6.5 according to IBC-2000. These increased drifts account for the total anticipated drifts including the inelastic effects. The allowable drift limit based on IBC-2000 is 0.025 times the story height which corresponds to 3.6 in. and 4.8 in. at a typical floor and first floor respectively. The calculated values of drift are less than these limiting values. It is to be noted that using IBC-2000 provisions, it is permissible to use the computed fundamental period of the structure without the upper bound limitation when determining the story drifts limits. However, the drift values shown are based on the calculated values of the fundamental period based on the code limits. Since the calculated drifts are less than the allowable values, further analysis based on the adjusted value of period was not necessary. In addition, the P-Δ effect need not to be considered in the analysis when the stability coefficient as defined by IBC-2000 is less than a limiting value. For the 12-story structure, the effect of P-Δ was found to be insignificant.

(c) Load Combinations: For design and detailing of structural components, IBC-2000 requires that the effect of seismic loads to be combined with dead and live loads. The loading combinations to be used are those prescribed in ASCE-95 as illustrated in Equation (10-2) except that the effect of seismic loads are according to IBC-2000 as defined in Equation (10-3).

To consider the extent of structural redundancy inherent in the lateral-force-resisting system, the reliability factor, $\rho$, is defined as follows for structures in seismic design category D as defined by IBC-2000:

$$\rho = 2 - \frac{20}{r_{\max}\sqrt{A_x}}$$

where

$r_{max}$ = the ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear, for a given direction of loading. For shear walls, $r_{max}$ is defined as the shear in the most heavily loaded wall multiplied by $10/l_w$ , divided by the story shear ($l_w$ is the wall length)

$A_x$ = the floor area in square feet of the diaphragm level immediately above the story

*Table 10-3.* Lateral displacements and Inerstory drifts Due to Seismic Loads (in.).

| | E-W Direction | | | N-S Direction | | |
|---|---|---|---|---|---|---|
| Story Level | displacement | drift | drift × $C_d^*$ | displacement | drift | drift × $C_d^*$ |
| Roof | 3.03 | 0.07 | 0.45 | 2.43 | 0.19 | 1.24 |
| 11 | 2.96 | 0.12 | 0.78 | 2.24 | 0.20 | 1.30 |
| 10 | 2.84 | 0.16 | 1.04 | 2.04 | 0.21 | 1.37 |
| 9 | 2.68 | 0.20 | 1.30 | 1.83 | 0.23 | 1.50 |
| 8 | 2.48 | 0.24 | 1.56 | 1.60 | 0.24 | 1.56 |
| 7 | 2.24 | 0.27 | 1.76 | 1.36 | 0.24 | 1.56 |
| 6 | 1.97 | 0.28 | 1.82 | 1.12 | 0.23 | 1.50 |
| 5 | 1.69 | 0.31 | 2.02 | 0.89 | 0.23 | 1.50 |
| 4 | 1.38 | 0.32 | 2.08 | 0.66 | 0.22 | 1.43 |
| 3 | 1.06 | 0.33 | 2.15 | 0.44 | 0.18 | 1.17 |
| 2 | 0.73 | 0.34 | 2.21 | 0.26 | 0.15 | 0.98 |
| 1 | 0.39 | 0.39 | 2.54 | 0.11 | 0.11 | 0.72 |

* $C_d = 6.5$

When calculating the reliability factor for dual systems such as the frame wall structure in the N-S direction, it can be reduced to 80 percent of the calculated value determined as above. However, this value can not be less that 1.0.

In the N-S direction, the most heavily single element for shear is the shear wall. Table 10-4 shows the calculated values for *r* over the 2/3 height of the structure. The maximum value of *r* occurs at the base of the structure where the shear walls carry most of the shear in the N-S direction. On this basis, the maximum value of $\rho$ determined was 1.0.

The load combinations used for the design based on $\rho$= 1.0 and $S_{DS}$=1.0 by combining

*Table 10-4.*Element story shear ratios for redundancy factor in N-S direction.

| Story Level | $V_i$ = shear force in wall | $V_i$ x $10/L_w$ | story shear | $r_i$ |
|---|---|---|---|---|
| 8 | 189 | 78 | 886 | 0.09 |
| 7 | 234 | 97 | 980 | 0.10 |
| 6 | 275 | 114 | 1074 | 0.11 |
| 5 | 317 | 131 | 1150 | 0.11 |
| 4 | 359 | 149 | 1208 | 0.12 |
| 3 | 408 | 169 | 1250 | 0.14 |
| 2 | 448 | 185 | 1278 | 0.15 |
| 1 | 570 | 236 | 1290 | 0.18 |

$$\rho = 2 - \frac{20}{r_{\max}\sqrt{A_x}}$$

$$\rho = 2 - \frac{20}{0.18 \times \sqrt{66 \times 182}} = 0.99 \qquad \text{but} \qquad \rho_{\min} = 1.0$$

equations (10-2) and (10-3) are as follows:

$$U = \begin{cases} 1.2\,D + 1.6\,L + 0.5\,L_r \\ 1.4\,D \pm 1.0\,Q_E + 0.5\,L \\ 0.7\,D \pm 1.0\,Q_E \end{cases} \tag{10-8}$$

The 3-D structure was analyzed using the above load combinations. The dead and live loads were applied to the beams based on tributary areas as shown in Figure 10-51. The effect of accidental torsion was also considered in the analysis.

To protect the building against collapse, IBC-2000 requires that in dual systems, the moment resisting frames be capable to resist at least 25% of prescribed seismic forces. For this reason, the building in the N-S direction was also subjected to 25% of the lateral forces described above without including the shear walls.

An idea of the distribution of lateral loads among the different frames making up the structure in the transverse direction may be obtained from Table 10-5, which lists the portion of the total story shear at each level resisted by each of the three groups of frames. The four interior frames along lines 3, 4,5, and 6 are referred to as Frame T-1, while the Frame T-2 represents the two exterior frames along lines 1 and 8. The third frame, T-3 represents the two identical frame-shear- wall systems along lines 2 and 7. Note that at the top (12th floor level), the lumped frame T-1 takes 126% of the total story shear. This reflects the fact that in frame-shear-wall systems of average proportions, interaction between frame and wall under lateral loads results in the frame "supporting" the wall at the top, while at the base most of the horizontal shear is resisted by

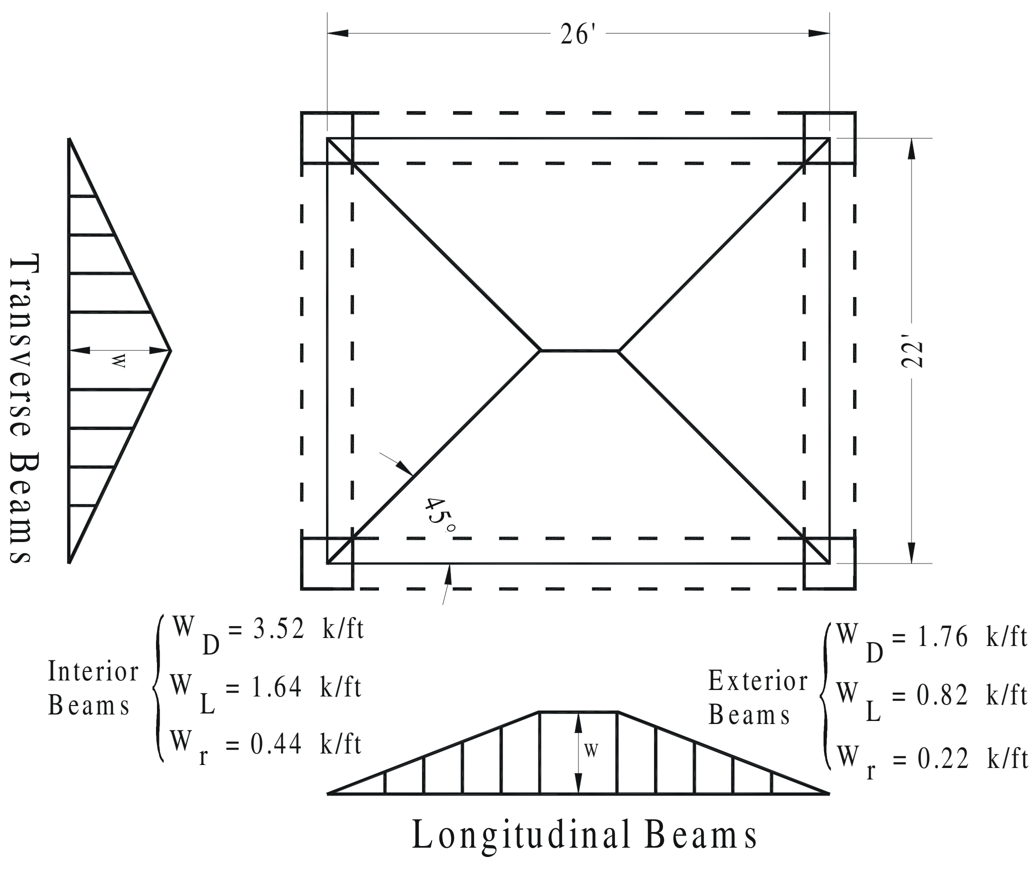



*Figure 10-51.* Tributary area for beam loading.

*Table 10-5.* Distribution of Horizontal Seismic Story Shears among the Three Transverse Frames.

| Story Level | Frame T-1 (4 interior frames) | | Frame T-2 (2 exterior frames) | | Frame T-3 (2 interior frames with shear walls) | | Total story shear, kips |
|---|---|---|---|---|---|---|---|
| | Story shear | % of total | Story shear | % of total | Story shear | % of Total | |
| Roof | 263.6 | 126 | 102.1 | 49 | -156.9 | -75 | 208.8 |
| 11 | 228.5 | 56 | 90.3 | 22 | 86.0 | 21 | 404.8 |
| 10 | 259.9 | 45 | 101.9 | 18 | 216.8 | 37 | 578.8 |
| 9 | 282.5 | 39 | 110.4 | 15 | 340.6 | 46 | 733.5 |
| 8 | 303.6 | 35 | 117.3 | 14 | 445.4 | 51 | 866.3 |
| 7 | 317.3 | 32 | 123.6 | 13 | 538.8 | 55 | 979.7 |
| 6 | 324.0 | 30 | 125.6 | 12 | 624.2 | 58 | 1073.8 |
| 5 | 320.0 | 28 | 124.0 | 11 | 705.9 | 61 | 1149.9 |
| 4 | 303.2 | 25 | 117.9 | 10 | 786.8 | 65 | 1207.9 |
| 3 | 269.6 | 22 | 104.4 | 8 | 876.4 | 70 | 1250.4 |
| 2 | 225.1 | 18 | 86.4 | 7 | 966.0 | 75 | 1277.5 |
| 1 | 96.0 | 7 | 34.8 | 3 | 1159.6 | 90 | 1290.4 |

the wall. Table 10-5 indicates that for the structure considered, the two frames with walls take 90% of the shear at the base in the transverse direction.

To illustrate the design of two typical beams on the sixth floor of an interior frame, the results of the analysis in the transverse direction under seismic loads have been combined, using Equation 10-8, with results from a gravity-load analysis . The results are listed in Table 10-6. Similar values for typical exterior and interior columns on the second floor of the same interior frame are shown in Table 10-7. Corresponding design values for the structural wall section at the first floor of frame on line 3 (see Figure 10-48) are listed in Table 10-8. The last column in Table 10-8 lists the axial load on the boundary elements (the 26 × 26-in, columns forming the flanges of the structural walls) calculated according to the ACI requirement that these be designed to carry all factored loads on the walls, including self-weight, gravity loads, and vertical forces due to earthquake-induced overturning moments. The loading condition associated with this requirement is illustrated in Figure 10-45. In both Tables 10-7 and 10-8, the additional forces due to the effects of horizontal torsional moments corresponding to the minimum IBC-2000 -prescribed eccentricity of 5% of the building dimension perpendicular to the direction of the applied forces have been included.

*Table 10-6.* Summary of design moments for typical beams on sixth floor of interior transverse frames along lines 3 through 6 (Figure 10-48a).

$$U = \begin{cases} 1.2D + 1.6L + 0.5L_r & (9-8a) \\ 1.4D + 0.5L \pm 1.0Q_E & (9-8b) \\ 0.7D \pm 1.0Q_E & (9-8c) \end{cases}$$

| BEAM AB | | Design moment, ft-kips | | |
|---|---|---|---|---|
| | | A | Midspan of AB | B |
| 9-8 a | | -76 | $\boxed{+100}$ | -202 |
| 9-8 b | Sides way to right | +91 | +83 | $\boxed{-326}$ |
| | Sides way to left | $\boxed{-213}$ | +85 | -19 |
| 9-8 c | Sides way to right | $\boxed{+127}$ | +35 | -229 |
| | Sides way to left | -177 | +37 | $\boxed{+79}$ |
| BEAM BC | | Design moment, ft-kips | | |
| | | B | Midspan of BC | C |
| 9-8 a | | -144 | $\boxed{+92}$ | -144 |
| 9-8 b | Sides way to right | -41 | +77 | $\boxed{-282}$ |
| | Sides way to left | $\boxed{-282}$ | +77 | -41 |
| 9-8 c | Sides way to right | $\boxed{+110}$ | +33 | -213 |
| | Sides way to left | -213 | +33 | $\boxed{+110}$ |

It is pointed out that for buildings located in seismic zones 3 and 4 (i.e., high-seismic-risk areas), the detailing requirements for ductility prescribed in ACI Chapter 21 have to be met even when the design of a member is governed by wind loading rather than seismic loads.

2.*Design of flexural member AB.* The aim is to determine the flexural and shear reinforcement for the beam *AB* on the sixth floor of a typical interior transverse frame. The critical design (factored) moments are shown circled in Table 10-6. The beam has dimensions $b$ = 20 in. and $d$ = 21.5 in. The slab is 8 in. thick, $f_c' = 4000$ lb/in.$^2$ and $f_y = 60{,}000$ lb/in.$^2$

In the following solution, the boxed-in section numbers at the right-hand margin correspond to those in ACI 318-95 .

(a) Check satisfaction of limitations on section dimensions:

$$\frac{width}{depth} = \frac{20}{21.5}$$

$$= 0.93 > 0.3 \text{ O.K } \boxed{21.3.1.3}$$

$$\boxed{21.3.1.4}$$

$$width = 20\,in. \begin{cases} \geq 10\,\text{in.} & \text{O.K.} \\ \leq (\text{width of suuporting column} & \\ \quad + 1.5 \times \text{depth of beam} & \\ = 26 + 1.5(21.5) = 58.25\,\text{in.} & \text{O.K.} \end{cases}$$

*Table 10-7.* Summary of design moments and axial loads for typical columns on second floor of interior transverse frames along lines 3 through 6 (Figure 10-48a).

$$U = \begin{cases} 1.2D + 1.6L + 0.5L_r & (9-8a) \\ 1.4D + 0.5L \pm 1.0Q_E & (9-8b) \\ 0.7D \pm 1.0Q_E & (9-8c) \end{cases}$$

| | | Exterior Column A | | | Interior Column B | | |
|---|---|---|---|---|---|---|---|
| | | Axial load, | Moment, ft-kips | | Axial load, | Moment, ft-kips | |
| | | kips | Top | Kips | kips | Top | Bottom |
| 9-8 a | | $\boxed{-1076}$ | -84 | +94 | $\boxed{-1907}$ | +6 | -12 |
| 9-8 b | Sides way to right | -806 | -33 | +25 | -1630 | +73 | -108 |
| | Sides way to left | -1070 | $\boxed{-110}$ | $\boxed{+134}$ | -1693 | $\boxed{-94}$ | $\boxed{+119}$ |
| 9-8 c | Sides way to right | -280 | +8 | -20 | -698 | +79 | -111 |
| | Sides way to left | -544 | -69 | +88 | -760 | -88 | +116 |

*Table 10-8.* Summary of design loads on structural wall section at first floor level of transverse frame along line 2 (or 7) (Figure *10*-48a).

$$U = \begin{cases} 1.2D + 1.6\,L + 0.5L_r & (9-8a) \\ 1.4D + 0.5L \pm 1.0\,Q_E & (9-8b) \\ 0.7\,D\ \pm 1.0\,Q_E & (9-8c) \end{cases}$$

| | Design forces acting on entire structural wall | | | Axial load# on boundary element, kips |
|---|---|---|---|---|
| | Axial Load, kips | Bending (overturning) Moment, ft-kips | Horizontal shear, kips | |
| 9-8 a | $\boxed{-5767}$ | Nominal | Nominal | -2884 |
| 9-8 b | -5157 | $\boxed{30469}$ | $\boxed{651}$ | $\boxed{-3963}$ |
| 9-8 c | -2293 | 30469 | 651 | -2531 |

# Based on loading condition illustrated in Figure 10-45 @ bending moment at base of wall

(b) Determine required flexural reinforcement:

(1) Negative moment reinforcement at support *B:* Since the negative flexural reinforcement for both beams *AB* and *BC* at joint *B* will be provided by the same continuous bars, the larger negative moment at joint *B* will be used. In the following calculations, the effect of any compressive reinforcement will be neglected. From $C = 0.85f_c'ba = T = A_sf_y$,

$$a = \frac{A_s}{0.85 f_c' b} = \frac{60A_s}{(0.85)(4)(20)} = 0.882A_s$$

$$M_u \le \phi M_n = \phi A_s f_y (d - a/2)$$

$$-(326)(12) = (0.90)(60)A_s \times [21.5 - (0.5)(0.882A_s)]$$

$$A_s^2 - 48.76A_s + 164.3 = 0$$

*or*

$$A_s = 3.64\,\text{in.}^2$$

Alternatively, convenient use may be made of design charts for singly reinforced flexural members with rectangular cross-sections, given in standard references.(10-79) *Use five No. 8 bars,* $A_s = 3.95$ *in.*$^2$ This gives a negative moment capacity at support *B* of $\phi M_n$ = 351 ft-kips.

Check satisfaction of limitations on reinforcement ratio:

$$\rho = \frac{A_s}{bd} = \frac{3.95}{(20)(21.5)} \qquad \boxed{21.3.2.1}$$

$$= 0.0092$$

$$> \rho_{\min} = \frac{200}{f_y} = 0.0033$$

$$> \rho_{\min} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4000}}{60{,}000} = 0.0032$$

and $< \rho_{max} = 0.025$ O.K.

(2) Negative moment reinforcement at support *A:*

$M_u$ = 213 ft-kips

As at support *B*, $a = 0.882A_s$.

Substitution into

$$M_u = \phi A_s f_y (d - a/2)$$

yields $A_s$ = 2.31 in.$^2$. *Use three No. 8 bars,* $A_s$ = 2.37 *in.*$^2$ This gives a negative moment capacity at support *A* of $\phi M_n$ = 218 ft-kips.

(3)Positive moment reinforcement at supports: A positive moment capacity at the supports equal to at least 50% of the corresponding negative moment capacity is required, i.e., $\boxed{21.3.2.2}$

$$\min M_u \text{(at support A)} = \frac{218}{2} = 109\, ft - kips$$

which is less than $M^+_{max}$ = 127 ft-kips at *A* (see Table 10-6), but greater than the required $M_u^+$ near midspan of AB (=100 ft-kips).

min $M_u^+$ (at support *B* for both spans *AB* and *BC*) $= \frac{351}{2} = 176\, ft - kips$

Note that the above required capacity is greater than the design positive moments near the mid-spans of both beams *AB* and *BC*.

Minimum positive/negative moment capacity at any section along beam AB or BC = 351/4 =87.8 ft-kips.

(4) Positive moment reinforcement at mid-span of beam AB- to be made continuous to supports: (with an effective T-beam section flange width = 52 in.)

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{60 A_s}{(0.85)(4)(52)} = 0.339 A_s$$

Substituting into

$$M_u = (127)(12) = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

yields $A_s$ (required) = 1.35 in.$^2$. Similarly, corresponding to the required capacity at support *B*, $M_u^+$ = 163 ft-kips, we have $A_s$ (required) =1.74 in.$^2$. *Use three No. 7 bars continuous through both spans. $A_s$ = 1.80 in.$^2$* This provides a positive moment capacity of 172 ft-kips.

Check:

$$\rho = \frac{1.8}{(20)(21.5)} = 0.0042$$

$$> \rho_{\min} = \frac{200}{f_y} = 0.0033 \qquad \text{O.K.} \qquad \boxed{10.5.1}$$

$$> \rho_{\min} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4000}}{60{,}000}$$

(c) Calculate required length of anchorage of flexural reinforcement in exterior column:

$$\text{Development length } l_{dh} \geq \begin{cases} f_y d_b / 65\sqrt{f_c'} \\ 8 d_b \\ 6 in. \end{cases} \qquad \boxed{21.5.4.1}$$

(plus standard 90º hook located in confined region of column). For the No. 8 (top) bars (bend radius, measured on inside of bar, $\geq 3d_b$ = 3.0 in.),

$$l_{dh} \geq \begin{cases} \dfrac{(60{,}000)(1.0)}{65\sqrt{4000}} = 15\, in. \\ (8)(1.0) = 8.0\, in \\ 6\, in. \end{cases}$$

For the No. 7 bottom bars (bend radius $\geq 3d_b$ = 2.7 in.), $l_{dh}$ = <u>13 in.</u>

Figure 10-52 shows the detail of flexural reinforcement anchorage in the exterior column. Note that the development length $l_{dh}$ is measured from the near face of the column to the far edge of the vertical 12-bar-diameter extension (see Figure 10-35).

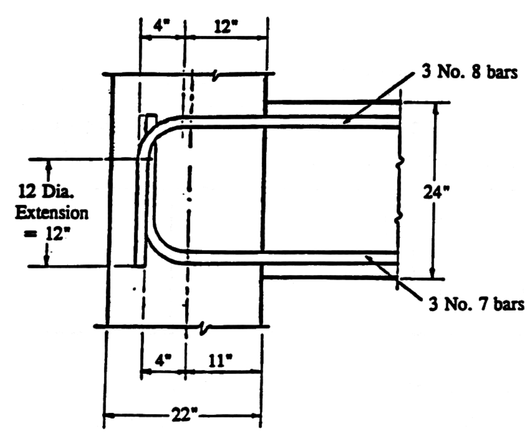



*Figure 10-52.* Detail of anchorage of flexural reinforcement in exterior column

(d) Determine shear-reinforcement requirements: Design for shears corresponding to end moments obtained by assuming the stress in the tensile flexural reinforcement equal to $1.25f_y$ and a strength reduction factor $\phi$ = 1.0, plus factored gravity loads (see Figure 10-16). Table 10-9 shows values of design end shears corresponding to the two loading cases to be considered. In the table,

$W_U = 1.2\ W_D + 1.6\ W_L = 1.2 \times 3.52 + 1.6 \times 1.64 = 6.85$ kips/ft

ACI Chapter 21 requires that the contribution of concrete to shear resistance, $V_c$, be neglected if the earthquake-induced shear force (corresponding to the probable flexural strengths at beam ends calculated using $1.25f_y$ instead of $f_y$ and $\phi$ = 1.0) is greater than one-half the total design shear and the axial compressive force including earthquake effects is less than $A_g f'_c/20$. $\boxed{21.3.4.2}$

For sidesway to the right, the shear at end *B* due to the plastic end moments in the beam (see Table 10-9) is

$$V_b = \frac{230+477}{20} = 35.4\ kips.$$

which is approximately 50% of the total design shear, $V_u$ = 69.6 kips. Therefore, the contribution of concrete to shear resistance can be considered in determining shear reinforcement requirements.

At right end *B*, $V_u$ = 69.6 kips. Using

$$V_c = 2\sqrt{f'_c}\,b_w d = \frac{2\sqrt{4000}(20)(21.5)}{1000} = 54.4 kips$$

we have

$$\phi V_s = V_u - \phi V_c = 69.6 - 0.85 \times 54.4 \qquad \boxed{11.1.1}$$
$$= 23.4\,kips$$

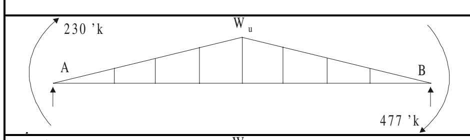
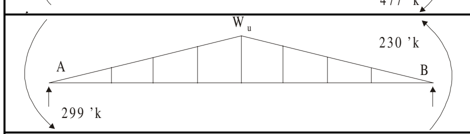
$$V_s = 27.5\,kips$$

Required spacing of No. 3 closed stirrups (hoops), since $A_v$ (2 legs) = 0.22 in.$^2$:

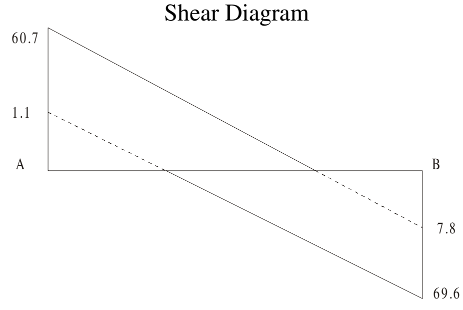
$$s = \frac{A_v f_y d}{V_s} = \frac{(0.22)(60)(21.5)}{27.5} \qquad \boxed{11.5.6.2}$$
$$= 10.3 \text{ in.}$$

Maximum allowable hoop spacing within distance 2d = 2(21.5) = 43 in. from faces of supports:

*Table 10-9.* Determination of Design Shears for Beam AB.

| Loading | $V_u = \dfrac{M_{pr}^A + M_{pr}^B}{l} \pm \dfrac{w_u l}{2}, (kips)$ | |
|---|---|---|
| | A | B |
|  | 1.1 | 69.6 |
|  | 60.7 | 7.8 |

Shear Diagram



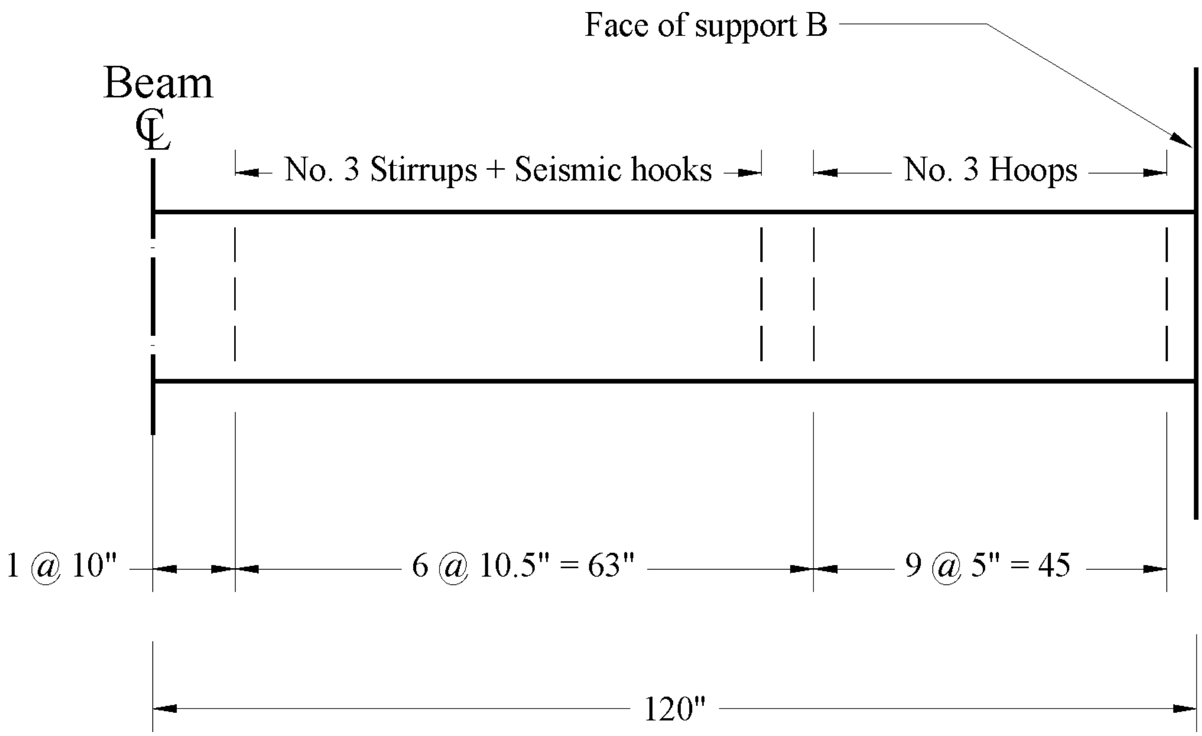



*Figure 10-53.* Spacing of hoops and stirrups in right half of beam AB

$$s_{\max} = \begin{cases} d/4 = 21.5/4 = 5.4\ in. \\ 8 \times (\text{dia. of smallest long. bar}) \\ = 8(0.875) = 7\text{ in.} \\ 24 \times (\text{dia. of hoop bars}) = 24(0.375) = 9\text{ in.} \\ 12\text{ in.} \end{cases}$$

$\boxed{21.3.3.2}$

Beyond distance 2d from the supports, maximum spacing of stirrups:

$$s_{\max} = d/2 = 10.75\ in. \qquad \boxed{21.3.3.4}$$

*Use No. 3 hoops/stirrups spaced as shown in Figure 10-53. The same spacing, turned around, may be used for the left half of beam AB.*

Where the loading is such that inelastic deformation may occur at intermediate points within the span (e.g., due to concentrated loads at or near mid-span), the spacing of hoops will have to be determined in a manner similar to that used above for regions near supports. In the present example, the maximum positive moment near mid-span (i.e., 100 ft-kips, see Table 10-6) is much less than the positive moment capacity provided by the three No. 7 continuous bars (172 ft-kips). $\boxed{21.3.3.1}$

(e) Negative-reinforcement cut-off points: For the purpose of determining cutoff points for the negative reinforcement, a moment diagram corresponding to plastic end moments and 0.9 times the dead load will be used. The cut-off point for two of the five No. 8 bars at the top, near support *B* of beam *AB,* will be determined.

With the negative moment capacity of a section with three No. 8 top bars equal to 218 ft-kips (calculated using $f_s = f_y$ = 60 ksi and $\phi$ = 0.9), the distance from the face of the right support *B* to where the moment under the loading considered equals 218 ft-kips is readily obtained by summing moments about section *a—a* in Figure *10-*54 and equating these to -218 ft-kips. Thus,

$$51.8x - 477 - 3.2\frac{x^3}{60} = -218$$

Solution of the above equation gives $x = 5.1$ ft. Hence, two of the five No. 8 bars near support *B* may be cut off (noting that $d = 21.5$ in.> $12d_b = 12 \times 1.0 = 12$ in.) at $\boxed{12.10.3}$

$$x + d = 5.1 + \frac{21.5}{12} = 6.9\,ft \quad \text{say } 7.0 \text{ ft}$$

from the face of the right support *B*. With $l_{dh}$ (see figure 10-35) for a No. 8 top bar equal to 14.6 in., the required development length for such a bar with respect to the tensile force associated with the negative moment at support *B* is $l_d = 3.5\, l_{dh} = 3.5 \times 14.6/12 = 4.3$ ft < 7.0 ft. Thus, *the two No. 8 bars may be cut off 7.0 ft from the face of the interior support B.* $\boxed{21.5.4.2}$

At end *A,* one of the three No. 8 bars may also be cut off at a similarly computed distance of 4.5 ft from the (inner) face of the exterior support *A. Two bars are required to run continuously along the top of the beam.* $\boxed{21.3.2.3}$

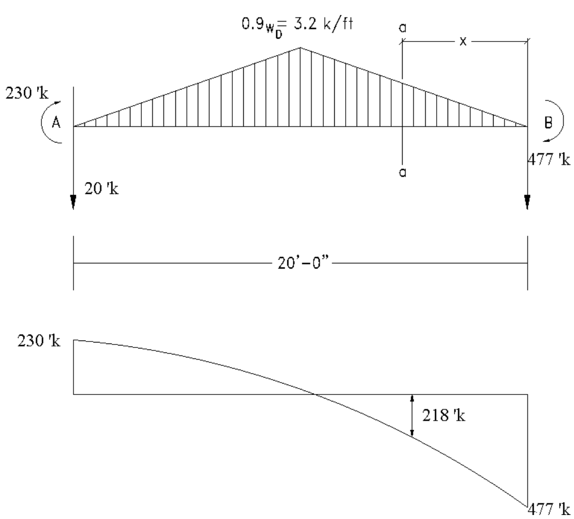



*Figure 10-54.* Moment diagram for beam AB

(f) Flexural reinforcement splices: Lap splices of flexural reinforcement should not be placed within a joint, within a distance 2d from faces of supports, or at locations of potential plastic hinging. Note that all lap splices have to be confined by hoops or spirals with a maximum spacing or pitch of *d*/4, or 4 in., over the length of the lap. $\boxed{21.3.2.3}$

(1) Bottom bars, No. 7: The bottom bars along most of the length of the beam may be subjected to maximum stress. Steel area required to resist the maximum positive moment near midspan of 100 ft-kips (see Table 10-6), $A_s = 1.05$ in.$^2$ Area provided by the three No. 7 bars = 3 (0.60) = 1.80 in.$^2$, so that

$$\frac{A_{s(provided)}}{A_{s(required)}} = \frac{1.80}{1.05} = 1.71 < 2.0$$

Since all of the bottom bars will be spliced near midspan, use a class B splice. $\boxed{12.15.2}$

Required length of splice = 1.3 $l_d \geq 12$ in.

where

$$l_d = \frac{3}{40}\frac{d_b f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c + k_{tr}}{d_b}\right)} \qquad \boxed{12.2.3}$$

where

$\alpha = 1.0$ (reinforcement location factor)
$\beta = 1.0$ (coating factor)
$\gamma = 1.0$ (reinforcement size factor)
$\lambda = 1.0$ (normal weight concrete)

$$c = 1.5 + 0.375 + \frac{0.875}{2} = 2.31 \qquad \text{(governs)}$$

(side cover, bottom bars)

or

$$c = \frac{1}{2}\left[\frac{20 - 2(1.5 + 0.375) - 0.875}{2}\right] = 3.84 \text{ in.}$$

(half the center to center spacing of bars)

$$k_{tr} = \frac{A_{tr} f_{yt}}{1500sn}$$

where

$A_{tr}$ = total area of hoops within the spacing s and which crosses the potential plane of splitting through the reinforcement being developed (ie. for 3#3 bars)

$f_{yt}$ = specified yield strength of hoops = 60,000 psi

s = maximum spacing of hoops = 4 in.

n = number of bars being developed along the plane of splitting = 3

$$k_{tr} = \frac{(3 \times 0.11)60{,}000}{1500 \times 4.0 \times 3} = 1.1$$

$$\frac{c + k_{tr}}{d_b} = \left(\frac{2.31 + 1.1}{0.875}\right) = 3.90 > 2.5 \text{, use } 2.5$$

$$\therefore l_d = \frac{3}{40} \frac{0.875 \times 60{,}000}{\sqrt{4000}} \frac{1}{2.5} = 24.9 \text{ in.}$$

Required length of class B splice = 1.3 × 24.9 = <u>32.0 in.</u>

(2) Top bars, No. 8: Since the mid-span portion of the beam is always subject to a positive bending moment (see Table 10-6), splices in the top bars should be located at or near midspan. Required length of class A splice = 1.0 $l_d$.

For No. 8 bars,

$$l_d = \frac{3}{40} \frac{d_b f_y}{\sqrt{f'_c}} \frac{\alpha\beta\gamma\lambda}{\left(\frac{c + k_{tr}}{d_b}\right)}$$

where $\alpha$ = 1.3 (top bars), $\beta$ = 1.0, $\gamma$ = 1.0, and $\lambda$ = 1.0

$$c = 1.5 + 0.375 + \frac{1.0}{2} = 2.375 \text{ in. (governs)}$$

$$c = \frac{1}{2}\left[\frac{20 - 2(1.5 + 0.375) - 1.0}{2}\right] = 3.81 \text{ in.}$$

$k_{tr}$= 1.1

$$\frac{c + k_{tr}}{d_b} = \frac{2.375 + 1.1}{1.0} = 3.5 > 2.5 \text{ use } 2.5$$

$$\therefore l_d = \frac{3}{40} \frac{1.0x60000}{\sqrt{4000}} \frac{1.3}{2.5} = 37.0 \text{ in.}$$

Required length of splice = 1.0 $l_d$ = <u>37.0 in.</u>

(g) Detail of beam. See Figure 10-55.

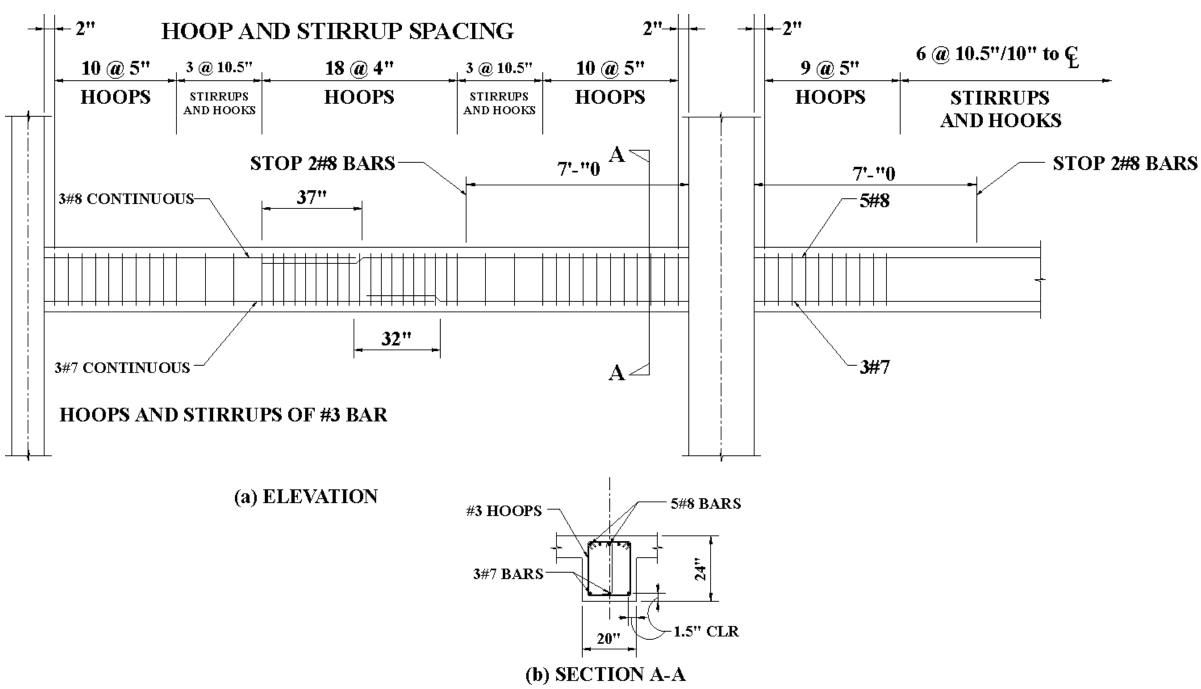



*Figure 10-55.* Detail of reinforcement for beam AB.

3. *Design of frame column A.* The aim here is to design the transverse reinforcement for the exterior tied column on the second floor of a typical transverse interior frame, that is, one of the frames in frame T-1 of Figure 10-48. The column dimension has been established as 22 in. square and, on the basis of the different combinations of axial load and bending moment corresponding to the three loading conditions listed in Table 10-7, *eight No. 9 bars arranged in a symmetrical pattern* have been found adequate.[10-80,10-81] Assume the same beam section framing into the column as considered in the preceding section. $f_c' = 4000 lb/in.^2$ *and* $f_y = 60{,}000 lb/in.^2$

From Table 10-7, $P_u$(max) = 1076 kips:

$$P_u(\max) = 1076 kips > \frac{A_g f_c'}{10} = \frac{(22)^2(4)}{10} = 194 kips$$

Thus, ACI Chapter 21 provisions governing members subjected to bending and axial load apply. $\boxed{21.4.1}$

(a) Check satisfaction of vertical reinforcement limitations and moment capacity requirements:

(1) Reinforcement ratio:

$0.01 \le \rho \le 0.06$

$$\rho = \frac{A_{st}}{A_g} = \frac{8(1.0)}{(22)(22)} = 0.0165 \qquad \text{O.K.}$$

$\boxed{21.4.3.1}$

(2) Moment strength of columns relative to that of framing beam in transverse direction (see Figure 10-56)

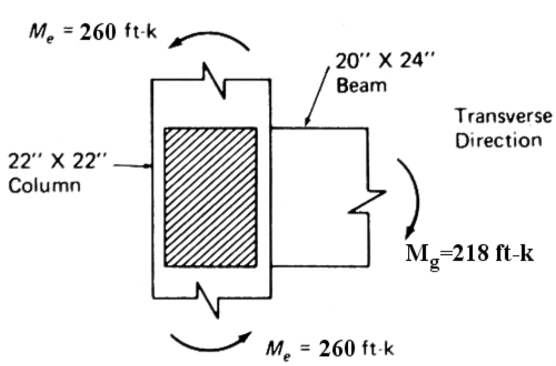



*Figure 10-56.* Relative flexural strength of beam and columns at exterior joint— transverse direction.

$$M_e(columns) \ge \frac{6}{5} M_g(beams) \qquad \boxed{21.4.2.2}$$

From Section 10.5.2, item 2, $\phi M_n^-$ of the beam at *A* is 218 ft-kips, which may be mobilized during a sidesway to the left of the frame. From Table 10-7, the maximum axial load on column *A* at the second floor level for sidesway to the left is $P_u$ = 1070 kips. Using the *P-M* interaction charts given in ACI SP-17A,[10-81] the moment capacity of the column section corresponding to $P_u = \phi P_n$ = 1070kips, $f_c'$ = 4 ksi, $f_y$ = 60 ksi, $\gamma$ = 0.75 ($\gamma$ = ratio of distance between centroids of outer rows of bars to dimension of cross-section in the direction of bending, and $\rho$ = 0.0165 is obtained as $\phi M_n = M_e$ = 260 ft-kips). With the same size column above and below the beam, total moment capacity of columns = 2(260) = 520 ft-kips. Thus,

$$\sum M_e = 520 > \frac{6}{5} M_g = \frac{(6)(218)}{5}$$

= 262 ft-kips O.K.

(3) Moment strength of columns relative to that of framing beams in longitudinal direction (see Figure 10-57): Since the columns considered here are located in the center portion of the exterior longitudinal frames, the axial forces due to seismic loads in the longitudinal direction are negligible. (Analysis of the longitudinal frames under seismic loads indicated practically zero axial forces in the exterior columns of the four transverse frames represented by frame on line 1 in Figure 10-48) Under an axial load of 1.2 D + 1.6 L + 0.5 $L_r$ = 1076 kips, the moment capacity of the column section with eight No. 9 bars is obtained as $\phi M_n = M_e$ = 258 ft-kips. If we assume a ratio for the negative moment reinforcement of about 0.0075 in the beams of the exterior longitudinal frames ($b_w$ = 20 in., $d$ = 21.5 in.), then

$$A_s = \rho b_w d \approx (0.0075)(20)(21.5)$$
$$= 3.23 \text{ in.}^2$$

Assume four No. 8 bars, $A_s$ = 3.16 in. Negative moment capacity of beam:

$$a = \frac{A_s f_y}{0.85 f_c^{'} b_w} = \frac{(3.16)(60)}{(0.85)(4)(20)} = 2.79\,in$$

$$\phi M_n^- = M_g^- = \phi A_s f_y \left( d - \frac{a}{2} \right)$$

$$= (0.90)(3.16)(60)(21.5 - 1.39)/12$$

= 286 ft-kips

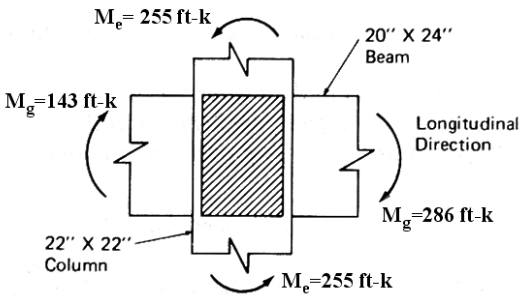



*Figure 10-57.* Relative flexural strength of beam and columns at exterior joint— longitudinal direction.

Assume a positive moment capacity of the beam on the opposite side of the column equal to one-half the negative moment capacity calculated above, or 143 ft-kips. Total moment capacity of beams framing into joint in longitudinal direction, for sidesway in either direction:

$$\sum M_g = 286 + 143 = 429\,ft - kips$$

$$\sum M_e = 2(258) = 516\,ft - kips$$

$$> \frac{6}{5} \sum M_g = \frac{6}{5}(429) = 515\,ft - kips$$

O.K. $\boxed{21.4.2.2}$

(b) Orthogonal effects: According to IBC-2000, the design seismic forces are permitted to be applied separately in each of the two orthogonal directions and the orthogonal effects can be neglected.

(c) Determine transverse reinforcement requirements:

(1) Confinement reinforcement (see Figure 10-38). Transverse reinforcement for confinement is required over a distance $l_0$ from column ends, where

$$l_0 \geq \begin{cases} depth\,of\,member = 22\,in.\ (governs) \\ \frac{1}{6}(clear\,height) = \frac{10 \times 12}{6} = 20\,in. \\ 18\,in. \end{cases} \qquad \boxed{21.4.4.4}$$

Maximum allowable spacing of rectangular hoops:

$$s_{\max} = \begin{cases} \frac{1}{4}(\text{smallest dimension of column}) \\ \quad = \frac{22}{4} = 5.5\,in. \\ 4\,in.\,(governs) \end{cases}$$

$\boxed{21.4.4.2}$

Required cross-sectional area of confinement reinforcement in the form of hoops:

$$A_{sh} \geq \begin{cases} 0.09 s h_c \dfrac{f_c^{'}}{f_{yh}} \\ 0.3 s h_c \left( \dfrac{A_g}{A_{ch}} - 1 \right) \dfrac{f_c^{'}}{f_{yh}} \end{cases}$$

$\boxed{21.4.4.1}$

where the terms are as defined for Equation 10-6 and 10-7. For a hoop spacing of 4 in., $f_{yh}$ = 60,000 lb/in.$^2$, and tentatively assuming No. 4 bar hoops (for the purpose of estimating $h_c$ and $A_{ch}$)' the required cross-sectional area is

$$A_{sh} \geq \begin{cases} \dfrac{(0.09)(4)(18.5)(4000)}{60{,}000} \\ = 0.44\ in^2 \\ (0.3)(4)(18.5)\left( \dfrac{484}{361} - 1 \right) \dfrac{4000}{60{,}000} \\ = 0.50\,in^2\ \text{(governs)} \end{cases} \qquad \boxed{21.4.4.3}$$

No. 4 hoops with one crosstie, as shown in Figure 10-58, provide $A_{sh}$ = 3(0.20) = 0.60 in.$^2$

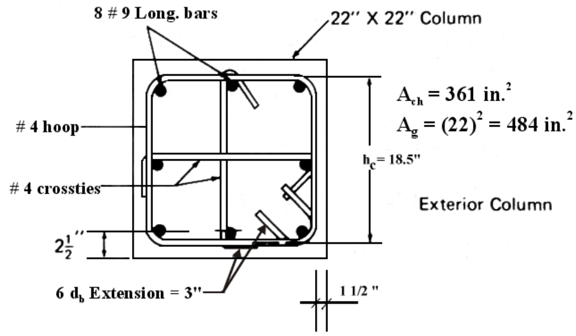



*Figure 10-58.* Detail of column transverse reinforcement.

(2) Transverse reinforcement for shear: As in the design of shear reinforcement for beams, the design shear in columns is based not on the factored shear forces obtained from a lateral-load analysis, but rather on the maximum probable flexural strength, $M_{pr}$ (with $\phi$ = 1.0 and $f_s$ = 1.25 $f_y$), of the member associated with the range of factored axial loads on the member. However, the member shears need not exceed those associated with the probable moment strengths of the beams framing into the column.

If we assume that an axial force close to P = 740 kips ($\phi$ = 1.0 and tensile reinforcement stress of 1.25 $f_y$, corresponding to the "balanced point' on the P-M interaction diagram for the column section considered – which would yield close to if not the largest moment strength), then the corresponding $M_b$ = 601 ft-kips. By comparison, the moment induced in the column by the beam framing into it in the transverse direction, with $M_{pr}$ = 299 ft-kips, is 299/2 = 150 ft-kips. In the longitudinal direction, with beams framing on opposite sides of the column, we have (using the same steel areas assumed earlier),
$M_{pr}$ (beams) = $M^-_{pr}$ (beam on one side) + $M^+_{pr}$ (beam on the other side) = 390 + 195 = 585 ft-kips, with the moment induced at each end of the column = 585/2 =293 ft-kips. This is less than $M_b$ = 601 ft-kips and will be used to determine the design shear force on the column. Thus (see Figure 10-42),

$V_u$ = 2 $M_u/l$ = 2(293)/10 = 59 kips

using, for convenience,

$$V_c = 2\sqrt{f_c'}\,bd$$

$$= \frac{2\sqrt{4000}(22)(19.5)}{1000} = 54\ kips$$

Required spacing of No. 4 hoops with $A_v$ = 2(0.20) = 0.40 in.$^2$ (neglecting crossties) and $V_s = (V_u - \phi V_c)/\phi = 14.8\,kips$:

$$s = \frac{A_v f_y d}{V_s} = \frac{(2)(2.0)(60)(19.5)}{14.8} = 31.6\,in.$$

11.5.6.2

Thus, the transverse reinforcement spacing over the distance $l_0$ = 22 in. near the column ends is governed by the requirement for confinement rather than shear.

Maximum allowable spacing of shear reinforcement: *d/2* = 9.7 in. 11.5.4.1

*Use No. 4 hoops and crossties spaced at 4 in. within a distance of 24 in. from the columns ends and No. 4 hoops spaced at 6 in. or less over the remainder of the column.*

(d) Minimum length of lap splices for column vertical bars:

ACI Chapter 21 limits the location of lap splices in column bars within the middle portion of the member length, the splices to be designed as tension splices. 21.4.3.2

As in flexural members, transverse reinforcement in the form of hoops spaced at 4 in. (<d/4 =19.5/4 = 4.9 in.) is to be provided over the full length of the splice. 21.3.2.3

Since generally all of the column bars will be spliced at the same location, a Class *B* splice will be required. 12.15.2

The required length of splice is 1.3$l_d$ where

$$l_d = \frac{3}{40}\frac{d_b f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c+k_{tr}}{d_b}\right)}$$

where $\alpha = 1.0$, $\beta = 1.0$, $\gamma = 1.0$, and $\lambda = 1.0$

$$c = 1.5 + 0.5 + \frac{1.128}{2} = 2.6 \text{ in. (governs)}$$

or $c = \dfrac{1}{2}\left[\dfrac{22 - 2(1.5+0.5) - 1.128}{2}\right] = 4.2$ in.

$$k_{tr} = \frac{A_{tr} f_{yt}}{1500sn} = \frac{(3\times 0.2)\times 60{,}000}{1500\times 4\times 3} = 2.0$$

$$\frac{c+k_{tr}}{d_b} = \frac{2.6+2.0}{1.128} = 4.1 > 2.5 \text{ use } 2.5$$

$$\therefore l_d = \frac{3}{40}\frac{1.128x60{,}000}{\sqrt{4000}}\frac{1.0}{2.5} = 32.1 \text{ in.}$$

Thus, required splice length = 1.3(32.1) = 42 in. *Use 44-in, lap splices.*

(e) Detail of column. See Figure 10-59.

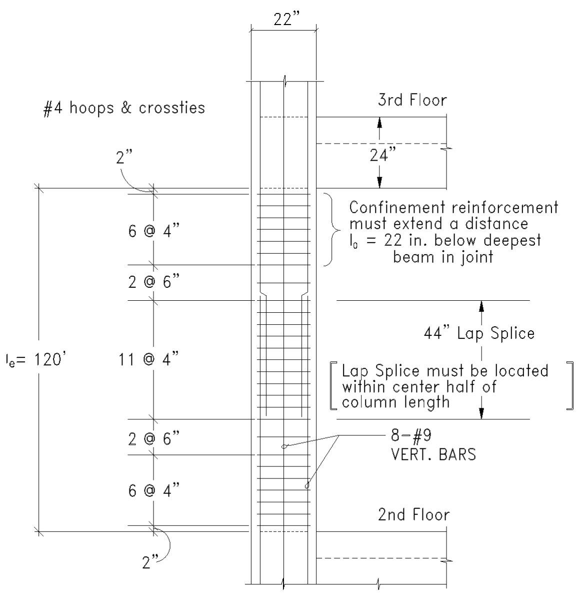



*Figure 10-59.* Column reinforcement details.

4. *Design of exterior beam—column connection.* The aim is to determine the transverse confinement and shear-reinforcement requirements for the exterior beam-column connection between the beam considered in item 2 above and the column in item 3. Assume the joint to be located at the sixth floor level.

(a) Transverse reinforcement for confinement: ACI Chapter 21 requires the same amount of confinement reinforcement within the joint as for the length $l_0$ at column ends, unless the joint is confined by beams framing into all vertical faces of the column. In the latter case, only one-half the transverse reinforcement required for unconfined joints need be provided. In addition, the maximum spacing of transverse reinforcement is (minimum dimension of column)/4 or 6 in. (instead of 4 in.).

21.5.2.1

21.5.2.2

In the case of the beam-column joint considered here, beams frame into only three sides of the column, so that the joint is considered unconfined.

In item 4 above, confinement requirements at column ends were satisfied by No. 4 hoops with crossties, spaced at 4 in.

(b) Check shear strength of joint: The shear across section x-x (see Figure 10-60) of the joint is obtained as the difference between the tensile force at the top flexural reinforcement of the framing beam (stressed to $1.25f_y$) and the horizontal shear from the column above. The tensile force from the beam (three No. 8 bars, $A_s = 2.37$ in.$^2$) is

(2.37)(1.25)(60) = 178 kips

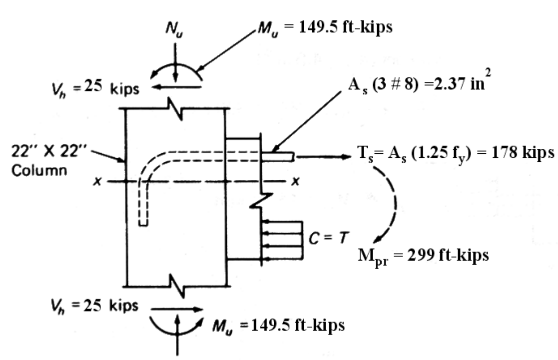



*Figure 10-60.* Horizontal shear in exterior beam-column joint.

An estimate of the horizontal shear from the column, $V_h$ can be obtained by assuming that the beams in the adjoining floors are also deformed so that plastic hinges form at their junctions with the column, with $M_p$(beam) = 299 ft-kips (see Table 10-9, for sidesway to left). By further assuming that the plastic moments in the beams are resisted equally by the columns above and below the joint, one obtains for the horizontal shear at the column ends

$$V_h = \frac{M_p(beam)}{story\,height} = \frac{299}{12} = 25\,kips$$

Thus, the net shear at section x-x of joint is 178 -25 = 153 kips. ACI Chapter 21 gives the nominal shear strength of a joint as a function only of the gross area of the joint cross-section, $A_j$, and the degree of confinement provided by framing beams. For the joint considered here (with beams framing on three sides),

$$\phi V_c = \phi 15\sqrt{f_c'}A_j$$
$$= \frac{(0.85)(15)(\sqrt{4000})(22)^2}{1000}$$
$$= 390\,kips > V_u = 153\,kips \quad \text{O.K.}$$

21.5.3.1

9.3.4.1

Note that if the shear strength of the concrete in the joint as calculated above were inadequate, any adjustment would have to take the form (since transverse reinforcement above the minimum required for confinement is considered not to have a significant effect on shear strength) of either an increase in the column cross-section (and hence $A_j$) or an increase in the beam depth (to reduce the amount of flexural reinforcement required and hence the tensile force *T*).

(c) Detail of joint. See Figure 10-61. (The design should be checked for adequacy in the longitudinal direction.)

*Note:* The use of crossties within the joint may cause some placement difficulties. To relieve the congestion, No. 6 hoops spaced at 4 in. but without crossties may be considered as an alternative. Although the cross-sectional area of confinement reinforcement provided by No. 6 hoops at 4 in. ($A_{sh}$ = 0.88 in.$^2$) exceeds the required amount (0.59 in.$^2$), the requirement of section 21.4.4.3 of ACI Chapter 21 relating to a maximum spacing of 14 in. between crossties or legs of overlapping hoops (see Figure 10-41) will not be satisfied. However, it is believed that this will not be a serious shortcoming in this case, since the joint is restrained by beams on three sides.

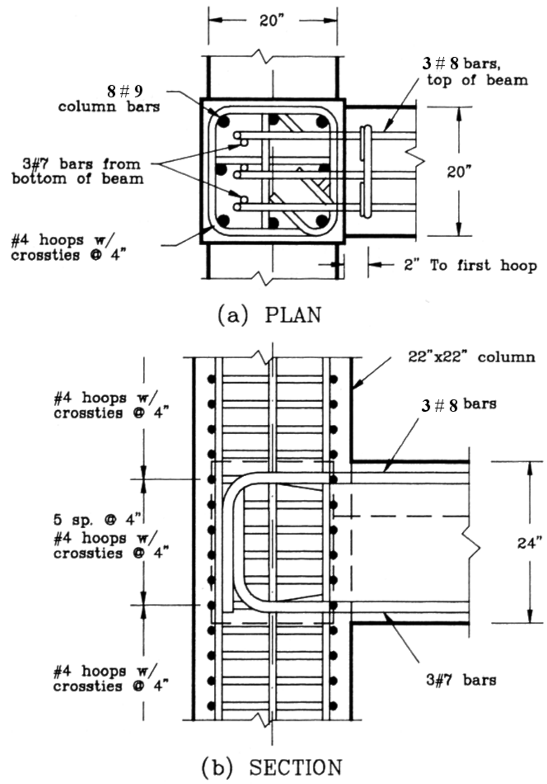



*Figure 10-61.* Detail of exterior beam-column connection.

5. D*esign of interior beam-column connection.* The objective is to determine the transverse confinement and shear reinforcement requirements for the interior beam-column connection at the sixth floor of the interior transverse frame considered in previous examples. The column is 26 in. square and is reinforced with eight No. 11 bars.

The beams have dimensions $b$ = 20 in. and $d$ = 21.5 in. and are reinforced as noted in Section item 2 above (see Figure 10-55).

(a) Transverse reinforcement requirements (for confinement): Maximum allowable spacing of rectangular hoops,

$$s_{\max} = \begin{cases} \frac{1}{4}(\text{smallest dimension of column}) \\ = 26/4 = 6.5\,in. \\ 6\ in.(governs) \end{cases}$$

$\boxed{21.5.2.2}$
$\boxed{21.4.4.2}$

For the column cross-section considered and assuming No. 4 hoops, $h_c$ = 22.5 in., $A_{ch} = (23)^2$ = 529 in.$^2$, and $A_g = (26)^2$ = 676 in.$^2$. With a hoop spacing of 6 in., the required cross-sectional area of confinement reinforcement in the form of hoops is

$$A_{sh} \geq \begin{cases} 0.09 s h_c \dfrac{f_c'}{f_{yh}} = \dfrac{(0.09)(6)(22.5)(4000)}{60{,}000} \\ = 0.81 in^2 \qquad (governs) \\ 0.3 s h_c \left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f_c'}{f_{yh}} \\ = (0.3)(6)(22.5)\left(\dfrac{676}{529} - 1\right)\dfrac{4000}{60{,}000} \\ = 0.75 in^2 \end{cases}$$

$\boxed{21.4.4.1}$

Since the joint is framed by beams (having widths of 20 in., which is greater than $\frac{3}{4}$ of the width of the column, 19.5 in.) on all four sides, it is considered confined, and a 50% reduction in the amount of confinement reinforcement indicated above is allowed. Thus, $A_{sh}$(required) ≥ 0.41 in.$^2$.
*No. 4 hoops with crossties spaced at 6 in. o.c. provide $A_{sh}$ = 0.60* in.$^2$. (See Note at end of item 4.)

(b) Check shear strength of joint: Following the same procedure used in item 4, the forces affecting the horizontal shear across a section near mid-depth of the joint shown in Figure 10-62 are obtained:

(Net shear across section x-x) = $T_1 + C_2 - V_h$

$$= 296 + 135 - 59$$
$$= 372 \text{ kips} = V_u$$

Shear strength of joint, noting that joint is confined:

$$\phi V_c = \phi 20 \sqrt{f_c'} A_j$$
$$= \frac{(0.85)(20)\sqrt{4000}(26)^2}{1000} \qquad \boxed{21.5.3.1}$$
$$= 726 \text{ kips}$$
$$> V_u = 372 \text{ kips} \quad \text{O.K.}$$

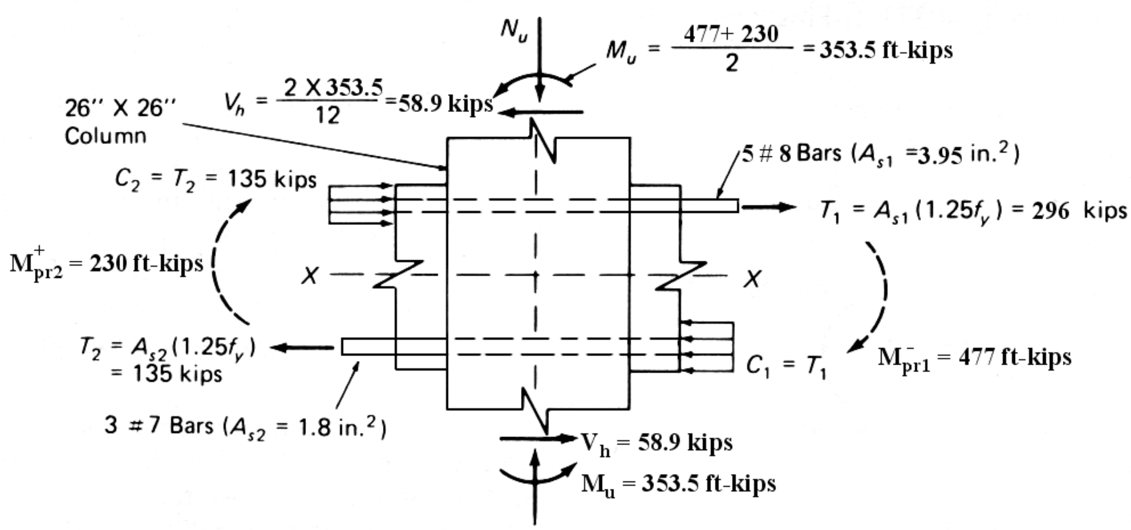



*Figure 10-62.* Forces acting on interior beam-column joint.

6.*Design of structural wall (shear wall).* The aim is to design the structural wall section at the first floor of one of the identical frame-shear wall systems. The preliminary design, as shown in Figure 10-48, is based on a 14-in.-thick wall with 26-in. -square vertical boundary elements, each of the latter being reinforced with eight No. 11 bars.

Preliminary calculations indicated that the cross-section of the structural wall at the lower floor levels needed to be increased. In the following, a 14-in.-thick wall section with 32 × 50-in. boundary elements reinforced with 24 No. 11 bars is investigated, and other reinforcement requirements determined.

The design forces on the structural wall at the first floor level are listed in Table 10-8. Note that because the axis of the shear wall coincides with the centerline of the transverse frame of which it is a part, lateral loads do not induce any vertical (axial) force on the wall.

The calculation of the maximum axial force on the boundary element corresponding to Equation 10-8b, $1.4\,\mathrm{D} + 0.5\,\mathrm{L} \pm 1.0\,\mathrm{Q_E}$, $P_u = 3963$ kips, shown in Table 10-8, involved the following steps: At base of the wall:

Moment due to seismic load (from lateral load analysis for the transverse frames), $M_b$ = 32,860 ft-kips.

Referring to Figure 10-45, and noting the load factors used in Equation 10-8a of Table 10.8,

W = 1.2 D + 1.6 L + 0.5 $L_r$
= 5767 kips
Ha = 30,469 ft-kips

$$C_v = \frac{W}{2} + \frac{Ha}{d}$$
$$= \frac{5157}{2} + \frac{30{,}469}{22} = 3963\,kips$$

(a) Check whether boundary elements are required: ACI Chapter 21 (Section 21.6.2.3) requires boundary elements to be provided if the maximum compressive extreme-fiber stress under factored forces exceeds $0.2 f_c^{'}$, unless the entire wall is reinforced to satisfy Sections 21.4.4.1 through 21.4.4.3 (relating to confinement reinforcement).

It will be assumed that the wall will not be provided with confinement reinforcement over its entire height. For a homogeneous rectangular wall 26.17 ft long (horizontally) and 14 in. (1.17 ft) thick,

$$I_{n.a.} = \frac{(1.17)(26.17)^3}{12} = 1747\,ft^4$$
$$A_g = (1.17)(26.17) = 30.6\,ft^2$$

Extreme-fiber compressive stress under $M_u$ = 30,469 ft-kips and $P_u$ = 5157 kips (see Table 10-8):

$$f_c = \frac{P_u}{A_g} + \frac{M_u h_w/2}{I_{n.a.}} = \frac{5157}{30.6} + \frac{(30{,}469)(26{,}17)/2}{1747}$$

= 397 ksf = 2.76 ksi > 0.2 $f_c^{'}$ = (0.2)(4)
= 0.8 ksi.

Therefore, *boundary elements are required,* subject to the confinement and special loading requirements specified in ACI Chapter 21.

(b) Determine minimum longitudinal and transverse reinforcement requirements for wall:

(1) Check whether two curtains of reinforcement are required: ACI Chapter 21 requires that two curtains of reinforcement be provided in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f_c^{'}}$, where $A_{cv}$ is the cross-sectional area bounded by the web thickness and the length of section in the direction of the shear force considered. $\boxed{21.6.2.2}$

From Table 10-8, the maximum factored shear force on the wall at the first floor level is $V_u$ = 651 kips:

$$2A_{cv}\sqrt{f_c^{'}} = \frac{(2)(14)(26.17\times 12)\sqrt{4000}}{1000}$$

= 556 kips
< $V_u$ = 651 kips

Therefore, *two curtains of reinforcement are required.*

(2) Required longitudinal and transverse reinforcement in wall:

Minimum required reinforcement ratio,

$$\rho_v = \frac{A_{sv}}{A_{cv}} = \rho_n \geq 0.0025 \qquad \text{(max. spacing = 18 in.)}$$ 21.6.2.1

With $A_{cv}$ = (14)(12) = 168 in.$^2$, (per foot of wall) the required area of reinforcement in each direction per foot of wall is (0.0025)(168) = 0.42 in.$^2$/ft. Required spacing of No. 5 bars [in two curtains, $A_s$ = 2(0.31) = 0.62 in.$^2$]:

$$s(required) = \frac{2(0.31)}{0.42}(12) = 17.7\ in. < 18\ in.$$

(c) Determine reinforcement requirements for shear. [Refer to discussion of shear strength design for structural walls in Section 10.4.3, under "Code Provisions to Insure Ductility in Reinforced Concrete Members," item 5, paragraph (b).] Assume two curtains of No. 5 bars spaced at 17 in. o.c. both ways. Shear strength of wall ($h_w/l_w = 148/26.17 = 5.66 > 2$):

$$\phi V_n = \phi A_{cv}\left(2\sqrt{f_c'} + \rho_n f_y\right)$$

where

$\phi$ = 0.60

$A_{cv}$ = (14)(26.17×12) = 4397 in.$^2$

$$\rho_n = \frac{2(0.31)}{(14)(12)} = 0.0037$$

Thus,

$$\phi V_n = \frac{(0.60)(4397)\left[2\sqrt{4000} + (0.0037)(60{,}000)\right]}{1000}$$

$$= \frac{2638.2[126.5 + 222]}{1000} = 919\,kips$$

$$> V_u = 651\,kips \qquad \text{O.K.}$$

Therefore, *use two curtains of No. 5 bars spaced at 17 in o. c. in both horizontal and vertical directions.* 21.7.3.5

(d) Check adequacy of boundary element acting as a short column under factored vertical forces due to gravity and lateral loads (see Figure 10-45): From Table 10-8, the maximum compressive axial load on boundary element is $P_u$ = 3963 kips.

21.5.3.3

With boundary elements having dimensions 32 in.×50 in. and reinforced with 24 No. 11 bars,

$A_g$ = (32)(50) = 1600 in.$^2$

$A_{st}$ = (24)(1.56) = 37.4 in.$^2$

$\rho_{st}$ = 37.4/1600 = 0.0234

$\rho_{min}$ = 0.01 < $\rho_{st}$ < $\rho_{max}$ = 0.06 O.K.

21.4.3.1

Axial load capacity of a short column:

$$\phi P_n(\max) = 0.80\phi\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

= (0.80)(0.70)[(0.85)(4)(1600 - 37.4) +(60)(37.4)]

= (0.56)[5313+ 2244] = 4232 kips > $P_u$ = 3963 kips O.K. 10.3.5.2

(e) Check adequacy of structural wall section at base under combined axial load and bending in the plane of the wall: From Table 10-8, the following combinations of factored axial load and bending moment at the base of the wall are listed, corresponding to Eqs. 10-8a, b and c:

9-8a: $P_u$ = 5767 kips, $M_u$ small

9-8b: $P_u$ = 5157 kips, $M_u$ = 30,469 ft-kips

9-8c: $P_u$ = 2293 kips, $M_u$ = 30,469 ft-kips

Figure 10-63 shows the $\phi P_n$-$\phi M_n$ interaction diagram (obtained using a computer program for generating *P-M* diagrams) for a structural wall section having a 14-in.-thick web reinforced with two curtains of No. 5 bars spaced at 17 in o.c. both ways and 32 in.×50-in. boundary elements reinforced with 24 No. 11 vertical bars, with $f_c'$ = 4000 lb/in.$^2$, and $f_y$ = 60,000 lb/in.$^2$ (see Figure 10-64). The design load combinations listed above are shown plotted in Figure 10-63. The point marked *a* represents the *P-M* combination corresponding to Equation 10-8a, with similar notation used for the other two load combinations.

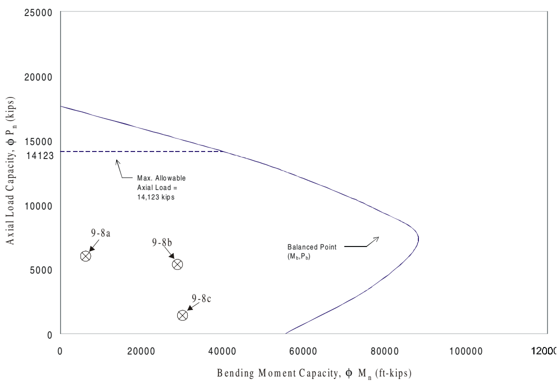



*Figure 10-63.* Axial load-moment interaction diagram for structural wall section.

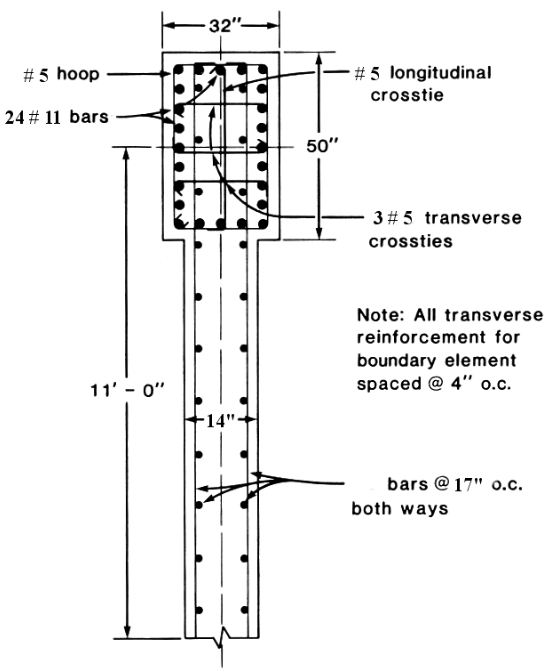



*Figure 10-64.* Half section of structural wall at base.

It is seen in Figure 10-63 that the three design loadings represent points inside the interaction diagram for the structural wall section considered. Therefore, *the section is adequate with respect to combined bending and axial load.*

Incidentally, the "balanced point" in Figure 10-63 corresponds to a condition where the compressive strain in the extreme concrete fiber is equal to $\varepsilon_{cu}$ = 0.003 and the tensile strain in the row of vertical bars in the boundary element farthest from the neutral axis (see Figure 10-64) is equal to the initial yield strain, $\varepsilon_y$ = 0.00207.

(f) Determine lateral (confinement) reinforcement required for boundary elements (see Figure 10-64): The maximum allowable spacing is

$$s_{\max} = \begin{cases} 1/4(\text{smallest dimension} \\ \text{of boundary element}) \\ = 32/4 = 8\,in. \\ 4\,in.\ (\text{governs}) \end{cases}$$

21.6.6.2
21.4.4.2

(1) Required cross-sectional area of confinement reinforcement in short direction:

$$A_{sh} \geq \begin{cases} 0.09 s h_c \dfrac{f_c'}{f_{yh}} \\ 0.3 s h_c \left(\dfrac{A_g}{A_{ch}} - 1\right) \dfrac{f_c'}{f_{yh}} \end{cases}$$

21.4.4.1

Assuming No. 5 hoops and crossties spaced at 4 in. o.c. and a distance of 3 in. from the center line of the No. 11 vertical bars to the face of the column, we have

$h_C$ = 44 + 1.41 + 0.625 = 46.04 in. (for short direction),

$A_{ch}$= (46.04 + 0.625)(26 + 1.41 + 1.25) =1337 in.$^2$

$$A_{sh} > \begin{cases} (0.09)(4)(46.04)(4/60) \\ = 1.10\,in^2\ (governs) \\ (0.3)(4)(46.04)(\dfrac{(32)(50)}{1337} - 1)(\dfrac{4}{60}) \\ = 0.72\ in.^2 \end{cases}$$

(required in short direction).

With three crossties (five legs, including outside hoops),

$A_{sh}$ (provided) = 5(0.31) = 1.55 in.$^2$ O.K.

(2) Required cross-sectional area of confinement reinforcement in long direction:

$h_c$ = 26 + 1.41 + 0.625 = 28.04 in.

(for long direction),
$A_{ch}$ = 1337 in.$^2$

$$A_{sh} \geq \begin{cases} (0.09)(4)\,(28.04)\,(4/60) \\ = 0.67 \text{ in.}^2 \text{ (governs)} \\ (0.3)(4)(28.04)(1.196 - 1)(4/60) \\ = 0.44 \text{ in.}^2 \end{cases}$$

(required in long direction).

With one crosstie (i.e., three legs, including outside hoop),

$A_{sh}$ (provided) = 3(0.31) = 0.93 in.$^2$ O.K.

(g) Determine required development and splice lengths:

ACI Chapter 21 requires that all continuous reinforcement in structural walls be anchored or spliced in accordance with the provisions for reinforcement in tension. 21.6.2.4

(1) Lap splice for No. 11 vertical bars in boundary elements (the use of mechanical connectors may be considered as an alternative to lap splices for these large bars): It may be reasonable to assume that 50% or less of the vertical bars are spliced at any one location. However, an examination of Figure 10-63 suggests that the amount of flexural reinforcement provided–mainly by the vertical bars in the boundary elements–does not represent twice that required by analysis, so that a class B splice will be required. 12.15.2

Required length of splice = 1.3 $l_d$ where $l_d$ = 2.5 $l_{dh}$ 12.15.1

and

$$l_{dh} \geq \begin{cases} f_y d_b / 65\sqrt{f_c'} \\ = \dfrac{(60{,}000)(1.41)}{65\sqrt{4000}} = 21\,in.\,(governs) \\ 8\,d_b = (8)(1.41) = 12\,in. \\ 6\ in. \end{cases}$$

21.5.4.2

Thus the required splice length is (1.3)(2.5)(21) = 68 in.

(2) Lap splice for No. 5 vertical bars in wall "web": Here again a class B splice will be required. Required length of splice = 1.3 $l_d$ , whre $l_d$ = 2.5 $l_{dh}$, and

$$l_{dh} \geq \begin{cases} f_y d_b / 65\sqrt{f_c'} \\ = \dfrac{(60{,}000)(0.625)}{65\sqrt{4000}} = 9\,in.\,(governs) \\ 8\,d_b = (8)(0.625) = 5.0\,in. \\ 6\ in. \end{cases}$$

Hence, the required length of splice is (1.3)(2.5)(9) = <u>30 in</u>.

Development length for No. 5 horizontal bars in wall, assuming no hooks are used within the boundary element: Since it is reasonable to assume that the depth of concrete cast in one lift beneath a horizontal bar will be greater than 12 in., the required factor of 3.5 to be applied to the development length, $l_{dh}$, required for a 90° hooked bar will be used [Section 10.4.3, under "Code Provisions Designed to Insure Ductility in Reinforced-Concrete Members", item 2, paragraph (f)]:

21.5.4.2

$l_d$ = 3.5 $l_{dh}$ , where as indicated above, $l_{dh}$ = 9.0 in. so that the required development length $l_d$ = 3.5(9) = <u>32 in.</u>

This length can be accommodated within the confined core of the boundary element, so that no hooks are needed, as assumed. However, because of the likelihood of large horizontal cracks developing in the boundary elements, particularly in the potential hinging region near the base of the

wall, the horizontal bars will be provided with 90° hooks engaging a vertical bar, as recommended in the Commentary to ACI Chapter 21 and as shown in Figure 10-64.
Required lap splice length for No. 5 horizontal bars, assuming (where necessary) 1.3 $l_d$ = (1.3)(32) = <u>42 in</u>.

(h) Detail of structural wall: See Figure 10-64. It will be noted that the No. 5 vertical-wall "web" reinforcement, required for shear resistance, has been carried into the boundary element. The Commentary to ACI Section 21.6.5 specifically states that the concentrated reinforcement provided at wall edges (i.e. the boundary elements) for bending is not be included in determining shear-reinforcement requirements. The area of vertical shear reinforcement located within the boundary element could, if desired, be considered as contributing to the axial load and bending capacity.

(i) Design of boundary zone using UBC-97 and IBC-2000 Provisions:

Using the procedure discussed in Section 10.4.3 item 5 (f), the boundary zone design and detailing requirements using these provisions will be determined.

(1) Determine if boundary zone details are required:

Shear wall boundary zone detail requirements to be provided unless $P_u \le 0.1 A_g f'_c$ and either $M_u/V_u l_u \le 1.0$ or $V_u \le 3\, A_{cv}\sqrt{f'_c}$. Also, shear walls with $P_u > 0.35\ P_0$ (where $P_0$ is the nominal axial load capacity of the wall at zero eccentricity) are not allowed to resist seismic forces.

Using 26 inch square columns; $0.1 A_g f'_c = 0.1 \times (14 \times 19.83 \times 12 + 2 \times 26^2) \times 4 = 1873$ kips $< P_u = 3963$ kips. Using 32 × 50 columns also results in the value of $0.1 A_g\ f'_c$ to be less than $P_u$. Therefore, boundary zone details are required.

Assume a 14 in. thick wall section with 32 × 50 in. boundary elements reinforced with 24 No. 11 bars as used previously. Also, it was determined that 2#5 bars at 17 in. spacing is needed as vertical reinforcement in the web. On this basis, the nominal axial load capacity of the wall ($P_0$) at zero eccentricity is:

$$P_0 = 0.85\ f'_c\,(A_g - A_{st}) + f_y\,A_{st}$$
$$= 0.85 \times 4 \times (6195\text{-}82.68) + (60 \times 82.68) = 25{,}743 \text{ kips}$$

Since $P_u = 3963$ kips $= 0.15\ P_0 < 0.35\ P_0 = 9010$ kips, the wall can be considered to contribute to the calculated strength of the structure for resisting seismic forces.

Therefore, provide boundary zone at each end having a distance of $0.15\ l_w = 0.15 \times 26.17 \times 12 = 47.1$ in. On this basis, a 32×50 boundary zone as assumed is adequate.

Alternatively, the requirements for boundary zone can be determined using the displacement based procedure. As such, boundary zone details are to be provided over the portion of the wall where compressive strains exceed 0.003. The procedure is as follows:

Determine the location of the neutral axis depth, $c'_u$.

From Table 10-8, $P'_u$ = 5767 kips; the nominal moment strength, $M'_n$ , corresponding to $P'_u$ is 89,360 k-ft (see Figure 10-63). For 32 × 50 in. boundary elements reinforced with 24 #11 bars, $c'_u$ is equal to 97.7 in. This value can be determined using the strain compatibility approach.

From the results of analysis, the elastic displacement at the top of the wall, $\Delta_E$ is equal to 1.55 in. using gross section properties and the corresponding moment, $M'_n$, at the base of the wall is 30,469 k-ft (see Table 10-8). From the analysis using the cracked section properties, the total deflection, $\Delta_t$, at top of the wall is 15.8 in. (see Table 10-3, $\Delta_t = 2.43 \times C_d = 2.43 \times 6.5 = 15.8$in.), also $\Delta_y = \Delta_E M'_n/M'_E = 1.55 \times 89{,}360/30{,}469 = 4.55$ in.

The inelastic deflection at the top of the wall is:

$\Delta_i = \Delta_t - \Delta_y = 15.8 - 4.55 = 11.25$in.

Assume $l_p = 0.5\, l_w = 0.5 \times 26.17 \times 12 = 157$ in., the total curvature demand is:

$$\phi_t = \frac{11.25}{(148\times12-157/2)\times157}+\frac{0.003}{26.17\times12}$$
$$= 5.176\times10^{-5}$$

Since $\phi_t$ is greater than $0.003/c'_u = 0.003/97.7 = 3.07\times10^{-5}$ , boundary zone details are required. The maximum compressive strain in the wall is equal to $\phi_t c'_u = 5.176 \times 10^{-5} \times 97.7 = 0.00506$ which is less than the maximum allowable value of 0.015. In this case, boundary zone details are required over the length,

$$\left(97.7-\frac{0.003}{0.00506}\times97.7\right)=39.8 in.$$

This is less than the 50 in. length assumed. Therefore, the entire length of the boundary zone will be detailed for ductility.

(2) Detailing requirements:

Minimum thickness:

$=l_u/16=\dfrac{(16\times12)-24}{16}=10.5\,in. < 32\,in.$ O.K.

Minimum length = 18 in. < 50 in. O.K.

The minimum area of confinement reinforcement is:

$$A_{sh}=\frac{0.09sh_c f'_c}{f_{yh}}$$

Using the maximum allowable spacing of $6d_b$ = 6 x 1.41 = 8.46 in. or 6 in. (governs), and assuming #5 hoops and crossties at a distance of 3 in. from the center line of #11vertical bars to the face of the column, we have

$h_c = 44 + 1.41 + 0.625 = 46.04$

$$A_{sh}=\frac{0.09\times6\times46.04\times4}{60}=1.66\,\text{in.}^2$$

With four crossties (six legs, including outside hoops), $A_{sh}$ provided = 6 (0.31) = 1.86 in.$^2$ O.K.

Also, over the splice length of the vertical bars in the boundary zone, the spacing of hoops and crossties must not exceed 4 in. In addition, the minimum area of vertical bars in the boundary zone is $0.005\times32^2 = 5.12$ in.$^2$ which is much less than the area provided by 24#11 bars. The reinforcement detail in the boundary zone would be very similar to that shown previously in Figure 10-64.

## REFERENCES

The following abbreviations will be used to denote commonly occurring reference sources:

- *Organizations and conferences:*

**EBRI** Earthquake Engineering Research Institute
**WCEE** World Conference on Earthquake Engineering
**ASCE** American Society of Civil Engineers
**ACI** American Concrete Institute
**PCA** Portland Cement Association
**PCI** Prestressed Concrete Institute

- *Publications:*

**JEMD** Journal of Engineering Mechanics Division, ASCE
**JSTR** Journal of the Structural Division, ASCE
**JACI** Journal of the American Concrete Institute

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